

TORSION MITIGATION OF EXISTING ASYMMETRIC STRUCTURES USING DAMPER

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ABSTRACT

Many existing buildings are irregular in plan or elevation because of asymmetric placement of masonry infills. The stiffness of masonry infill is a considerable value relating to that of the structure. Produced torsion from eccentricity because of infill stiffness leads to extra forces and deformations in structural members and diaphragms. An appropriate alternative to solve this problem especially in existing buildings is using dampers. Dampers can enhance structural performance by reducing seismically induced lateral displacements and by reducing inelastic behavior of beams and columns. In this paper some simple models are used to show structural modeling and a conceptual discussion is presented on the numerical results. An accurate model for masonry infill has been used in structural model. Numerical results show the efficiency and high performance of added dampers to reduce the torsion effects in the structural elements.

Keywords: Infill wall; damper; push-over; torsion

1. INTRODUCTION

The response of plan asymmetric structures during previous earthquakes have shown that the elastic and inelastic deformation demand may tend to concentrate in a few resisting planes [1]. As a result, design codes incorporate procedures to account for such irregular plan-wise displacement distribution, leading to different stiffness's and capacities of resisting planes. The inelastic seismic response of a class of one-way torsionally unbalanced structures is presented By Bozorgnia et al. [2], Tso [3] shown that much better correlation exists between inelastic torsional responses and strength eccentricity than the traditionally used stiffness eccentricity parameter. Tso

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and Ying [4] used a single mass three-element model; a study was made on the effect of strength distribution among elements on the inelastic seismic responses of eccentric systems. Additional ductility demands on elements and additional edge displacements are taken as response parameters of interest in optimizing the strength distribution. Previous research [5] showed that by controlling the strength of resisting planes, the inelastic properties of the building represented by the ultimate story shear and torque surface may be modified in order to reduce the inelastic rotations of the building plan. Once lateral-torsional coupling is controlled in the structure, the problem transforms into that of a nominally symmetric structure, implying simpler design procedures, more efficient use of structural members, and more reliable structures. Recently, the concept of "torsional balance" has been introduced by De la Llera et al. [6]. Yoon and Stafford Smith [7] presented a method to predict the degree of translational-torsional coupling of mixed-bent-type multistory building structures subject to dynamic loading.

Chopra and De la Llera [8] focused on the description of two recently developed procedures to incorporate the effects of accidental and natural torsion in earthquake analysis and design of asymmetric buildings. Basu and Jain [9] presented the definition of center of rigidity for rigid floor diaphragm buildings has been extended to unsymmetrical buildings with flexible floors. Stefano et al. [10] presents an overview of the progress in research regarding seismic response of plan and vertically irregular building structures. It is a property of an asymmetric structure that leads to a similar deformation demand in structural members equidistant from the geometric center of the building plan. This concept has led to a new statistical approach to optimally locate extra-structural dissipation devices. In particular, Garcia et al. [11] used this approach to investigate the optimal position of visco-elastic dampers in plan-asymmetric structures. They stated that optimal damper eccentricity values tend to increase linearly as the stiffness or mass eccentricities increase, and that visco-elastic dampers are equally effective in controlling torsional coupling of torsionally flexible structures.

De la Llera and Almazan [12] also studied the optimal location problem by considering frictional dampers, it was recognized that the optimal criteria are difficult to cast into simple expressions. Moreover, torsional amplification of the edge displacements of arbitrary asymmetric structures relative to the displacement of the symmetric counterparts can be approximately bounded by a factor of 2. As we know, the infill walls will increase the lateral stiffness of the frames and lead to reduce the period of the structure, therefore for a reliable design it is important to consider the effect of infill walls. We can say that in the irregularities and changes in structural properties raised from infill walls will not be considered, the structural design may be inefficient and the seismic response of the structures may not be acceptable. Tabeshpour and Ebrahimian have presented a global view on the design process of friction or yielding damping devices [13]. Considering the infill walls leads to determine the period of the structure in high accuracy and therefore, the seismic responses will be reliable and the design of a friction damper will be performing in a correct way. They showed that infill walls had brittle behavior and after a small drift, the stiffness of these elements decreased suddenly and it could change the response of the frame. Most of previous seismic codes have not been considered ductility parameter in seismic design of (steel) structures, so seismic vulnerability and retrofit of existing steel structure is becoming an important engineering problem. The usual way to seismic retrofit such as adding the shear wall or bracing the structure cause huge

forces in elements and foundation. Rehabilitation on the foundation has many constructional problems and leads to long down time. Considering these problems using the new methods for seismic retrofit of the structures are not only economic but also more practical. One of these methods is using friction damper. This has many benefits such as economical, do not disturb the main structure, the lowest effect on the architectural plan, and is ability to control dynamics response and rehabilitation of structure, can be easily manufactured and quickly installed. It makes use of material that provides very stable performance over many cycles, resists adhesive wear well and does not damage the steel plate surfaces, thus allowing multiple uses. In the recent decades increasing demands for the safety, reliability, durability, and serviceability of the structures lead to the theory of vibration control for restricting seismic excitation in civil engineering. In a number of cases, different control devices have been implementing on modern infrastructures such as high-rise buildings, long span suspension bridges, and offshore platforms. In vibration control, we have two purposes: First, to improve habitability during strong winds or in moderate earthquakes and reduce vibration response of structures. Many control devices have been developed to achieve the first purpose, and they have been applied to high-rise buildings and towers such as friction damper device (FDD) [14]. FDD is the simplest kind of dampers and easy to construction and installation. The second purpose of vibration control is to prevent of imparting damage to the main elements of a structure during severe earthquakes. In seismic design of FDD, the structural stiffness and fundamental period directly affect the damper properties. Several parametric studies of one-story asymmetric systems considering energy dissipation devices have been investigated recently [15, 17]. It was shown that the use of viscous supplemental damping may reduce the deformation demand about three times if a correct selection of the system parameters is made. More recently, the inelastic behavior of the primary structure using a bilinear model was also considered [16]. It was shown that the supplemental dampers reduce the deformation demand and the ductility demand of the primary structural members; the results were consistent with those obtained for the elastic system. Other investigations [18] have shown that the reduction in response of systems with supplemental viscous damper is highly dependent on the plan-wise distribution of the devices. Results for other more flexible structures as well as with other damper capacities follow similar trends [19].

Such study also indicates that Viscous Elastic dampers (VE dampers) are more effective than viscous dampers in controlling the response of asymmetric systems. Furthermore, two other studies [19, 20] use non-linear viscous and VE dampers in the torsional control of single-story axisymmetric structures. Analytical models were created using the open source finite element platform, OpenSees [21]. A schematic view of an irregular arrangement of infill is shown in figure 1. In this Study a three-dimensional model of the building is developed for the structure and friction damper is used to retrofit 3-story non-ductile steel structure with infill effects and irregularities. Numerical discuss on the results of pushover and time history analyses are presented.

