Performance of Structures with Dampers

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Abstract

The purpose of this paper is to evaluate the earthquake performance of structures with passive energy dissipators. This Paper addresses the following issues: 1) evaluation of seismic intensity levels at which frames incorporating the energy dissipating system (EDS) remain elastic; 2) performance evaluation of frames incorporating an EDS for high intensity ground shaking; and 3) evaluation of SEAOC Blue Book provisions. Linear time history analyses indicate that frames with an EDS generally remain elastic during earthquake events that do not greatly exceed the UBC Zone 4 response spectrum. Nonlinear time history analyses indicate the following: 1) the frames with an EDS can provide "immediate occupancy performance" for high intensity earthquakes; 2) the performance level of the frames with an EDS exceeds that of frames without an EDS; and 3) the performance of the frame with an EDS, which was designed per Blue Book provisions, can exceed life safety performance.

Introduction

The purpose of this paper is to evaluate earthquake performance of structures with passive energy dissipators. Time history analyses were conducted on three steel moment frame buildings: 1, 5, and 11 stories in height designed to conform to the strength and drift requirements of the Uniform Building Code (UBC). Linear and nonlinear computer models with and without EDS were subjected to various recorded and synthetic time histories of ground motion.

Input Ground Motions

A total of eight acceleration time histories were entered into computer models. Six time histories were selected from the (M 6.7) 1994 Northridge earthquake that produced a variety of strong ground motions, including those with strong near field effects. These six time histories represent a reasonable range of the combination of peak ground acceleration, peak ground velocities and peak ground displacements of spatial ground motion variability in the near field. These include the effects for seismic source directivity in the recordings from the forward and backward azimuth (Singh 1985). Two spectra matched time histories were also used in this study. One time history was matched to the UBC Zone 4, Soil type S_d response spectra. The other was matched to a site specific 500-year return spectra for an S_d type site in Redwood City, California, at a distance of approximately 6 kilometers from the San Andreas Fault. Both time histories were synthesized using procedures described by Singh (1994). The ground motion parameters (peak ground acceleration, peak ground velocity, and peak ground displacement) for selected time histories are listed in Table 1. The time histories are listed in the descending order of peak ground velocity.

Table 1. Ground Motion Parameters for Selected Time Histories

Station Name	*Site Geology*	*Peak Acceleration [g]*	*Peak Velocity [in./sec.]*	*Peak Displacement [in.]*
Sylmar, 360º	Alluvium	0.892	50.7	12.80
Redwood City	Alluvium	0.800	38.8	10.40
Newhall, 90°	Alluvium	0.610	29.4	6.93
UBC, Zone 4	S_d	0.400	19.6	11.70
Santa Monica, 90°	Alluvium	0.901	16.5	5.63
U.C.L.A., 360°	Alluvium	0.634	8.6	2.87
Moorpark, 180°	Alluvium	0.297	8.0	1.73
Downey360°	Deep Alluvium	0.223	5.0	0.75

Northridge earthquake recordings from stations Sylmar (360º) and Newhall (90º) and the synthetic record for Redwood City, which exceed the UBC spectra in terms of acceleration and velocity, represent the enhanced ground motions found in the forward azimuth of near field recordings. The 5 and 20 percent damped acceleration spectra are shown in Figure 1. Northridge earthquake recordings from stations Santa Monica (90º) and UCLA (360º) represent the ground motions from back or side azimuth in the near field and are closer to the code type ground motions.

Description of Structures

Three steel moment resisting frame buildings with 1, 5, and 11 stories in height were used in the analysis. The typical floor height is 14 ft. with the exception of first floor height of 16 ft. 4 in. (see Figure 2). The footprint is 105 ft. x 130 ft. A typical bay is 21 ft. x 26 ft. The lateral load resisting system consists of perimeter steel moment frames. There are a total of two 2-bay moment frames in each principal direction. The typical bay length of the moment frames is 26 ft. The last bay is not used. The floor diaphragms were composed of cast-in-place concrete over metal deck. Floor dead load, including partition load, varies between 80 psf and 100 psf. Floor live load is 50 psf and is typical of office usage. Tributary weight of the frame is 773^k at the 2^{nd}, 456^k at the 3^{rd}, 448^k at all other floors and 556^k at roof level. Each building is designed to conform to the special moment frame requirements of the UBC for Seismic Zone 4 with S_d soil conditions. Drift criteria was the governing factor for designing member sizes. The first mode is the predominant mode shape for all structures. The fundamental periods of the 1, 5, and 11 story buildings are 0.5, 1.4, and 2.5 seconds, respectively. Period shift by added damping is negligible for practical purposes. Dampers are added at each floor. Quantity of damping is estimated as equivalent to 20 percent of critical damping for the first mode. The damping constant is calculated in proportion to the story stiffness.

