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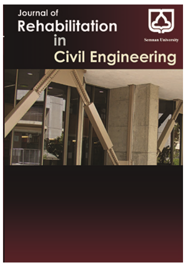

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Evaluating Equivalent Damping and Response Modification Factors of Frames Equipped by Pall Friction Dampers

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ABSTRACT

A study on the evaluating equivalent damping and response modification factors of frames equipped by Pall Friction Dampers is presented. To do so, buildings with various stories were considered. Nonlinear and linear dynamic time history analysis has been performed to evaluate equivalent damping for Pall Friction Dampers. Ground motions with various frequency characteristics scaled with Iranian earthquake resistant design code. In addition, nonlinear incremental dynamic analysis has been performed to evaluate the response modification factors. In this article, equivalent damping and seismic response modification factor for moment resisting frames with and without Pall Friction Dampers have been determined separately. The governing parameters were identified and their influence was traced and summarized along with implications for practical design. The results show that the equivalent damping of these frames using damper is higher than frames without it. Also we have this result for the response modification factors. It was also found that the number of stories have a great effect on these two characteristics of the buildings.

1. Introduction

In recent years, research has been focused on the development of various damping mechanisms intended to provide positive control of structural vibration induced by earthquake. Of all the methods so far available to extract kinetic energy from a moving body, the most widely adopted is undoubtedly the friction brake. Mechanical engineers have successfully used this concept for centuries to stop the motion of equipment, automobiles, railway trains, airplanes etc. It is an effective, reliable and economical mean to dissipate kinetic energy. Similar to automobiles, the motion of vibrating building can be slowed down by dissipating seismic energy in friction. Inspired by the principle of friction brake in mid1970's, Pall Friction Dampers were pioneered for the seismic control of buildings. Pall Friction Dampers significantly reduce the initial cost of construction while dramatically increasing the earthquake resistance against damage.

Over the years, Pall Dynamics has earned an international reputation for excellence and is a world leader in friction dampers for seismic control of buildings. Pall Friction Dampers have successfully undergone rigorous proof testing in the U.S and Canada. In 1985, the National Research Council of Canada tested 3-story frame structures on a shaking table at the University of British Columbia, Vancouver [1]. In 1986-1987, the U.S. National Science Foundation tested a 9-story frame structure on a shaking table at the University of California at Berkeley [1]. The structures were subjected to more than 20 different major earthquake records. Even for an earthquake 5 times stronger than the 1985-Mexico earthquake, the frames equipped with friction dampers remained damage free. Pall Friction Dampers are well recognized and accepted by the building codes in Canada, U.S and many other countries. Salient Features Pall Friction Dampers are foolproof in construction. Basically, these consist of series of steel plates, which are specially treated to develop very reliable friction. These plates are clamped together and allowed to slip at a predetermined load. Decades of research and testing have led to perfecting the art of friction. Their performance is reliable, repeatable and they possess large rectangular hysteresis loops with negligible fade. Their performance is independent of velocity and hence exerts constant force for all future earthquakes, design-based earthquake (DBE) or maximum credible earthquake (MCE). A much greater quantity of energy can be dissipated in friction than any other method involving the yielding of steel plates, viscous or viscoelastic dampers. Therefore, fewer Pall Friction Dampers are required to provide the required amount of energy dissipation. Pall Friction Dampers are passive energy dissipation devices and, therefore, need no energy source other than earthquake to operate it.

A lot of investigations on friction devices has been done, e.g. Pall and Marsh [2], Aiken and Kelly [3], Fitzgerald et al. [4], Constantinou et al. [5], Grigorian and Popov [6], Nims et al. [7], and many of them have been implemented in buildings around the world. Much research was devoted to developing the theory of passive control systems as well.

Definitely, due to its proven efficiency and simplicity, the concept of seismic protection based on friction damping systems is gaining momentum within the engineering community worldwide. Many studies have been conducted in our country. Since this is a relatively new topic of research in construction industries in Iran, it is still needed to be studied. Integh elahi and Jalali [8] have investigated on the retrofitting of reinforced concrete structures equipped with dampers in 2002. Zahrai and Kheirolahi [9] focused on the role of friction dampers on improving seismic friction on a five floor-steel building in 2006. In 2008, Shariatmadar and Sadeghi [10] have studied about evaluating slip load spectrum and attitude of Pall Friction Damper's design. Abedini [11] researched about analysis and design of steel frame with friction damper in 2009.

