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Inelastic Behavior of Structures with Passive Energy Dissipators and Its Effect on SEAOC Provisions

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ABSTRACT

The purposes of this paper are as follows: 1) determine the level of seismic intensity at which frames with an Energy Dissipating System (EDS) remain elastic; 2) determine the performance level of these frames with EDS for high intensity earthquakes; 3) determine the difference in performance between frames with EDS and without EDS; and 4) verify the Structural Engineers Association of California (SEAOC) provisions, which allow the base frame to be designed for strength only, with EDS provided to control drift. Linear and non-linear time history analyses were conducted on three steel moment frame buildings: 1-story; 5-story; and 11-story. The base frame of these buildings were designed to conform to the strength and drift requirements of the UBC. In addition, the base frame of the 5-story building was redesigned per SEAOC provisions. Discrete damping elements were added at each floor. Linear and non-linear computer models with and without EDS were subjected to various recorded and synthetic time histories of ground motion.

Introduction

The addition of an Energy Dissipation System (EDS) in structures for reducing seismic demands provides a new and innovative option to improve seismic performance. Yet, the concept of an EDS is not new. As early as 1968 the United States issued a patent for structures outfitted with Fluid Viscous Dampers (FVDs) (Cardan, 1968). This study evaluates the performance of steel moment frame structures, some with an EDS and others without, as they are subjected to various recorded and synthesized seismic time histories. The specific type of the EDS unit chosen was the Linear Fluid Viscous Damper. Not only is it one of the simplest form of EDS, but its dynamic stiffness is out of phase and the effects on the base frame are easy to capture.

Input Time Histories

Some of the highest accelerations ever recorded were obtained in the M 6.7 Northridge earthquake on January 17, 1994. Natural time histories used for this study were recorded by the California Strong Motion Instrumentation Program (CSMIP). Six records were chosen from 193 stations based on high magnitude of velocity and acceleration at typical alluvium sites (table 1).

Table 1
CSMIP Ground-Response Records

Station Name	SiteGeology	PeakAcceleration [g.]	PeakVelocity [in./sec.]	PeakDisplacement [in.]
Downey, 360^0	Deep Alluvium	-0.223	5.0	0.75
Moorpark, 180^0	Alluvium	0.297	8.0	1.73
Santa Monica, 90^0	Alluvium	-0.901	16.5	5.63
U.C.L.A., 360^0	Alluvium	0.634	8.6	2.87
Newhall, 90^0	Alluvium	-0.610	29.4	6.93
Sylmar, 360^0	Alluvium	0.892	-50.7	-12.80

Two synthesized time histories were used in this study. One, is compatible with the UBC Zone 4 S2 response spectrum. The second is a 500 year return event for Redwood City, California. The site soil is equivalent to the UBC S2 soil, and the site is located approximately 4 miles from the San Andreas Fault. The time histories were synthesized by Singh (1996) (see figure 1). Figure 2 shows the 5% damped response spectrum envelope for all time histories used for this study.

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Description of Structures

Three different steel moment resisting buildings are used for analysis: 1-story; 5-story; and 11-story. The typical floor height is 14'-0" with the exception of 16'-4" at the 1st. Footprint is 105 ft x 130 ft. Floor diaphragm is composed of cast-in-place concrete over metal deck. Floor dead load varies from 80 psf to 100 psf including movable partition load. Floor live load is 50 psf, typical of office usage. Lateral load resisting system consists of perimeter steel moment frames.

There are a total of six bays of moment frame in each principal direction. Each building is designed to conform to the special moment frame requirement of the 1994 UBC for seismic zone 4 and site soil S2. Drift criteria is a governing factor for designing member sizes. The 1st mode is the predominant mode shape for all structures. The fundamental periods of the 1-story, 5-story, and 11-story buildings are 0.5 seconds, 1.4 seconds, and 2.5 seconds respectively. These natural periods cover the highest spectral acceleration range.

