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The Effect of Easy-Going Steel on KBF's Seismic Behavior

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ABSTRACT: The knee bracing steel frame (KBF) is a new type of energy dissipating frame which enjoys exceptional ductility and lateral stiffness. Rather than the beam-column joint, one end of the diagonal brace in KBF is attached to the knee element. Indeed, the knee element as a hysteretic damper is designed and detailed to behave like a structural fuse by sustaining controlled inelastic deformations as well as by dissipating seismic energy, yet other parts and connections remain elastic. Simultaneously, the lower strength steel is utilized in knee element based on the general concept of easy-going steel (EGS). As the current paper takes into account the effect of easy going steel on KBF's response modification factor, several frames with similar dimensions but varying heights are designed based on the Iranian code of practice. For this purpose, initially the knee elements are substituted with the one made of EGS and subsequently the seismic parameters such as response modification factor and seismic performance levels are compared based on non-linear incremental dynamic analysis (IDA). The average values of response modification factor for these frames have been obtained 11.4 and 11.6 for KO and KE frames respectively. The results reveal that the frames' stiffness and ductility factor with EGS augments by 10% and 6% respectively.

Keywords: Easy-Going Steel (EGS), Increment Dynamic Analysis (IDA), Knee Bracing Steel Frame, Response Modification Factor (R).

INTRODUCTION

In the early twentieth century, structural engineers became conscious of the potential disaster induced by strong earthquakes. In fact, structures designed to resist moderate tremors must enjoy sufficient stiffness and strength to control for the deflection and prevent any possible damage (Maheri and Akbari, 2003). To put it differently, they must be strong and ductile enough to avoid collapses under extreme seismic exposure. With regard to the lateral load resistance in steel frames, the moment resisting frame (MRF) and the concentrically braced frame (CBF) were the two frames frequently applied. Despite the fact that MRF possesses fine ductility owing to flexural yielding

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beam elements yet it suffers from limited stiffness (Mofid and Lotfollahi, 2006). On the other hand, CBF benefits from acceptable stiffness yet it suffers from inadequate ductility due to buckling of the diagonal brace (Maheri and Akbari, 2003). As a matter of fact although stiffness and ductility are two opposing properties, neither the MRF nor the CBF alone could economically fulfill these criteria. Combining the positive features of these two frames into an economical seismic resistant structural system, Roeder and Popov (Roeder and Popov, 1978) proposed the eccentrically braced frame (EBF) where the brace is placed eccentric into the beam-column joint (Maheri and Akbari, 2003). Of late, Ochoa (1986) proposed the knee braced frame (KBF), an alternative system which acts like a ductile fuse to prevent structural collapse under extreme seismic exposures by dissipating energy through flexural yielding. A diagonal brace with at least one end attached to the knee element affords the most elastic lateral stiffnes.

Yet in this system the brace was not designed for compression hence it was allowed to buckle (Naeemi and Bozorg, 2009). Subsequently, Balendra et al. (Balendra et al., 1990; Balendra et al., 1994) re-assessed the system and modified it by another system called the knee braced frame (KBF). In this system, the non-buckling diagonal brace provides the lateral stiffness. To put it another way, the flexural or shear yielding of the knee element provides the ductility under severe tremors. Consequently, the damage is concentrated on a secondary member which can be easily repaired at a low cost (Maheri and Akbari, 2003).

The current paper is an attempt to apply easy-going steel (EGS) into the knee elements in order to improve their seismic behavior such as response modification, ductility factors and performance levels as well. To this purpose, twenty KBFs were designed utilizing the Iranian code for seismic resistant of building and AISC89, further the knee elements were replaced by ones made of EGS. Subsequently, their seismic performances have been evaluated through non-linear incremental dynamic analysis (IDA) and linear dynamic analysis.

Given that the EGS is exploited in very small parts of the structure like active links, it does not affect the total cost of the structures to a great extent (Bahrampoor and Sabouri-Ghomi, 2008).

EASY-GOING STEEL CONCEPT

Human beings have long endeavored to enhance the steel strength and to reduce the size of structural members so that the total weight of structures decreases, moreover it is economical. Yet, it should be taken into account that increasing the steel strength and decreasing the cross section of structural members is not always efficient. In some cases, it is needed to decrease the steel strength as much as possible to improve the structural behavior (Sabouri-Ghomi and Ziaei, 2008; Sabouri-Ghomi, 2004). Examples for such situation are steel structures exposed to an earthquake or severe windy conditions. To augment the energy absorption of frames, applying lower strength steel for knee elements is a useful method. Generally, this lower strength steel is called easy-going steel (EGS) and the best EGS is the pure iron with yield stress between 90 N/mm^2 and 120 N/mm^2. The percentage of the typical elements such as carbon, manganese, silicon and chromium added to iron to make steel are much lower in EGS than other constructional steels. The elasticity modulus of EGS is also equal to that of other constructional steels which significantly enhances its ductility since the part made of EGS yields in smaller displacement as the energy absorption increases. Figure 1 highlights the stress-strain curves of the constructional steel (ST37) and the iron EGS (Sabouri-Ghomi and Ziaei, 2008). Following are some of the advantages of utilizing very low strength steel vis-à-vis carbon steel:

i) Modulus of elasticity is equal in both ST37 and EGS.

ii) Although shear displacement decreases in the system with low strength steel vis-à-vis carbon steel, yet both systems have equal strength (Figure 1.).

iii) Ductility of low strength steel is much greater than carbon steel (Figure 2.) (Sabouri-Ghomi and Ziaei, 2008).

