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Seismic Evaluation of Braced Steel Structures with and without Viscous Dampers for Near Fault Ground Motions

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Abstract. In the last few decades the Viscous Damper (VD) is used for retrofitting or seismic designing in existing or new buildings. In this paper the effect of VD is evaluated in the steel Diagonal-Braced frames with 5, 10 and 15 stories under near fault excitation. For this purpose after designing of prototype structures and defining the parameters of dampers based on performance point, nonlinear dynamic analysis are used to assess the nonlinear behaviour of structures in the form of IDA and fragility curves. The results show the reduction of the responses especially in displacement and drift of prototype structures with greater height. The mechanism of structures with and without dampers is compared together.

1 INTROUCTION

The VD's are energy dissipating devices which use in seismic design and modify the responses of structures against earthquake. The usage of VD results to mitigate the response of structure without increasing stiffness of structure. In the absence of stiffness the relation in the VD's which represents the force in the damper based on relative velocity between each end of the device is:

$$F_D = C|\dot{u}|^{\alpha} sgn(\dot{u}) \tag{1}$$

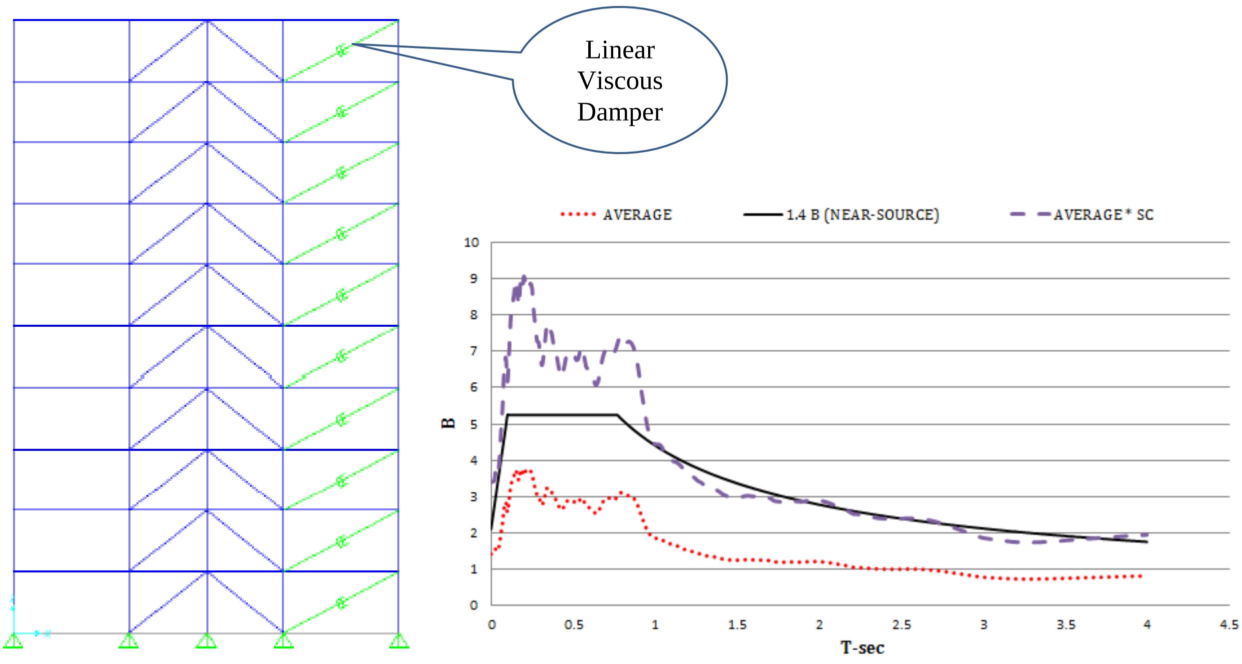
Where C is the damping coefficient,$\dot{u}$ is the relative velocity and α is the velocity exponent. For economic use and maximum efficiency of these devices, careful selection of damper parameters is important [1]. FEMA 356 proposes an approximate and easy-to-use design formula to account the linear viscous damper parameters, in the form of an effective damping ratio arising from the supplemental linear viscous dampers which is added to the inherent damping ratio of the structure. The additional damping ratio, assuming first mode response, is computed by [2]:

