Sixth International Conference on Urban Earthquake Engineering
March 3-4, 2009, Tokyo Institute of Technology, Tokyo, Japan

STUDY ON ENERGY-BASED SEISMIC DESIGN METHOD AND THE APPLICATION FOR STEEL BRACED FRAME STRUCTURES

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Abstract: An energy-based seismic design framework for building structures is proposed with comprehensive understanding of the relationship between energy-based and performance-based seismic design methods. Some fundamental aspects are especially concerned, including spectral input energy E_I, cumulated hysteresis energy ratio E_H/E_I of both SDOF and MDOF systems, system capacity design philosophy of controlling the distribution of hysteresis energy E_H in MDOF systems, methods of determining E_H distribution in building structures as well as the limitations of this method. The proposed framework of seismic design is applied to braced-steel frames to show its effectiveness. The scope of future researches for energy-based seismic design is also outlined, including ground motion intensity indices, distribution of hysteresis energy in MDOF systems, energy-based and displacement-based design methods for individual structural components and energy-based seismic design of irregular structures.

1. INTRODUCTION

Since the first proposal by G. Housner (1956), great progress has been made by researchers in the fundamental aspects of the energy-based seismic design method. One of the major differences between the energy-based seismic design method with other methods, such as the conventional force-based methods and the displacement based method recently, is that a preferable energy-dissipating mechanism is required in the design with the hope that the structure subjected to severe earthquakes will follow the presumed energy-dissipating mechanism and the total energy dissipation capacity of the structure must be greater than the input energy of expected earthquake ground motions. In this way, Engineers are required to fully understand the energy-dissipating behavior of the structure and to control the distribution of structural damages and energy dissipating mechanisms.

Energy dissipation patterns in building structures can be either concentrated or distributed. For seismically isolated buildings, the energy dissipation is concentrated at the isolation layer. Besides, the isolated structure behaves more like a SDOF system, which makes the analysis of such systems much easier. Thus, the energy-based seismic design method was firstly applied to isolated structures (AIJ 2001). On the other hand, energy-based seismic design method was not so far well-developed for structures with distributed energy dissipating mechanisms, such as passive controlled structures with continuously distributed dampers. One of the major difficulties is to estimate the energy distribution among the structural system under severe earthquake, in which case some components of the system were already considerably damaged. This can be very difficult if the damage mode of the system is too arbitrary.

In order to overcome this difficulty, "system capacity design methodology" was proposed by the first author (Ye *et al.* 2002). The main idea is to control the structural damage mode by keeping a main sub-system nearly within its elastic region even when the system is subjected to very severe earthquake. Once the structural system damage mode being controlled, the distribution of energy dissipation will be easier to be estimated. Based on this approach, an energy-based seismic design framework is proposed in this paper.

2. FRAMEWORK OF ENERGY-BASED SEISMIC DESIGN

It is not intended to take the place of the traditional force-based or the recently emerging displacement-based design method by the energy-based design method. Rather, the energy-based seismic design is a supplement to those current design methods, which is expected to make the performance-based seismic design more comprehensive. Strength is a fundamental issue to ensure the structure safety under design earthquake as IO in Figure 1 while for severer earthquakes like LS in Figure 1, displacement limitations proved very helpful to reduce casualty and economic losses. Such displacement limitations are usually dependent on the building occupancies, which are not directly related with the structural seismic capacity or dynamic characteristics. The structural seismic behavior is actually a process of the structure to dissipate the input energy of the ground motion. This is the fundamental concept of the energy-based seismic design. To ensure the structural safety under very rare earthquakes, the LS state in Figure 1 should be kept far enough away from the collapse point (CP in Figure 1), although the determination of such collapse point is still very difficult. Disastrous earthquakes in the last two decades such as Northridge earthquake in 1994, Kobe earthquake in 1995, Chichi earthquake in 1999 and Wenchuan earthquake in 2008 caused astonishing casualty and economic losses. It is of essential importance to study the collapse behavior of building structures under very rare mage earthquakes. Although it is already out of the scope of this paper, some researches have shown that well-designed energy-dissipating mechanisms are helpful to reduce the collapse probability of building structures under very rare mage earthquakes. As mentioned before, engineers to conduct energy-based seismic design are required to comprehensively understand the energy dissipating mechanism of the structure subjected to severe earthquakes. As a result, the aim of energy-based seismic design is to improve the structure design against severe earthquake (as around LS in Figure 1) by applying more reasonable energy-dissipating mechanisms. Furthermore, energy-based seismic design is not only suitable for seismic-resistant structures, but also for seismic passive control and seismic isolation structures.

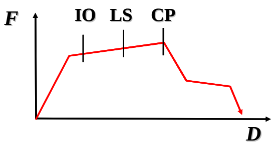


Fig.1 Seismic performance objects of building structures

A design framework as shown in Figure 2 is proposed to achieve the above aim, where the route through ①-③-④-⑤-⑨ is the current displacement-based design and ①-②-⑥-⑦-⑧-⑨ is energy-based design. The energy-based design emphasizes the design of the energy-dissipation mechanism, which is very important for the structure to meet the structural performance objects. The design procedure should be able to ensure that the structure follows the presumed energy-dissipating mechanism under severe earthquakes.

Besides the strength and deformability demands, the following aspects need to be further considered in the energy-based design: (1) Demand of total input energy EI; (2) Hysteresis energy ratio EH/EI (and damping energy ratio ED/EI for structures with viscous dampers); (3) Distribution of hysteresis energy in the structure; (4) Evaluation of component damage.

3. DEMAND OF TOTAL INPUT ENERGY E_I

A lot of researches have been devoted in determining the demand of total input energy E_I. Acceptable design energy spectra (E_I spectra) are already obtained despite of some detailed problems which need further studies. In early researches, E_I spectra were established for elastic SDOF systems. Later, E_I spectra for inelastic SDOF

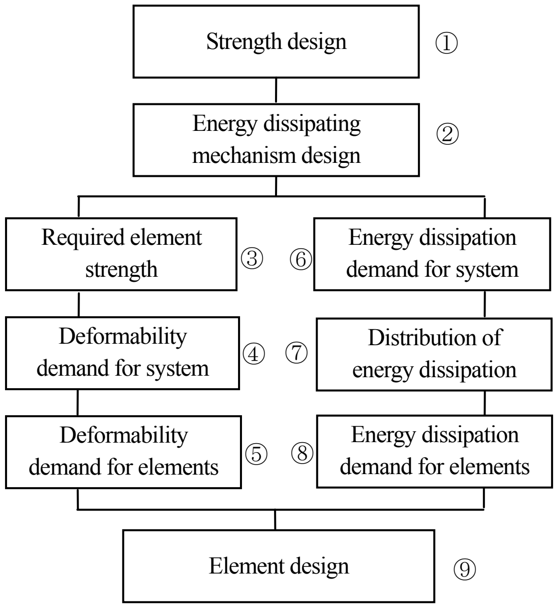


Figure 2 Framework of energy-based seismic design

systems were also established. The E_I spectra used in this study is as below.

