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Evaluation of performance of eccentric braced frame with friction damper

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Abstract. Nonlinear dynamic analysis and evaluation of eccentric braced steel frames (EBF) equipped with friction damper (FD) is studied in this research. Previous studies about assessment of seismic performance of steel braced frame with FD have been generally limited to installing this device in confluence of cross in concentrically braced frame such chevron and x-bracing. Investigation is carried out with three types of steel frames namely 5, 10 and 15 storeys, representing the short, medium and high structures respectively in series of nonlinear dynamic analysis and 10 slip force values subjected to three different earthquake records. The proper place of FD, rather than providing them at all level is also studied in 15 storey frame. Four dimensionless indices namely roof displacement, base shear, dissipated energy and relative performance index (RPI) are determined in about 100 nonlinear dynamic analyses. Then average values of maximum roof displacement, base shear, energy dissipated and storey drift under three records for both EBF and EBF equipped with friction damper are obtained. The result indicates that FD reduces the response compared to EBF and is more efficient than EBF for taller storey frames.

Keywords: friction damper (FD); slip load; nonlinear dynamic analysis; performance indices

1. Introduction

The historical buildings that have survived a number of severe earthquakes show that seismic design based on intuition and experience, has a very long history. This approach continued well in to this century. The traditional approach to seismic hazard mitigation is to design structures with sufficient strength capacity and the ability to deform in a ductile manner. The use of wind- resistant cross-bracing after the 1906 San Francisco earthquake leads to introduction of moment-resisting beam-to-column connections and the incorporation of shear walls in the reinforced concrete frames, all represented the first steps towards seismic design. In the early 1980s brought to light some new advanced concepts to the seismic design of multi-storey buildings. Newer concepts of structural control, including both passive and active control systems, have been growing in acceptance and may preclude the necessity of allowing for inelastic deformations in the structural system. A passive control system may be defined as a system which does not require an external power source for

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operation and utilizes the motion of the structure to develop the control forces. Control forces are developed as a function of the response of the structure at the location of the passive control system. An active control system may be defined as a system which typically requires a large power source for operation. Friction dampers are the prevalent of passive control systems, because of being used in different kinds of braces, low cost and suitable efficiency.

Pall, Marsh and Fazio developed friction damper to be used at confluence of cross bracing in 1980. Then in 1987 a 3-Storey frame equipped with this damper, was tested on a shake table by Filiatrault and cherry. In 1988 Aiken, Kelly and Pall equipped a nine-Storey frame with friction damper and tested it on a shake table in the University of California. Then this damper was used in diagonal and chevron braces namely Eaton building Palais Des Conngres of Montreal in 2000. In 1989 Fitzgerald proposed a friction device that utilized slotted bolted connections (SCB) in concentrically braced frames. One end of the brace is connected to the building frame gusset plate using the channel bracing members. Then Grigorian and Popov (1993) presented the experimental and analytical results of the testing of individual SBCs and the testing of a large test structure, equipped with twelve SBCs, on the shake table. Aiken and (1993) presents an overview of seven different passive energy dissipation systems were studied in experimental research programs at the Earthquake Engineering Research Center of the University of California at Berkeley. Their studies, describing the different types of devices, the results of the shake table experiments, and associated analytical work. Tehrani and Maalek (2006) studied the effect of the use of several rehabilitation methods to improve the seismic performance of an existing 9 story steel structure has been investigated using nonlinear dynamic analyses. These methods included the use of the EBF systems; RC Shear Walls and use of Passive energy dissipators such as etallic TADAS, viscous, viscoelastic and friction dampers. Jagadish *et al.* (2008) present the comparative study and performance of variable semi-active friction dampers by using recently proposed predictive control law with direct output feedback. The numerically evaluated optimum parametric value is considered for the analysis of structure with dampers. The numerical results of variable friction damper show better performance over the passive damper in reducing the seismic response of structures. Lee *et al.* (2008) deals with the numerical model of a bracing-friction damper system and its deployment using the optimal slip load distribution for the seismic retrofitting of a damaged building. The Slotted Bolted Connection type friction damper system was tested to investigate its energy dissipation characteristic. It was found by distributing the slip load that minimizes the given performance indicies based on structural response. Numerical results for the damaged building retrofitted with this slip load distribution showed that the seismic design of the bracing-friction damper system under consideration is effective for the structural response reduction.

For friction dampers, the main step is to determine the slip load. If the slip load is chosen big, the structural system acts like a braced frame and if this amount is chosen small, the damper does not slip and cannot control drift in the structure. The value of dissipated energy subjected to friction damper (FD) is product of slip load and drift of all dampers. The energy dissipation in the brace is proportional to the product of slip load and the slip travel during each excursion. For very high slip loads, the energy dissipation in friction will be zero, as there will be no slip if the slip load is very low, the amount of energy dissipation again will be negligible. Between these extremes, there is an intermediate value to give the maximum energy dissipation Therefore, when the difference between input energy and energy dissipated is minimum, the optimum slip load is obtained (Pall *et al.* 1982).

This paper presents a nonlinear parametric assessment on the seismic performance of multistory eccentric braced steel frames equipped with friction damper. The paper includes assessing the 2-

dimensional seismic performance of 5, 10 and 15 storey steel frames equipped with FD and slip force distribution based on storey weight. Dampers are installed on the whole floors of the building structure. However, it may be more effective when FD placed in upper storey or storey with maximum relative displacement. Therefore the 15 storey frame is studied with five FD that placed at optimal placement too. Although EBFs are known for their energy dissipation properly, using FD in them can further their performance by reducing base shear and energy dissipation in the first step. Four dimensionless performance indices are introduced to characterize the seismic efficiency of FD system (Moreschi 2003). Each index is described and discussed in the following subsections.

1.1 Roof displacement index (Rd)

Roof displacement index defined as follows

$$Rd = \frac{D_f}{D_p} \tag{1}$$

Where, D_f is, maximum roof displacement value in nonlinear dynamic analysis for FD system and D_p is the same value for frame without damper.

1.2 Relative Performance Index (RPI)

A simplified seismic design procedure is proposed by Filiatrault and Cherry (1990) for structures equipped with friction-damping system. The system has been shown experimentally to perform very well and could represent a major new development in earthquake-resistant design. The hysteretic properties of the friction dampers are derived theoretically is used to perform a parametric study of the optimum slip-load distribution for the friction dampers. They introduced a relative performance index (RPI) for a given slip load distribution defined as

$$RPI = \frac{1}{2}\left(\frac{ASE}{ASE_0} + \frac{U_{\max}}{U_{\max 0}}\right) \tag{2}$$

where ASE and $U_{\max}$ are, respectively, the area under the elastic strain-energy time history and the maximum strain energy for a FD structure; ASE_0 and $U_{\max 0}$ are the respective quantities of the original uncontrolled structure. The selection of this performance index was motivated by the direct relation that exists between the amount of elastic strain energy imparted into a building and the resulting structural response.

1.3 Energy dissipation by FD index (Re)

This index is defined as Eq. (3)

$$\mathrm{Re} = \frac{E_i - E_f}{E_i} \tag{3}$$

Where E_f as shown energy dissipated by friction damper for FD frame and E_i is total value for input energy. This index does not provide relevant information about the performance of the frame but quantifies the energy dissipated by FD.