$$\beta_{eff} = \beta + \frac{T\sum_j C_j cos^2_{\theta_j} \varphi^2_{rj}}{4\pi \sum_i \left(\frac{w_i}{g}\right) \varphi_i^2} \tag{2}$$

where θ_j is the angle of inclination of the jth device from the horizontal, C_j is the damping coefficient of the jth device, φ_{rj} is the first modal relative displacement between the ends of the jth device in the horizontal direction, T is the first mode period, φ_i is the first modal displacement at ith floor, w_i is the weight of the ith floor and β is inherent damping of structure.

In this paper, the seismic behaviour of 3 diagonal braced steel frames with 5,10and 15 stories retrofitted with linear viscous damper in diagonal brace form evaluated under near fault excitation (Fig.1a). According to previous investigation the records of accelerogram in the near fault region impose large demands on structures compared to ‘ordinary’ ground motions[3]. Recordings suggest that near-fault ground motions with ‘forward’ directivity are characterized by a large pulse, which is mostly orientated perpendicular to the fault.

At the first step the prototype structures were designed based on design spectrum of the Iranian Standard No. 2800 without VD's such that all plastic hinges form at the braces simultaneously. For specifying the VD parameters 3 design earthquakes (Tabas,Manjil,Bam) were selected and scaled them to design spectrum of the Iranian Standard No. 2800 with applying amplification factor for near fault excitation(N_a,N_v based on UBC97)(Fig.1b).By using linear dynamic analysis and a trial-error approach the damping coefficient(C) was identified based on target performance (Drift ratio=0.005)[4].Based on equation 2 the additional damping ratio (β_{eff}) is10,15and20% for 5,10and15 stories respectively, which are under 30% (condition of using Eq.2).



(a) (b)

Figure1:The model of prototype structure,(a) the ten stories model,(b)Scaling of design spectrum

The Maxwell model was used for simulation of dampers (Fig.2). Singh noted that increased brace flexibility tends to reduce the effectiveness of the VD's[5]. Therefore in this paper the stiffness of damper element (k_b) was considered adequately for this purpose(five times the story stiffness)[1].

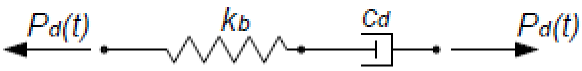


Figure2: Maxwell model - serial arrangement of linear spring and viscous dashpot

2 VALIDATION

The numerical model is validated by experimental model[6]. The validation is achieved in two steps, the first step is validation of Maxwell model of one damper in numerical method which the results of the corresponding experimental model is available, and the second is the comparison of the results of 3 stories moment frame experimental model with viscous damper on the shaking table.

The curve of force-velocity of viscous damper experimental model under harmonic force compares with the numerical model. The results show the behaviour of numerical model depends on the value of stiffness of viscous damper severely (Fig.3). This is approved with the results of previous investigations [5] and it shows that with increasing the stiffness of the bracing connected to dampers the effectiveness of VD increases correspondingly. Because of the experimental model is related to a damper alone, so with increasing the stiffness of VD in numerical model, it is captured well.

