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SPECIAL ECCENTRICALLY BRACED STEEL FRAME TO EFFECTIVELY RESIST SEISMIC LOADS

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ABSTRACT

In this research, the seismic performance of a proposed efficient energy dissipating eccentrically braced frame (EEDBF) in relation to that of moment resisting frame (MRF) and chevron braced frame (CBF) is studied through nonlinear static pushover (NLSP), nonlinear time history (NLTH) and damage analyses. The analyses results revealed that the EEDBF has a more stable lateral force-deformation behavior compared to CBF. The energy dissipation capacity of the EEDBF is comparable to that of the MRF. The drift of the EEDBF at small to medium intensity ground motions is comparable to that of the CBF and smaller than that of the MRF. At high intensity ground motions, the drift of the EEDBF is smaller than those of both CBF and MRF. Furthermore, the EEDBF is found to experience less damage compared to other frames.

Introduction

Steel structures are designed for ductility, where the earthquake energy is dissipated in plastic hinges that occur in frame members. Under this design philosophy, when a conventional steel structure such as MRF or CBF is subjected to a strong earthquake, yielding of the plasticized zones and the resulting large permanent displacements may induce nearly irreparable damage to the structure. Thus, it may be required to rebuild the essential structural members of the structure following a strong earthquake to ensure a satisfactory performance under service loads.

MRF and CBF are commonly used in the construction of steel buildings. The MRF has large ductility capacity compared to other frame types. However, the it requires large member sizes to keep the lateral drifts within code-mandated limits. Even then, the flexibility of the MRF may result in large drift-induced nonstructural damage under seismic loading. Furthermore, in an MRF, as the inelastic deformation of the beams results in dissipation of energy, substantive damage to these members may be induced. Consequently, costly post-earthquake rehabilitation of the structure may be required. On the other hand, CBF possesses high elastic stiffness to prevent large drifts. Material saving could also be achieved as the frame members are subjected to less bending effect due to the presence of the braces. However, its ability to dissipate energy solely depends on the unstable hysteretic behavior of the braces due to buckling effects producing loss of lateral stiffness and strength of the frame (Khatib et al 1988). This in turn, results in soft-story formations, instability (Tremblay 2001) and hence substantial damage to the frame members.

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In light of the above discussion, it is clear that in addition to the advantages of each frame type, there are numerous disadvantages. Furthermore, both frames are prone to substantial structural damage during a strong earthquake. Consequently, a novel frame system with a design philosophy that will concentrate on minimizing seismic damage to its essential structural members is needed to ensure satisfactory post-earthquake performance of steel buildings under service loads with minimal rehabilitation cost.

Proposed Special Eccentrically Braced Steel Frame

The research study is focused on a proposed EEDBF configured to minimize seismic damage to its essential structural members by combining the advantages and eliminating the disadvantages of MRF and CBF. It is composed of a rigid frame with chevron braces and a conventional energy dissipating shear element (SE) connected between the braces and the beam as shown in Figs. 1(a) and (b). The SE is designed to yield in shear before the compression brace buckles. It is built using a compact HP section that is capable of undergoing large inelastic deformations (Dicleli and Albhaisi 2004) due to its stocky configuration. The proposed EEDBF is intended to behave similar to a CBF prior to yielding of the SE under small to moderate intensity earthquakes to minimize inter-story drifts. Under large intensity earthquakes, the EEDBF is intended to combine properties such as the high lateral stiffness of the CBF and the energy dissipating capacity of the MRF combined with that of the SE to minimize seismic damage to the essential structural components of the frame. Most of the damage is anticipated to occur within the SE, which may be replaced after a potential earthquake. In the subsequent sections, the performance of the proposed EEDBF will be assessed in comparison to those of the MRF and CBF under monotonic and seismic loading using the program ADINA (2004).