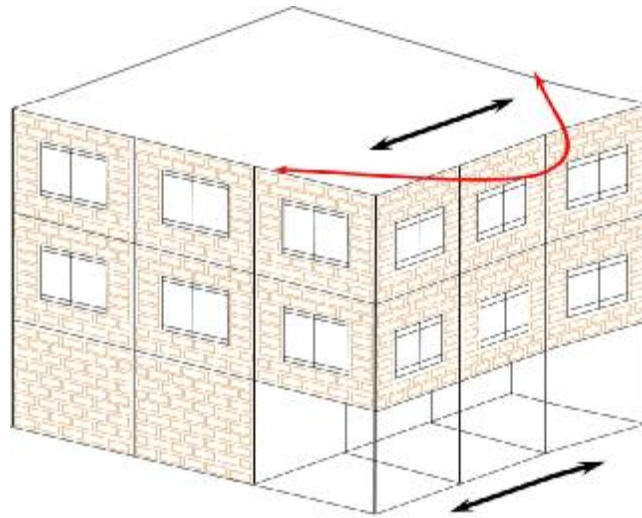
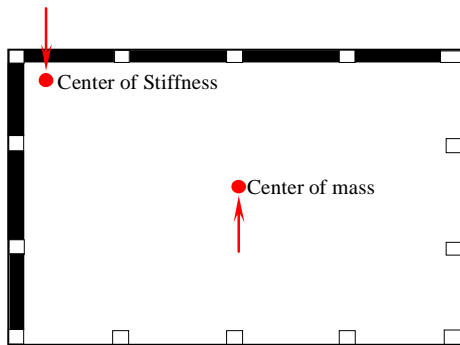


Figure 1. Torsional behavior of 3-story infilled frame

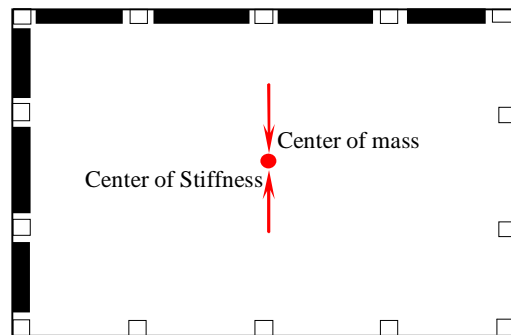
An example of torsional fracture because of ignoring the effect of infills in design and construction is shown in Figure 2. Note the difference between plan b and c.



(a)



(b) Interaction between infill wall and frame



(c) Interaction between infill wall and frame

Figure 2. Torsional fracture because of ignoring the effect of infills (Kobe,1995)

2. THE IRANIAN SEISMIC CODE, STANDARD NO. 2800 (SIMILAR TO UBC-97)

Standard 2800, section: 1-8. building classification according to configuration [22]

Buildings are 'regular' or 'irregular' base on their configuration as fallow:

Standard 2800, section: 1-8-1. regular buildings

Buildings are regular if they conform to all of the following criteria:

Standard 2800, section: 1-8-1-1. plan regularity:

- a) The plan of the building shall be symmetric or almost symmetric about its principal axes, where the lateral load resisting elements are generally aligned with. In case there is any setback or projection, their dimension in each direction shall not exceed 25% of the respective building dimension in that direction.
- b) In each Story, the distance between centers of mass and stiffness in each orthogonal direction shall not exceed 20% of the building's dimension in that direction.
- c) Abrupt variation in diaphragm stiffness relative to the adjacent stores shall not exceed 50% moreover; the total area of opening in each diaphragm shall not be greater than 50% of the total area of the diaphragm.
- d) There shall be no discontinuity in the lateral load path toward the base, such as out-of-plan offset of the lateral load resisting elements.
- e) In each story the maximum drift, including accidental torsion, at one end of the structure shall not exceed 20% of the average of the story drift of the two ends of the structure.

Standard 2800, section: 2-3-10 horizontal distributions of the seismic forces:

Standard 2800, section: 2-3-10-1 the design story shears, determined from the vertical distribution of seismic forces are above shall be distributed among the various vertical lateral-force-resisting systems in proportion to their rigidities. The shear due to horizontal torsion resulting from eccentricities of the applied design lateral forces at levels above any story shall also be included. Where diaphragms are not rigid, the effect of their deformations shall also be considered in horizontal distribution of shears.

Standard 2800, section: 2-3-10-2 the torsional design moment at a given story, i , shall be determined from the following formula:

$$M_i = \sum_{j=1}^n (e_{ij} + e_{aj}) F_j \quad (1)$$

where:

e_{ij} : eccentricity between the lateral force at level j and the center of stiffness at level i (the horizontal distance between the center of mass at level j and he center of rigidity at level i)

e_{aj} : accidental eccentricity at level j determined in accordance with Clause 2-3-10-3

F_j : the lateral force at level j

Standard 2800, section: 3-10-3-2. The accidental eccentricity at any floor level, e_{aj} account for the possible variations in mass and stiffness distributions and also the forces due to the torsional component of the earthquake. This eccentricity shall be considered in both

directions with a minimum value equal to 5 percent of the buildings dimensions in that Story and along with the direction perpendicular to the direction of the force under consideration at the level. Based on the Iranian seismic code of practice (Standard No. 2800), if torsional irregularity exists, this effect shall be accounted for by increasing the accidental eccentricity at each level by amplification factor, A_j determined from the following formula:

$$A_j = \left(\frac{\Delta_{\max}}{1/2\Delta_{\text{ave}}} \right)^2 \quad 1 \leq A_j \leq 3 \quad (2)$$

where:

Δ_{\max} = the maximum displacement at level j

Δ_{ave} = the average of the displacement of the extreme points of the structure at level j

3. MODELING OF MASONRY INFILL WALLS

Infilled frames are complex structural type including numerous parameters. From experimental observations, it is evident, this type of structure exhibits a complex nonlinear inelastic behavior. Material nonlinearity originates from the material properties degradation of both the infill panels and the surrounding frame, the loss of bond-friction mechanism at the interfaces, and variation of contact length, etc. Geometric nonlinearity can also affect the behavior of the infilled frame significantly, especially when the structure is resisting large horizontal displacements. The nonlinear effects mentioned above introduce analytical complexities, which require sophisticated computational techniques in order to be properly considered in the modeling. Due to the stiffness and strength degradation occurring under cyclic loading, the infilled frame structures cannot be modeled as elasto-plastic systems, while models that are more realistic should be used to obtain valid results, especially in the dynamic analysis of short period structures, such as infilled frames. The aim of this chapter is to introduce the modeling of a masonry infill panel, which will implement in the analysis in the following chapter. The elastic in-plane stiffness of a solid unreinforced masonry infill panel prior to cracking shall be represented with an equivalent diagonal compression strut of width, a , given by the following equation:

$$a = 0.25 (I h_{\text{col}})^{-0.4} r_{\text{inf}} \quad (3)$$

$$I_1 = \left(\frac{10E_{me} t_{\text{inf}} \sin 2q}{E_{fe} I_{\text{col}} h_{\text{inf}}} \right)^{0.25} \quad (4)$$

where:

h_{col} = Column height between centerlines of beams, cm

h_{inf} = Height of infill panel, cm.

