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SEISMIC RESPONSES OF A FULL-SCALE STEEL FRAME WITH ADDED VISCOELASTIC DAMPERS UNDER BILATERAL EXCITATIONS

Y.-L. Chu[1] and K.-C. Chang[2]

ABSTRACT

Over the past few decades, viscoelastic (VE) dampers have been verified to be capable of providing extra stiffness and damping to structures to reduce the dynamic responses of structures. Nevertheless, the studies performed in the past merely discussed the two dimensional behavior of the scaled down structures with added VE dampers. In order to achieve a comprehensive understanding of the dynamic behavior of structures with VE dampers under actual earthquake excitations, experimental studies of a 3-storey full-scale single bay steel frame subjected to bi-lateral earthquake ground motions simultaneously have been carried out. In this study, a new VE material, which is insensitive to shear strain and behaves viscoelastically even under large deformation, was used. Shaking table test results show that the seismic responses of structure which was added with VE dampers in both bays can be reduced significantly under bi-lateral mild and strong earthquake ground motions. Furthermore, analytical studies had been carried out to predict the seismic responses of the steel frame with added VE dampers. The analysis results show that the responses of the damped frame can be precisely captured by applying equivalent stiffness and damping ratio provided by the added VE dampers into linear dynamic analyses programs, such as ETABS and SAP2000.

Introduction

The two dimensional seismic behavior of scaled or non-scaled structures with added viscoelastic dampers has been extensively examined through many studies of shaking table tests (Aiken *et al.*, 1990; Chang *et al.*, 1992 and Chang *et al.*, 1995). However, the three dimensional behavior of a viscoelastically damped structures under shaking table test has not been studied yet. Therefore, a full-scale, three-story steel frame with added VE dampers in both bays subjected to bi-lateral earthquake ground motions has been carried out in this study. According to the past studies, it is clear that the vibration frequency, ambient temperature and shear strain would greatly affect the dynamic properties of VE dampers (Soong *et al.*, 1997). A new material, recently developed in Japan, was used in this study. This new VE material is less sensitive to shear strain such that it still behaves viscoelastically even under large deformation. Based on the material test, the hysteresis loop for this new type VE dampers behaves rounded in shape, indicating that the seismic response of the structures with new VE dampers can be simulated precisely by applying appropriate stiffness and

[1] Ph.D. Student, Dept. of Civil, Structural and Environmental Engineering, University at Buffalo, Buffalo, NY 14260; formerly, research assistant fellow, National Center for Research on Earthquake Engineering, Taipei, Taiwan 106
[2] Chair and Professor, Dept. of Civil Engineering, National Taiwan University, Taipei, Taiwan 106

damping ratio into the linear dynamic analysis program, such as DRAIN2D+ (Tsai *et al.*, 1994) and ETABS (Ashraf, 1994). In this study, the shaking table test results of a full-scale steel frame with added VE dampers are described. The seismic responses of the bare frame and damped frame under mild and strong bi-lateral earthquake ground motions were also compared. In addition, the symmetry of the steel frame and responses independence of the damped structure in both directions under bi-lateral scaled earthquake ground motions were discussed as well. Subsequently, the numerical model of the VE dampers mentioned above was used to simulate the seismic responses of the damped structure in the dynamic analysis program. Through comparing the experimental results to the simulated responses, the modeling method was proved to be accurate in describing the behavior of the new VE dampers.

Dynamic properties of the VE damper

Damper test

Before conducting the shaking table test, sinusoidal cycling tests were conducted to determine the dynamic behavior of this new material damper. The test frequencies and maximum strain ranged from 1Hz to 4 Hz and 5% to 100%, respectively. The ambient temperature remained at 29^{o}C. Table 1 lists the details of the applied strain and frequency within 10 cycles. Fig. 1 shows the hysteresis loops of the damper under frequency ranging from 1 Hz to 4 Hz and maximum strain remaining at 100%. It is shown that the stiffness of the damper becomes larger as the excitation frequency increases. At the same time, Fig. 1 also indicates that this new viscoelastic damper still behaves linearly even subjected to 100% strain deformation. Fig. 2 illustrates the hysteresis loops of the damper undergoing 1Hz sinusoidal excitation with maximum strain ranging from 30% to 100%. It can be observed from the figure that the stiffness of the damper is slightly dependent on strain.

