STATE-OF-THE-ART APPLICATIONS OF PASSIVE ENERGY DISSIPATORS

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SUMMARY

Passive Energy Dissipators (PED) have been used in over forty structures in the US in recent years to improve their earthquake response performance. This paper gives a description of two applications that were designed and constructed during the period of 1994 through 1998, and new design guidelines developed by Structural Engineers Association of California (SEAOC).

INTRODUCTION

DESCRIPTION OF HOTEL WOODLAND

Hotel Woodland is a 4-story reinforced concrete building constructed during the latter part of 1927. The building is a National Historic Registered building. The ground level footprint is approximately 168 feet by 95 feet, the upper three levels have a footprint of 168 feet by 50 feet, and the total square footage is approximately 50,000 ft^2. The total height of the structure is about 53 feet (see figure 1).

The 2nd, 3rd, and 4th floors are cast-in-place concrete joist-beam construction. Typical columns are 16" square reinforced concrete. Typical exterior frame consist of 48" deep by 10 3/4" thick concrete spandrel beams and 48" wide by 6 3/4" thick concrete piers. The concrete wall pier-spandrel beam construction is terminated at the 2nd floor. In addition, 6 3/4" thick concrete bearing-shear walls exist at the East and West end of the structure at the ground floor. No lateral resisting elements are found at the North and South elevation of the building at the ground floor, except for 16" square lightly reinforced concrete columns. This type of structure in the East-West direction is often defined as a non-ductile soft/weak story structure.

SEISMIC RETROFIT CRITERIA

This was an owner-option seismic retrofit, therefore maintaining the historical appearance of the building, and cost were the primary considerations in establishing the retrofit design. The design team and owner decided that the seismic retrofit objective should be limited to preventing the collapse of the four-story super structure, since it would present a major threat to life safety. The Design Basis Earthquake (DBE) is a 20% probability of occurrence in a 50 year duration. This event is consistent with the California Seismic Safety Commission Recommendations for the " Acceptable Seismic Risk for State Buildings" report. The Maximum Capable Earthquake (MCE) selected for the retrofit is a 10% probability of occurrence in a 100 year duration. Three pairs of Time Histories for each DBE and MCE event were constructed to analyze the structure.

EXPECTED PERFORMANCE OF THE EXISTING BUILDING

The fundamental period of the building in the East-West direction was approximately .7 second. Maximum displacement and story shear are shown in table 1. The concrete columns at the ground floor level were overstressed in bending and shear due to excessive deflection and the lack of ductility detailing and strength.

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Most of the non-linear behavior of this building was concentrated at the ground floor level columns. This type of adverse behavior could cause total collapse of the superstructure.

TABLE 1

Story	Story Displacement (inch)	Story Shear (kips)
Roof	0.01	290
4th	0.02	890
3rd	0.01	1484
2nd	2.00	2200 (.43G)

SEISMIC RETROFIT SCHEMES

Since this building is a National Historical Registered building, there were unique challenges that the design team had to meet, including 1) keeping the historical appearance of the landmark hotel, and 2) avoiding disturbance to tenants living in apartments on the 2nd floor and above.

The steel moment frames were designed to provide stiffness, strength, and redundancy, which the existing lightly-reinforced concrete columns lacked. Fluid Viscous Dampers (VDs) were provided to control drift at the 1st floor and to keep steel moment frames in the elastic range. VDs were attached to the top of the steel Chevron Braces (see figure 2) and VDs-Chevron Braces were strategically located to meet the above requirements.

Time history analyses were performed using the computer program ETABS 6.04 (CSI, 1994) which utilized the Step-by-Step Linear Acceleration Method. The link elements represented the VDs. A total of 20 mode shapes were extracted. Twelve mode shapes belonged to the structural stiffness and mass matrix, and 8 mode shapes belonged to the link elements. Each floor had X, Y translation and Z rotation, which constitute a rigid diaphragm. The top of the Chevron Braces were disconnected from the 2nd floor rigid diaphragm, and the link elements connected the top of the Chevron Braces to the 2nd floor to emulate VDs.

EXPECTED PERFORMANCE OF THE RETROFITTED STRUCTURE

The fundamental period of the retrofitted building in the East-West Direction was .46 second. A total of 16 steel moment frames with. A total of 8-VD assemblies with 16-50 kip output dampers were provided. The damping constant for each VD was 9.4 kip-second/inch. The exponential constant was set as a unity, which produced linear viscous behavior. The maximum design axial force of the VDs was 100 kip with safety factor 2.0. The maximum displacement, velocity, and story shear for DBE are shown on Table 2 for 5% of critical damping without VDs and on Table 3 the ground level with VDs.