Damping Ratio = Cw/2K
C = Total story damping constant
W ☐ = Circular frequency at the first mode
K = Story stiffness
For 1-story building:

Damping ratio = 0.2
$W = 12.56\ ^{rad}/_{sec}$ (T = 0.5 sec)
$K = 314\ ^{k}/_{in}$

Using the above equation, a value of 10 $^{k\text{-}sec}/_{in}$ (total story damping) was obtained for this model. Each discrete viscous damping element is mounted on Chevron-type “driver” braces. Braces are TS10x10x5/8. Maxwell elements were used for all analyses. It is very important to use these elements to capture finai stiffness of bracing elements, even when damper forces were limited to reasonable values (300 - 400^{k} max damper force) for buildings. NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273) was used for earthquake performance criteria including plastic hinge rotation (ATC 1997). The performance criteria are described below.

Linear Time History Analysis

1-Story Model. Linear 2-dimensional models were constructed using ETABS 6.1 (CSI, 1996). Fluid viscous dampers are modeled as discrete nonlinear damping elements mounted on chevron driver braces. Nonlinear elements are limited to these discrete damping elements. Beams and column elements are limited to linear response. The definition in ETABS describes this procedure as nonlinear time history. However, to make a distinction with Drain 2DX analyses described later in this section, a term “linear time history” was used for this paper. Twenty percent of critical damping is provided for the FVDs as described above. Structures are assumed to have 2 percent inherent damping. The results of the linear time history analyses show that the frame remains elastic in all cases except for the Newhall and Sylmar records. For the Sylmar record the frame shows a maximum stress ratio of 1.74. The stress ratio is the ratio of force demand to force capacity. Force demand is defined as 1.2 DL + 0.5 LL + 1.0 EQ. Force capacity is defined as load and resistance factor design (LRFD) strength using an appropriate phi factor. Maximum interstory drift in all cases, with the exception of Newhall, Sylmar and Redwood City records is within the drift ratio of 0.007. The drift ratios for the Newhall, Sylmar, and Redwood City records are higher because they exceed the code motions due to seismic source directivity effects in the near field but are still well within the drift ratio of 0.025 (see Table 2). The drift ratio may be a useful tool to gauge architectural and structural damage. The drift ratio of less then 0.01 may be considered to provide immediate occupancy and less than 0.025 to provide life safety. Drift ratio calculated by linear analysis could approximate drift ratio in an inelastic structure. This concept “equal displacement approximation” is used in the *Uniform Building Code.*

5-Story Model. The linear time history analyses show that the frames remain elastic in all cases except for the Newhall, Sylmar, and Redwood City recordings. For the Sylmar record, the frame shows a maximum stress ratio of 2.70. Maximum drift ratio in all cases except the Newhall, Sylmar, Redwood City and UBC Zone 4 is within the drift ratio of 0.008 for the Newhall, Sylmar, and Redwood City records the frame is within the drift ratio of 0.025 (see Table 3). The damping ratio is much higher than 20% for high modes (second mode and higher). This is very beneficial for reducing the earthquake response parameters at high modes in 5 and 11-story models.

11-Story Model. The results are similar to the 1 and 5 story models. The linear time history analyses show that the frame remains elastic for all cases except for the Newhall, Sylmar, and Redwood City records. The Santa Monica station record produces a maximum stress ratio of 1.1, which is near the elastic response. The Sylmar record results in maximum stress ratio of 2.33. Maximum drift for all cases except Santa Monica, Newhall, Sylmar, and Redwood City station records is within the 0.008 drift ratio. For the Sylmar and Redwood City recordings the frame is within 0.025 drift ratio (see Table 4).