In this paper the effectiveness of the damping system employing Pall Friction Damper in frames in 3, 5 and 7 stories are evaluated. Regardless of how to obtain the slip load of Pall Friction Damper that are described in the studies described above extensively, will concentrate on equivalent damping and response modification factors of frames caused by using Pall frictional dampers.

2. Pall Friction Dampers

Pall Friction Dampers significantly reduce the initial cost of construction while dramatically increasing the earthquake resistance against damage. Pall friction dampers are customized to suit site conditions and allow greater adaptability than is possible with other systems. These dampers can be bolted or welded into place. Pall Friction Dampers are available for long slender tension-only cross bracing, single diagonal tension compression bracing and chevron bracing Fig.1. The damper for cross bracing is a unique mechanism. When one of the brace in tension forces the damper to slip, the damper mechanism forces the other brace to shorten and thus avoid buckling. In this manner, the other brace is immediately ready to slip the damper on reversal of cycle. This cyclic motion continued during the earthquake and prevents transmission of loads to other structural member. Modeling Pall Friction Damper in software and achieving the value of Pall's slip load is really important, which are described further briefly.

All buildings try to damp the lateral loads they are encountered. It is clear that if there is a way to increase the structural system's damping, it would be a major step to reduce the dynamic response. The present study focused on using Pall dynamic friction damper and its influence on increasing damping. In a typical undamped structure, the inherent damping is merely 1-5% of critical. With the introduction of Pall Friction Dampers, structural damping of 10-30% of critical can be easily achieved. As the dampers dissipate a major portion of the seismic energy, forces and deformations on the structure are significantly reduced [1].

Seismic design codes consider a reduction in design loads, taking advantages of the fact that the structure posses significant reserve strength and capacity to dissipate energy which are named overstrength and ductility respectively. The overstrength and ductility are incorporated in structural design through a force reduction or a response modification factor. This factor represents ratio of maximum seismic force on a structure during specified ground motion if it was to remain elastic to the design seismic force. Thus, actual seismic forces are reduced by the factor "R" to obtain design forces. The basic flaw in code procedures is that they use linear methods but rely on nonlinear behavior [12].

The response modification factors were first proposed in ATC3-06, ATC-19 and ATC-34, was calculated as the product of three factors: Over-Strength factor, Ductility factor, and Redundancy factor [13]. The response modification factors for special moment resisting frames with Pall devices should be relatively computed, defining the system according to its ductility and performance in a manner consistent with the factors already established for ordinary moment resisting frames. The present study focuses on the evaluation of over-strength, force reduction due to ductility and response modification factors of ordinary moment resisting frames (OMRFs) and Pall equipped ordinary moment resisting frames (P-OMRFS), designed in accordance with Iranian code of particle for seismic resistance design of buildings [14] (BHRC, 2005) and Iranian National Building Code (Part 10) for Structural Steel Design [15]. Nonlinear Incremental Dynamic Analysis and linear dynamic analysis were carried out to obtain such factors.

Fig. 1. Pall Friction Dampers

2.1. Design of Pall Friction Dampers

The computer modeling of Pall Friction Dampers is easy. Since the hysteretic loop of the friction dampers is perfectly rectangular, similar to perfectly elasto-plastic material. The friction dampers can be modeled as fictitious plasticity element having yield force equal to slip load.

The single diagonal tension/compression brace with friction damper Fig. 2. can be modeled as a damped brace using the following link object.

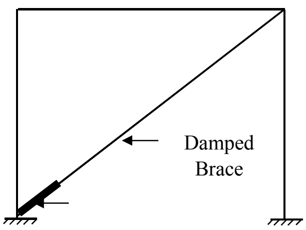

Fig. 2. Single Diagonal Tension-Compression Brace with Pall Friction Damper

The Cross Brace with Pall Friction Damper Fig. 3. can be modeled as said for the single diagonal tension/compression brace and the hysteretic loop of each tension brace is equal to hysteretic loop of one single diagonal tension-compression brace having half the slip load. However, the brace and the connections should be designed considering the full slip load. [13].

