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ORIGINAL RESEARCH

Investigation of the nonlinear seismic behavior of knee braced frames using the incremental dynamic analysis method

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Abstract In knee braced frames, the braces are attached to the knee element rather than the intersection of beams and columns. This bracing system is widely used and preferred over the other commonly used systems for reasons such as having lateral stiffness while having adequate ductility, damage concentration on the second degree convenience of repairing and replacing of these elements after Earthquake. The lateral stiffness of this system is supplied by the bracing member and the ductility of the frame attached to the knee length is supplied through the bending or shear yield of the knee member. In this paper, the nonlinear seismic behavior of knee braced frame systems has been investigated using incremental dynamic analysis (IDA) and the effects of the number of stories in a building, length and the moment of inertia of the knee member on the seismic behavior, elastic stiffness, ductility and the probability of failure of these systems has been determined. In the incremental dynamic analysis, after plotting the IDA diagrams of the accelerograms, the collapse diagrams in the limit states are determined. These diagrams yield that for a constant knee length with reduced moment of inertia, the probability of collapse in limit states heightens and also for a constant knee moment of inertia with increasing length, the probability of collapse in limit states increases.

Introduction

Structures designed to resist moderate and frequently occurring earthquakes must have sufficient stiffness and strength to control deflection and to prevent any possible damage. However, it is inappropriate to design a structure to remain in the elastic region, under severe earthquakes, because of the economic constraints. The inherent damping of yielding structural elements can advantageously be utilized to lower the strength requirement, leading to a more economical design. This yielding usually provides the ductility or toughness of the structure against the sudden brittle type structural failure. It is desirable to devise a structural system that combines stiffness and ductility, in the most effective manner without excessive increase in the cost. In building seismic design codes, two main goals are aimed for. First, during weak and average Earthquakes the structure should have sufficient strength and stiffness to prevent structural damages and to control the displacement. Second, during strong Earthquakes, the structures must have the ability to absorb energy properly and display suitable ductility. Stiffness and ductility are two elements that are frequently at odds, thus it is desirable for the structural system to have rational and sensible balance between the two (Balendra and Sam 1990, 1991a).

At present, in designing seismic resistant steel buildings, systems such as moment frames, concentrically braced frames and eccentrically braced frames are widely used. Moment resisting frames show a good ductility due to the bending yield of the beam but its stiffness in low. Concentrically braced frames have a high stiffness but due to

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the buckling of the compressional brace, it displays a low ductility. To overcome the problem of stiffness in moment resisting frames and also the problem of low ductility in concentrically braced frames, Hjelmstad and Popov (1983) proposed the eccentric brace. By considering the appropriate amount of eccentricity, the system contains sufficient stiffness and through the shear and bending yield of the intermediary beam, the ductility will be provided. This system has fitting ductility and stiffness but to supply its ductility, the intermediary beam, which is one of the main elements of the structure, must yield and that results in severe damage to the ceiling which makes the post-earthquake repairing difficult (Balendra and Sam 1991b, 1994).

Ochao (1986) proposed a new system which was later modified by Balendera (1990, 1991a, 1994). In this system which is called the (KBF), the tag end of the diagonal braces are attached to a skewed (knee) element which itself is attached to the beam and the column or the beam and the support, rather than being annexed to the intersection of the beam and the column. In this system the knee element remains in an elastic state during small earthquake and they yield before the main members of the structure in strong earthquakes, and this causes the energy to be dissipated without the lateral strength being attenuated. In this system, the damages brought about by the earthquake will be concentrated on the knee members which are not key structural members and can be replaced and repaired after the earthquake (William and Denis 2004).

In recent years, different studies have been conducted on determining dimensions, shape, characteristics and other optimal parameters of these systems in order to achieve the most desirable stiffness—ductility combinations. The later experimental work on directly knee braced model frames by Zahrai and Jalali (2014) and Sutat et al. (2011) also showed the applicability of this method. Knee braces were
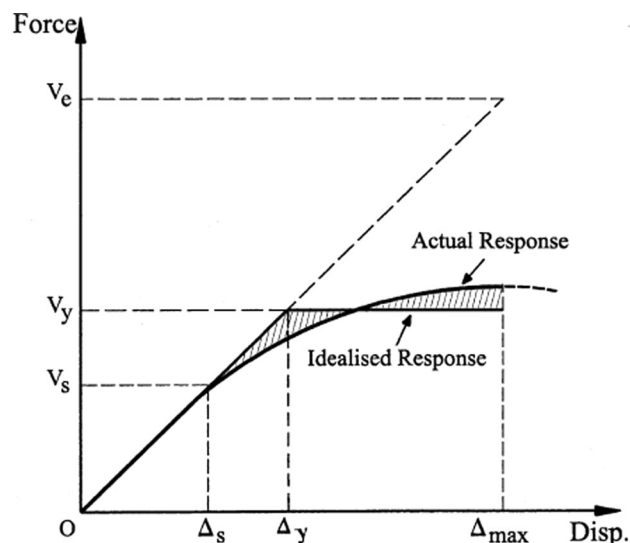

