

Applications of vertical steel pipe dampers for seismic response reduction to steel moment frames

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ABSTRACT

A newly developed vertical steel pipe damper is introduced to improve the seismic performance of steel frames. The damper exhibits large lateral stiffness and excellent capability to dissipate energy due to earthquakes. It provides a reliable, compact, inexpensive, and replaceable damper. Improved performance of the structure is verified analytically using a four-story steel moment frame equipped with steel pipe dampers. Vertical steel pipe dampers are placed between any two points where large relative motion exists during earthquake excitation. A nonlinear dynamic analysis of the structure using PERFORM-3D software demonstrated the significant benefit of equipping the structure with steel pipe dampers. All structural components, except the steel pipe dampers, remain elastic during earthquake excitation. Structures properly designed with vertical steel pipe dampers will only require minimum post-earthquake inspection and limited damage. Some practical issues associated with the application of vertical steel pipe dampers to building structure for seismic response reduction are presented in this paper.

Keywords: Energy dissipation, seismic response, pipe damper

1. INTRODUCTION

A newly developed vertical steel pipe damper is introduced to improve the seismic performance of steel frames. The damper has large lateral stiffness, small yield displacement and excellent capability to dissipate a tremendous amount energy. The damper is intended to be installed in low to medium rise structures. Improved performance of a four-story steel moment frame with and without steel pipe dampers was verified analytically using north-south components of the El-Centro 1940, Chi-Chi 1999, Fukushima-Hamadori 2011 and Padang 2009 earthquakes accelerograms. The structure was not intended to be designed for a specific site. Therefore no spectral matching, to obtain design ground motion time histories to match a target response spectra of a site, was done. The results of the nonlinear dynamic analysis of the structure demonstrated the significant benefit of equipping the structure with vertical steel pipe dampers. All structural components, except the dampers, remain elastic during four strong earthquake excitations.

2. VERTICAL STEEL PIPE DAMPER

Steel pipes in vertical position are sensitive to the diameter (D) to thickness ratio (t) of the pipes. The schedule 80 carbon steel pipe which has small D/t ratio was chosen to avoid buckling in middle part of the pipe. Pipe having $D = 114.3$ mm, $t = 8.6$ mm and height (h) = 200 mm was used as the material of the damper. Abebe et al. [1] proposed the ratio of height to diameter of the pipe as $\sqrt{3}$ so that bending and shear stress yield simultaneously. The results of simple tensile tests to obtain the material properties of the steel pipe and plate are shown in Table 1. Figure 1 shows the results of numerical simulation using ABAQUS [2] for bare pipe fixed at both ends subjected cycles of increased amplitude by $1 \times \delta_y$ (δ_y is the yield displacement of the damper) in each consecutive cycle. As shown in Figure 1, no local buckling happened at the middle part of the pipe but local buckling happened at the ends of the pipe manifesting in unstable hysteretic curve. The specimen of the bare pipe had been tested, and fracture occurred at the heat affected zone area (HAZ) close to the ends of the pipe. Some kind of strengthenings to the bare pipe are required to avoid buckling at ends of the pipe, to relieve the stresses at the ends of the pipe, and to relocate the fracture away from HAZ area.

Figure 2 shows the two improved model of vertical steel pipe dampers. The model of vertical steel pipe dampers shown in Figure 2a had been tested, and fracture occurred at the heat affected zone area (HAZ) close to the ends of the pipe. The details of the plate strengtheners outside the pipe were improved so that: (1) buckling at the pipe was eliminated; (2) connection failure at the ends of the pipe was avoided; (3) fractures at HAZ region were avoided; (4) early fractures at points of high intense stress were postponed, and (5) extensive yielding was concentrated in the middle part of the pipe. As shown in Figure 2b, curved and tapered plate strengtheners were used to prevent buckling at the ends of the pipe, to concentrate the extensive yielding in the middle part of the pipe and to postpone and to shift fracture locations away from HAZ areas.

