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SEISMIC BEHAVIOR OF STEEL STRUCTURES EQUIPPED WITH CYLINDRICAL FRICTIONAL DAMPERS

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ABSTRACT

During severe seismic excitations, a large amount of kinetic energy is fed into a structure **.**In this investigation, seismic response of steel structures utilizing Cylindrical Frictional Dampers (CFD) is studied. CFD is an innovative frictional damper which comprises two principal elements, the shaft and the hollow cylinder. These two elements are assembled such that one is shrink-fitted inside the other. If the damper's axial force overcomes the static friction load, the shaft inside the cylinder will move and results in considerable mechanical energy absorption. To assess the efficacy of CFD, various steel frames are constructed and analyzed using OpenSees software. Nonlinear time history analyses and Incremental Dynamic Analysis (IDA) are applied to the frames and clear distinction has been drawn between the frames comprising CFD and the counterparts without CFD to emphasize the effectiveness of CFD in altering seismic responses. The results show that CFD extremely improves the seismic response of the structure

Keywords: Passive control; cylindrical frictional damper; incremental dynamic analysis; seismic response.

1. INTRODUCTION

During severe seismic excitations, a large amount of kinetic energy is fed into a structure. Structural engineers recognized that it is not economical to dissipate this seismic energy within the elastic capacity of the materials. As a consequence, it is a common design principle to accept some seismic damage in a building; however it is preferable to anticipate yielding in some controlled locations of the structure. In braced buildings, braces are primarily responsible for energy dissipation but buckling in compression results in sudden loss of stiffness and progressive degrading behavior which confines the amount of energy dissipation. Various innovative methods have been proposed to override this deficiency in

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steel braces. Arguably the most promising one is to use frictional damper within the bracing members.

Frictional devices dissipate energy through friction caused by two solid bodies sliding relative to each other. The idea of using frictional dampers was first proposed by Pall (1979). Pall and Marsh [1] proposed frictional dampers installed at the crossing joint of the X-brace. Tension in one of the braces forces the joint to slip thus activating four links, which in turn force the joint in the other brace to slip. This device is usually called the Pall frictional damper (PFD). B. Wu et al. [2] introduced improved Pall frictional damper (IPFD) which replicates the mechanical properties of the PFD, but offers some advantages in terms of ease of manufacture and assembly. Sumitomo friction damper [3] utilizes a more complicated design. The pre-compressed internal spring exert a force that is converted through the action of inner and outer wedges into a normal force on the friction pads. Fluor Daniel Inc., has developed and tested other type of friction device which is called Energy Dissipating Restraint (EDR) [4]. The design of this friction damper is similar to Sumitomo friction damper since this device also includes an internal spring and wedges encased in a steel cylinder. The EDR utilizes steel and bronze friction wedges to convert the axial spring force into normal pressure on the cylinder. A full description of the EDR mechanical is given in [5]. Constantine et al. [6] proposed frictional dampers composed of a sliding steel shaft and two frictional pads clamped by high strength bolts. Li and Reinhorn [7] verified the seismic performance of a reinforced concrete building with frictional dampers through a combined experimental and analytical study. Grigorian et al. [8] studied the energy dissipation effect of a joint with slotted holes both analytically and experimentally. Mualla and Belev [9] proposed a friction damping device and carried out tests for assessing the friction pad material. Cho and Kwon [10] proposed a wall-type friction damper in order to improve the seismic performance of the reinforced concrete structures. S.-H. Lee et al. [11] proposed a design methodology of friction damper–brace systems, to determine the quantity and slip load of the frictional damper and the brace stiffness systematically for an elastic multistory building structure based on the story shear forces. Recently Mirtaheri et.al. [12] proposed an innovative type of frictional damper called cylindrical friction damper (CFD). In contrast with other frictional dampers the CFDs do not use high-strength bolts to induce friction between contact surfaces. This reduces construction costs, simplifies design computations and increase reliability in comparison with other types of frictional dampers.

In this paper, passive control of steel structures utilizing cylindrical friction dampers (CFD) is investigated. Firstly, finite element models of a moment resisting and a braced steel frames are made using OpenSees software. Subsequently, nonlinear zero-length elements, with elastic–perfectly plastic behavior are used to model the CFD at the middle of bracing members. Nonlinear time history analyses and Incremental Dynamic Analysis (IDA) are applied to the frames with CFD and the frames without CFD, and the results were compared. The results show that CFD extremely improves the seismic response of the structure.