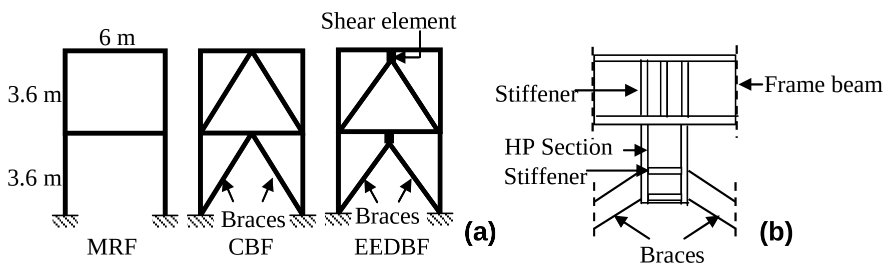

Figure 1. (a) Typical two-story frames, (b) Details of shear element and connections in EEDBF.

EEDBF Design Methodology

A design methodology based on preventing the buckling of the braces is adopted for the EEDBF to achieve a more stable hysteretic behavior and to avoid damage to the braces and other structural members. The buckling of the braces is prevented by allowing the SE to yield in shear before the compression brace reaches its buckling capacity. At the verge of buckling of the compression brace, the axial forces in the tension and compression braces will be both equal to the buckling load, P_b. Consequently, to prevent buckling of the compression brace, the SE is designed to have a plastic shear capacity, V_p, less than twice the horizontal component of the buckling load of the brace. Thus;

$$\phi_o V_p \leq 2 P_b Cos\theta \tag{1}$$

where, θ is the angle that the chevron braces make with the horizontal, ϕ_o is an over-strength factor for the SE and V_p is expressed as (AISC 2001);

$$V_p = 0.6 F_y d_w t_w \tag{2}$$

where F_y is the yield strength of steel, and d_w and t_w are the depth and thickness of the web. The failure mode for the SE is inelastic web shear buckling. To ensure a stable energy dissipation mechanism, this mode of failure may be delayed by the addition of web stiffeners (Kasai and Popov 1986).

Furthermore, the length, L_s, of the SE needs to be determined such that yielding occurs in shear before its plastic moment capacity, M_p, is reached. For the calculation of L_s, assuming a rectangular V_p-M_p interaction and including the strain hardening effect, the length of the cantilever SE is calculated as (Kasai and Popov 1986);

$$L_s = \frac{0.8M_p}{V_p} \tag{3}$$

Details of the Frames Considered for Analyses

In this study, single and multiple story frames are considered. Typical two-story MRF, CBF and EEDBF are demonstrated in Fig. 1(a). To enable a direct comparison of the performance of the three frames, their member sizes are determined such that all the frames have identical lateral strengths. Additionally, since the design of an MRF is typically controlled by its drift limit (or stiffness), another frame, MRF*, is designed to have the same code-mandated drift limit per AISC (2002) seismic design provisions as that of the CBF and EEDBF under the same seismic loading. This new frame (MRF*) will allow the comparison of the seismic responses of the frames based on their drift limit (or stiffness) rather than their strength. All the frames are designed to exhibit nonlinear behavior under moderate to large intensity ground motions (A_p= 0.35g and 0.5g).

For the single story studies, a set of typical frames for NLSP analyses and three sets of frames, MRF/CBF/EEDBF 1, 2 and 3 for NLTH analyses are considered to study the effect of the brace contribution to the lateral strength and brace slenderness ratio on the seismic response of the frames. In CBF/EEDBF 1 the brace contribution to the lateral strength of the frames is 40% and the slenderness ratio of the braces is 96. In CBF/EEDBF 2 and 3, the braces resist 60% of the lateral load applied to the frames and the slenderness ratios of the braces are 96 and 125 respectively.

For the multiple story studies, two, four and eight story MRF, CBF, EEDBF and MRF* are considered. First, the eight-story frame is configured such that each two-story levels have the same member sizes and the lateral strength of the frame gradually decreases at the higher story levels. The two and four-story frames are then assumed to form the bottom two and four stories of the eight-story frame respectively. This was done to ensure comparable base shear capacities for all the frames for studying the effect of the number of stories on the seismic performance of the frames.

Ground Motions Considered for Analyses

Seismic ground motions are generally characterized by their peak ground acceleration, A_p, to peak ground velocity, V_p, ratios (Dicleli and Buddaram 2006) which represent their dominant frequency and energy content. Consequently, a set of seven ground motions with A_p/V_p ratios ranging between 5.5 and 21.5 s^{-1} are considered (Table 1). The ground motions are scaled to have A_p=0.20g, 0.35g and 0.50g representing respectively, small, moderate and large intensity earthquakes.