1.4 Base shear index

This index (R_f) can be expressed as

$$R_f = \frac{V_f}{V_p} \tag{4}$$

Where V_f as shown maximum value of base shear of FD system in nonlinear dynamic analysis and V_p is the same value for frame without damper.

2. Modeling and assumptions

Three hypothetical buildings are chosen as reference buildings for this study namely 5, 10 and 15 storey frames that have an identical 3 bay layout in plan, 6 m span and 3 m storey height (Fig. 1). Under the assumption that seismic responses in two perpendicular directions are independent, a two-dimensional plane frame model is used in all design analyses and seismic response simulations. The mid span is eccentrically braced that critical eccentric value for the brace has been calculated as 0.5 m. The gravity loads are live and dead load. For each frame a realistic model has been prepared and several nonlinear dynamic analyses have been performed on the models. The frames (Fig. 1) are assumed to be located on a soil type III and in a seismically active area based of 2800-Category (Iranian Code of practice for Seismic Resistance Design of building, Standard No.2800-05 (3th Edition)). All connections are considered to be rigid. The frame members are selected to support gravity and lateral loads determined in accordance with the minimum requirements of Iran Building National Manual (6th subject, Irancategory).

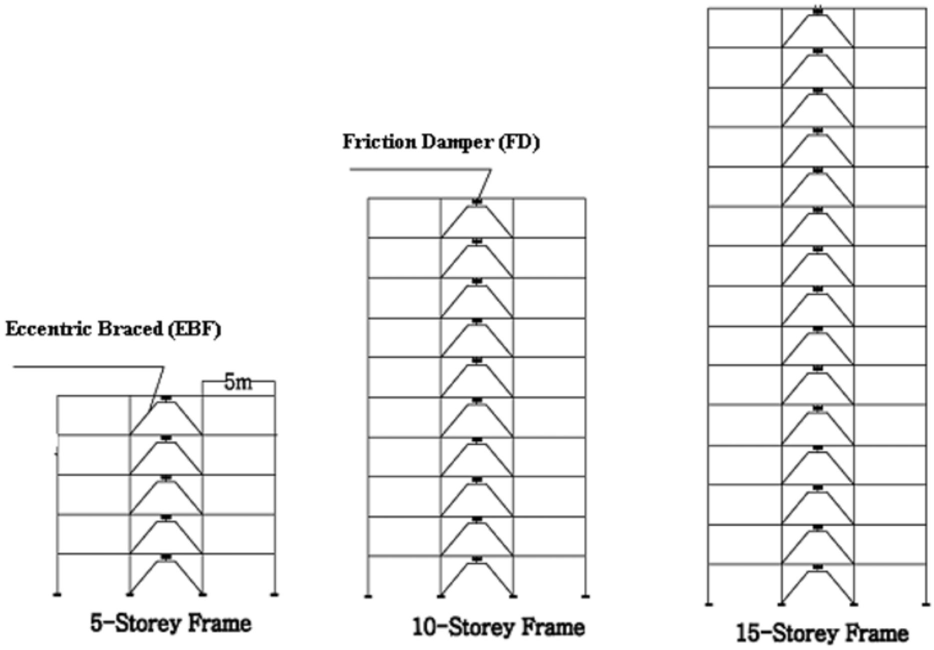

Fig. 1 Geometry of three basic frames

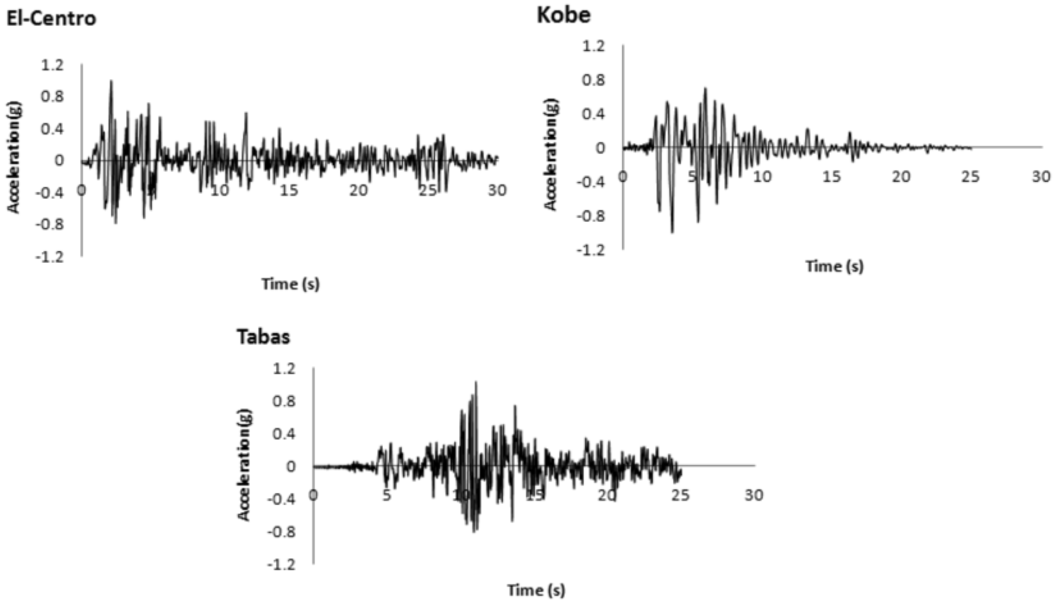

Fig. 2 Real acceleration records of earthquakes

In all of models, the top story is 25% lighter than the others. H-EB, IPE and UNP sections, according to DIN standard, are chosen for columns, beams and braces, respectively. For this reason 2-D nonlinear time-history dynamic analyses are carried out, using the computer program SAP2000 (Nonlinear version), developed by Computers and Structures Inc. (Ibrahimbegovic and Wilson). In the first step, under the code type design (AISC-ASD-89), each frame is analyzed under linear static analysis and once the members are selected, the entire design is checked for code drift limitations and refined to meet the requirements, if necessary.

Nonlinear dynamic analysis is carried out for determining a suitable slip force for each frame. Slip force values are selected as 0, 1, 5, 8, 10, 12, 15, 20, 25 and 30 percent of total storey weight of each building. The weight of each building is defined according to equivalent static lateral loads for a normal building with importance factor equal to 1. For severe earthquakes the structure must remain linear and FD acts as a nonlinear member. Damping ratio is taken 0.05 for the first few effective modes. To investigate the accuracy of different methods to predict the seismic response of eccentric braced frames, 3 ground motions covering a broad variety of conditions in terms of frequency content, peak ground acceleration, velocity and duration, are selected namely El Centro (1940, Imperial Valley, USA), Kobe (1995, Hyogoken-Nanbu, Japan) and Tabas (IRAN 1978). The real acceleration time histories are shown in Fig. 2.

2.1 Description of FD modeling

Most friction dampers produce a stable rectangular hysteresis that Fig. 3 shows a typical FD and its hysteresis loop. The hysteretic behavior of friction devices and connections can be modeled using Wen Plastic element In SAP2000 (Ibrahimbegovic and Wilson) that has an ideal elasto-plastic diagrams (representing Coulomb friction) in Fig. 3.