Table 1. Material properties of the steel used for numerical simulation.

Steel	Modulus of elasticity (MPa)	Yield stress (MPa)	Ultimate stress (MPa)	Breaking strain (%)
Pipe	200.000	330	465	37
Plate	200.000	360	500	25

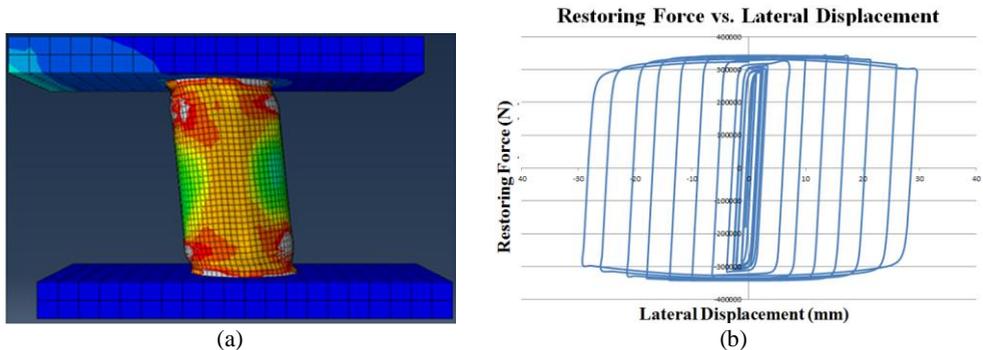


Fig. 1. Local buckling at the ends of the bare pipe: (a) Bare pipe and (b) Hysteretic curve

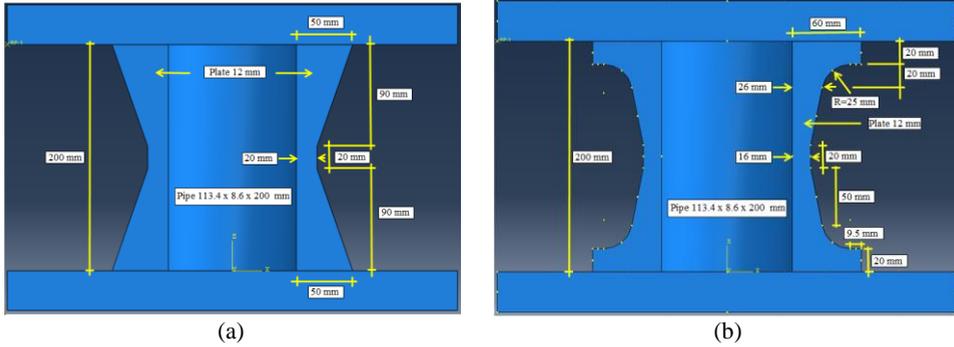


Fig. 2. Details of the model of vertical steel pipe dampers: (a) Strengthened with a pair of trapezoidal shape plates and (b) Strengthened with a pair of curve shape plates.

Figure 3 shows the von Mises stress distribution of the improved model shown in Figure 2b. The results of the numerical simulation shows: (1) The point of high intense stresses where fractures are expected to happened has been shifted away from HAZ region near the ends of the pipe; (2) Welded connections are placed at low stress areas, and (3) The high stresses at the ends of the pipe are relieved. The improved model of the vertical steel pipe damper has been tested using ATC-24 loading protocol using cycles of increased amplitude by $1x\delta_y$ in each of three consecutive cycles.