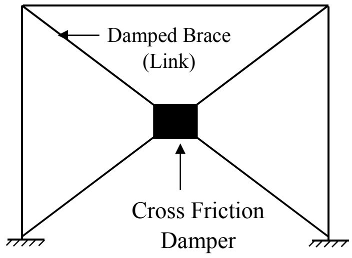

Fig. 3. Tension Only Cross Brace with Pall Friction Damper

The chevron friction damper Fig. 4. can be modeled also using the link object. The brace is modeled as frame element. Braces are from joints A and E and joints B and E. The beams at top are from joints C and D and joints D and F. The friction damper is modeled as a nonlinear axial link element between joints D and E. Make sure that joint E is disconnected from the diaphragm otherwise the damper will not work or move.

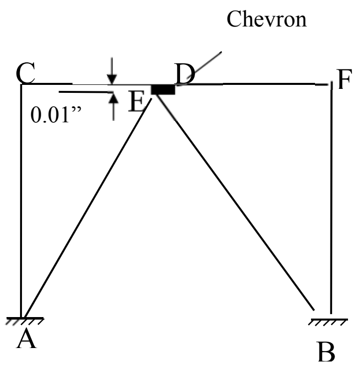

Fig. 4. Chevron Brace with Pall Friction Damper

2.2. Slip Load of Friction Damper

The friction dampers are designed not to slip during wind. During a major earthquake, they slip prior to yielding of structural members. In general, the lower bound is about 130% of wind shear and the upper bound is 75% of the shear at which the members will yield [1]. As seen in Fig. 5, if the slip load is very low or very high, the response is very high. Several parametric studies have shown that the slip load of the friction damper is the principal variable with the appropriate selection of which it is possible to tune the response of structure to an optimum value. Optimum slip load gives minimum response. Selection of slip load should also ensure that after an earthquake, the building returns to its near original alignment under the spring action of an elastic structure. Studies have also shown that variations up to ±20% of the optimum slip load do not affect the response significantly. Therefore, small variations in slip load (8- 10%) over life of the building do not warrant any adjustments or replacement of friction damper [1].

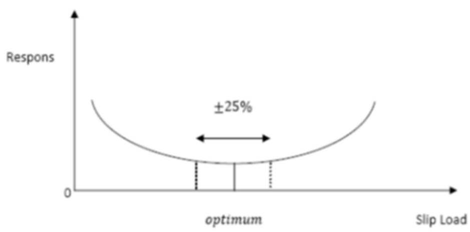

Fig. 5. Response versus Slip Load

As this article is not about evaluating slip load of frictional dampers, although it is an important issue, further study in this regard is deposited to reader.

3. Equivalent damping

In this study, in order to calculate the equivalent damping, all under-studied ordinary moment resisting frames equipped with Pall Friction Dampers are investigated. Always in the linear analysis, the value of base shear load obtained for a given frame is more than the value obtained in the nonlinear analysis. This issue has been used in calculating the equivalent damping of frames with Pall Friction Damper.

4. Response modification factor

Elastic analysis of the structures under earthquake can create base shear force and stresses which are so noticeably bigger than real structural response. Overstrength in structures is related to the fact that the maximum lateral strength of a structure generally exceeds its design strength. Hence, seismic codes reduce design loads, taking advantage of the fact that structures possess overstrength and ductility. In fact the response modification factor includes inelastic performance of structure and indicates overstrength and ductility [12].

Mazzolani and Piluso [16] addressed several theoretical approaches such as maximum plastic deformation, energy and low cycle fatigue approaches to compute response modification factor. As it is shown in Fig. 6, usually real nonlinear behavior is idealized by a bilinear elasto-plastic relation. The yield force of structure is shown by v_y and the yield displacement is Δ_y. In this figure V_e $(V_{\max})$ correspond to the elastic response strength of the structure. The maximum base shear in an elasto perfectly behavior is V_y [12]. The response modification factor is determined as follows [14]:

$$R=R_{\mu}.R_S \tag{1}$$

where R_u is a reduction factor due to ductility and R_s and R_s is the overstrength factor.