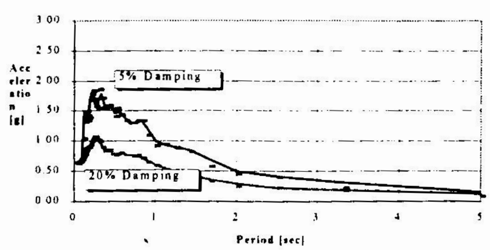

Figure 1. Redwood City Synthetic Response Spectrum

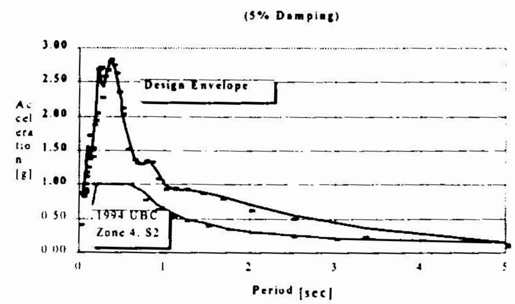

Figure 2. Design Envelope Response Spectrum

Linear Time History Analysis

1-Story Model Linear 2-dimensional models are constructed using ETABS 6.1 (CSI, 1996). Fluid Viscous Dampers are modeled as discrete damping elements and mounted on chevron braces. Approximately 20% of equivalent critical damping is provided by FVDs. Structures are assumed to have 2% inherent modal damping. The result of the linear time history analyses show that in all cases except for the Newhall and Sylmar excitations, the frame remained elastic. The stress ratio is calculated as ratio of Force Demand over Force Capacity. Force Demand is defined as: 1.2DL + 0.5 LL + 1.0 EQ. Force Capacity is defined as LRDF strength using appropriate phi factor. Maximum interstory drift for all except the Newhall, Sylmar, and Redwood City records is within the 0.007 drift ratio limit for immediate occupancy performance, as defined in FEMA 273 (1998). The results for the Newhall, Sylmar and Redwood City records are well within the 0.025 drift ratio for life safety performance level (see table 2).

Table 2

Linear Time History (T_1 = 0.5 sec.), 1-Story Model

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

5-Story Model The linear time history analyses show that in all cases except for the Newhall, Sylmar, and Redwood City stations, the frame remains elastic. Maximum drift ratio in all cases except the UBC Zone 4, Newhall, Sylmar, and Redwood City records, is within the 0.007 drift ratio limit for immediate occupancy performance. For the Newhall, Sylmar, and Redwood City records, the frame is within the 0.025 drift ratio for life safety performance (see table 3)

Table 3
Linear Time History (T_1 = 1.4 sec.),5-Story Model

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

11-Story Model The results are similar to the 1-story and 5-story models. The linear time history analyses show that except for the Newhall, Sylmar, and Redwood City cases, the frame remains elastic. Maximum drift for all except Santa Monica, Newhall, Sylmar, and Redwood City station records is within the 0.007 drift criterion. For the Sylmar and Redwood City stations the frame is within 0.025 drift ratio limit (see table 4).

Table 4
Linear Time History (T_1 = 2.5 sec.),11-Story Model

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

Non-linear Time History Analysis

Two-dimensional models are constructed using DRAIN 2 DX (Prakash, Powell, & Campbell, 1993). Steel beams and columns are modeled with plastic hinge beam-column elements. Bilinear behavior is assumed with 5% plastic hardening. The Fluid Viscous Dampers are modeled as discrete damping elements which are mounted on chevron braces. Two computer models are created; one is a frame with FVDs ('damped' frame) and the other is a frame without FVDs ('bare' frame). Five percent inherent modal damping is assumed for the bare frame.

1-Story Model Records of the Newhall and Sylmar stations are used for analysis, since overstresses occurred in the linear analysis in both of these cases. For the damped frame, the maximum plastic hinge rotation is 0.9% (0.009 radian) for the Sylmar record, which is lower than the 1% hinge rotation limitation for immediate occupancy performance as defined. For the bare frame, the numbers and magnitudes of plastic hinges significantly increase. Maximum hinge rotation is 2.0% with the Sylmar record. It should be noted that the base shear coefficient is higher for the damped frame, since the damping force is higher, and the natural period of the bare frame is shifted higher due to plastic hinge formation, thus moving the frame into a lower acceleration range (see figure 1).

