SEISMIC PERFORMANCE OF STEEL MOMENT RESISTING FRAMES WITH FLUID VISCOUS DAMPER

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Keywords: Steel MRFs, Linear and Nonlinear Fluid Viscous Damper, Far Field Ground Motion, Incremental Dynamic Analysis (IDA), Fragility Curve

ABSTRACT

In this paper, the performance of relatively high-rise steel moment resisting frame (MRFs) under far field earthquakes is investigated with and without supplemental fluid viscous dampers. In this order, three structures of six, eight and twelve stories are designed according to ASCE 7-10 with and without damper, and the characteristics of linear and nonlinear dampers (α = 0.5) are also calculated with equal damping ratios (20% for the models of six and eight stories, 25% for the model of twelve stories). Then, the structures with and without dampers are modeled in Opensees with plastic hinges. Afterwards, the probability of structure collapse is investigated using Incremental Dynamic Analysis (IDA) under far field records, and the probability of collapse of structures are extracted from fragility curves. The results show that the higher the height is, the higher the probability of collapse is due to the effects of P-Delta. Moreover, use of damper improves performance of the structure and reduces its collapse probability in comparison with structure without damper. On the other hand, it was observed that structure with linear damper shows better performance and it has less collapse probability, in comparison against structure with nonlinear damper with equal damping ratio.

INTRODUCTION

In conventional methods, buildings show strength using a combination of stiffness and ductility, and energy dissipation against earthquake. The amount of damping is very low in these buildings and therefore dissipated energy is very poor in elastic range. Thus, building would behave out of elastic range and cause to absorb and dissipate the transferred energy to the structure and prevent collapse under strong earthquakes by inelastic cyclic displacements in members. In this method, generation of plastic hinges will cause damages to structure, which in some cases, damages are such that are unrepairable.

For this purpose, in 1972 Kelly et al. have proposed the idea of using energy dampers in structure to control seismic vibrations (Kelly et al., 1972). Moreover, due to the efforts in this context and the results for application of energy absorbent devices, reliable codes for application of these types of systems are developed so that American Society of Civil Engineers has proposed the analysis of damper systems using three methods: response spectrum, equivalent lateral load, and nonlinear methods, and described its design and seismic loadings in ASCE-7. These systems are categorized into three main groups based on their usage of energy resources: passive, active, and semi-active (American Society of Civil Engineering (ASCE), 2010). Passive control systems are the most common among them, because damping effect is obtained without application of external loads on the damping system, and they perform with the displacements due to the earthquake (Soong and Dargush, 1999).

VISCOUS DAMPER MODELING PROCEDURE AND RESULTS VERIFICATION

To model a damper in SAP 2000, a section of Damper type is used and then it is assigned to a Link element. It is noteworthy that to prevent subsequent convergence issues it is better to assign a low mass to the damper (Computers and Structures INC, 2011).

To model a viscous damper in Opensees, the new ViscousDamper material proposed by Lingos is used and then it is assigned to a twoNodeLink element.

To verify the modeling procedure of viscous dampers, a one-story frame with one span is modelled both in SAP and Opensees and the results are then compared. Details of model are shown in Fig. 1. Box 200*200*20 section is used for columns and IPE 160 section is used for beams. The model is loaded under distributed load of 0.05 kN/mm and the period of that is T = 0.8s. Finally, the model is analyzed using time history method under Kobe earthquake records with scale factor of 0.5. As can be seen in Figs. 2, the results of force-displacement from SAP and Opensees are in good accordance for both linear and nonlinear viscous dampers which would verify the results of the modeling procedure in Opensees.