2. CYLINDRICAL FRICTIONAL DAMPERS

CFD comprises two principal elements, the shaft (Fig. 1a) and the hollow cylinder (Fig. 1b). At a predefined length called L0 the inside diameter of cylindrical element is slightly smaller than the diameter of the shaft. Taking advantage of thermal expansion and warming the cylindrical part its diameter will increase and the shaft can be easily placed into the cylinder. A longitudinal section of the CFD is shown in Fig. 1c. Manufactured shaft and cylinder and assembled CFD are shown in Fig..2a and Fig.2b respectively. Reaching thermal equilibrium, the contact pressure will be developed between the outer surface of the shaft and inner surface of the cylinder, which results in friction between these surfaces [12].

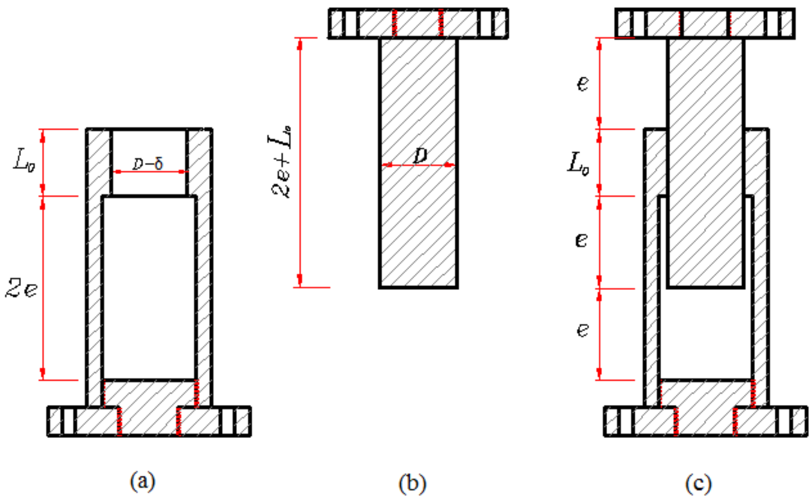
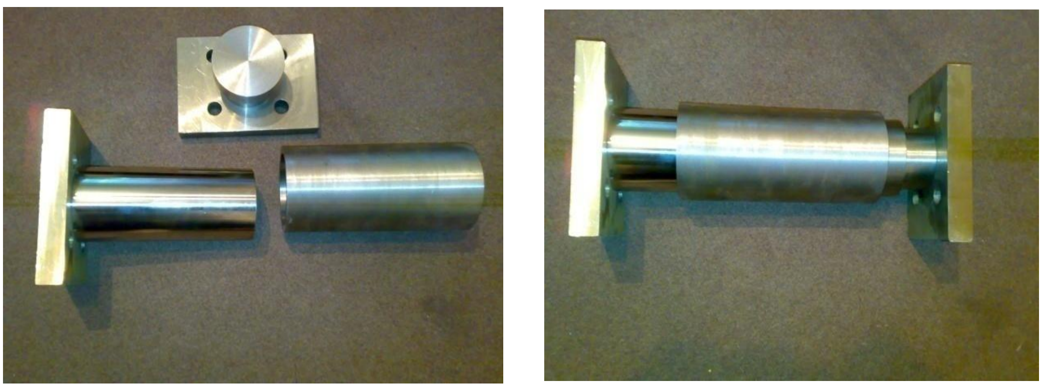


Figure 1. Main parts of CFD: (a) Tubular cylinder; (b) Solid shaft; (c) Longitudinal section of CFD



(a) (b)