3.1 Energy spectra for elastic SDOF systems

The normalized energy spectra for elastic SDOF systems with damping ratio ξ=0.02 are obtained for different site types as shown in Figure 3a, which consists of three branches as below:

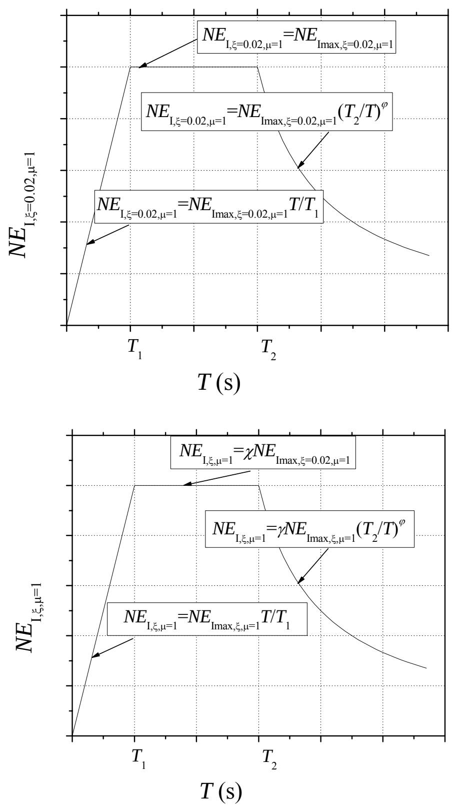


Figure 3 Energy spectra of elastic SDOF systems:
(a) Normalized energy spectrum at ξ=0.02 and

(b) Normalized energy spectra at various damping ratios

I: For short period $T<T_1$, a linearly ascending branch, reaching its peak N_{EImax}, ξ=0.02, μ=1 at $T=T_1$;

II: For moderate period $T_1 \leqslant T<T_2$, a plateau equal to N_{EImax}, ξ=0.02, μ=1;

III: For long period $T \geqslant T_2$, a descending branch with parameters φ given in Table 1.

Table 1 Parameters of elastic energy spectra

Site Type	S1	S2	S3	S4
T_1(s)	0.2	0.3	0.4	0.5
T_2(s)	1	1.8	2	2.5
φ	2.2	2.8	2.8	2.8
$NE_{Imax,\xi=0.02,\mu=1}$	1.0	1.2	1.2	1.3

The characteristic periods T_1 and T_2 are determined following the proposal by Decanini *et al.* (2001) as shown in Table 1. The normalized peak values N_{EImax}, ξ=0.02, μ=1 for different site types are also listed in Table 1, which are obtained as the mean plus one standard deviation of the ground motion spectral energy in the period region of $T_1 \leqslant T<T_2$. Based on the analysis by the authors, the energy spectra peak values E_{Imax}, ξ=0.02, μ=1 was suggested as,

$$E_{Imax,\xi=0.02,\mu=1}=NE_{Imax,\xi=0.02,\mu=1}(9.11I^{1.91}) \quad (1a)$$

$$I = \dot{x}_{gmax} t_D^{0.15} \quad (1b)$$

where I is earthquake intensity index; $\dot{x}_{gmax}$ is peak ground velocity and $t_D^{0.15}$ is strong ground motion duration.

Table 2 Damping modification factor

Damping ratio	0.05	0.1	0.2	0.3
χ (Mean)	0.743	0.569	0.426	0.359
χ (Mean+StdDev)	0.819	0.669	0.526	0.452

Considering the influence of system damping ratios on the elastic energy spectra, a set of two damping modification factors χ and γ are proposed to transform the energy spectra at a standard damping ratio of 0.02 to various damping ratios, as shown in Figure 3b. Factor χ, as given in Table 2, is used to modify the normalized peak value. Factor γ, given by Equation 2, is used to modify the descending branch of the spectrum.

$$\gamma = \frac{(4.5\xi - 0.0663)T + 1}{(4.5\xi - 0.0663)T_2 + 1} \qquad T \geq T_2 \quad (2)$$

The design elastic energy spectrum corresponding to different site types and ground motion intensities can then be obtained by multiplying the normalized energy spectrum by the peak values given by Equation 1, as shown in Figure 4.

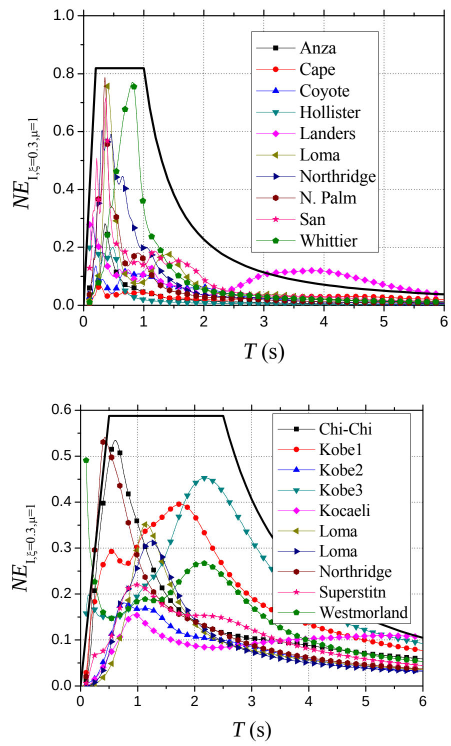


Figure 4 Proposed elastic energy spectra:
(a) S1, ξ=0.1 and (b) S4, ξ=0.3

3.2 Energy spectra for inelastic SDOF systems

Inelastic energy spectra are much more sophisticated than elastic ones. The following factors may have considerable influence on them.

(1) Hysteresis model: It was commonly believed that the influence of hysteresis models on energy spectra is insignificant and that it is conservative to use the bilinear model to generate design spectra. However, the author discovers that such a consensus is only applicable in the moderate and long period region. In short period region, the energy spectra of systems with degrading stiffness are generally larger than those with the ideal bilinear model. This effect should be taken into account if short-period structures are to be studied. In this study, only moderate and long period structures are covered. So the energy spectra yielded with bilinear models can be still regarded as conservative.

(2) Post-yielding stiffness: Most researchers didn't give enough attention to the influence of the post yielding stiffness. It is generally believed that energy spectra with post-yielding stiffness ratio η=0 or 0.05 is conservative. In some cases, however, the input energy for hardening systems can be larger than for perfect plastic-elastic systems. In studying the energy spectra of SDOF systems in this paper, bilinear model with η=0.05 is used herein to simulate the hardening behavior. Hardening systems with η>0.05 will be discussed for MDOF systems.

(3) Damping ratio and ductility factor: Damping and inelastic hysteresis are the two aspects of energy dissipation. They both have significant influence over the energy spectra.