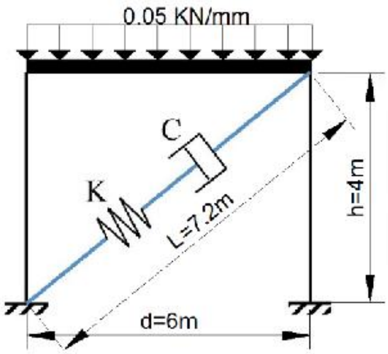

Figure 1. Details of the verification model

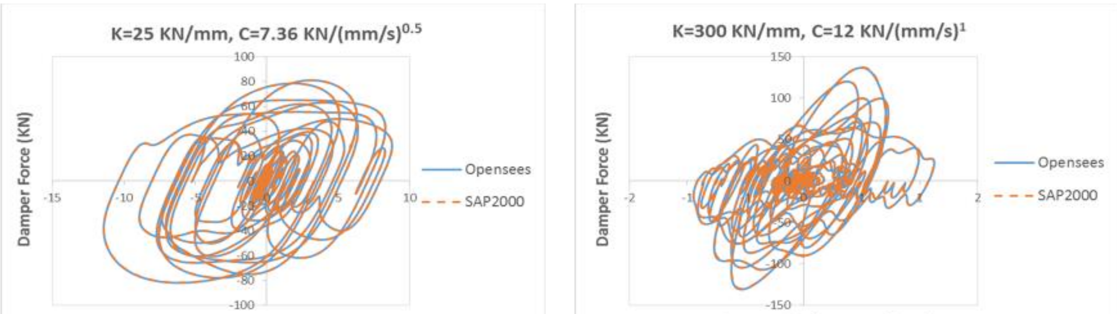

Figure 2. Comparison between the force-displacement results from SAP and Opensees for linear and nonlinear viscous damper.

DESIGNING AND MODELING OF STRUCTURES

In this study, three regular steel structures with 6, 8, and 12 stories are modelled 3D in SAP 2000 as the final design and model, and then one of the frames is selected for modeling in Opensees. As can be seen in Fig. 3, special moment frames are used in circumferential members, all the inner members are under gravitational loading, and all the beam-to-column joints are pinned. Circumferential spans are bending moment frames in two middle spans, and the two others are simple frames. In the structures with dampers,

dampers are located in simple frame spans. The structure is regular and symmetric. All stories are of 4m height and spans are of 6m length.

Records are extracted from USGS website introduced in ASCE 7-10, and are from Los Angeles, USA, Details are given in Table 1.

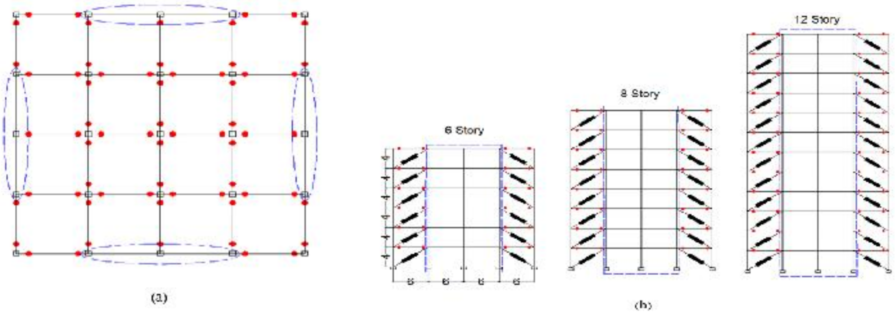

Figure 3. (a) Plan of 3D steel structure (b) Lateral load bearing system of structure (dashed line indicates bending moment frame).

Table 1. Site specifications

Site Class		D - "Stiff Soil"	
Risk Category		I/II/III	
Seismic Design Category		E	
S_S	2.432 g	S_1	0.853 g
S_{MS}	2.432 g	S_{M1}	1.279 g
S_{DS}	1.622 g	S_{D1}	0.853 g
T_0	0.105 s	T_L	8 s
T_S	0.526 s		

DESIGN OF STRUCTURE WITHOUT DAMPER

The structures without damper are designed with %100 of base shear. Relative displacement is controlled in all these structures, whilst the principle of strong column and weak beam has been met according to AISC 341 (American Institute of Steel Construction (AISC), 2010).

