SHAKING TABLE STUDY ON DISPLACEMENT-BASED DESIGN FOR SEISMIC RETROFIT OF EXISTING BUILDINGS USING NONLINEAR VISCOUS DAMPERS

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ABSTRACT :

This paper presents the experimental and analytical results of a shaking-table study on the elastic and inelastic behavior of a 2/3-scale three-story steel structure retrofitted by the nonlinear viscous dampers. The properties of the dampers used in the test are designed based on the displacement-based design procedure. The retrofitted frame exhibits moderate inelastic behavior under the design ground motion of the 275% El Centro earthquake. The lateral floor displacements, story drifts, floor accelerations, story shears and damper axial forces measured from the frame tested are compared with those obtained from the displacement-based method as well as the nonlinear time-history analysis. It's shown from the study that the addition of nonlinear viscous dampers to the structure results in displacement and force reduction by about 68% to 80% under the 30% El Centro earthquake (PGA $\approx$ 0.1g). Higher-mode responses are significantly diminished. In addition, the displacement and acceleration responses of the structure with dampers are appropriately captured by analytical models in the elastic range. The differences between the experimental and analytical results become noticeable in the inelastic range. The displacement-based evaluation procedure tends to underestimate the responses of the damped structure in the elastic range and overestimate them in the inelastic range.

KEYWORDS: inelastic behavior, shaking-table test, retrofit of existing structures, nonlinear viscous dampers, displacement-based design.

1. INTRODUCTION

This paper presents a displacement-based design procedure for retrofit of buildings using the nonlinear viscous dampers first, and then provides experimental and analytical comparisons. An inelastic shaking-table test on a 2/3-scale 3-story steel building structure retrofitted with the nonlinear viscous dampers are conducted. The dampers are designed by the displacement-based retrofit procedure under the design earthquake ground motion, the 275% El Centro earthquake. In addition to the performance of nonlinear viscous dampers in nonlinear structures, inelastic seismic responses including floor displacements, story drifts, floor accelerations, story shears and damper axial forces measured from the frame tested are compared with those obtained from the displacement-based method and the nonlinear time-history analysis.

2. TEST SETUP AND PROCEDURE

The structure for shaking-table tests is a 2/3-scale 3-story moment resisting steel frame as shown in Fig.1. Overall dimensions of the test frame are 3.0 m$\times$2.0 m in plane with story height of 2.0 m for the first story and 1.75m for the other two. The weight of each floor is 33.3 kN (1F), 32.9 kN (2F) and 22.7 kN (3F). Dimensions of the beams, columns and braces in the direction of input ground motion are H100×100×6×8, H125×60×6×8 and TUBE891, respectively (unit=mm). The frame is retrofitted by the nonlinear viscous dampers at each story. The yield stresses obtained from the tensile tests of the samples are 0.34 kN/mm^2 for the beams and 0.32 kN/mm^2 for the columns. In addition, all their ultimate stresses are 0.45 kN/mm^2.

The test setup and instrumentation are designed to measure the lateral floor displacements, floor accelerations, column strains and shears, beam moments and curvatures, damper deformations and forces. In general, low-level white noise excitations are carried to identify the dynamic characteristics of the structure before every major earthquake-simulation test. The El Centro 1940 N-S earthquake record scaled to various peak ground accelerations (PGA) is used as the seismic input for the shaking-table studies. Three series of tests are conducted. In Test Series I, the moment resistant frame (M.F., bare frame) is tested under the 20% and 30% El Centro earthquakes (PGA $\approx$ 0.07 and 0.1 g). The earthquake intensities used here are not high because these cases are just for checking the validity of the analytical model of the bare frame. Therefore, inelastic behavior is not allowed in the frame in this stage. In Test Series II, the frame retrofitted by nonlinear viscous dampers (N.D.) is tested under the 30%, 70% and 100% El Centro earthquakes (PGA $\approx$ 0.1, 0.24, 0.34 g) to investigate the effectiveness of the dampers when the structure remains elastic. Moreover, the validity of the analytical model of the retrofitted frame can also be checked in the elastic stage. In Test Series III, the retrofitted structure is tested under the 275% El Centro earthquake (PGA $\approx$ 0.98g), which is the design earthquake of the retrofitted frame. In this stage, the frame experienced moderate yielding in some beams and columns.

