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ASSESMENT OF SEISMIC PERFORMANCE OF ECCENTRICALLY BRACED FRAME WITH VERTICAL MEMBERS

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ABSTRACT

This paper evaluates comprehensively seismic inelastic demand of eccentrically braced frames (EBFs) with vertical members. In this configuration, two zipper-struts are added to connect each end-point of the shear links in all stories. To investigate the efficiency of this system versus conventional EBFs, pushover and incremental dynamic analyses were performed. Results show that in this system, the fully-plastic hinges are almost simultaneously developed in all stories while damage in a conventional EBF concentrates only in a few floors. Moreover, the ductility and energy absorption capacity of this configuration are noticeably higher than those of the conventional EBFs.

Keywords: Eccentrically braced frames; zipper-struts; steel buildings; seismic performance; ductility; energy absorption capacity.

1. INTRODUCTION

Eccentrically braced frame (EBF) is a lateral load resisting system used for steel structures to resist forces induced by strong ground motions. In the EBFs, lateral forces are resisted by a combination of flexure, shear and axial forces in the frame members. An EBF is essentially a hybrid system which combines the stiffness of concentrically braced frames and the moment frames ductility.

EBFs are expected to accommodate inelastic deformation through ductile shear yielding of the link when subjected to earthquake loading. The link becomes the focal point in the design and detailing of an EBF. This member acts as a structural fuse which can dissipate seismic input energy without degradation of strength and stiffness. To achieve this behavior, links, as the weakest elements in the frame, can experience very large inelastic rotations during an earthquake. Other members of an EBF (including braces, columns and beams

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segments outside of the links) are considered to remain essentially elastic. Hence these members should be necessarily stronger than the links.

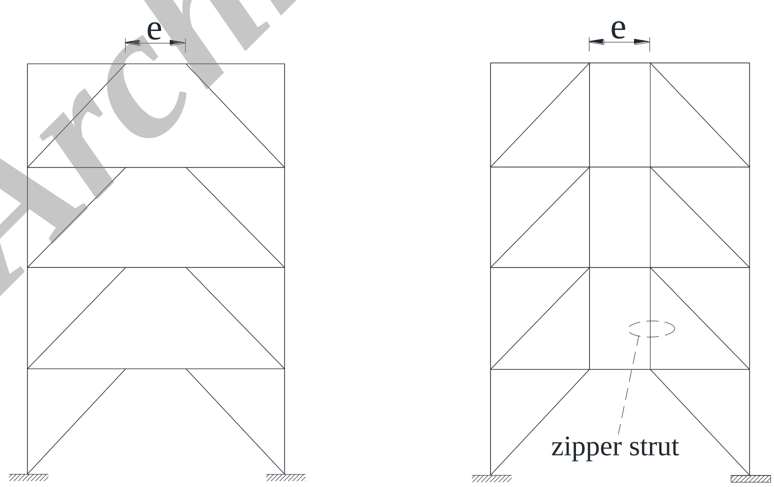
EBF technology is relatively modern as this system was first developed in Japan in the early 1970s. Popov et al. conducted extensive researches on seismic response of EBFs through 1978–1992. These investigations were followed by numerous theoretical and experimental studies on the seismic response of EBFs [1- 4]. Several structural systems have been proposed to mitigate the concentration of inelastic demand in steel braced frame. Those included zipper braced frame [5-10]. In order to enhance the seismic behavior of EBFs, frames with vertical members connecting link end joints was explored by Martini et al. [11]. This modified system named tied eccentrically braced frames (TBFs). Then this structural system was studied briefly by Popov et al. [12], Ghersi et al. [13, 14] and Rossi [15].

In this study, focus is comprehensively on the modification of conventional EBFs using the zipper-struts which connect two end-points of shear links in all stories. The method is applied to frames with different heights and number of stories subjected to various earthquake ground motions. The TBF is verified by means of non-linear static and dynamic analyses.

Results of numerous analyses show that the ductility and energy absorption capacity of the TBFs are significantly higher than those of conventional EBFs. Furthermore, in the TBFs, fully-plastic hinges are almost simultaneously developed in all stories while damage in a conventional EBF concentrates only in a few floors.

2. TBFS CONFIGURATION

In the TBF modification, two pinned zipper-struts are added to connect each end-point of the shear links in all stories. The link of the lowest story is not connected to the base level of the frame by any strut. The geometries of the conventional EBF and TBF are shown in Fig. 1.



(a) The conventional EBFs (b) The TBFs

Figure 1. The geometries of two systems

3. ASSESSMENT OF THE TBFS

To investigate the advantages of the TBFs versus conventional EBFs, a series of EBFs case study were chosen. Pushover and incremental dynamic analyses were performed to evaluate the seismic responses and ductility of two systems, using PERFORM-3D software [16]. The specifications and assumptions are given in the following sections.

3.1 Description of the case study structures

The EBF prototypes selected in this study consist of regular 3, 7, 10, 12 and 15-story EBFs with and without zipper-struts. The conventional EBF models of these structures were previously evaluated by Furukawa et al. [17]. The uniform story height and bay length are 144 and 360 in, respectively. The length of the shear link was chosen as 48 in. All frames have five bays of which two bays include EBF and the others have simple connections. In the EBF bays, the brace-to-beam and the beam-to-column connections are fully restrained. Typical configuration of 2-D frames is shown in Fig. 2.