TABLE 2
STEEL MOMENT FRAME WITHOUT VDs (DBE), 5% DAMPING

Story	Story Displacement (inch)	Story Shear (kips)	Story Velocity (inch/sec)
Roof	0.01	410	0.09
4th	0.03	1256	0.25
3rd	0.03	2090	0.41
2nd	1.31	3087 (.60G)	16.80

TABLE 3
STEEL MOMENT FRAMES WITH VDs (DBE): FINAL DESIGN

Story	Story Displacement (inch)	Story Shear (kips)	Story Velocity (inch/sec)
Roof	0.01	183	0.05
4th	0.01	558	0.13
3rd	0.01	927	0.22
2nd	0.41	1374 (.26G)	5.50

The above tables show that by providing VDs , both base shear and 2nd floor displacement were reduced by approximately 60%. Plastic deformation of both existing concrete and new steel moment frames were precluded, and the majority of the seismic energy was absorbed by VDs .

CONCLUSION

Total construction cost of the seismic strengthening was approximately $500,000 US which equates to $10.0 per square foot. The above figure satisfied the construction cost requirement of the project. Construction was completed in 1995.

DESCRIPTION OF THE MONEY STORE

The National Headquarters for this financial institution, located in West Sacramento, California, became one of the first new buildings in the United States to employ seismic dampers to control a building's response during a seismic event. This 11-story pyramid-shaped steel moment frame building occupies approximately 450,000 total square feet, including a partial basement and exterior deck areas. The ground floor footprint is 300 by 300 feet. There is a 10 foot set-back at each story. The typical story height is 14 feet. The total height of the structure is 156 feet (see Figure 3). The typical floor system consists of structural steel beams with cast-in-place concrete over metal deck. The structure's lateral system consists of elastic steel moment frames with fluid viscous dampers (FVDs).

PERFORMANCE-BASED DESIGN CRITERIA

Earthquake performance, cost effectiveness, and architectural requirements were the primary considerations in designing this building. The owner's requirements were as follows: 1) disruption of business operations should be minimal after a major seismic event, and 2) the construction cost should not exceed that of a minimum code-conforming building. The design team and the owner agreed upon the following criteria to satisfy the first requirement: 1. All structural members and connections shall remain below yield levels for the Design Basis Earthquake (DBE). 2. The maximum inter-story drift ratio shall be less than 0.005 at the DBE to protect non-structural elements. 3. An unreduced Time History Analysis shall be utilized to study the actual behavior of the structure during the DBE. The Design Basis Earthquake is defined as a seismic event with a 10% probability of occurrence in a 50 year duration

Steel moment frames with Fluid Viscous Dampers (FVDs) were selected for the following reasons (see Figure 4): 1. The additional cost for FVDs was less than 1% of the total construction cost. 2. The natural period of the structure was kept out of the high acceleration spectra range, yet displacement was controlled by FVDs. 3. Plastic hinge formation within structural elements and connections were prevented.

Fluid Viscous Dampers were selected over other damping devices for several reasons. Since FVDs are velocity-dependent systems, the forces are out of phase with the axial loading of the columns, and the change in the natural period of the building is insignificant. Additionally, the long history of military application proved the system's reliability. Fluid Viscous Dampers operate on the principle of fluid flowing through orifices. A stainless steel piston travels through chambers filled with silicone oil, which flows through an orifice in/around the piston head. During a seismic event, the seismic energy is transformed into heat, which dissipates into the atmosphere. The orifice construction utilized in FVDs is similar to that of classified applications for the U.S. Armed Forces, and is considered state-of-the-art (Constantinou and Symans, 1992).