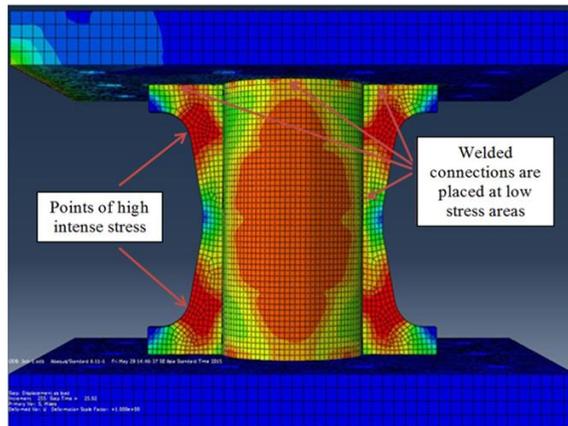


Fig. 3. von Mises stress distribution of the improved model

The stable parts of the hysteretic curves of the test results of the dampers were plotted in Figure 4. Figure 4a shows the multiple plots of the bare pipe damper and the vertical pipe dampers strengthened with a pair of trapezoidal shape plates on one graph, and Figure 4b shows the multiple plots of the bare pipe damper and the vertical pipe dampers strengthened with a pair of curve shape plates on one graph. The number and distribution of plastic cycles determined the energy dissipation capacity of the dampers. Table 2 shows the number of plastic cycles and the energy dissipated by each of damper due to the following cyclic displacement loadings: (1) For the bare pipe and the pipe strengthened with a pair of trapezoidal shape plates, the displacement loading amplitude was increased in each consecutive cycle by $1x\delta_y$, and (2) For the pipe strengthened with a pair of curve shape plates, the displacement amplitude was increased in each three consecutive cycles by $1x\delta_y$.

(ATC-24). The number of plastic cycles from the test results reflects the quality of the detailing of the dampers. Applying smooth geometry configuration to the plate strengtheners outside the pipe manifested in the increase capacity of the energy dissipation of the damper significantly.

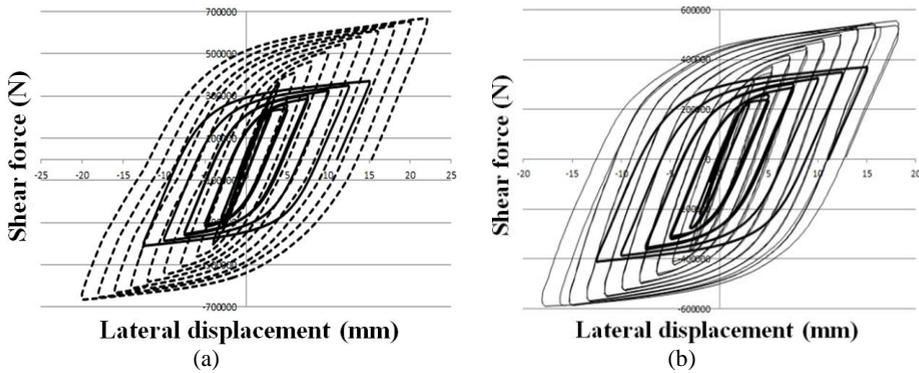
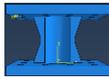


Fig.4. Plot of stable parts of hysteretic curves of vertical steel pipe dampers: (1) Bare pipe vs. pipe strengthened with a pair of trapezoidal shape plates, and (2) Bare pipe vs. pipe strengthened with a pair of curve shape plates.

Table 2. Number of plastic cycles and increase capacity of energy dissipation.

Vertical steel pipe dampers	Number of Plastic Cycles	Total Dissipated strain energy (N.mm)	Increase capacity of energy dissipation (%)
	7	0.325e+08	-
	8	1.09e+08	236.72
	29	2.45e+08	654.08

3. APPLICATIONS OF VERTICAL STEEL PIPE DAMPERS

The vertical steel pipe damper strengthened with a pair curve shape plates will be installed in a four-story steel moment frame to reduce the seismic response of the building due to four strong earthquakes. Nonlinear dynamic procedure shown in Figure 5 was used to access the performance capability of the building. Limiting the lateral deflection of the damper to $10 \times \delta_y$ ($\delta_y = 1.80$ mm is the yield lateral displacement of the damper) due to four strong earthquakes was selected as the performance objective of the building. A preliminary designed to estimate the required number of dampers in each story of the building was done. A computational model of the building that incorporate the nonlinear load-deformation characteristic of the individual damper was built using PERFORM-3D [3]. The computational model was then subjected to the north-south component of the Chi-Chi 1999,

El-Centro 1940, Fukushima-Hamadori 2011, and Padang 2009 ground motion time-histories. The resulting maximum absolute deflection of the dampers was directly compared to the performance objective of the building.