The uniform dead and live loads of all floors are 0.12 and 0.056 kips/in, respectively and the lateral loading for the frames was based on the ASCE 7-2010 [18]. All prototypes are assumed to be founded on firm soil class C of NEHRP and located in the region of highest seismicity. The yield strength of steel is assumed to be $F_y = 50$ ksi for all structural members. The frames design based on the AISC (LRFD, 2010) [19] that satisfied allowable drift ratio criteria of 2%. The section sizes of the TBFs and the conventional EBFs are the same except that zipper- struts are added to the TBFs. Final section sizes of all frames are summarized in Table 1.

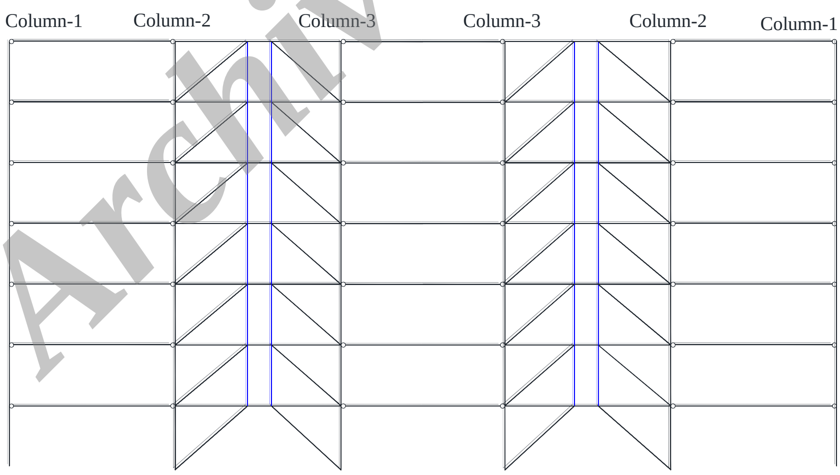


Figure 2. Typical configuration of 2-D frames

3.2 Nonlinear static analysis

After designing steel frame members, a non-linear pushover analysis is carried out to evaluate structural seismic response. For pushover analysis, the structure is subjected to lateral load pattern based on first mode shape of the structure. The use of this load pattern shall be permitted only when more than 75% of the total mass participates in this mode [20]. In the present paper, all models satisfy this requirement. Also the target displacement values of two systems based on FEMA 356[20] are listed in Table 2.

3.3 Incremental dynamic analysis

Incremental dynamic analysis (IDA) is a powerful tool which includes performing a set of nonlinear time-history analyses for a series of ground motion records scaled at increasing intensity levels. In this study, IDA is employed to verify the results of static analysis and determining the peak sustainable acceleration for the TBFs in comparison with conventional EBFs. Ten different ground motions are considered for this purpose based on studies of Vamvatsikos and Cornell [21]. The important objective in the selection of these causative records is distance from fault lines which ranges from 15-30 km. The basic parameters of the records are summarized in Table 3.

Table 1: Section sizes of models

n_s	Story	Column1 W	Column2 W	Column3 W	beam &link W	Simple beam (all stories)W	brace &zipper HSS- TBFs
	3	14x68	14x120	14x120	14x68	14x99	10x1/2
3	2	14x82	14x176	14x176	14x74		10x1/2
	1	14x99	14x233	14x283	14x82		12x1/2
	7	14x68	14x120	14x120	14x68	14x99	10x1/2
	6	14x82	14x176	14x176	14x74		10x1/2
	5	14x99	14x233	14x283	14x82		12x1/2
7	4	14x120	14x283	14x342	14x109		12x1/2
	3	14x132	14x370	14x426	14x120		12x1/2
	2	14x176	14x550	14x550	14x120		14x1/2
	1	14x176	14x550	14x550	14x82		10x1/2
	10	14x34	14x48	14x43	14x22	14x99	10x1/2
	9	14x43	14x68	14x61	14x34		10x1/2
	8	14x61	14x90	14x90	14x48		10x1/2
	7	14x68	14x120	14x120	14x68		10x1/2
	6	14x82	14x176	14x176	14x74		10x1/2
10	5	14x99	14x233	14x283	14x82		12x1/2
	4	14x120	14x283	14x342	14x109		12x1/2
	3	14x132	14x370	14x426	14x120		12x1/2
	2	14x176	14x550	14x550	14x120		14x1/2
	1	14x176	14x550	14x550	14x82		10x1/2
	12	14x34	14x34	14x34	14x22	14x99	8x1/2
	11	14x34	14x34	14x34	14x22		8x1/2
	10	14x34	14x48	14x61	14x34		8x1/2
	9	14x48	14x68	14x82	14x48		10x1/2
12	8	14x61	14x90	14x120	14x68		10x1/2
	7	14x82	14x132	14x176	14x82		12x1/2
	6	14x90	14x233	14x233	14x120		14x1/2
	5	14x99	14x283	14x342	14x120		14x1/2

	4	14x120	14x370	14x426	14x132		14x5/8
	3	14x132	14x550	14x550	14x132		14x5/8
	2	14x176	14x665	14x665	14x159		16x5/8
	1	14x176	14x550	14x665	14x68		12x1/2
	15	14x34	14x34	14x34	14x22	14x109	8x1/2
	14	14x34	14x34	14x34	14x22		8x1/2
	13	14x34	14x43	14x43	14x34		8x1/2
	12	14x34	14x48	14x43	14x34		8x1/2
	11	14x34	14x61	14x61	14x34		8x1/2
	10	14x48	14x90	14x90	14x61		10x1/2
	9	14x61	14x120	14x132	14x68		10x1/2
15	8	14x82	14x176	14x233	14x82		12x1/2
	7	14x90	14x233	14x283	14x120		14x1/2
	6	14x99	14x342	14x342	14x132		14x5/8
	5	14x120	14x370	14x426	14x132		14x5/8
	4	14x132	14x550	14x550	14x145		14x5/8
	3	14x176	14x550	14x665	14x159		16x5/8
	2	14x176	14x665	14x665	14x159		16x5/8
	1	14x233	14x665	14x665	14x74		12x1/2