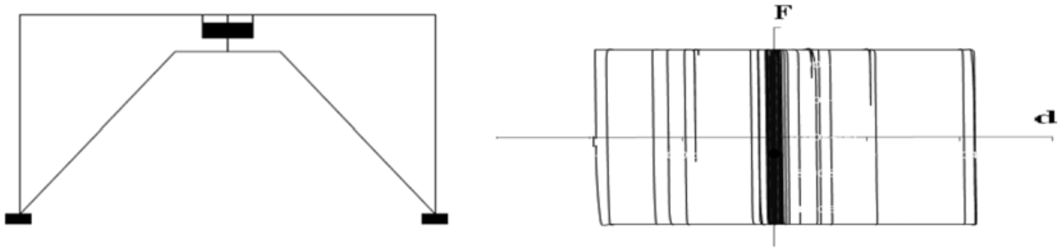

Fig. 3 FD in Eccentric Brace with hysteretic loop

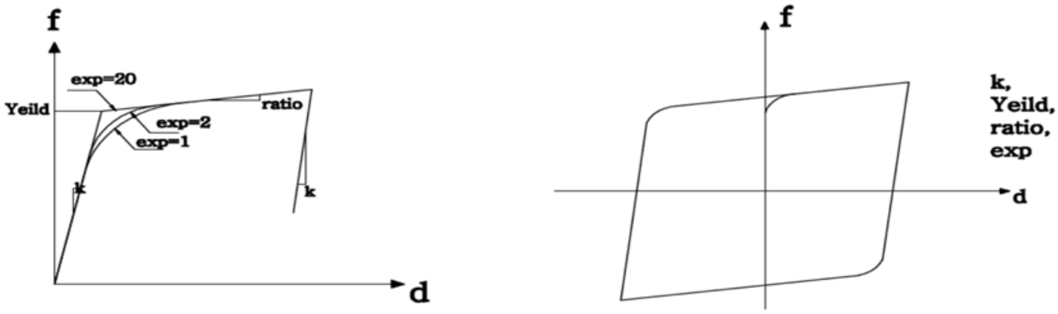

Fig. 4 Hysteretic loop of Wen Plastic element in Sap 2000

In Fig. 4, equation of force-drift of Wen Plastic can be expressed as Eqs. (5) and (6)

$$f = ratio(k) \cdot d + (1 - ratio) yield(z) \tag{5}$$

$$if\ \dot{d} > 0 \Rightarrow \dot{z} = \frac{k}{yeild}\dot{d}(1 - |z|^{\exp})$$

$$if\ \dot{d} < 0 \Rightarrow \dot{z} = \frac{k}{yeild}\dot{d} \tag{6}$$

Where, k is stiffness coefficient ratio and yield is the yield force of Wen plastic element. exp is an index greater or equal to 1. If exp index is greater than 1, sharpness of hysteresis diagram is increased. Quantity 20 for exp can produce the hysteresis diagram for FD to be more reality.

3. Results and discussion

For the purpose of numerical evaluation, several response indicators were estimated: energy dissipated by the damper as a percentage of the total seismic input energy, maximum response displacement, maximum total base shear, and maximum base shear in the primary frame only. According these parameters is obtained results of four performance indices for determining of suitable slip load for each frame. Then, with suitable slip load, time history of roof displacement and base shear for EBF frame with FD and without FD for each frame are established. After comparison of these results in time history diagrams under each earthquake, the average percentage envelopes in base shear, roof displacement, input energy and dissipated energy by FD is compared with EBF.

3.1 Evaluation of performance index for FD system

For each earthquake the value of slip load is varied as a percentage of the weight of floor from 0 to 30 and as discussed in section 1, four performance indices is used to determining the optimal slip force.

Figs. 5, 6 and 7 shows the results of performance indices. In these diagrams the horizontal axis is the value of slip load (kN) and the vertical axis is the amount of four performances indices.

Each four performance indices are equal 1 for value zero slip load. Because in this state the amount of energy dissipation will be zero and FD is not effective.

Fig. 5 compare four performance indices for 5, 10 and 15 storey frames subjected to the El-Centro earthquake. This Fig. shows that for 5-storey frame if slip load is greater than 120 kN, the performance indices, except Rf, almost converge to steady values. In 5-storey frame the suitable value of slip load is 40 kN (0.08w = 0.08 of total weight of frame) for RPI, 60 kN for Rf and 120 kN for Re and finally 140 kN for Rd.

In 10-storey frame the suitable value of slip load is 100 kN (0.15w = 0.15 of total weight of frame) for RPI, 120 kN for Rf and Re. The main difference between 5-storey and 10-storey frames diagrams is variation of Rf as the value of Rf for 5-storey frame is growth for slip load exceeds from 120 kN. For 15-Storey frame the best value of RPI is between 100 to 120 kN. The main difference between 15-storey frame under El-Centro earthquake and 5 storey frame is variation RPI so that is about 30% for 15-storey. This means that energy dissipation in higher frame is more

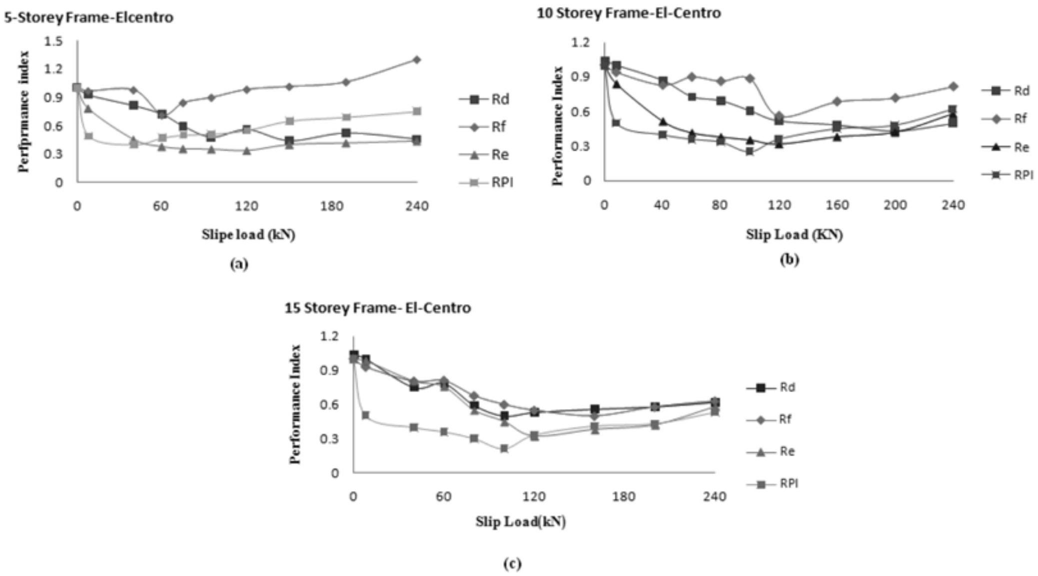

Fig. 5 Performance Indices subjected to El-Centro earthquake (a) 5-storey frame, (b) 10-Storey frame, (c) 15-Storey frame