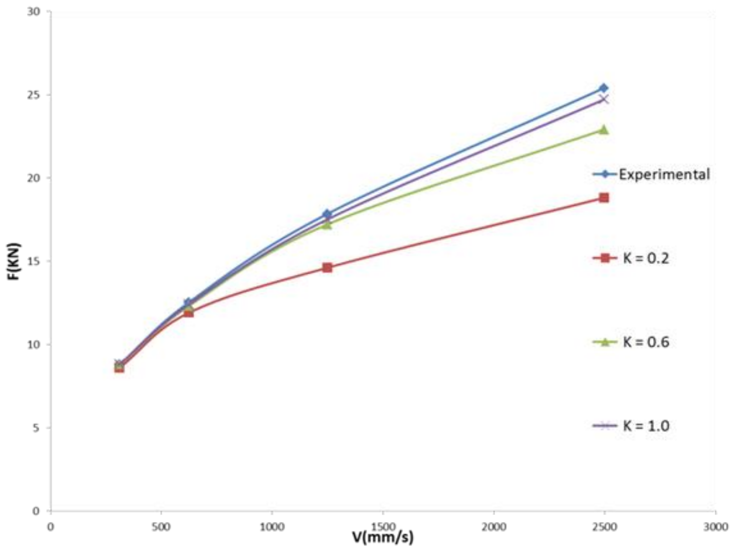


Figure3:Comparison of experimental and numerical viscous damper

In the second step, for verifying the numerical model of VD's in the structure, the results of the 3 stories moment frame with scaling 2/3 on the shaking table under Elcentro record excitation compares with the numerical model (Fig4) [6].The summary of results is presented in Table1 which shows the satisfactory results. The mean of the reduction of response is 70 percent.

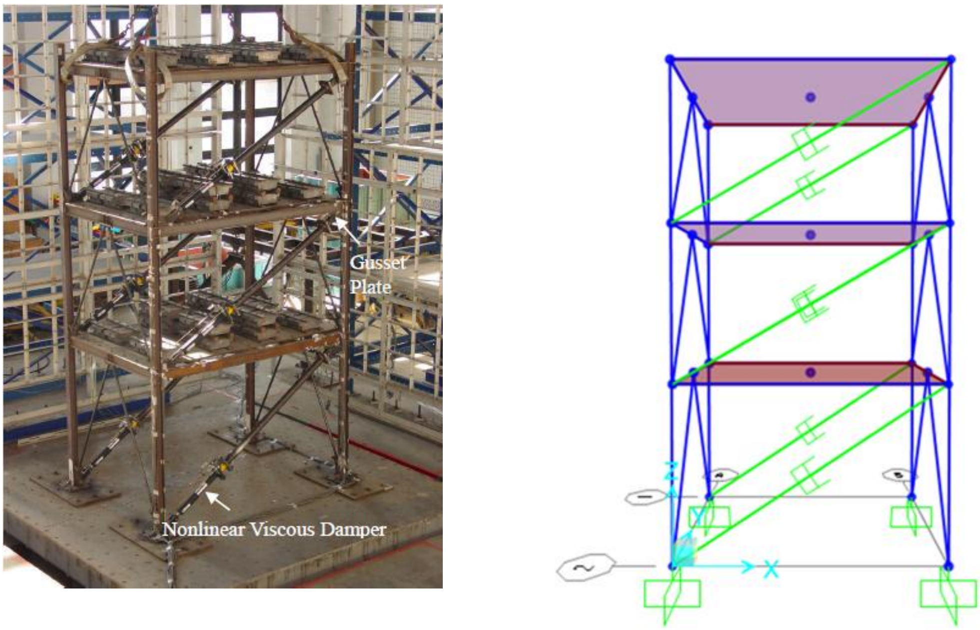


Figure4:The view of model.(a) Experimental Model,(b) Numerical Model

Table1:The comparison of results between Exp&Num. Models.

Comparison of Results :		Displacement (mm)			Story Drift (%)			Accelaration (g)		
Frame ↓	Story →	1	2	3 Roof	1	2	3 Roof	1	2	3 Roof
MF	***Experimental***	***15.4***	***30.8***	***40.7***	***0.77***	***0.89***	***0.58***	***0.38***	***0.44***	***0.72***
	Numerical	***16.3***	***31.2***	***40.5***	***0.81***	***0.85***	***0.53***	***0.38***	***0.48***	***0.69***
NVD	***Experimental***	***4.6***	***8.9***	***11.3***	***0.23***	***0.25***	***0.15***	***0.12***	***0.14***	***0.15***
	Numerical	***4.3***	***8.3***	***10.6***	***0.22***	***0.25***	***0.15***	***0.11***	***0.15***	***0.17***
NVD/MF	***Experimental***	***0.3***	***0.29***	***0.28***	***0.3***	***0.28***	***0.26***	***0.31***	***0.32***	***0.2***
	Numerical	***0.27***	***0.27***	***0.26***	***0.27***	***0.29***	***0.28***	***0.29***	***0.31***	***0.24***