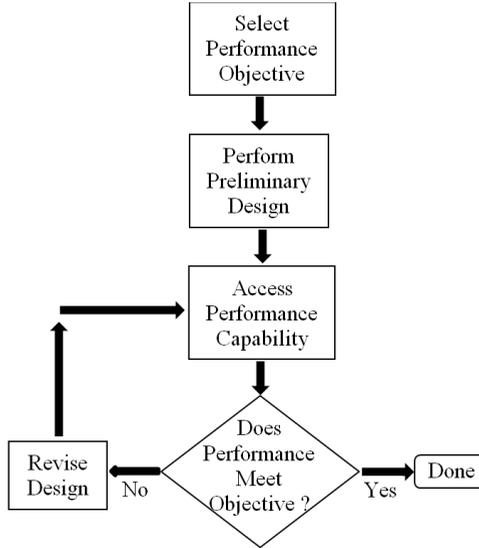


Fig. 5. Nonlinear dynamic procedure

3.1. Steel moment resisting frame

A four-story steel moment resisting frame shown in Figure 6 was used to study the application of the vertical steel pipe damper. A number of end releases were applied so that the lateral load resisting systems in the building are the peripheral frames. One-way slab systems were applied to all floors. The secondary beams were not shown in Figure 6.

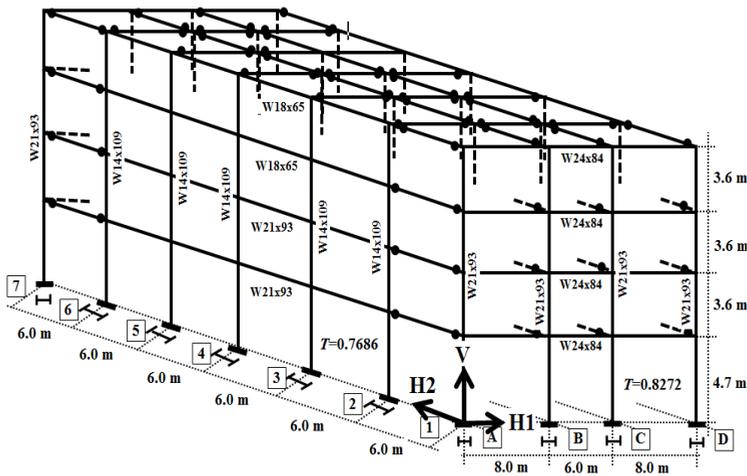


Fig. 6. Four-story steel moment resisting frame.

The superimposed specified loads are

Gravity loading:

- Live load: Roof LL = 1.0 kPa, Floor LL = 2.4 kPa
- Dead load: Floor weight plus topping = 3.6 kPa, Partitions = 1 kPa

Materials:

- Concrete: 24.0 MPa
- Reinforcing steel: ASTM Grade 60 $f_y = 415$ MPa

Elements dimension:

- Exterior columns: W21x93 (along H1), and W14x109 (along H2)
- Interior columns: W12x72 (along H2)
- Exterior beams: W24x84 (along H1), W21x93 and W18x95 (along H2)
- Interior beams: W24x84 (along H1), W21x93 and W18x95 (along H2)

Fundamental periods:

- $T = 0.8272$ seconds (along H1)
- $T = 0.7689$ seconds (along H2)

Originally all the columns height were 3.6 m and the four-story steel moment frames had been designed to withstand moderate earthquake. The height of the first floor column was modified to 4.6 m and the double plates at panel zones were eliminated to purposely create problems into the four-story steel moment frames. These problems will be eliminated by installing dampers in the peripheral frames of the buildings.