Fig. 1 Parameters used in calculation of ductility, overstrength and performance factor

used in the past for wind-resistant design and have been recently explored in various forms for seismic applications (Inouel et al. 2006; Lee and Bruneau 2005). The design of the proposed KBMF structural system is based on a capacity-design concept that results in ductile behavior. For this system, the frames are designed so that the knee braces will yield and buckle under seismic loads; this is followed by plastic hinging of beams at the ends of the beam segments outside the knee portions. All inelastic activities are directed away from the critical areas, decreasing the dependence of the performance on the material and quality of the welded joints. In this study, also, in the interest of completing the former studies, the principles of incremental dynamic analysis have been employed.
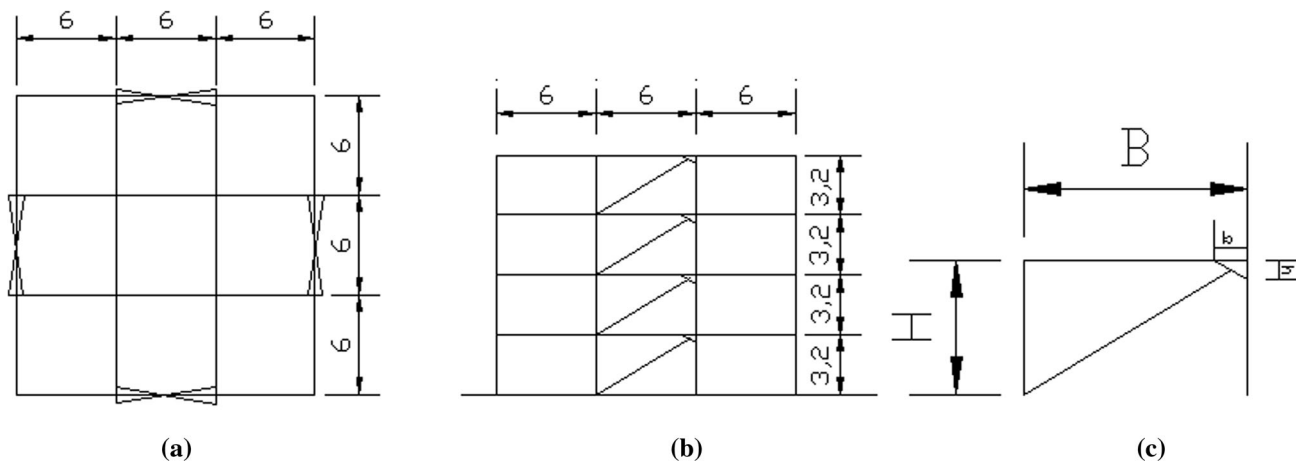

Fig. 2 Geometrical specifications of the building. **a** Plan of building. **b** View of building. **c** Knee braced
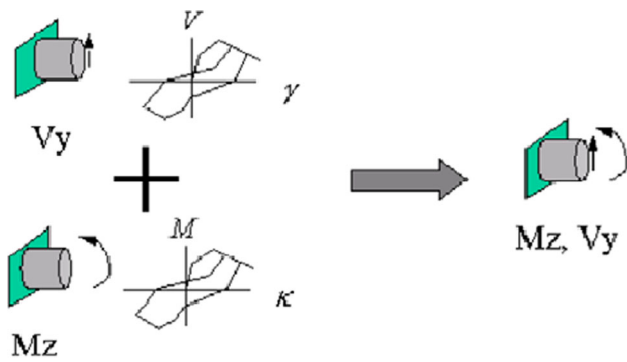
Fig. 3 Section used for knee element in OpenSees software (McKenna et al. 2007)

Estimating the behavior factor

Most of the design codes decrease the base shear by dividing the elastic earthquake factor over the larger number of a unit called "the behavior factor". The below equation can be used as a low approximation of the behavior factor (R) (Chopra 1998).

$$R = R_\mu \times R_S \times Y \tag{1}$$

where R_μ is the ductility factor of the system and presents the energy dissipation capacity. R_S is the excess strength factor of the system and Y is the allowable stress factor. These parameters will be determined through Eqs. 2, 3 and 4 (Fig. 1) using the capacity spectra diagram, where V_d is the designing shear, V_s is the shear that corresponds to the development of the first plastic joint, V_{ym} is the base shear established upon the maximum inelastic displacement and V_e is the maximum seismic need for the elastic response, F_1, F_2 and F_3 are respectively the ratio of

actual stress over the nominal stress, the effect of the speed of loading in increasing the yield stress and the effect of nonstructural (Chopra 1998; Jinkoo 2005).

$$R_S = F_1 \times F_2 \times F_3 \times R_{SO} \tag{2}$$

$$R_\mu = V_e / V_{ym} \tag{3}$$

$$R_{so} = V_{ym} / V_s \tag{4}$$

In this study the behavior factor is obtain through the multiplication of R_μ and R_S, and R_{so} is equal to V_{ym}/V_d.

The subjected models

To assess the stiffness, the factor of ductility, the factor of excess strength, the behavior factor and the probability of failure of knee braced systems, a building like the one depicted in Fig. 2 with the below characteristics has been studied.

