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Using friction dampers in retrofitting a steel structure with masonry infill panels

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Abstract. A convenient procedure for seismic retrofit of existing buildings is to use passive control methods, like using friction dampers in steel frames with bracing systems. In this method, reduction of seismic demand and increase of ductility generally improve seismic performance of the structures. Some of its advantages are development of a stable rectangular hysteresis loop and independence on environmental conditions such as temperature and loading rate. In addition to friction dampers, masonry-infill panels improve the seismic resistance of steel structures by increasing lateral strength and stiffness and reducing story drifts. In this study, the effect of masonry-infill panels on seismic performance of a three-span four-story steel frame with Pall friction dampers is investigated. The results show that friction dampers in the steel frame increase the ductility and decrease the drift (to less than 1%). The infill panels fulfill their function during the imposed drift and increase structural strength. It can be concluded that infill panels together with friction dampers, reduced structural dynamic response. These infill panels dissipated input earthquake energy from 4% to 10%, depending on their thickness.

Keywords: masonry-infill panel; seismic retrofitting; pall friction damper; earthquake energy; structural analysis

1. Introduction

The damages and losses caused by recent severe earthquakes have increased concern for finding ways to improve structural behavior against earthquake. Applying new methods in structural seismic design and improving the quality of the structural materials are currently among the common approaches to this objective. Various methods based on distributing energy in structures have been recently developed to control seismic vibrations and reduce the effect of earthquakes forces. Large amounts of energy are generally transferred to structures during an earthquake, which must somehow be dissipated within the structure in different forms like kinetic

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and hysteretic energies. Without a damping system, the structure vibrates continuously, but in practice structural properties provide some damping that reduces structural vibrations.

The masonry infill panels have the ability to absorb and dissipate some portions of the input earthquake energy. The masonry-Infill panels cover, partially or fully, the spans in the steel or concrete frames and are usually surrounded by beams and columns. These panels prevent the in-plane deformation of the frame, and a part of the lateral force applied to the frame is therefore transferred to the panel. Previous studies and experiments showed that the lateral strength and stiffness of the frame with infill panel are much higher than those of the frame without the infill panel.

The main problem is that these elements crack suddenly during an earthquake and fail completely. If a crack develops, then the earthquake component perpendicular to the wall deforms the wall outside of its plane. This issue for instance caused severe damage during the recent Manjil and Bam Earthquakes in Iran (Zahrai and Heidarzadeh 2007). A previous report, which included studies from 1948 to 1990, has shown that the effect of interaction of the frame and infill panel on the structural behavior is not negligible.

Crisafulli *et al*. (2000) presented a general review of the different procedures used for the analysis of infilled frames using finite element formulation and the equivalent truss mechanism. Generally, the interaction of frame and infill panel increases the lateral stiffness, strength, and ductility of the structure, improving the seismic behavior of the structure significantly (Naeim 2001). Dolsek and Fajfar (2008) investigated the effect of masonry infill panels on the seismic response of a 4-story reinforced concrete building based on pushover analysis and the inelastic spectrum approach. Their results showed that the infill panels can completely change the distribution of damage throughout the structure. Moreover, results showed that if infill panels are placed symmetrically within the structure, they can have a beneficial effect on the structural response by avoiding shear failures of columns.

Pujol and Fick (2010) conducted the experimental study on a full-scale 3-story RC structure with and without masonry infill walls. The results showed that the added walls increase base shear strength and lateral stiffness by approximately 100% and 500% respectively. It was also observed that the drift capacity of the structures with infill walls is 1.5% which shows the masonry infill walls are able to control inter-story drift.

Koutromanos *et al*. (2011) investigated the behavior of masonry-infilled RC frames subjected to seismic loads by nonlinear finite element modeling. The constitutive models have been validated with experimental shaking table test performed on masonry-infill panels in a non-ductile reinforced concrete frame. The results have indicated the capabilities of the finite element method in capturing the nonlinear cyclic load–displacement response and failure mechanisms of the structure. The results have also indicated that even though infill walls are considered as non-structural elements in design, they can significantly contribute to the seismic resistance of a structure.

Asteris *et al*. (2011) reviewed different macromodels used for the analysis of infilled frames, pointed out the advantages and disadvantages of each macromodel and indicated a practical recommendation for the implementation of the different models. Asteris and Cotsovos (2012) performed nonlinear finite element analysis and showed that infill walls in RC frames act as diagonal compression ‘struts’ offering relief to certain structural elements of the frame. They showed from dynamic case studies that the structural elements of the story of the RC frame without infill walls sustained more damages.

Uva *et al*. (2012a) showed that in the seismic assessment of infilled RC frames, it is crucial to adopt multi-strut systems for equivalent strut model. This is in fact the only model able to include the observed brittle behavior triggered at the nodes by the presence of the infill. In another study by Uva *et al*. (2012b), non-linear static (pushover) analyses were performed on the structural building models with and without infill walls, in order to evaluate the effect of infill walls on the failure mechanisms. The results showed that the presence of a strong infill with high strength and stiffness significantly changes the structural response to horizontal movement and the collapse mechanism changes substantially for both high-intensity and low-intensity earthquakes.

Fiore *et al*. (2012a) proposed a method to model an infill panel by two equivalent struts whose position is expressed in function of the aspect ratio of the panel. The results showed that panels increase the lateral stiffness of the frame and contribute on the possible local effects. This proposed method is not completely acceptable for low seismicity since the two-strut mechanism is not necessarily activated under small horizontal loads. Fiore *et al*. (2012b) performed push over analysis on one low rise RC building and one tall RC building, both located in a high seismic hazard area to investigate the influence of infill panels. They showed that the infill panels increase the stiffness and strength of the overall structure both in low rise and tall building while they reduce the displacement capacity of the structure just in the low rise building. The reduction of the ductility is almost negligible for tall building.