Table 1. Frequency and strain applied to material test

Temperature, ^{o}C	Frequency, Hz	Strain, %	Cycles
29	0.1, 0.5, 1, 1, 2, 3, 4	5, 10,30, 50,100	10

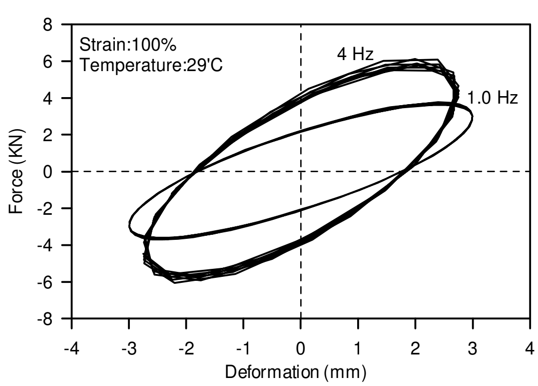


Figure 1. The effect of excitation frequency.

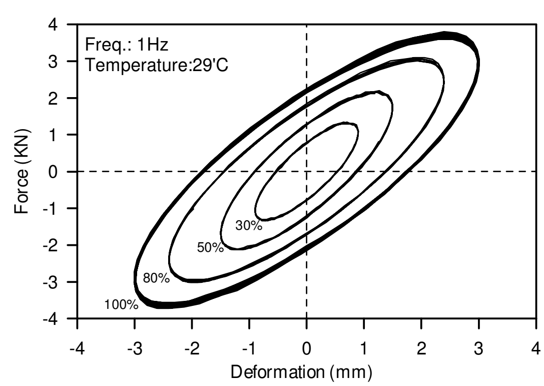


Figure 2. The effect of strain.

Based on the collected sinusoidal cycling test data, the parameters, G', G'' and η of the dampers can be calculated (Chang, 1996). The calculated values of G', G'' and η at excitation frequency 1Hz and 2 Hz with maximum strain varying from 5% to 100% were listed in Table 2. As indicated in early study (Chang *et al.*, 1996), the damper property is dependent on its characteristic strength so that dynamic behavior of the damped structure is indeed nonlinear. However, according to Table 2, these parameters are less sensitive to strain. With this characteristic, it would be easier for engineers to apply this new type VE damper in practical construction works for seismic resistant control and the analyses remain easy as well. Because the iteration process to find the effective frequency and maximum strain is not necessary, only the predication of damping ratio provided by added VE dampers and effective stiffness locating at the same place of the added damper of the bracing are needed, which will be considered later.

Table 2a. Material test results (Excitation frequency at 1 Hz)

$\gamma(\%)$	G' (psi)	G'' (psi)	η
5	17.5489	12.9578	0.7384
10	18.0291	13.3142	0.7385
30	17.5833	12.9541	0.7367
50	17.1142	12.5762	0.7348
80	15.2879	11.0164	0.7206
100	14.9562	10.9695	0.7334

Table 2b. Material test results (Excitation frequency at 2 Hz)

$\gamma(\%)$	G' (psi)	G'' (psi)	η
5	22.5075	18.6417	0.8282
10	23.3779	19.1122	0.8175
30	22.5947	18.5690	0.8218
50	21.9104	18.2982	0.8351
80	19.5557	15.9556	0.8159
100	19.2167	15.9684	0.8310

Empirical Formulae

For practical engineering application, the parameters describing the dynamic property of VE damper can also be expressed in terms of ambient temperature, excitation frequency and maximum shear strain from regression analyses of the test data. The proposed formulae are in forms of:

$$G'(f,T,\gamma_0) = e^a f^b T^c \gamma^d \qquad (1)$$

$$\eta(f,T,\gamma_0) = e^g f^h T^i \gamma^j \qquad (2)$$

where *e* is exponential function; *f* , *T* and γ_0 are excitation frequency (Hz), ambient temperature (^{o}C) and strain(%), respectively.