3. DESIGN AND TEST OF DAMPERS

On the basis of the displacement-based design procedure presented in Chang et al. (2008), the damping coefficient $C_{N,j}$ and velocity exponent β_j for the *j*-th floor viscous damper are determined as $C_{N,1F}$ =0.47 $kN\cdot(s/mm)^{0.51}$, β_{1F} =0.51, $C_{N,2F}$ =0.44 $kN\cdot(s/mm)^{0.48}$, β_{2F} =0.48, $C_{N,3F}$ =0.17 $kN\cdot(s/mm)^{0.55}$, and β_{3F} =0.55. The total design maximum roof displacement (*D*) is 164 mm (drift ratio= 164/5500 mm= 2.98%). Note that since the original properties of the dampers designed are somewhat different from those obtained from the damper property tests, the damping coefficients and velocity exponents used here are obtained from the damper property tests under sinusoidal axial deformation at specified frequencies (0.5, 1.0, 2.0, 4.0 Hz). The weight, capacity force and maximum piston movement for every damper are 0.156 kN, 22.25 kN and $\pm$ 101 mm, respectively. The braces for installing the dampers are diagonal and concurrent at the beam-column center line joints (Fig.1).

4. TEST RESULTS

4.1 Dynamic Characteristics of Structure

Initial dynamic characteristics of the test structure before and after retrofit are identified using 0.05-0.30g white noise excitations. Fig.2 shows the transfer functions (= roof acc./ base acc.) of the structure with and without retrofitted by the nonlinear viscous dampers. It can be seen that the spectral responses of the structure with dampers are obviously smaller than those of the structure without dampers. Additionally, the higher mode responses of the retrofitted frame are insignificant as compared to those of the un-retrofitted case (i.e., the behavior of the structure with dampers are primarily dominated by the first mode). Furthermore, the band width of the first mode for the frame with dampers is clearly larger than that for the frame without dampers. This means that the first modal damping ratio of the former is apparently greater than that of the latter.

The first three modal frequencies of the frame without dampers are 1.81, 5.92, 10.42 Hz (period= 0.55, 0.17, 0.096 sec), and the corresponding damping ratios estimated by the half power method are about 1.5%, 0.4% and 0.2%, respectively, while the first modal frequency of the frame retrofitted with dampers is 1.95 Hz (0.51 sec), and the corresponding damping ratio is 14.3 %. Although it has been analytically shown that viscous dampers have no influence on the stiffness of structures, the first modal frequency of the test structure actually increases from 1.81 Hz to 1.95 Hz after the nonlinear viscous dampers are installed. This is mainly due to the presence of braces and gusset plates used for installing the dampers and for connecting the braces to beam-column joints. Fig.3 presents the response spectra of the El Centro earthquake with damping ratios of 1.5% and 14.3%, respectively. Reductions in the values of pseudo-acceleration due to the added damping alone are about 60%.

4.2 Structural Response

Fig.4 shows the relative displacements and the absolute accelerations at the roof of the structure with and without dampers under the 30% El Centro earthquakes. The overall responses of the structure with dampers are reduced significantly. Table 1 summarizes the maximum response envelopes of relative displacement, inter-story drift, absolute acceleration and story shear for the frame with and without dampers under the 30% El Centro earthquake. It can be seen that not only the displacement and inter-story drift but also the acceleration and story shear can be obviously decreased after the addition of dampers. The maximum responses of the retrofitted structure are only about twenty to thirty percent of those of the structure without dampers. Comparisons of the column shear-drift loops and horizontal component of damper force-drift loops at the first story for the structure with and without dampers under various intensities of the scaled El Centro earthquakes are provided in Fig.5.

The maximum response envelopes of the frame retrofitted with dampers under various earthquakes are summarized in Table 2. Of these cases, only for the one under the 275% El Centro earthquake, inelastic deformations occur in the structure. The frame underwent moderate yielding in the beams of the second and third floors, the bottom of the first story columns, and the top of the second story columns under. However, there is no any joint failure. Fig.5(c) shows the column shear and damper axial force at the first story under the 275% El Centro earthquake. It can be observed that the hysteretic behavior of the frame is obvious and stable, and the damper force-drift loops are still stable and full under the severe earthquake. The residual displacement between the roof and the base of the structure after the test is about 5.5 cm (drift ratio= 1.0%). The maximum responses of the structure under the 275% El Centro earthquake is 16.5 cm for the roof displacement, 3.95% for the first story drift, 1.65g for the roof acceleration and 1.15W for the first story shear. Fig.6 presents the roof lateral displacement and acceleration of the frame under the 275% El Centro earthquake. Detailed responses of the structure tested can be found in Chang et al. (2008).