Celarec and Dolsek (2013) investigated the effect of modelling uncertainty on the seismic performance assessment of three selected RC frame buildings by using a simplified approach. The results of sensitivity analysis show that the rotational capacity of the plastic hinges in the columns and beams has the greatest impact on the seismic response parameters. Meslem and D'Ayala (2013) showed that modelling building as bare frame structure lead to lowest risk of damage, however, the building is found to be more vulnerable if the infilled frame system is adopted.

Fiore *et al*. (2014) investigated the variability of structural capacity due to the variation of the nonlinear cyclic law of equivalent strut. They showed for low intensity earthquakes, the structural capacity of regular building, depends much more on the collapse limit state than mechanical characteristics of the infills. Porco *et al*. (2015) presented some considerations about the effects induced by strengthening interventions involving the tying of the infill panels to the RC frame. They provided an appraisal of the actual displacement capacity and the possible alteration induced on the global collapse mechanism.

Although some researchers have studied the influence of infill walls on lateral behavior of RC frames and few of them as presented in Section 1.2 investigated the role of dampers on retrofitting building with infill panels, no one has evaluated the effect of masonry infill panels on behavior of friction dampers. Seismic vibration control of structures is one of the novel approaches to enhance building efficiency and reduce structural displacement or acceleration. There are four vibration control systems: Passive, Active, Semi-Active and Hybrid. A passive control system operates without requiring an external power source. This system consists of one or more devices designed to modify the structural properties such as stiffness, ductility and dissipated energy, leading to reduction in structural vibrations. The Pall friction damper as one of the passive control systems is investigated in this research (Friedriechs 1997).

1.1 Pall friction damper

In 1980, Pall and his colleagues began investigating dissipation of energy in structures during earthquakes by means of friction. In that study, the structural movements were limited by devices that worked like an automobile brake to dissipate kinetic energy. These studies led to the development of the Pall friction damper in 1982 (Fiore *et al*. 2012a).

The Pall friction damper consists of a set of special steel plates that can generate the necessary frictional performance. These plates are bolted together with high-strength screws, and they are designed not to slip during wind. These dampers slide over each other at the determined optimum slip load before structural members yield, and they dissipate a large portion of earthquake energy. As a result, the structure remains in the elastic range, and thus undesired yielding of structural members during major earthquakes is delayed. The related hysteresis loop is rectangular showing an elastic perfectly plastic behavior. Strength fades negligibly during cyclic movement. As Fig. 1 shows, for a given force and displacement, friction dampers essentially dissipate more energy than other kinds of passive dampers. Fig. 2 shows the details of the Pall friction damper and its components (Fiore *et al*. 2012a).

Friction dampers have been used for seismic retrofitting in many modern structures. For example, Pall friction dampers were installed in 58 bracing spans in the 50,000 m^2 Canadian Space Agency Headquarters; whose construction was completed in 1992. In the 5- and 9-story concrete McConnell library at Concordia University (Montreal, Quebec), 143 Pall friction dampers were used. In 2004, Pall and his colleagues showed that using friction dampers could save 6.5% of the structural cost over use of a concrete shear wall. In the precast-concrete school building in Sorel, Canada, Pall and his colleagues used two rehabilitation methods: the conventional use of a cast-in-place concrete wall and use of Pall friction dampers. Investigations show that using the Pall friction damper saved 40% of construction costs and also 60% of construction time (Soong and Dargush 1997).

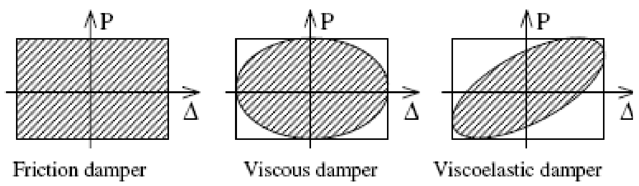

Fig. 1 Hysteresis loops for different dampers (Friedriechs 1997)

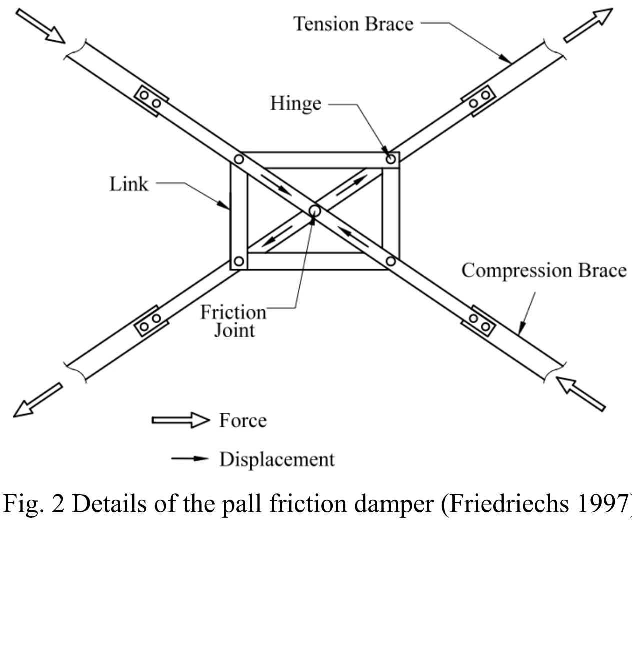

Fig. 2 Details of the pall friction damper (Friedriechs 1997)

Evaluation of the behavior of the Quebec Provincial Police Headquarters building during rehabilitation revealed that the story drifts were high and the building had low resistance to lateral load which was calculated based on national building code of Canada. According to national building code of Canada, if the seismic resistance in a building is less than 60% of that in equivalent building designed based on the new specification, the building then needs to be retrofitted. Sixty-two friction dampers were used to retrofit this structure, and the performance of the structure was then evaluated by time-history analysis. The friction dampers absorbed 50% of the energy, and the roof displacement was decreased to 0.18 m (Pall *et al*. 2002).