DESIGN PROCEDURES

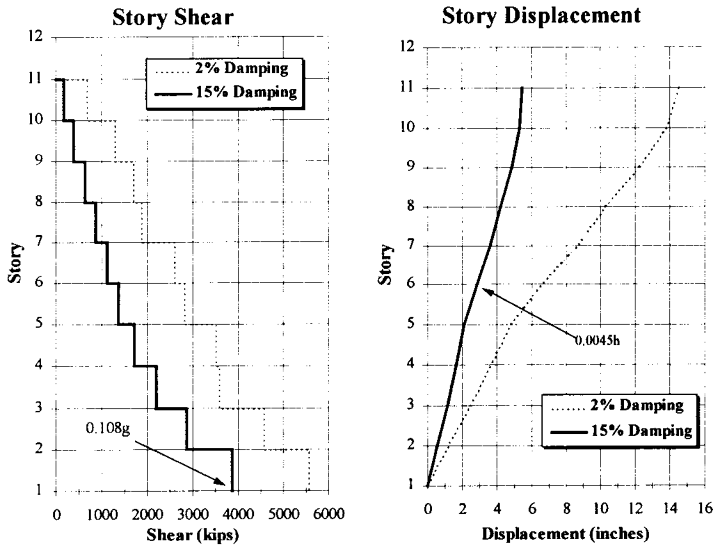
Step 1: In order to expedite the plan approval process and determine a design starting point, steel moment frames were designed to conform to ordinary steel moment frames, as specified by the 1994 UBC, discounting the effect of FVDs. Step 2: Using member sizes determined in step 1, unreduced time history analyses were performed to study the effects of FVDs. FVD design criteria is as follows: 1. Maximum allowable drift ratio = 0.005. 2. All members, connections, and foundations are to remain elastic using the L.R.F.D. strength factors. Time History analyses were performed using ETABS 6.04 (CSI, 1994). All FVDs were modeled as discrete link elements in order to study the interaction between the moment frames and the FVDs.

RESPONSE PERFORMANCE OF THE STRUCTURE

The maximum story displacement and story shear for the DBE/MCE are shown graphically below for 2% critical damping and 15% critical damping by FVDs.

Table 4 shows that by providing FVDs, base shear was reduced by 30%, and displacement was reduced by 60%. Maximum inner story drift was limited to 0.0045, and all structural members remained well below yield level for DBE/MCE event.

Table 4



COST ANALYSIS

Total structural construction cost, including structural steel, FVDs, metal deck, concrete, and foundations was approximately $10.3 million (1996, present) which equals $23.00 per square foot. The above figure satisfied the construction cost requirement, which was not to exceed that of a minimum code-conforming building. Construction started in May, 1996. FVDs eliminated the plastic hinge formations in the structure, thereby reducing the uncertainty in the structure's behavior.

THE STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA (SEAOC) PROVISIONS

The SEAOC provisions for Energy Dissipation Devices (SEAOC 1999) 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 code conforming frames. The 5-Story building is designed 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. Fluid Viscous Dampers are modeled as discrete damping elements. Approximately 20% of critical damping is provided by FVDs to the SEAOC frame.

A two-dimensional model is constructed using Drain 2DX. Two synthesized and two natural time histories used in this study. One synthetic history, is compatible with the UBC Zone 4 S2 response spectrum. The second is a 500 year return event for Redwood City, California. Natural Time Histories used for this study were obtained at Sylmar & Newhall stations in the 1994 Northridge earthquake. Nonlinear time history analyses are carried out. For all records, the SEAOC frame shows less number and magnitudes of plastic hinges, less 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. As a minimum, the frame satisfies life safety requirement of FEMA 273 for all ground motions analyzed (see figures 5, 6, & 7).

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. Assuming 40 damper devices at an average cost of $3000 per unit, the EDS total cost is $120,000. 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 0.5%, which is negligible.

CONCLUSION

The following is a summary of the finding for the 5-story model, which conformed to SEAOC provisions. 1) The fundamental period of the structure was increased by approximately 30% in comparison with the UBC Frames. 2) The SEAOC frame with EDS can provide the 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.

