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PERFORMANCE EVALUATION OF VISCOELASTIC DAMPERS IN NEAR-FAULT EARTHQUAKES USING NONLINEAR TIME HISTORY ANALYSIS

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Records from near-fault earthquakes at close the distance where the wave propagation source is a special property that show their behavior makes them different from other records. Mostly near-fault earthquakes have strong pulse velocity (pulsatile wave) with a great period accompanied with permanent deformation of earth. Velocity pulse occurs in horizontal component perpendicular to the moving surface of fault which is the resultant of directionality effect of fault rupture. The properties of the pulse such as velocity record in near-fault earthquakes cause the response spectrum to show non-ordinary behavior in a pulse period. Also, due to imposing of much energy to structure during short period by these pulses, most of earthquake energy is absorbed at first making hinges instead of extension of non-linear behavior and plastic hinges in height of structure and the extension of non-linear behavior is not observed. This absorption of energy causes large relative inter-story displacements. Considering today's using energy dissipation systems is current due to reducing the earthquake vibrations of structures, where these energy dissipation systems are passive viscoelastic dampers. In these dampers whose energy dissipation mechanism depends on the velocity of motion or in other words, on loading frequency and to activate these dampers there is no need to determine level of external excitation and they act in every earthquake. For this purpose, a number of structural models have been modeled in 2D form in "Open Sees" software for different damping ratios due to the added viscoelastic damper, non-linear dynamic analysis has

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been done under acceleration of horizontal earthquake and the amount of reduction of displacement response and base shear have been studied.

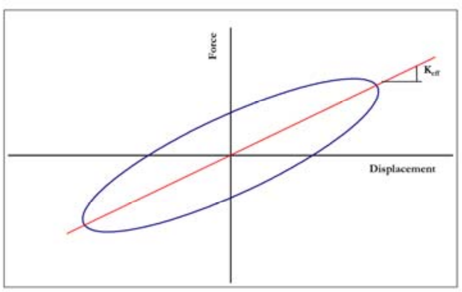
Keywords: passive control, viscoelastic damper, near-fault earthquake, nonlinear dynamic analysis, OpenSees

1. Introduction

Considering the research conducted about land records which are obtained from strong ground motion in the proximity of fault and also considering the effect of such records on different structures, greater attention has been given to studies on these records and their effects on structures during the past two decades. Due to shear waves properties and the cumulative effects of these waves in front of the path of failure, significant differences can be found between the characteristics of near-fault earthquakes and those of far-fault earthquakes, among which the following can be cited in records of near-fault earthquakes the presence of pulse-like motion with a long period at the beginning of records, larger component perpendicular to the fault than component parallel to it, accumulation and transfer of energy in short durations, impact-like force applied to structures existing in the leading failure path, high maximum velocity to maximum acceleration ratio, and the presence of maximum acceleration and speed and higher displacement (John, 1995). Northridge (1994) and Kobe (1995): earthquakes caused entire damage or serious injuries to many modern structures, most of which were attributed to the effects of near-fault earthquakes, after much research was done. John. F. Hall offered a lengthy report entitled "Parametric study of steel moment frames’ response to near-fault earthquakes" in December 1995, with an investment of the U.S. Federal Emergency Management Agency (FEMA) (John, 1995). Although the results of this study showed that inelastic stress typically occurs in beams, the yield may significantly occur in columns too. In addition, the results of relative displacement values under the intended records for both 6- and 20-story frames suggest that there are higher displacement-demands of the intended records as near-fault records compared to the limits of current seismic regulations. At the end, they concluded that the effects of the earthquakes of this sort (near-fault records) are more than those presented in the regulations. Therefore, to consider the effects of near-fault records in the seismic regulations, force level in design of regulations for near-fault earthquakes must be increased (John, 1995). Effects of earthquakes in the proximity of the fault (especially in the leading direction of failure path) cause severe damages to structures (especially high-period structures) due to pulse-like motions with a long period. It has also been empirically observed in Duzce, Chi-Chi, Kobe, Kocaeli and Northridge earthquakes, which led to regard it as one of the determining factors of urban development so that Rauch & Smolka in their article (1996) introduced proximity to fault and placement of buildings in failure path of the fault as two important factors involved in selecting and developing future cities as well as in designing large cities, after they studied the earthquakes of two large and modern cities of the world (i.e. Northridge, California (1994) and Kobe, Japan) and the damages caused (Rauch & Smolka, 1996). To perform nonlinear dynamic analysis and energy absorbability, Andre & Filiatrault in 1998 analyzed the moment steel frames to examine the actual behavior of the moment steel frames using a conventional six-story structure. The analysis was conducted on a regular six-story structure with a moment steel frame which is designed based on the current codes of regulations. The above structure with two different damping systems affected by the ground motion, which are representative of near-fault conditions, have been placed under the records obtained in Los Angeles area with a probability of 10 percent in 50 years; and the behavior of both systems for energy absorption and energy amortization has been examined by the system (Andre & Filiatrault, 1998). Studies on the response of structure to the near-fault earthquakes show that the time-history analysis is better than the response spectrum analysis, because the specifications of the frequency domain of earthquakes (during the response spectrum) expresses the process in which there is a relatively uniform distribution of energy during the motion. Thus, when the energy is concentrated in a few pulses of motion, the phenomenon of resonance is thought to be provided by the response spectrum should not have enough time for formation (Somerville & Paul, 2001). Also the damages incurred in the structures as the result of Kobe (1995, (Mw= 6.7) Northridge (1994) and Izmit (1999) earthquakes have shown that there are significant differences between the response of structures to the near-and far-fault earthquakes (Akkar & Gulkan, 2003).