Pall Friction Dampers were also used in the steel bracing system in La Gardenia Towers in New Delhi, India built in 2000. There was no need to use costly concrete shear walls in this building, which had been equipped by Pall friction dampers (Chandra *et al*. 2000).

Wu et al. developed a new type of Pall friction damper with a simple configuration and compared the operation and damper force of the improved PFD (IPFD) with those of the original PFD (OPFD). Results have showed that the resisting forces generated by those two dampers are identical with similar frictional forces. Therefore, the IPFD has the same mechanical properties as the OPFD, while because of its simpler configuration, the IPFD is cheaper and easier to analyze than the OPFD (Wu *et al*. 2005).

1.2 Using pall friction dampers with masonry infill panel

In this section, the seismic performance of some buildings with masonry infill panels and/or Pall friction dampers is presented.

Dihog and Miyamoto (1999) compared the performance of a viscous damper versus friction damper in retrofitting of a 12-story concrete building in Seattle, Washington. This concrete structure did not have appropriate ductility. From the basement to the second floor, the lateral resistance system was concrete shear wall. From the second floor to the tenth floor, it was concrete moment frame with clay block partition wall and brick-infill panel with opening. From the tenth floor to the roof, it was concrete wall column on the perimeter of the structure. In this building, the stiffness of the complex beam was two to four times higher because of the infill panels. In the study of this structure, lateral displacement of the stories was assigned to be 0.6% of the performance point of the structure displacement. The results showed that the masonry-infill panels performed appropriately until lateral relative displacement reached 1%. After this displacement level, masonry walls were damaged substantially, and the concrete frame worked alone as the lateral resistance system. Finally, the combination of nonlinear viscous damper and friction dampers were considered for retrofitting this building.

In 2003, Pall and his Colleagues also investigated the effect of Pall friction dampers in 9-story Eaton building located in Montreal Canada. This building had concrete and steel frames and concrete slab as the flooring system. Lateral resistance system for this building was external masonry infill panel, interior walls and concrete frame. This building did not have enough lateral resistant based on building codes. Eaton building was retrofitted by installing 161 friction dampers in the new structural. Analytical simulations showed that the use of friction dampers could provide a practical and economical solution for the seismic upgrade of the Eaton Building (Pall *et al*. 2002).

Although in these few studies, the role of dampers on retrofitting building with infill panels was investigated, there has been no evaluation on the effect of masonry infill panels on behavior of friction dampers. The objective of this study is to determine if friction dampers can effectively work in frames with masonry infill panels and also to find out whether and how the effect of masonry infill panels needs to be considered while retrofitting a building with Pall friction dampers. In this study, the impact of Pall friction dampers on the seismic response of the steel frame with masonry infill panel as one of the passive control system is investigated. Contrary to previous research projects, simple frame is considered here. Also, the effect of masonry infill panel on performance of the Pall friction damper is evaluated.

2. Using pall friction dampers in a 4-story steel frame with masonry-infill panels

Most of the old steel frames in many countries like Iran are simple frames with no lateral-resisting system. Only masonry-infill panels create some resistance against lateral loads. For the present study, Pall friction dampers are used to retrofit the simple steel frame with masonry-infill panels. On the basis of the Iranian Standard Seismic Code No. 2800, the following assumptions are applied in the modeling (BHRC 2005): the floor system is hollow tile, the building is a hospital in a region with high risk of earthquake, and the soil is type 3. Dead loads for the floors and roof are 650 and 600 kg/m^2, respectively, and live loads for the floors and roof are 300 and 150 kg/m^2, respectively. These values are determined on the basis of the Iranian National Building Regulation part 6: Loading. Table 1 presents information on the structural members.

Table 1 Properties of the simple frame members

Story	Beams	Interior columns	Exterior columns
4	IPE330 + PL120 × 5	BOX 100 × 5	BOX 100 × 5
3	IPE330 + PL130 × 10	BOX 150 × 10	BOX 120 × 8
2	IPE330 + PL130 × 10	BOX 150 × 10	BOX 120 × 10
1	IPE330 + PL130 × 10	BOX 200 × 10	BOX 150 × 10