4.1. Reduction factor due to ductility

R_u is a parameter that measures the global nonlinear response of a structure, due to the hysteretic energy. The ratio of maximum base shear considering elastic behavior V_e to maximum base shear in elasto perfectly behavior V_y is called force reduction factor due to ductility:

$$R_u = \frac{V_e}{V_y} \tag{2}$$

Several proposals have been put forward for R_u. In a simple version of the methods, proposed by Fajfar [17], the reduction factor R_u is written as:

$$\begin{aligned} R_u &= (\mu-1)\frac{T}{T_c}+1 \qquad (T<T_c) \\ R_u &= \mu \qquad (T \geq T_c) \end{aligned} \tag{3}$$

where, T is the fundamental period, T_c is the characteristic of ground motion equal to 0.5 for the soil type II that has been considered here based on the Iranian Earthquake Resistance Design Code (Standard No. 2800) [14] and μ is the structural ductility factor defined as:

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{4}$$

where, Δ_{max} is the maximum displacement for the first life safety performance in structure and Δ_y is the yield displacement observed there.

4.2. Overstrength factor

As observed in some of the intermittent quake incidents, it seems building structures could take the forces considerably larger than those were designed for. The presence of significant reserve strength that was not accounted in design, explains this phenomenon [18]. Overstrength helps structures stand safely not only against sever tremors but reduces the elastic strength demand, as well. This object is performed using the force reduction factor [19]. The design overstrength factor (R_{sd}) is defined as follows [18].

$$R_{sd} = \frac{V_y}{V_d} \tag{5}$$

Here, V_d is the design base shear in the building and V_y is the base shear in relevance to the first life safety performance in structural members. The concept of overstrength, redundancy and ductility, which are used to scale down the earthquake forces need to be clearly defined and expressed in quantifiable terms.

In this equation, the overstrength factor is based on the applied nominal material properties. Meanwhile, the actual overstrength factor should consider the help of some other effects [12]:

$$R_s = R_{sd}.F_1.F_2...F_n. \tag{6}$$

In this equation, F_1 is used to account for difference between actual static yield strength and nominal static yield strength. For structural steel, a statistical study shows that the value of F_1 may be taken as 1.05

[20]. Parameter F_2 may be used to consider the increase in yield stress as a result of strain rate effect during an earthquake excitation. A value of 1.1, a 10% increase to account for the strain rate effect, could be used [12]. In this paper the steel type St-37 was used for all structural members. Parameter F_1 and F_2 equal to 1.05 and 1.1 were considered taking into 1.155 as material overstrength factor. Other parameters such as nonstructural component contributions, variation of lateral force profile could be included once a reliable data is available [12].