The steel building has three bays along their two main directions and the number of the stories is 4 and 8. The length of the bays and the height of the stories are 6 and 3.2 meters, respectively. The building is braced in both of its lateral sides to resist earthquake. The weight of each story is 274 ton and it's been assumed that the building are constructed in Tehran with very high earthquake risk, soil type 3 (BHRC: Standard No. 2800, 2005) whose shear velocity is approximately between 175 and 375 m/s and residential application. The connection between the beam and the column has been considered as a joint. These buildings have designed in accordance with the AISC-ASD design code (2005). The specifications of the knee brace is presented in Fig. 2a and is described as follow. The knee element is placed on one side and at the top of the brace. The type of connection between the knee and the beam and

Table 1 Specifications of the accelerograms use in the incremental dynamic analyzes

No.	Earthquake	Station	Moment Magnitude	Distance (km)
1	Friuli-Italy 1976	Codroipo	6.5	33.4
2	Cape Mendocino	Eureka	7.01	41.97
3	Cape Mendocino	Eureka	7.01	41.97
4	Imperial Valley	El Centro	6.5	18.20
5	Imperial Valley	Calipatria	6.5	23.17
6	Loma Prieta	Gilory	6.9	16.1
7	Loma prieta	Palo Alto	6.9	36.3
8	Northridge	La Saturn	6.7	30
9	Chi Chi	CWB Chy002	7.62	24.21
10	Chi Chi	CWB Chy004	7.62	24.21
11	Duzce	Yarimca	7.14	97.53
12	Kocaeli	Ambarli	7.51	69.62
13	Kocaeli	Ambarli	7.51	69.62
14	Kobe	Kakogava	6.9	22.5
15	Kobe	Kakogava	6.9	22.5

(a) 4H0.15I0.20

(b) 4H0.15I0.10
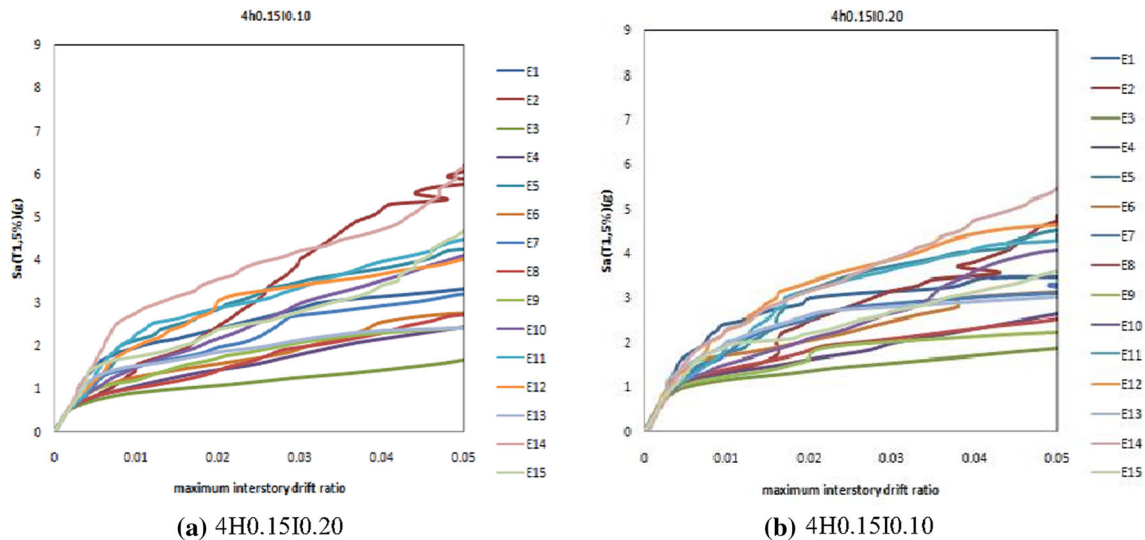
Fig. 4 The results of the incremental dynamic analysis of 4 story frames