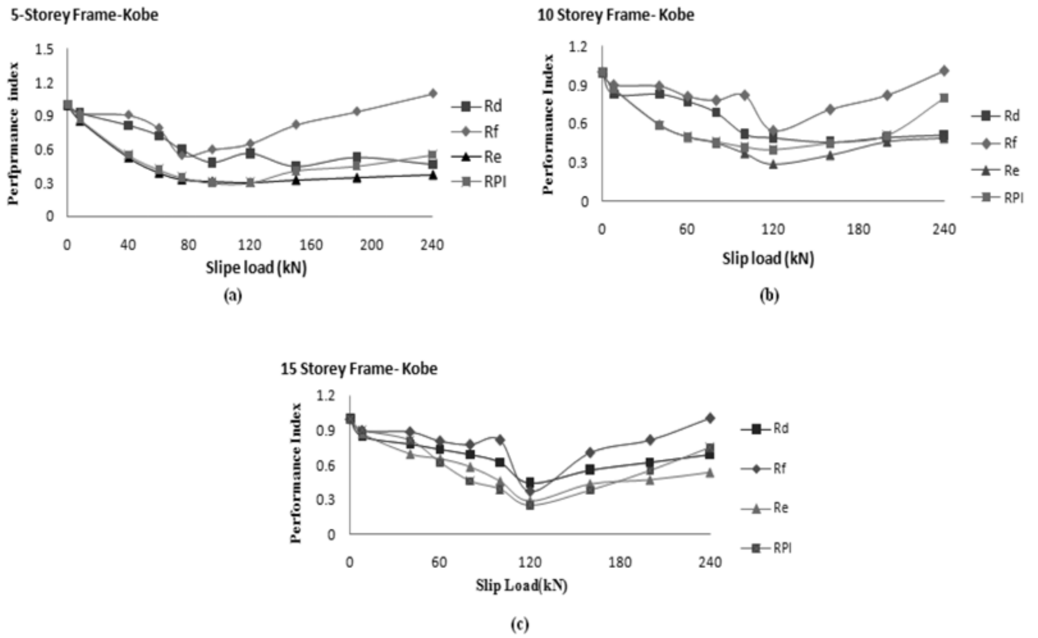

Fig. 6 Performance Indexes subjected to Kobe earthquake (a) 5-storey frame, (b) 10-Storey frame, (c) 15-Storey frame

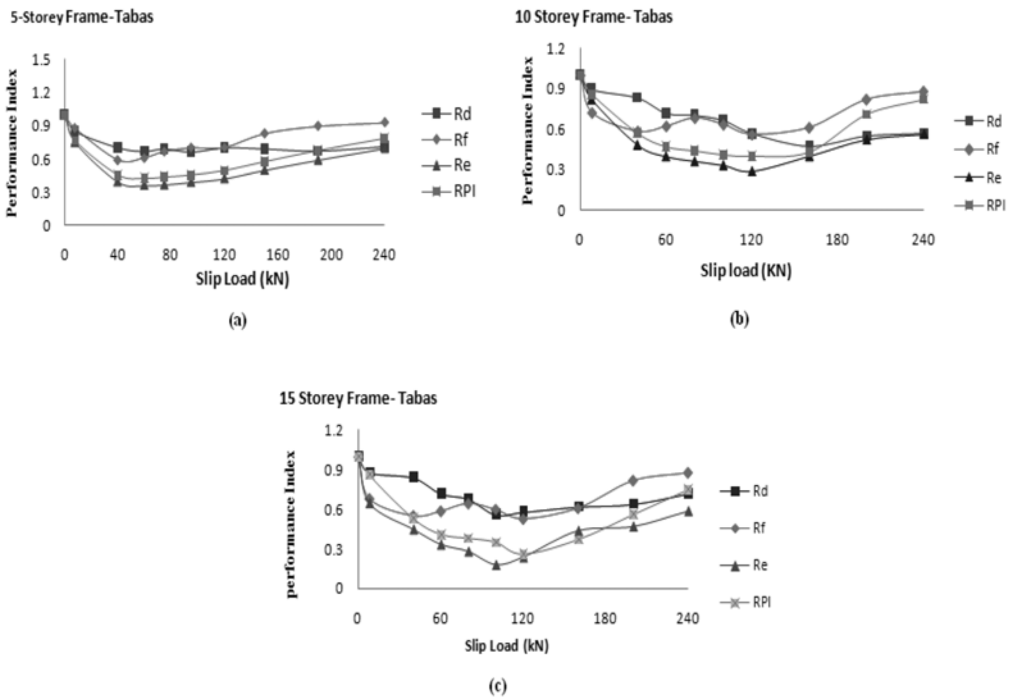

Fig. 7 Performance Indices subjected to Tabas earthquake (a) 5-storey frame, (b) 10-Storey frame, (c) 15-Storey frame

considerable than in shorter ones.

Diagrams in Fig. 6 are obtained taking again 5, 10 and 15 storey frames under Kobe earthquake. This Fig shows that, the value for Rd, Rf, Re and RPI are equal 1 for slip load equal to 0. In 5-storey frame the adequate value of slip load is 95 kN (0.1w = 0.1 of total weight of frame) for RPI, 75 kN for Rf and 95 kN for Re. In 10-storey frame the adequate value of slip load is 120 kN (0.15w = 0.15 of total weight of frame) for RPI, 120 kN for Rf and 120 kN for Re. These value for 15-storey frame is 120 kN for RPI, Re and Rf.

Diagrams in Fig. 7 again contain 5, 10 and 15-storey frames subjected to Tabas earthquake. This Fig allows deriving similar conclusions than those obtained from two previous figures. The main difference is that the plots for Rd reach minimum value in 160-190 kN, in 5-storey frame. In this case FD is useful to reduce roof displacement in higher slip load if compared to other performance indices.

3.2 Evaluation of time-history result

In this section, results of nonlinear dynamic analysis of frames with and without FD are compared. At all analysis slip force is selected which suitable slip force was calculated in ago section. The time history of roof displacement and base shear responses of the three frames with and without FD is shown in Figs. 8 to 13.

From Fig. 10, it is observed that, in reducing of the peak roof displacement, FD acts more efficiently than EBF, in 5-storey frame, but from Figs. 8, 9 it is observed that FD haven't more efficiency in 10-storey frame under Kobe and El-Centro earthquake, wherein the peak roof displacement for 15-storey frame in kobe earthquake in FD is increased about 30% from the second 5 to 10. This is because, the sway of the taller frame in this case 10 and 15-storey frames