5. ANALYTICAL SIMULATION OF STRUCTURAL RESPONSE

The computer programs, SAP2000N V8.3 (CSI 2003), for inelastic dynamic analysis of structures are used for simulating the elastic and inelastic responses of the structure with and without nonlinear viscous dampers. The damper properties inputted in both programs are those obtained from the damper property tests. The yield and ultimate stresses adopted in analyses for the beam and column members are those derived from the tensile tests mentioned above. Besides, the post yield stress ratio for beam and column members is taken as 5%. The first modal frequencies derived from SAP2000N for the frame tested are 0.53 sec, which are close to those obtained from the experiments (0.55 sec for M.F. and 0.51 sec for N.D.). Fig.7 presents the experimentally obtained and numerically simulated force-deformation curves of the nonlinear viscous damper located at selected stories under the 100% and 275% El Centro earthquakes, respectively. It can be observed that when the structure exhibits elastic behavior (the 100% El Centro earthquake), the force-deformation curves derived from SAP2000N is acceptable in comparison with the test result. Nevertheless, the differences between them clearly increase after the structure yields (the 275% El Centro earthquake).

Fig.8 shows the experimentally obtained and numerically simulated time-history responses of the roof displacement and acceleration for the retrofitted structure under the 100% El Centro earthquake. It can be seen that because the structure is still elastic under the earthquake, the analytical results predict the experimental data quite well. Even the acceleration response, which is usually more difficult than the displacement response to be predicted, can also be appropriately captured. With the increase of the degree of inelastic deformation in the frame, the differences in displacement and acceleration responses between experiments and analytical results increase. Fig.9 presents the experimentally obtained and numerically simulated time-history responses of the roof displacement and acceleration for the retrofitted structure under the 275% El Centro earthquake. The maximum roof displacement and acceleration obtained from SAP2000N are 16.0 cm and 1.68g, respectively, which are very close to those obtained from the test. Table 3 summarizes the simulated maximum response envelopes of displacement, interstory drift, acceleration, story shear, column shear and damper axial force calculated from SAP2000N for each floor of the retrofitted frame under the 30% to 275% El Centro earthquakes.

6. COMPARISONS WITH DISPLACEMENT-BASED RETROFIT DESIGN

On the basis of the displacement-based design procedure presented in Chang et al. (2008), the responses of the structure with nonlinear viscous dampers under a given level of earthquakes can be estimated. The results including floor displacements, story drifts, floor accelerations, story shears and damper axial forces for the test structure under various earthquakes are summarized in Table 4. Herein, the equivalent viscous damping ratios (ξ_{vm}) provided by nonlinear viscous dampers are computed by two different equations, Eqs.(3) and Eq.(8) of

Chang et al. (2008). The story shear is computed from equilibrium of floor inertial forces, floor mass times absolute floor acceleration. The damper axial force is computed as damping coefficient times damper axial velocity computed as damper axial displacement times $2\pi / T_m$ (i.e., pseudo-velocity). The damper axial displacement is computed as story drift times $\cos\theta_j$.

Table 4 also includes the maximum experimental responses. It can be seen that floor displacements, story drifts and damper axial forces are, in general, underestimated by approximately -8% ~ -36% regardless of the use of Eq.(3) or (8) for the structure remaining elastic, whereas they are generally overestimated by approximately +4% ~ +37% for the structure with nonlinear behavior (i.e., the ELC_275 case). In addition, the errors of responses derived from Eq.(3) is basically greater than those derived from Eq.(8). Nevertheless, Eq.(3) gives good estimations of floor displacements in the inelastic range.

The differences between the experimental and displacement-based outcomes maybe result from: (a). The distribution pattern of the nonlinear static lateral force (F_{im}) for conducting the pushover curve may affect the value of yield displacement. Accordingly, the ductility ratio, the equivalent period and equivalent viscous damping of the equivalent linear systems will be different; (b) For use of Eqs.(3) and (8), the periods and mode shapes of the damped structure are assumed to be the same as those of the structure exclusive dampers even if the damped structure is actually non-classically damped. Moreover, the modal analysis procedure valid only for linear structures is assumed to be applicable to nonlinear structures with nonlinear dampers. (c). The fundamental of the displacement-based retrofit design method is the equivalent linear system which is just an approximate method. Although it can averagely predict the maximum nonlinear responses of structures well, the error between them may be obvious for an individual case.