Table 2: Target displacement and effective period of two systems

parameter	system	3-story	7-story	10-story	12-story	15-story
Target displacement	EBF	2.755	10.39	20.39	24.917	35.856
(inch)	TBF	2.742	10.403	20.232	24.624	35.942
Effective Period	EBF	0.4121	0.8365	1.336	1.535	2.12
(second)	TBF	0.4088	0.8434	1.333	1.521	2.109

Table 3: List of the ground motions based on studies of Vamvatsikos [21]

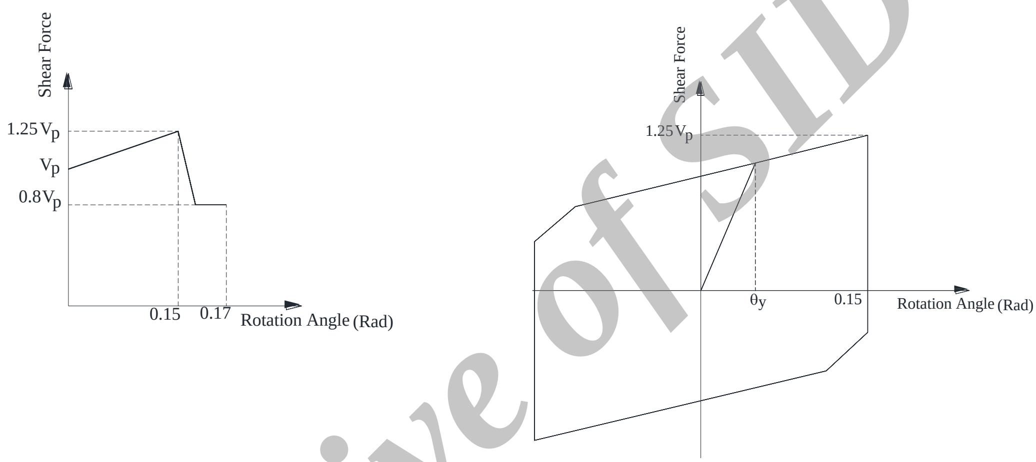
No.	Event	Station	Component	Moment	Distance to fault (km)	PGA(g)
1	Loma Prieta,1989	Agnews State Hospital	090	6.9	28.2	0.159
2	Imperial Valley,1979	Plaster City	135	6.5	31.7	0.057
3	Imperial Valley,1979	Cucapah	085	6.5	23.6	0.309
4	Imperial Valley,1979	Westmoreland Fire Station	090	6.5	15.1	0.074
5	Superstition Hills,1987	Wildlife liquefaction Array	090	6.7	24.4	0.18
6	Loma Prieta,1989	WAHO	090	6.9	16.9	0.638
7	Imperial Valley,1979	Chihuahua	282	6.5	28.7	0.254
8	Imperial Valley,1979	El Centro Array #13	140	6.5	21.9	0.117
9	Loma Prieta,1989	Coyote Lake Dam Downstream	285	6.9	22.3	0.179
10	Loma Prieta,1989	Hollister Diff.Array	165	6.9	25.8	0.269

3.4 Non-linear properties of the shear link

One of the main requirements of nonlinear analysis is the definition of non-linear formation of plastic hinges. As previously noted, members of the EBF system are designed to concentrate non-linear behavior on shear link while other members remain linear elastic. The ductility of the link is provided by stiffener plates, satisfying the width-to-thickness requirement and setting lateral bracing at the end of the link (AISC 341-2010)[22]. The non-linear model of the shear link based on FEMA 356 [20] is shown in Fig. 3. The plastic shear strength of the link (V_p) is determined using the following equation:

$$V_p = 0.55 F_{ye} A_w \tag{1}$$

where F_{ye} and A_w are the expected yield stress and the web area of the link, respectively.



(a) For pushover analysis (b) For incremental dynamic analysis

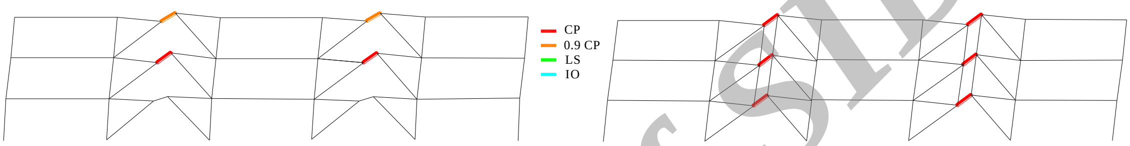
Figure 3. The nonlinear model of shear link mode [20]

4. COMPARISON AND DISCUSSION OF THE ABOVE TWO SYSTEMS

In order to assess the TBFs configuration versus the conventional EBFs, some seismic parameters such as ductility, energy dissipation and inter-story drift ratio are compared in these two systems. To determine the ultimate capacity of the system, two criteria are monitored; either one of the links reaches ultimate rotation or one of the stories attains peak drift.