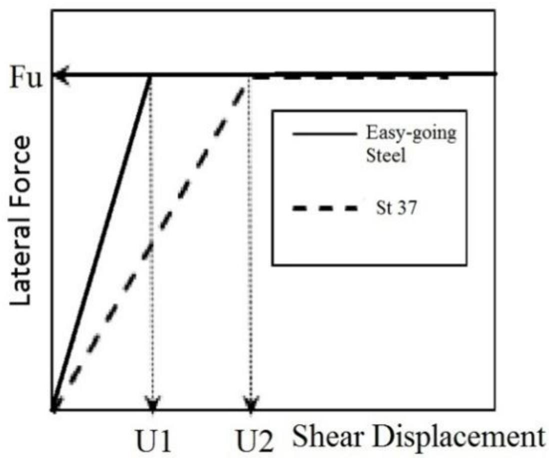

Fig. 1. Comparison between shear force-displacement of members made of EGS and common constructional steel (Sabouri-Ghomi and Ziaei, 2008).

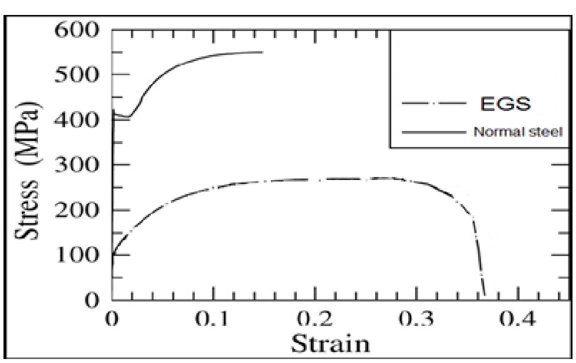

Fig. 2. Stress-strain curve for Iron EGS and constructional steel (Sabouri-Ghomi and Ziaei, 2008).

Given that the ultimate load carrying capacity should not alter by applying EGS in a structure, the thickness of these members should be enhanced due to fewer yield stress of the EGS. To state it differently, the thickness of structural members made of the EGS is greater than the ones made of common constructional steel (Sabouri-Ghomi and Ziaei, 2008). Comparatively, since the thickness of the EGS knee element is increased, local buckling in the flange and webs do not occur and the hysteresis loops are more stable.

The use of the EGS in lateral resisting systems such as KBFs, especially in knee element significantly augments shear stiffness and reduces shear displacement in different stories of a steel structure. As such the moments in the vertical load-carrying members like columns are also decreased with decrement in the lateral displacement. Moreover, undesirable P-Δ effects are considerably diminished by the decrease in lateral displacements.

In order to improve ductility and energy dissipation capacity of the knee braced frames, the current paper utilizes steel with nominal yield stress of 95.4 MPa as the EGS grounded on experimental tests (Susantha et al., 2005).

SAMPLE MODELS OF FRAMES

Figure 3 presents shape of the KBF. As the figure shows, the optimal angle of the knee element is achieved when the tangential ratio of (b/h)/ (B/H) is approximately one (Naeemi and Bozorg, 2009). It implies that the knee element should be parallel to the diagonal direction of the frame and the diagonal element pass through the midpoint of the knee element as well as beam-column intersection (Naeemi and Bozorg, 2009). In the present study, the framing system has been taken equal to 5m length and 3m height. The number of frames are chosen at five levels i.e. 3-story, 5-story, 7- story, 10-story and 12-story level. Figure 4 indicates the 3-story frame.

$H = 3\text{m}$

$B = 5\text{m}$

$$\frac{h}{H} = 0.2 \rightarrow h = 0.6\text{m}$$

$$\frac{b}{B} = 0.2 \rightarrow b = 1.0\text{m}$$

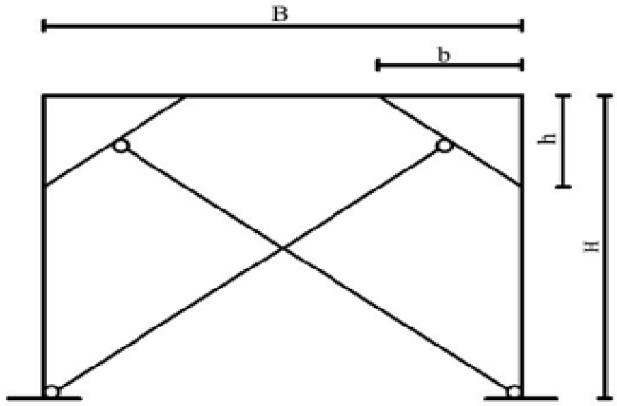

Fig. 3. The shape and dimension of KB.

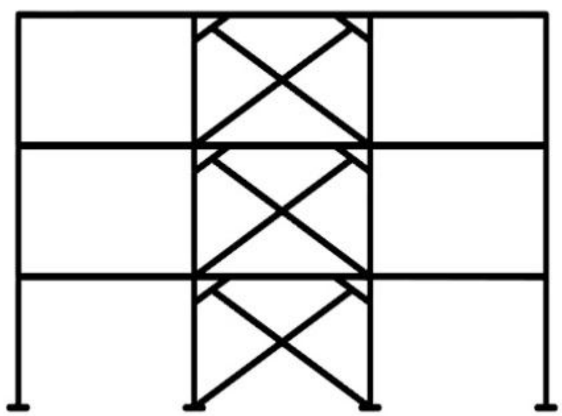

Fig. 4. Geometry of the 3-story frame.