Peak spectral input energy decreases with the increasing of the damping ratio and ductility factor. Based on the analysis by the authors, the average ratio of peak spectral input energy of inelastic systems with various damping ratios $E_{\mathrm{Imax},\xi,\mu}$ to that of elastic systems with ξ=0.02 $E_{\mathrm{Imax},\xi=0.02,\mu=1}$ is suggested as Equation 3.

$$\zeta = \frac{E_{\mathrm{Imax},\xi,\mu}}{E_{\mathrm{Imax},\xi=0.02,\mu=1}} = \left(0.6845 - \frac{0.6393}{\mu} - 0.0882\ln\xi - \frac{0.1517}{\mu}\ln\xi\right)\mu^{-0.57} \tag{3}$$

Considering the standard deviation of ζ is 0.05, the normalized peak spectral input energy for inelastic SDOF system $NE_{\mathrm{Imax},\xi,\mu}$ can be determined by Equation 4.

$$NE_{\mathrm{Imax},\xi,\mu} = (\zeta + 0.05)NE_{\mathrm{Imax},\xi=0.02,\mu=1} \tag{4}$$

The energy spectrum slightly decreases in short period region (T<0.5s) and dramatically increases in moderate and long period regions (T>0.5s) with the increase of the damping ratio when ductility is constant, as shown in Figure 5. On the other hand, the energy spectrum significantly increases in short period region (T<0.5s) and decreases in moderate and long period regions (T>0.5s) with the increase of the ductility factor when the damping ratio is constant, as shown in Figure 6.

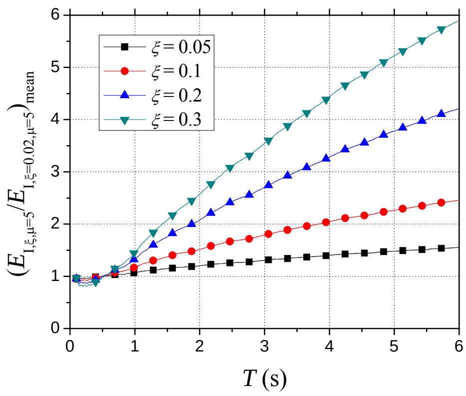


Figure 5 Influence of damping ratio ξ (μ= 5)

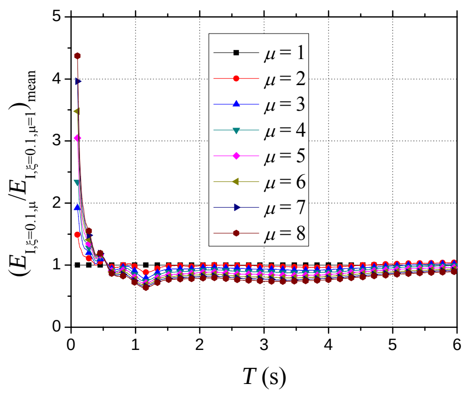


Figure 6 Influence of ductility factor μ (ξ=0.1)

The following modification is applied to the energy spectra in period region T>T_2 according to the analysis results in Figure 5 and 6.

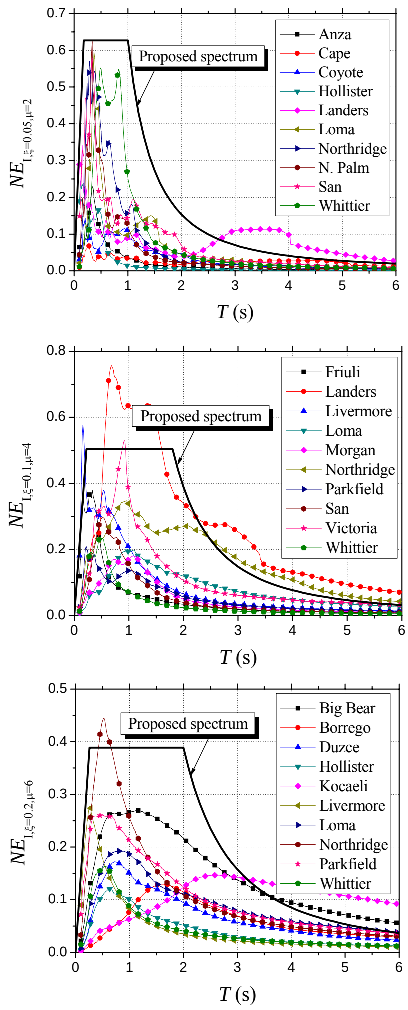
$$\frac{E_{\mathrm{I},\xi,\mu}}{E_{\mathrm{I},\xi=0.02,\mu=1}} = (4.5\xi - 0.0663)(13-\mu)\frac{T}{12} + 1.0 \tag{5}$$

where $T \geq T_2$, $\mu \leq 5$

Energy spectrum significantly increases with the increase of ductility factor in short period region. This is mainly due to the increase of the equivalent period of the system. Equation 6 is recommended to calculate the equivalent period.

$$T_{1,\mu} = \frac{T_1}{1+0.121(\mu-1)^{0.939}} \tag{6}$$

Figure 7 compares the energy spectra given by time history analysis and the proposed equations for various combinations of damping ratios and ductility factors. It is shown that the proposed energy spectra are accurate enough and conservative.



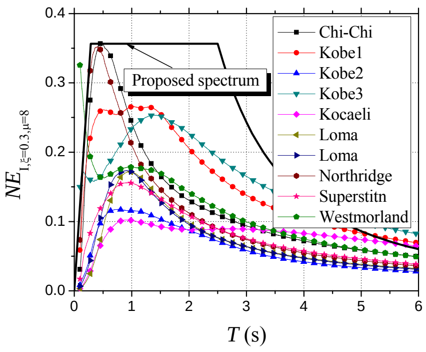


Fig. 7 Proposed inelastic energy spectra: (a)S1 ($\xi = 0.05$, $\mu = 2$), (b) S2 ($\xi = 0.1$, $\mu = 4$), (c) S3 ($\xi = 0.2$, $\mu = 6$) and (d) S4 ($\xi = 0.3$, $\mu = 8$)

3.3 Energy spectra for MDOF systems

Besides the factors influencing the SDOF systems, the earthquake input energy to inelastic MDOF systems are also influenced by the system parameters and ground motion characteristics. It will be extremely complicated to consider all these influencing factors. Bearing in mind that the input energy E_I is a global parameter, the scattering of the influence of different factors on the relationship between the input energy of SDOF and MDOF systems is relatively insignificant if the global system parameters of the SDOF and MDOF are identical. For this reason, the input energy to inelastic SDOF systems $E_{I,SDOF}$ is generally taken as an approximation of that to inelastic MDOF systems $E_{I,MDOF}$. It has been proved accurate enough for engineering practice to use the input energy of SDOF system $E_{I,SDOF}$ as an approximation of that of the MDOF system with the same fundamental period T_0 (Housner *et al.* 1956, Kato *et al.* 1982 and Akiyama *et al.* 1985), i.e.

$$E_{I,MDOF} = E_{I,SDOF}\left(\xi_1, \mu, T_0\right) \tag{7}$$

4. DEMAND FOR HYSTERESIS ENERGY RATIO E_H/E_I

4.1 Demand for hysteresis energy ratio in SDOF systems

Factors influencing the hysteresis energy ratio E_H/E_I of SDOF systems include the damping ratio, the ductility factor and the initial period. For brevity, the influence of these factors on the hysteresis energy ratio E_H/E_I is firstly studied for inelastic SDOF system with bilinear hysteresis and post-yielding stiffness ratio η=0.05. Figure 8 presents some results of E_H/E_I, and following observations are made:

(1) E_H/E_I decreases for larger periods. For given damping ratio and ductility factor, E_H/E_I is approximately linearly descending along the whole range of period.