3.2. Modeling Damper as Inelastic Component

Test results data of the vertical steel pipe damper were used to model shear vs. lateral displacement of the damper. Seismic isolator component in PERFORM-3D [3] was used to model the damper as shown in Figure 7.

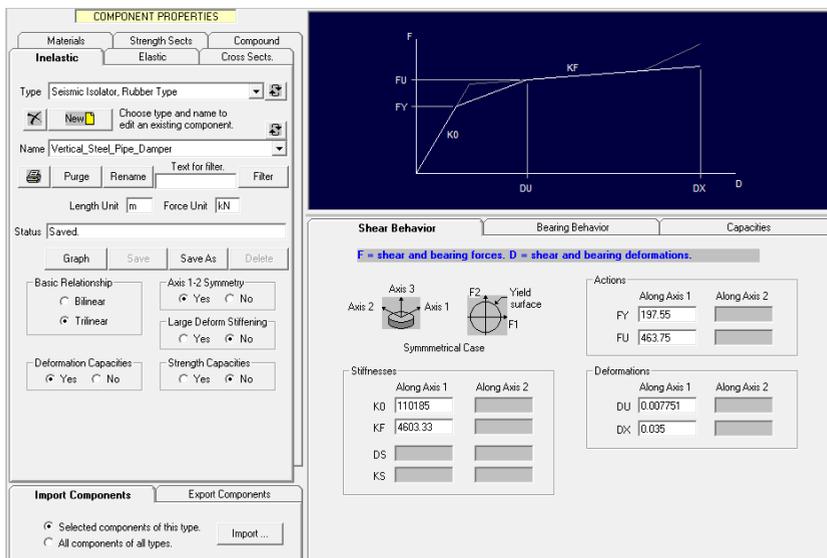


Fig. 7. Trilinear model of the force vs. displacement of the damper

The performance objectives of the steel frames equipped with dampers are:

1. Energy dissipation is concentrated in dampers meaning other components of the steel frames, except the dampers, remain elastic.
2. Interstory drifts reduction at all level are significant
3. Absolute lateral deflection of the dampers is less than $10 \times \delta_y$.

3.3. Estimating the Number of Dampers in Each Story

The required number of dampers in each story had been estimated using energy-based method proposed by Benavent-Climent [4]. The required lateral stiffness and lateral strength of the damper for near-fault ground motion were determined from input energy spectra for moderate-seismicity regions proposed by Benavent-Climent et al. [5]. The configuration of the dampers in the peripheral frames along H1 and H2 are shown in Figure 8. Auxilliary structures in the form of triangular bracings are needed to intall dampers between two points where large relative motion exists during earthquake. Auxilliary structures shown in Figure 8 were chosen to minimize the influence of axial forces to the dampers.

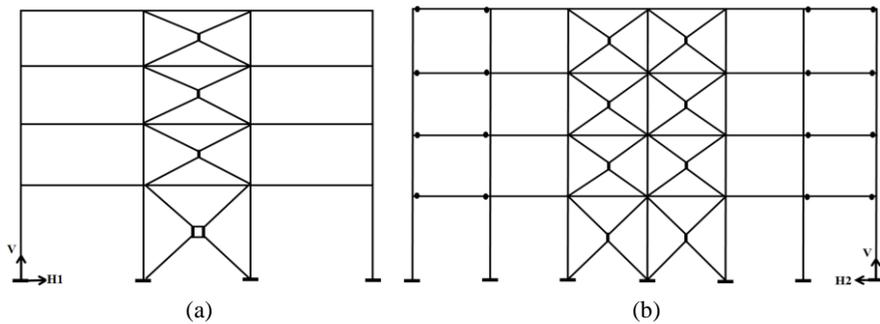


Fig. 8. Configuration of the dampers in peripheral frames: (a) Two peripheral frames along H1, and (b) Two peripheral frames along H2.