DESIGN OF STRUCTURE WITH DAMPER

To design the structures with damper, the expressions given in ASCE 7-10, chapter 18 are used (American Society of Civil Engineering (ASCE), 2010). To calculate damper specifications such as damping ratio and stiffness of spring element, an appropriate pattern of dampers ought to be chose at first, from which magnification factor can be calculated. The pattern used in this study is diagonal. The magnification factor for this pattern is equal to cos θ, but this magnification factor can be used only for horizontal displacement. According to Hwang et al. to get more accurate results in calculation of damping ratio in addition to horizontal displacements, vertical displacements must be considered, especially for structures with high altitude (Hwang et al., 2008). Magnification factors for various patters are given in Table 2.

Table 2. Magnification Factors of Various Damper-Installation Schemes. (Hwang et al., 2008)

Installation configuration	Magnification factor: Horizontal direction f_h	Magnification factor: Vertical direction f_v
Diagonal-brace damper	$\cos\theta$	$\sin\theta$
K-brace damper	1	H/D
Upper toggle-brace damper	$\dfrac{\sin\theta_2\cos(\theta_1-\theta_1)}{\cos(\theta_1+\theta_2)}+\sin\theta_4$	$\dfrac{\cos\theta_2\cos(\theta_1-\theta_1)}{\cos(\theta_1+\theta_2)}$
Lower toggle-brace damper	$\dfrac{\sin\theta_2\sin(\theta_1+\theta_3)}{\cos(\theta_1-\theta_2)}$	$\dfrac{\cos\theta_2\sin(\theta_1+\theta_3)}{\cos(\theta_1+\theta_2)}-\sin\theta_3$

To calculate damping coefficient of damper (C) Eq. 1 given in (Hwang et al., 2008) is used. In this study, expected damping ratio for six- and eight- story structure is assumed to be 20%, and that of structure with 12 stories is assumed to be 25% (inherent damping of 5% is also of high values).

$$\xi = \frac{T^{2-\alpha}\sum_j \eta_j C_j \lambda_j \left|(f_h)_j(\phi_h)_{rj} - (f_v)_j(\phi_v)_{rj}\right|^{1+\alpha}}{(2\pi)^{3-\alpha} A^{1-\alpha} \sum_i m_i (\phi_h)_i^2} \tag{1}$$

Where $(\O_h)_{rj}$ and $(\O_v)_{rj}$ are vertical and horizontal modal relative displacements of two links of damper to the top and bottom floors of story which is normal to the displacement of the roof; $(\O_h)_i$ is the modal displacement of structure which is normal to the roof. Two different magnification factor are used in this expression, one for horizontal movement, f_h, one for vertical movement, f_v .The values of these two factor can be calculated using expressions given in Table 2. m_i is the mass of i^{th} story and η_j is in fact number of dampers in j^{th} story. λj can be obtained from Eq. 2 where Γ is Gamma function, α is damping exponent, C_j is damping coefficient of damper in j^{th} story, and ξ is damping ratio of structure that should be generated by dampers.

$$\lambda = 2^{2+\alpha}\frac{\Gamma^2(1+\frac{\alpha}{2})}{\Gamma(2+\alpha)} \tag{2}$$

The parameter A in Eq. 1 is in fact displacement of the roof of the structure in expected damping according to Hwang et al. (Hwang et al., 2008). The best way to determine this parameter is nonlinear time history analysis in structure with goal damping. In case of three records, A is the maximum response and in case of seven or more records A is the average of their responses. Values of A are given in Table 3. In the design procedure of the existing dampers it is assumed that damper's specifications are the same at different stories.