Table 1. Earthquake records used in the analyses.

Earthquake	Station	A_p (g)	V_p (cm/s)	A_p/V_p (1/s)
San Fernando, 1971	8244 Orion Blvd	0.13	23.9	5.5
Loma Prieta, 1989	Oakland Outer Wharf	0.22	35.4	6.1
Northridge, 1994	Arleta & Nordhoff Fire Station	0.34	40.4	8.4
Imperial Valley, 1940	El Centro	0.35	32.3	10.6
Northridge, 1994	Santa Monica City Hall.	0.37	24.9	14.6
Whitter Narrows, 1987	90079 Downey Birchdale	0.24	13.7	17.4
Parkfield, 1966	Cholame, Shandon	0.24	10.8	21.5

Single Story Frames

Nonlinear Static Pushover Analyses Results

Comparative Performance of MRF, CBF and EEDBF for Various Brace Slenderness Ratios

The NLSP analyses results having a comparison of CBF, EEDBF and MRF for various slenderness ratios of λ=40, 80 and 120 are demonstrated in Fig. 2 (a). In addition to these, a hypothetical case is also studied in which the buckling of the braces is prevented. It is observed that the elastic stiffness of the MRF (77,300 kN/m) is much smaller than those of the EEDBF (104,000 kN/m) and CBF (122,000 kN/m). Thus, both CBF and EEDBF will have limited drift and hence a more desirable performance under the effect of small to moderate intensity earthquakes where buckling of the braces in CBF and yielding of the SE in EEDBF may be minimal. It is also observed that the EEDBF has a more stable tri-linear force-displacement relationship as compared to CBF. Moreover, for λ≤80, the base shear capacities of the EEDBF and MRF are comparable. This shows that if the EEDBF is designed using braces with λ≤80, its monotonic energy dissipating capacity may be comparable to that of an MRF which has much larger member sizes.

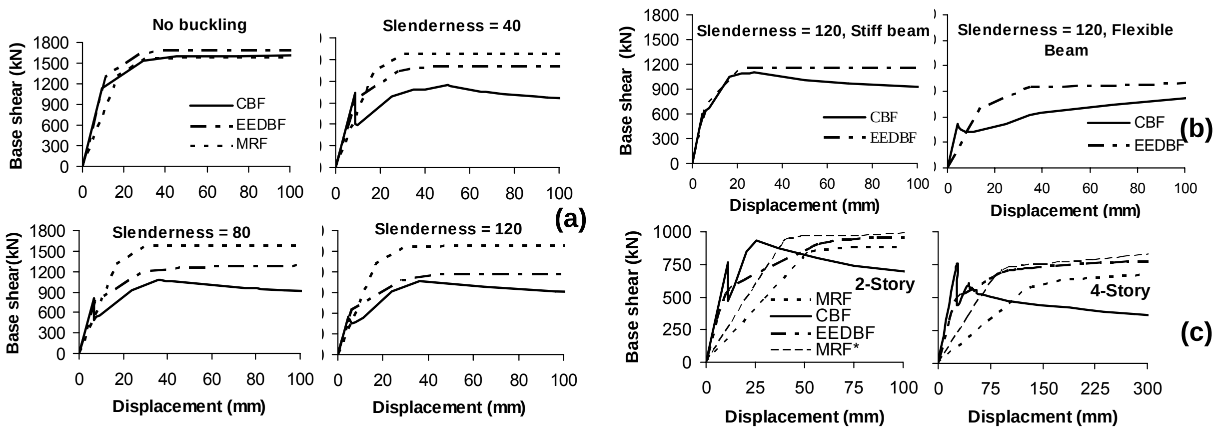

Figure 2. (a) Base shear versus frame displacement for various brace slenderness ratios (b) Effect of beam stiffness on force-displacement response (c) Base shear versus top displacement for two and four story frames.