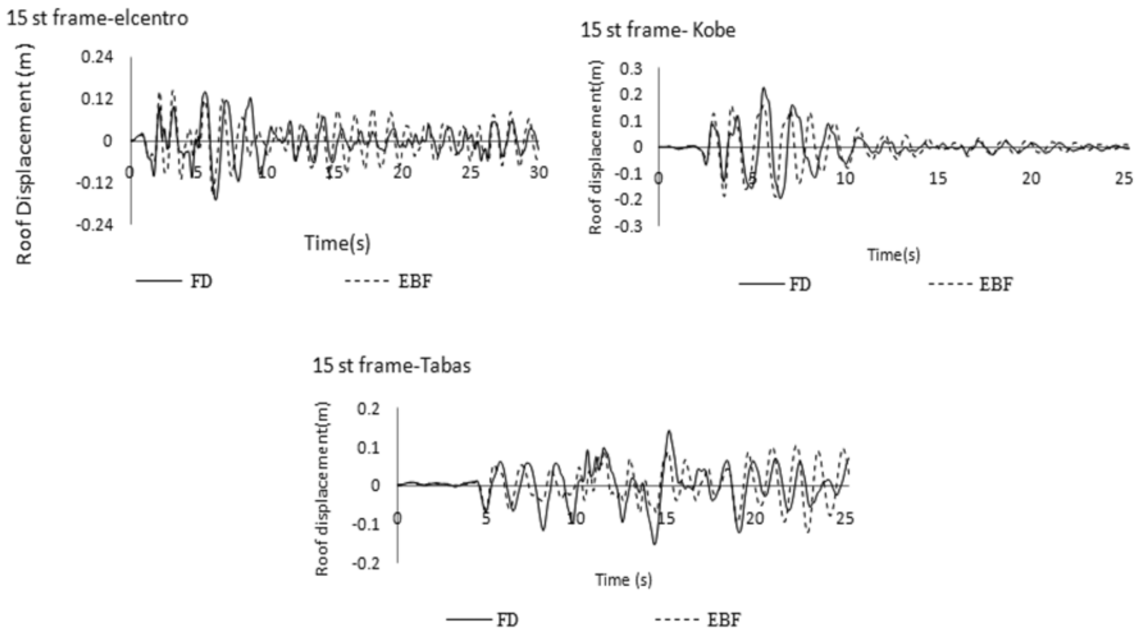

Fig. 8 Time histories of roof displacement of the 15-storey frame

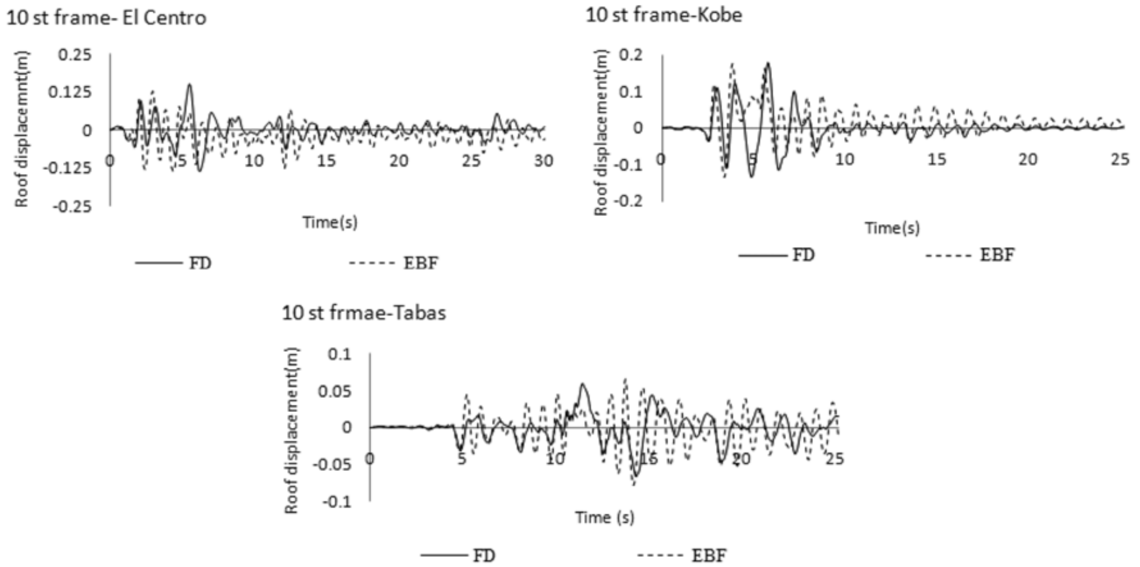

Fig. 9 Time histories of roof displacement of the 10-storey frame

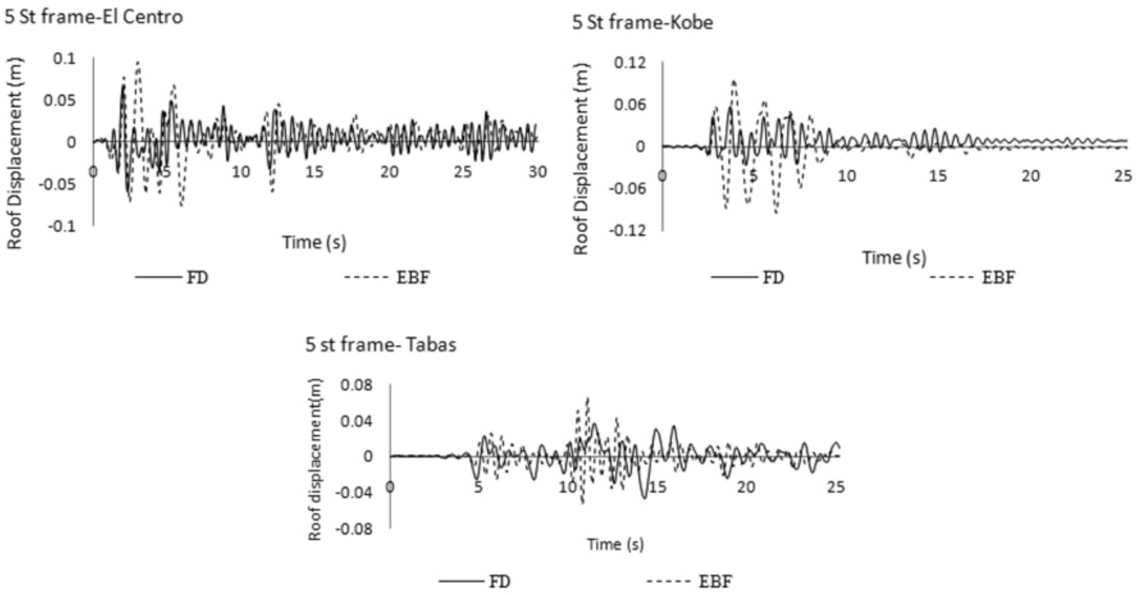

Fig. 10 Time histories of roof displacement of the 5-storey frame

is more severe and the roof displacement increases in FD frame compared to EBF. Figs. 11, 12 and 13 show time history of base shear for each frame. These Figs clearly indicates the effectiveness of dampers in controlling the earthquake responses of three frames in all of seconds of earthquake.