Based on the regression analysis of the test data, the empirical formulae of this new type VE material at 29 ^{o}C are as follows:

$$G'(f,T,\gamma_0) = e^{2.998} f^{0.254} \gamma^{-0.043} \text{ (psi)} \qquad (3)$$

$$\eta(f,T,\gamma_0) = e^{-0.429} f^{0.217} \gamma^{-0.017} \qquad (4)$$

Fig. 3 and Fig. 4 show the comparison of the G', G' and loss factor η regressed from above-mentioned empirical formulae. It is seen that the empirical formulae can describe these parameters precisely for engineering application purposes.

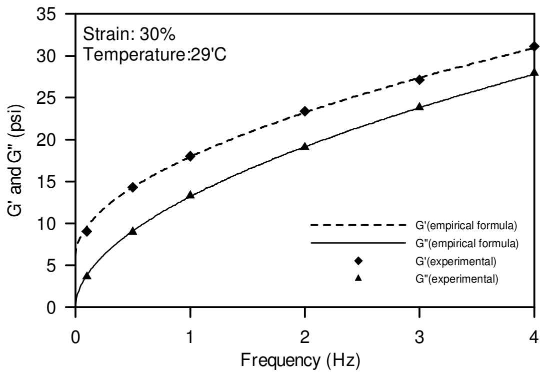


Figure 3. The comparison of G' and G'' estimated from and empirical formula and experimental results.

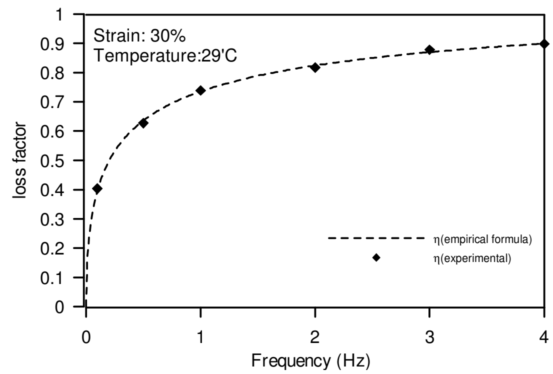


Figure 4. The comparison of η estimated from and empirical formula and experimental results.

Test Description

Test setup

The frame model used in this study, built by National Center for Research on Earthquake Engineering, Taiwan (NCREE), was a 3-story single bay full-scale steel frame with story height of 3 meters for each floor. The center to center distance between columns in long direction and short direction are 4.5m and 3m respectively. The major axis of the column is in short direction. Each floor was added with a concrete block, weighted 10.91 tons for the roof block and 11.21 tons for the rest two blocks, to simulate the inertia mass. The properties of beams, columns and braces of the frame in both directions are listed in Table 3. Two types of installation methods for VE dampers were used in this study. One is named as diagonal-damper-brace used in short direction; the other is floor-damper-brace used in long direction, as shown in Fig. 5.