Effect of Beam Flexural Stiffness

The performance of the CBF is known to be sensitive to the flexural stiffness of the beam (Khatib et al. 1988). Thus, in this section, the sensitivity of the EEDBF to the stiffness of the beam is studied in comparison to CBF. For this purpose, the flexural stiffness of the beam of the frames considered in the analyses is modified by a factor of 10 to obtain a flexible and a stiff beam. Fig. 2(b) displays a comparison of the lateral force-displacement relationship of the EEDBF and CBF for stiff and flexible beam cases for a brace with λ=120. In the case of the CBF, a large unbalanced vertical load is produced on the beam due to the difference between the axial loads in the tension and buckling braces. Thus, the flexible beam is forced to displace down and reach its flexural yield capacity. This phenomenon produces a reduction in the lateral load capacity of the CBF compared to the stiff beam case as observed from Fig. 2(b). However, for the EEDBF, since the tension and compression brace forces remain identical due to the controlled yielding of the SE, the flexural stiffness of the beam affects only the elastic stiffness of the frame and the magnitude of the lateral displacement at which the ultimate strength of the frame is reached. Thus, a more stable behavior is observed in the case of the EEDBF regardless of the beam stiffness.

Nonlinear Time History Analyses Results

Performance of the Frames versus Intensity and A_p/V_p Ratio of Ground Motions

Fig. 3 displays the maximum story drifts of the MRF (and MRF* for frame set 1), CBF and EEDBF for each frame set as a function of the A_p/V_p ratio of the ground motions for low (A_p=0.20g), medium (A_p=0.35g) and high (A_p=0.50g) ground motion intensities respectively.

For A_p=0.20g and 0.35g, Fig. 3 shows that while the lateral drift responses of the CBF and EEDBF are comparable, the MRF (and MRF* for frame set 1) produces higher drifts for the range of A_p/V_p ratios and for all the frame sets considered. It is also observed that while the peak drifts for the MRF occur at low to intermediate A_p/V_p ratios, those for the MRF*, CBF and EEDBF are generally more uniform.

For A_p=0.5g, there is a considerable increase in the value of lateral drifts of the frames, especially in the case of the MRF and CBF. It is also observed that while the MRF, MRF* and EEDBF display a relatively uniform response over the range of A_p/V_p ratios and ground motion intensities considered, CBF seems to be highly sensitive to the A_p/V_p ratio of the ground motion. Close examination of the behavior of the CBF revealed that the sensitivity of the frame's seismic response to the A_p/V_p ratio mainly depends on the buckling behavior of the brace. For ground motions with lower intensities, either no buckling or limited buckling behavior of the brace is observed. This produces a more uniform response of the frame over the range of A_p/V_p ratios considered. However, for ground motions with higher intensities, the buckling behavior of the brace becomes more dominant and the frame becomes more sensitive to the A_p/V_p ratio due to the degradation in the stiffness and strength of the frame associated with buckling phenomenon.

In summary, both the CBF and EEDBF display a more desirable response than that of the MRF and MRF* for low to medium intensity ground motions over the range of A_p/V_p ratios considered. However, for high intensity ground motions the response of the CBF becomes highly unstable due to the effect of brace buckling. The EEDBF however, displays a highly stable response for the range of A_p/V_p ratios considered. The story drift of the EEDBF is smallest of all the frames considered in this study.