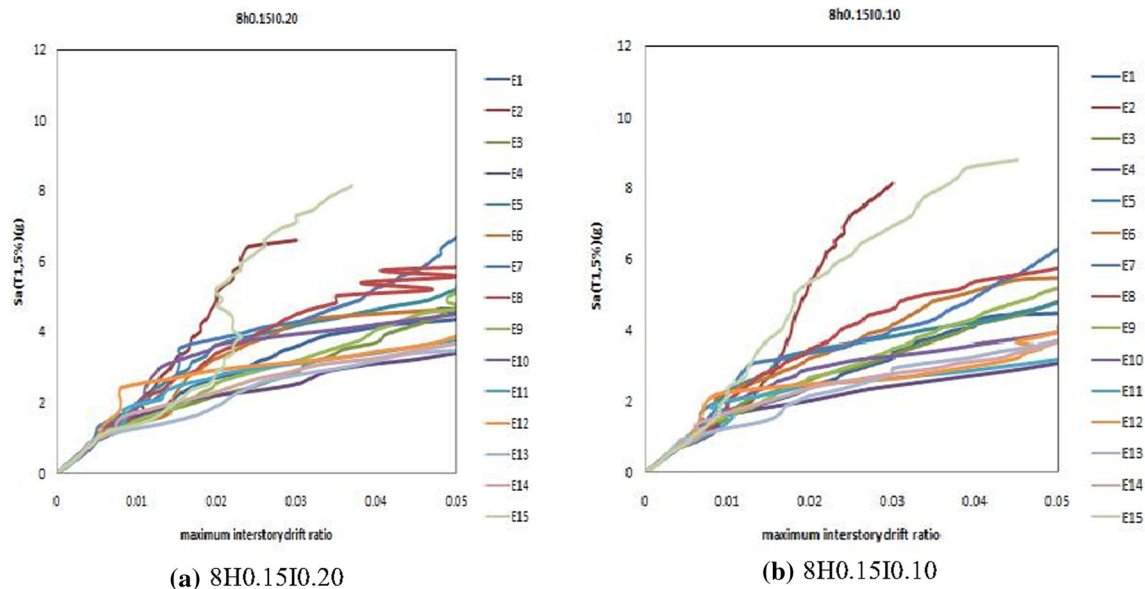**(a)** 8H0.15I0.20

(b) 8H0.15I0.10

Fig. 5 The results of the incremental dynamic analysis of 8 story frames

column and the connection between the brace and the knee are considered rigid and joint, respectively. The knee element is parallel to the other hypotenuse of the frames in such a way that the ratios b/h and B/H are equal and the along of the bracing member goes through the point at which the beam and the column are connected (Mofid and Khosravi 2000; Mofid and Lotfollahi 2006). Parametrical studies conducted on knee brace frames for different lengths and moment of inertia of the knee member are shown below..

$$h/H = 0.0, 0.05, 0.10, 0.15, 0.20, 0.25, 0.30$$

$$I_K/I_C = 0.10, 0.15, 0.20, 0.25, 0.30$$

where h, H, I_C and I_k are the vertical length of the knee element, the height of the story, the moment of inertia of the column and the moment of inertia of the knee respectively.

The frames are generally named using the format "xBHaIb" in which x, a and b are the number of stories, the ratio of h/H and the ratio of I_C/I_K, respectively. For example 8BH0.15I0.25 would means the frame with 8 story knee brace with the ratios of $h/H = 0.15$ and $I_k/I_c = 0.25$, the diagonal brace, also, is shown using "gh".
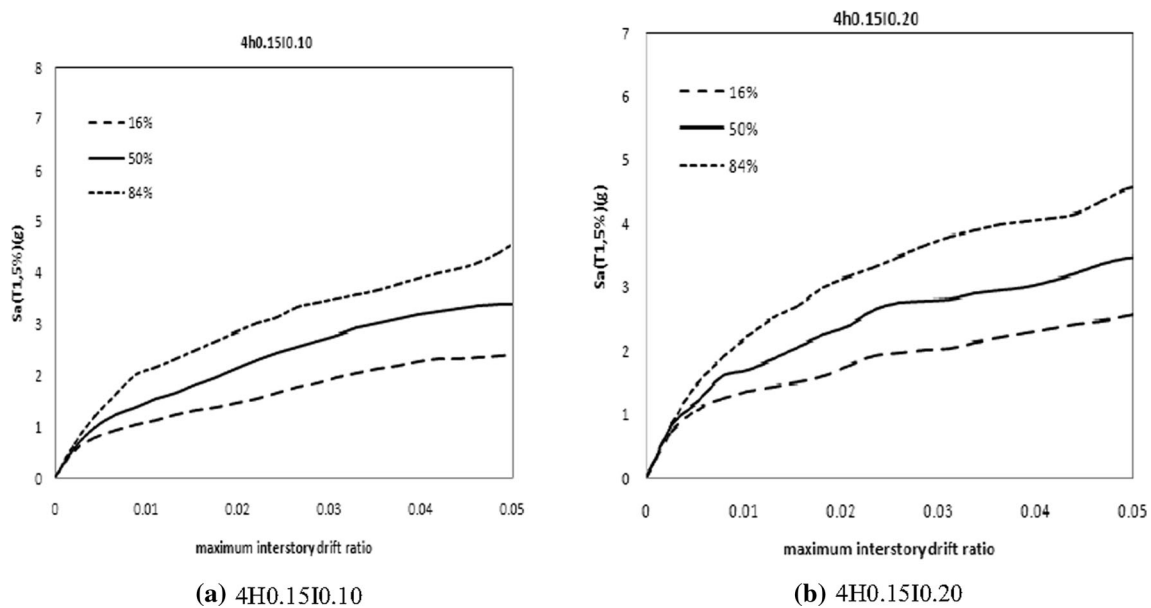
(a) 4H0.15I0.10

(b) 4H0.15I0.20

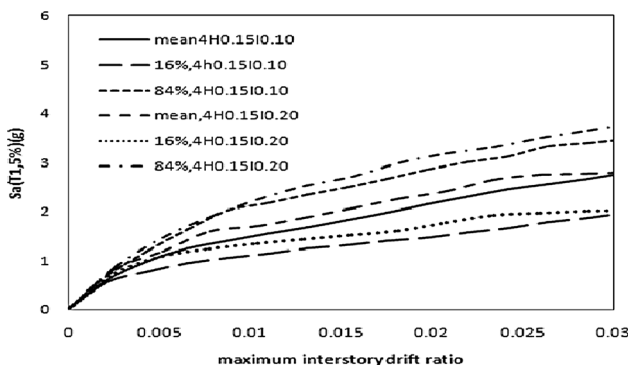**Fig. 6** A summary of the IDA results of 4 story buildings