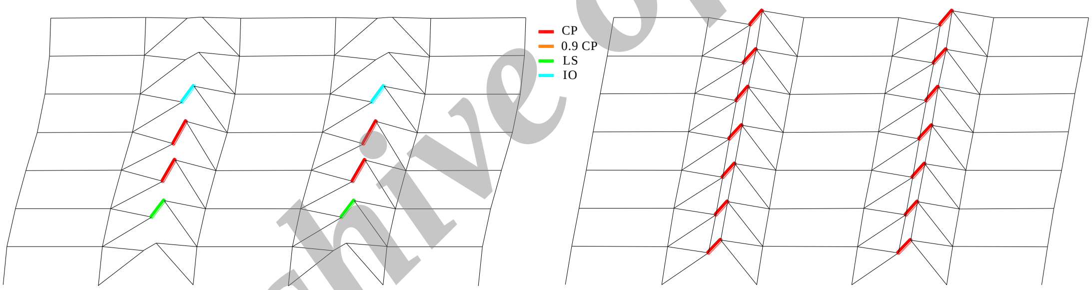
4.1 Formation of the plastic hinges in two systems

As noted before, link, as a structural fuse, can dissipate seismic input energy with formation of plastic hinge. Figs. 4 to 8 show formation of the plastic hinges in the links at target displacement for two systems. For example, in the case of a 3-story frame (Fig. 4) in the conventional EBF system, the fully-plastic hinge is formed in the second floor and only this story reaches Collapse Prevention (CP) limit criterion [20]. In the third floor, the plastic hinge is partially developed and does not reach the CP criterion. Also in the first floor, the plastic hinge has not formed and the shear link remains elastic. As a result, a large deformation is observed in the second story while the small deformation is obtained in the others without using the full capacity of the structure. On the other hand, for the TBF in corresponding frame, fully- plastic hinges are formed in three stories. This is unlike in the conventional EBF which concentrates the damage in a few floors. The TBF distributes the damage along the height of the frame in three stories and almost all links reaches the CP criterion. In addition, the assessment of deformed geometry and plastic hinges formation in other case studies show that this rule also applies. Shear link participation in energy dissipation for frames with zipper members is noticeably higher than that of conventional EBFs. In the TBF configuration, almost the shear links of all stories enter the plastic zone while in the conventional EBFs, plasticization and failure mode are concentrated only in a few stories and a limited number of these stories reach the collapse prevention level. More detailed information on the formation of plastic hinges in the two systems is presented in Figs. 4 to 8.