7. CONCLUSIONS

The advantage of the displacement-based design is that the structure can be directly designed by allowable displacements not by forces. This characteristic is especially useful for retrofit design of structures since the retrofit target of them is commonly the floor lateral displacements or story drift ratios. The displacement-based design results of a 2/3-scale 3-story steel structure seismically retrofitted using nonlinear viscous dampers are discussed and compared with the results of shaking-table tests. The conclusions are drawn as follows.

It can be observed no matter from the transfer function (Fig.2) or from the test results of the frame with and without dampers under the 30% El Centro earthquake (Fig.3, Table 1) that the responses (including the displacement, inter-story drift, acceleration and story shear) of the test frame are significantly reduced after the nonlinear viscous dampers, which provide a damping ratio of 14.3%, are added for retrofit. Under the 30% El Centro earthquake, the addition of the nonlinear viscous dampers results in floor displacement and story drift reduction by 70% to 74%, floor acceleration reduction by 68% to 79%, and shear force reduction by 69% to 80%. Besides, even under the severe motions of the 100% and 275% El Centro earthquakes, the energy dissipation behavior of the dampers is still stable and reliable (Figs.5 and 7). Furthermore, the responses caused by higher modes can be effectively reduced by the dampers (Fig.2). Therefore, if the dampers are applied to irregular structures, the effects of higher modes in such structures may be obviously decreased.

Theoretically, the fundamental vibrating period of structures is not affected by viscous dampers. Nevertheless, it is shown from the test that the stiffness of the structure with dampers is slightly larger than that of the structure without dampers because of the presence of the gusset plates and the braces for installing the dampers. The gusset plates installed in beam-column joints will lessen the effective length of the beam and column members. In the elastic range, the displacement and acceleration responses of the structure retrofitted with nonlinear viscous dampers can be appropriately estimated by SAP2000N (Fig.8). However, with the increase of the inelastic behavior in beams and columns, the differences between the experimental and numerically simulated results increase (Fig.9).

In the elastic range, the maximum seismic responses of displacements and damper axial forces estimated by the displacement-based evaluation procedure are underestimated by nearly -8% to -36%, whereas they are overestimated by nearly +4% to +37% in the inelastic range. Eq.(3), the equivalent damping provided by nonlinear viscous dampers, gives good estimations of floor displacements for the structure in the inelastic range. However, in the elastic range, the errors of responses obtained from the equation is usually larger than those obtained from Eq.(8).

8. REFERENCES

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Table 1. Response envelopes obtained from shaking-table tests under 30% El Centro earthquake.

Structure	Rel. floor disp. (mm)			Story drift (%)			Abs. floor acc. (g)			Story shear /W		
	2F	3F	RF	1st	2nd	3rd	2F	3F	RF	1st	2nd	3rd
M.F.	15.4	30.8	40.7	0.77	0.89	0.58	0.38	0.44	0.72	0.36	0.31	0.20
N.D.	4.6	8.9	11.3	0.23	0.25	0.15	0.12	0.14	0.15	0.11	0.08	0.04
N.D./M.F	0.30	0.29	0.28	0.30	0.28	0.26	0.31	0.32	0.20	0.31	0.26	0.20

Table 2. Response envelopes obtained from shaking-table tests for retrofitted structure under 30%-275% El Centro earthquakes.

Intensity	Rel. floor disp. (mm)			Story drift (%)			Abs. floor acc. (g)			Story shear /W			Column shear /W			Damper axial force /W		
	2F	3F	RF	1st	2nd	3rd	2F	3F	RF	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
ELC_030	4.6	8.9	11.3	0.23	0.25	0.15	0.12	0.14	0.15	0.11	0.08	0.04	0.11	0.08	0.03	.048	.031	.016
ELC_070	16.6	31.0	39.2	0.83	0.85	0.49	0.33	0.44	0.51	0.37	0.27	0.12	0.37	0.28	0.13	.097	.060	.032
ELC_100	23.6	45.6	57.4	1.18	1.26	0.69	0.46	0.61	0.72	0.54	0.39	0.19	0.52	0.39	0.19	.113	.072	.034
ELC_275	79	138	165	3.95	3.38	1.59	0.90	1.29	1.65	1.07	0.77	0.38	1.03	0.81	0.41	.172	.109	.078

Note: ELC_030= 30% El Centro, ELC_070= 70% El Centro, etc.

Table 3. Response envelopes obtained from SAP200N under 30%-275% El Centro earthquakes.