A computational model of the four-story frames that incorporate the nonlinear load-deformation characteristic of the individual damper shown in Figure 7 was built using PERFORM-3D [3]. For assesment of the performance of the building equipped with dampers, the computational model was then subjected to the north-south of the four strong earthquakes time histories, and the results were evaluated.

3.4. Combined Acceleration Spectrum

The Peak Ground Accelerations (PGAs) of the Chi-Chi 1999, Fukushima-Hamadori 2011 and Padang 2009 were scaled down to El-Centro 1940. Figure 9 shows he combined acceleration spectra of the four earthquake time-histories. Spectral matching would reduce the peaks and valleys in each earthquake time history. No spectral matching was done because the four-story steel moment resisting frame was not designed for specific site. Therefore peaks and valleys in Figure 9 of each acceleration spectra are very obvious.

Two cases of the peripheral frames along H1 considered:

1. Case 0 is the four-story frame without dampers ($T = 0.8272$ seconds)
2. Case 1 is the four-story frame with dampers ($T = 0.4918$ seconds)

It is well understood that each point in a response spectra represents the energy content of the earthquake at a certain frequency. For the frame without damper along H1 subjected to Padang earthquake accelerograms, the corresponding point for $T=0.8271$ seconds lies in a valley. Therefore significant response reduction is not expected for Padang earthquake. Significant response reductions are expected for three other earthquakes.

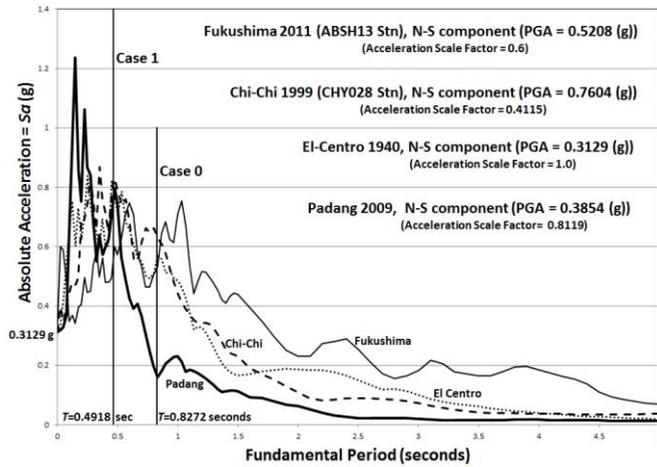


Fig. 9. Combined acceleration spectra

3.5. Interstory Drift Reduction

The results of nonlinear dynamic analysis of Case 0 and Case 1 were used to quantify the lateral displacement (drift) and interstory drift ratio of the four-story steel moment frame. The drift and interstory drift for Case 0 and Caser 1 are shown in Figure 10 and Figure 11 respectively.

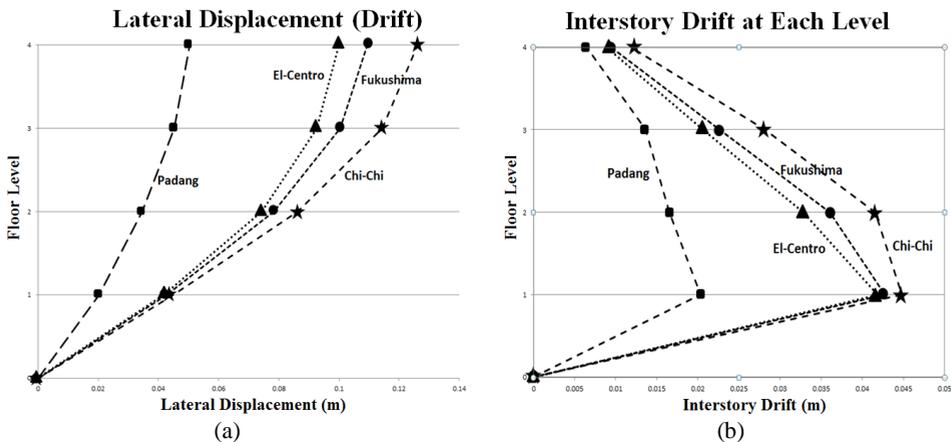


Fig. 10. Drift and interstory drift for Case 0: (a) Drift, and (b) Interstory drift.