5-Story Model The result is similar to the 1-story model. The records of the Newhall, Sylmar, and Redwood City stations are used for analyses, since overstresses occurred in the linear analyses. For the damped frame, the maximum plastic hinge rotation is 0.77% due to the Sylmar record, which is lower than the 1% limitation. For the bare frame, the number and magnitudes of plastic hinges significantly increase. Maximum hinge rotation is 2.0% due to the Redwood City record. Again, the base shear coefficient is higher for the damped frame (see figure 2).

11-Story Model The result is similar to the 1 and 5-story models. Newhall, Sylmar, and Redwood City stations are used for analysis. For the damped frame, maximum plastic hinges rotation is 0.76% at Sylmar station. For the bare frame, quantity and magnitude of plastic hinge significantly increase. Maximum hinge rotation is 1.2% at Sylmar station. Base shear is again higher for the damped frame at Sylmar station (see figure 3).

The SEAOC Provisions

The proposed blue book provisions for Energy Dissipation Devices (SEAOC 1998) allow the base frame to be designed for strength only, with EDS provided to control drift. This design procedure produces lighter moment frames than the one described in above. The 5-Story building is redesigned to conform to the special moment frame for SEAOC requirement. Seismic Zone is 4, site soil is S2. The first mode natural period is 1.8 seconds, approximately 30% longer than the 1.4 second 1st mode period for the UBC frame. Maximum column slenderness factor K is 2.08, significantly increased from 1.7 for the frame conforming to the UBC frame. A linear two-dimensional model is constructed using ETABS 6.1. Fluid Viscous Dampers are modeled as discrete damping elements. Approximately 20% of critical damping is provided by FVDs. The results of the linear analyses show that the lateral frame member stress ratios typically increase from the UBC frame. UBC Zone 4 and Santa Monica records, for which the UBC frames remained elastic, cause overstress in the SEAOC frame. Drift ratio is within 0.025 limit for life safety for all records (see table 5).

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

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

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

Cost Study

The reduction in steel weight for the SEAOC frame compared with the UBC frame is significant; approximately 100,000 pounds, which equates to a construction cost of $100,000 U.S. Dollars. Assuming 40 damper devices at an average cost of $6000 per unit, the EDS total cost is $240,000. Assuming $100/ft^2 total construction cost for this type of building, the net cost increase in the 'damped' UBC frame is approximately 3 % . This figure agrees with findings by Jokerst & Soyer (1996) and Miyamoto & Scholl (1996). The net cost increase in the SEAOC frame with EDS is a mere 2.1%, which is negligible (see table 6).

Table 6
Cost Study

	SEAOC Frame w/ EDS	U.B.C. Frame w/ EDS	Difference
Steel Frame Weight	320,136 lbs.	421,056 lbs.	100,920 lbs.
Steel Frame Cost	$320,136	$421,056	$100,920
Damper Cost	$240,000	$240,000	$0
Lateral System Cost	$560,136	$661,056	$100,920

Lateral System Cost

	Cost	% Total Cost	Difference
Bare UBC Frame	$421,056	6.1%	-
SEAOC Frame w/ EDS	$560,136	8.2%	2.1%
UBC Frame w/ EDS	$661,056	9.7%	3.6%

Summary and Conclusion

1) Linear time history analyses indicated that the damped frame can remain elastic and provide immediate occupancy performance for earthquake events which do not greatly exceed UBC, Zone 4 S2 spectrum. 2) Nonlinear time history analyses indicated that performance level of the damped frame far exceeded that of the bare frame. For the bare frame, the number and magnitudes of plastic hinging significantly increase. 3) Base shear in the damped frame can be larger than for the bare frame. 4)The damped frame can provide immediate occupancy performance as defined in FEMA 273 for high intensity earthquakes. The following is a summary of the finding for the 5-story model, which was redesigned to conform SEAOC provisions. 1) The fundamental period of the structure was increased by approximately 30%. 2) The SEAOC frame with EDS can provide at least life safety performance as defined in FEMA 273. 3) The performance of the SEAOC frame with EDS was improved over the bare UBC frame. 4) The cost increment for EDS was offset by the cost saving in the steel weight. This study shows that a structure may exhibit only a linear seismic response if EDSs are used. Linear structural behavior is well understood, and the uncertainty associated with non-linear response is eliminated. For high intensity earthquake events, the structures with EDS will behave as inelastic frames. However, the nonlinear behavior is significantly reduced by EDS. Therefore performance beyond minimum life safety is possible using EDS, without significant construction cost increase