E_{fe} = Expected modulus of elasticity of frame material, kg/cm

E_{me} = Expected modulus of elasticity of infill material, kg/cm

I_{col} = Moment of inertia of column, cm

L_{inf} = Length of infill panel, cm.

r_{inf} = Diagonal length of infill panel, cm.

t_{inf} = Thickness of infill panel and equivalent strut, cm

q = Angle whose tangent is the infill height-to length aspect ratio, radians

I_1 = Coefficient used to determine equivalent width of infill strut

The equivalent strut shall have the same thickness and modulus of elasticity as the infill panel it represents, Figure 3 and Table 1 shows the required and calculated parameters.

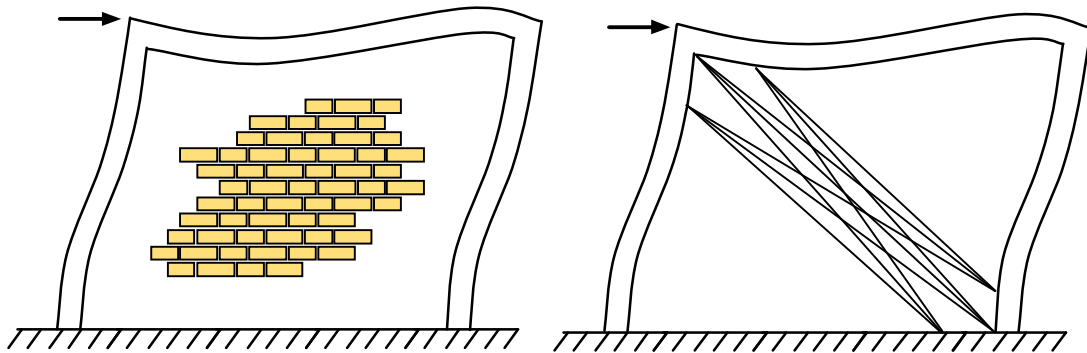


Figure 3. Proposed infill model using equivalent diagonal strut

Table 1: Summary of calculated masonry parameters

E_{me} (N / m^2)	t_{inf} (m)	I_1	a (m)
1.2e9	0.02	0.009043	0.7195

The material properties of masonry strut with consideration of usual mortar and bricks in Iran is taken from [23]. The following stress-strain curve is for one kind of infill walls (0.23m). Stress-strain curve of masonry material in compression is like Table 2. In the shape of parabolic curve till maximum stress f_{mo} then it decreases linearly and after that remains constant. Equivalent masonry strut's material properties is presented in table 2. Proposed effective width of diagonal strut and module of elasticity by researchers is shown in tables 3 and 4 respectively.

Table 2: Equivalent masonry strut's material properties

Parameter	Value
Thickness	0.23 (m)
f_{mo}	4 (Mpa)
e_{mo}	0.0014
f_{mu}	0.8 (Mpa)
e_{mu}	0.0028

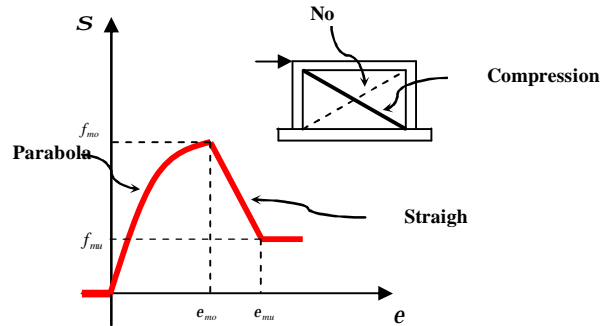


Table 3: Proposed effective width of diagonal strut

Researcher	Effective width (b_w)	
Holmes [28]	$b_w = [0.33]d_w$	
Mainstone [29]	$b_w = 0.16(lh)^{-0.3}d_w$	
Klingner and Bertero [30]	$b_w = 0.175(lh)^{-0.4}d_w$	
Liau and Kwan [31]	$b_w = 0.95h_w \cos q (lh)^{-0.5}$	
Decanini and Fantin [32]	Regarding to Figure 4.	
Paulay and Priestley [33]	$b_w = [0.25]d_w$	
Present Study [34]	Upper bound, Negative Effects	$b_w = [0.2]d_w$
	Lower bound, Positive Effects	$b_w = [0.1]d_w$

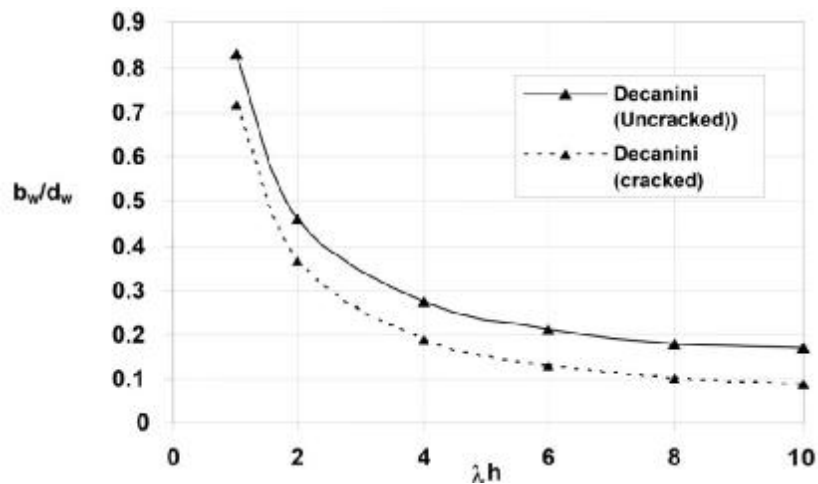


Figure 4. Effective width of infill wall which is suggested by Decanini and Fantin [32]

Table 4: Proposed module of elasticity

Researcher	Module of elasticity	
Sahlin [35]	$E_m = 750 f_m$	
Sinha and Pedreschi [36]	$E_m = 1180 f_m^{0.83}$ (Mpa)	
San Bartolomé [37]	$E_m = 500 f_m$	
Hendry [38]	$E_m = 2116 f_m^{0.5}$ (Mpa)	
Paulay and Priestley [33]; Sahlin [35]	$E_m = 1000 f_m$	
Paulay and Priestley [33]	$E_m = 750 f_m$	
Present study [34]	Negative Effect	$E_m = 800 f_m$
	Positive Effect	$E_m = 600 f_m$

4. DAMPER DESCRIPTION AND PRINCIPLE OF ACTION

Friction dampers have often been employed as a component of these systems because they present high energy-dissipation potential at relatively low cost and are easy to install and maintain. A friction damper is usually classified as one of the displacement-dependent energy dissipation devices, because its damper force is independent from the velocity and frequency-content of excitations. A friction damper is activated and starts to dissipate energy only if the friction force exerted on its friction interface exceeds the maximum friction force (slip force); otherwise, an inactivated damper is no different from a regular bracing. This device used to dissipate the energy not only in the usual structure (building) but also it used in platforms and jackets (offshore structure) as well [35].