Figure 10a shows the maximum drift at each floor of the four-story frame without damper. The maximum drift due to Padang 2009 earthquake is small as expected (see Figure 9). Figure 10b shows the maximum interstory drift at each floor of the four-story frame without damper.

Figure 11a shows the maximum drift at each floor of the four-story frame equipped with dampers. Figure 11b shows the maximum interstory drift at each floor of the four-story frame with dampers installed. Comparing Figure 10 and Figure 11, it can be seen that the lateral displacement and the interstory drift were reduced significantly due to the present of dampers in the four-story steel frame.

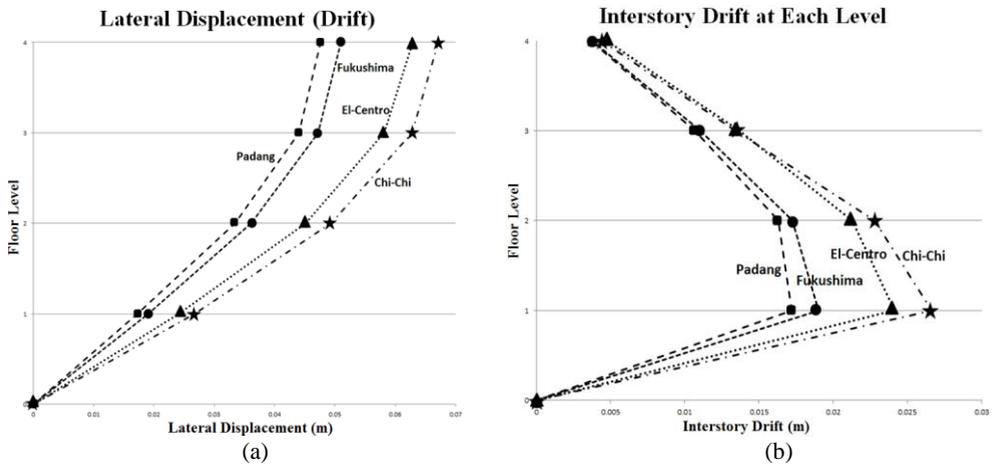


Fig. 11. Drift and interstory drift for Case 1: (a) Drift, and (b) Interstory drift.

The reduction of interstory drift at each floor due to the dampers is shown in Figure 12. Parameter β_i^1 [6] is used to quantify the ratio of maximum drift at each floor for the frame with dampers to the frame without dampers (Tovar and Lopez 2004). The interstory drift reduction is quantified as $1-\beta_i^1$. Except for Padang 2009 earthquake where the interstory drift reduction is expected to be small, the interstory drift reduction at each floor due to the other earthquakes is significant. On average the interstory drift reduction at each floor is about 40%.

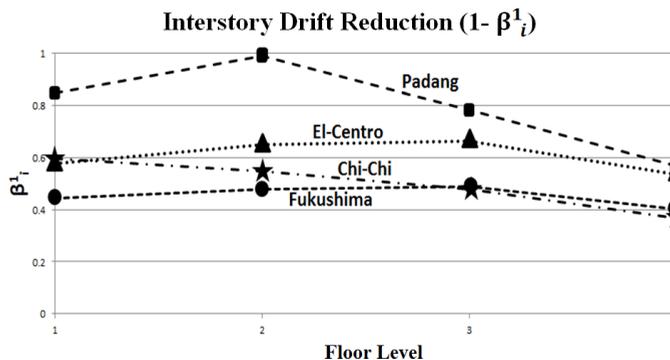


Fig. 12. Interstory drift reduction

3.6. Dissipated Inelastic Strain Energy

Dissipated strain energy quantified at each component of the four-story steel moment frame in nonlinear dynamic analysis is shown in Table 3. Only the vertical steel pipe dampers dissipate energy. All other components except the dampers remain elastic. The present of the dampers installed at strategic locations in the four-story steel moment frame were able to protect the structure against strong earthquakes. Extensive yielding in dampers is the energy dissipation mechanism. By dissipating strain energy, the dampers control the vibration of the structure during strong earthquakes.