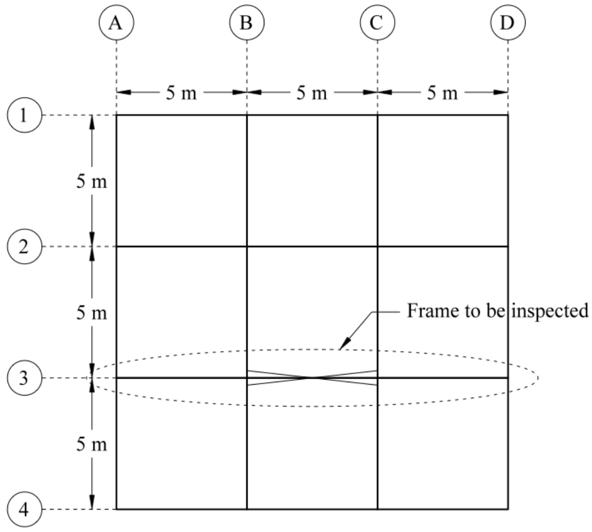

Fig. 3 Floor plan of the four-story steel building

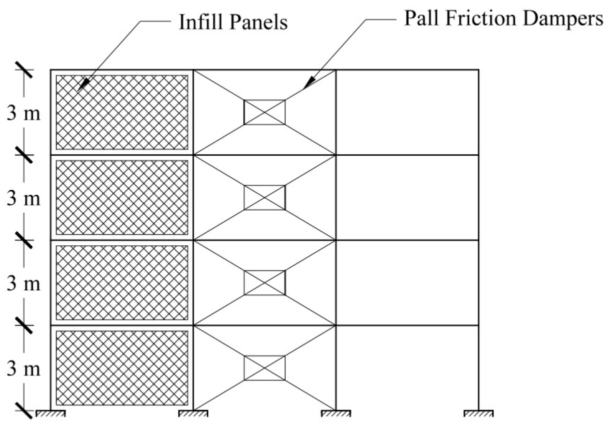

Fig. 4 Configuration of the infill panel and friction damper in the steel frame

Infill panels are made of solid brick with 10 cm and 20 cm thickness, in two different cases. Each span is 5 m long and 3 m high. Fig. 3 shows the floor plan of the building, where the specified frame is shown. Locations of the infill panels and Pall friction dampers are shown in Fig. 4.

3. Design of the pall friction damper

The crucial step in the design of a friction damper is determination of the optimum slip load. The movement of the damper in an elastic brace constitutes nonlinearity. Moreover, the amount of energy dissipation is proportional to the displacement. Therefore, nonlinear time-history dynamic analysis, which was used in the present study, is the most accurate procedure for finding the value of the optimum slip load. With this method, structural response can be evaluated during and after an earthquake (Fiore *et al*. 2012a).

The hysteresis loop of the damper is similar to the rectangular loop of a material with elastic perfectly plastic behavior (Fig. 1). Therefore, sliding load is considered as virtual yielding force in bracing. PERFORM-3D (CSI 2006) was used for nonlinear time-history dynamic analysis of the frame with the dampers.

In this study the performance of the frames under an earthquake with hazard level 1 (BHRC 2005) is reported. This hazard level designates, on the basis of the 2800 Iranian standard code third edition, an earthquake with a 10% probability of occurring within 50 years, equal to a return period of 475 years. Three accelerographs, information about which is shown in Table 2, were used in the analyses.

Table 2 Specification of the earthquake records. PGA, peak ground acceleration

Earthquake	Year	Station	PGA (g)	Soil type	Duration (sec)
Tabas	1978	9101	0.836	III	32.84
Imperial Valley	1979	Bonds Corner	0.588	III	37.61
Cape Mendocino	1992	89156 Petrolia	0.59	III	36

In the analyses, yielding stress of the braces in tension is set equal to the stress in the braces during sliding. The maximum displacements of the stories are considered to be the frame response in nonlinear dynamic analysis. This procedure was performed for different values of slip forces. The sliding design load, which corresponds to the minimum structural response, is considered the optimum sliding load.

Fig. 5 shows the maximum story drift in terms of sliding force under the Tabas earthquake, and Fig. 6 illustrates the hysteresis loops of the friction damper installed in the bracing of the frame under the Tabas earthquake.

As shown in Fig. 5, inter-story relative displacement is primarily reduced by increasing sliding load, but for sliding loads more than 25 tons, lateral drifts increases. Therefore, optimum sliding load is equal to 25 tons in all stories.

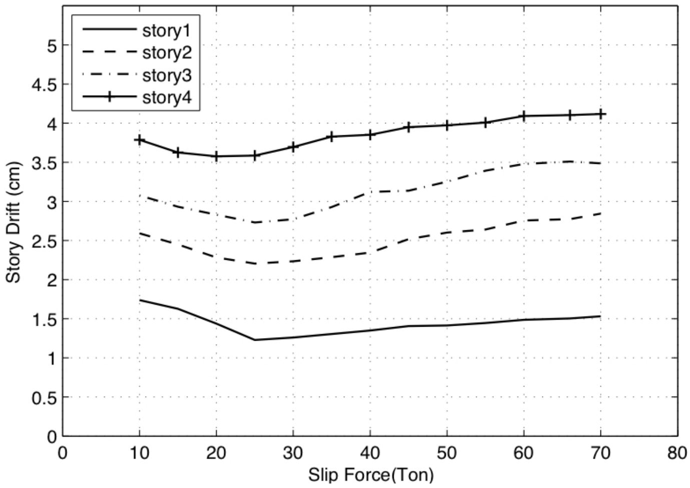

Fig. 5 Maximum story drift in terms of slip force under the Tabas earthquake

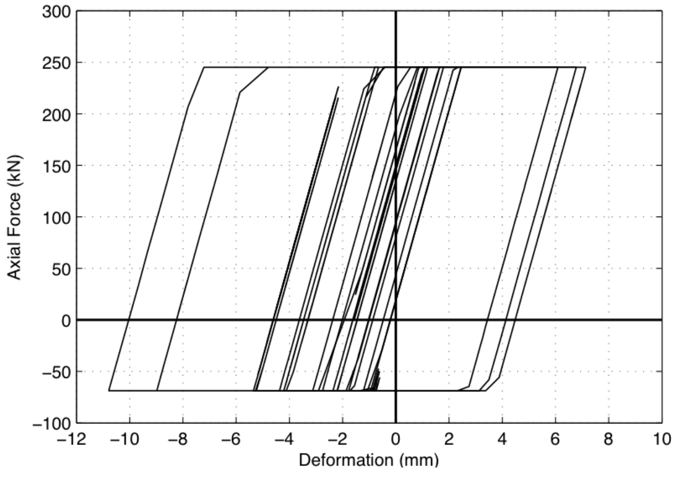

Fig. 6 Hysteresis loop of the friction damper under the Tabas earthquake

4. Stiffness of the masonry-infill panel

In PERFORM, diagonal struts of width *a* and thickness *t* are used to model infill panels. In this study, two infill panels with thicknesses of 10 and 20 cm were modeled, and the effect of their interaction with the friction damper on structural behavior was investigated. The procedure is based on FEMA 356 (2000). The area of the strut A_e and the value of *a* are calculated by means of Eqs. (1), (2), (3) and (4).