The expressions given by Lu et al. are used to calculate the stiffness (Lu et al., 2012). The stiffness of the damper can be calculated using Eq. 3 in which C_e is for linear damper. For nonlinear dampers, equivalent damping coefficient should be calculated using Eq. (4) or (5), then the stiffness of the damper can be calculated by substituting equivalent damping coefficient in Eq. (3). The results for damping coefficient and stiffness are given in Table 3.

$$\frac{K^2}{K^2+(C_e\omega)^2} \geq \%95 \tag{3}$$

$$Energy-equivalence: C_e = \frac{\lambda C_\alpha \omega^{\alpha-1} u_0^{\alpha-1}}{\pi} \tag{4}$$

$$Power-equivalence: C_e = \frac{2 C_\alpha \omega^{\alpha-1} u_0^{\alpha-1}}{\alpha+1} \tag{5}$$

Table 3. Damping coefficient and Damping stiffness.

Structure	Pattern	Damping ratio	Roof displacement (A)	Magnification factor	Damping Coefficient (ton.sec/m)		Damper Stiffness (ton/m)	
					α = 0.5	α = 1.0	α = 0.5	α = 1.0
6 story	diagonal	20%	0.38	f_h = 0.83 f_v = 0.55	130.93	350.34	1637.20	3887.90
8 story	diagonal	20%	0.59		168.74	604.66	1846.20	5510.90
12 story	diagonal	25%	0.78		269.19	1025.6	2628.30	8326.50

The periods of the structures with damper are greater than the structures without damper because their weights are 7 kg/m^2 lighter.

Table 4. Period of 2D and 3D structures with and without damper.

Structure	Type	3D	2D
6 story	Without damper	1.31	1.27
	With damper	1.62	1.58
8 story	Without damper	1.69	1.68
	With damper	2.11	2.046
12 story	Without damper	2.07	2.05
	With damper	2.48	2.45

To control the response of the designed structures the method given in (Hwang et al., 2008) is used. In this method, the structure with damper should be analyzed once with damper and once without damper and equivalent damping using time history analysis, then the results should be controlled. In fact, this method shows that if the designed damper is capable of generating expected damping or not. The Northridge Beverly Hills records which are scaled to DBE spectra are used in this section. The history response of the 6-story structure with damper and the structure with equivalent damping are shown in Fig. 4.

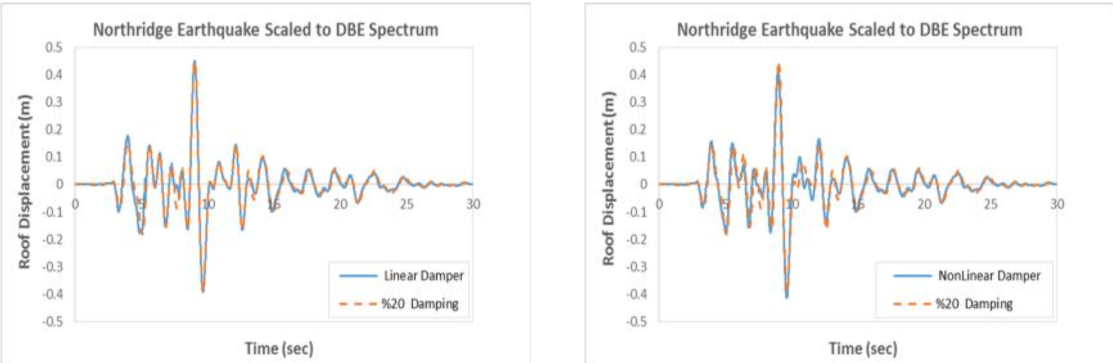

Figure 4. Comparison of the roof displacement for the 6-story structure with linear & nonlinear damper and the structure with equivalent damping.

2D NONLINEAR MODELING IN OPENSEES

To model nonlinear behavior of beams and columns is represented using the concentrated plasticity concept with rotational springs. The rotational behavior of the plastic regions follows a bilinear hysteretic response based on the Modified Ibarra Krawinkler Deterioration Model (Lignos and Krawinkler, 2012).

