SEISMIC ENERGY RESPONSE OF TWO-STORY X-BRACED FRAMES

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ABSTRACT

A structure subjected to strong ground motion is expected to show nonlinear behaviour. Energy parameters are a way to specify the structural damage. Energy input depends on the characteristics of the structure and ground motion. Structural design in energy-based design can be defined as the equilibration of the energy input and the energy dissipation capacity of the structure. Structures subjected to earthquake are supposed to dissipate all the energy input. Studies based on energy concepts are usually applied to single-degree-of-freedom (SDOF) system. For multi-degree-of-freedom (MDOF), more researches and new simpler methodologies are still needed in performance based evaluation including energy parameters. In this study, low– and medium-rise steel two-story X-braced frames will be studied through linear and nonlinear dynamic time history analysis. The results obtained from these analysis will be presented in terms of drift ratios and seismic energy demands.

KEYWORDS: Energy Demand, Seismic Design, Seismic Response, Steel X-Braced Frame

1. INTRODUCTION

Seismic design of structures is generally strength-based. The structural members are selected based on the principle that the strength supply from the structure should not be less than the strength demand on it. Nevertheless, the assigned level of the seismic force in the seismic provisions provides only a minimum lateral resistance level, but is not intended to design the structure to be without damage in the design earthquake. A structure subjected to strong ground motion is supposed that it shows nonlinear behavior. Energy parameters is a way to specify the structural damage. In consequence of the damage including inelastic deformation, to estimate the damage in a structure is considered nonlinear behavior. Performance-based seismic design of structures calls for a rational and rigorous design approach in its conceptual framework and has been widely used for the last two decades. Performance-based design (PBD) approach accounts for realistic seismic performance of a structure and is considered to be more rational seismic design approach in its structural design stage.

Energy-based design approach provides an alternative in PBD. Energy and energy-based parameters has long been considered to be the most rational parameters to design a structure subjected to moderate or severe EQGMs. The earthquake effect on the structure can be considered as energy input, a function of the structural properties and characteristics of the earthquake ground motion. The structure must dissipate all the energy imparted to it during an EQGM to survive. Despite high irregularity of EQGMs, the energy input to a structure is considered to be a stable quantity. The energy input is a function of the characteristics of the EQGM (effective strong motion duration, amplitude, frequency content, etc.) and structural properties (mass, fundamental natural period, etc.).

Structural design can be defined as the equilibration of the input energy and the energy dissipation capacity of the structure. Structures subjected to earthquake are supposed to dissipate all the input energy. A part of input energy turns into elastic strain energy and kinetic energy, others are dissipated damping and inelastic behaviour of the structural and non-structural members. Studies based on energy concepts are usually applied to single-degree-of-freedom (SDOF) system. For multi-degree-of-freedom (MDOF), more researches and new simpler methodologies are still needed in performance based evaluation including energy parameters.

In this study, low and medium rise steel moment frames and will be studied in linear and nonlinear time history analysis. Two story X- braced steel frames in 4- and 8- story with same span length representing typical low and medium- -rise steel buildings are designed based on the seismic design requirement in ASCE 7-10 and AISC 341-10. An ensemble of ground motions are selected so that the seismic response of each of the three 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 are used in nonlinear first mode incremental dynamic time history analyses. The results obtained from these analysis are reviewed for drift ratios, hysteretic energy and total input energy.

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 (supply). The demand should be equal or smaller than the supply for a proper design. The following equations of motion and energy are useful to defined the energy concept.

The equation of motion for a viscous damped SDOF system subjected to a horizontal earthquake ground motion can be written as:

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

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) can be rewritten as:

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

İt is necessary to derive the energy equations to develop reliable design methods based on an energy approach. 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 (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 right side of equation (3) represents the total (or seismic) energy input, $E_I(t)$, to the structure and is defined as the work done by the effective seismic force (the mass times ground acceleration) over the structural deformation.

$$E_I(t) = -\int_0^u m\ddot{u}_g(t)du \tag{4}$$

The first term on the left side of equation (3) is the kinetic energy, $E_K(t)$ and can be found by multiplying half of the mass with its motion relative to the ground. $E_K(t)$ is proportional to relative velocities of masses at time t, which is only related to the instant response of the structure at time t.