Table 3. Dissipated strain energy at each component of the peripheral frames along H1.

Group Name	Earthquake			
	El-Centro - 1940	Fukushima - 2011	Padang - 2009	Chi Chi - 1999
Perimeter Columns	0	0	0	0
Perimeter Beams	0	0	0	0
Interior Columns	0	0	0	0
Interior Beams	0	0	0	0
Connection Panel Zones - along H1	0	0	0	0
Connection Panel Zones - along H2	0	0	0	0
Vertical Steel Pipe Dampers	261.49	422.4	202.35	233.29
Bracing HSS-H1-1st floor	0	0	0	0
Bracing HSS-H2-1st floor	0	0	0	0
Bracing HSS-H2-other floors	0	0	0	0
Bracing HSS-H1 other floors	0	0	0	0
Total for All Groups	261.49	422.4	202.35	233.29

The strain energy dissipated by individual damper is shown in Table 4. The largest value of strain energy dissipated by individual damper located between the 2nd and 3rd floor is due to Fukushima-Hamadori 2011 earthquake. The values of dissipated strain energy of individual damper reflect the damage (the degree of yielding) experienced by the dampers.

Table 4. Dissipated strain energy of individual damper along H1.

Earthquake	Dissipated Energy of Individual Damper (kN.m)			
	Damper btw 1 st and 2 nd floor	Damper btw 2 nd and 3 rd floor	Damper btw 3 rd and 4 th floor	Damper btw 4 th floor and roof
El-Centro	36.40	46.30	11.65	0.0
Fukushima	59.32	76.71	15.86	0.0
Padang	26.37	35.18	13.26	0.0
Chi-Chi	34.85	37.38	9.58	0.0

3.7. Maximum absolute shear force and shear displacement

The maximum absolute value of the shear force and the maximum absolute value of the lateral displacement shown in Table 5 were obtained from the histories of the shear force and lateral displacement after the nonlinear dynamic analysis was completed. The maximum absolute shear force is 513.21 kN which is less than the maximum shear force of the vertical steel pipe damper. The maximum absolute shear displacement is 18.5 mm which is very close to the $10\delta_y$ of the damper (δ_y of the damper is 18.00 mm). Therefore the performance objective of the structure has been achieved. Revising of the computational model of the building structure and rerun the nonlinear dynamic analysis are not necessary.

Table 5. Maximum absolute shear force and lateral displacement.

Earthquake	Maximum Absolute Shear Force (kN)		Maximum Absolute Shear Displacement (m)	
	Damper btw 1 st and 2 nd floor	Damper btw 2 nd and 3 rd floor	Damper btw 1 st and 2 nd floor	Damper btw 2 nd and 3 rd floor
El-Centro	502.63	501.88	0.0162	0.0161
Fukushima	480.78	484.38	0.0115	0.0122
Padang	470.01	480.17	0.0099	0.0113
Chi-Chi	513.21	508.34	0.0185	0.0175

4. CONCLUSION

The newly developed vertical steel pipe damper, which has sufficient lateral stiffness and excellent capability to dissipate a tremendous amount of energy due to strong earthquakes, was successfully applied to reduce the seismic response of the moment frame due to four strong earthquakes. The energy dissipation were concentrated in the dampers so that other components of the structure, except the dampers, remain elastic. The performance of the four-story steel moment frame with slender columns at the first floor has been improved significantly by installing dampers at strategic locations. After strong earthquakes, dampers that already experience yielding can be replaced easily. Structures properly designed with vertical steel pipe dampers will only require minimum post-earthquake inspection and experience limited damage.

Acknowledgment

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