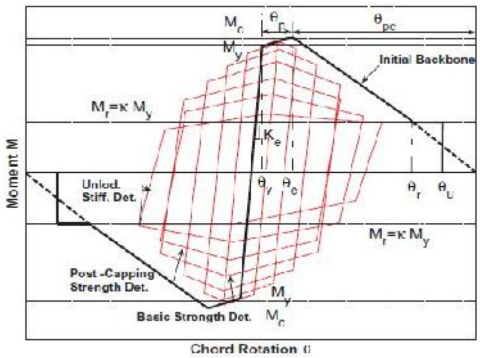

Figure 5. Modified Ibarra-Krawinkler (IK) model. Backbone curve, basic modes of cyclic deterioration and associated definitions (Lignos and Krawinkler, 2012).

INCREMENTAL DYNAMIC ANALYSIS AND ASSESSMENT OF THE STRUCTURES COLLAPSE

The incremental dynamic analysis is a powerful method to predict capacity and demand of structure using a series of nonlinear dynamic analyses and scaling records (Vamvatsikos and Cornell, 2002). The records which are used in incremental dynamic analysis are 22 pairs of far field records given in appendix of FEMA P695 (FEMA P695, 2009).

Collapse indicator is one of the following criteria:

1- When the slope in IDA curve is equal to 20% of initial elastic slope.

2- When the maximum relative inter-story displacement is greater than 0.1.

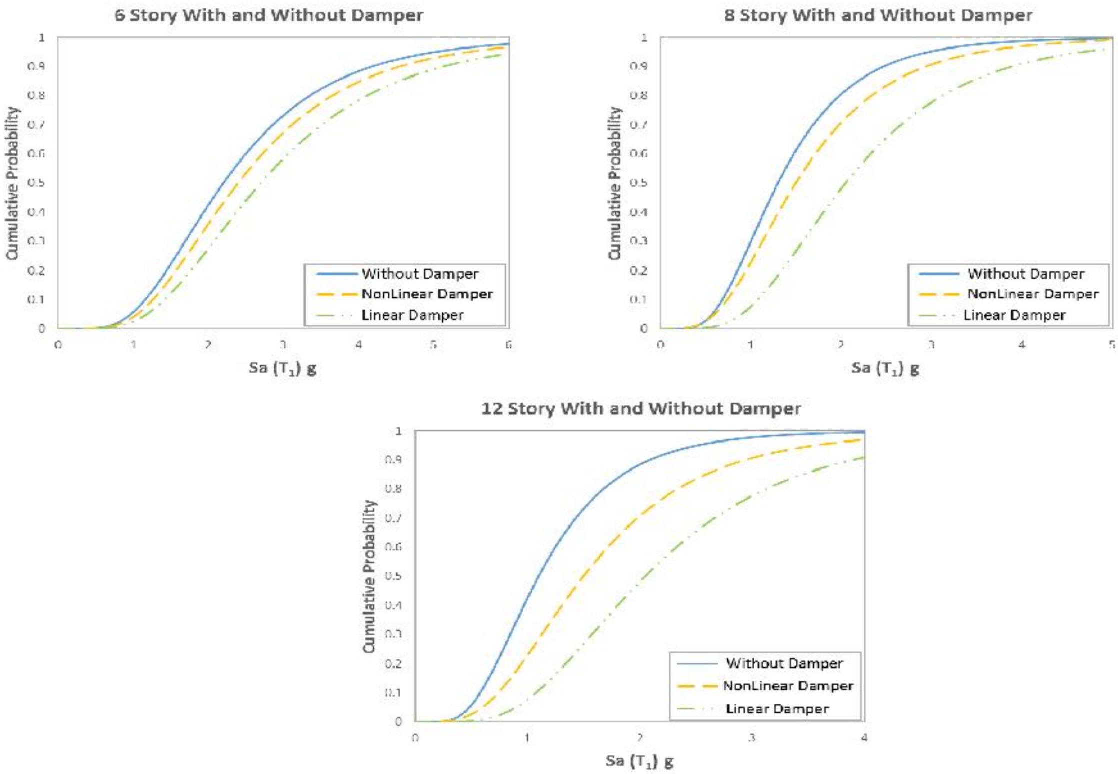

Figure 6. The fragility curves of the structues with and without damper resulting from IDA curves.