Intensity	Rel. floor disp. (mm)			Story drift (%)			Abs. floor acc. (g)			Story shear /W			Column shear /W			Damper axial force /W		
	2F	3F	RF	1st	2nd	3rd	2F	3F	RF	1st	2nd	3rd	1st	2nd	3rd	1st	2nd	3rd
ELC_030	4.5	8.7	11.3	0.23	0.24	0.14	0.12	0.15	0.17	0.12	0.08	0.04	0.09	0.07	0.03	.047	.043	.018
ELC_070	15.4	30.1	38.9	0.77	0.85	0.50	0.34	0.46	0.54	0.35	0.26	0.12	0.33	0.25	0.11	.090	.070	.039
ELC_100	22.1	43.2	55.4	1.1	1.21	0.71	0.47	0.64	0.76	0.51	0.37	0.17	0.48	0.36	0.17	.112	.081	.040
ELC_275	77.4	132	160	3.87	3.25	1.74	1.07	1.38	1.68	0.99	0.73	0.37	0.95	0.71	0.36	.180	.120	.071

Table 4. Summary of results obtained from displacement-based design procedure for test structure.

Intensity	Equivalent Damping (ξ_{vm})	Rel. floor disp. (mm)			Story drift (%)			Abs. floor acc. (g)			Story shear /W			Damper axial force /W		
		2F	3F	RF	2F	3F	3rd	2F	3F	RF	1st	2nd	3rd	1st	2nd	3rd
ELC_030	Experimental	4.6	8.9	11.3	0.23	0.25	0.15	0.12	0.14	0.15	0.11	0.08	0.04	.048	.031	.016
	Eq.(3)	3.5	7.0	9.0	0.18	0.20	0.12	0.08	0.14	0.18	0.12	0.09	0.04	.038	.028	.012
	Eq.(8)	4.1	8.1	10.4	0.20	0.23	0.13	0.08	0.15	0.19	0.13	0.10	0.05	.041	.030	.013
ELC_070	Experimental	16.6	31.0	39.2	0.83	0.85	0.49	0.33	0.44	0.51	0.37	0.27	0.12	.097	.060	.032
	Eq.(3)	10.7	21.4	27.5	0.54	0.61	0.35	0.21	0.36	0.47	0.32	0.25	0.12	.069	.047	.020
	Eq.(8)	12.5	25.0	32.0	0.62	0.71	0.41	0.22	0.39	0.51	0.35	0.27	0.13	.075	.051	.021
ELC_100	Experimental	23.6	45.6	57.4	1.18	1.26	0.69	0.46	0.61	0.72	0.54	0.39	0.19	.113	.072	.034
	Eq.(3)	17.8	35.5	45.6	0.89	1.02	0.58	0.31	0.56	0.72	0.49	0.39	0.18	.091	.060	.025
	Eq.(8)	20.1	40.3	51.6	1.01	1.15	0.65	0.33	0.61	0.78	0.53	0.42	0.20	.097	.064	.026
ELC_275	Experimental	79	138	165	3.95	3.38	1.59	0.90	1.29	1.65	1.07	0.77	0.38	.172	.109	.078
	Eq.(3)	64	128	164	3.20	3.66	2.08	1.00	1.75	2.27	1.53	1.20	0.58	.184	.112	.048
	Eq.(8)	67	134	172	3.36	3.84	2.18	1.03	1.77	2.30	1.55	1.22	0.59	.189	.114	.050

Fig.1. Layout of 2/3-scale 3-story test structure.

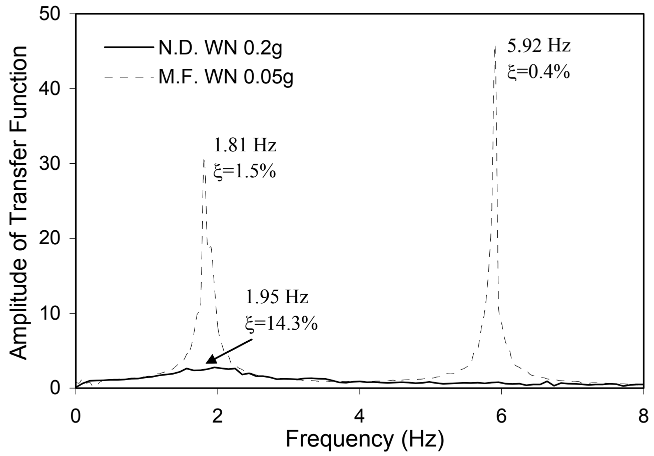

Fig.2. Transfer functions for structures with and without dampers.