$$E_K(t) = \int_0^u m\ddot{u}(t)du = \int_0^u m\dot{u}(t)d\dot{u} = \frac{m\dot{u}^2}{2} \tag{5}$$

The second term on the left side of equation (3) is the energy dissipated by damping, $E_D(t)$.$E_D(t)$ is physically interpreted as the energy dissipated by the viscous damping of the system and a cumulative quantity, ever increasing with the time during the vibration.

$$E_D(t) = \int_0^u f_D(t)du = \int_0^u c\dot{u}(t)du \tag{6}$$

The third term on the left side of equation (3) is the sum of the hysteretic energy, $E_H(t)$, and the elastic strain energy, $E_e(t)$. $E_e(t)$ is an instant quantity depending on the current elastic deformation level at time t.

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

where k is the initial stiffness of the system. And the hysteretic energy, $E_H(t)$ is a cumulative quantity over the plastic deformation throughout the entire duration of the vibration, and will be zero if the structure remains elastic.

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

E_H includes the inelastic deformation of structural members and is directly related to the cyclic deformation capacity of structural components. In an elastic response, E_H is equal to zero, whereas E_e is negligible compared to E_H in an inelastic response. At an instant time t, E_k and E_e can be computed from Eqs. 5 and 7, respectively. Based on the above-defined energy quantities, the energy response of a nonlinear system can be written as:

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

If Eq. (9) were considered as a design equation (demand$\leq$ capacity), 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 instant kinetic energy and elastic strain energy consist of relatively small portion of the E_I at any time during the vibration and vanish at the end of the vibration. The E_D and E_H, therefore, are major contributors for dissipating the E_I. Thus, E_k and E_e are negligible in an inelastic response and Eq. (9) can be practically written as:

$$E_I(t) = E_D(t) + E_H(t) \tag{10}$$

The quantities in Eq. (10) at the end of the EQGM can be determined and the distribution of hysteretic energy throughout the structure can be evaluated for a given structure and EQGM. Therefore, the energy terms in equation (9) represent the energy values obtained from the motion of the system relative to the base rather than the due to the total motion.

3. EVALUATION OF THE SEISMIC ENERGY DEMANDS IN ANALYTICAL STUDY

3.1 *Description of the Buildings*

Two story X- braced steel frames with 4 and 8-story, representing typical low and medium high-rise steel buildings were designed based on the seismic design requirements for special concentrically braced frames

(SCBFs) in accordance with ASCE 7-10 , AISC 360-10 , AISC 341-10. The plan dimensions of buildings are (width) 45 m and (depth) 45m with constant span length of 9.0 m (five equal spans) for all stories.