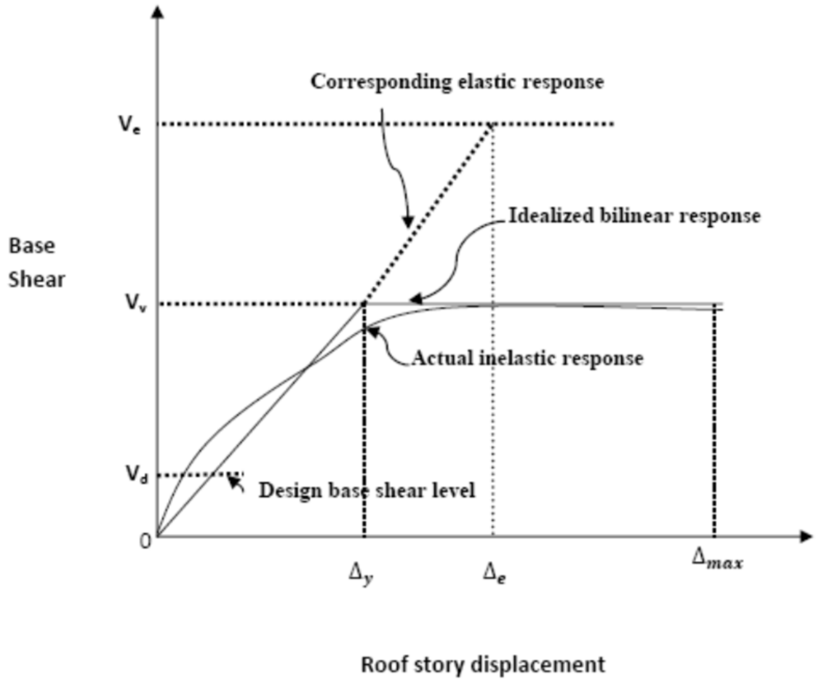

Fig. 6. Lateral load-roof displacement relationship of a structure

5. Structural models

To evaluate the equivalent damping, overstrengh, ductility and response modification factors of ordinary resisting frames with pall Friction Dampers, 3, 5 and 7 story buildings with the bay length of 4m were designed as per the requirement of Iranian earthquake resistance design code [14] and Iranian national building code, part 10, steel structure design [15]. The story height of the models was considered as 3m. The dead and live loads of 6 and 2 kN/m^2 were used. The important factors are presented in the Table 1.

For the nonlinear modeling of the structural frames the required performance level must be selected. The level of building performance is being determined based on structural and non-structural components. Structural components include components of the lateral and vertical load resistance that includes the beam, column and bracing. Non-structural components are such as mechanical, electrical and other instrument. In this study, the performance level of life safety for structural components is selected for components.

The life safety performance level definition refers to the prediction that damage in the structure occurs caused by earthquake, but the failure rate is not enough to cause life's damage. At this level of performance structural and non-structural would be damaged. Additional load-bearing capacity and lateral stiffness decreases, while the building still has a resistance to collapse.

Using time history non-linear design of structures selecting the life safety performance level for the frames, sections of the selected frames are presented in Tables 2, 3 and 4.

Also according to the explanations given about slip load design of the Pall Friction Damper, using the section of 2UNP100 for bracings Slip load design of the Pall Friction Damper value of the structures 3, 5 and 7 floors respectively are 12, 17 and 20 tons. In addition it must be noticed that Specifications of the selected acceleration records are presented in the table 5.

Table 1. The important factors of modeling

Story	7	5	3
Importance factor (I)	1	1	1
Based acceleration scheme (A)	0.35	0.35	0.35
Response modification factor (R)	7	7	7
Dead load on floor (kg/m^2)	600	600	600
Live load on floor kg/m^2	200	200	200
Weight (ton)	645.12	460.8	276.48
The main period of construction (T)	0.7848	0.6097	0.4157
Seismic coefficient (C)	0.093	0.1095	0.125

Table 2. 3-story frames sections

story	Columns (cm)	beams (cm)
1	20*20*1.2	30*1.5-45*1.5
2	20*20*1.2	35*1.2-25*1.5
3	20*20*1.2	28*1.2-20*1.2

Table 3. 5-story frames sections

story	Columns (cm)	beams (cm)
1	20*20*2	50*1.5-35*1.5
2	20*20*2	45*1.5-35*1.5
3	20*20*2	45*1.5-30*1.5
4	20*20*1.2	35*1.2-25*1.5
5	20*20*1.2	28*1.2-20*1.2

Table 4. 7-story frames sections

story	Columns (cm)	beams (cm)
1	20*20*2	50*1.5-35*1.5
2	20*20*2	50*1.5-35*1.5
3	20*20*2	45*1.5-35*1.5
4	20*20*1.2	45*1.5-35*1.5
5	20*20*1.2	45*1.5-30*1.5
6	20*20*1	35*1.2-25*1.5
7	20*20*1	28*1.2-20*1.2

Table 5. Specifications of the selected acceleration records

no	Distance from the fault	Magnitude	Date of occurrence	Record station	Earthquake	Specification
1	>20	7/4	1978	Dayhook	Tabas	P0140
2	22/6	6/7	1994	Old Ridge Route	Northridge	P0883
3	21/4	6/9	1989	Anderson Dam	Loma	P0743

6. Results

6.1. Equivalent damping

The method used in calculating the equivalent damping, is as follows:

At first base shear load for each of the Pall Friction Dampers equipped frames are achieved affected by Tabas spectrum by the maximum acceleration of 0.35g while nonlinear behavior for materials is defined.

Then, each of the frames is analyzed sequential steps in a dynamic time history cases while linear behavior for materials is defined. In each step damping would be increased till achieve to a base shear load equal to one achieved by defining nonlinear behavior for materials. Finally damping ratio which by base shear load of the frame equipped with Pall Friction Damper in the state of nonlinear analysis is equal to base shear load of linear analysis of that frame, is the equivalent damping caused by additional frictional damper. Equivalent damping percent of the frames equipped with dampers are presented in tables 6 till 8.