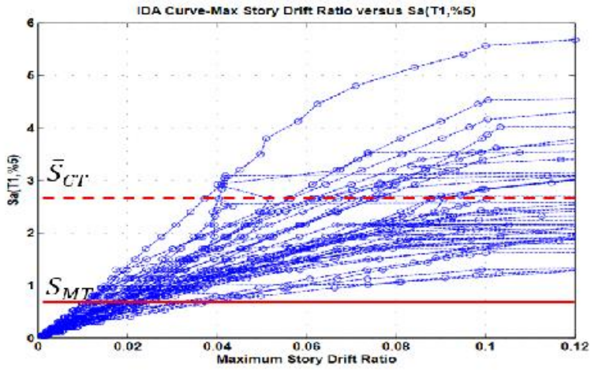

Figure 7. IDA curves

Table 5. Evaluate the performance of structures with and without damper according to FEMA P695.

Structure	Static Ω	μT	SCT	SMT	CMR	SSF	ACMR	Accept. ACMR	Pass/Fail
6 Story without Damper	2.86	9.27	2.2	1.00	2.20	1.54	3.39	1.52	Pass
6 story with Nonlinear Damper	3.06	7.36	2.4	0.80	2.98	1.58	4.71	1.52	Pass
6 story with Linear Damper	3.06	7.36	2.7	0.80	3.36	1.58	5.30	1.52	Pass
8 Story without Damper	2.87	8.40	1.3	0.76	1.71	1.61	2.75	1.52	Pass
8 story with Nonlinear Damper	2.30	4.85	1.6	0.63	2.56	1.43	3.66	1.52	Pass
8 story with Linear Damper	2.30	4.85	2.05	0.63	3.28	1.43	4.69	1.52	Pass
12 Story without Damper	2.67	6.57	1.1	0.62	1.76	1.526	2.69	1.52	Pass
12 story with Nonlinear Damper	2.51	3.58	1.5	0.63	2.40	1.364	3.27	1.52	Pass
12 story with Linear Damper	2.51	3.58	1.8	0.63	2.88	1.364	3.93	1.52	Pass

CONCLUSIONS

The results of this study can be summarized as follows:

- To design Damper and obtain its characteristics, is necessary to be taken into consideration effect of vertical displacement of the ends of the damper.
- Structure with damper designed for %75 base shear and continue controlled for force from chapter 18 of ASCE 7-10. Its about 7 kilograms per square meter lighter than structure without damper are designed.
- For modeling fluid viscous damper in Opensees we used new element who made by Lignos recently. To verify the modeling procedure of viscous dampers, a one-story frame with one span is modelled both in SAP and Opensees and was observed that the results from SAP and Opensees are in good accordance.
- The results of the incremental dynamic analysis of structures showed with increasing of structure height, probability of collapse in %50 of records occurs in smaller Sa. In fact, in tall structures with increasing altitude and subsequently increase the P-Delta effect under far field earthquake, probability of collapse increases.
- Use of damper improves performance of the structure and reduces its collapse probability in comparison with structure without damper. However, opposite result is obtained from short structures under near-field records (Ahmadi Namin, 2013).
- On the other hand, it was observed that structure with linear damper shows better performance and it has less collapse probability, in comparison against structure with nonlinear damper with equal damping ratio.
- To describe the worse performance of nonlinear damper against linear damper with equal damping, it can be stated that the nonlinear damper shows less capacity as the intensity of earthquake increases and the velocity at extremes of the damper increases as well. This will cause less energy dissipation in nonlinear damper in comparison to linear damper, therefore the remaining input energy will be dissipated through structural members (such as columns and beams) which will cause earlier failure and subsequently earlier collapse of structure. It is noteworthy that this behavior occurs when the structure inputs the plastic zone. When the structure is in elastic zone or in other words, the intensity of earthquake is low there is no significant difference between two dampers.

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