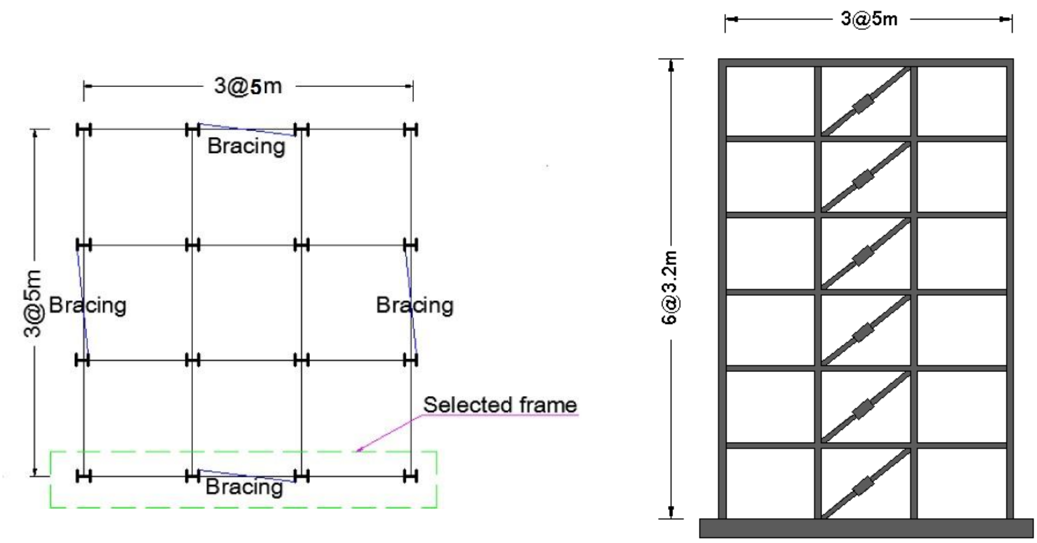
Figure 2. Manufactured CFD: (a) Manufactured shaft and cylinder; (b) Assembled CDF

The shaft will move inside the cylinder if the applied axial force to the damper overcomes the static friction load. This movement results in considerable mechanical energy absorption. The slippage load remains constant during the motion since the main parts are in contact at a certain constant length that is L0. [12].

3. EFFECT OF CYLINDRICAL FRICTION DAMPERS ON SEISMIC RESPONSE OF STEEL STRUCTURES

3.1 numerical modeling of Structures

To investigate the effectiveness of CFDs in a real building, two analytical models are made and studied comparatively. First, the two-dimensional model of Fig. 3 is modeled using OpenSees software. Beams and columns are modeled using force-based nonlinear fiber beam–column elements with five integration points along their length. The element cross-section is discretized into uni-axial fibers. Column bases have been fully fixed. Gravity loads are supposed to be similar to common residential buildings in the region. CFD dampers are added to the model subsequently. Nonlinear zero-length elements, with elastic–perfectly plastic behavior are used to model the CFD at the middle of bracing members. The framing members are designed according to AISC seismic provisions for seismic zone 2 with a response factor of 6 in allowable stress design.



(a) (b)

Figure 3. 6-storey frame(a)Plan of the buildings and (b) elevation

3.2 Record selection

Four earthquake records which belong to a bin of distances of 50 to 150 km, bearing no mark of directivity are used to determine the response modification factor of the frames. Specifications of selected earthquake records are shown in Table 1.

Table 1: Specification of selected Earthquake records

Earthquake	PGA (g)	Duration (s)	Magnitude
Elcentro (1940)	0.318	31.16	6.4
Kobe (1995)	0.599	47.98	6.9
Northridge (1994)	0.416	29.98	6.7
Tabas (1978)	0.836	32.82	7.4

3.3 Optimum slip load

The dissipated energy of a friction damped braced frame E_d is expressed as follows:

$$E_d = \sum_{i=1}^{n} E_{di} = \sum_{i=1}^{n} \int F_{si} |y_i| dt \tag{1}$$

where n is the total number of dampers, E_{di} and F_{si} are the dissipated energy and the slip load of the ith CFD and y_i is the displacement of the ith CFD. If the slip load of the CFD is too high (greater than buckling load of the bracing member in which the CFD is engaged) the dissipated energy is equal to zero since no slip occurs. In this case the frame behaves like a braced frame. On the other hand, if the slip load is too low, excessive slip occurs but due to small amount of slip load the dissipated energy is negligible. In this case the frame behaves like a moment resisting frame. Between these to limit states, one could find a slip load which result in the optimum energy dissipation. This slip load is called optimum slip load.