*MF: Moment Frame
*NVD: Nonlinear Viscous Damper

3 SEISMIC RESPONSE

In this paper the nonlinear dynamic analysis was used for seismic response assessment of prototype structures in the form of the incremental dynamic analysis (IDA) based on 7 double near fault accelerogram records. A set of 14 near-fault ground motion records with forward directivity is used to evaluate elastic and inelastic demands of structures. Table 2 lists the basic properties of the recorded motions[3]. The plastic hinged was defined for braces and columns based on FEMA 356. The hysteresis type is Kinematic model which is proper model for steel structures.

3-1 Displacement and Base shear of the models

For evaluation of the effect of the dampers in the displacement response of the models, the average of displacements of the roof is selected based on increment of PGA in the set of records.(see Table 2).According to the results viscous dampers decrease the displacement of the roof and the effectiveness of the dampers is better for 15 stories structure. Averagely the value response reduction for 5,10 and 15 stories structures are 10,30 and 40% respectively. In the other hand the effectiveness of VD's increases with increasing the height of the structure.

Table2: Properties of recorded ground motions used in this study

Earthquake	Station	Mw	R (km)
Erzincan 1992	Erzincan	6.7	2
Imperial Valley 1979	Elcentro - A6	6.5	1.2
Kobe 1995	JMA	6.9	0.6
Morgan Hill 1984	Anderson D	6.2	4.5
Landers 1992	Lucerne	7.3	1.1
Northridge 1994	Sepulveda	6.7	8.9
Northridge 1994	Olive View	6.7	6.4

Table3: The reduction of displacement in the roof of structures(cm)

	5 Stories			10 Stories			15 Stories		
	B.F	VD	R(%)	B.F	VD	R(%)	B.F	VD	R(%)
0.1g	1.77	1.51	14.69	4.61	3.42	25.81	7.98	4.96	37.84
0.2g	3.53	3.01	14.73	9.21	6.84	25.73	15.96	9.93	37.78
0.3g	6.84	5.77	15.64	15.29	10.27	32.83	24.56	14.91	39.29
0.4g	9.24	8.46	8.44	22.75	15.76	30.73	38.49	20.31	47.23
0.5g	11.38	10.65	6.41	36.72	21.08	42.59	44.12	28.62	35.13
0.6g	13.13	12.05	8.23	38.03	25.05	34.13	67.37	36.25	46.19

*B.F: Models without dampers
**VD: Models with linear viscous dampers
***R: Reduction of displacement in roof in percent

Similar to roof displacement of models Table 4 represents the average base shear of the models based on increment of PGA in the set of records. Decreasing trend is shown in the base shear response until the structures are in the elastic region, but with entering the structures in the inelastic region the base shear response increases. It could be resulted from the change of frequency of structure due to inelastic behaviour. In the near fault excitation the inelastic response is sometimes outside the elastic response range [7]. It is shown in the Fig.5 with increasing ductility and period of structure (near to 1.12) the acceleration increases.

Table4: The change of Base shear in the models (ton.f)

	5 Stories			10 Stories			15 Stories		
	B.F	VD	R(%)	B.F	VD	R(%)	B.F	VD	R(%)
0.1g	34.75	30.91	11.05	41.41	31.24	24.56	46.27	33.55	27.49
0.2g	69.51	62.83	9.61	82.12	62.48	23.92	91.31	67.11	26.5
0.3g	96.85	90.93	6.11	103.46	92.45	10.64	126.89	100.66	20.67
0.4g	102.93	109.91	-6.78	113.82	116.23	-2.12	136.96	131.66	3.87
0.5g	107.11	115.72	-8.04	125.35	138.76	-10.7	152.16	161.51	-6.14
0.6g	110.17	122.34	-11.05	177.84	199.81	-12.35	161.13	185.55	-15.16