It is important to consider that the equivalent determined damping is in excess of 5% damping intended for general analysis of the frames. At the end of this section to investigate the effect of damping on the number of stories of the frames mean value of the equivalent damping are compared in Table 9.

Table 6. Equivalent percent damping in 3-stories frame

Tabas earthquake (slip load=12ton)			
linear-analysis (kN)	Damping%	Non-linear-analysis	Equivalent damping %
623.5	10	847.22	19.5
	15	674.07	
	17	649.77	
	16	661.72	
	18	638.20	
	19	627.00	
	19.5	623.4	
	20	616.1	
Northridge earthquake (slip load=11ton)			
linear-analysis (kN)	Damping%	Non-linear-analysis	Equivalent damping %
619.67	5	828.54	31.2
	10	769.61	
	15	737.29	
	20	664.23	
	30	627.59	
	31.2	619.45	
	32	613.38	
C) Loma earthquake (slip load=9ton)			
linear-analysis (kN)	Damping%	Non-linear-analysis	Equivalent damping %
595.88	5	1495.36	35.4
	10	916.68	
	20	741.32	
	30	638.22	
	35	599.55	
	35.4	595.83	
	36	592.50	

Table 7. Equivalent percent damping in 5-stories frame

Tabas earthquake (slip load=17ton)			
linear-analysis (kN)	Damping%	Non-linear-analysis	Equivalent damping %
861.68	5	1582.37	16.2
	10	1177.04	
	15	914.83	
	16	873.45	
	16.2	862.54	
	16.5	853.87	

Table 8. Equivalent percent damping in 7-stories frame

Tabas earthquake (slip load=20ton)			
linear-analysis (kN)	Damping%	Non-linear-analysis	Equivalent damping %
968.13	5	1703.16	12.2
	10	1113.61	
	12	980.83	
	12.2	969.43	
	12.5	952.87	
	13	926.62	

Table 9. Equivalent percent damping in frame

story	Equivalent damping %
3	19.5
5	16.2
7	12.2

To calculate v_y, the Incremental Nonlinear Dynamic Analysis of the models subjected to strong ground motions, matched with the design spectrum, was carried out. The response spectrums of the time history of Tabas, Northridge and Loma earthquake are considered. In this analysis under above-mentioned time histories, their PGA's with several try and errors had changed in a way that the gained time history resulted in the structure reaching to the life safety structural performance level as well as the nonlinear behavior of elements as suggested by FEMA-356[10] Fig. 7. The maximum nonlinear base shear of this time history is the inelastic base shear of structure [12]. Finally the material overstrength factor of 1.155 was considered for actual overstrength factor.

The values of the overstrength factor R_s for frames with and without dampers are presented in tables 10 and 11.

To calculate R_μ, the nonlinear dynamic analysis and linear dynamic analysis were carried out. By the use of nonlinear dynamic analysis and try and error on PGA of earthquake time histories, the nonlinear base shear v_y was calculated as described. Then by Linear dynamic analysis of the structure under the same time history, the maximum linear base shear v_e was calculated and finally the ductility reduction factor was evaluated [12]. The evaluated values are presented in the tables 12 till 17.

In this section with the achieved parameters affecting the response modification factor response, modification factor of the ordinary moment frames in two cases of with and without dampers are calculated. Results are presented in Tables 18 and 19.

As it can be understood of results usage of Pall Friction Damper increases the response modification factor. Also decreasing of the building's height causes the response modification factor's increase.

The response modification factor of ordinary frames is presented in the Iranian Earthquake Resistance Design Code (Standard No. 2800). The response modification factor which is proposed in this standard for the ordinary frames without dampers is 7, while with presented analyzes the mean value of the response modification factor for the frames in 3, 5 and 7 floors is 5.14. So the difference in calculated response modification factor and what proposed in standard 2800 is about 26%. Although this difference is not small, but with attention to lack of considering height of the structures, dimensions and section of structural members in the standard 2800 and design assumption, it is justified.