In order to find the optimum slip-load, various slip loads must be examined. As the first trial load, 80% of the buckling load of the brace member is selected as the CFD slip load. Subsequently, a parametric study is conducted and the slip load is bracketed until the minimum displacement of the top of the frame is reached. The result of parametric study for the 6-story frame is shown in Fig. 4

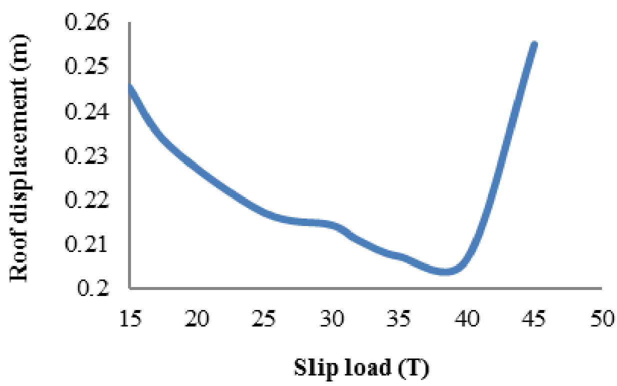


Figure 4. Optimum slip-load for the 6-story frame (Whitter narrows earthquake)

Optimum slip loads of the frame for ten different records are presented in Table 2.

Records belong to a bin of distances of 50 to 150 km, bearing no mark of directivity. The earthquakes are scaled to produce a peak ground acceleration of 1 g. As can be seen optimum slip load is completely dependent to external seismic load.

Table 2: Optimum slip load of earthquake records

Earthquake	Optimum slip load (kN)	Deviation from average (%)
Coalinga	750	0.47
Elcentro	620	0.22
Imperial valley	500	-0.02
Loma perita	420	-0.17
N.palm spring	240	-0.53
Northridge	550	0.08
Victoria,mexico	480	-0.06
Whitter narrows	400	-0.21
Kobe	430	-0.16
Tabas	700	0.38
Average	509	

In order to assess the sensitivity of the response of the frame to the selected slip load the following parameters are defined:

$$\alpha = 1 - \frac{F_s}{F_{so}} \tag{2}$$

$$\beta = \frac{R_{df}}{R_{dfo}} - 1 \tag{3}$$

$$\eta = 1 - \frac{R_{df}}{R_d} \tag{4}$$

where, F_s is the slip load, F_{so} is the optimum slip load, R_d is the displacement response of the frame without CFD, R_{df} is the displacement response of the frame with CFD and finally R_{dfo} is the displacement response of the frame utilizing CFDs with optimum slip load.

Fig. 5a shows β versus α for Victoria earthquake. As can be seen when $\alpha = -20\%$, that is the slip load is 20% less than optimum slip load, β is equal to 23% that is the maximum displacement response of the frame is increased 23% once compared to the maximum displacement response of the frame with optimum slip load. Fig. 5b shows η versus α. As can be seen when $\alpha = -20\%$, η is equal to 38%. In other words when the slip load is 20% less than optimum slip load, the maximum displacement response of the frame is decreased 38% once compared to the maximum displacement response of the frame without damper.