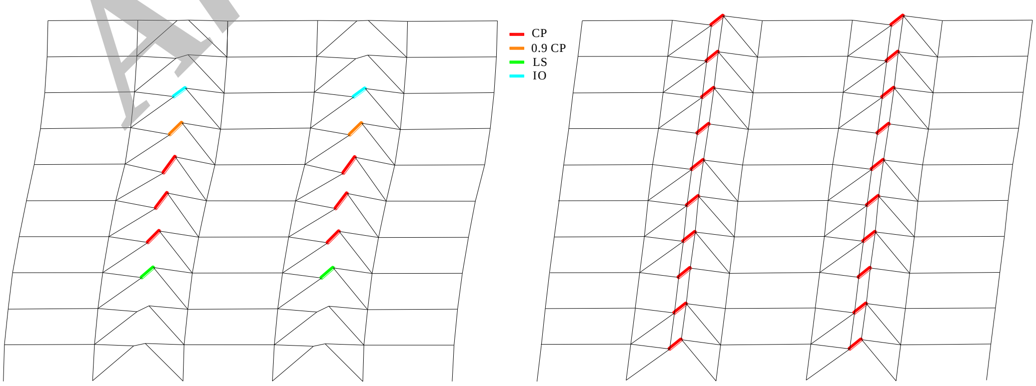
(a) EBF (b) TBF

Figure 4. Formation of plastic hinges and performance levels of 3-story frames



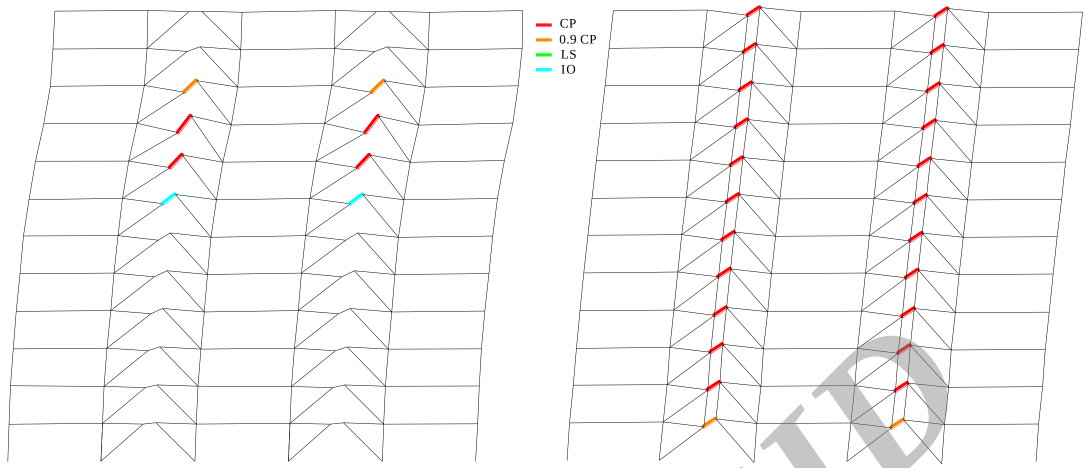
(a) EBF (b) TBF

Figure 5. Formation of plastic hinges and performance levels of 7-story frames



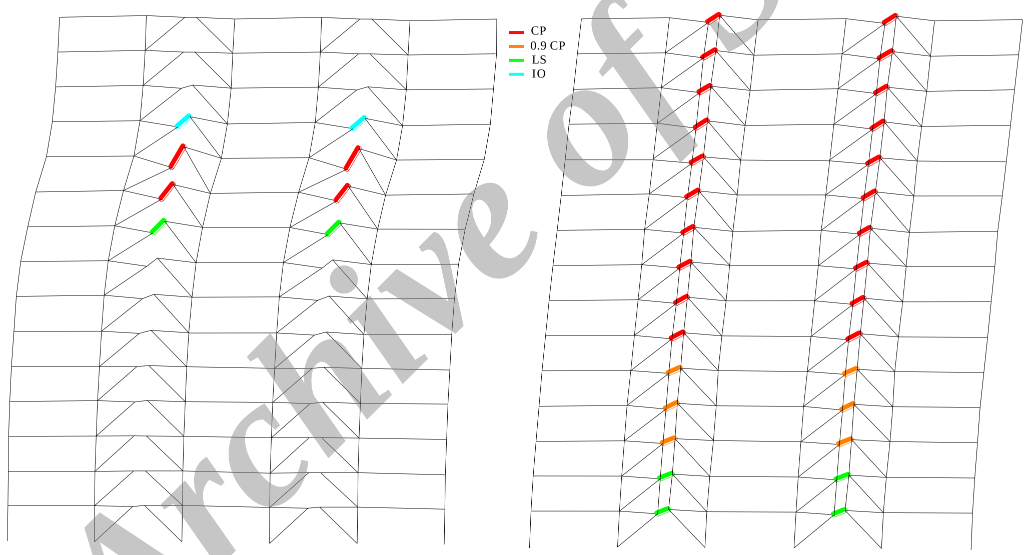
(a) EBF (b) TBF

Figure 6. Formation of plastic hinges and performance levels of 10-story frames



(a) EBF (b) TBF

Figure 7. Formation of plastic hinges and performance levels of 12-story frames



(a) EBF (b) TBF

Figure 8. Formation of plastic hinges and performance levels of 15-story frames

4.2 The ductility and energy dissipation evaluation

The ductility coefficient for two systems are calculated as the ratio of maximum lateral displacement ($\Delta_{\max}$) to yield lateral displacement (Δ_y).

$$\mu_s = \frac{\Delta_{\max}}{\Delta_y} \tag{2}$$

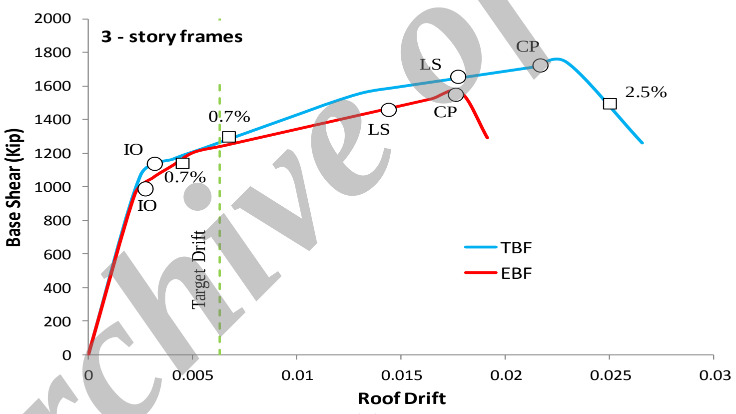
$\Delta_{\max}$ and Δ_y are obtained from pushover curves (see Figs. 9a-9e).

The obtained values of ductility factor for the two systems are presented in Table 4. As shown in this table and also in Fig. 9a, in the case of a 3-story frame, the ductility of the modified system is increased by about 40% compared with the corresponding value in the conventional EBF. The rate of increase in ductility for the case of 7, 10 and 12-story frames are around 60, 50 and 70%, respectively and particularly for the 15- story, it is more than 200%.

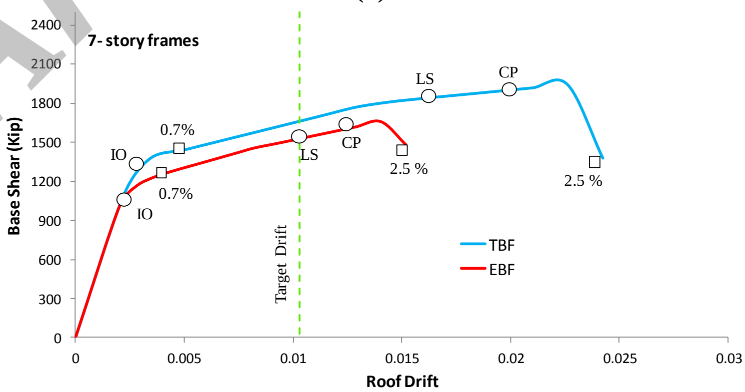
Also the values of energy dissipation in the plastic hinges of two systems are listed in Table 5. As seen from this table, the energy dissipation of the TBFs configuration in some cases such as15-story frame is tripled.

These observations highlight some important points: Firstly, the ductility of the TBF configuration is remarkably higher than that of the conventional EBF, demonstrating that the TBF is successful in energy dissipation capacity. The second observation is that an increase in height leads to a noticeable increase in the ductility of the TBF in comparison with conventional EBF.