(2) E_H/E_I decreases for larger damping ratios when constant ductility system is studied.

(3) For given damping ratio, the influence of ductility factors is insignificant in the whole period region when the ductility factor $\mu \geqslant 3$. E_H/E_I increases with the increase of ductility factor. This increase is relatively large in short period region and negligible in long period region.

(4) The standard deviation of E_H/E_I is relatively small. The influences of site soil conditions and ground motion characteristics on E_H/E_I is negligible.

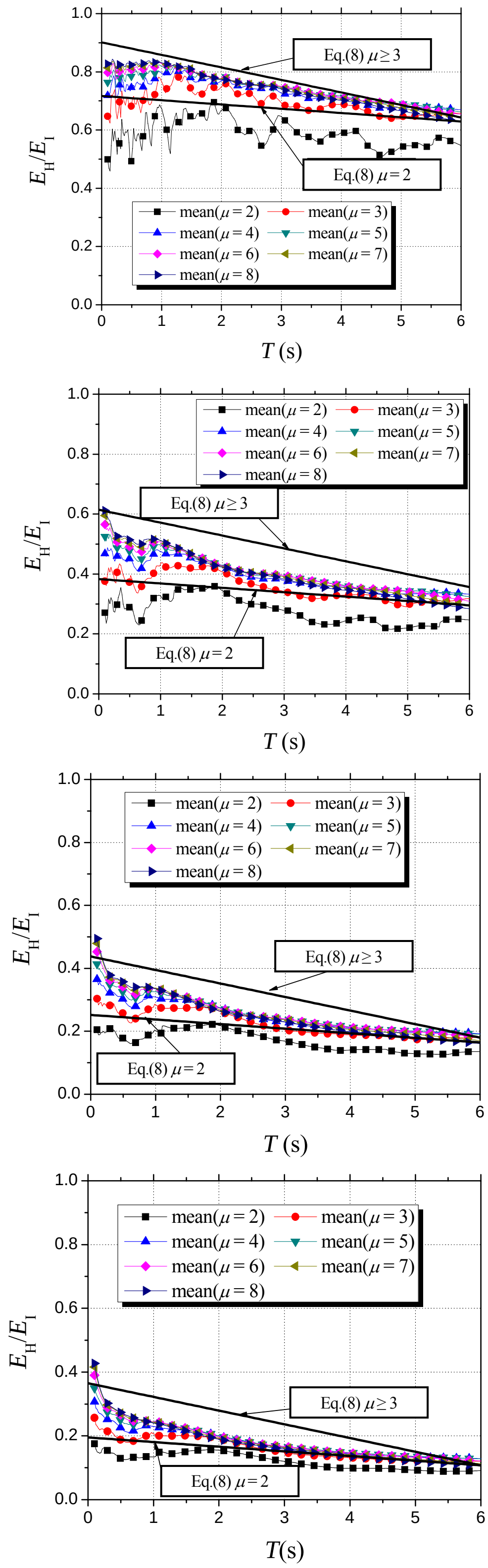


Fig. 8 E_H/E_I for 4 ground motions at various damping ratios: (a) $\xi = 0.02$, (b) $\xi = 0.1$, (c) $\xi = 0.2$ and (d) $\xi = 0.3$

The calculation of E_H/E_I ratio is recommended as Equation 8, which is based on the mean plus one standard deviation value of E_H/E_I ratios from extensive calculations. Values given by Equation 8 are compared with time history analysis results in Figure 8.

$$\frac{E_H}{E_I} = \begin{cases} 1 & (\mu = 1) \\ -0.0146T + b_1 & (\xi \neq 0, 1 < \mu \leq 2) \\ \left[-0.0146 - 0.0284(\mu - 2)\right]T + b & (2 < \mu < 3) \\ -0.043T + b_2 & (\mu \geq 3) \end{cases} \tag{8a}$$

where

$$\begin{aligned} b_1 &= -43.6\xi^3 + 29.87\xi^2 - 7.22\xi + 0.85 \\ b &= b_1 + (b_2 - b_1)(\mu - 2) \\ b_2 &= -17.48\xi^3 + 15.69\xi^2 - 5.25\xi + 1 \end{aligned} \tag{8b}$$

4.2 Demand for hysteresis energy ratio in MDOF systems

Based on extensive analysis results, E_H/E_I ratio of MDOF systems can be obtained by modifying those of corresponding SDOF systems in moderate and long period regions. The following equations are suggested.

$$\frac{E_H}{E_I} = \begin{cases} b_1 & (\xi \neq 0, 1 < \mu \leq 2) \\ b & (2 < \mu < 3) \\ b_2 & (\mu \geq 3) \end{cases} \tag{9}$$

where the values of b_1, b and b_2 are the same as given in Equation 8(b).

5. SYSTEM CAPACITY DESIGN AND ENERGY DISTRIBUTION

5.1 System capacity design

The distribution of hysteresis energy E_H in a structural system is essential to perform the energy-based seismic design. E_H distributions can be very different when the structure is subjected to different earthquake ground motions if its deformation pattern or damage mode is not well-controlled. This has been a major difficulty for a long time to perform energy-based seismic design. As a result, the energy-based method has been applied mainly for SDOF systems. Researches on E_H distribution are quite very limited. Fajfar (1996) proposed a method of determining E_H distribution for reinforced concrete frames and shear wall-frame structures based on pushover analysis. This method was further developed by Chou through modal pushover analysis (MPA) and was applied to steel frames. These methods can still be categorized into some displacement-based design framework and are not fully compatible with the basic principle of the energy-based design, although they found some rational bases for determining of E_H distribution. The energy-based design requires the structure's energy-dissipation capacity to be greater than its energy-dissipation demand. The energy dissipation mechanism should be considered as the first step and be taken as a premise in the following design procedure. It was shown by the author (Ye *et al.* 2002) that structures without carefully chosen energy dissipation mechanisms may suffer from story-mechanisms and the location of damage concentration will be quite arbitrary due to the randomness of earthquake ground motions, even if its mass, stiffness and strength are uniformly distributed along the height. In order to effectively control the energy dissipation mechanism and maximize the system capacity of energy dissipation, "system capacity design" was proposed by the first author (Ye 2004). In system capacity design, the structural system is further divided into two or more sub-systems, one of which is taken as the major sub-system and is responsible for controlling the global structural behavior under severe earthquakes. The major sub-system is designed not to be damaged and most of the energy dissipation occurs in the secondary sub-systems. E_H distribution of thus-designed structural system will be controlled by the major sub-system. The whole structure will behave as a hardening system. Further researches by the authors show that the scattering of the inelastic seismic responses is significantly smaller for hardening systems, which makes the seismic simulation more reliable (Ye *et al.* 2008). As mentioned previously, the design of energy dissipation mechanism is the major difference between the energy-based method and other methods. It is also the basis of energy-based design.