LOADING AND DESIGN

The gravity loads include dead and live load of 600kg/m^2 and 200kg/m^2 respectively. Eq. 1 calculates the equivalent static lateral seismic loads assuming that the response modification factor R for the knee-bracing system is 7.

$$V = CW \rightarrow C = \frac{ABI}{R} \tag{1}$$

where V represents the base shear, A is the design base acceleration ratio (for very high seismic zone = 0.35g), B is the response factor of building (depending on the fundamental period T), and I the importance factor of building (depending on its performance, taken equal to 1.0 in this paper), and A × B the design spectral acceleration (Figure 5) (Naeemi and Bozorg, 2009). All of the frames are designed according to the AISC89's allowable stress design. Table 1 summarizes the size of members in frames. As observed, the buildings contain H-shaped columns, I-shaped beams, knee elements as well as box braces. The columns, beams and braces were made of ST37 while the knee elements were made of ST37 and EGS.

The beam–column joints were assumed to be pinned at both ends and rigid connection between knee elements and beam-column to ensure the energy dissipation.

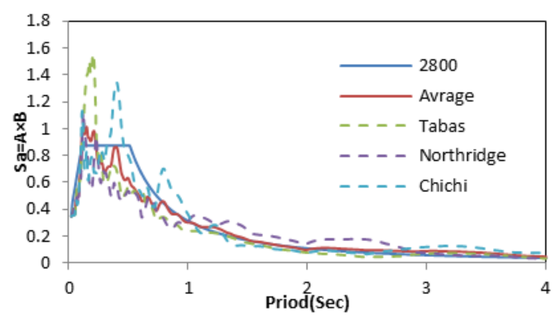

Fig. 5. Variation of spectral acceleration with period of structure.

Design of Knee Element with St37 and Easy Going Steel

As mentioned previously, KBFs are expected to withstand significant inelastic deformations while subjecting to forces resulting from earthquake. Link plastic rotation angle (γ_p) can easily be estimated by frame geometry assuming the rigid-plastic behavior of the frame members. Depending upon the section properties, the knee elements may yield either in shear extending over full length or in flexure at the ends or a combination of both. However, it is worth mentioning that the plastic rotation angle of the knee elements is the same whether the link yields in shear or in flexure. To put it differently, yielding mechanism of the knee elements depends on material properties of links such as moment capacity, shear capacity and strain hardening. Equations to determine the length ranges and permissible knee elements' inelastic rotation angles were assumed identical.

Eccentrically braced frames (EBFs) that were developed for the compacted sections are specified in seismic provisions of AISC (American Institute of Steel Construction, 2005). Short (shear yielding) links:

$$1.6\frac{M_p}{V_p}\gamma_p \le e \le 2.6\frac{M_p}{V_p}\gamma_p \ = 0.08 \text{ radians} \qquad (2)$$

Long (flexural yielding) links:

$$e \ge 2.6\frac{M_p}{V_p}\gamma_p \ = 0.02 \text{ radians} \qquad (3)$$

Intermediate length (combination of shear and flexural yielding) links:

$$1.6\frac{M_p}{V_p}\gamma_p \le e \le 2.6\frac{M_p}{V_p}\gamma_p \ = \text{interpolation between 0.08 and 0.02 radians} \qquad (4)$$

where e is the link length, $M_p=Z.F_y$ is the nominal plastic flexural strength, Z is the plastic section modulus and, F_y is the specified minimum yield stress. $V_p=0.6F_y(d_b-2t_f)t_w$ is the nominal shear strength, d_b is the overall beam depth and t_f and t_w are the thicknesses of the flange and web, respectively.

Given that the inelastic action occurs primarily within the knee elements, other elements such as beam segments, diagonal braces, and columns should be designed following the capacity design approach. These elements should remain essentially elastic under the maximum forces generated by the fully yielded and strain-hardened knee elements. A soft story is formed if plastic hinges in columns are combined with the yielded knee element; consequently plastic hinges should be avoided (Bahrampoor and Sabouri-Ghomi, 2010). To ensure that yielding and energy dissipation in the KBF occur primarily in the knee elements, capacity design approach is adopted for designing the diagonal brace in order to resist the forces generated by the fully yielded and strain hardened knee elements. The KBFs columns are further designed utilizing the capacity design. Columns were designed to resist the maximum forces developed by fully yielded and strain hardened knee elements (Bahrampoor and Sabouri-Ghomi, 2010).

Table 1 presents the properties of knee elements designed through AISC89. As discussed earlier, all sections consist of ST37. In the next step, all of the previous knee elements are designed through utilizing EGS. In order to have equal shear capacity in knee elements made of EGS, the sections should be increased. In this paper, the nominal plastic flexural strength (M_p) and nominal shear strength (V_p) of knee element with carbon steel and knee element with EGS are equal and hence new size is obtained for t_f and t_w as demonstrated in Table 2.

Table. 1. The member size for specimens.