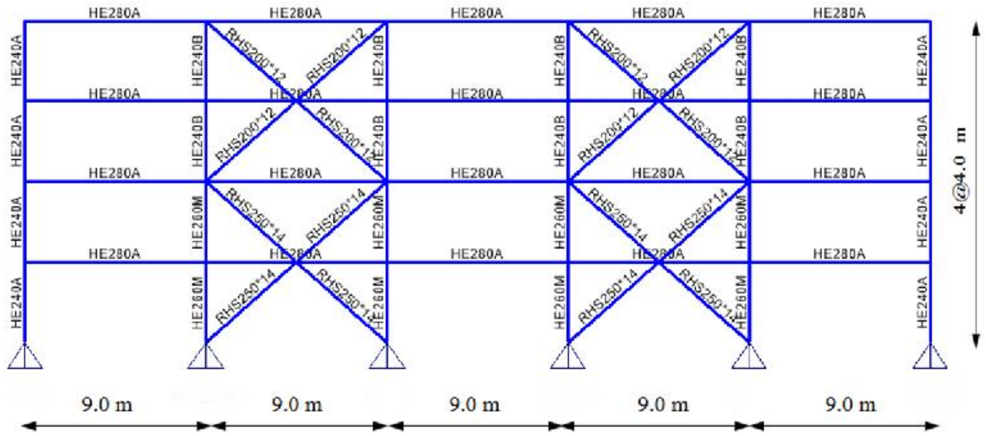

Figure 1. Elevation of the 4-story frame

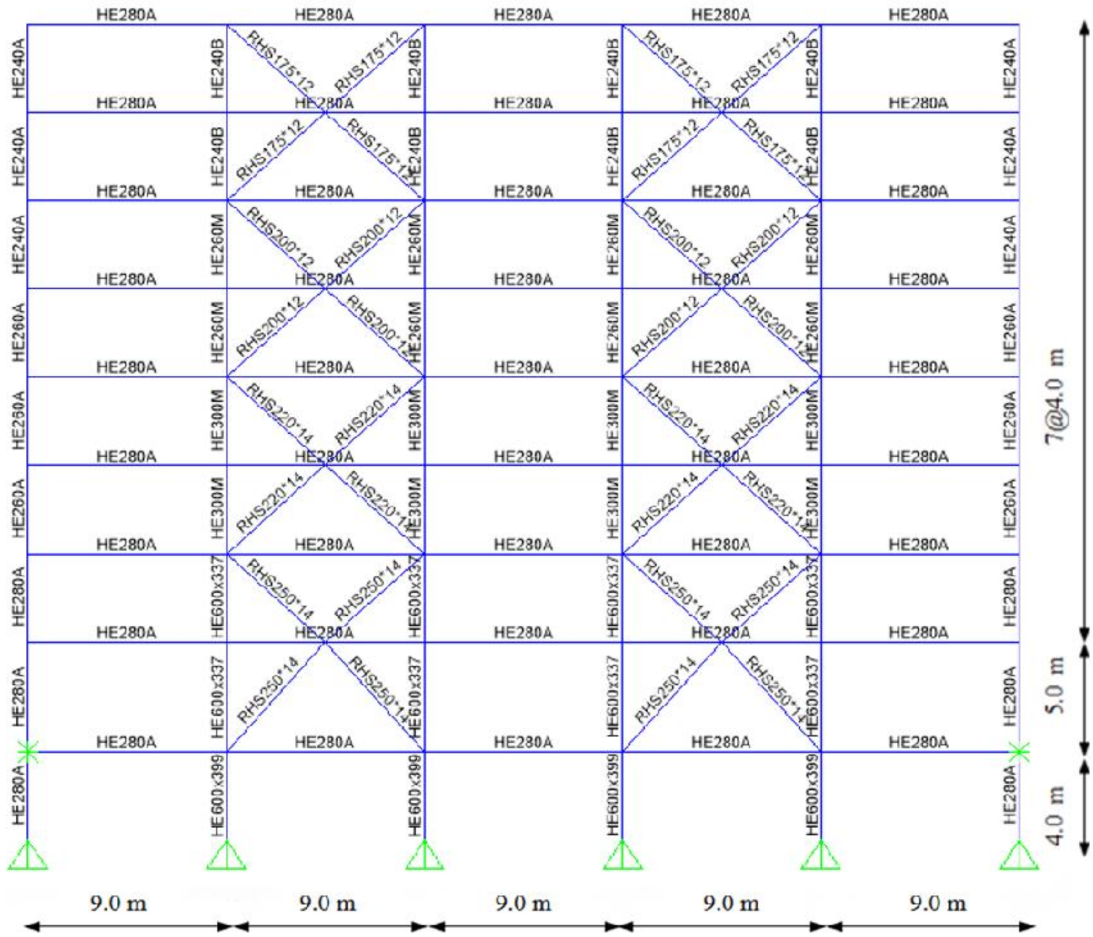

Figure 2. Elevation of the 12-story frame

The typical story height for the 4- and 8- stories frame is 4.0 m and the height of ground for the 8-story frame is 5.0 m in Fig. 1 and 2. For 8- 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 are applied by using Load and Resistance

Factor Design (LRFD) specification in accordance with AISC 360-10 standard. 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 in except at the roof level, where it is 4.0 kN/m^2 and 1.4 kN/ m^2, respectively. European wide flange profiles are preferred with quality of S355. The yield stress of braces in the building are preferred with quality of S275. The structural system for each building consists of steel perimeter moment resisting frames and interior simply-connected framing for gravity, i.e. lateral loads are carried by perimeter frames and interior frames are not explicitly designed to resist seismic loads in the direction of the earthquake.