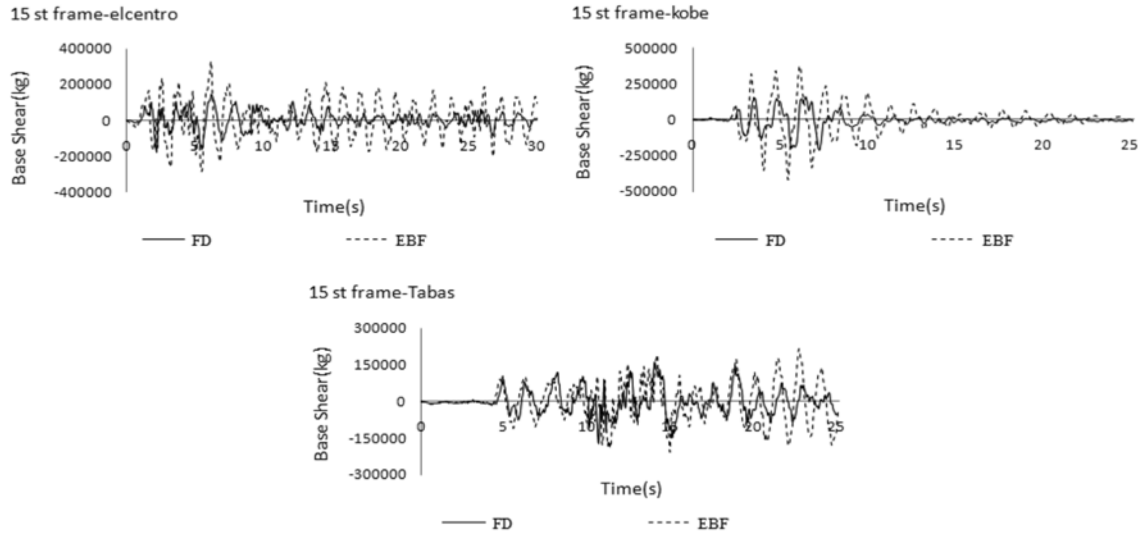

Fig. 11 Time histories of the base shear of the 15 storey frame

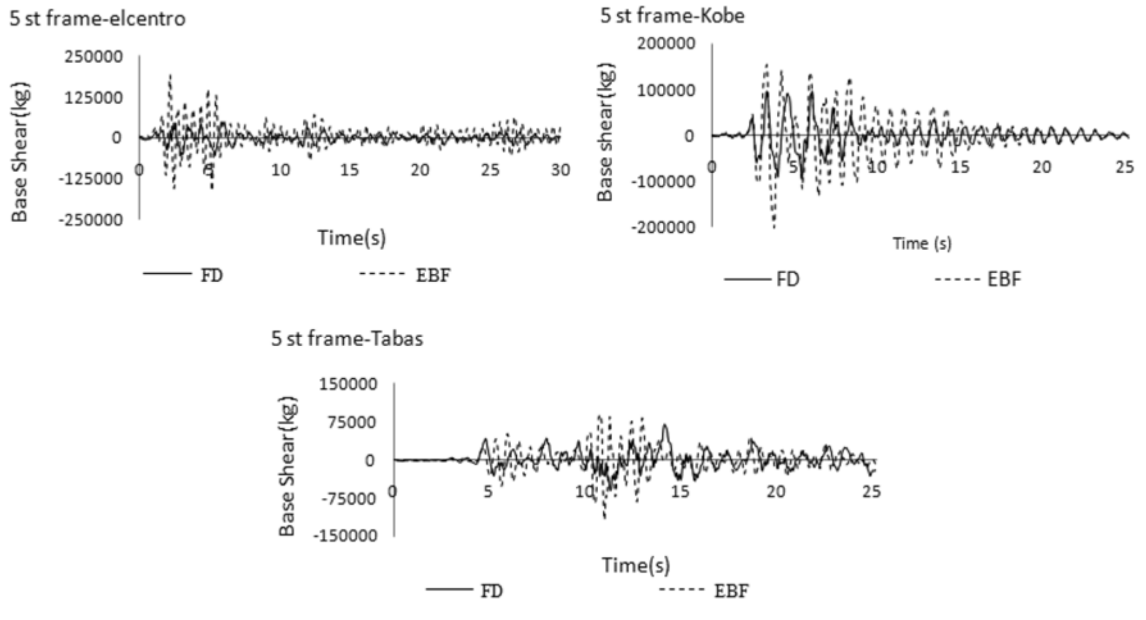

Fig. 12 Time histories of the base shear of the 10 storey frame

3.3 Evaluation of average result

Figs. 14, 15, 16 and 17, illustrates the average percentage reductions in peak values of three earthquake of roof displacement, base shear and input energy experienced by FD frames with the most suitable slip load compared with the EBF without FD. In order to do so, maximum value of roof displacement, storey drift, base shear and input energy for FDs and EBFs investigated in this

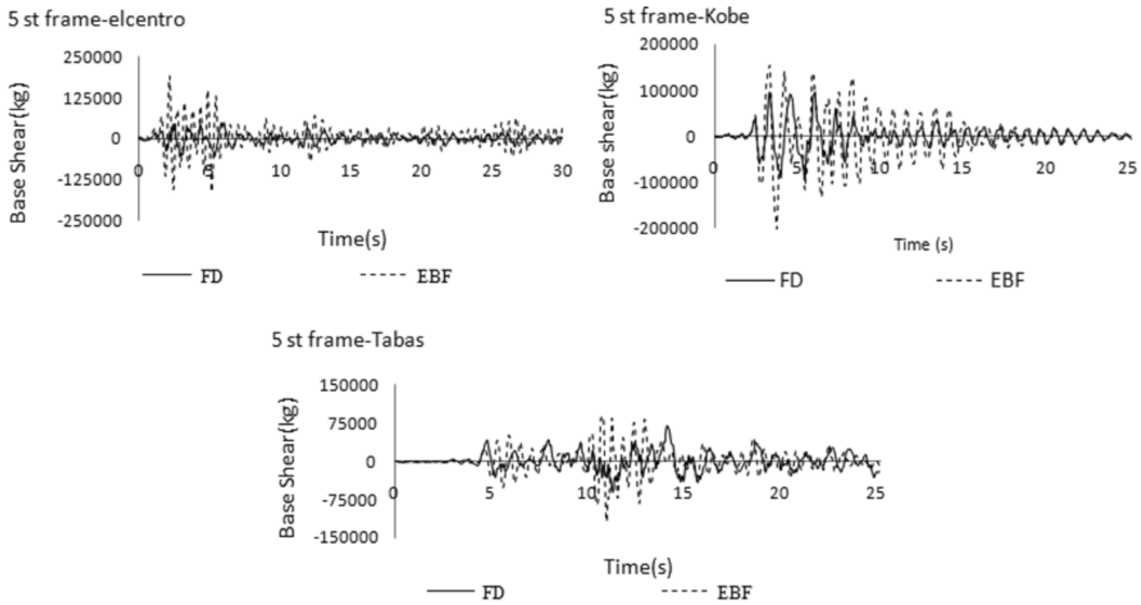

Fig. 13 Time histories of the base shear of the 5 storey frame

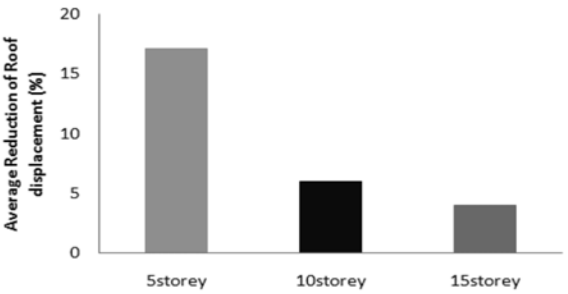

Fig. 14 Average percentage reduction in roof displacement for 5, 10 and 15 storey frames