Number of Story	Beam		Mid Column		Side Column		Knee Elements		Diagonal Elements	
	Similar Story	Dimensions	Similar Story	Dimensions	Similar Story	Dimensions	Similar Story	Dimensions	Similar Story	Dimensions
3	1,2,3	IPE270	1	IPB180	1,2,3	IPB100	1,2	IPE160	1	Box100x100x14.2
	–	–	2,3	IPB160	–	–	3	IPE140	2	Box90x90x12.5
	–	–	–	–	–	–	–	–	3	Box80x80x10
5	1,2,3,4,5	IPE270	1	IPB240	1,2	IPB120	1	IPE200	1	Box120x120x12.5
	–	–	2	IPB220	3,4,5	IPB100	2,3	IPE180	2,3,4	Box100x100x14.2
	–	–	3	IPB200	–	–	4	IPE160	5	Box80x80x10
	–	–	4,5	IPB140	–	–	5	IPE140	–	–
7	All stories	IPE270	1	IPB320	1,2,3,4	IPB140	1	IPE220	1,2,3,4	Box120x120x12.5
	–	–	2,3	IPB260	5,6,7	IPB100	2,3,4	IPE200	5	Box100x100x14.2
	–	–	4,5,6,7	IPB200	–	–	5,6	IPE180	6	Box90x90x12.5
	–	–	–	–	–	–	7	IPE160	7	Box80x80x10
10	All stories	IPE270	1	IPB500	1,2	IPB160	1,2,3,4,5	IPE220	1	Box120x120x17.5
	–	–	2	IPB450	3,4,5,6	IPB140	6,7,8,9	IPE200	2	Box120x120x14.2
	–	–	3	IPB360	7,8,9,10	IPB120	10	IPE160	3,4,5,6	Box120x120x12.5
	–	–	4,5	IPB300	–	–	–	–	7,8	Box100x100x14.2
	–	–	6,7,8	IPB240	–	–	–	–	9	Box90x90x12.5
	–	–	9,10	IPB200	–	–	–	–	10	Box80x80x10
12	All stories	IPE270	1	IPB650	1	IPB180	1,2,3,4,5	IPE220	1	Box120x120x17.5
	–	–	2	IPB550	2,3,4	IPB160	6,7,8,9	IPE200	2,3	Box120x120x14.2
	–	–	3,4,5,6	IPB450	5,6,7	IPB140	10,11	IPE180	4,5,6,7,8	Box120x120x12.5
	–	–	7,8	IPB280	8,9,10,11,12	IPB120	12	IPE160	9,10	Box100x100x14.2
	–	–	9,10	IPB220	–	–	–	–	11	Box90x90x12.5
	–	–	11,12	IPB200	–	–	–	–	12	Box80x80x10

In this article two types of frames have been exploited: knee braced system made of ordinary steel (ST37) called "KO" and knee braced system with knee elements made of EGS called "KE". To state it differently, "KO3" refers to a 3-story knee braced frame with knee elements made of EGS.

Table. 2. New size for thickness of the flange and web of the knee elements made of EGS.

Section	New tw (cm)	New tf (cm)
IPE10	1.473	1.898
IPE12	1.491	2.013
IPE14	1.537	2.145
IPE16	1.603	2.304
IPE18	1.668	2.446
IPE20	1.75	2.64
IPE22	1.83	2.81
IPE24	1.92	3.04

DEFINING THE RESPONSE MODIFICATION FACTORS

Elastic analysis of structures under earthquake could create base shear force and stress which are noticeably larger than the real structure response. The structure is capable of absorbing a lot of earthquake energy and resisting when it enters the inelastic range of deformation (Naeemi and Bozorg, 2009). In force-based seismic design procedures, the response modification factor (UBC code and NEHRP provisions, Federal Emergency Management Agency, 1997) is utilized to reduce the linear elastic response spectra from the inelastic response spectra. In other words, the response modification factor is the ratio of the strength required to maintain elastic to inelastic design strength of the structure. The response modification factor R therefore accounts for the inherent ductility and overstrength of a structure as well as the difference in the level of stresses considered in its design (Miri et al., 2009). As shown in Figure 6 the real nonlinear behavior is usually idealized by a bilinear elasto-perfectly plastic relation (Miri et al., 2009; Naeemi and Bozorg, 2009).

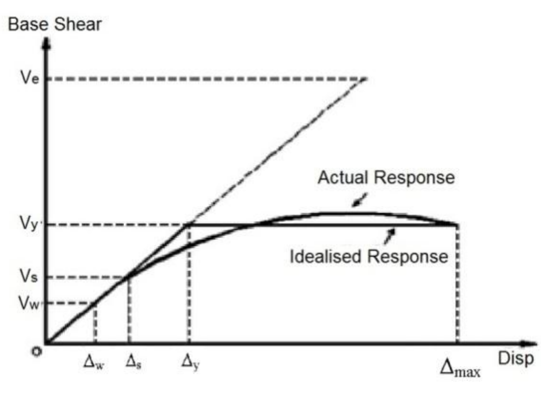

Fig. 6. General structural response (Uang CM, 1991).

Yield force and yield displacement of the structure are represented by V_y and Δ_y, respectively. In this figure, $V_e(V_{max})$ corresponds to the elastic response strength of the structure. The maximum base shear in an elasto-perfectly behavior is V_y (Uang, 1991). It is generally expressed in the following form taking into account the above three components (Miri et al., 2009; Naeemi and Bozorg, 2009):

$$R = R_{\mu}.R_s.Y \tag{5}$$

where R_{μ} represents ductility-dependent component also known as ductility reduction factor, R_s the overstrength factor, and Y the allowable stress factor (Miri et al., 2009; Naeemi and Bozorg, 2009).