Shear wave velocity (V_s) of site class is between value of 300 m/s and 770 m/s. These ground motions were scaled to comply with the response spectra given in Figure 5. The acceleration time histories are given in Fig. 5 and 6. Design response spectrum curve are developed in accordance with ASCE 7-10 as indicated in Fig. 5 and 6. Modal Response Spectrum Analysis are used in seismic design based on the structure's seismic design category D. Also, equivalent lateral load procedure are calculated to compare the base shears and if needed , forces are scaled. The calculated fundamental period of the structure (T) are checked with the approximate fundamental period (T_a) and coefficient of upper limit (C_u). Where the calculated fundamental period (T) exceeds C_uTa , C_uT_a is used instead of T in accordance with ASCE 7-10. Structural system is checked by using amplified seismic load effect including overstrength factor. The fundamental periods of vibration, the equivalent lateral force procedure base shear (V), the base shear from the required modal combination (V_t) for the frames are given in Table 1.

Table 1. Fundamental periods and base shears for the moment frames

Story	**T (sec)**	**T_a (sec)**	**C_u**	**C_uT_a (sec)**	**T_{used} (sec)**	**Total Mass (kN.sec^2/m)**	**V (kN)**	**V_t (kN)**
4-STORY	0.593	0.390	1.4	0.546	0.546	2173	4337	4364
8-STORY	1.582	0.656	1.4	0.918	0.918	4485	5322	4812

The floor system of the buildings is assumed to provide diaphragm action and to be rigid in the horizontal plane. In design of steel moment 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. The buildings were designed for a site where S_S is 2.0g and S_1 is 1.0g. Seismic parameters for design spectrum are S_{DS} =1.333 (g), S_{D1}= 0.666 (g) and long-period transition period, T_L = 12.0s. Two sets of ground motion records were selected from PEER Strong Ground Motion Database corresponding to 10% and 2% probability of exceedance. Each set consists of 3 records. The frames were designed by selecting an assumed 2% target drift in ASCE7-10. The columns of the buildings are assumed to be simply connected to the foundation.

3.2 *Analyses Results*

The Incremental Dynamic Analysis (IDA) were carried out to study the energy response parameters on the 4- and 8-story frames. The frames were subjected to two sets of ground motions (a total of 6 ground motions) .Table 2 lists the detailed information of these ground motions. Two sets of ground motions corresponding to 10% and 2% probability of exceedance are used in IDA. Column and beams are modeled by PERFORM 3D which is nonlinear time history analysis software. Braces were modeled as an "Inelastic Steel Bar". Beams in braced by are modeled as a column element because of the unbalanced force caused by braces. The interaction between the axial force and bending moment was considered in columns. P-Δ effects were always included in the time-history analyses. As specified in Fig. 3 and Fig. 4, ground motions corresponding to 10% probability of exceedance are presented as GM 1, GM 2 and GM 3 that cause moderate structural damage and ground motions corresponding to 2% probability of exceedance are presented as GM 4, GM 5 and GM 6 that cause heavy structural damage. Fig. 5 and 6 summarizes the elastic response spectra of the ground motions.

Table 2. Earthquake Ground Motion Characteristics from PEER Database

	NGA#	Record	Scale Factor	Duration (sec)	PGA (cm/sec^2)

GM 1 (%10)	1612	Düzce	3.5588	41.0	531.89
GM 2 (%10)	4284	Basso,Tirreno	3.9035	29.00	572.00
GM 3 (%10)	587	New Zealand	1.451	50.00	404.17
GM 4 (%2)	1111	Kobe	1.3103	41.00	620.973
GM 5 (%2)	4099	Park Field	2.2397	21.00	1047.708
GM 6 (%2)	4481	L'Aquila	1.7551	61.00	828.945

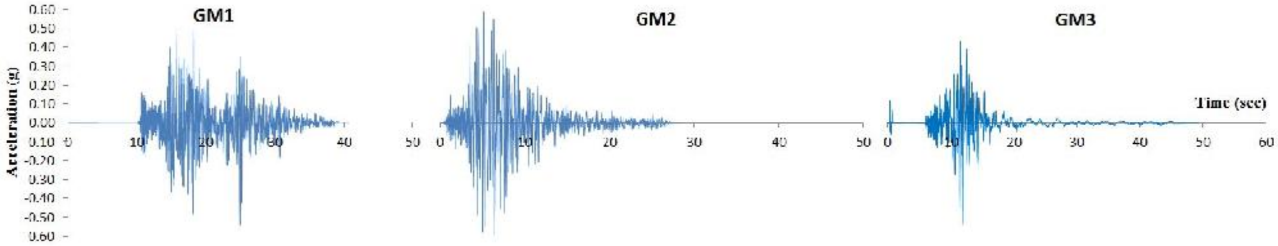

Figure 3. Time histories of the ground motions corresponding to 10% probability of exceedance