2. Viscoelastic Damper

This damper is designed on the basis of energy dissipation due to shear deformation in solids. The damper, as Figure 1 shows, consists of the plates between which the polymer materials have been filled and kinetic energy is wasted with the shear deformation of the polymer layers, so that viscoelastic material has a polymer molecule structure; and in other words, their molecules are linked together as chain. As the result of the molecular network above, viscoelastic material shows a resistance against the deformation – the resistance that is one characteristic of the material. In fact, stiffness of structural systems will be increased by using this material in the structure. On the other hand, while deformation is applied to this material, some of the molecular bonds are broken down and the heat is produced, depending on temperature and the loading frequency. So, some energy is spent to break the bonds, and is wasted. Damping of these materials is due to the breakdown of intermolecular bond. After loading over time, the material recovers their initial strength, which the amount of this recovery depends on the temperature of the material, stimulant frequency and strain amplitude. In short, one will face an increase in stiffness and damping in the structural system by using the material above in the structure. Installation of the dampers should not be limited only to braces, but they can be used with special arrangements throughout the structure in which shear deformations occur. The experimental results suggest that the effects of higher modes are reduced and can be ignored using the viscoelastic damper in the building. On this basis, there is a good coordination between with the results of analyses (which were performed according to the first mode) and the experimental results – the subject which, as one of the strengths of this damper, makes the calculations simple and easy. Meanwhile, the Kelvin model is used as conventional one to model the dynamic behavior of the viscoelastic damper, which contains a spring and a linear damper arranged in parallel. As it is shown in Figure 2, hysteresis graph of this damper is an ellipse, and behaves like the devices whose properties of damping depend on the speed. To activate them, no level of external stimulation is needed, and they act as the result of each earthquake, and dissipate the energy. That is a property that represents the distinction between elastic dampers and friction dampers, which cannot be activated for the forces less than slip force. In addition, unlike the viscoelastic dampers, velocity-dependent damping is a linear function of speed, in which the damping power is 1 for dampers of this kind (Trevor & Kelly, 2001).