The ratio of maximum base shear considering elastic behavior V_e to maximum base shear in elasto perfectly behavior V_y is called ductility reduction factor (Miri et al., 2009; Naeemi and Bozorg, 2009).

$$R_{\mu} = \frac{V_e}{V_y} \tag{6}$$

The overstrength factor is defined as the ratio of maximum base shear in actual behavior V_y to first significant yield strength in structure V_d (Miri et al., 2009; Naeemi and Bozorg, 2009).

$$R_s = \frac{V_y}{V_s} \tag{7}$$

The overstrength factor demonstrated in Eq. (7) is based on the use of nominal material and other factors. Representing this overstrength factor by R_{so}, the actual overstrength factor R_s which can be utilized to formulate R should take into account the beneficial contribution of some other effects (Uang CM, 1991):

$$R_s = R_{S0} F_1 F_2 \tag{8}$$

In this equation, F_1 accounts for the difference between the actual static yield strength and the nominal static yield strength. For structural steel, a statistical study reveals that the value of F_1 may be taken as 1.05 (Schmidt and Bartlet, 2002). Parameter F_2 might be applied to take into consideration the augmentation in the yield stress as a result of strain rate effect during an earthquake. A value of 1.1, a 10% increase to account for the strain rate effect, could be used (Uang, 1991). Parameters F_1 and F_2 are taken equal to 1.05 and 1.1 while material overstrength factor equal to 1.155 (Naeemi and Bozorg, 2009). To design the permissible stress method, the design codes reduce design loads from V_s to V_w.

$$Y = \frac{V_s}{V_w} \tag{9}$$

This paper utilizes the design base shear V_w, instead of V_s, hence the allowable stress factor Y becomes unity and the overstrength factor is defined as:

$$R_s = \frac{V_y}{V_w} \tag{10}$$

MODELING THE STRUCTURES IN OPENSEES SOFTWARE

The computational model of the structures was developed applying modeling capabilities of the OpenSees software framework (Mazzoni et al., 2004). This software has been specifically designed to measure performance of soil and structure under earthquake. In order to model the members in nonlinear range of deformation, the following assumptions were made (Asgarian and Shokrgozar, 2009).

The frame members, i.e., beams, columns and braces assume pin-ended shapes yet connections between the knee elements and beam-column are assumed rigid. Consequently, earthquake lateral forces are carried only by vertical braces; while gravity loads are sustained mainly by columns. In order to carry out dynamic analysis, story masses were placed in the story levels considering rigid diaphragms action. To model braces, nonlinear beam and column elements with the material behavior of Steel01 were exploited. Figure 7 demonstrates the idealized elasto-plastic behavior of the steel material.

Compressive and tensional yield stresses were taken equal to steel yield stress based on experimental tests (Susantha et al., 2005) as shown in Table 3.

The section utilized for each member is the uniaxial section. A strain hardening of 3% was assumed for the member behavior in inelastic range of deformation (Figure 7). For linear and non-linear dynamic analysis a damping coefficient of 5% was assumed. Regarding the nonlinear buckling prediction, a uniaxial section and nonlinear Beam Column element were employed for plastification of the element over the cross section and member length.

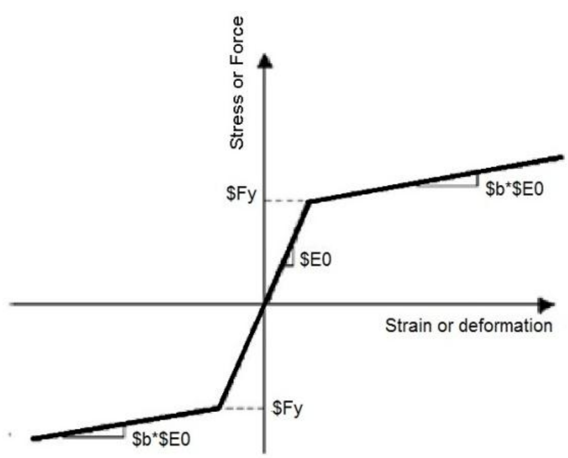

Fig. 7. Steel 01 material for nonlinear elements (Mazzoni et al., 2004).

Table. 3. Properties of steel type (Susantha et al., 2005).

Steel Type	Fy (MPa)	Fu (MPa)	Strain at Failure (%)	Initial Elastic Modulus (GPa)
EGS	95.4	274.0	39.6	200
ST37	240	370	Over17	206

To predict linear buckling an initial mid span, imperfection of 1/1000 for all braces was assumed. In order to account for geometric nonlinearities, the simplified P-Δ stiffness matrix was used (Asgarian and Shokrgozar, 2009).

To verify the results, some numerical analyses were carried out by another software (SAP 2000 software) and subsequently the results obtained from the two modeling were compared. Roof displacements of the frames were utilized to compare the results. The results give weight to the accuracy of the modeling. It implies that the roof displacements obtained are approximately the same in both modeling. For example, Figure 8 shows the time history of chichi ground motion for the top floor displacement of KO and KE 3-story frames.

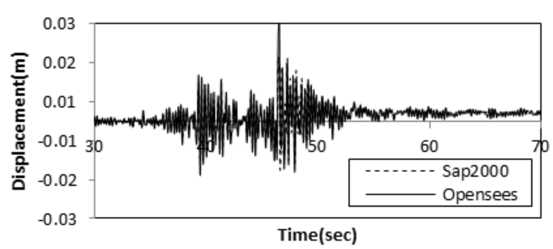

Fig. 8. Time history of top floor displacement of KO and KE frames (three story frames subjected to chichi ground motion).