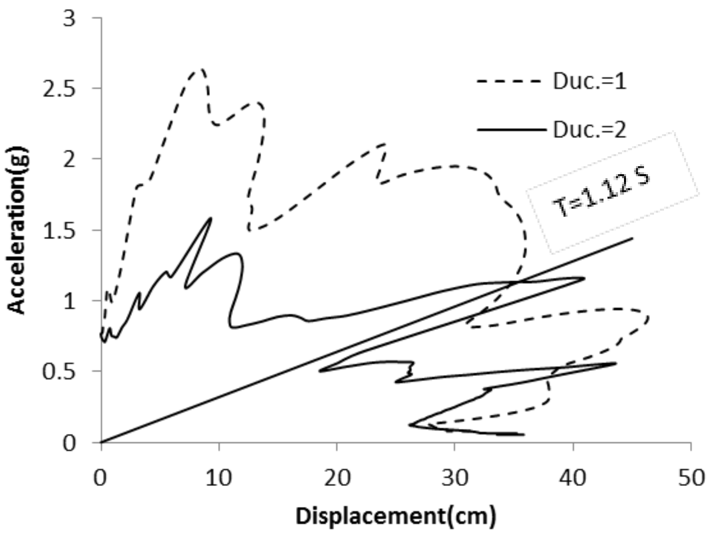


Figure5: The ADRS for record of Kobe (1995)[JMA]

3-2 Fragility curves

The fragility curves represent the probability of exceeding of a certain damage state at a seismic intensity measure, are a suitable tool for the seismic assessment [8].(see Eqs.3&4).

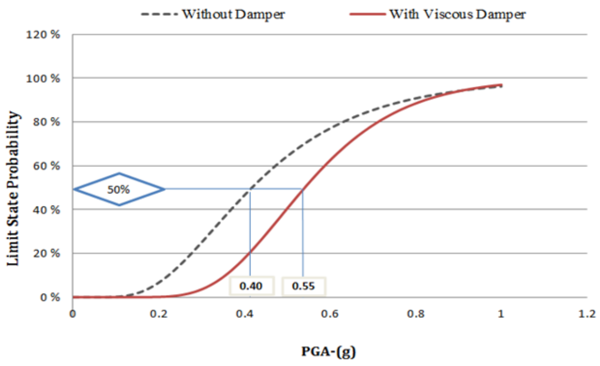
$$P(LS/s) = P[(d_{LS} \leq d_{max}) \,/\, PGA] = 1 - \varphi(r) \quad (2)$$

Where d_{LS} and d_{max} are limit state capacity and maximum demand, respectively. Assuming that the response follows a log-normal distribution, φ is the standard normal cumulative distribution and the standard normal r can be expressed as:

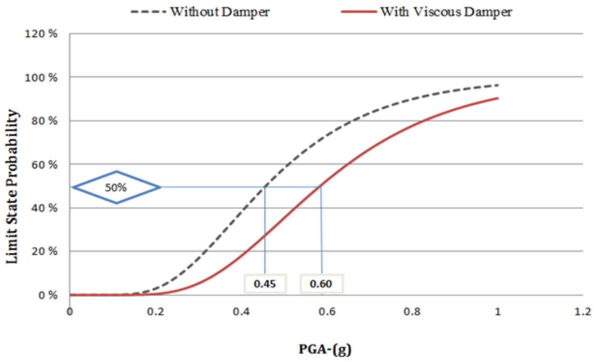
$$r = \frac{lnd_{LS} - lnd_{max}}{\sqrt{\beta_{LS}^2 + \beta_D^2}} \quad (3)$$

β_{LS} and β_D are the lognormal standard deviations of limit state and the displacement demand, respectively. The discrete probabilities were transformed to continuous form by using lognormal probability paper.