$$A_e = a.t \tag{1}$$

$$a = 0.254[\lambda_1 h]^{-0.4} r \tag{2}$$

$$\lambda_1 = \left[\frac{10 E_i t \sin 2\theta}{E_f I_c h}\right]^{0.25} \tag{3}$$

$$\theta = \arctan\left(\frac{h}{l}\right) \tag{4}$$

where

h, l, and t are the height, length, and thickness of the infill panel respectively,
r is diagonal length of the infill panel,
E_i and E_f are the elastic moduli of the infill panel and the frame, respectively,
I_c is the moment of inertia of the column.

The changes in the lengths of the beam and columns are negligible. Therefore, the stiffness of the frame is equal to

$$K = \frac{A_e E_f}{r}\cos^2\theta \tag{5}$$

In this frame, the height of the infill panel is 500 cm, and its length is 300 cm. Elastic modulus of the brick is 2,000,000 kg/cm^2, and its compressive strength is 50 kg/cm^2. According to the FEMA 356 (2000), the elastic modulus of masonry-infill panels is 550 times the brick compressive strength. Therefore, the elastic modulus of the masonry-infill panel in the present study was 27,500 kg/cm^2. Substituting these values into equation 4 reveals that the infill panels with thicknesses of 20 and 10 cm have lateral stiffness values of 39,400 kg/cm^2 and 21,100 kg/cm^2, respectively.

5. Seismic performance of friction dampers with masonry-infill panel

5.1 Energy dissipation

In this section, it is investigated whether infill panels fulfilled their intended functions during various earthquakes and allowed the friction dampers in dissipating the earthquake energy or if they cracked and degraded and therefore had no major contribution to building performance.

Table 3 and Fig. 7 show the capacity for nonlinear displacements of the structure, which determines the capacity for story drift. These values are based on the building seismic retrofitting code, FEMA 356 (2000).

In Fig. 7, "*Q*" is the generalized force in a component, "Q_y" is yield strength of a component, "Δ_{eff}" is differentiated displacement between the top and bottom of the wall h_{eff} is effective height of wall "*d*" and "*e*" are parameters used to measure deformation capacity, "*c*" parameter used to measure residual strength.

In Table 3, "V_f" represents the frame shear strength without infill panels, "V_i" is shear strength of the infill panels and "*d*" represents the nonlinear capacity, which should be represented as a story drift in percent (Fig. 7).

In this frame, the ratio of length to height of the infill panel is 1.66. The ratio of shear strength of the frame to shear strength of the infill panel is 1.3. The value of 1% for *d* is obtained by interpolation between the values of Table 3. In other words, nonlinear deformation capacity of the infill panel is 1% of the story height (3 cm), and the infill panel can have a deformation of 3 cm before its strength decreases. As Fig. 8 shows, the maximum story drift was less than 0.8% under the Tabas earthquake. Infill panels are therefore not degraded and can fulfill their function during earthquakes, and they dissipate part of the earthquake energy. Fig. 9 shows the contribution of the infill panel and friction damper toward total earthquake energy dissipation during earthquake. It can be seen that most of the energy dissipation occurs between second 5 and 10 of the earthquake

Table 3 Force-displacement relation (β) for the masonry-infill panel in the nonlinear dynamic method. d, nonlinear capacity, represented as a story drift in percentage; *l*, *h*, length and height of the infill panel; V_f, V_i, frame shear strength without and with infill panels

d (%)	*l*/*h*	$\beta = V_f/V_i$
0.5	0.5	
0.4	1	$\beta < 0.7$
0.3	2	
1	0.5	
0.8	1	$0.7 \le \beta \le 1.3$
0.6	2	
1.5	0.5	
1.2	1	$\beta \ge 1.3$
0.9	2	

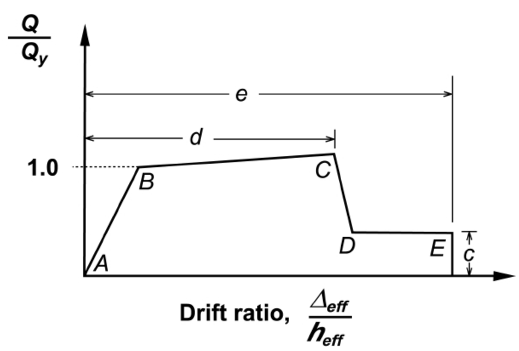

Fig. 7 Generalized force-deformation relations for masonry elements or components (FEMA 356 (2000))

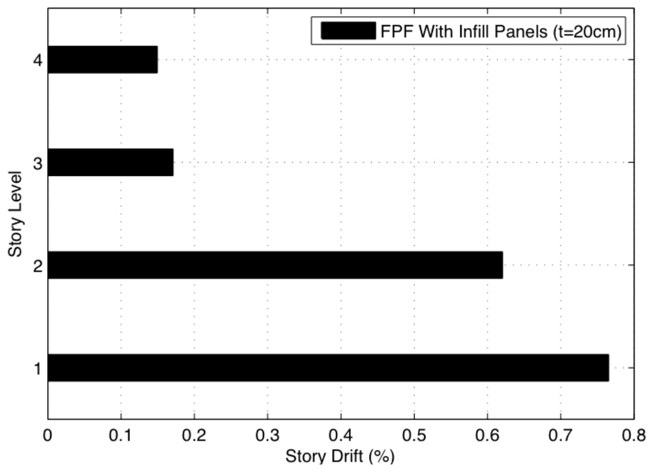

Fig. 8 Maximum story drift in friction-damper-pinned frame (FPF) with Infill panel with thickness of 20 cm (FPFI20) under the Tabas earthquake