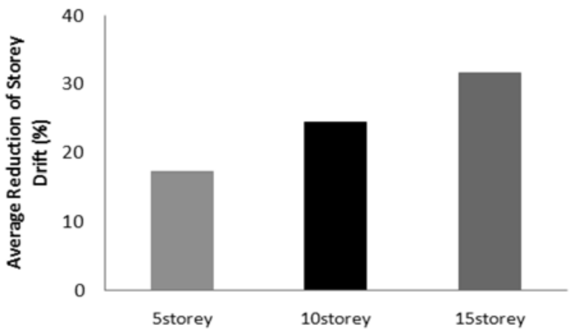

Fig. 15 Average percentage reduction in storey drift for 5, 10 and 15 storey frames

paper for each earthquake, is gained. A then, average reduction percentage value, which is obtained by subtracting each value for FD from the corresponding EBF and then dividing it by the initial value without FD and averaging out results for each earthquake, are gained. Fig. 14 shows the average percentage reductions in roof displacement for FD in comparison EBF. In this diagram the vertical axis is the value of average roof displacement of three earthquakes. From this Figure it is observed that the best reduction in roof displacement is occurred in 5 storey frame with an average reduction value about 17%. This value for 10 and 15-storey frames is gained about 6% and 4%.

Fig. 15 show the average percentage reductions in storey drift for FD in comparison EBF. In this diagram the vertical axis is the value of average storey drift of three earthquakes for three frames and the horizontal axis is the value of storey of frame. From this Fig it is observed that the best reduction in storey drift has been occurred in 15 storey frame with an average reduction of 32%. The second reduction is occurred in 10 storey frame with an average reduction of 24% and for 5 storeys this value is 17%. Therefore the best performance of FD in reduction of storey drift is achieved in higher frame.

In Fig. 16 the average percentage reductions in base shear for FD in comparison with the EBF is presented. In this diagram the vertical axis is the average percentage reduction of maximum base shear of frame with FD to EBF. From this fig it could be observed that the best reduction in base shear has been occurred in 15 storey with an average reduction of 66%. The second reduction is occurred in 10 storey frame with an average reduction of 55% and lowest performance has been recorded for 5 storey frame with an overall reduction of 41%.

The best way to comparison between FD frame and EBF frame without FD, is assessing of results of input energy to each frame by earthquake record and energy dissipated with each frame (FD and EBF). Therefore in Fig. 17 the reduction value of input energy of FD frame to EBF without FD is presented. This value is percentage of average reduction for three earthquakes for each frame.

From this Fig it is observed that the input energy in frame with FD is less than frame without FD for short, medium and high frame and the best performance is occurred in higher frame (15 storey) with an average reduction of 38% in comparison EBF.

As observed from Fig. 18, the average percentage increasing in peak values of three earthquake of energy dissipated experienced by FD frames with the most suitable slip load compared with the

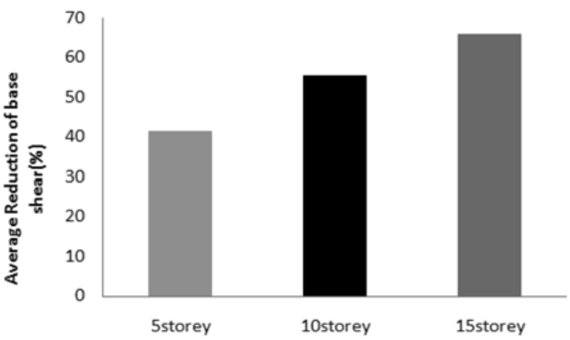

Fig. 16 Average percentage reduction in base shear for 5, 10 and 15 storey frames

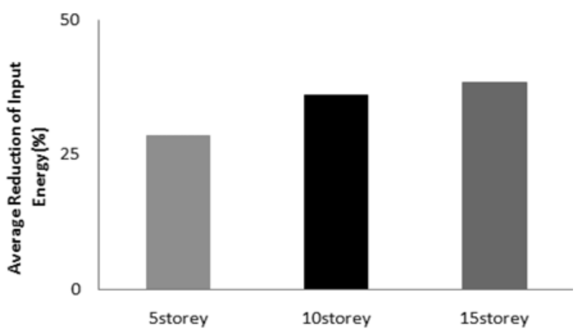

Fig. 17 Average percentage reduction in total input energy for 5, 10 and 15 storey frames

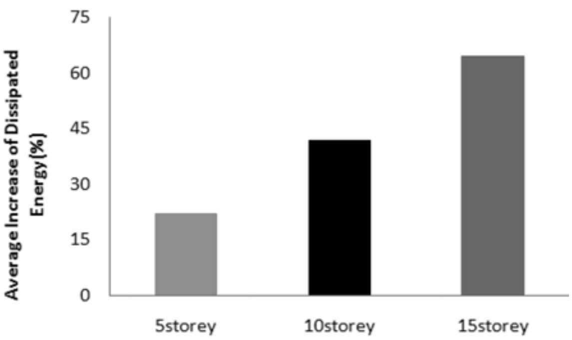

Fig. 18 Average percentage increase in dissipated energy for 5, 10 and 15 storey frames

EBF without FD. Therefore, maximum value of dissipated energy for FDs and EBFs investigated in this paper for each earthquake is gained. Then average increase percentage values, which are obtained by subtracting each value for EBF from the corresponding FD and then dividing it by the initial value without EBF and averaging out results for each earthquake, are gained. Fig. 18 clearly indicates that the performance of each frame that dissipated more input energy is better, about 65% of input energy is dissipated by FD in 15-storey frame, therefore the remaining energy in that left in the FD in 15-storey is approximately 35%. The increasing in dissipated energy for FD into EBF is about 40% for 10-storey frame and 22% for 5-storey frame. Therefore dissipation of input energy and performance of damper in higher frames is better than others frame, because the higher frames has more flexibility.

3.4 Evaluation of placement of FD

In order to study the influence of location and to minimize the cost of FD, the responses of the 15-storey frame with FD fitted only a few upper storeys are investigated by proper slip. The 15 storey frame whichever has the maximum relative displacement are selected to place the FD. Fig. 19 show the variation of the maximum floor displacement and Fig. 20 show time history of base shear for three different cases under Kobe, Tabas and El-Centro earthquake, when FD replaced at all the floors, FD replaced only at 11, 12, 13, 14 and 15 floors and frame without FD. It can be observed from the figures that the dampers are more effective when they are placed only at five upper floors. When the dampers are attached to theses floors, the maximum displacements in all the stories and base shear in all times are reduced much more than when they are connected at all the floors.