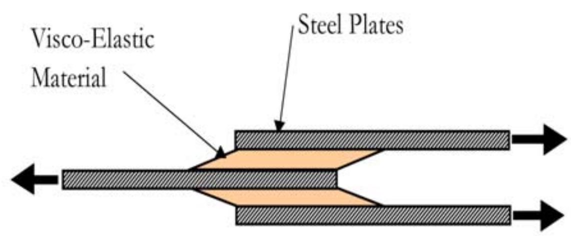


Figure 1. Viscoelastic damper (Trevor & Kelly, 2001)

Figure 2. Force-displacement relationship of the viscoelastic damper (Trevor & Kelly, 2001)

The amount of force in these devices is determined as follows:

$$F_D = K_{eff} .\Delta + C\dot{u} \tag{1}$$

In viscoelastic dampers, shear-storage modulus and shear-loss modulus are function of the main vibration frequency of structure, and are used to determine the effective stiffness and damping (Ramirez et al, 2003).

$$K_e = \frac{G'A_d}{h_d} \text{ and } C_e = \frac{G''A_d}{h_d\,\omega} \tag{2}$$

Where , A_d = cross section of the damper added , h_d = thickness of the added damper , ω = the main vibration frequency , K_e = effective stiffness of damper, and C_e = effective damping of damper.

To determine these two modules, the graphs in Figure 3, can be used according to the studies by Zimmer (2000). Based on different ambient temperature and shear strains, these diagrams show a relationship between the main vibration frequency of the structural system, shear-storage modulus, and shear-loss modulus. Studies also show that the properties of the damper depend on the number of cycles of the incurred load and range of deformations; but its importance is likely to be obscured simply because the large accelerations occur only in a limited number of cycles during an earthquake; and the properties of damper can be assumed constant during the earthquake (Zimmer, 2000).

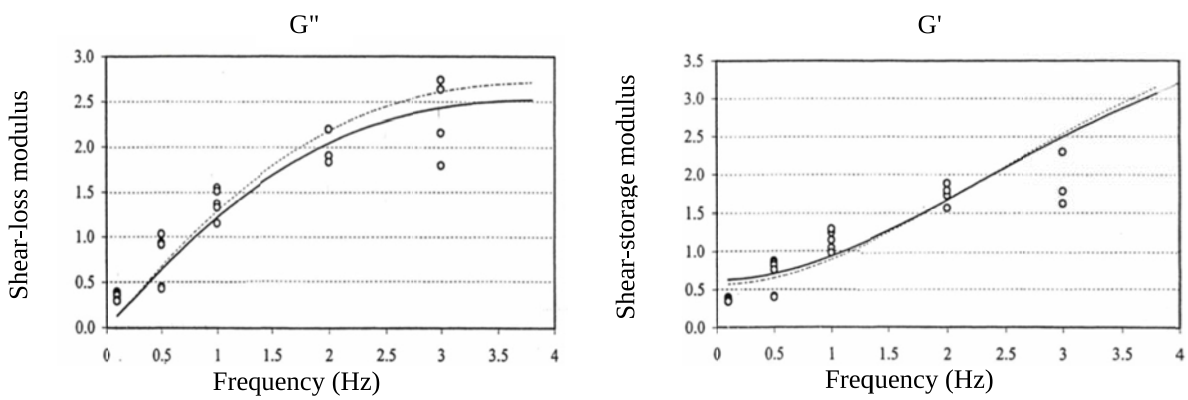


Figure 3. Relationship between frequency- shear-loss modulus and shear-storage modulus, according to Zimmer's studies, (Zimmer, 2000)

3. Characteristics of Analytical Models

In this study, three special two-dimensional models were used for moment frames having the number of stories 9, 14, 17 with a variable number of openings (each with a length and a height of 5 and 3 meters, respectively). Profiles used in this paper include IPE and IPB. So, the value of dead load on roof for all stories is equal to 3000 kg/m, while the amounts of live loads for the stories and the roof are 1000 kg/m and 800 kg/m, respectively. In addition, structural importance coefficient is considered equal to one, with selected land of type C. They are designed on the basis of the topic 10 of the National Building Regulations and Standard 2800 Iran, assuming F_y = 2400 kg/cm^2. ETABS software was used to develop structural models. Also to perform nonlinear dynamic analyses, OpenSees2.5 software was used. Structural members were modeled by the Force Beam Column Element that is a model based on a fiber element with extensive plasticity. In this software, the elastoplastic behavior of steel 01 is used for steel with strain hardening of 3 percent; and Newmark method with $\alpha = 0.5$ and $\beta = 0.25$ was also used to analyze the system. For example, the sections selected for a 9-story structural model with and without the added viscoelastic damper are shown in Figures 4 and 5.