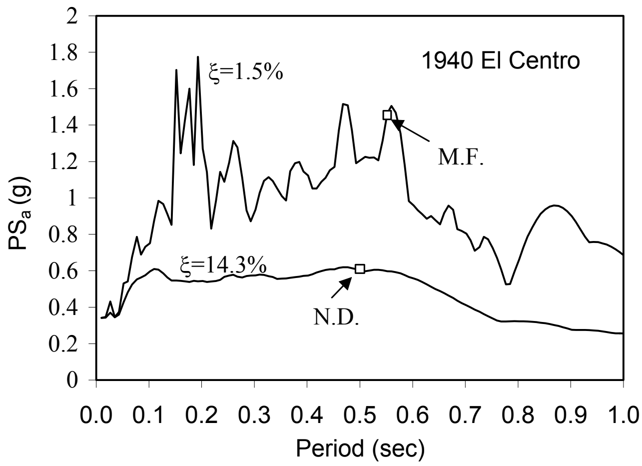

Fig.3. Response spectra of 1940 El Centro Earthquake.

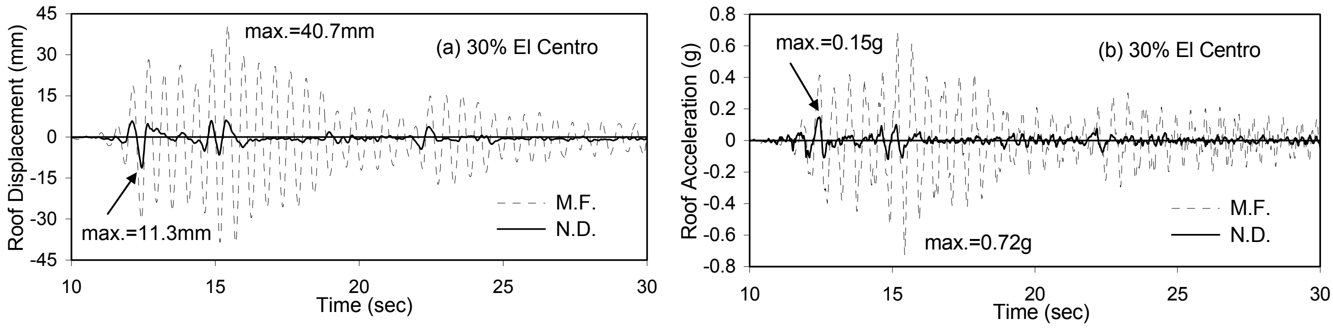

Fig.4. Roof lateral (a) displacement and (b) acceleration under 30% El Centro earthquake.