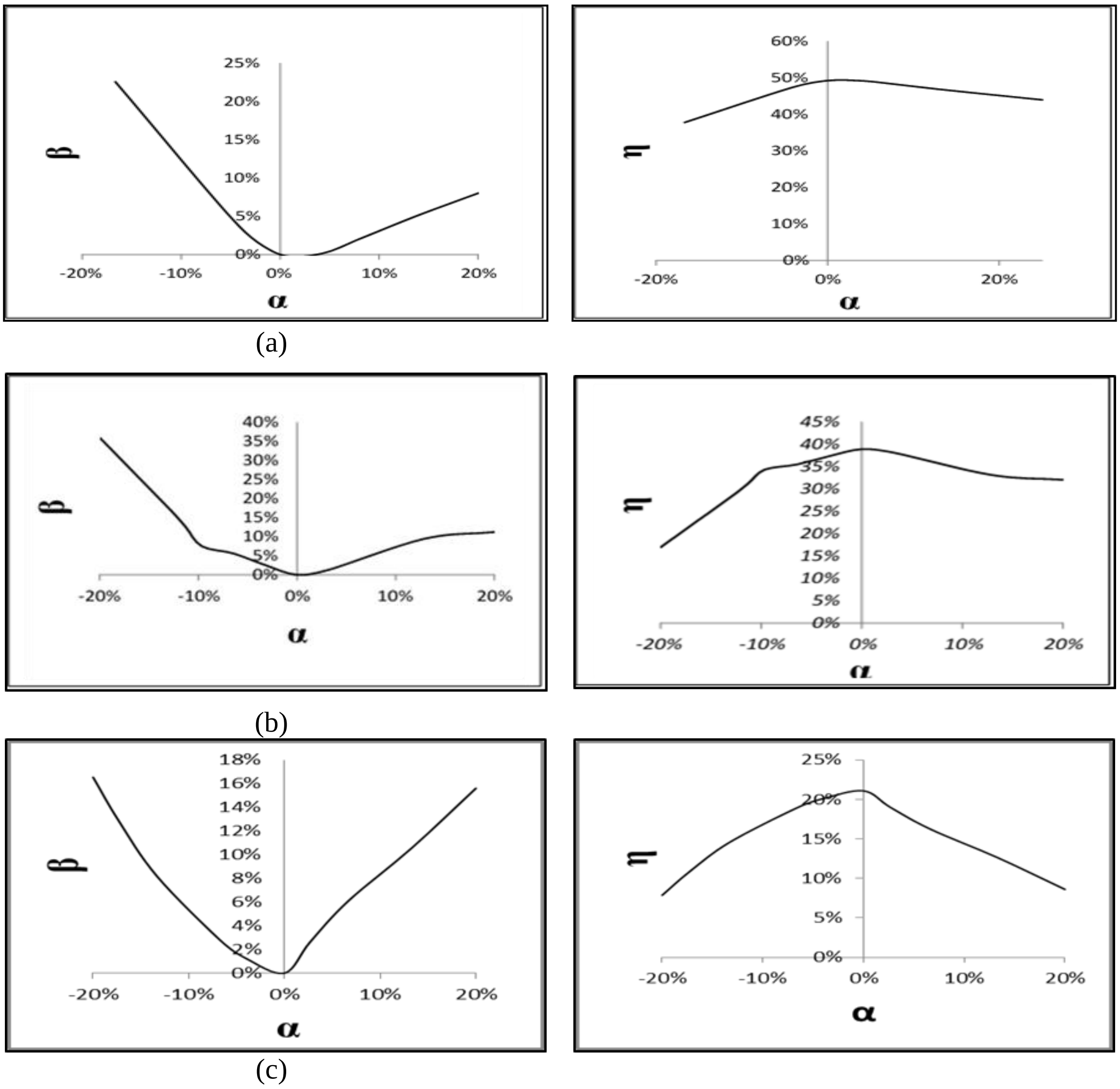


Figure 5. Sensitivity of the frame response to the selected slip load for (a) Victoria earthquake (b) Elcentro (c) Coalinga

The values of β and η for other ground motion records are presented in Table 3. As can be seen, the maximum value of β is 36% which is related to Elcentro earthquake. The average value for β is about 16% and 9% at $\alpha = -20\%$ and $\alpha = 20\%$ respectively.

3.4 Non-linear time history analysis

The response of the frame with and without dampers is compared using three of selected earthquake as shown in Table 4. Note that the records are not scaled this time and they are used as recorded.

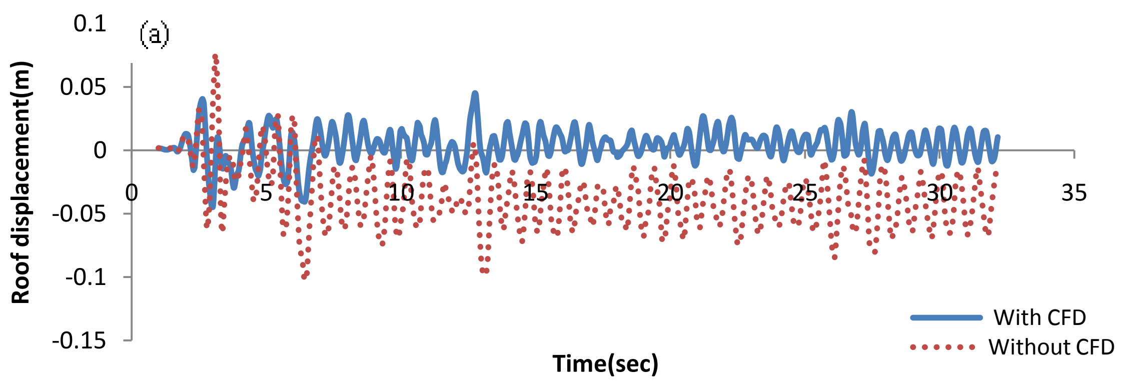
By using the CFD with a slip load of 600 kN, maximum displacement of the roof is reduced by 56% and maximum base shear is reduced by 72%. The peak responses of the frame for all earthquake records are shown in Table 4. The comparative plots of displacement, velocity and acceleration responses at the top of the frame and the base shear for Elcentro earthquake are shown in Fig. 6.