Also the base shear capacity of the frames which was obtained through pushover analysis is shown in Table 6. For instance, the percentage of increase in the base shear for the 15-story frame is about 40%. Hence, as anticipated, the TBF provides more capacity for the base shear in all case studies compared with those of the EBFs.

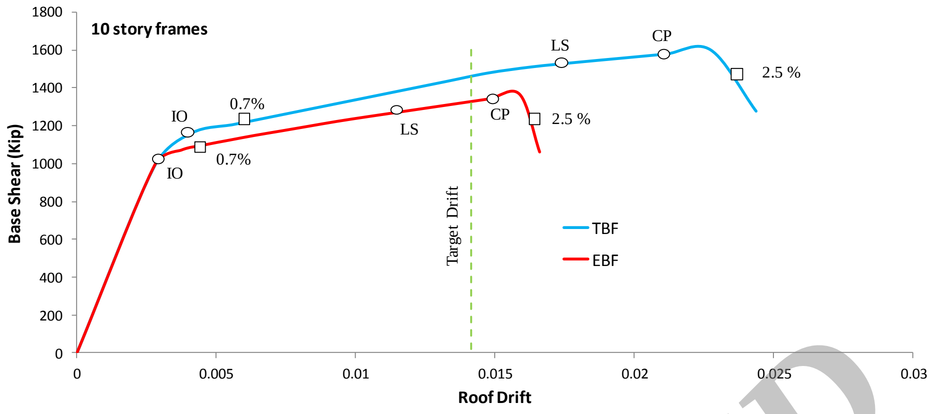


(a)



(b)

Figure 9. Pushover curves for TBFs in comparison with conventional EBFs



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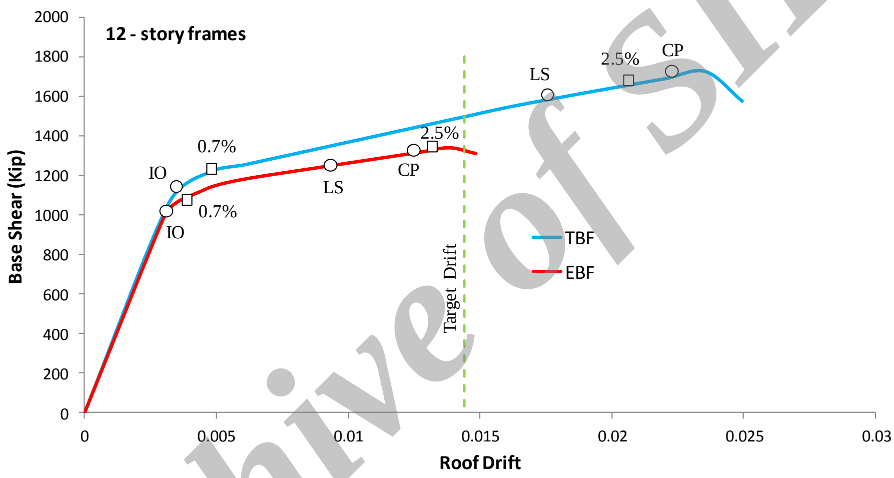

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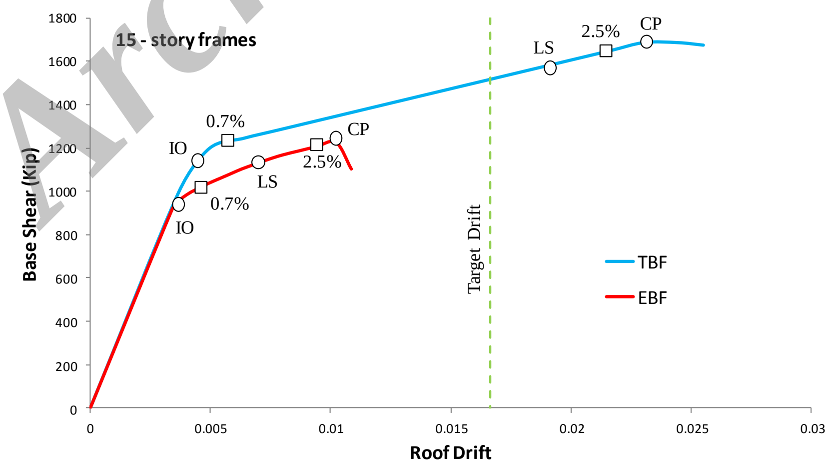


(e)

Figure 9. Pushover curves for TBFs in comparison with conventional EBFs (continued)

Table 4: Ductility coefficient for two systems

system	3-story	7-story	10-story	12-story	15-story
EBF	7.6	5.36	5.93	4.93	2.7
TBF	10.8	8.68	8.71	8.33	6.89
Ratio	1.42	1.62	1.47	1.68	2.55

Table 5: Energy dissipation (kip-in) in plastic hinges for two systems

system	3-story	7-story	10-story	12-story	15-story
EBF	6203.84	12951.61	17743.29	15970.94	10574.88
TBF	8944	21821.8	26065.46	32199.8	38251.52

Table 6: Base shear capacity (kips) for two systems

system	3-story	7-story	10-story	12-story	15-story
EBF	1600	1700	1400	1300	1200
TBF	1800	2000	1600	1700	1700

4.3 Drift and displacement evaluation

The drifts of the nonlinear static analysis are assessed based on the acceptance criteria of FEMA-356 [20]. As previously mentioned, frames were subjected to pushover analysis and the target displacements of two systems were approximately the same (Table 2). The base shear versus roof drift relationships for all models is illustrated in Figs. 9a to 9e. Valuable information is obtained through these figures, such as drift values due to formation of the first plastic hinge related to immediate occupancy, life safety and collapse prevention limits of acceptance criteria [20] are shown by Points IO, LS and CP, respectively. For example, in the 15-story building (Fig. 9e) for the TBF, the ultimate drift of the top story at the CP acceptance criterion is almost 50% higher than that of the conventional EBF. As observed in all cases, the points IO in the two systems are almost adjacent, but for the TBF, point LS and particularly point CP are considerably apart compared with those of the conventional EBF. The issue reveals the fact that the TBF reaches the acceptance criteria levels of FEMA356 [20] later than the conventional EBF. It means that the system experiences larger displacements in order to reach these levels without suffering any damage.