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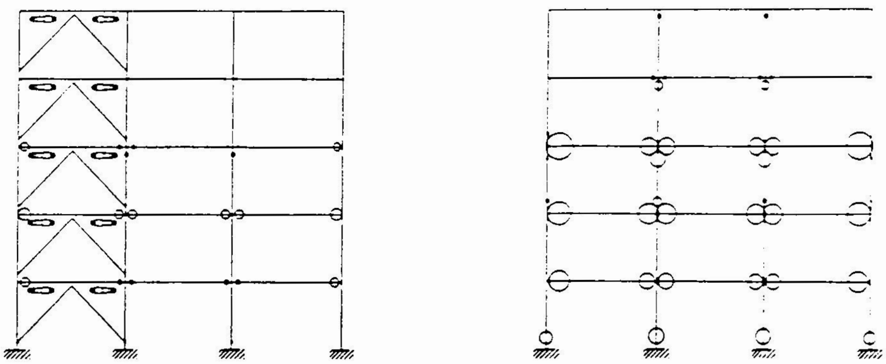
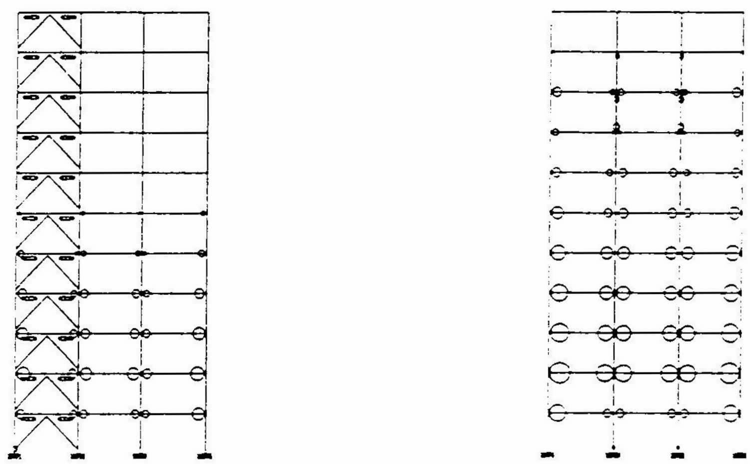
Figure 1 Sylmar Station, (T_1=0.5 SEC)



V MAX. = 0.57w
MAX. DRIFT = 0.017
MAX. PLASTIC ROTATION = 0.77%

V MAX. = 0.38w
MAX. DRIFT = 0.026
MAX. PLASTIC ROTATION = 1.8%

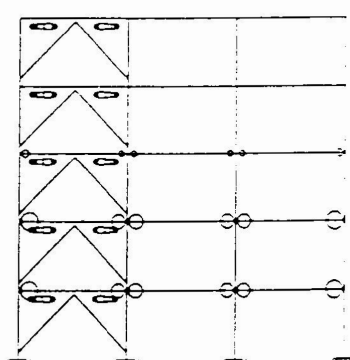
Figure 2. Sylmar Station, (T_1=1.4 SEC)



V MAX. = 0.28w
MAX. DRIFT = 0.016
MAX PLASTIC ROTATION=0.76%

V MAX. = 0.20w
MAX. DRIFT = 0.019
MAX PLASTIC ROTATION=1.2%

Figure 3. Sylmar Station, (T_1=2.5 SEC)



V MAX. = 0.38w
MAX. DRIFT = 0.021
MAX. PLASTIC ROTATION = 1.26%

Figure 4. Sylmar Station, (T_1=1.8 SEC)