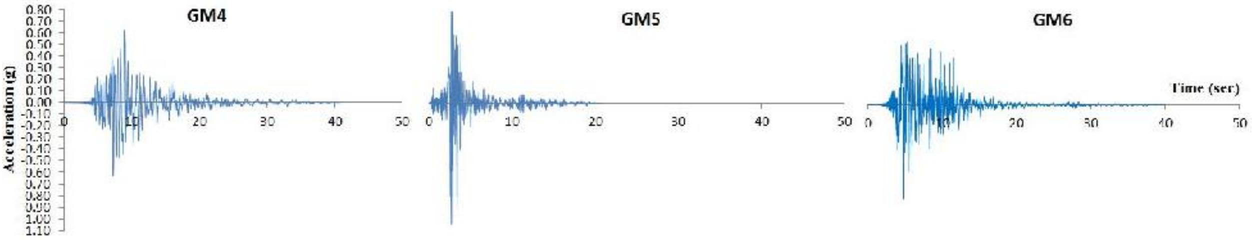

Figure 4.Time histories of the ground motions corresponding to 2% probability of exceedance

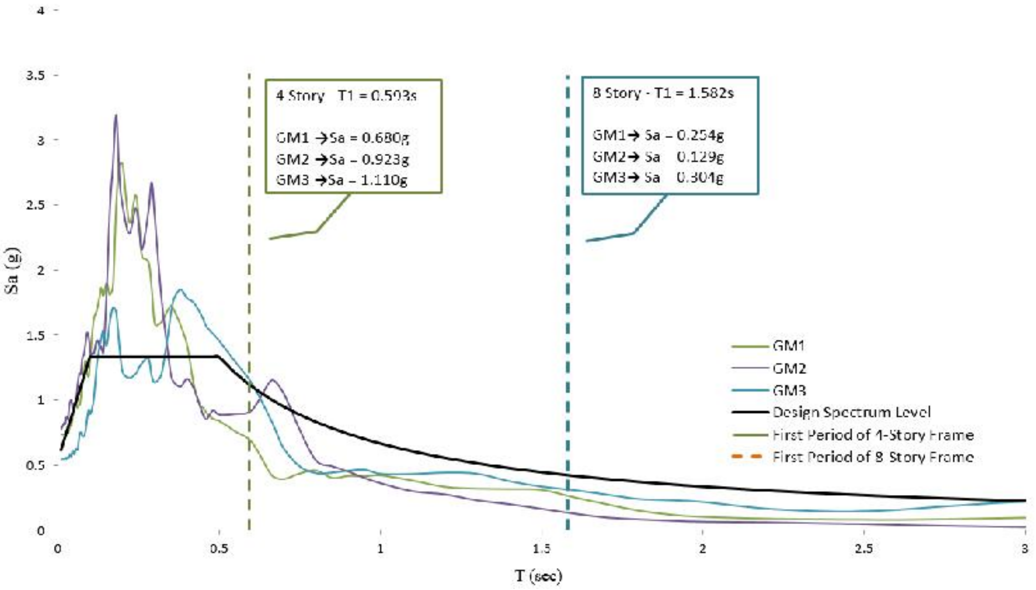

Figure 5. Response Spectra of the ground motions (GM 1, GM 2, GTM 3) selected from PEER database for design earthquake level corresponding to 10% probability of exceedance