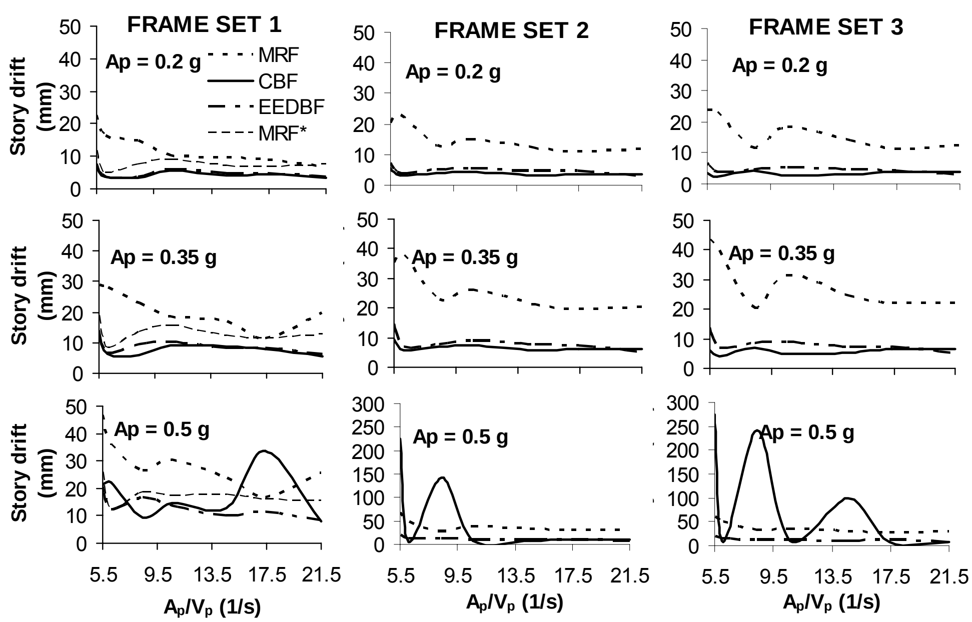

Figure 3. Maximum story drift versus A_p/V_p ratio of the ground motions for the three frame sets and for various A_p.

Effect of Brace Contribution and Slenderness

In this section, the effect of the brace slenderness and contribution to the lateral strength of the frame on the seismic response of the EEDBF and CBF are investigated in relation to MRF. Fig. 3 demonstrates that for A_p=0.20g and 0.35g, the EEDBF generally yields a response comparable to that of the CBF regardless of the properties of the frame for the range of brace contribution (40%-60%) and brace slenderness ratios (96 and 125) considered. Obviously, since the brace contribution to the lateral strength is smaller in CBF 1 the story drift for this frame is larger than those of the other two frames (CBF 2 and 3).

For A_p=0.5g, CBF 2 and 3 produce exceptionally higher drifts compared to those of the CBF 1, EEDBF 1,2,3 and even MRF 1,2,3 and MRF*. This may be attributed to the more dominant buckling behavior of the braces resulting from the larger brace contribution to the lateral strength in CBF 2 and 3. Furthermore, the seismic response becomes more sensitive (unpredictable) to the A_p/V_p ratio of the ground motion for CBF 2 and 3 as observed from the humps and undulations displayed in the two graphs at the bottom right of Fig. 3. Moreover, Fig. 3 clearly shows that the lateral drift of the EEDBF for low to medium intensity ground motions is smaller than that of the MRF (and MRF* for frame set 1) and comparable to that of the CBF for all the frame sets considered. However, for high intensity ground motions the lateral drift of the EEDBF is considerably smaller than those of all the frames considered at certain A_p/V_p ratios.

In summary, the performance of the CBF is highly dependent on the slenderness and contribution of the brace to the lateral strength of the frame. However, the performance of the EEDBF is more stable and is independent of such parameters. Furthermore EEDBF yields the smallest drifts of all the frames when subjected to high intensity ground motions. This is indicative of less potential damage to the essential structural members of the EEDBF.

Multiple Story Frames

Nonlinear Static Pushover Analyses Results

NLSP analyses of the multiple-story frames are performed assuming a triangular lateral load pattern along the height of the frames. The results of the NLSP analyses of the two and four story frames are depicted in Fig. 2(c). The figure displays the base shear force as a function of the drift at the top story level. In the case of the CBF the loss of lateral strength associated with buckling behavior is clearly observed. In all the cases the elastic stiffness of the MRF and MRF* is lower than those of the other two frames and the frames reach their ultimate lateral strength at a higher drift value. On the other hand, the EEDBF exhibits a higher elastic stiffness compared to MRF and MRF* and a more stable monotonic force-deformation relationship compared to CBF. The observations from the NLSP analyses of multiple story frames confirm the findings from the one story studies.

Nonlinear Time History Analyses Results

Performance of Multiple Story Frames in Relation to A_p/V_p Ratio and Intensity of Ground Motions

Fig. 4(a) displays the maximum inter-story drifts of the MRF, MRF*, CBF and EEDBF for two, four and eight story frames as a function of the A_p/V_p ratio of the ground motions considered for low (A_p=0.20g), medium (A_p=0.35g) and high (A_p =0.50g) intensities respectively.