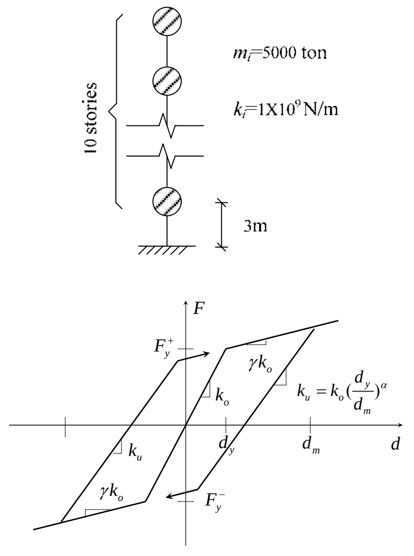
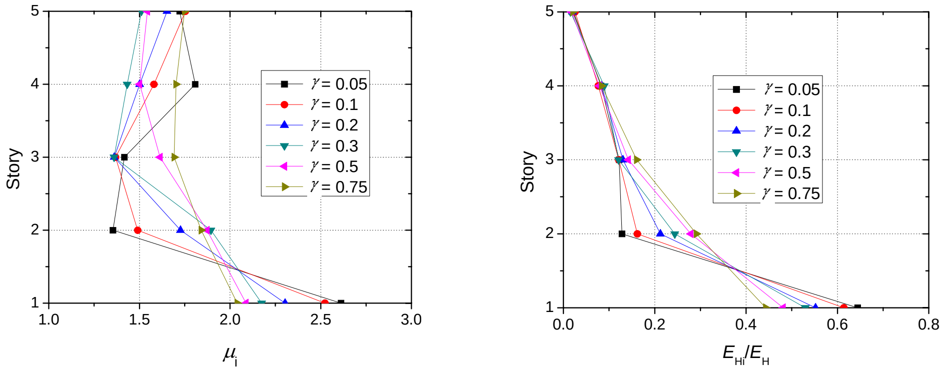
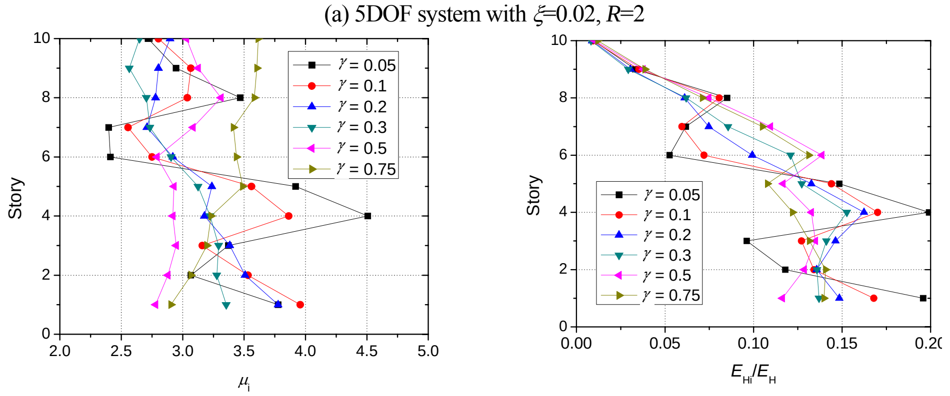


Figure 9 MDOF shear-story model:
(a) Model and (b) Story shear-deformation model

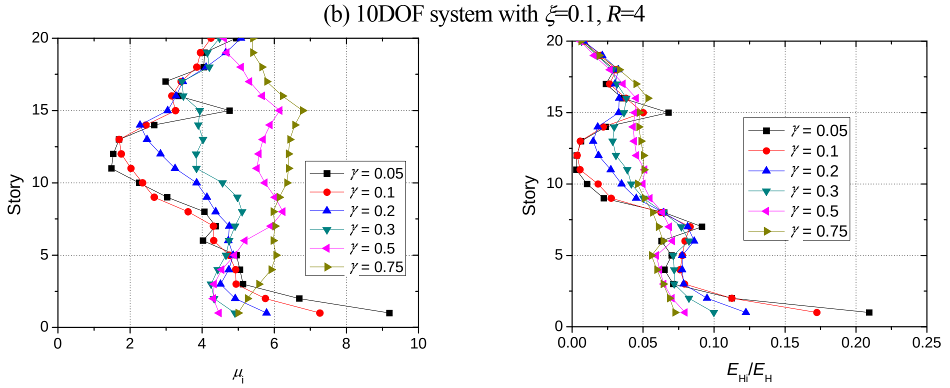
E_H distributions in hardening structure systems are demonstrated by the lumped mass MDOF systems with 5, 10, 20 and 30 degrees of freedom, as shown in Figure 9, are analyzed. Bilinear hysteresis model is adopted. The post-yielding stiffness ratios γ are taken to be 0.05, 0.1, 0.2, 0.3, 0.5 and 0.75, the damping ratios ξ to be 0.02, 0.1, 0.2 and 0.3, the strength reduction factor R to be 1, 2, 4, 6 and 8. Ductility factors μ and cumulated hysteresis energy E_H of individual stories under the El Centro NS ground motion are shown in Figure 10. When the post-yielding stiffness ratio is small (γ<0.5), μ and E_H are badly distributed and tend to concentrate at certain stories.



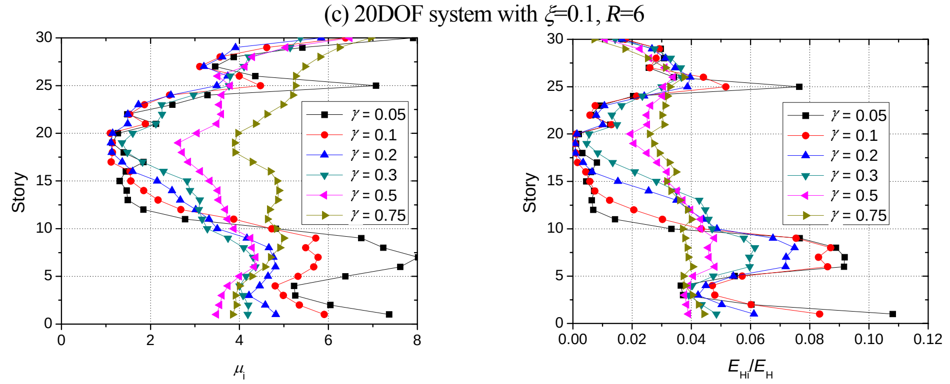
(a) 5DOF system with ξ=0.02, R=2



(b) 10DOF system with ξ=0.1, R=4



(c) 20DOF system with ξ=0.1, R=6



(d) 30DOF system with ξ=0.02, R=8

Figure 10 Distribution of cumulated hysteresis energy

The concentration becomes even severer for systems with more degrees of freedom. When $\gamma \geq 0.5$, μ and E_H distribution becomes uniform and $\mu \approx R$. Larger post-yielding stiffness ratios lead to much smaller inter-story drifts and more uniformly distributed inelastic deformation and damage more uniform.