The damper main parts are the central (vertical) plate, two side (horizontal) plates and two circular friction pad discs placed in between the steel plates as shown in Figure 5. The central plate has length h and is attached to the girder mid span in a frame structure by a hinge. The hinge connection is meant to increase the amount of relative rotation between the central and side plates, which in turn enhances the energy dissipation in the system. The ends of the two side plates are connected to the members of inverted V-brace at a distance r from the FDD center. The bracing makes use of pretension bars in order to avoid compression stresses and subsequent buckling. The bracing bars are pin-connected at both ends to the damper and to the column bases. The combination of two side plates and one central plate increases the frictional surface area and provides symmetry needed for obtaining plane action of the device. When a lateral force excites a frame structure, the girder tends to displace horizontally. The bracing system and the forces of friction developed at the interface of the steel plates and friction pads will resist the horizontal motion. Figure 5 explain the functioning of the FDD under excitation. As is shown, the device is very simple in its components and can be arranged within different bracing configurations to obtain a complete damping system.

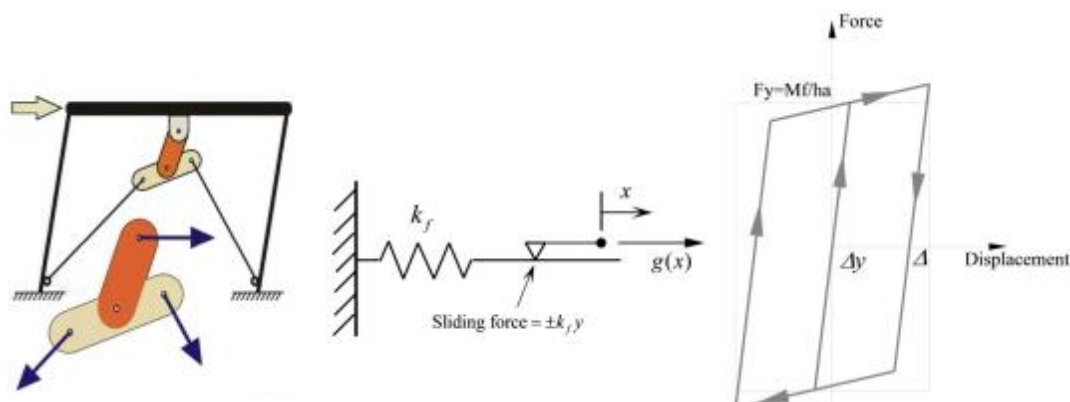
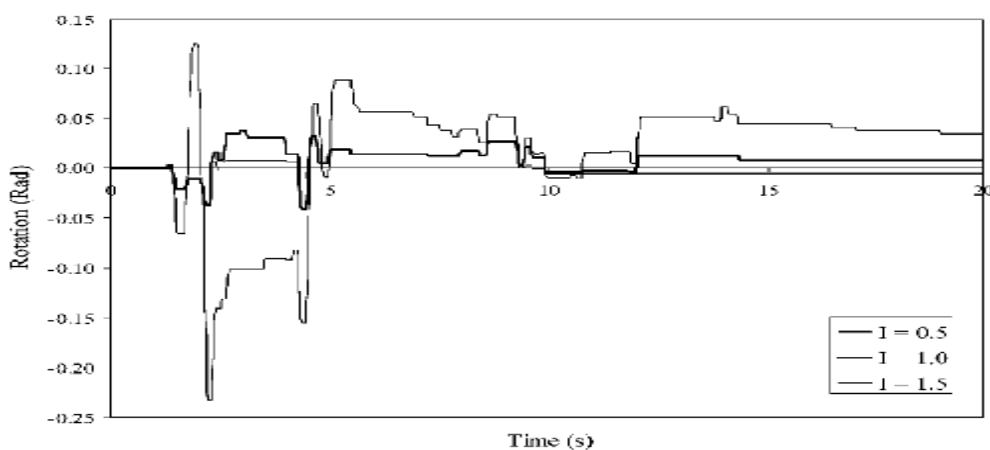


Figure 5. Component of FDD

5. MODELING THE DAMPER IN OPENSEES SOFTWARE

An element with zero length has been used in OpenSees program in order to model the friction hinge. For checking the accuracy of the modeling hypotheses, the structure which studied by Mualla [14] has been modeled in this program. The structure was a steel frame with 7.6m bay and 4.6m height. The frame's beam was assumed to be rigid while the base plates had rigid connections.

Moment of inertia was $34 \times 10^6 \text{mm}^4$ for the beams. For presumed weight of 450kN, vibration period was one second and the damping ratio was 5% of the critical value. This frame was equipped with a damper whose specifications were as follows: $h_a = 0.2\text{m}$, $r = 0.165\text{m}$, $A_b = 603\text{mm}$ and $M_f = 22\text{kN.m}$. This frame was exposed to the north-south component of El-Centro record with maximum acceleration of 3.417m/s^2 . Figure 6 depicts the rotation time History of friction hinge for three different values of scale factor of earthquake. In this Figure, (a) is for Mualla's paper and (b) for our results. Once more, it can be seen that the results are in reasonable agreement.