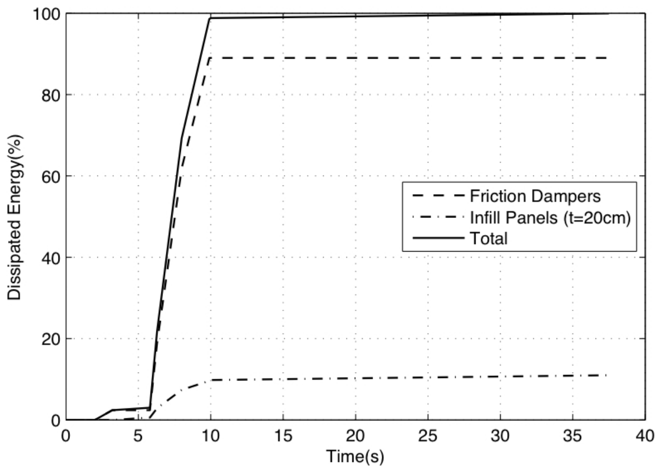

Fig. 9 Percentage of energy dissipated by friction dampers and the 20-cm-thick infill panels under the Imperial Valley earthquake

and infill panel has the considerable contribution on total energy dissipation. The percentage of energy dissipation for the friction dampers and infill panels with 20-cm thickness are 89 and 11%, respectively.

5.2 Lateral displacement

Figs. 10-11 show roof displacements in the frame with and without 20-cm infill panel under the Imperial Valley and Tabas earthquakes. It can be seen under both earthquakes that since frame with infill panel is stiffer than the one without, the former has the lower natural period than the latter. Lower natural period is the reason of lower story drift. Therefore, inclusion of infill panels in structures with friction dampers can be considered to reduce the drifts. Actually, an increase in

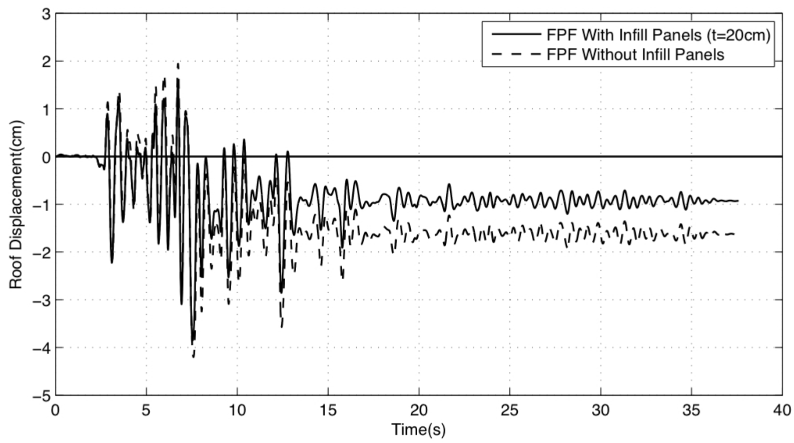

Fig. 10 Roof displacement versus time for FPF and FPFI20 under the Imperial Valley Earthquake

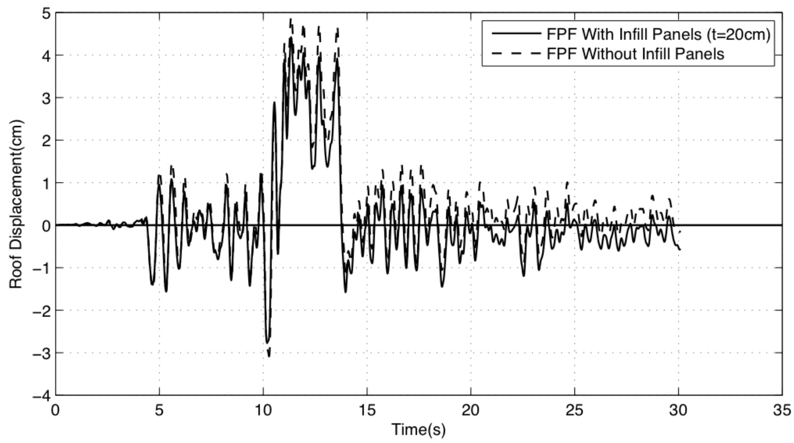

Fig. 11 Roof displacement versus time for FPF and FPFI20 under the TABAS Earthquake

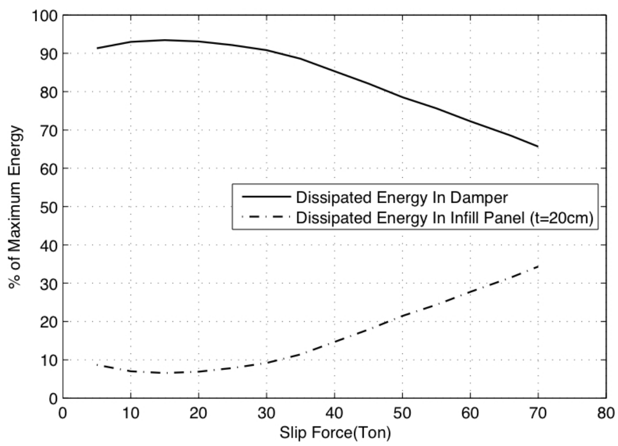

Fig. 12 Percentage of dissipated energy in damper and infill panel of FPFI20 in terms of slip load under the Tabas Earthquake