Table 3: Values of β and η at $\alpha = -20\%$ and $\alpha = -20\%$

Earthquake	$\alpha = -20\%$		$\alpha = 20\%$	
	β	η	β	η
Coalinga	16.54	7.87	15.61	8.62
Elcentro	35.96	16.98	11.22	32.09
Imperial valley	8.63	67.55	8.40	67.62
Loma perita	24.02	-6.42	6.11	5.62
N.palm spring	1.73	50.42	4.36	49.13
Northridge	5.14	3.55	3.55	5.01
Victoria,mexico	22.58	37.82	10.36	44.01
Whitter narrows	20.38	9.39	10.54	16.08
Kobe	17.35	22.01	14.26	27.46
Tabas	12.35	27.43	8.42	33.58
Average	16.47	23.66	9.28	28.92

Table 4: Peak responses of the frame

Earthquake	PGA	Duration (sec)	Maximum displacement at the top of the frame (m)			Maximum base shear (kN)		
			Without CFD	With CFD	Reduction (%)	Without CFD	With CFD	Reduction (%)
Elcentro	31.16	0.318	0.1026	0.0451	56	1367.13	377.64	72
Kobe	47.98	0.599	0.4654	0.1769	62	1621.20	809.05	50
Tabas	32.82	0.836	0.1342	0.1204	10	1390.64	998.46	28