For A_p=0.20g and 0.35g, Fig. 4(a) shows that while the seismic lateral displacement responses of the CBF and EEDBF are comparable for the two and four story frames, the MRF and MRF* produce higher drifts for the range of A_p/V_p ratios considered. In the case of the eight story frames, while the CBF and MRF exhibit comparable inter-story drifts, the EEDBF and MRF* produce lower inter-story drifts. It is also observed that while the peak drifts for the MRF and MRF* generally occur at low to intermediate A_p/V_p ratios, those for the CBF and EEDBF are generally more uniform.

For A_p=0.5g, there is a considerable increase in the inter-story drifts of the frames, especially in the case of the MRF and CBF. The dramatic increase in the drifts of the CBF is due to the buckling of the braces resulting in soft-story formations. It is also observed that while the MRF, MRF* and EEDBF display relatively stable response over the range of A_p/V_p ratios and ground motion intensities considered, CBF is highly sensitive to the A_p/V_p ratio at higher ground motion intensities due to the buckling of the braces.

In summary, both the CBF and EEDBF display a stable and a more desirable response than that of the MRF and MRF* for low to medium intensity ground motions for two and four story frames over the range of A_p/V_p ratios considered. However, for high intensity ground motions and for increasing number of stories, the response of the CBF becomes highly unstable due to the effect of brace buckling. The EEDBF and MRF* display a stable response for the range of A_p/V_p ratios considered.

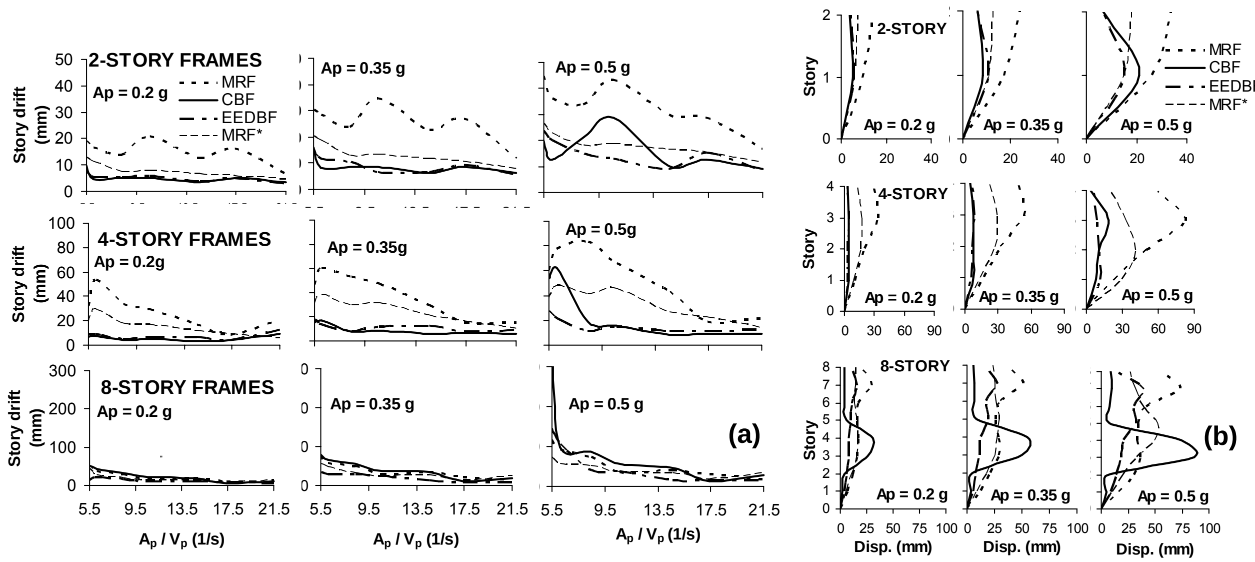

Figure 4. (a) Maximum inter-story drift versus A_p/V_p ratio of the ground motions for various A_p for the two, four and eight story frames. b) Displacement profile of the two, four and eight story frames for various A_p (A_p/V_p=8.4 s^{-1}).

Displacement Profile of the Frames

Fig. 4(b) compares the deformed shapes of the two, four and eight-story CBF, MRF, MRF* and EEDBF for a ground motion with A_p/V_p=8.4 s^{-1} scaled for A_p=0.2g, 0.35g and 0.5g. The deformed shapes of the frames are obtained at the instant when the maximum inter-story drift occurs.