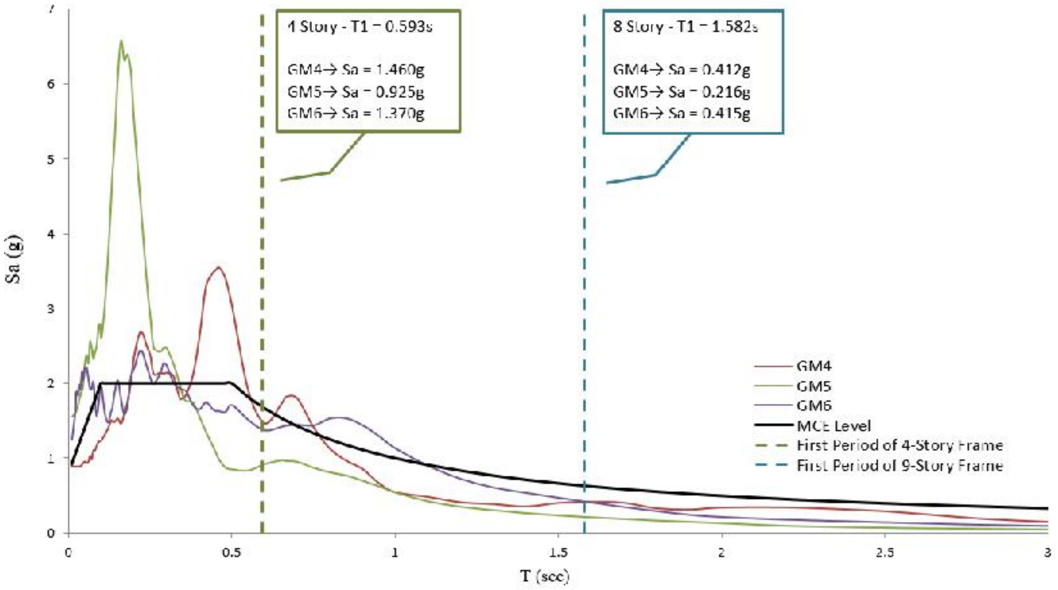

Figure 6. Response Spectra of the ground motions (GM 4, GM 5, GTM 6) selected from PEER database for MCE level corresponding to 2% probability of exceedance

In The Incremental Dynamic Analysis (IDA), selected ground motions are scaled corresponding to their first period spectral acceleration. The scale factor (SF) was determined in such a way that the ground motion intensity with a scale factor of SF would have a spectral acceleration, Sa, equal to (SF) g. For example, a scale factor of 1.00 means that the scaled ground motion has a 1.00g spectral acceleration at the fundamental period. In each step from 0.1g to 2.5g, 0.1g scale factor increment is used for both 4- and 8- stories frames. Fig. 5 and Fig. 6 also show spectral accelerations values of selected ground motions for fundamental periods of 4-story, 8 story frames.

Maximum drift ratios in frames for each steps under selected ground motions are given in Fig.11.

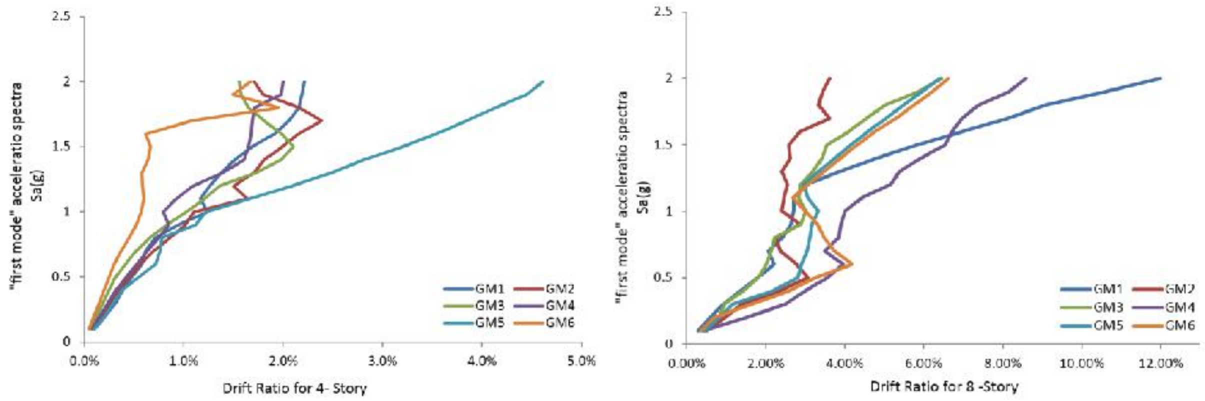

Figure 11. Peak drift ratio development of frames subject selected ground motions

The seismic energy response parameters; total energy input and hysteretic energy are normalized with respect to mass in each case (S_a =1.0 g and S_a=2.0 g) are given in Fig.7 , 8, 9, 10.