Table 3. Section properties of beams, columns and braces

	Beam	Column	Brace
A(cm^2)	37.92	62.08	32.06
I_x(cm^4)	2766	4610	---
Z_x(cm^3)	368.8	451.96	---
I_y(cm^4)	---	1601	---
Z_y(cm^3)	---	160.1	---

	Damper Size	
	Floor Damper	Diagonal Damper
Area(cm^2)	41×10=410	33×15=495
Thickness(cm)	0.3	0.3

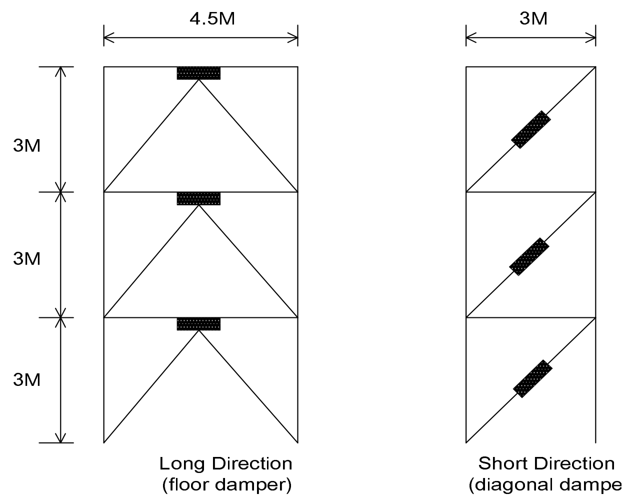


Figure 5. Configurations of the dampers installed.

Test program

El Centro 1940 record was selected as the input earthquake ground motion. The two scaled components of the El Centro, North-South direction (NS) and East-West direction (EW), were imposed to the test frame in long and short direction respectively. This presented the real condition of the structures suffering during earthquake strong motions. In this study, two cases of this experimental study were carried out. Case 1 was conducted to obtain the dynamic characteristics and structural responses of the bare frame (without dampers). According to the preliminary analysis, the peak accelerations of scaled ground motion were limited to 0.12g and 0.15g for long and short direction respectively to prevent the bare test frame from undergoing inelastic deformation. Case 2 was focused on the study of the responses of the test frame with added VE dampers subjected to moderate and strong ground motions. The preliminary analysis also shows that the model remains in elastic range during Case 2. All tests were performed at room temperature set to 29 ^{o}C.

Test Results

Dynamic characteristics of the test frame

To identify the dynamic characteristics of the bare frame and damped frame, scaled 0.1g white noise excitation was used to determine the fundamental characteristics. The transfer functions of the bare frame and damped frame in both directions are plotted in Fig.6. It can be seen from Fig.6 that the responses of the damped frame are smaller than those of bare frame in both directions. In addition, the higher modes responses of the damped frame become insignificant as compared to those of bare frame. From Fig.6, it reveals that the fundamental frequencies of the bare frame are 1.07Hz and 1.5 Hz in long and short direction respectively. The damping ratio, 1.5%, was assumed for the bare steel frame in this study. The frequencies of the damped frame are 2.14 Hz in long direction and 2.34 Hz in short direction. Based on the plotted curves, the damping ratios are about 20% for both directions, which were estimated by half-power method.(Clough, R. W. and Penzien, J., 1993) Therefore, there is a significant increase of fundamental frequency and damping ratio due to the stiffness and viscous damping provided by added VE dampers.

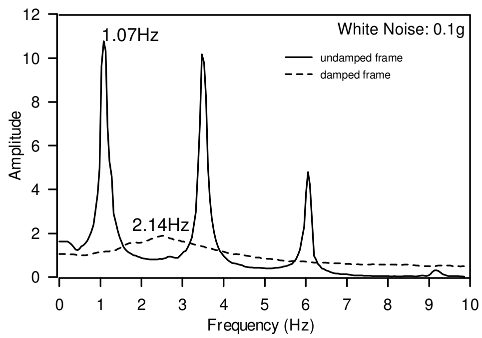


Figure 6a. Transfer function for the damped and undamped frame (Long direction).