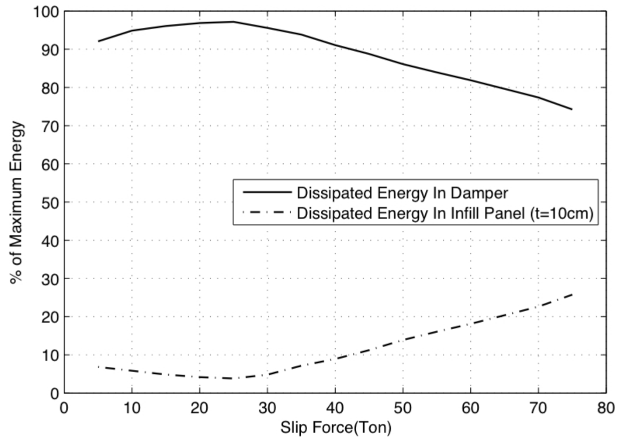

Fig. 13 Percentage of dissipated energy in damper and infill panel of FPFI10 in terms of slip load under the Tabas Earthquake

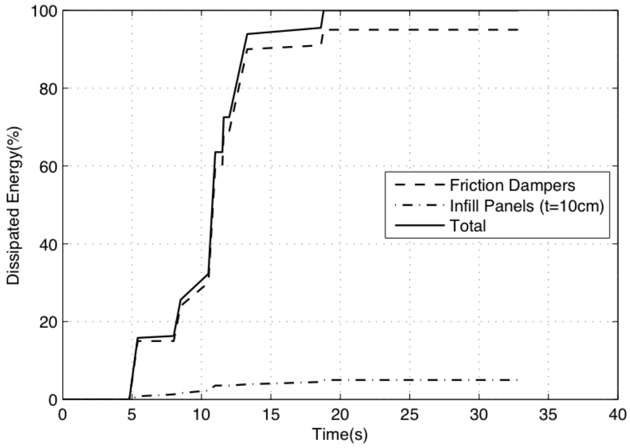

Fig. 14 Percentage of energy dissipated by 20-cm-thick Infill panel under the Tabas earthquake

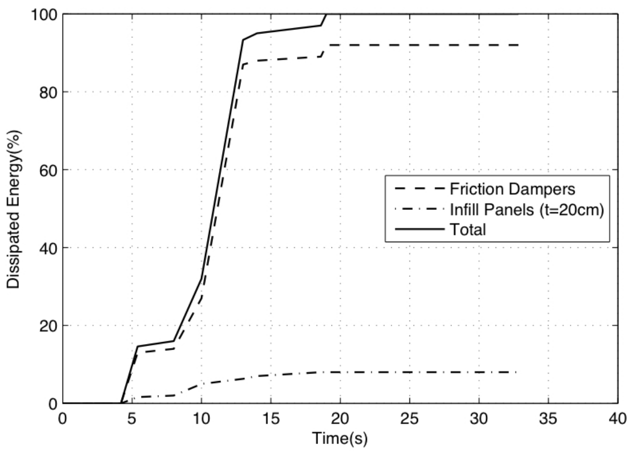

Fig. 15 Percentage of energy dissipated by 10-cm-thick Infill panel under the Tabas earthquake

the frame stiffness justifies this 12% to 20% reduction.

5.3 Impact of panel thickness on energy dissipation

In this section, the behavior of simple frames with friction dampers (FPF) having infill panels of 20-cm thickness (FPFI20) and of 10-cm thickness (FPFI10) is presented and compared.

Figs. 12 and 13 depict the percentage of energy dissipation by two thicknesses of infill panels (20-cm and 10-cm respectively) in terms of slip load under the Tabas earthquake. Figs. 14 and 15 also show the percentage of energy dissipation by two thicknesses of infill panels, 8% and 4% respectively. It can be seen that the thicker the infill panel, the more energy is dissipated due to its higher stiffness and proportional contribution.

Figs. 16 and 17 present the maximum story drift under the Tabas and Imperial Valley earthquakes, respectively. Because PGA and frequency content for the Tabas earthquake is higher