(a)

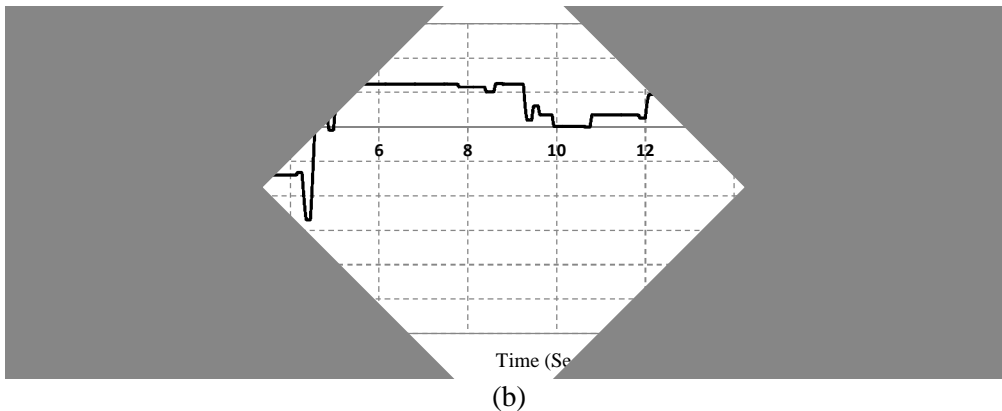


Figure 6. Verification of Rotation in friction hinge; (a) Mualla's paper, (b) our result

6. NUMERICAL STUDY

A 3-story frame with 3 bays has been investigated in this study. Plan and elevation of the building are shown in Figures 7 and 8. Frame A has been filled by masonry walls with thickness of 0.23 m. Lateral force resisting system is intermediate steel moment frame and the type II of soil according to Iranian seismic code of practice (Standard No. 2800). Since investigating the effects of masonry infill walls is the main goal of this research, the considered frames are designed according to last version of Iranian building codes without considering infill walls. Dimensions of the elements have been shown in Table 5. Dead and live loads of stories are considered 600 (kg/m^2) and 200 (kg/m^2) respectively. These parameters are considered 550 (kg/m^2) and 150 (kg/m^2) respectively for roof story. Dead load is considered 133 (kg/m^2) for 23 cm thick walls respectively.

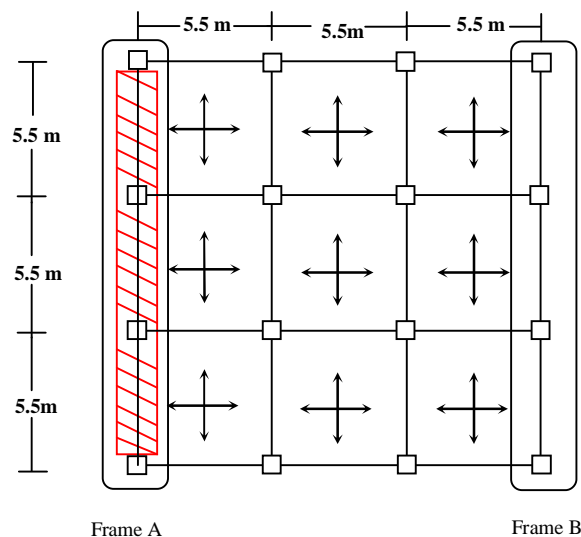


Figure 7. Plan and infill position

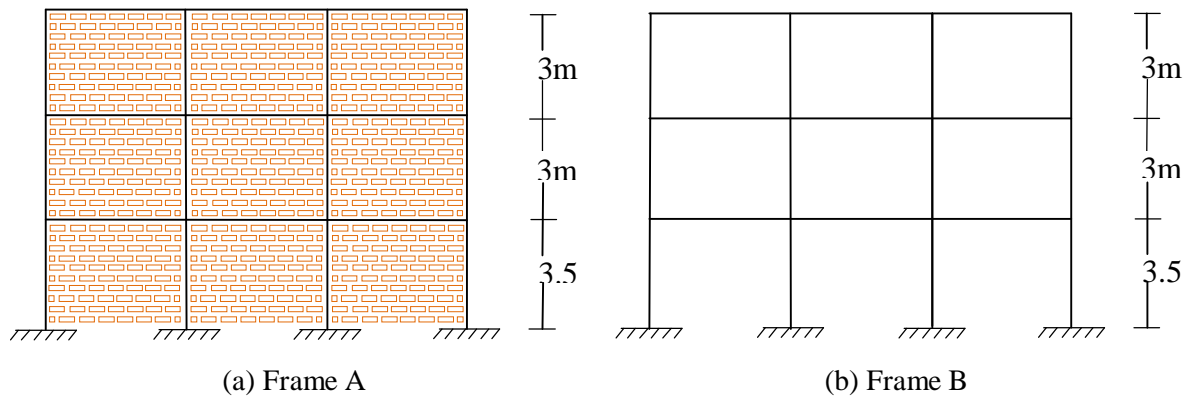


Figure 8. Building elevation

Table 5: Details of element sections

Story	Column		Exterior beam		Interior beam	
	Dimension (m)	Thickness (m)	Web (m)	Flange (m)	Web (m)	Flange (m)
1	0.25×0.25	0.01	0.3×0.01	0.12×0.01	0.3×0.01	0.15×0.01
2	0.2×0.2	0.01	0.3×0.01	0.12×0.01	0.3×0.01	0.15×0.01
3	0.2×0.2	0.01	0.3×0.01	0.12×0.01	0.3×0.01	0.15×0.01

In order to compare the behavior of the original structure and equipped structure, the pushover curves of three cases are shown in Figure 9. Infill walls lead to increase stiffness and strength of buildings compared to a building without considering infill walls. Changing the slope in pushover curves shows this phenomenon. In the push over curves with friction damper, stiffness and strength of buildings in the elastic part of analysis are increased. Since infill walls are brittle material and have a high stiffness, these walls absorb a large amount of lateral load till they fail. After failure of infill walls, we have a drop of stiffness (slope) and strength in curves. As it can be seen in the figure after failure of the infill walls, the slope of the curve will be the same as bare frame. The local interaction between frame A and infill walls is not considered. These results are achieved when the shear strength of columns are sufficient. This can be supposed in steel structures. In order to have a clear sense of the effect of infill and friction damper on the structural behavior, Figure 10 shows the scaled deformations of 3 cases which presented in Figure 9a, b and c.