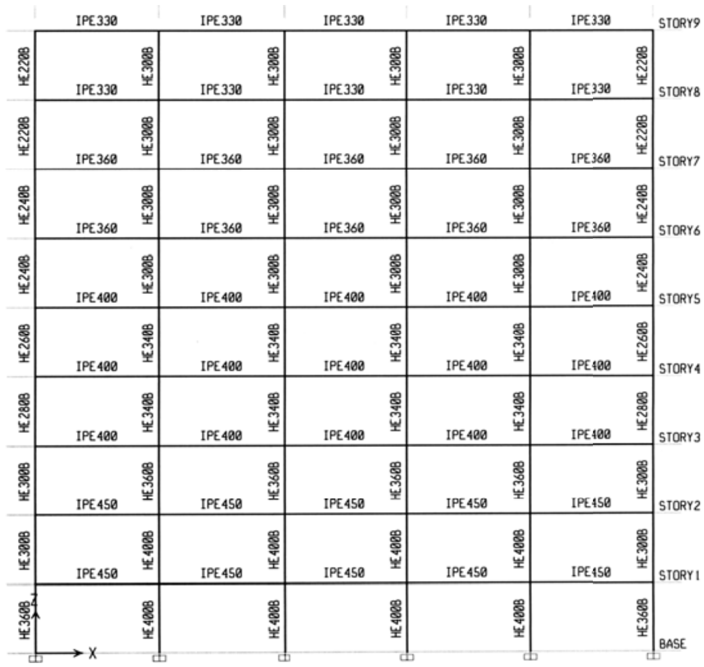


Figure 4. Sample frame without added viscoelastic damper

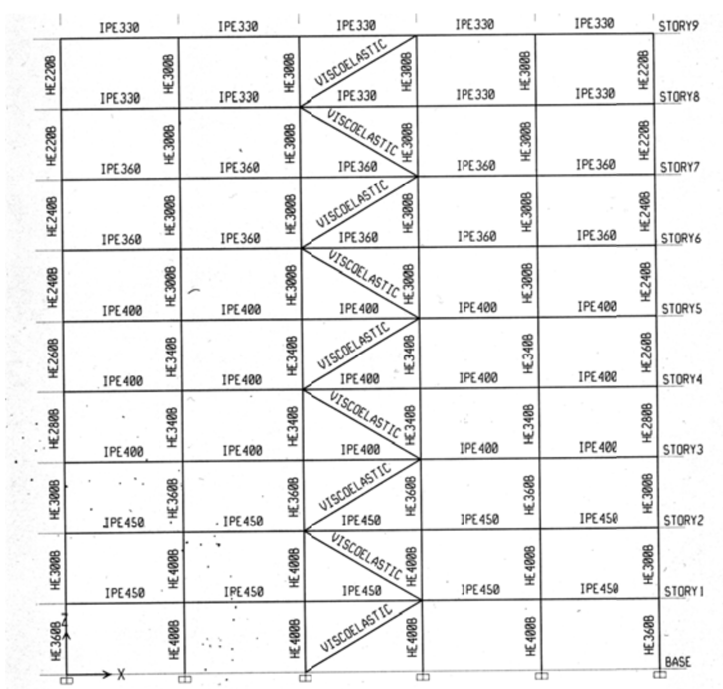


Figure 5. Sample frame with a added viscoelastic damper

4. Scaling Using ASCE 7-05

According to ASCE7-05 for analyzing two-dimensional frame, horizontal acceleration of the ground shall be selected from a real event recorded. When selecting the acceleration, consideration should be given to their magnitude, the distance of the fault and source mechanisms. If the appropriate number of recorded history of ground motion is not available, simulation of the motion should be used. Ground motions must be so scaled that average acceleration response spectra of selected records with damping ratio of 5% in the interval of 0.2T-1.5T (T is the main period of structure) is not less than the standard design spectrum of region [9]. Also according to ASCE7-05 for earthquakes with a probability of 2% in 50 years (MCE), and all active faults known in region, average acceleration response spectra of selected records must be examined for 1.5 times of the area's standard design spectrum. To draw standard design spectrum, it is proposed that values of Ss and S1 be 1.5 and 0.6, respectively; where Ss and S1, are acceleration response spectrum in the short period and the acceleration response spectrum in one-second period in a damping ratio of 5%, respectively (American Society of Civil Engineers, 2005).