Based on the analysis of lumped mass MDOF, Nakashima *et al.* (1996) concluded for frames with displacement-related dampers that the deformation concentration can be avoided if no column yields at both of its ends and the post-yielding stiffness ratio $\gamma \geq 0.75$. The work by Connor and Wada *et al.* (1997) shows that this can be done for frame models if $\gamma \geq 0.33$.

The basic concept of system capacity design is to obtain hardening behavior in the system level. Various measures should be taken to provide a global major sub-system in the structural system. This major sub-system should have very high strength and enough deformability and the damaging sequence in the whole structural system should be well-organized.

5.2 Method for calculating E_H distribution

Three methods of determining E_H distribution have been so far proposed: (1) Uniform distribution (Akiyama 1985); (2) Linear distribution along height (Shen *et al.* 1999 and Akbas *et al.* 2001); (3) Pushover analysis-based method (Chou *et al.* 2003). Uniform distribution has proved inaccurate. The following discussion shows that the linear distribution and the pushover-based distribution can be achieved under certain conditions.

(1) Linear distribution

Akbas *et al.* (2001) concluded that E_H distribution is almost linear for regular steel frames with damping ratio of 0.02. According to the present research, however, E_H distribution can approximately be taken as linear only when the damping ratio $\xi > 0.1$. As a result, the assumption of linear E_H distribution is applicable for structures with additional damping devices. The following linear equations are suggested to calculate E_H distribution:

$$\frac{E_{\mathrm{H}i}}{E_{\mathrm{H}}} = \begin{cases} \dfrac{2(N+1-i)}{N(N+1)} & N < 5 \\ \dfrac{2(N-i)}{N(N-1)} & N \geq 5 \end{cases} \tag{10}$$

where N is the total number of story and i is the story number. The equation is compared with E_H distribution given by time history analysis in Figure 11. For structures with large damping ratios, Equation 10 agrees well with time history analysis results.

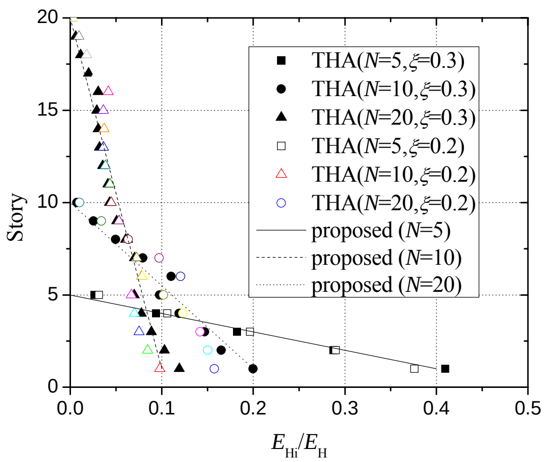


Fig. 11 Proposed linear distribution equation (Eq. 10) of E_H

(2) Pushover analysis-based method

E_H distribution is highly affected by the structure's characteristics when the damping ratio $\xi \leq 0.1$. Pushover analysis is used to determine E_H distribution by Chou *et al.* (2003)[0]. Pushover analysis is capable of estimating the structural peak responses while E_H is cumulated through the whole dynamic process of the structural response to earthquake excitations. As a result, the relationship between E_H and the structural peak response need to be first established. A linear correlation between them can be found when the post-yielding stiffness ratio $\eta \geq 0.5$. Figure 12 compares the plastic deformation energy E_{pi} and cumulated hysteresis energy in individual stories. The plastic deformation energy E_{pi} is defined in Equation 11, which is taken herein as a typical peak response.

$$E_{\mathrm{p}i} = (1-\eta_i)(\mu_i - 1) F_{\mathrm{y}i} d_{\mathrm{y}i} \tag{11}$$

Good correlations are observed for systems with various numbers of DOF and damping ratios. Equation 12 is suggested to determine E_H distribution from peak story drifts.

$$\frac{E_{\mathrm{H}i}}{\sum_{i=1}^{N} E_{\mathrm{H}i}} = \frac{E_{\mathrm{p}i}}{\sum_{i=1}^{N} E_{\mathrm{p}i}} \tag{12}$$

Modal pushover analysis should be used to determine the plastic deformation energy E_{pi} in Equation 11 for high-rise buildings and other structural systems where higher modes contribute a lot.

6 ENERGY-BASED SEISMIC DESIGN OF BRACED STEEL FRAMES

Two parameters are introduced to better describe the concept of system capacity design: capacity coefficient and capacity ratio. The capacity coefficient of an individual component or a sub-system is the ratio of its actual strength capacity to required strength capacity. The capacity ratio is taken as the ratio of the capacity coefficients of different components or sub-systems. By adjusting the capacity ratios of different sub-systems, secondary sub-systems are expected to be damaged and dissipate energy while the major sub-system keeps elastic and controls the global response of the whole system when the system is subjected to a certain level of earthquake. The capacity ratios can be adjusted in the following two ways:

(1) Increase the capacity coefficient of the major sub-system, especially the elastic deformation capacity of the major sub-system in order to increase the post-yielding stiffness of the whole system. High strength and high performance materials are preferable for the major sub-system.