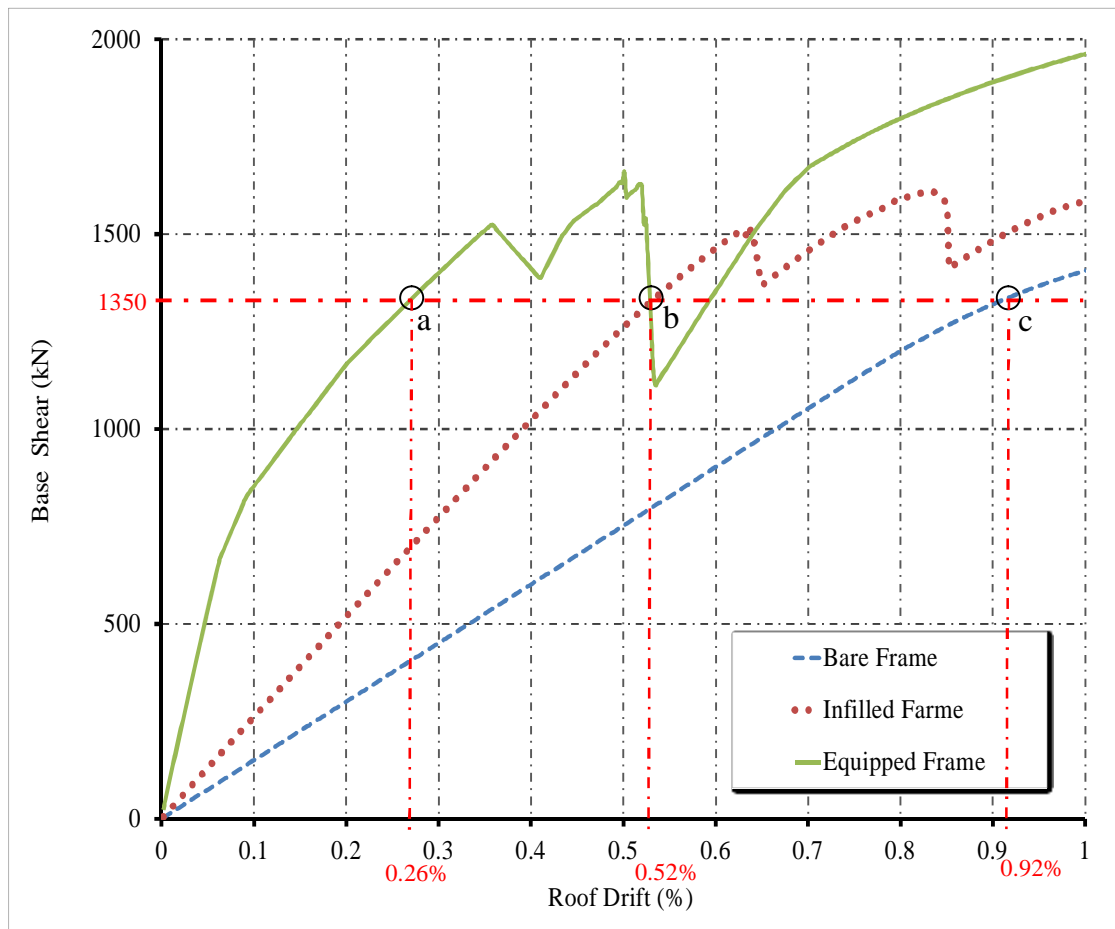
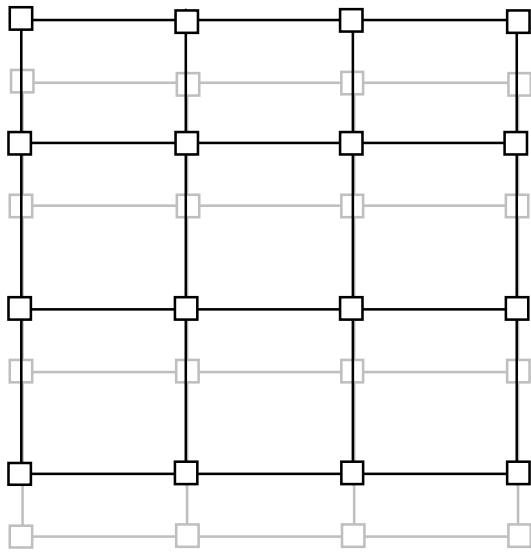
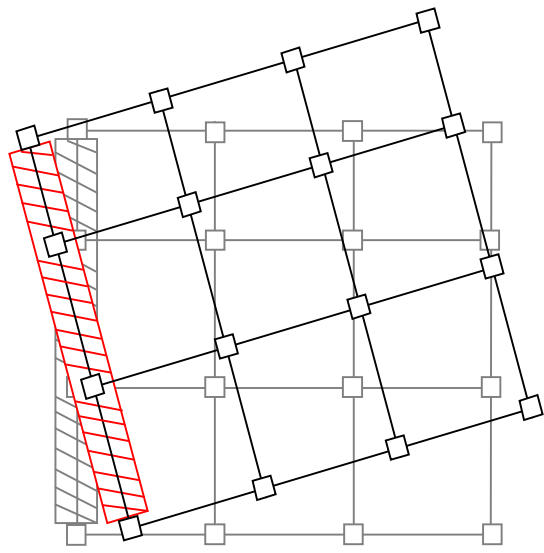


Figure 9. Pushover curves

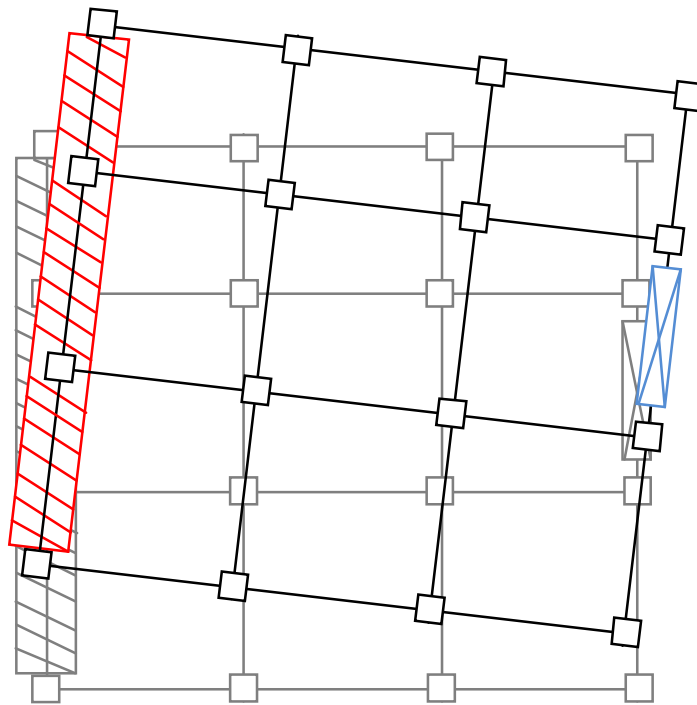
As shown in figure 11 by adding infill walls to the bare frame they lead to increase the stiffness of the system and the torsional problems occurs. This torsion leads to structural failure because of concentration of stress in one side and concentration of deformations in the other side. By using friction damper, eccentricity can be omitted and the distance between the center of mass and center of stiffness will be controlled to satisfy code requirements [26]. In region I with increasing lateral deformation, rotation is increased both in infilled frame and equipped frame. However in this region the rotation in this region at equipped frame considerably less than infilled frame. This reduction of rotation is because of transforming asymmetric system (infilled frame) to a symmetric system (equipped frame). When infill walls start to fail, rotation starts to reduce Region II in the case of equipped frame. But for asymmetric infilled frame, the rotation increases with increasing lateral drift. In region III failing the infill walls cause to reduce the eccentricity and torsional rotation. Therefore the rotation decreases with increasing lateral displacement. In the case of equipped frame, failing the infill walls leads to increase rotation clockwise.