Summary of Linear Analyses

The above analyses indicate that the frames with an EDS can provide equivalent to immediate occupancy performance levels for earthquake events that do not exceed the UBC Zone 4 response spectrum. The drift ratio of less than 0.01 may be considered to provide immediate occupancy performance, and less than 0.025 to provide life safety occupancy.

Table 2. Linear Time History Analysis of 1-Story Model (T_1=0.5 Seconds)

Time History	*Drift Ratio*	*Base Shear Coefficient*	*Column Stress Ratio*	*Beam Stress Ratio*
Sylmar, 360º	0.014	1.25	1.74	1.03
Redwood City	0.008	0.75	1.06	0.63
Newhall, 90º	0.009	0.83	1.16	0.69
UBC, Zone 4, S_2	0.006	0.58	0.84	0.49
Santa Monica, 90º	0.005	0.50	0.62	0.36
U.C.L.A., 360º	0.004	0.36	0.51	0.30
Moorpark, 180º	0.003	0.29	0.41	0.24
Downey, 360º	0.002	0.21	0.30	0.17

Note: Letter T_1 is undamped first mode of structure.

Table 3. Linear Time History Analysis of 5-Story Model (T_1=1.4 Sec.)

Time History	*Drift Ratio*	*Base Shear Coefficient*	*Column Stress Ratio*	*Beam Stress Ratio*
Sylmar, 360º	0.022	0.63	1.65	2.70
Redwood City	0.015	0.39	1.10	1.72
Newhall, 90º	0.011	0.36	0.91	1.39
UBC Zone 4, S_2	0.008	0.23	0.71	1.00
Santa Monica, 90º	0.007	0.19	0.54	0.85
U.C.L.A., 360º	0.005	0.13	0.41	0.53
Moorpark, 180º	0.003	0.15	0.37	0.42
Downey, 360º	0.002	0.07	0.21	0.29

Table 4. Linear Time History Analysis of 11-Story Model (T_1=2.5 Sec.)

Time History	*Drift Ratio*	*Base Shear Coefficient*	*Column Stress Ratio*	*Beam Stress Ratio*
Sylmar, 360º	0.017	0.29	1.80	2.33
Redwood City	0.011	0.21	1.32	1.59
Newhall, 90º	0.008	0.16	0.91	1.11
UBC Zone 4, S_2	0.007	0.14	0.88	1.02
Santa Monica, 90º	0.008	0.15	0.92	1.10
U.C.L.A., 360º	0.004	0.08	0.58	0.59
Moorpark, 180º	0.002	0.07	0.40	0.32
Downey, 360º	0.001	0.04	0.30	0.18

Nonlinear Time History Analysis

Two-dimensional models were constructed using DRAIN 2DX (Prakash et al. 1993). Steel beams and columns are modeled with plastic hinge beam-column elements. Bilinear behavior is assumed with 5 percent plastic hardening. The fluid viscous dampers are modeled as discrete damping elements mounted on chevron braces. The specification of models is the same as linear models. This study is limited to examination of linear dampers. Using nonlinear dampers such as damping force proportional to a fractional power of the velocity may provide larger energy absorption. Two computer models for each prototype building were created. One is a frame with FVD ('damped' frame), and the other is frame without FVD ('bare' frame). Five percent inherent damping is provided for both models. Inherent damping is modeled as mass proportional damping at each mode.

1-Story Model. The Newhall and Sylmar records were used for this analysis because they produced overstress using the linear analysis. For the damped frame, the Sylmar record resulted in the maximum plastic hinge rotation of 0.9 percent (0.009 radian), which is lower than the immediate occupancy hinge rotation limitation of 1.0 percent. For the bare frame, however, there is a significant increase in number and magnitude of plastic hinges. Maximum hinge rotation on 2.0 percent occurs for this record. The base shear coefficient for the damped frame is higher that the bare frame. The formation of plastic hinging in the bare frame results in a natural period shift into the lower acceleration range (see Figure 3). In Figure 3, V indicates total base shear including the damping force, and W indicates total structure weight.

5-Story Model. The results are similar to the 1-story model. Sylmar, Newhall, and Redwood City records which produced overstress in the linear analyses were used for the nonlinear analysis. For the damped frame, the Sylmar record produced a maximum plastic hinge rotation of 0.77 percent, which is lower than the 1 percent limitation. For the bare frame, there was a significant increase in the number and magnitude of plastic hinges with a maximum hinge rotation of 2.0 percent for the Redwood City record. The base shear coefficient is higher for the damped frame (see Figure 4).