For A_p=0.2g and 0.35g, the figure reveals that for the two and four story frames, both the CBF and EEDBF display similar deformed shapes. The deformation of both frames is mostly concentrated at the first story level with the deformation at the upper story levels being relatively modest. In the case of the MRF and MRF*, the deformation at the first story level is also large. However, the MRF and MRF*, also experience notable deformations at the upper story levels due to the flexibility of the frames. For A_p=0.5g, the buckling of the braces in the two and four story CBF dominates the behavior of the frame where inter-story drifts larger than those of the EEDBF are observed. The MRF and MRF*, also experience deformations larger than those of the EEDBF due to the larger flexibility of the frames and yielding of the beams. The EEDBF exhibits generally a more stable deformation pattern for the range of ground motion intensities considered.

For the eight story frames, the buckling of the braces dominates the behavior of the CBF for the range of ground motion intensities considered. This resulted in soft story formations as observed from Fig. 4(b). It is also observed that the displacement profile of the CBF is highly sensitive to the A_p/V_p ratio of the ground motion and the number of stories. Consequently, the design of the CBF becomes highly unreliable. Compared to CBF, the MRF and MRF* exhibit a better lateral deformation pattern and hence a more even distribution of energy dissipation along the height of the frame. Nevertheless, the EEDBF exhibits even a more uniform lateral displacement profile and a smaller inter-story drifts compared to the other frames.

Effect of Number of Stories on the Seismic Response of the Frames

In this section, the analyses results for the multistory frames are consolidated and the average of the maximum inter-story drifts from the seven earthquakes are presented in the form of bar charts for two, four and eight story frames corresponding to low, medium and high intensities in Fig. 5. The figure reveals that for two and four story frames, the MRF and MRF* yields higher inter-story drifts than those of the CBF and EEDBF due to the flexibility of the frames and yielding of the beams. Generally, the CBF displays a reasonably good response at low and moderate intensities of ground motions and for smaller number of stories. However, for larger number of stories, a sudden increase in the inter-story drifts of the CBF is observed, indicating an unstable behavior due to the presence of more slender braces and flexible beams at the upper stories promoting buckling behavior. EEDBF, on the other hand exhibits a more uniform increase of the inter-story drift as the height of the frame increases indicating a more desirable and reliable response compared to the CBF. Furthermore, the EEDBF is observed to have generally smaller inter-story drift that is indicative of less potential structural and nonstructural damage.

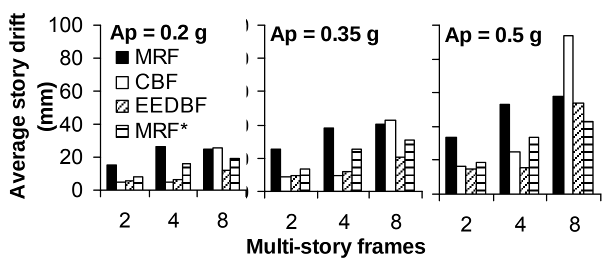

Figure 5. Average of maximum inter-story drifts as a function of the number of stories for various A_p.

Damage Analyses of Multiple Story Frames

In this section, damage analyses of the three four-story frames are performed for two ground motions (A_p/V_p=5.5 s^{-1} and A_p/V_p=5.5 s^{-1}) and two intensities (A_p=0.35g and 0.5g) to further assess the performance of the proposed EEDBF in relation to CBF, MRF and MRF*.

Seismic damage quantification is generally represented by damage indices that range between 0 (no damage) and 1 (complete collapse). Many researchers have proposed a number of damage models that calculate damage indices (Park and Ang 1985, Kunnath et al. 1997, Khashaee 2005). However, most of these damage models have concentrated almost exclusively on flexural modes of failure. Thus, they may not be applicable to the EEDBF and CBF due to the presence of shear yielding and brace buckling. Nevertheless, the damage model proposed by Hindi and Sexsmith (2001) is primarily based on the monotonic energy dissipating capacity of structural elements before and after the application of reversed cyclic loading. Therefore, it may be universally applicable to structural members exhibiting failure modes other than flexure, including steel members failing in the shear or buckling mode. Consequently, it is used for the damage assessment of the frames considered in this study.