Fig. 7 Comparing the summary of the IDA results of 4 story buildings

Incremental dynamic analysis

The incremental dynamic analysis has been carried out using the software opensees. For beams, columns and the bracing members, the fiber section for the nonlinear beam column element has been used and for the knee members, shear yield specifications to be exertable the section aggregator nonlinear beam column element has been used (Fig. 3). For the material specifications, the steel 02 material with the yield stress of 2400 kg/cm^2 has been used and the modulus of elasticity and the post yield stiffness are considered equal 2.1e6 kg/cm^2 and 2 %, respectively. The allowable drift according to the Iranian standard design code (2005) and the UBC for the frames of the both building are considered equal to 0.025. one of the following has been considered as the collapse mode: the drift exceeding the allowable limit or the buckling of the brace. The loading was continued until one of these two collapse modes were observed in the shear–displacement diagram.

The incremental dynamic analysis (IDA) has been employed to analyze the frames. The intensity measure (IM) has been taken into account in the spectral acceleration analysis of the first vibration mode of the structure [s_a (T_1)], and the maximum inter story drift ratio (MIDR) has been considered as the damage measure (DM). For these analyses, 15 accelerograms whose specifications are demonstrated in Table 1 have been used.

Result of dynamic analysis of 4 and 8 story frames with constant knee length and variable moment of inertia

The result of dynamic analysis of 4 and 8 story frames with constant knee length and variable moment of inertia are presented in Figs. 4 and 5. From linear responses to strictly nonlinear responses of the structure are observable in these figures. The structures response to some of the accelerograms is intensive and not so severe to some others, i.e., for a very small increase in the spectral acceleration of the accelerogram, the maximum intensity measure of the drift expands rapidly (softening), meanwhile, for some other accelerogram by drastically increasing the intensity measure, the stiffness measure wont experience any significant change (hardening), and all these responses in the incremental dynamic analysis are not unexpected. In fact in these diagrams, we can observe all of the probable responses during the future earthquakes.
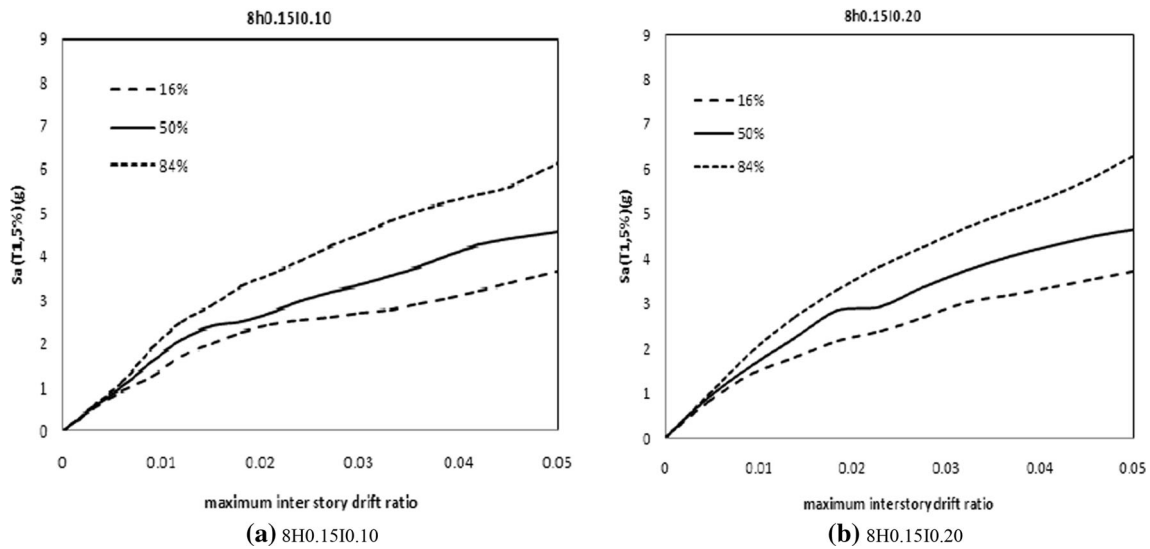
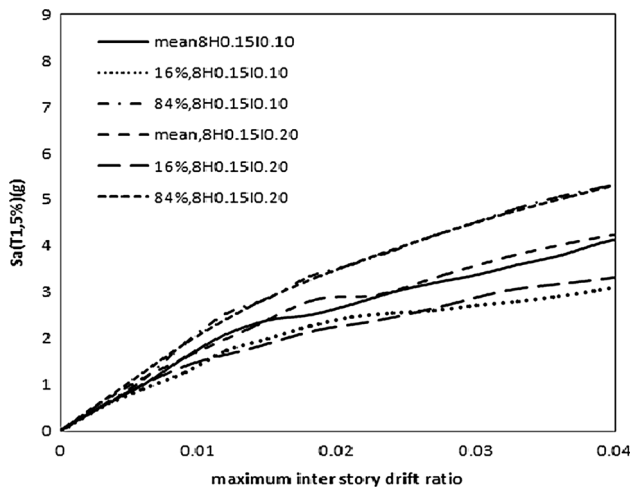Fig. 8 A summary of the IDA results of 8 story buildings

Fig. 9 Comparing the summary of the IDA results of 8 story buildings

In Figs. 6 and 7, summary of the IDA's results is presented which includes the average curve and the 16 and 84 % quintiles. The quintile, actually, show the dispersion of the results of the analysis.