Besides, 0.7 and 2.5% drift levels were selected for damage control based on the FEMA 356 [20]. Evaluation of drift criterion shows that for instance, in the 3-story frame (Fig. 9a), the TBF model achieves a 2.5% drift while that of the conventional EBF reaches 0.7% and fails before 2.5% drift. As can be observed from Figs. 9b to 9e, it is also clear that in other cases, the conventional EBFs reach these criteria earlier and with less energy dissipation in comparison with the TBF configuration.

Results shown in Figs. 9a to 9e, illustrate that the conventional EBFs in comparison with the new system reach premature collapse prior to attaining the target drift.

Other important results for the seismic assessment of the structures are the distribution of inter-story drifts over the building height. The peak inter-story drift profiles for both systems are presented in Fig. 10. As anticipated, in the TBFs, the inter-story drifts are distributed uniformly over the building height while scatter values in a wide range of the height are seen in the conventional EBFs.

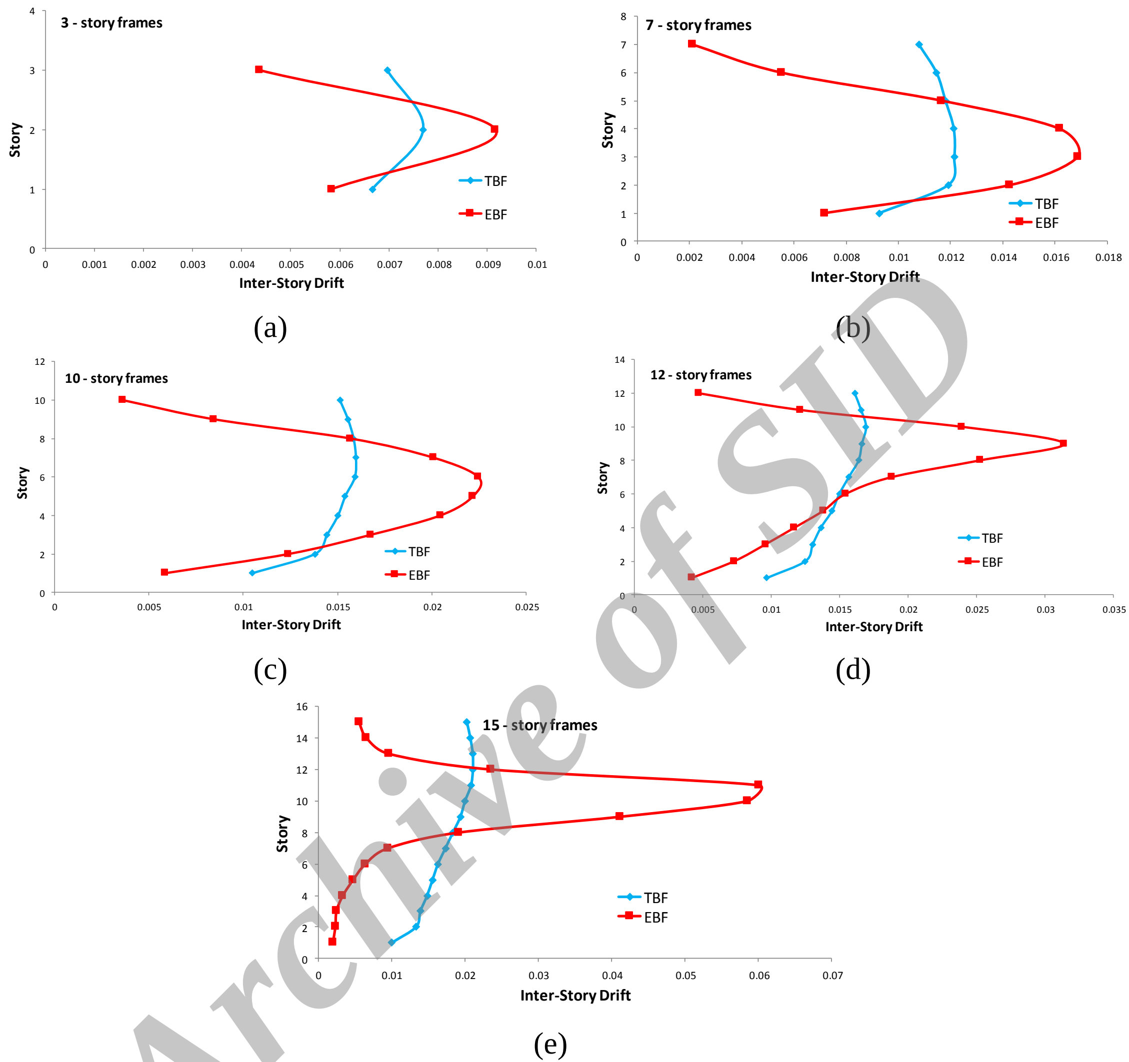
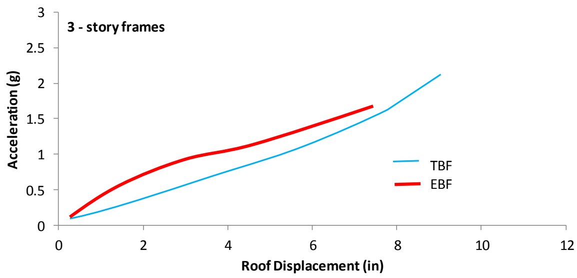