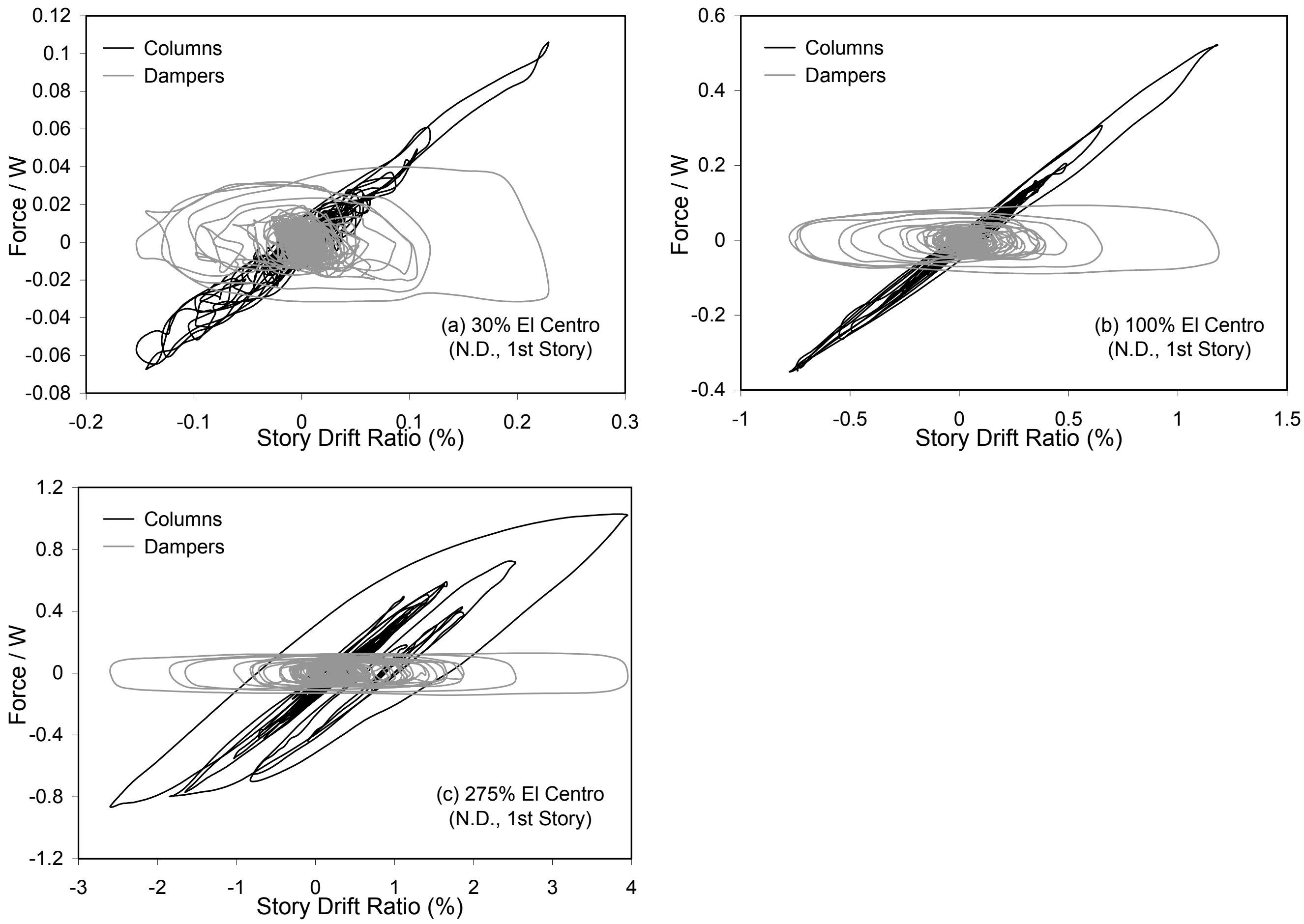

Fig.5. Column shear and damper force at the first story under various earthquakes.

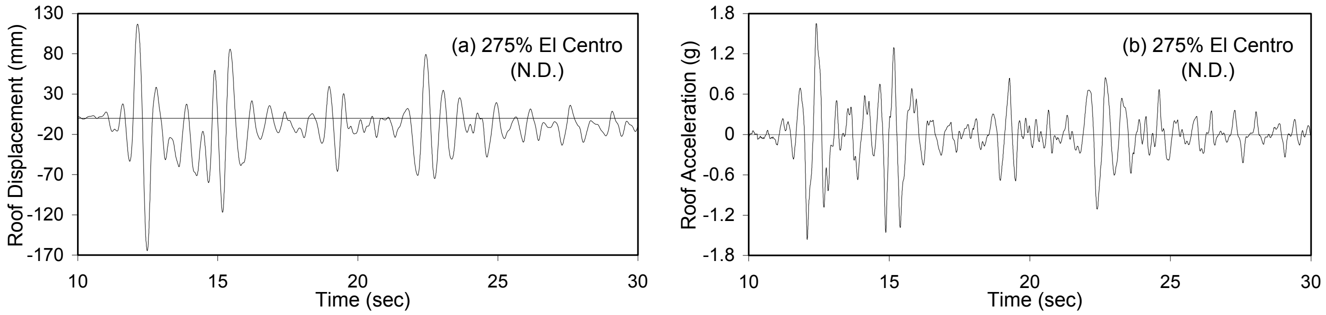

Fig.6. Roof lateral (a) displacement and (b) acceleration of retrofitted structure under 275% El Centro earthquake.