The summary of the IDA curve makes the interpretation of the countless results of the analysis possible. The 50 % curve shows the average dynamic response of the structure to earthquakes and yields the maximum drift of the stories in terms of the magnitude of the earthquake. The 16 and 84 % diagrams show the changeability of this average value. It is worth to point out that although the incremental dynamic analysis is the most powerful and reliable method at hand to analyze a structure subjected to seismic loads, still it is accompanied with excessive uncertainties in the

exact response of the structure. To put that in perspective's pace as it is seen, the distance between the quintiles of the results of the analysis and the average diagram is considerable and this phenomenon enhances in extremely non-linear domains, which makes the practical use of the results difficult. Despite all of that, the aforementioned results have the lowest deviation from the intensity and damage measures. For this reason, it seems that because of the above short comings, the path will be opened to exacting the incremental dynamic analysis, e.g., assessing the sensitivity of the results to different intensity parameters.

As it can be seen in the diagrams that compare the Figs. 8 and 9, the average diagrams and the quintiles overlap on a certain drift which in fact shows the elastic limit and the linear behavior of the structure. Prior to this drift, the structures response to different accelarograms will not go through any kind of change and the displacement is proportionate to the force and in this range, even the hooks low can be implemented to obtain the stiffness of the structure. ($F = KX$). After this drift the quintile diagrams deviate from the average diagram, but still the average diagrams illustrate equal responses.

This part shows the development of linear behavior in the structures, but still, inducing the moment of inertia has not affected the behavior of the structure. After this point, the average diagram of the frames with low knee moment of inertia will be under the frame with higher moment of inertia and this shows that reduction in the moment of inertia causes the frames strength to decrease and that for a slight increase in the earthquake force, the frame experiences a larger drift.

In order to more accurately show the behavior of the structure along the changes of the drifts have been used and
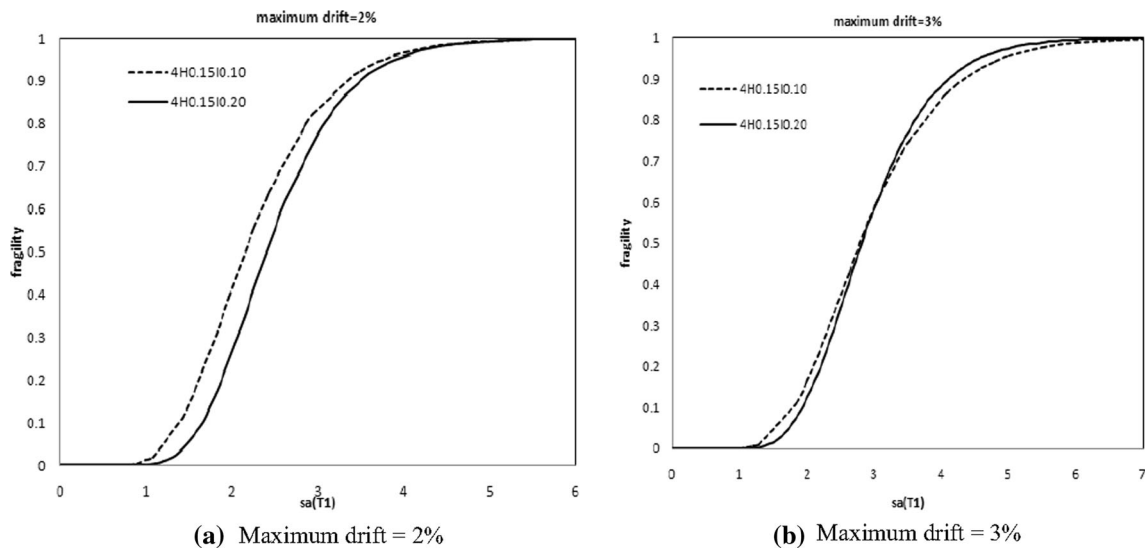
(a) Maximum drift = 2%   (b) Maximum drift = 3%