4.1. Accelerographs Used

For nonlinear dynamic analysis by using time-history method to evaluate seismic characteristics of frames, eight accelerations different recorded in type C soils (according to USGC classification) can be seen in Table 1.

Table 1. Characteristics of earthquake records used in this study

NO	Earthquake	Station	Magnitude	PGA(g)
1	CapeMendocino(1989)	Petrolia	$M_s = 7.1$	0.662
2	Chi-Chi (1999)	Taiwan	$M_s = 7.6$	0.653
3	Duzce, Turkey (1999)	Duzce	$M_s = 7.3$	0.535
4	Imperial Valley (1979)	El Centro Array #8	$M_s = 6.9$	0.454
5	Kocaeli Turkey (1999)	Yarimca	$M_s = 7.8$	0.349
6	Loma Prieta (1989)	Gilroy Array #2	$M_s = 7.1$	0.322
7	Northridge (1980)	Newhall - Fire Sta	$M_s = 6.7$	0.59
8	Tabas (1978)	Tabas	$M_s = 7.4$	0.852

Since the main period in 9-story structures is equal to 1.5 sec, the interval will be between 0.3 to 2.25 seconds. As it is shown in Diagram (6), scale factor of 1.40 is obtained, which is multiplied in all accelerographs, and is used to analyze the time history. A scale factor of 1.50 can be obtained for 14- and 17-story buildings as well.

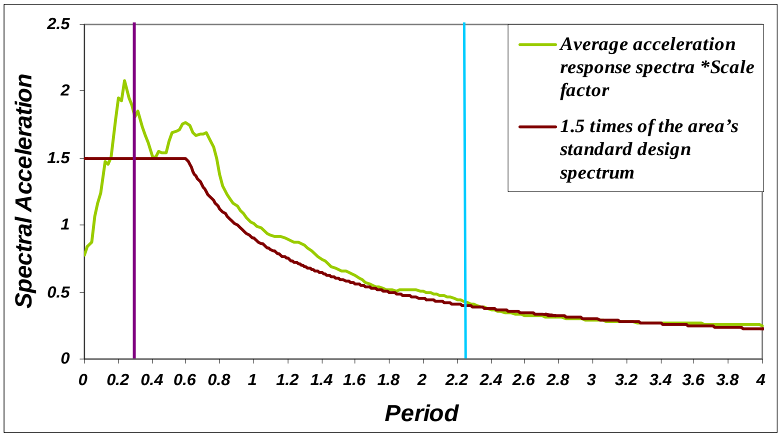


Figure 6. Comparison between chart of scaled response spectrum and chart of 1.5 times the standard design spectrum

5. Determining Damping Ratio Due To the Added Viscoelastic Damper

Given the governing elastic conditions, uniform distribution of damping in the height of the frame, and given that effective damping, vibration modes, and the way that one can arrange dampers are specified, damping ratio of added damper in *m*th mode can be determined from the following equation for multi-story structural frames (NEHRP, 2001):

$$\beta_{Vm} = \frac{T_m}{4\pi} \cdot \frac{\sum_{i=1}^{n} C_i f_i^2 \phi_{ri}^2}{\sum_{i=1}^{n} \frac{W_i}{g} \phi_{im}^2} \qquad (3)$$

Where, T_m = *m*th period of the building with the added viscoelastic damper, W_i = weight of any story of the structure, C_i = damping factor of the damper, ϕ_m = *m*th mode of vibration, f_i = the coefficient of damper placement. Considering that the damper was diagonally installed in the frame (as shown in Fig. 5), i.e. $f_i = \cos\theta$, then $\phi_{ri} = \phi_{im} - \phi_{(i-1)m}$. β_{Vm} =damping ratio due to the added damper in *m*th mode. Considering Equation (3), ratio of damping resulting from the desired added damper is obtained by selecting an appropriate shear cross section and shear thickness for the viscoelastic elements. In other words, the equivalent cross section of brace element relating to the added damper can be determined using a repeatable process to select the right size and the main vibration frequency resulting from it. The result is shown in Table 2 for various models.