Damage Model of Hindi and Sexsmith (2001)

The damage model takes as a reference the monotonic energy dissipation capacity of a structure in the undamaged, virgin state, which is defined as the area, A_o, under the static pushover curve up to the point of failure. With the actual 'n' cycles of load-displacement history applied on the structure due to a potential earthquake, the remaining monotonic energy dissipation capacity of the structure compared to that in its virgin state defines the extent of damage. The remaining monotonic energy dissipation capacity of the structure is defined as the area, A_n, under the static pushover curve obtained from the end of the last cycle 'n' to the failure point. Accordingly, the damage index is the ratio;

$$D_n = \frac{(A_o - A_n)}{A_o} \tag{4}$$

Discussion of Damage Analyses Results

The results of the damage analyses of the frames are presented in Table 2. It is observed that earthquakes with low A_p/V_p ratio and high intensity are particularly damaging to CBF, as it loses up to 40% of its monotonic energy absorption capacity due to the buckling of the braces. The MRF and to a lesser extent MRF* display small to moderate damage indices for the A_p/V_p ratios and intensities considered in the analyses. On the other hand, EEDBF has generally lower damage indices compared to the other frames. This indicates a greater reserve energy dissipation capacity, less damage and smaller rehabilitation cost after a major seismic activity.

Table 2. Damage indices for the four-story MRF, MRF*, CBF and EEDBF.

		A_p/V_p=5.5 s^{-1}				A_p/V_p=8.4 s^{-1}			
		A_p=0.35 g		A_p=0.5 g		A_p=0.35 g		A_p=0.5 g	
Frame	**A_o (kN-m)**	**A_n (kN-m)**	**D_n**	**A_n (kN-m)**	**D_n**	**A_n (kN-m)**	**D_n**	**A_n (kN-m)**	**D_n**
MRF	129.1	103.2	0.201	86.5	0.330	123.8	0.042	114.3	0.115
*MRF**	200.9	184.2	0.083	173.4	0.137	195.6	0.026	180.5	0.102
CBF	87.5	75.7	0.135	51.8	0.408	79.4	0.094	74.3	0.151
EEDBF	133.7	124.8	0.066	119.9	0.103	129.7	0.030	128.8	0.036

Conclusions

The seismic performance of the proposed EEDBF is investigated analytically in comparison to MRF, MRF* and CBF through NLSP, NLTH and damage analyses using single and multiple story frames. The conclusions derived from this study are presented below.

The NLSP analyses results revealed that the proposed EEDBF exhibits a more stable lateral force-displacement relationship compared to that of the CBF as no degradation in the lateral strength is observed due to the buckling of the braces. Moreover, although the EEDBF has higher elastic stiffness and smaller member sizes compared to MRF, its monotonic energy-dissipation capacity is comparable to that of the MRF for low and intermediate brace slenderness values. Thus, while the EEDBF combines the advantages of both frames, it eliminates most of the disadvantages particular to each frame.

The NLTH analyses of the frames revealed that MRF and MRF* generally yield large inter-story drifts for small to medium intensities of ground motions compared to the other frames. On the other hand CBF displays a good response for low to moderate intensity earthquakes and for lower number of stories. Nonetheless, for high intensity earthquakes and for larger number of stories, a sudden deterioration in the strength and stiffness of the frame is observed due to the effect of brace buckling. Furthermore, the behavior of the CBF is found to be highly dependent on the brace contribution to the overall strength of the frame, the slenderness of the braces and the A_p/V_p ratio. On the other hand, EEDBF displays a more stable behavior over a wide range of structural and ground motion properties. It also exhibits a more even distribution of earthquake input energy over the height of the frame and generally yields lower drifts as compared to the other frames studied. Moreover, the behavior of the EEDBF is independent of the brace properties. Thus, EEDBF combines the advantages of both CBF and MRF and therefore, displays an overall more desirable behavior as compared to the other frames. Damage analyses of the frames also revealed that the EEDBF generally exhibits less damage and larger reserve lateral deformation capacity compared to the other frames. Thus, in the event of an earthquake, it is anticipated that the yielding of the SE in the EEDBF will prevent buckling of the braces and minimize damage to the structural components of the frame. The damaged SE can be easily replaced for a relatively small cost. This may result in a minimal post earthquake rehabilitation cost compared to the other frames.

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