Fig. 10 The collapse diagrams to surpass the 2 and 3 % drifts of 4 story frames

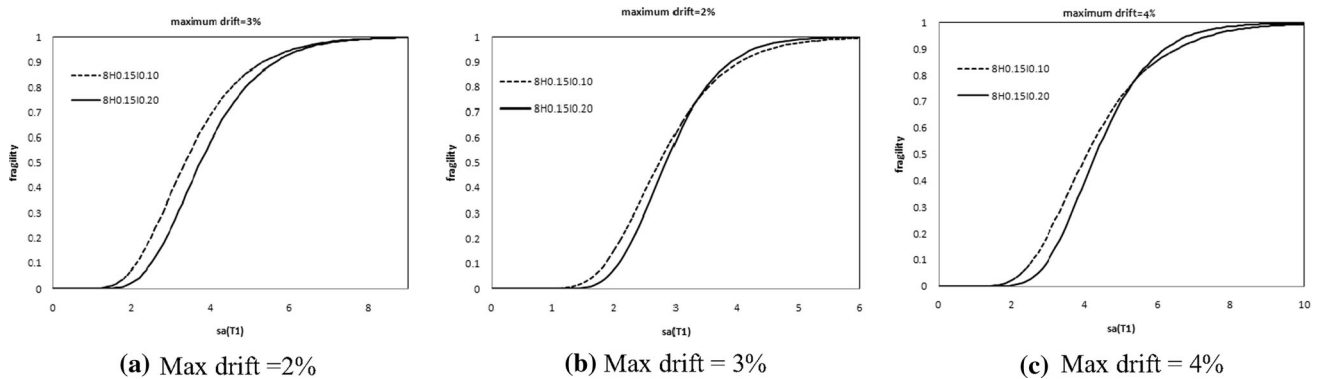(a) Max drift =2%   (b) Max drift = 3%   (c) Max drift = 4%

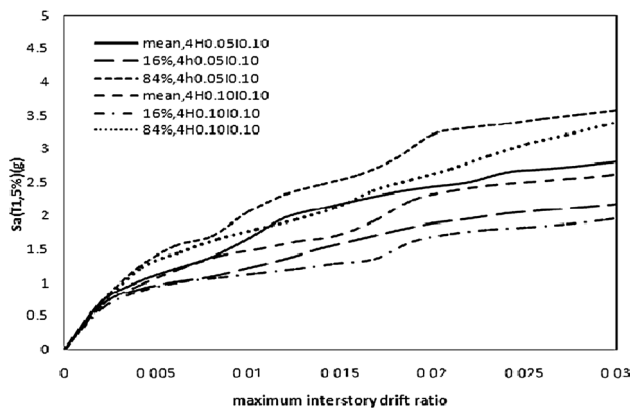Fig. 11 The collapse diagrams to surpass the 2, 3 and 4 % drifts of 8 story frames

Fig. 12 Comparing the summary of the IDA results of 4 story buildings
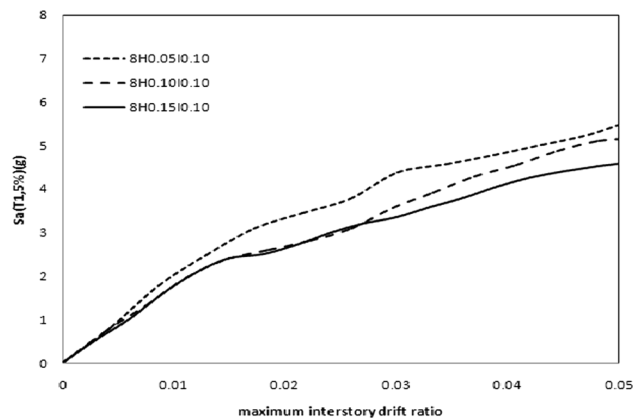

Fig. 13 Comparing the summary of the IDA results of 8 story buildings

the collapse diagrams of 2, 3 and 4 % the frames in the aforementioned drifts are compared. The collapse diagrams can be observed in Figs. 10 and 11.

As it can be seen from the collapse diagrams, for the mentioned drifts with low knee moment of inertia are above those with high knee moment of inertia and so the
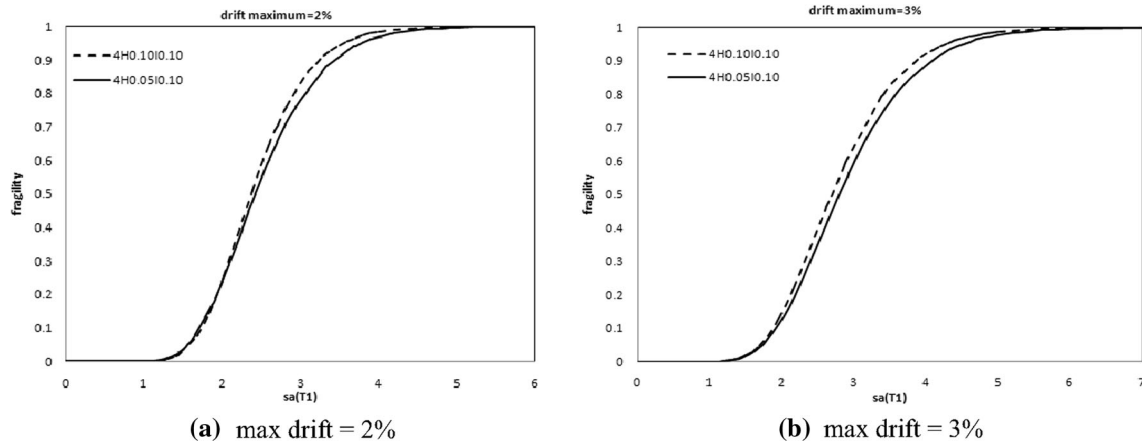
Fig. 14 The collapse diagrams to surpass the 2 and 3 % drifts of 4 story frames
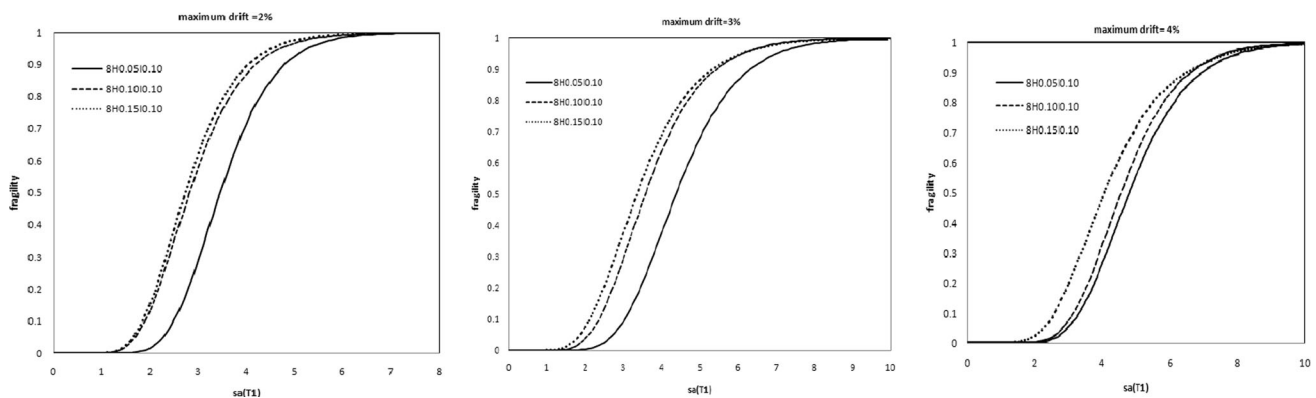