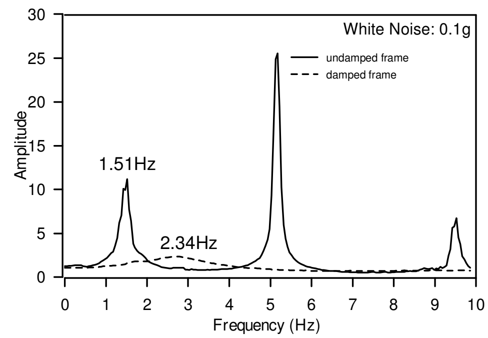


Figure 6b. Transfer function for the damped and undamped frame (Short direction).

Structural responses

During the test, the test structure was subjected to scaled El Centro ground motion bilaterally. In order to check whether the torsion phenomenon exists during the shaking table test process, the responses comparisons for the two bays in long and short direction are presented. Fig.7 shows the comparison results of the structural responses under the scaled 0.15g (long)+0.093g (short) El Centro earthquake, which indicates the structure used is regular and symmetric in geometry. Therefore, the torsion effect is not considered in the simulation study. The force-deformation relationship for VE dampers are shown in Fig. 8. It is obvious that the earthquake energy can be dissipated by VE dampers effectively as shown by the area enclosed in the hysteresis loops. Moreover, from this figure, it is found that the VE dampers remained elastically even experiencing a large deformation. This is concord to the material test results, which means the stiffness of this new material behaves slightly different as the deformation changed. When the peak ground acceleration of the input earthquake is greater than 0.12g and 0.15g for long and short direction respectively, numerically simulated responses of the bare frame obtained from DRAIN2D+ program are used to compare the corresponding experimental results of the damped frame because the bare frame may be damaged if the ground motions greater than these levels are imposed. The roof lateral displacement and acceleration of the frame with and without dampers subjected to scaled 0.5g (long)+0.31g (short) El Centro earthquake in long and short direction are plotted in Fig.9 and Fig.10, respectively. As shown in the figure, the structural responses of displacement and acceleration were effectively suppressed due to the presence of viscoelastic dampers. The floor displacement and acceleration envelopes of the bare frame and damped frame were summarized in Table 4. As read from Table 4, the reduction of the floor acceleration is between 30% and 50%, for the displacements; it is between 40% and 75%.

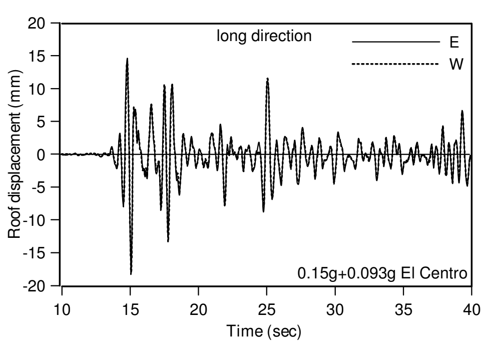


Figure 7a. The roof displacement of east side and west side in long direction.

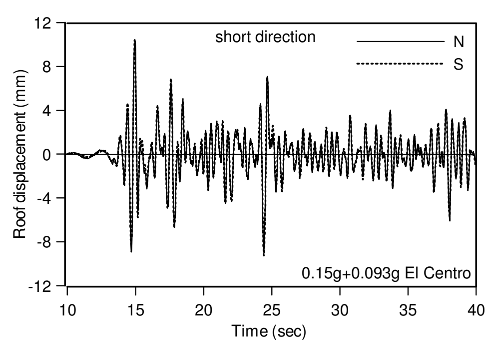


Figure 7b. The roof displacement of north side and south side in short direction.