Figure 10. Inter-story drifts for TBFs in comparison with conventional EBFs

4.4 Evaluation of sustainable acceleration

In order to verify the results obtained from static analysis, IDA procedure is also performed. Figs. 11a to11e illustrate the sustainable acceleration curves for five case studies, which are obtained by IDA. These curves present the mean acceleration of earthquake records versus roof displacement. As can be seen from these figures and also from Table 7, in a 7-story frame (Fig. 11b), the TBF could resist 2.8 g at peak acceleration while corresponding frame in the conventional EBF could withstand about 1.4 g. The same rule governs other cases.

It can be inferred that the acceleration and displacement values of the TBFs in all cases are larger than those of the conventional EBFs. This proves that in the dynamic analysis, the TBF not only has more ductility and energy absorption capacity, but also has the capability to withstand severe acceleration.



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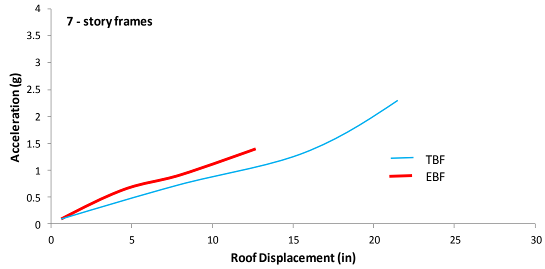

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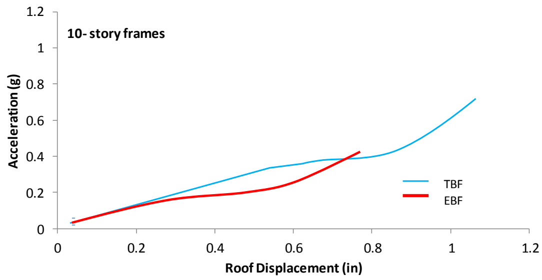

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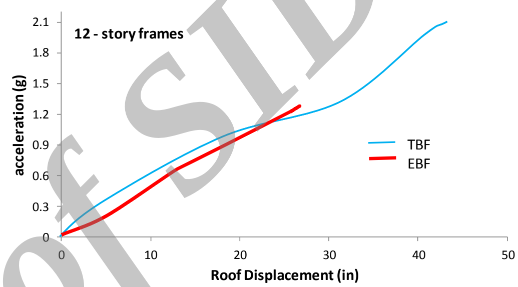

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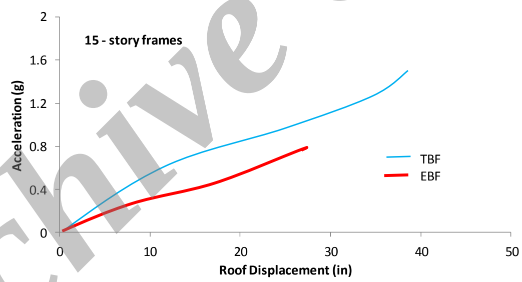

(e)

Figure 11. Sustainable acceleration for TBFs in comparison with conventional EBFs

Table 7: Sustainable acceleration (g) for two systems.

System	3-story	7-story	10-story	12-story	15-story
EBF	1.6	1.4	1.0	1.3	o.75
TBF	2.1	2.8	1.8	2.1	1.3

5. CONCLUTIONS

This paper evaluates comprehensively seismic inelastic demand of EBFs with vertical members and attempts to provide solutions for energy dissipation potential in more members of the structure. A number of 2-D EBFs with different number of stories were chosen to investigate the efficiency of the vertical members. The results of the pushover and incremental dynamic analyses showed that seismic performance of the TBF configuration was noticeably improved in comparison with the conventional EBFs for all models. The main results obtained in this study are summarized as follows:

1. The zipper-struts play an important role in better distribution of the plastic hinges from the lowest story to the top with more energy concentration in the shear links while the conventional EBFs concentrate damage only in a few stories.
2. Adding the zipper-struts to the system, leads to excellent ductility and energy absorption capacity in all models. Furthermore, the TBF provides ductile responses in all stories so that each story participates in the overall failure of structure and prevents the formation of soft-story mechanism. The effects of height on the TBF configuration were also evaluated in this study. It was found that an increase in height leads to more increment in the ductility percentage of the TBFs rather than the conventional EBFs. Moreover, according to the pushover analysis in the modified system, the base shear capacity of the structure increases before an overall failure occurs. This is due to nonlinear responses of all stories.
3. The pushover analysis shows that adding vertical members at the end of the link leads to coherent motion of stories and uniform distribution of inter-story drifts over building height.
4. The target displacements of two systems are almost the same. While the TBF exhibits more displacements and falls farther than the target displacement, the conventional EBF in most cases collapses before reaching this displacement.
5. Sustainable accelerations obtained from IDA indicate that the TBFs survive more earthquake acceleration compared with the conventional EBFs.

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