Table 2. Effective stiffness and effective damping of the added dampers

Frequency (Hz)	Effective stiffness K_e (KN/cm)	Equivalent cross section of brace element (cm^2)	Period of the main mode	Effective damping C_e (kN.sec/cm)	ratio of the added damper (β_V)	Type of structural model
0.7	62.84	1.74	1.432	16.1	5%	
0.77	212	5.89	1.3	55.96	15%	9 floors
0.87	482.62	13.4	1.143	107.3	25%	
0.57	114.82	3.19	1.767	29.98	5%	
0.64	435.19	12.08	1.556	107.77	15%	14 floors
0.72	804.6	22.34	1.393	205.82	25%	
0.49	163.96	3.8	2	44.16	5%	
0.57	612	17	1.76	164.76	15%	17 floors
0.66	1350.12	37.49	1.5	331.68	25%	

As it was expected, the results indicate that the effective stiffness in the combined frames will be increased with the damping resulting from the addition of elastic damper; and the effective damping applied has also an upward trend, by increasing the main frequency of frames; and the damping ratio resulting from the added dampers show an increase.

6. Seismic Response of Frames vs. Damping Ratio

In this study, structural analysis is carried out for three different damping percentages (5, 15, and 25) due to the added damper. A comparison of amounts of displacement and base shear of a 9-story structure in uncontrolled and controlled modes by inserting viscoelastic damper and 25% damping due to Tabas earthquake is shown in Figures 7 and 8. The results indicate that viscoelastic damper can significantly reduce the seismic responses of structures against earthquakes.

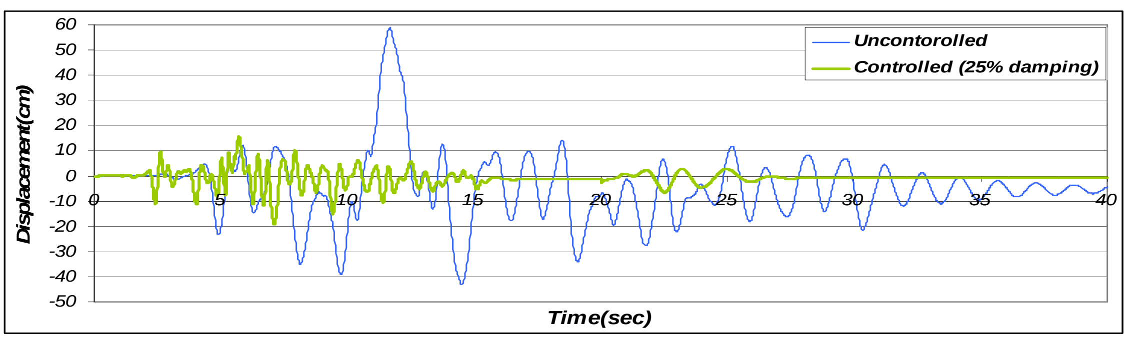


Figure 7. Displacement response of 9-story structure in both uncontrolled and controlled modes with a viscoelastic damper (added damping of 25%) under Tabas earthquake