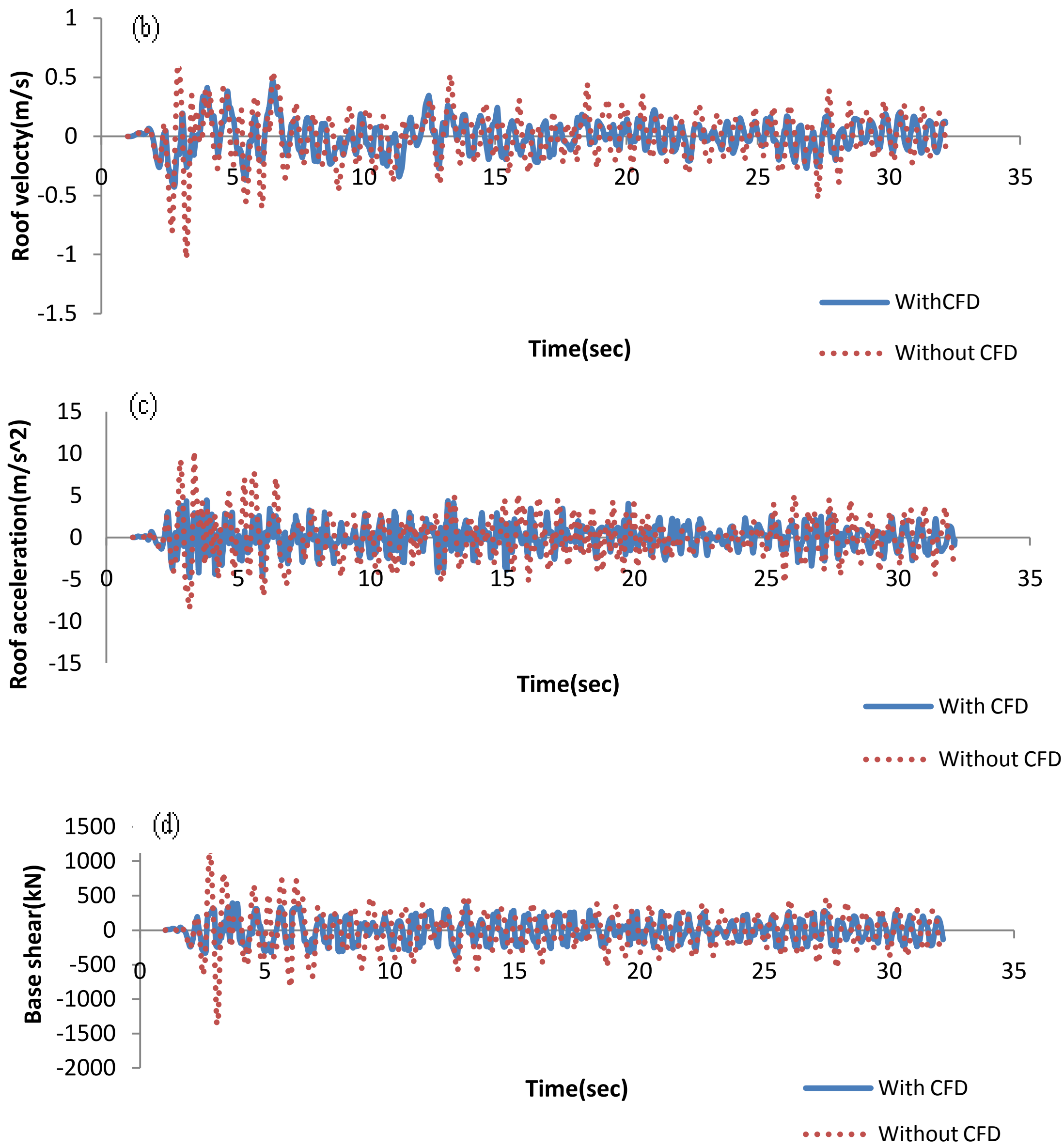


Figure 6. Comparative plots for Elcentro earthquake a)displacement b) velocity c) acceleration responses at the top of the frame d) base shear

3.5 Incremental dynamic analysis

To investigate the performance of the frame more thoroughly under seismic loads with various intensities, Incremental Dynamic Analysis (IDA) is applied to the frame. The comparative plot of the IDA curves of the frame for Caolinga earthquake is shown in Fig. 7. As can be seen, CFD significantly improves the performance of frame subjected to earthquake loads [13].

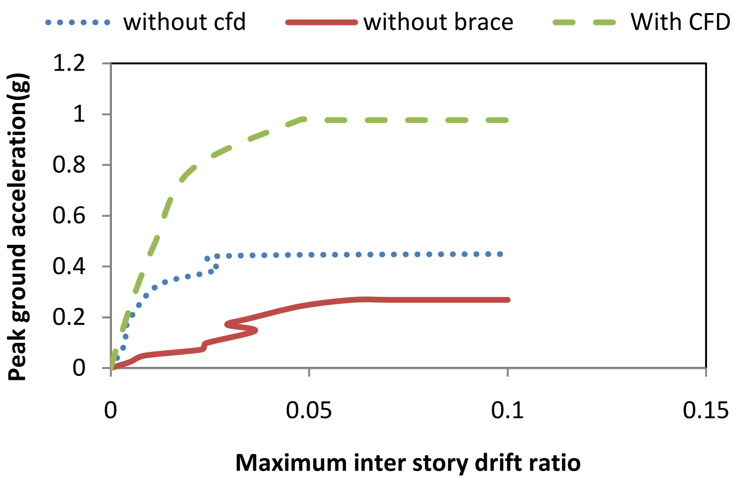


Figure 7. IDA curves of the frame for Caolinga earthquake

4. CONCLUSIONS

Seismic control of steel structures utilizing cylindrical friction dampers (CFD) was investigated. Two-dimensional model of a 6-storey steel braced frame was modeled using OpenSees software. Time history and IDA analyses were applied to the frame with CFD and the same frame without CFD and the results were compared. It was shown that CFD can extremely improve the seismic response of the structure. The results show that optimum slippage load is completely dependent to external seismic load. It was shown when the design slippage load has a difference up to 20% from optimum slippage load, the maximum displacement response can increases up to 35%. However, it is still less than the maximum displacement response of the frame without CFD.

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