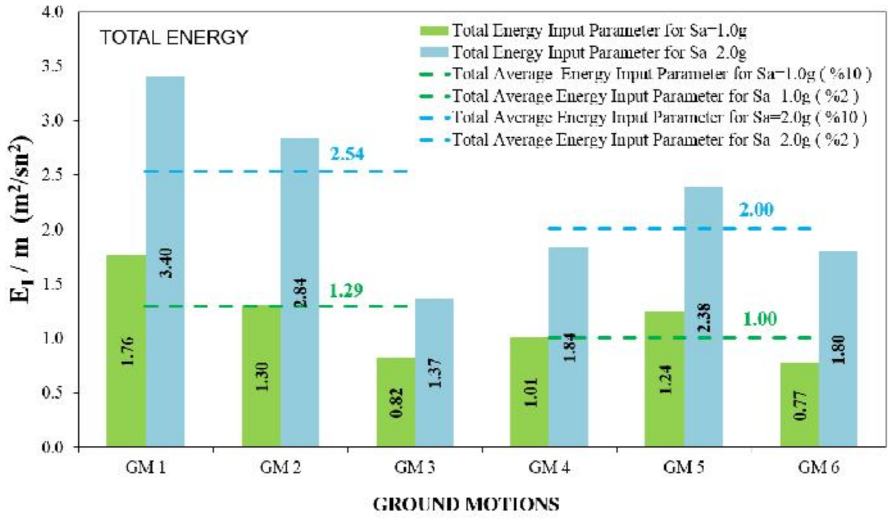

Figure 7. E_I/m for the 4-story frame corresponding to Sa=1.0g and Sa=2.0g

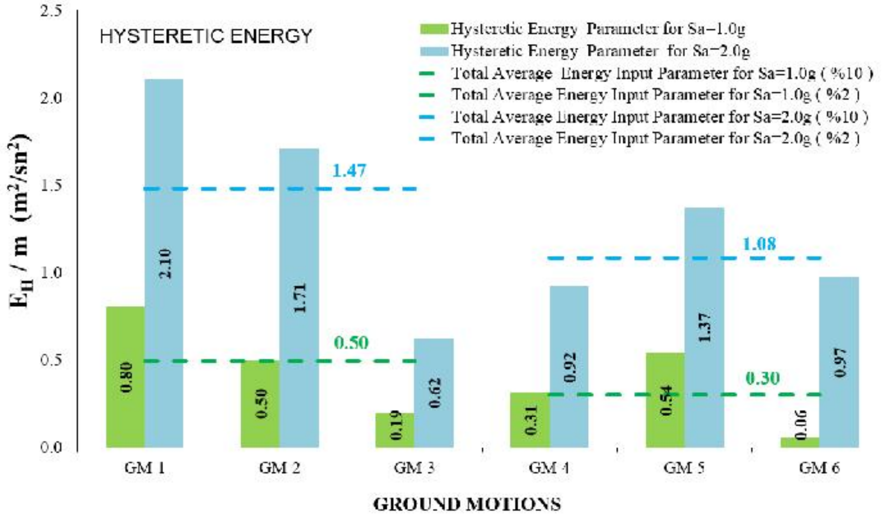

Figure 8. E_H/m for the 4-story frame corresponding to Sa=1.0g and Sa=2.0g

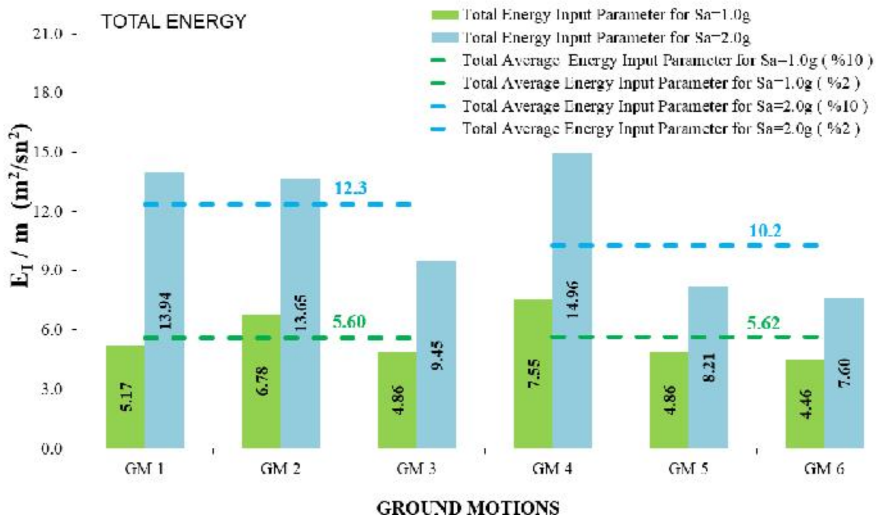

Figure 9. E_I/m for the 8-story frame corresponding to Sa=1.0g and Sa=2.0g

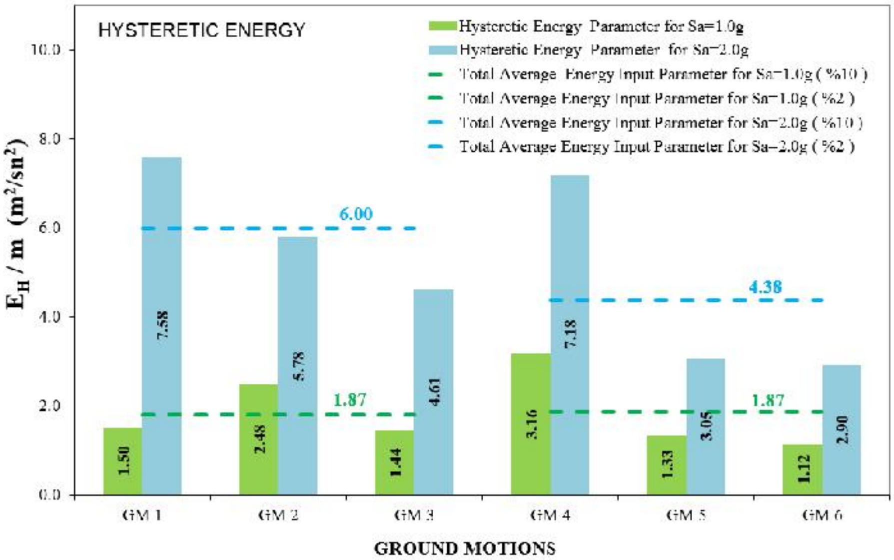

Figure 10. E_H/m for the 8-story frame corresponding to Sa=1.0g and Sa=2.0g

4. CONCLUSION

An energy-based design evaluation of typical low- and medium -rise steel buildings in the framework of performance-based earthquake engineering is presented in this study. Special concentrically two strory X-braced frame (SCBFs) is selected as the structural system to demonstrate the rationality, feasibility and further research needs. Two typical steel braced frames with same span lengths in 4- and8- story representing typical low and medium-rise steel buildings are designed according to and incremental dynamic analysis (IDA). An ensemble of ground motions corresponding to 10% and 2% probability of exceedance are selected so that the seismic response of each of the two frames with differen height would range from moderate to severe and the seismic energy demand would be evaluated based on the response of the frames. Based on the results obtained in this study, the following observations can be made.