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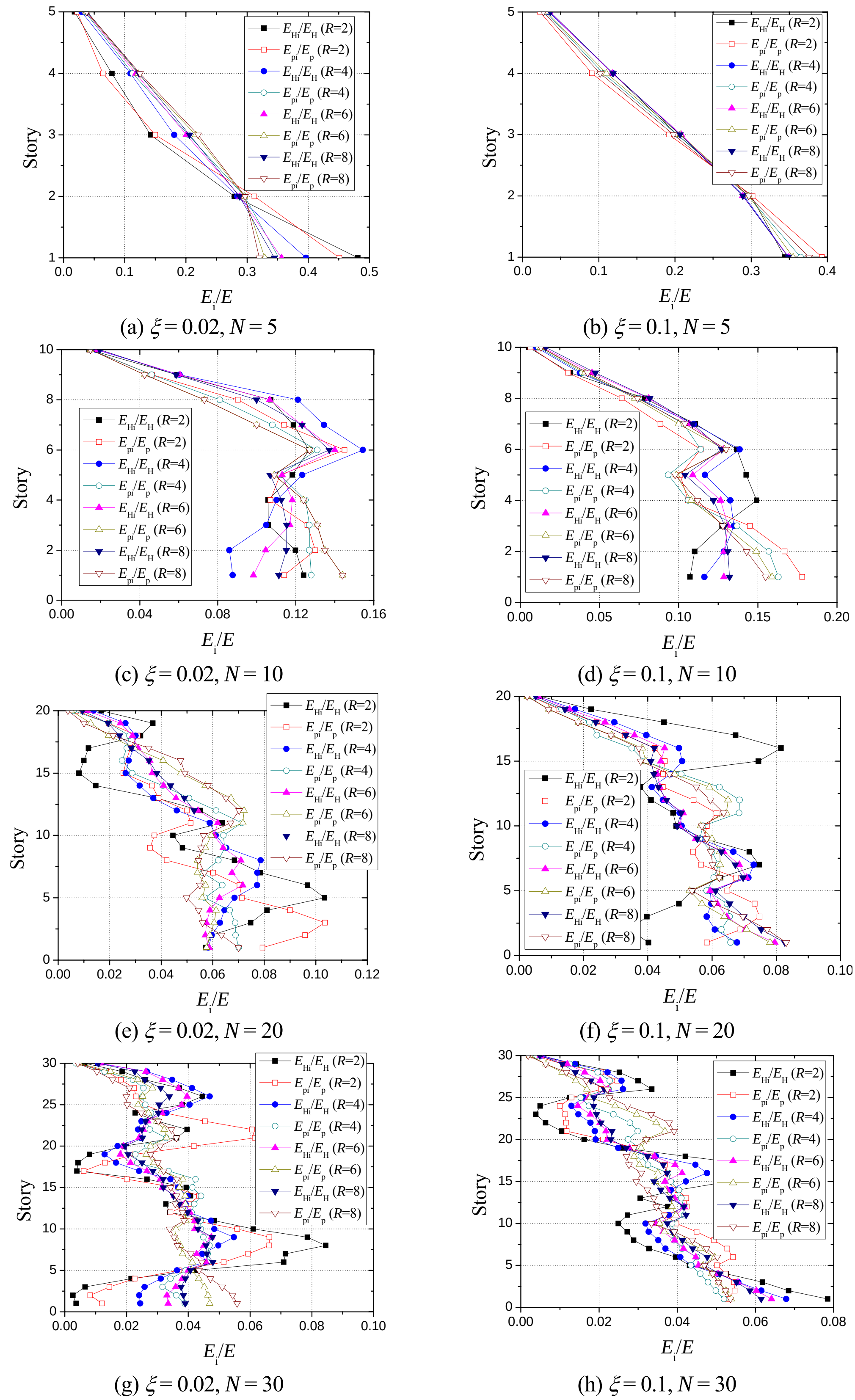


(a) $\xi = 0.02$, $N = 5$ (b) $\xi = 0.1$, $N = 5$

(c) $\xi = 0.02$, $N = 10$ (d) $\xi = 0.1$, $N = 10$

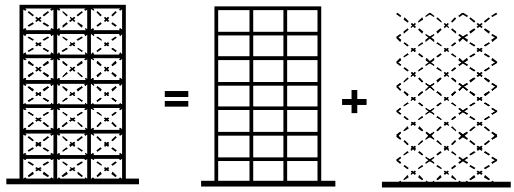
(e) $\xi = 0.02$, $N = 20$ (f) $\xi = 0.1$, $N = 20$

(g) $\xi = 0.02$, $N = 30$ (h) $\xi = 0.1$, $N = 30$

Figure 12 Energy distribution ($\eta = 0.5$)

(2) Distinguish the major sub-system from secondary ones by changing the structural layout. A way of doing this is to make components in the major sub-system insensitive to lateral deformations while those in secondary sub-systems are sensitive

The above energy-based seismic design method is applied to a steel braced frame as shown in Figure 13. As the vertical load is mainly carried by the frame columns and beams, their performance determines the safety of the whole structural system. As a result, the frame becomes the major sub-system in this system and columns are even more important. Steel braces mainly resist the lateral forces. Their yielding will not affect the vertical capacity of columns. At the same time, braces are more sensitive to lateral deformations than columns. So braces belong to the secondary sub-system.



(a)Whole system (b) Major system (c) Secondary system

Figure 13 Braced steel frame and its system hierarchy

The performance objects for the major and the secondary sub-systems are prescribed as below according to the 3-level fortification criterion in Chinese seismic design code:

(1) When subjected to minor earthquakes, both the major and secondary sub-system keep elastic.

(2) When subjected to moderate or design earthquakes, the secondary sub-system (braces) may be damaged and the major sub-system keeps elastic. Rehabilitation is only required checking for the secondary sub-system after the earthquake.

(3) When subjected to major earthquakes, the secondary sub-system (braced) can be heavily damaged but not exceed its deformation capacity. Some frame beams in the major sub-system may be moderately damaged and the columns keep elastic.

High strength steel is used for frame columns in order to achieve the desirable energy dissipation mechanism and the above performance objects and to ensure that the post-yielding stiffness ratio of the whole system $\eta \geq 0.5$. Mild steel with large elongation ratio and low yield strength is used for braces. Material properties are listed in Table 3.

Table 3 Material properties

Comp.	Steel Grade	Steel Type (mm)	f_y(MPa)	f_u(MPa)	Elongation (%)
Column	HT590	19~100	440~540	590~740	≥20
Beam	Q235B	10~40	205~235	375	≥20
Brace	LY100	6~12	90~130	200~300	≥50

The structure is located in VIII seismic fortification area with site condition of Grade II. Peak ground accelerations associated with minor, moderate and major earthquake levels are taken to be 70gal, 200gal and 400gal, respectively. The story height is 5m for the first story and 4m for other stories. The total height of the building is 61m. Every story carries the same dead load of 6kN/m^2 and live load of 2kN/m^2. The structural layout is shown in Figure 14.