Table 4a. Displacement envelopes under scaled El Centro earthquake

***Long Direction* Displacement(mm)**									
	Bare frame			Damped frame			Reduction		
	PGA			PGA			PGA		
Floor	**0.15g**	**0.3g**	**0.5g**	**0.15g**	**0.3g**	**0.5g**	**0.15g**	**0.3g**	**0.5g**
3F	74.63	90.85	145.97	18.27	36.90	62.50	75.5%	59.4%	57.2%
2F	55.61	67.62	114.12	14.16	38.60	51.45	74.5%	42.9%	54.9%
1F	25.09	33.67	65.42	7.27	15.30	32.69	71.0%	54.6%	50.0%
***Short Direction* Displacement(mm)**									
	Bare frame			Damped frame			Reduction		
	PGA			PGA			PGA		
Floor	**0.093g**	**0.186g**	**0.31g**	**0.093g**	**0.186g**	**0.31g**	**0.093g**	**0.186g**	**0.31g**
3F	29.45	56.97	81.46	10.30	19.08	32.00	65.0%	66.5%	60.7%
2F	21.16	40.95	59.49	7.50	13.58	22.95	64.6%	66.8%	61.4%
1F	8.80	17.00	24.88	4.62	8.70	13.89	47.5%	48.8%	44.2%

Table 4b. Acceleration envelopes under scaled El Centro earthquake

***Long Direction* Acceleration(gal)**									
	Bare frame			Damped frame			Reduction		
	PGA			PGA			PGA		
Floor	**0.15g**	**0.3g**	**0.5g**	**0.15g**	**0.3g**	**0.5g**	**0.15g**	**0.3g**	**0.5g**
3F	422.4	649.3	951.6	249.9	480.2	674.2	40.8%	26.0%	29.1%
2F	373.4	670.3	868.9	212.7	429.2	498.8	43.0%	36.0%	42.6%
1F	304.8	603.5	747.7	157.8	306.7	500.8	48.2%	49.2%	33.0%
***Short Direction* Acceleration(gal)**									
	Bare frame			Damped frame			Reduction		
	PGA			PGA			PGA		
Floor	**0.093g**	**0.186g**	**0.31g**	**0.093g**	**0.186g**	**0.31g**	**0.093g**	**0.186g**	**0.31g**
3F	320.5	631.5	902.1	163.7	316.5	484.1	48.9%	49.9%	46.3%
2F	237.8	455.7	709.4	140.1	247.9	441.0	41.1%	45.6%	37.8%
1F	133.6	265.7	422.1	129.4	227.4	311.6	3.2%	14.4%	26.2%

Simulation

Prediction of the damping ratio

To predict the damping ratio of the damped frame, the model strain energy (MSE)(Chang *et al.* 1992) was adopted in this study. This method proposed that, if the inherent damping of the bare frame is small, the damping ratio of the frame with added VE dampers can be predicted by the following forms:

$$\xi_i = \left[\frac{1}{2} \left(1 - \frac{1}{\sqrt{1+\eta_i}} \right) \right]^{1/2} \quad (5)$$

$$\eta_i = \eta_{vb} \left(1 - \frac{\phi_i^T K_0 \phi_i}{\phi_i^T K s \phi_i} \right) \quad (6)$$

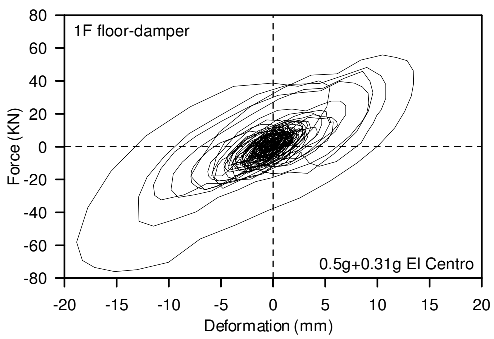
where η_i is the effective loss factor of the i^{th} mode; K_0 is the stiffness matrix of the bare frame; Ks is the stiffness matrix of the damped frame; ϕ_i is the i^{th} mode shape of the VE frame; η_{vb} is the effective loss factor of the damper; η_{vb} can be derived as followings:

$$\frac{1}{K_{vb} + i\eta_{vb} K_{vb}} = \frac{1}{K_v + i\eta_v K_b} + \frac{1}{K_b} \quad (7)$$

$$\eta_{vb} = \frac{K_b}{K_v \left(1+\eta_v^2\right) + K_b} \eta_v \quad (8)$$

where η_v is the loss factor of the VE material; K_v is the storage stiffness of the VE damper; K_b is the axial stiffness of the brace with dampers. Eq. (7) considered the effect of the brace connecting damper and structure, which may cause stiffness change (Chang *et al.* 1998).