(a) In 1400kN as a base shear we have 2.7% drift



(b) In 1400kN as a base shear we have 2.7% drift



(c) In 1350kN as a base shear we have 2.7% Drift

Figure 10. The response of the structure in 1350kN as a base shear

Figure 12 shows an interactive view between pushover curve and rotation of the structure with no damper. There are two drops in pushover curve, exactly after each drop in strength,

there is a step-like reduction in rotation. After these two drops in pushover curve rotation remains constant but drift increases. It means after failure in masonry infill (and or yielding in structural frame), rotation will remain constant.

After adding the friction damper to the infilled frame, rotation is controlled when the structure is elastic, rotation increases by increasing base shear after stiffness reduction because of sliding of damper direction of rotation changes. Each point that shows reduction in stiffness or drop in pushover curve, relates to a meaningful point in rotation curve. Furthermore, in Figures 9 and 11, a constant relation between base shear and rotation for two cases are found (with and without FDD).

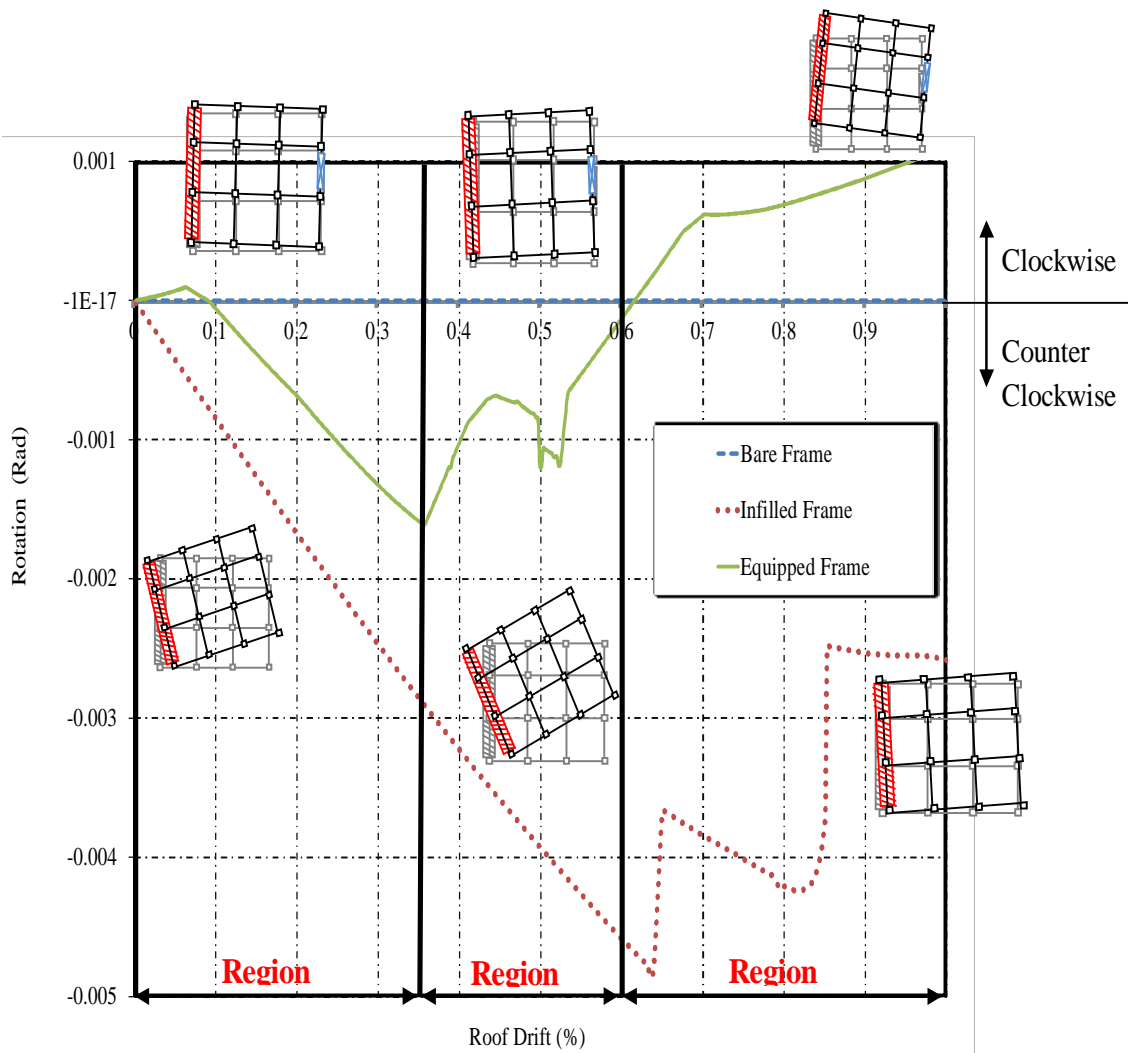


Figure 11. Rotation curves for center of mass

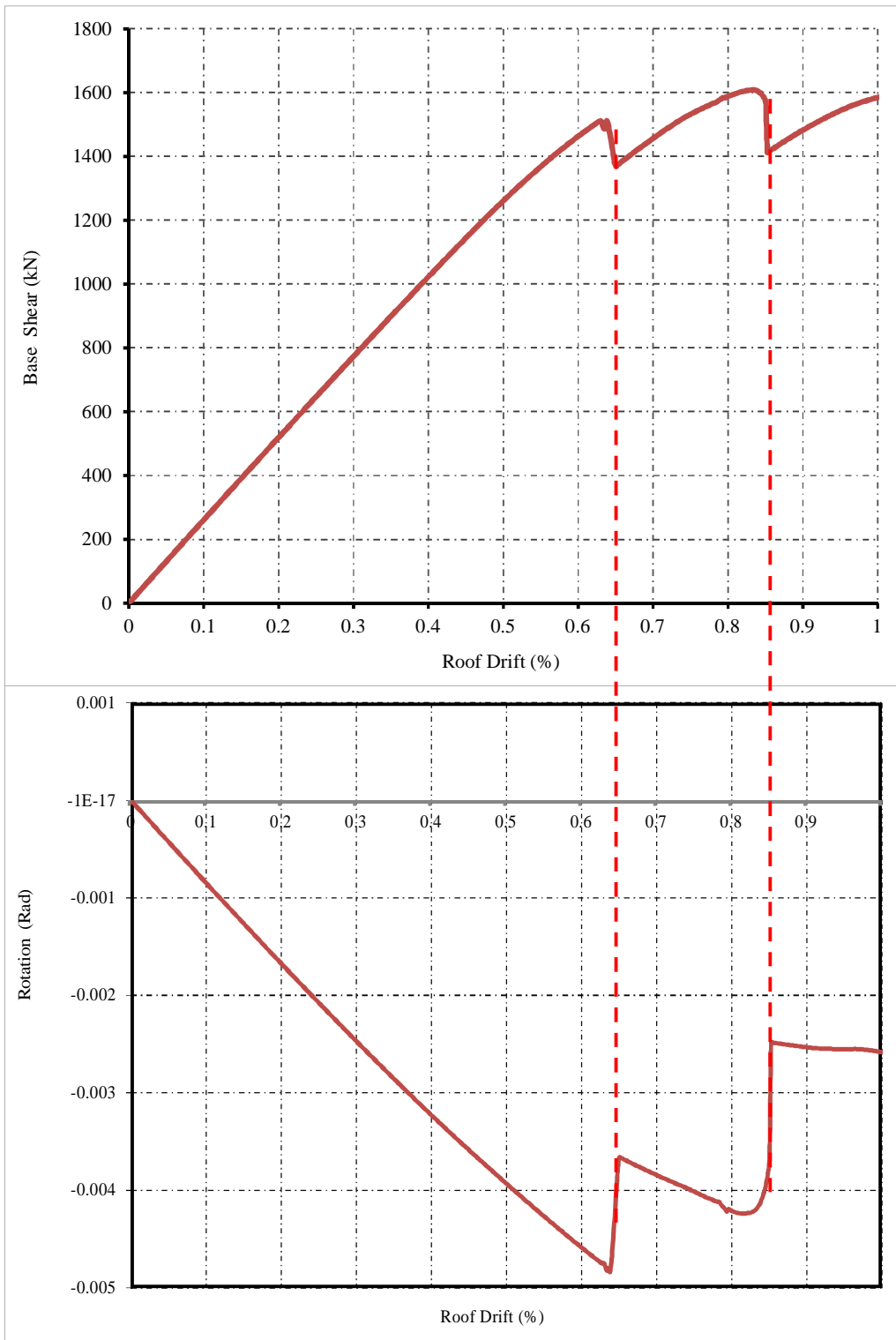


Figure 12. Comparing the pushover curve and rotational behavior of infilled frame

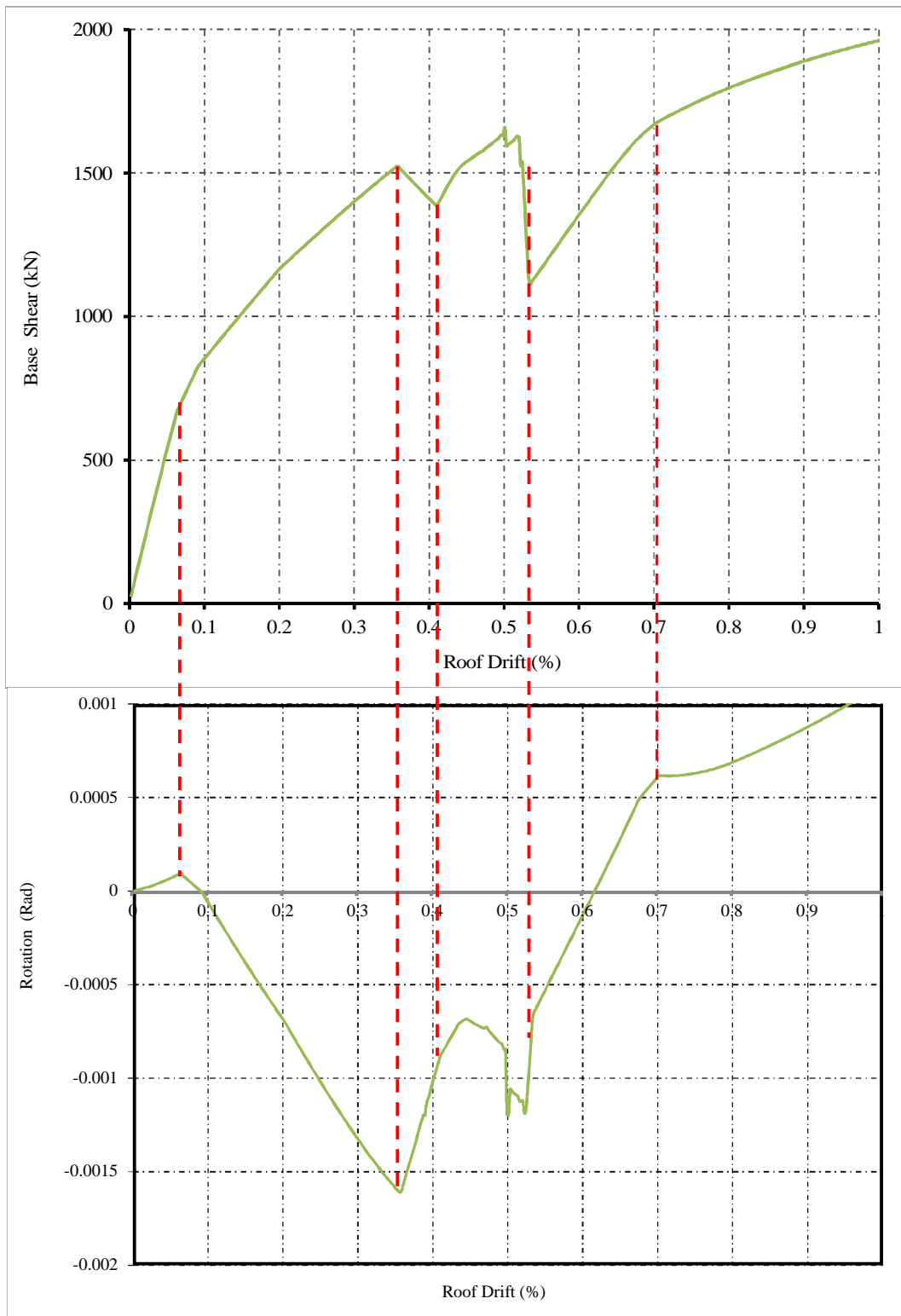


Figure 13. Comparing the pushover curve and rotational behavior of equipped frame

7. CONCLUSION

Because of high stiffness of the infill walls, considering them as structural elements leads the initial stiffness of structures to increase. Such elements show high strength at the first step of seismic loading, but by reaching to the maximum strength, the infill walls fail and high loss of strength occurs in the small drifts. This drop down of strength can be seen in push over curves of structures. Existing of these walls causes high distance between the center of mass and the center of stiffness. Therefore by applying the lateral forces in center of mass, high torsional torque is generated in the diaphragm. For solving this problem, the FDD was used. Sensitivity analysis on effective variables on the FDD behavior showed that increasing sliding force causes decreasing the differences between the center of mass and center of stiffness, so the problem would be solved. It can be seen that, the FDD modifies structural torsion under earthquake excitation. By increasing PGA the positive effect of FDD in structural behavior is reduced, but equipped structure has better performance related to other structures without FDD. Seismic code requirements were considered for torsion. A detailed structural model has been produced using OpenSees. Static nonlinear analyses have been carried out. Because of sensitivity of the friction damper to pulse type excitation, near filed input motion has been considered as excitation force.

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