In this paper the fragility curves are obtained by using lognormal probability paper[8] and damage state is considered Immediate Occupancy limit state (IO). For this purpose the limit drift ratio is considered 0.005 for each story. Fig 6 shows the fragility curves for prototype structures. It is shown that the PGA corresponding to 50 percent probability of damage increases for structures with VD's.



(a)



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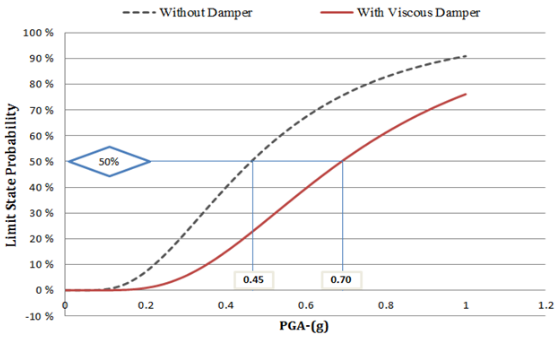
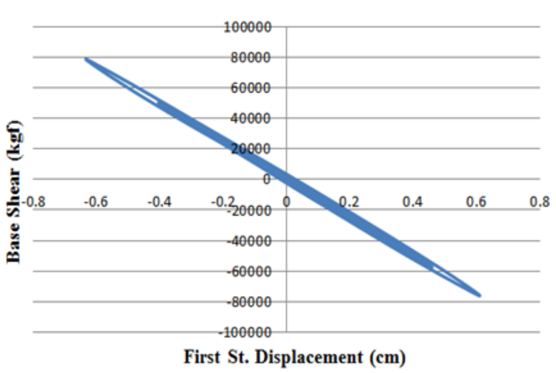


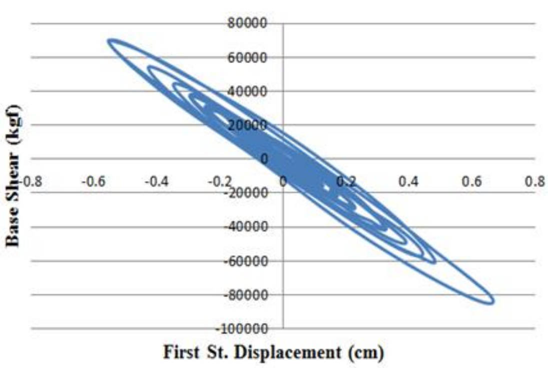
(c)

Figure6: Fragility curves for models. (a) 5 stories. (b) 10 stories. (c) 15 stories.

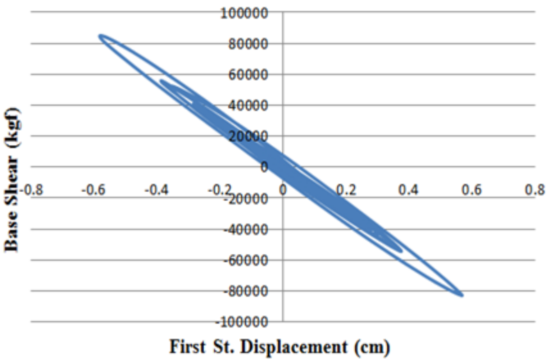
3-3 The Hysteresis curves and mechanism of models

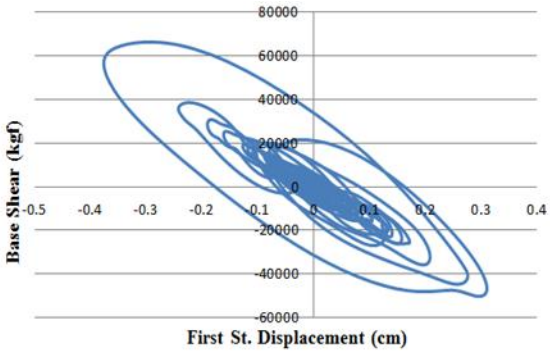
For representing the effect of VD's in absorption and dissipation of input energy, the hysteresis curves are presented based on base shear and displacement of first story in the models for Morgan Hill record scaled to 0.4g. As the Fig.7 shown, in the structure with damper the value of absorption and dissipation of energy increases while the value of force and displacement decreases. This trend is more considerable in taller structures.