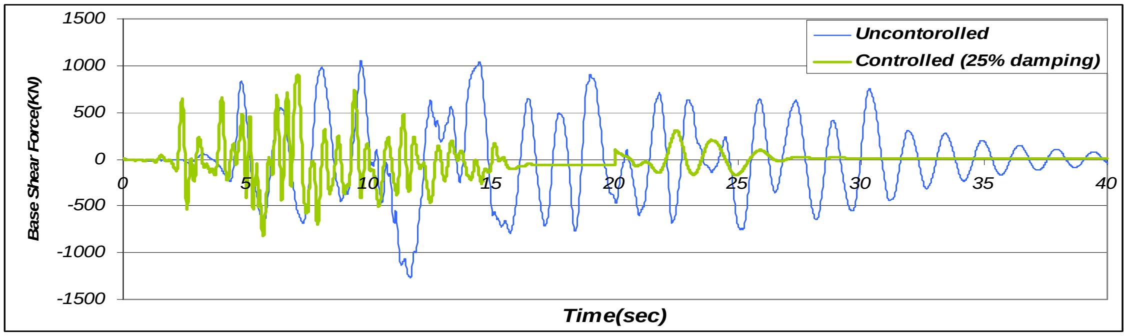


Figure 8. Base shear response of a 9-story structure in both uncontrolled and controlled modes with a viscoelastic damper (added damping of 25%) under Tabas earthquake

Also according to ASCE7-05, if the scaled accelerations in time-history analysis are more than seven in number, the final reflection of structure will be equal to the average values of earthquake response records. So in the following Figures, the maximum base shear of the stories has been compared with the maximum roof displacement under eight scaled accelerations in uncontrolled and controlled modes by added viscoelastic damper and different cases of damping from the intended structures.

Results of maximum base shear of stories for the intended structures are shown in Figure 9, indicating that the maximum base shear for all three structures with a damping ratio of 0.25 due to the added damper can be reduced on average up to 25%; while according to ASCE7-05, minimum seismic base shear used for designing seismic-resistant systems should not be less than $V_{\min} = 0.75V$. For this purpose, the maximum amount of roof displacement will also be examined for the damping ratio of up to 0.25 due to the added damper.

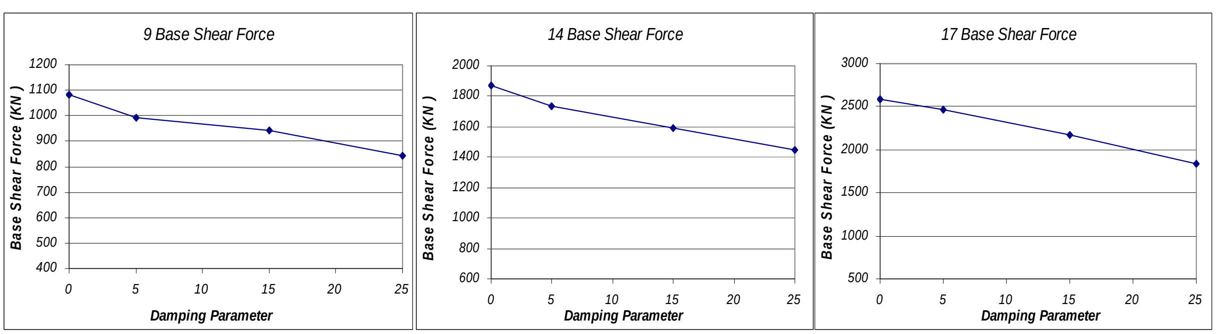


Figure 9. Results of maximum base shear of structures

Results of the maximum roof displacement for the intended structures, which are shown in Figure 10, indicate that increasing the damping ratio leads to a constant downward trend for the maximum story displacement of the roof, and that in case of a damping ratio of 0.25% due to the added damper, the maximum roof displacement will be reduced on average up to 56% for all three structures.