Fig. 15 The collapse diagrams to surpass the 2, 3 and 4 % drifts of 8 story frames

probability of failure of frames with low knee moment of inertia is higher than those with low knee moment of inertia. These collapse diagrams show the probable passing of the frames drift from the particular drift (threshold of the limit state) mentioned in the diagram for the future earthquakes.

Dynamic analysis results of 4 and 8 story frames with constant knee moment of inertia and variable length

Here, for the sake of brevity, only the results of the incremental dynamic analysis of the 4 and 8 story frames are presented.

As it can be seen in the average diagrams of Figs. 12 and 13, increasing the knee length in these frames has caused the average response of the frame to increase, i.e., in a lower earthquake intensity, the frame experiences a larger drift and the average diagrams of the frames with larger knee length is below that of the frames with the smaller knee length. This means that the strength of the

frames with a larger knee length is lower than that of the frames with smaller knee length to better understand the subject, for an increase in the earthquake's intensity, the collapse diagrams in 2, 3 and 4 % are presented in Figs. 14 and 15.

As it can be observed in the collapse diagrams for the aforementioned drift, the diagram of the larger knee length is above the diagram of the smaller knee length. i.e., the frame with the larger knee length is more lightly to fail than the frame with the smaller knee length.

Conclusion

Results of the incremental dynamic analysis show that the response of a particular structure different accelerograms varies according to the specifications of the structure and the earthquake such as duration of the earthquake, frequency, amplitude, etc., at times these differences are so significant that the responses of multiple accelerograms cannot be compared. The diagrams represent the efficient

behavior of the knee bracings as compared to the diagonal bracings. In the diagonal-braced frames, the drifts of the building greatly increase in one or two stories due to the severe buckling of the bracings under seismic forces, but in the knee-braced frames, before buckling of the bracing, the knee members will yield, start to absorb the energy, and prevent buckling of the bracing.

The drift of the stories under seismic forces is higher in the knee members with great length and weak moment of inertia, and, as a result, when the length of the knee is greater, the use of lower moments of inertia for the knee is not recommended.

The summary of the average IDA diagram for frames with smaller knee length, in all conditions (4 and 8 story frame) is higher than the average diagram of the frames with larger knee lengths, i.e., the frames with a large knee length experience a larger drift as the intensity of the earthquake increases. Thus, increasing the knee length causes the earthquake resistance of the frames to diminish.

The summary of the average IDA diagrams for frames with larger knee moment of inertia, in all conditions is higher than the average diagrams of frames with smaller knee moment of inertia, i.e., the frames with low knee moment of inertia experience a larger drift as the intensity of the earthquake increases. Thus decreasing the moment of inertia causes the earthquake resistance of the frame to diminish.

Collapse diagram of the frame with the larger knee length was above the collapse diagram of the frame with the smaller knee length, i.e., the frame with the larger knee length more probable to collapse than the frame with the smaller knee length.

At small knee lengths, the use of the strong moments of inertia leads to the buckling of the brace; therefore, using the knee element with a small length and a high moment of inertia is not recommended. However, even in the worst conditions, the behavior of this system is more efficient than that of the diagonal system.

One of the most influential parameters in the collapse diagrams is the selected accelerograms to analyze the structure. As it was seen in the IDA diagrams, the structure shows hardening behavior to some accelerograms and softening behavior to some others and thus it possible for the accelerogram to be selected in such away that all or most of them have stiffening behavior which makes the resulted collapse diagram to show a low collapse probability. In contrast, the accelerograms can be selected in such a way that the diagrams display softening behavior and the resulted collapse diagram show a high collapse probability. Thus the selection of the accelerograms affects the collapse diagrams and the acquired collapse diagrams here in are based on the accelerograms used in this study.

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