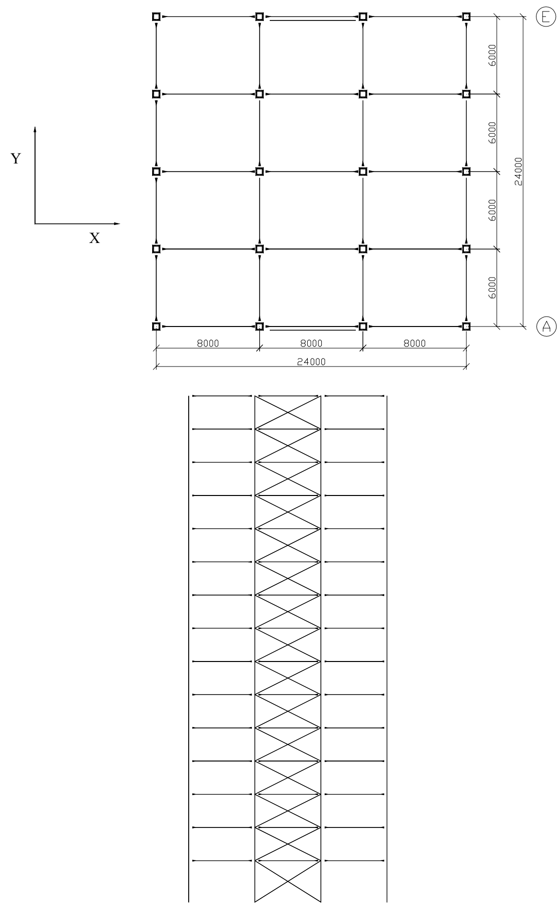


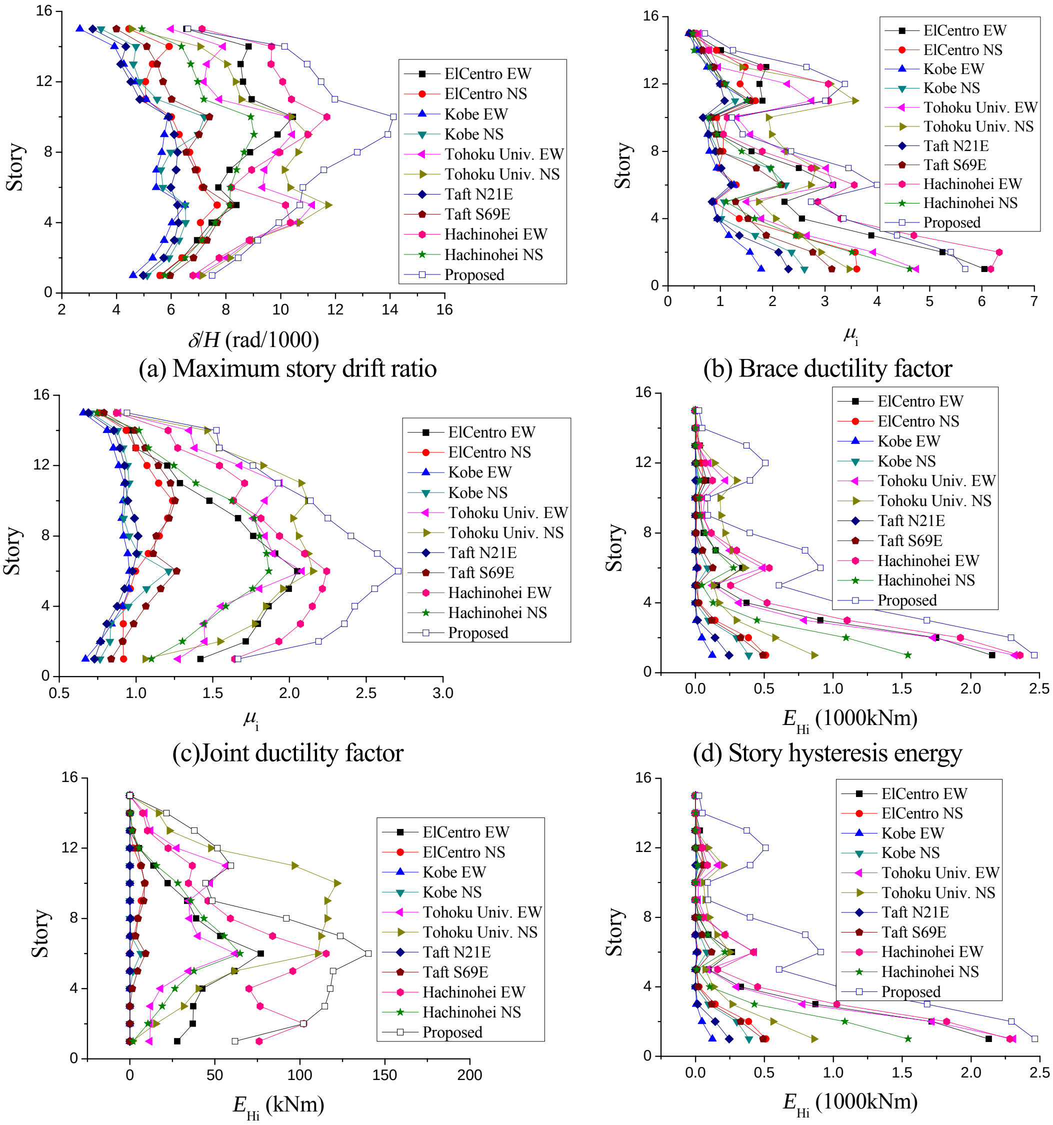
Fig. 14 Structural layout: (a) Plan and (b) Elevation A-E

The cross section geometries of all the components are listed in Table 4. The lateral stiffness of the secondary sub-system is designed to be about 2 times that of the major sub-system. The fundamental period of the structure is 1.62s.

Table 4 Component section dimensions

Story	Column(mm)	Beam(mm)	Brace(mm^2)
1~5	□500×500×20×20	I 500×300×12×20	24464
6~10	□400×400×20×20	I 500×300×12×20	18400
11~15	□350×350×20×20	I 500×300×12×20	13216

By evaluating the E_H distribution and energy dissipation demands for individual components in accordance with the proposed energy-based design method, the braced steel frame proves capable of meeting the energy dissipation demand under major earthquakes. Story responses given by the proposed method and by the time history analysis of 10 ground motion records are compared in Figure 15. The total cumulated hysteresis energy and its distribution given by the proposed method is conservative for engineering purpose compared with the time history analysis results.



(a) Maximum story drift ratio (b) Brace ductility factor

(c)Joint ductility factor (d) Story hysteresis energy

(e) Joint hysteresis energy (f) Brace hysteresis energy

Figure 15 Comparison between nonlinear time-history analysis and the proposed method

7. CONCLUSIONS

The energy-based seismic design is reviewed and developed in the following aspects: (1) By considering the influences of ground motion characteristics and structural properties on E_I spectra of SDOF systems, elastic E_I spectra for various damping ratios and inelastic E_I spectra for various damping ratios and ductility factors are established. Spectra of hysteresis energy ratio E_H/E_I considering the influences of both ductility factors and damping ratios are also established.

(2) Influences of structural properties on the input energy and its distribution are examined based on inelastic MDOF shear systems. Relationships between energy input and its distribution of inelastic MDOF systems and inelastic SDOF systems are established.

(3) The influence of the post-yielding stiffness ratio on E_H distribution is examined in accordance with the system capacity design concept and the requirement for system hardening. Methods of determining the E_H distribution are proposed and their limitations are summarized.

(4) Energy-based seismic design method for steel braced frames is proposed in accordance with the system capacity design concept. Its validity is demonstrated through a case study.

Energy-based seismic design is an important part in the performance-based seismic design framework. It can be used to comprehensively assess the structural performance and hence ensure the structural safety under severe earthquakes together with structure control concepts and displacement-based methods. More studies are still needed in the following aspects:

(1) Peak ground accelerations have proved not adequate as an earthquake intensity index to represent the structural seismic capacity. More advanced earthquake intensity indices, which can represent the complete characteristics of earthquake ground motions, are required as

a basis to improve energy spectra.

(2) More widely applicable and simpler methods of determining the hysteresis energy distribution in MDOF systems are still needed.

(3) Energy-based seismic design is a supplement to displacement-based methods. Besides the strength and the deformation, the cumulated hysteresis energy is also a design index for individual components. The establishment of methods of evaluating and designing the energy capacity of various structural components requires comprehensive experimental and theoretical studies.

(4) The current study mainly focuses on regular structures. In theory, energy-based seismic design method is also applicable in irregular structures. More researches are needed in this field.

Acknowledgement:

The present research is sponsored by the Ph.D Foundation of Education Ministry of China (No. 200800030001) and the key project of National Natural Science Foundation of China (No. 90815025).

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