- Spectral accelerations varies between 1.20g and 1.70g when the 4-story frame reach its 2 % design drift ratio approximately for ground motions having 10% and 2% probability of exceedance.
- Spectral accelerations varies between 0.25g and 0.70g when the 8-story frame reach its 2 % design drift ratio approximately for ground motions having 10% and 2% probability of exceedance.
- Total maximum average energy input parameter used to specify seismic response , E_I/m , for the 4-story frame are 1.29 m^2/sec^2 in spectral acceleration , S_a of 1.0g while it is about 2.54m^2/sec^2 in spectral acceleration , S_a of 2.0g.
- Total maximum average energy input parameter used to specify seismic response , E_I/m , for the 8-story frame are 5.62 m^2/sec^2 in spectral acceleration , S_a of 1.0g while it is about 12.3 m^2/sec^2 in spectral acceleration , S_a of 2.0g.
- Total maximum average hysteretic parameter used to specify seismic response , E_H/m , for the 4-story frame are 0.50 m^2/sec^2 in spectral acceleration , S_a of 1.0g while it is about 1.47 m^2/sec^2 in spectral acceleration , S_a of 2.0g.
- Total maximum average hysteretic parameter used to specify seismic response , E_H/m , for the 8-story frame are 1.87 m^2/sec^2 in spectral acceleration , S_a of 1.0g while it is about 6.00 m^2/sec^2 in spectral acceleration , S_a of 2.0g.
- The ratio of seismic energy demands of the 4- and 8-story frames is approximately same in total energy and hysteretic energy parameters.

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