11-Story Model. Newhall, Sylmar, and Redwood City records are used in the analyses. The results are similar to the 1-and 5-story models. For the damped frame, the Sylmar record results in a maximum plastic hinge rotation of 0.76 percent. For the bare frame, the maximum hinge rotation is 1.2 percent for the Sylmar record. Base shear is higher for the damped frame for Sylmar record (see Figure 5).

Summary of Nonlinear Analysis

The above analyses indicate that 1) when inelastic behavior is considered, the damped frame can provide immediate occupancy performance for high intensity earthquake event; 2) the performance level of the damped frames exceeds that of the bare frames; and 3) the base shear for the damped frame can be larger than for the bare frame.

The Blue Book Provisions

The 1999 Recommended Lateral Force Requirements and Commentary ("Blue Book") (SEAOC 1999) allows the bare frame to be sized for strength only using the code static level force. Displacement of the frames is controlled by EDS. Time history analyses are required for this procedure. This procedure produces significantly lighter moment frames than the frames designed to conform to both strength and drift criteria using the code formula, since design of moment frames is usually controlled by drift criteria. The 5-story building was redesigned to conform to the SEAOC requirement. The first mode is 1.8 seconds, approximately 30 percent longer than the 1.4 second for the UBC frame described previously. Maximum column slenderness factor K is 2.08, significantly increased from 1.7 for the frame conforming to the UBC frame (see Figure 6). A linear two-dimensional model was constructed using ETABS 6.1. Fluid viscous dampers were modeled as discrete damping elements. Approximately 20 percent of critical damping is provided by the FVD. The results of the linear analyses show that the member stress ratios typically increase from the UBC frame with EDS. UBC Zone 4 and Santa Monica records, for which the UBC frames with EDS remained elastic, cause overstress in the SEAOC frame with EDS. The drift ratio is within 0.025 limit for life safety for all records except Sylmar, which was 8 percent over the limit (see Tables 3 & 5).

Table 5. Linear Time History (T_1 = 1.8 Sec.)

Time History	*Drift Ratio*	*Base Shear Coefficient*	*Column Stress Ratio*	*Beam Stress Ratio*
Sylmar, 360º	0.027	0.44	2.10	3.98
Redwood City	0.015	0.27	1.34	2.35
Newhall, 90º	0.012	0.25	1.09	1.82
UBC Zone 4, S_2	0.011	0.20	0.96	1.67
Santa Monica, 90º	0.010	0.15	0.85	1.50
U.C.L.A., 360º	0.005	0.10	0.56	0.93
Moorpark, 180º	0.004	0.12	0.42	0.67
Downey, 360º	0.003	0.05	0.25	0.36

A two-dimensional model was constructed using Drain 2DX with FVD added as described earlier. Nonlinear analyses were carried out for UBC Zone 4, Newhall, Sylmar, and Redwood City records. For all records, the SEAOC frame EDS shows greater numbers and magnitudes of plastic hinges, larger story drift ratios, and lower base shears than the UBC frames with EDS. The lower base shear was caused by an increased fundamental period, which places the frame in a less critical region of the excitation. At a minimum, the frame satisfies the life safety requirement of FEMA 273 for all ground motions analyzed. Compared to the bare frame described above, the number and magnitudes of plastic hinges, the drift ratios and the base shears are significantly less (See Figure 7). Therefore, the seismic performance is improved.

Conclusion

1) Linear time history analyses show that the damped frame can remain elastic and provide immediate occupancy performance for earthquake events that do not greatly exceed UBC Zone 4 spectra.
2) Non-linear time history analyses show that the performance level of the damped frame exceeds that of the bare frame. Plastic hinging in a bare frame can be significantly higher for ground motions that exceed the code spectra.
3) Base shear in the damped frame can be larger than base shear in the bare frame.
4) The damped frame can provide immediate occupancy performance as defined in FEMA 273 for high intensity earthquakes.
5) The SEAOC frame with EDS can provide higher performance (less quantity and magnitude of plastic hinges) over the bare UBC frame.