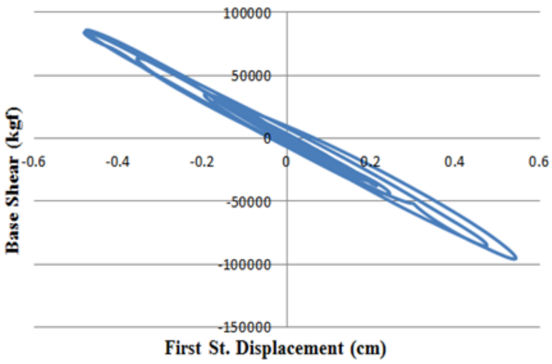


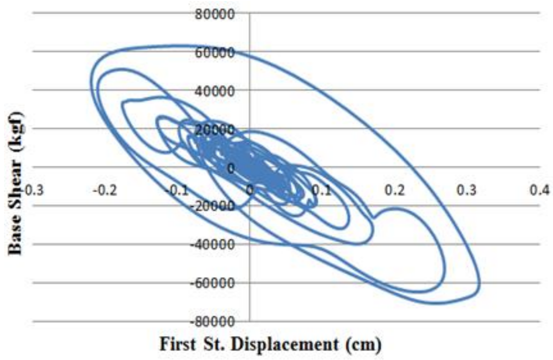
5 stories





10 stories



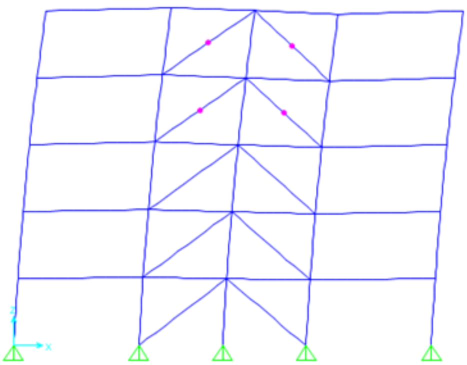


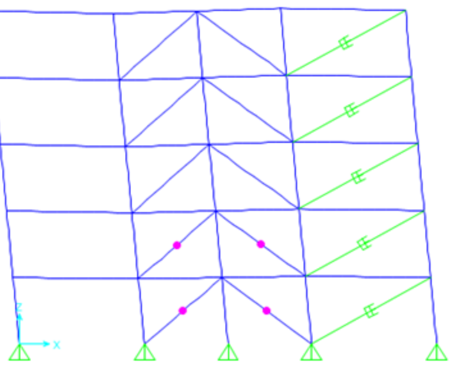
15 stories

(a)without Damper (b)with Damper

Figure 7: Hysteresis curves for prototype structures (a) without Damper (b) with Damper

For evaluation of mechanism of structures, the formation of plastic hinges in the models is compared between structures with and without dampers. In the structures without damper hinges form from top to bottom of structures while this trend is in contrast in models with damper (Fig.8). It could be resulted from the near fault excitation. The previous studies demonstrated that structures with a period longer than the pulse period of near fault record, early yielding occurs in higher stories [3]. As a result with adding VD's to structures the performance of structure would be better in comparison to structures without damper.





(a) without damper (b)with damper

Figure 8: The mechanism for 5 stories structure(a)without damper(b)with damper

4 CONCLUSIONS

For evaluation the effect of Viscous Damper in the seismic behaviour of structures,3 prototype steel braced frames with 5,10 and 15 stories was considered under 7 near fault records. The results show the response of structure is modified and would be under control with adding viscous dampers, but the base shear in the inelastic range of structure is increased. In general the reduction achieved by increasing damping ratio depends on the period of the structure and the frequency content of the excitation. The results of this investigation show that the effectiveness of viscous dampers in steel braced frame under near fault excitation would be better with increasing the height of structure.

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