The damping ratios calculated by MSE method for this added damper set are 21.4% and 22.3% for long direction and short direction respectively, which are close to the results estimated from the half-power method based on the plotted transfer function.



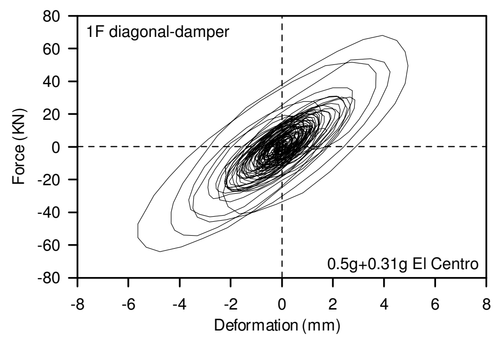


Figure 8. Force-deformation loops of the dampers under scaled 0.5g+0.31g El Centro earthquake.

Structural responses analyses

Once the stiffness and damping ratio provided by the VE dampers were estimated, the structural responses of the damped frame under ground excitations can be simulated in structural analysis program, such as DRAIN2D+, SAP2000 and ETABS etc., by applying the corresponding stiffness and damping ratio provided by the VE dampers into these programs. The simulated time history responses of the roof displacement and acceleration of the damped frame obtained from ETABS program and the corresponding experimental results of the damped structure under scaled 0.5g (long)+0.31g (short) bilateral El Centro earthquake were plotted together in Fig. 11 and Fig. 12. From this figure, it can be seen that the simulated results are in good agreement with the experimental results for both acceleration and displacement terms in both direction. Therefore, it can be concluded that the ETABS program can be used to predict the time history responses of the damped VE frame precisely in both directions.

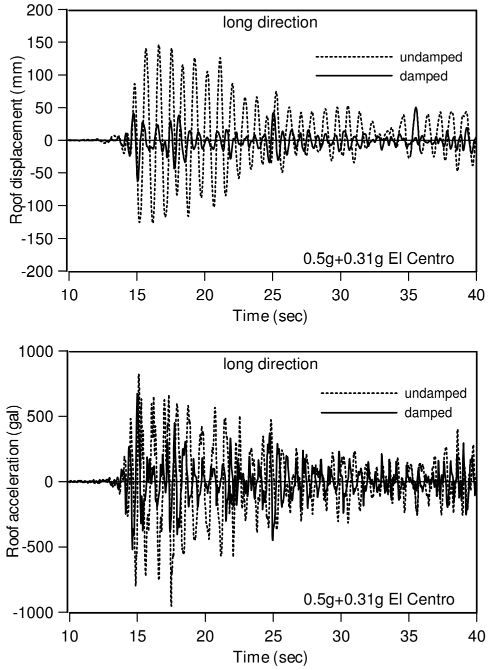


Figure 9. Lateral roof displacement and acceleration of damped and undamped frame in long direction

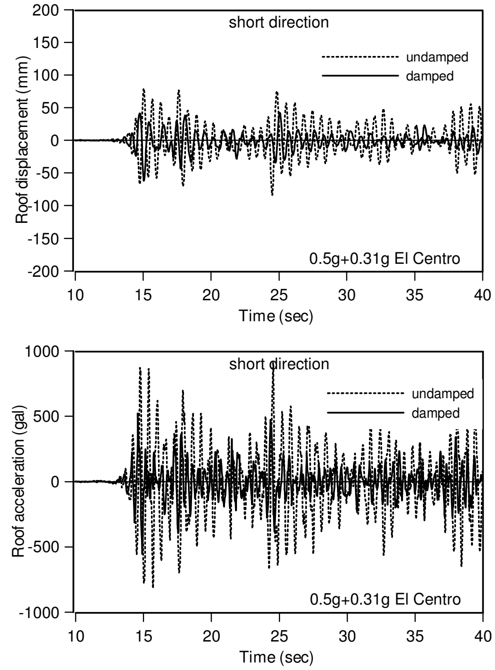


Figure 10. Lateral roof displacement and acceleration of damped and undamped frame in short direction