The study shows that the use of EDS's in structures for code level earthquake motion can reduce the seismic demand to levels that bring the response of many structures within the linear range. Because linear structural behavior is well understood, it eliminates the uncertainty associated with the estimation of nonlinear response. It is believed that incorporation of an EDS into the design of a structure will be a key to future code development for performance based design.

This paper was written in memory of Dr. Roger Scholl and Dr. Ajit Virdee.

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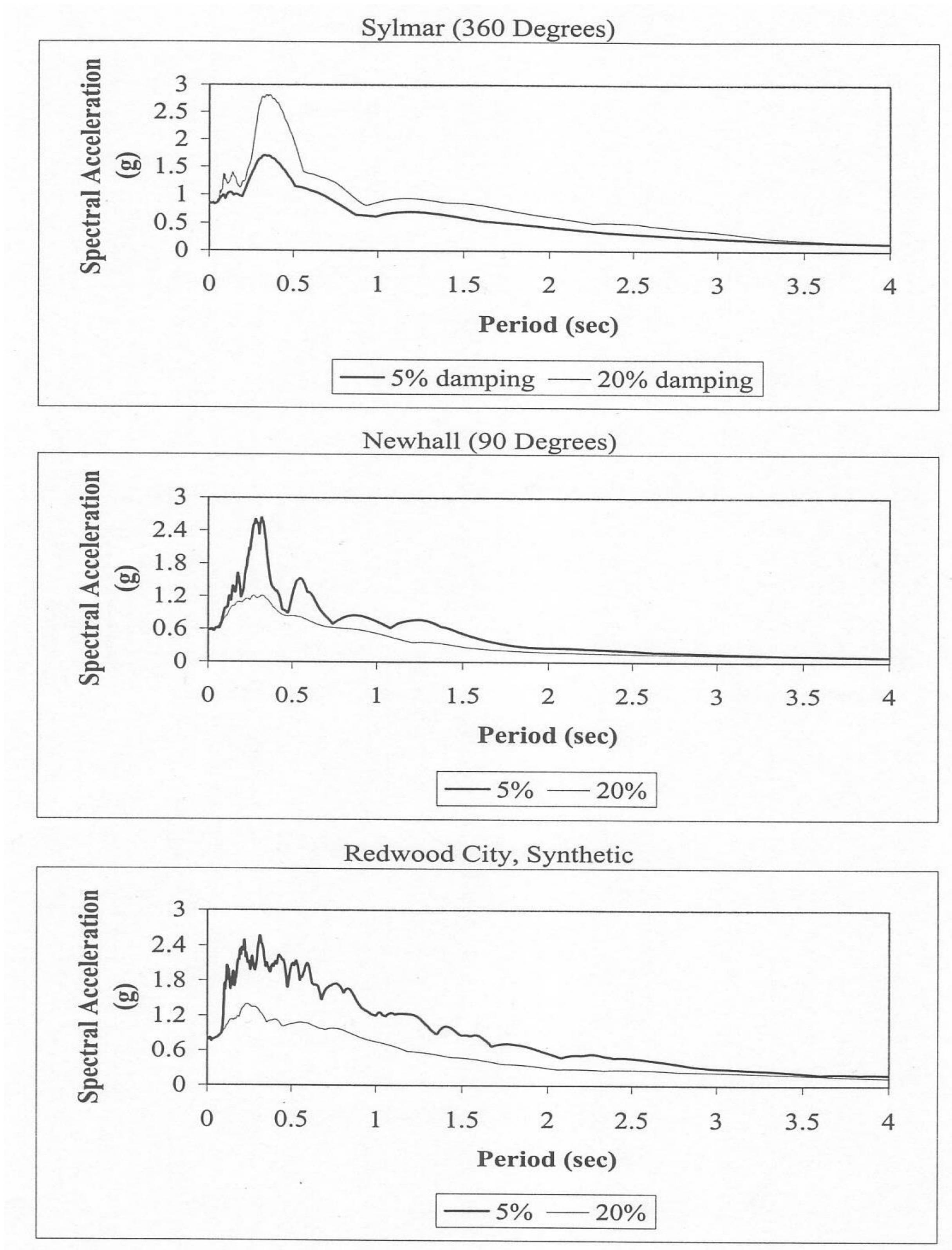

Figure1. Acceleration response spectra for Sylmar, Newhall records and Redwood City synthetic record for 5 and 20 percent damping