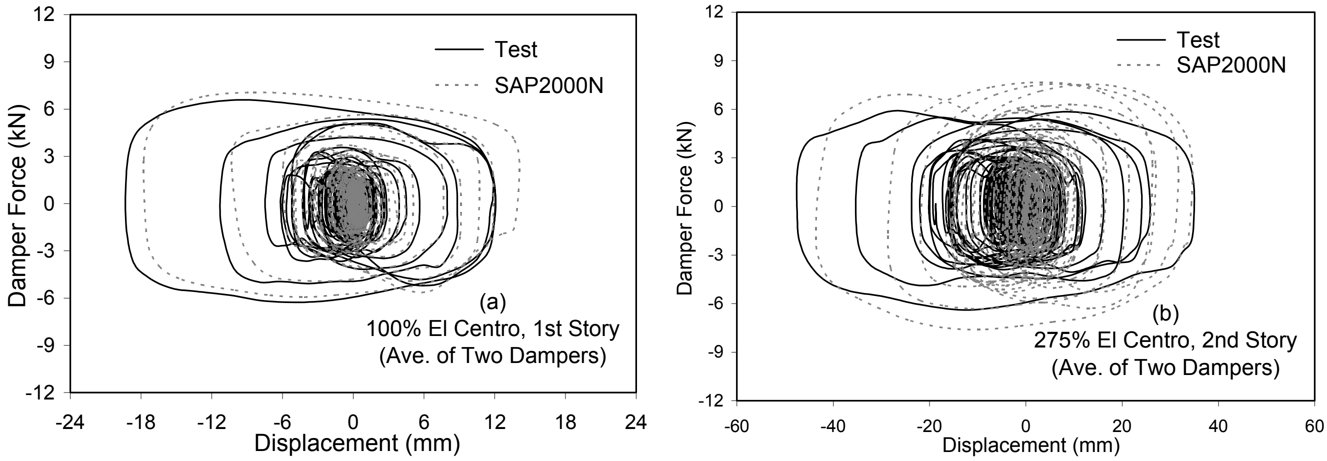

Fig.7. Simulation of damper force at the 1st story under (a) 100% and (b) 275% El Centro earthquakes.

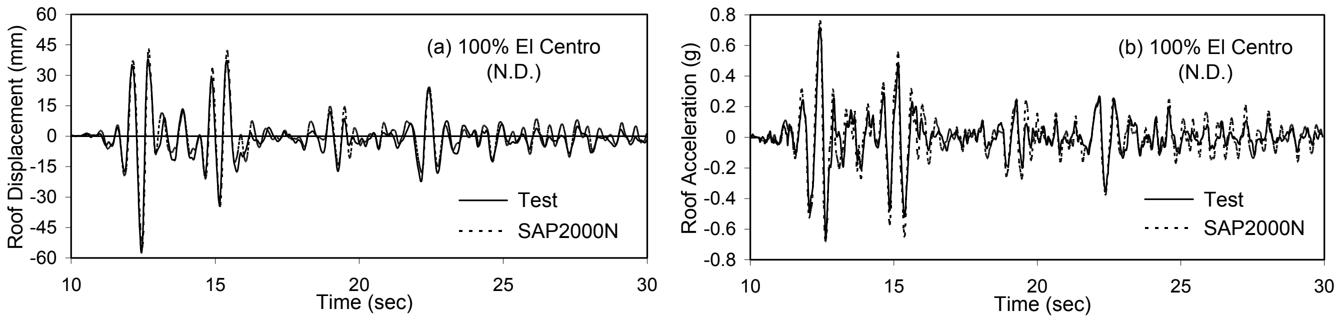

Fig.8. Simulation of (a) roof displacement and (b) roof acceleration for retrofitted structure under 100% El Centro earthquake.

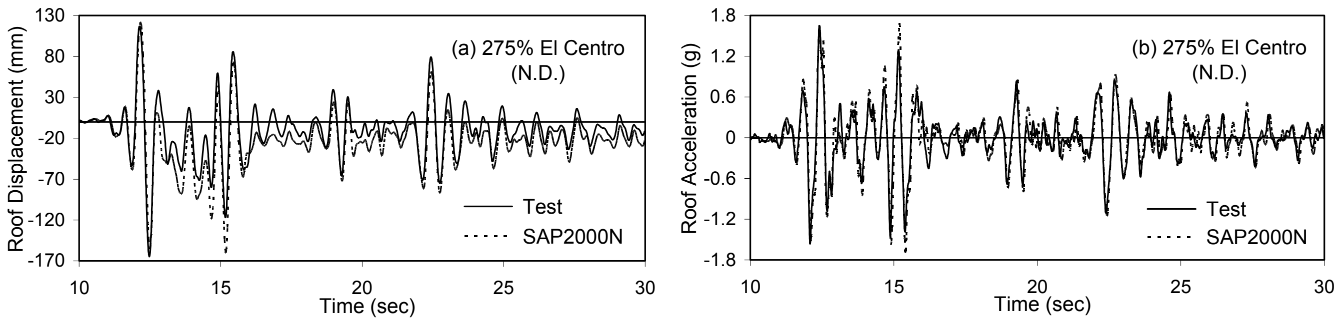

Fig.9. Simulation of (a) roof displacement and (b) roof acceleration for retrofitted structure under 275% El Centro earthquake.