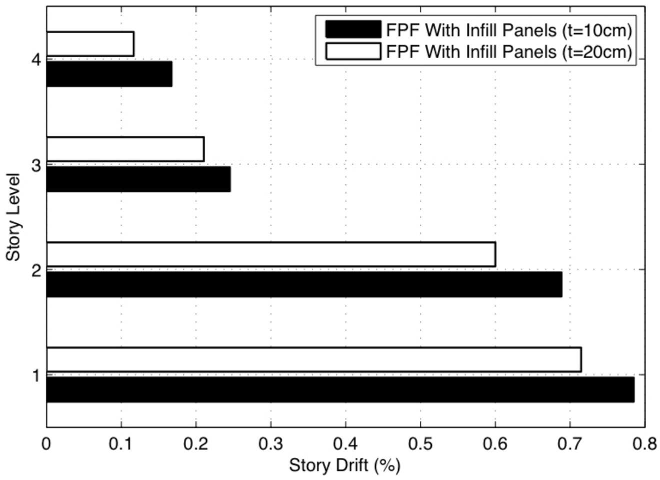

Fig. 16 Story drifts of the FPFI10 and FPFI20 under the Tabas earthquake

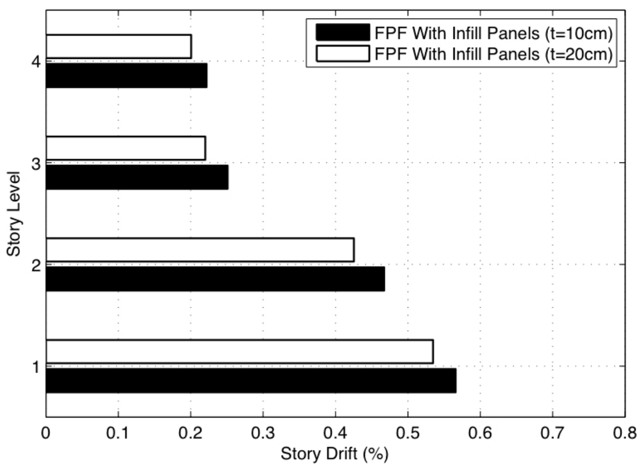

Fig. 17 Story drifts of the FPFI10 and FPFI20 under the Imperial Valley earthquake

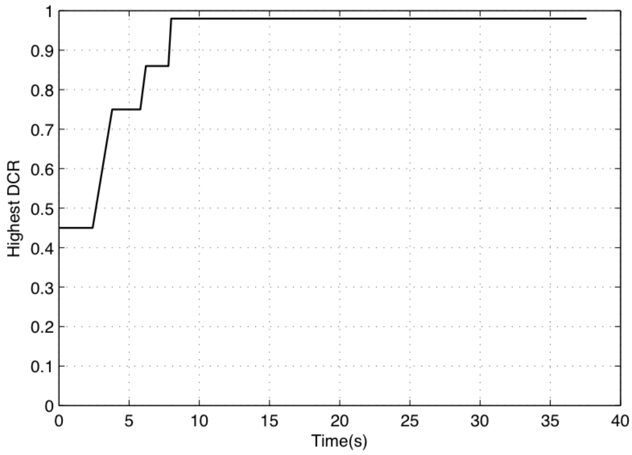

Fig. 18 Highest DCR versus time for the FPF under the Imperial Valley Earthquake

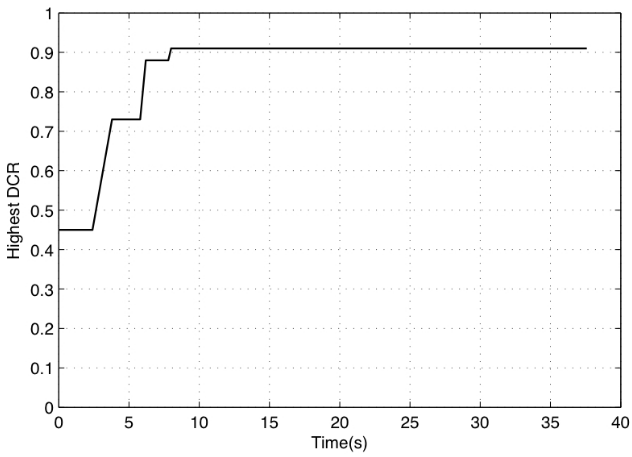

Fig. 19 Highest DCR versus time for the FPFI20 under the Imperial Valley Earthquake

than those for the Imperial Valley earthquake, the frame has responded more under the Tabas earthquake. Moreover, as shown in these figures, increasing the thickness of the infill panel can be considered to reduce the story drift by 10% and to improve the performance of the frame with friction dampers. Larger drifts at lower stories are due to the fact that seismic shear forces are larger at lower floors and thus stronger and/or stiffer damper and infill panels might be needed to control drift.

6. Column demand/capacity ratios

The highest column demand/capacity ratios under the Imperial Valley earthquake were compared for the frame without infill panels (Fig. 18) and the frame with infill panel, t = 20 cm (Fig. 19). The simulations show that the highest column demand/capacity ratio in FPFI20 (which is 0.91) is 8% lower than that in FPF (which is 0.98). Actually, infill panels reduce the amount of the axial force, which is developed in the columns. This is another evidence which demonstrates that infill panels in a building with friction dampers improve the performance of the structure during earthquakes.

7. Conclusions

A simple 4-story steel frame with infill panels 20 cm and 10 cm thick was retrofitted by adding Pall friction dampers. Nonlinear time history dynamic analyses revealed the optimum sliding load under conditions of the Imperial Valley, Tabas, and Cape Mendocino earthquakes, and the effect of infill panels on performance of the Friction-damper Pinned Frame (FPF) was investigated. Results are apparent as following:

(1) If the story drift in the building is less than 1%, there is no considerable damage and infill panels improve the performance of the FPF.
(2) If the story drift was more than 1%, considerable damage occurs in the building and the simple frame without infill panels resist against earthquake loads.
(3) The friction dampers can begin sliding at a maximum drift of 0.8%. As it was also mentioned, for drift less than 1%, the infill panels improve the performance of the FPF. Therefore for drift less than 1% the infill panels and Pall friction dampers work together to dissipate energy.
(4) The contribution of the infill panels varies, with thickness of the panel (10 to 20cm), from 4 to 10% of total energy.
(5) Drift in FPFI20 is 12% to 20% less than that in simple FPF, which means using infill panels in FPF increase the stiffness and thus reduce the roof drift by up to 12% and improve the performance of the structure during earthquake.
(6) Infill panels decrease the forces acting on the columns which makes the columns demand/capacity ratio for FPFI20 becomes 4% to 7% smaller than that of simple FPF and FPFI10.

In general, frames with infill panels show better and more reliable seismic performance. Infill panels help friction dampers in dissipating earthquake energy. Based on thickness of the infill panels, from 4% to 10% of the earthquake energy is dissipated by infill panels and 90 to 96% of that is dissipated by friction dampers.

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