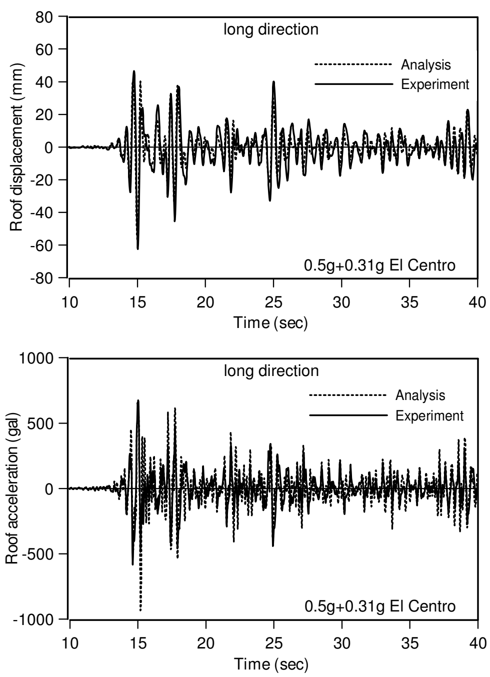


Figure 11. Simulated lateral roof displacement and acceleration of damped frame in long direction

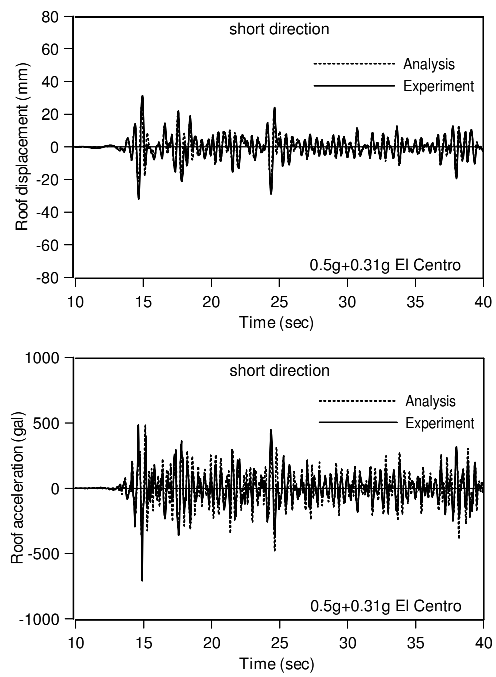


Figure 12. Simulated lateral roof displacement and acceleration of damped frame in short direction

Conclusion

A shaking table test study on the seismic behavior of a full-scale steel frame with added VE dampers subjected to bilateral ground excitations scaled to various peak accelerations simultaneously had been carried out. According to material test and shaking table test, it can be drawn that the new type VE damper used in this study is slightly sensitive to strain and behaves elastically even subjected to a large deformation. The displacement and acceleration in both bays for both directions were compared. The results show that the structure with added VE dampers is symmetric and regular when experiencing bilateral excitations simultaneously. The experimental responses of the damped frame were compared to those of the bare frame. From the comparison results, it shows that the VE dampers still function well and the seismic responses can be significantly reduced due to the presence of VE dampers under bilateral excitations. Further, damping ratios estimated by MSE method and the stiffness calculated by the proposed approach are used to predict the seismic responses of the damped frame under bilateral excitations. The analysis results are consistent with the experimental results. Therefore, based on this study result, it is feasible to apply VE dampers into engineering applications.

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