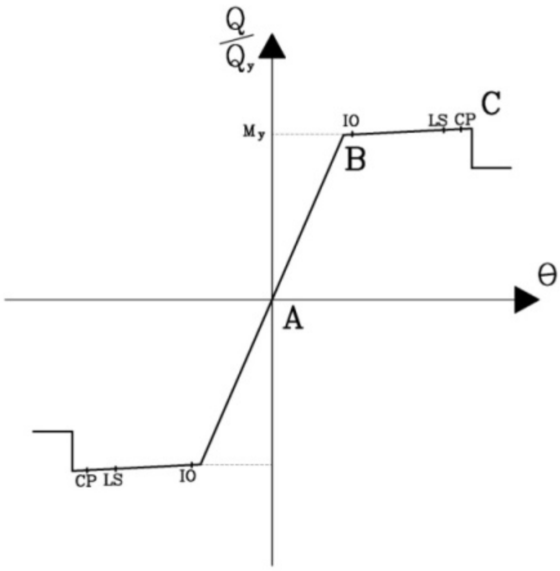

Fig. 7. Generalized force-deformation relation for steel elements (FEMA-356)

Table 10. The value of the overstrength factor R_s for frames without dampers

story	$V_d(kN)$	$V_y(kN)$	Ω	R_s
3	112.50	414.65	3.686	4.257
5	164.25	592.73	3.609	4.168
7	223.30	724.81	3.246	3.749

Table 11. The value of the overstrength factor R_s for frames with dampers

story	$V_d(kN)$	$V_y(kN)$	Ω	R_s
3	112.50	726.72	6.460	7.461
5	164.25	963.94	5.869	6.778
7	223.30	980.87	4.390	5.073

Table 12. Mean value of the base shear load equivalent to the first yielding point V_y in the frames without dampers

story	Tabas	Northridge	Loma	$V_y(avg.)KN$
3	427.99	395.15	420.8	414.65
5	615.29	574.93	587.96	592.73
7	787.49	647.12	739.82	724.81

Table 13. Mean value of the base shear load equivalent to the first yielding point V_y in the frames with dampers

story	Tabas	Northridge	Loma	$V_y(avg.)KN$
3	785.67	708.52	685.96	726.72
5	939.31	1021.78	930.73	963.94
7	1018.99	1062.67	980.96	1020.87

Table 14. Mean value of the base shear load equivalent to the final point V_e in the frames without dampers

story	Tabas	Northridge	Loma	$V_y(avg.)kN$
3	536.13	492.87	463.38	497.46
5	829.87	704.16	772.84	768.96
7	903.45	914.47	1027.56	948.49

Table 15. Mean value of the base shear load equivalent to the final point V_e in the frames with dampers

story	Tabas	Northridge	Loma	$V_y(avg.)kN$
3	2123.09	1875.53	2346.32	2114.98
5	2896.65	2127.75	2748.29	2424.23
7	2983.74	2644.61	2943.01	2857.2

Table 16. Value of the ductility reduction factor R_μ in the frames without dampers

story	$V_e(kN)$	$V_y(kN)$	R_u
3	497.46	414.65	1.200
5	768.96	592.73	2.297
7	948.49	724.81	1.308

Table 17. Value of the ductility reduction factor R_μ in the frames with dampers

story	$V_e(kN)$	$V_y(kN)$	R_u
3	2114.98	726.72	2.910
5	2424.23	963.94	2.515
7	2857.2	1020.87	2.80

Table 18. Value of the response modification factor R for frames without dampers

story	R_u	R_s	R

3	1.200	4.257	5.108
5	2.297	4.168	5.406
7	1.308	3.749	4.904

Table 19. Value of the response modification factor R for frames with dampers

story	R_u	R_s	R
3	2.910	7.461	21.711
5	2.515	6.778	17.047
7	2.80	5.073	14.204

7. Conclusion

- Usage of Pall Friction Damper increases damping of the frame and decreases percentage of destruction of non-structural components encountering earthquake.
- Effectiveness of Pall Friction Damper is more noticeable in shorter buildings.
- Usage of Pall Friction Damper increases response modification factor of the frame and decreases percentage of destruction of non-structural components encountering earthquake.
- Effectiveness of Pall Friction Damper is more noticeable in shorter buildings, the more the stories the less the damper increases response.

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