DETERMINING THE RESPONSE MODIFICATION FACTOR

In the present paper, the two factors R_s and R_μ have been calculated as follows:

Overstrength Factor (R_s)

To calculate V_y, the incremental nonlinear dynamic analyses of models subjected to strong ground motions were carried out. In these analyses time history of Tabas, Northridge and Chichi earthquakes (Table 4) were utilized. Figure 5 demonstrates the response spectra as well as the design spectrum. Subsequent to several trials, their PGA altered such that the gained time history resulted in one of the following failure criteria. The maximum nonlinear base shear of this time history represents the inelastic base shear of the structure (Mwafy and Elnashai, 2002). Finally, the material overstrength factor was taken equal to 1.155 for the actual over-strength factor (Asgarian and Shokrgozar, 2009). The failure criteria are defined by the following two levels:

i) The relative floor displacement: The maximum limitation of the relative story displacement was selected utilizing the Iranian Standard Code No. 2800 (BHRC, 2005).

a) For frames with the fundamental period less than 0.7 sec:

$$\overline{\Delta_M} < 0.025H \tag{11}$$

b) For frames with the fundamental period more than 0.7 sec:

$$\overline{\Delta_M} < 0.02H. \tag{12}$$

in which ‘H’ is the story height.

ii) Reaching the life safety structural performance: Generally, the component behavior induced by nonlinear load-deformation relations is defined through a series of straight line segments suggested by FEMA-273 (Naeemi and Bozorg, 2009; FEMA, 1997). As Figure 9 demonstrates the nonlinear dynamic analysis was stopped and the last scaled earthquake base shear was selected as the one reaching to the level of life safety structural performance; the figure further indicates the nonlinear behavior of elements as suggested by FEMA-356 (Naeemi and Bozorg, 2009; FEMA, 2000).

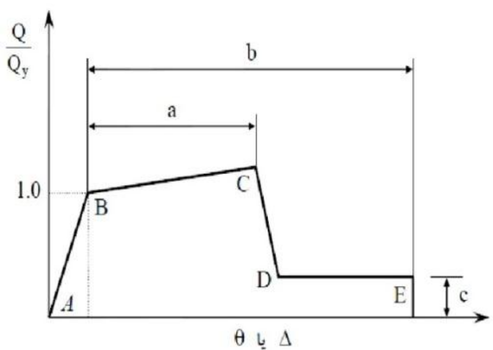

Fig. 9. Generalized force-deformation relation for steel elements (Federal Emergency Management Agency, 1997).

Table. 4. Ground motion data.

Record	Year	Magnitude	PGA
Chi-Chi	1999	M (7.6)	0.378
Northridge	1994	M (6.7)	0.37
Tabas	1978	M (7.4)	0.328

R_μ Calculation

To calculate R_μ, linear and nonlinear dynamic analyses were carried out. The nonlinear base shear V_y was calculated utilizing incremental nonlinear dynamic analysis as well as trials on PGA of earthquake time histories as aforementioned. Subsequently, the maximum linear base shear V_e was computed through linear dynamic analysis of the structure under the same time history; and ultimately the ductility reduction factor was evaluated (Asgarian and Shokrgozar, 2009; Uang, 1991; Mwafy and Elnashai, 2002).

RESULTS

Figure 10 compares nonlinear dynamic analyses based on Standard No. 2800 (BHRC, 2005) employing scaled ground acceleration (PGA) of 0.35g for 3, 5, 7, 10 and 12 story. Results reveal that the drift decreases by 10% when EGS is exploited instead of ST37 in the knee elements.

Table 5 demonstrates ultimate base shear V_y and maximum acceleration obtained from nonlinear dynamic analysis under Tabas, Northridge and Chichi events for KO and KE frames. Table 6 presents maximum elastic base shear V_e, resulted from linear dynamic analysis for the aforesaid time histories. Tables 7 and 8 indicate the overstrength factor, the ductility factor as well as the response modification factor for KO and KE specimens. As can be observed, overstrength, ductility and response modification factors increase as the height of the building reduces. The response modification factor for different specimen was calculated statistically as:

1. KO bracing system R=11.46, $R\mu$ =1.85
2. KE bracing system R=11.64, $R\mu$ =1.97

Figures 11-13 highlights the comparison of overstrength, ductility and response modification factors for difference types of bracing. As observed, ductility factor decreases at a greater speed vis-à-vis the overstrength factor by increasing the number of stories. Ductility and overstrength factors gradually become stable in the high stories. For all types of bracings, the response modification factor reduces as the height of the building increases (Figures 10 to 12.).

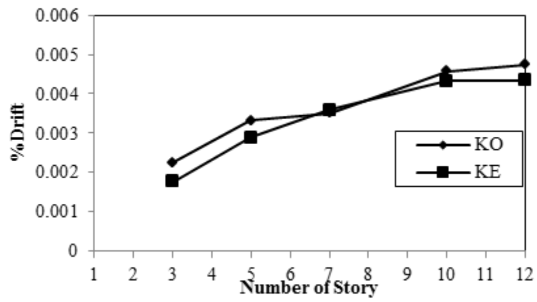

Fig. 10. Comparison of average of maximum drift of KO with KE frames under incremental nonlinear dynamic analysis with scaled ground acceleration (PGA) to 0.35g.

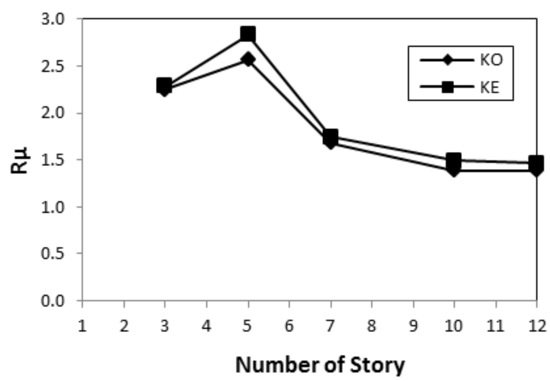

Fig. 11. Number of story- ductility factor.