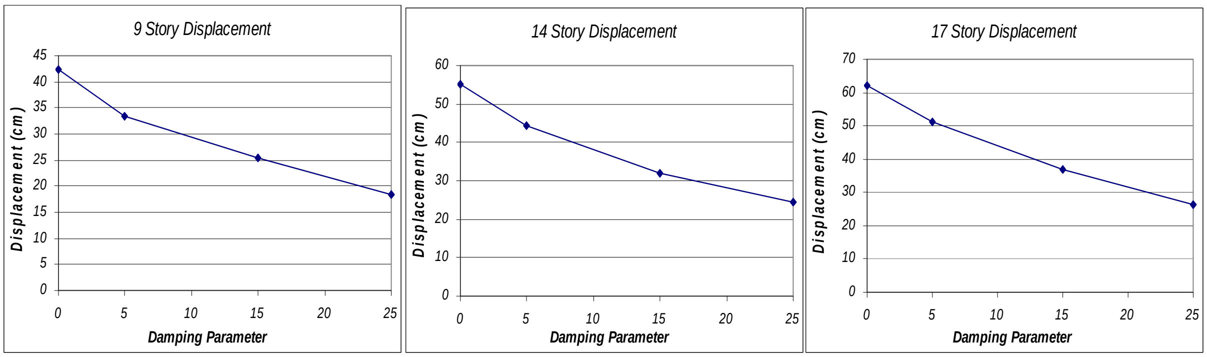


Figure 10. Results of the maximum story displacement of roof

In Figures 11 and 12, structural hysteresis curve in a 9-story structure in uncontrolled and controlled modes by inserting viscoelastic damper are compared with 25% damping due to Tabas earthquake. These results show a very high amount of imposed energy, ductility-demands and displacement-demands of near-fault records. So, to deal with the imposed energy, a structure of high ductility is required. However, added viscoelastic damper and increasing the damping resulted from it lead to reducing the condition for entering within the limits of nonlinear behavior in structural members, and provide dissipation of a part of the energy caused by dampers.

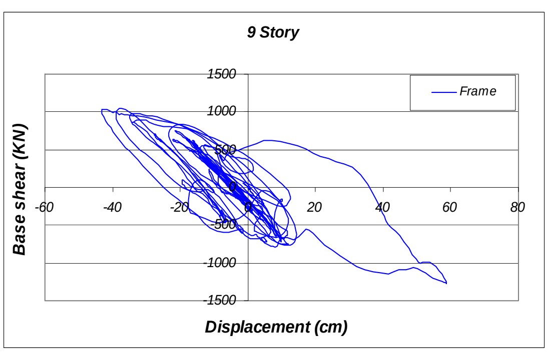


Figure 11. hysteresis curve of a 9-story structure in uncontrolled mode under Tabas earthquake

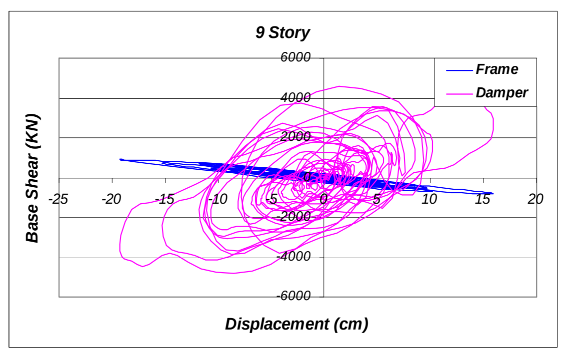


Figure 12. hysteresis curve of a 9-story structure controlled mode with a viscoelastic damper (added damping of 25%) under Tabas earthquake

7. Conclusions

This research has studied the effect of passive viscoelastic dampers on reducing seismic vibration of structures. For this purpose, after two-dimensional models of three of 9-, 14- and 17-story structures were prepared by OpenSees software; the structures with different cases of damping due to the added damper were tested by horizontal accelerations of earthquake, to evaluate the performance of viscoelastic damper. The following were observed from the studies on the structures:

1- Maximum base shear for all three structures with a damping ratio of 0.25 due to the added damper is reduced on average up to 25%. Also, according to ASCE-7, minimum seismic base shear used for designing seismic-resistant systems should not be less than $V_{min} = 0.75V$.

2- Increasing the damping ratio leads to a constant downward trend for maximum story displacement of the roof, and in case of a damping ratio of 0.25 due to the added damper, average of the maximum roof displacement will be reduced on average up to 56% for all three structures.

3- The imposed energy, ductility-demands and displacement-demands of near-fault records are very high. So, to deal with the imposed energy, a structure of high ductility is required. Added viscoelastic damper and increasing the damping resulted from it lead to reducing the conditions for entering within the limits of nonlinear behavior in structural members, and provide dissipation of a part of the energy caused by dampers.

4- The results indicate the very strong effect of passive viscoelastic dampers in reducing seismic response of structures. So, these dampers can be used to make the new structures light, or retrofit existing structures.

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