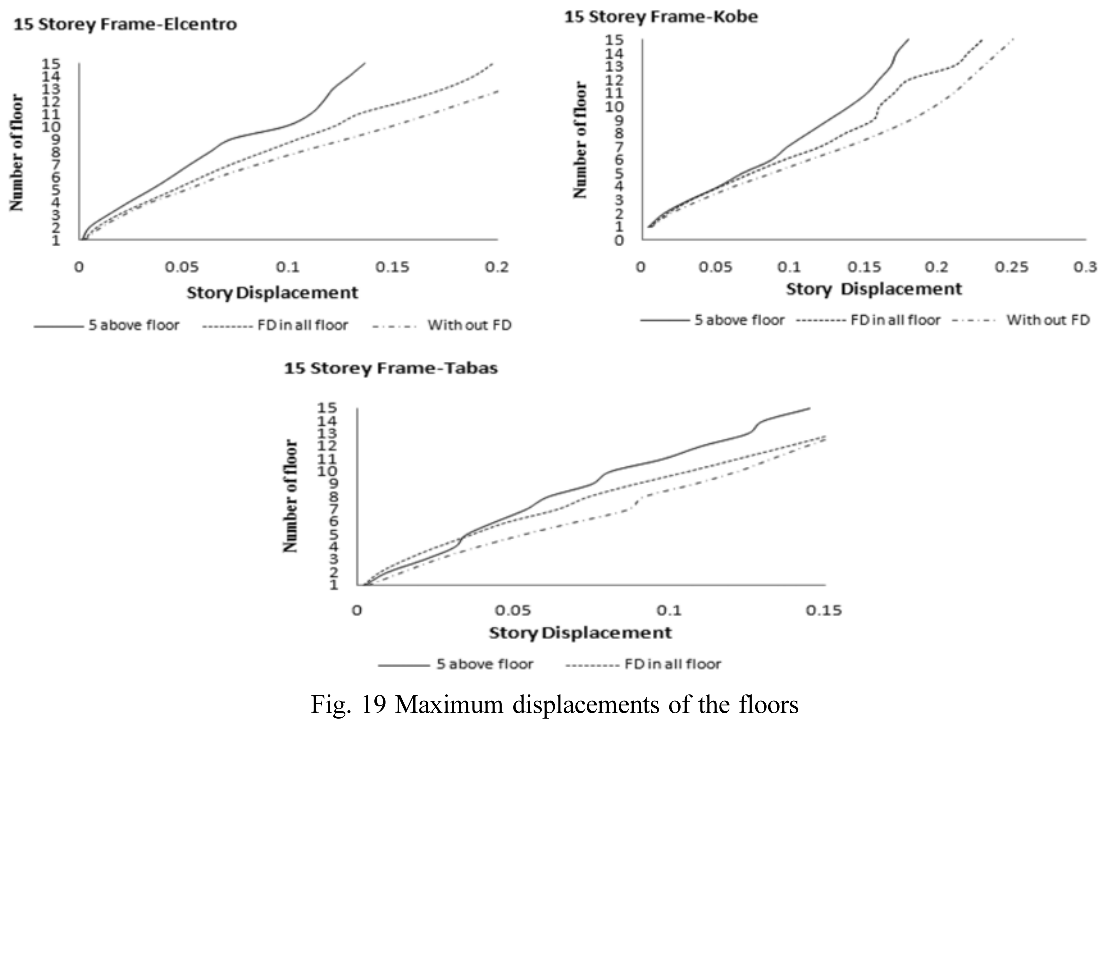

Fig. 19 Maximum displacements of the floors

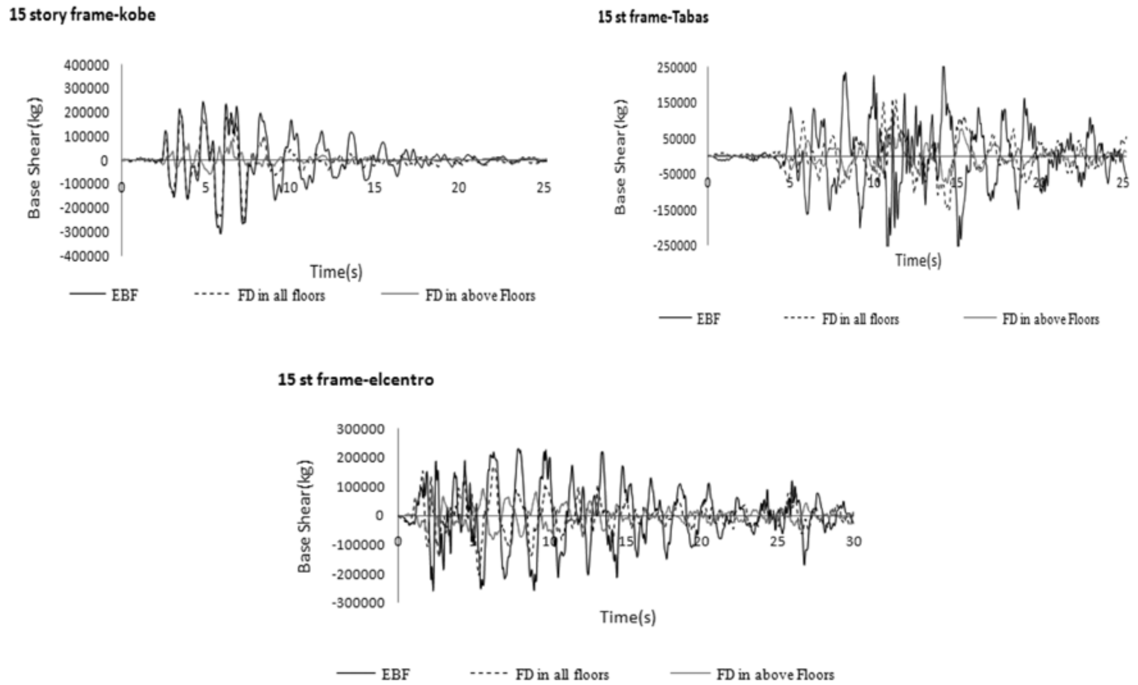

Fig. 20 Time history of base shear

4. Conclusions

In the design of the friction damper for seismic structural control, it is the most important factor to determine the quantity, location and slip-load of the damper friction. In this study slip load is varied from 0% to 30% building total weight that equally-distributed in friction damper at all of storeys. Also In order to study the influence of location friction damper the responses of the 15-storey frame with FD fitted only a few upper storeys are investigated by proper slip With the consideration of the case study under investigation and the alternative methods rehabilitation suggested here, are summarize the following concluding remarks:

1. With compared different performance indices of frames, is estimated that slip load equal 8% to 15% is more appropriate.
2. The influence of FD in EBF is investigated with average value of roof displacement, storey drift, base shear and energy dissipated. For each frame FD is reduced these values into EBF. In higher frame with FD, because has less stiffness, roof displacement is increased.
3. Peak value of Base shear in of FD frame is reduced in of up to 40% for shorter frame and to 65% in higher frame. This reduction for storey drift is up to 35% for higher frame and 17% for shorter frame. But the best reduction in absorption of input energy is up to 22% in shorter frame, 40% in 10-storey frame (medium frame) and 65% in higher frame. Therefore when FD placed in higher frame gave the maximum increasing in dissipated energy.
4. From time-history diagram of roof displacement it is observed that in higher frame because of major sway in roof than to short frame, EBF performed better for its more rigidity to FD.
5. The function of FD when placed only a few upper floor is better than at all floors, therefore

replacing them in upper floors reduces the seismic response.

Hence, this study investigated the evaluation of FD in eccentric braced frame under three earthquake records. It has been demonstrated that it is possible to improve eccentric braced frame performance under seismic behavior by using FD in conflation of this brace. This system is more suitable for frames that have more than 10 storeys.

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