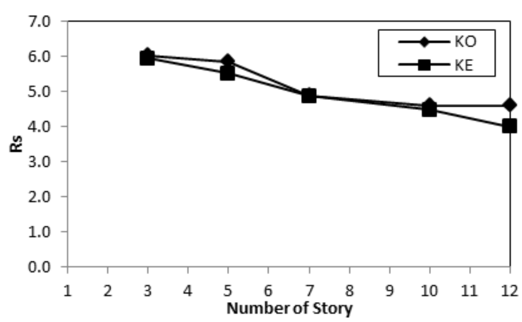

Fig. 12. Number of story-overstrength factor.

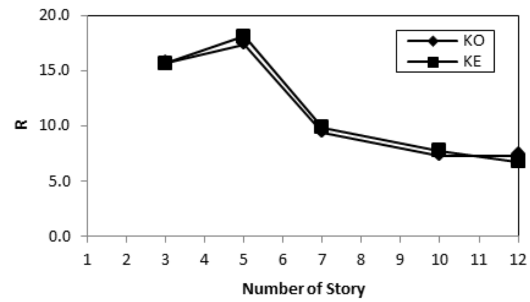

Fig. 13. Number of story- response modification factor.

CONCLUSIONS

During the course of the present study, the EGS instead of constructional steel was utilized in the knee elements of braced frames.

To this purpose, five frames with different height were assumed. Ductility, overstrength and response modification factors of the models were assesses through incremental nonlinear and linear dynamic analyses. The result of the study is best summarized as follows:

1. Regardless of the increase in ductility factor for KE vis-à-vis KO, the response modification did not enhance significantly owing to the reduction in overstrength factor.
2. The overstrength factor's mean obtained for KO and KE specimen is 6 and 5.7, respectively.
3. The ductility factor's mean obtained for KO and KE specimen is 1.85 and 1.96, respectively.
4. The response modification factor's mean for KO and KE specimens is suggested 11.4 and 11.6, respectively.
5. The stiffness enhanced by 10 percent once the EGS was utilized instead of constructional steel in the knee elements.
6. The mean of ductility factor enhanced by 6% once the EGS was utilized instead of constructional steel in the knee elements.

Table. 5. Nonlinear maximum base shear and PGA for KO and KE under Tabas, Northridge and Chichi ground motion.

No. of Story	KO							KE						
	Tabas		Northridge		chichi			Tabas		Northridge		chichi		
	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	$V_{e(avg)}$ (KN)	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	$V_{e(avg)}$ (KN)
3	1	599.1	0.95	599.2	0.65	606.5	601.6	0.78	594.3	0.85	588.9	0.58	594.5	593
5	1.9	1025.8	1.15	953.9	0.78	976.6	985.5	1	860.1	0.75	925.3	0.50	999.8	928
7	1.025	1167.0	0.90	1242.5	1	1091.4	1167.0	1	1259.9	0.68	1156.9	0.7	1075.3	1164
10	1.075	1404.6	0.93	1403.1	0.9	1255.5	1354.4	0.88	1300.2	0.73	1300.2	0.72	1328.5	1310
12	1.05	1342.5	0.80	1426.6	1.2	1442.6	1403.9	0.88	1308.8	0.48	1330.9	0.93	1372.5	1337.4

Table. 6. Linear maximum base shear and PGA for KO and KE under Tabas, Northridge and Chichi ground motion.

No. of Story	KO							KE						
	Tabas		Northridge		chichi			Tabas		Northridge		chichi		
	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	$V_{e(avg)}$ (KN)	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	PGA (g)	V_e (KN)	$V_{e(avg)}$ (KN)
3	1	1566.0	0.95	1391.9	0.65	1104.3	1354.1	0.78	1254.9	0.85	1793.0	0.58	1009.3	1352
5	1.9	2593.9	1.15	2316.3	0.78	2681.3	2530.5	1	2416.5	0.75	2689.9	0.5	2789.2	2632
7	1.02	1858.1	0.90	1948.1	1	2045.2	1950.5	1	2219.7	0.68	1878.6	0.72	1995.1	2031
10	1.08	1822.9	0.93	1844.4	0.9	1930.0	1865.8	0.88	1805.1	0.73	1936.4	0.72	2165.5	1969
12	1.05	1893.1	0.80	2307.8	1.2	2244.5	2148.5	0.88	1926.7	0.48	1721.5	0.93	2069.1	1906

Table. 7. Average of overstrength, ductility and response modification factors of KO.

Number of Story	R_{SO}	R_S	R_μ	R
3	6.1	7.0	2.3	15.7
5	5.9	6.8	2.6	17.4
7	4.8	5.6	1.7	9.5
10	4.6	5.3	1.4	7.4
12	4.6	5.3	1.4	7.4
Average	5.2	5.0	1.9	11.4

Table. 8. Average of overstrength, ductility and response modification factors of KE.

Number of Story	R_{SO}	R_S	R_μ	R
3	5.9	6.9	2.3	15.7
5	5.5	6.4	2.8	18.1
7	4.8	5.6	1.7	9.8
10	4.5	5.2	1.5	7.7
12	4.0	4.6	1.5	6.8
Average	5.0	5.7	2.0	11.6

REFERENCES

American Institute of Steel Construction (AISC), (2005) "Seismic provisions for structural steel buildings", Chicago.

Aristizabal-Ochoa, J.D. (1986). "Disposable knee bracing: improvement in seismic design of steel frames", *J. Structural Engineering*, ASCE, 112(7), 1544-1552.

Asgarian, B. and Shokrgozar, H.R. (2009). "BRBF response modification factor", *Constructional Steel Research,* 65, 290-298.

ATC. (1995). "A critical review of current approaches to earthquake-resistant design", ATC-34, Applied Technology Council, Redwood City, CA, 6–31.

ATC. (1995). "Structural response modification factors", ATC-19. Applied Technology Council, Redwood City, CA. 5–32.

Bahrampoor, H. and Sabouri-Ghomi, S. (2010). "Effect of easy-going steel concept on the behavior of diagonal eccentrically braced Frames", *International Journal of Civil Engineering*, 8(3), 242-255.

Balendra, T., Lim, E.L. and Lee, S.L. (1994). "Ductile knee-braced frames with shear yielding knee for seismic resistant structures", *Engineering Structures*, 16(4), 263–9.

Balendra, T., Sam, M.T. and Liaw, C.Y. (1990). "Diagonal brace with ductile knee anchor for a seismic steel frame", *Earthquake Engineering and Structural Dynamics*, 19(6), 847-858.

BHRC (2005). "Iranian code of practice for seismic resistance design of buildings", Standard No. 2800 (3rd ed.), Building and Housing Research Center.

Cengizhan Durucan, Murat Dicleli. (2010). "Analytical study on seismic retrofitting of reinforced concrete buildings using steel braces with shear link", *Engineering Structures,* 32, 2995–3010.

European Committee for Standardization. (1998). European Pre-standard ENV (Eurocode 8): Design provisions for earthquake resistance of structures, Bruxelles.

Federal Emergency Management Agency (FEMA). (1997). "NEHRP provisions for the seismic rehabilitation of buildings", Report FEMA 273 and 274, Washington DC.

Federal Emergency Management Agency (FEMA). (2000). "NEHRP provisions for the seismic rehabilitation of buildings", Report FEMA-356, Washington DC.

International Conference of Building Officials. (1997). "Uniform Building Code", Whittier, California.

Ker-Chun Lin, Chu-Chieh J. Lin, Jung-Yu Chen, Heui-Yung Chang. (2010). "Seismic reliability of steel framed buildings", *Structural Safety*, 32, 174–182.

Lotfollahi, M. and Mofid, M. (2006). "On the design of new ductile knee bracing", *Constructional Steel Research*, 62, 282–294.

Luigi Mastrandrea, Vincenzo Piluso. (2009). "Plastic design of eccentrically braced frames, I Moment shear interaction", *Constructional Steel Research*, 65, 1007-1014.

Maheri, M.R. and Akbari, R. (2003). "Seismic behaviour factor, R, for steel X-braced and knee-braced RC buildings", *Engineering Structures*, 25, 1505–1513.

Mazzolani, F.M. and Piluso, V. (1996). *Theory and design of seismic resistant steel frames*, Routledge, London.

Mazzoni, S. and McKenna, F. (2007). "OpenSees command language manual" version 2.1.0. Pacific earthquake engineering research center.

Miri, M., Zare, A.R. and Abbas Zadeh, H. (2009). "Seismic behavior of steel frames investigation with knee-brace based on pushover analysis", *World Academy of Science*, *Engineering and Technology,* 50, 932-938.

Mofid, M. and Lotfollahi, M. (2006). "On the characteristics of new ductile knee bracing systems", *Constructional Steel Research,* 62, 271–281.

Mwafy, A.M. and Elnashai, A.S. (2002), "Calibration of force reduction factors of RC buildings", *Earthquake Engineering*, 6(22), 239–73.

Naeemi, M. and Bozorg, M. (2009). "Seismic Performance of Knee Braced Frame", World Academy of Science, Engineering and Technology, 50, 976-980.

Roeder, C.W. and Popov, E.P. (1978). "Eccentrically braced steel frames for earthquakes", *J. Structural Division,* ASCE, 104, 391-411.

Sabouri-Ghomi, S. (2004). *Lateral load resisting systems and innovative idea to application of Easy- Going Steel*, EGS, Angizeh publishing Company.

Sabouri-Ghomi, S. and Ebadi, P. (2008). "Concept improvement of behavior of X-Bracing systems by using Easy-Going Steel", The 14th World Conference on Earthquake Engineering, October 12-17, Beijing, China.

Sabouri-Ghomi, S. and Ziaei, M. (2008). "A study on the behavior of shear link beam made of easy-going steel in eccentrically braced frames", The 14th World Conference on Earthquake Engineering, October 12-17, Beijing, China.

Schmidt, B.J. and Bartlett, F.M. (2002). "Review of resistance factor for: Resistance distributions and resistance factor calibration", *Canadian Journal of Civil Engineering*, 29, 109–18.

Sheng-Jin Chen, Chyuan Jhang. (2011). "Experimental study of low-yield-point steel plate shear wall under in-plane load", *Constructional Steel Research*, 67, 977–985.

Susantha, K.A.S., Aoki, T., Kumano, T. and Yamamoto, K. (2005). "Applicability of low-yield-strength steel for ductility improvement of steel bridge piers", *Engineering Structures*, 27, 1064–1073.

Uang, C.M. (1991). "Establishing R (or Rw) and Cd factor for building seismic provision", *Structure Engineering*, 117(1), 19–28.

Vulcano, A. and Mazza, F. (2000). "Comparative study of the seismic performance of frames using different dissipative braces", *12th World Conference on Earthquake Engineering*, January 30-February 4, Auckland, New Zealand.