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Figure 1

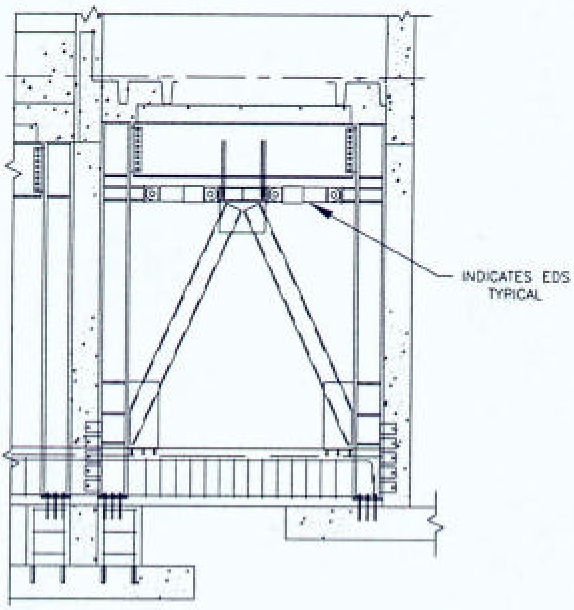


Figure 2

Figure 3

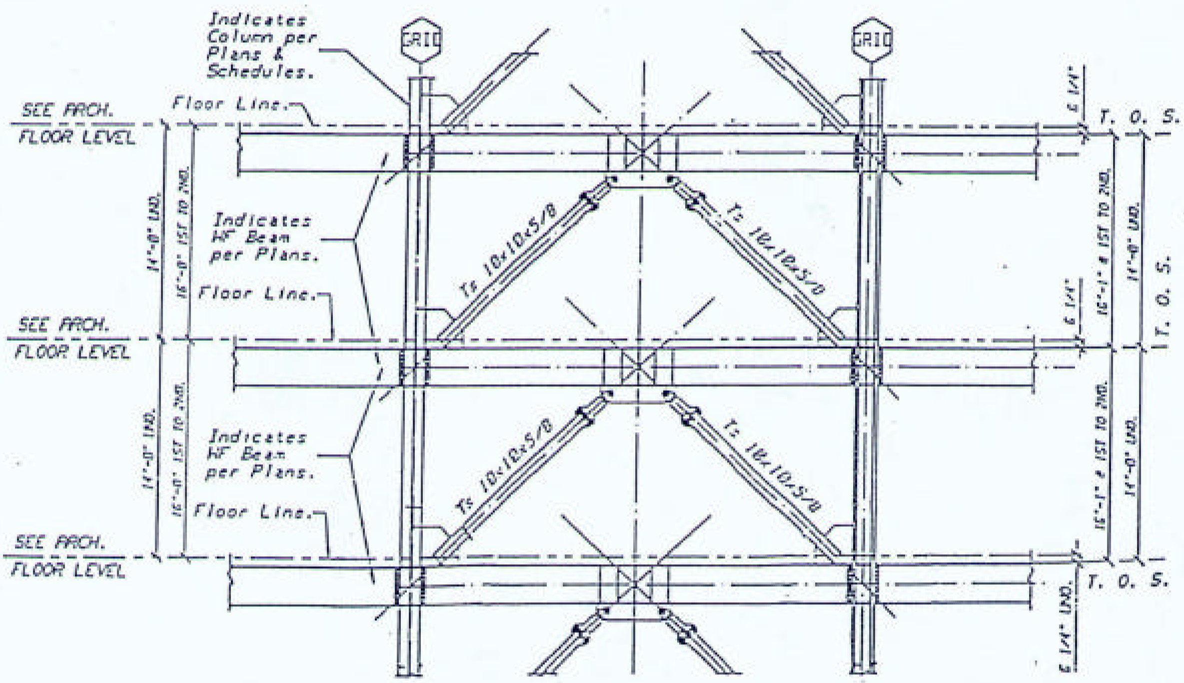
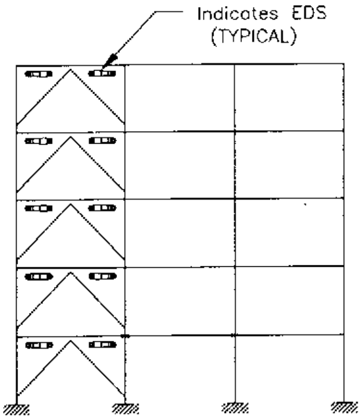
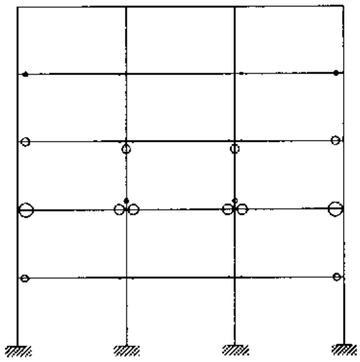


Figure 4



V MAX. = 0.26w
MAX. DRIFT = 0.011
MAX. PLASTIC ROTATION = 0.3%

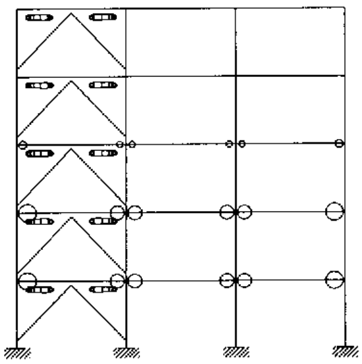
Newhall Station, (T_1=1.8 SEC)



V MAX. = 0.37w
MAX. DRIFT = 0.019
MAX. PLASTIC ROTATION = 0.95%

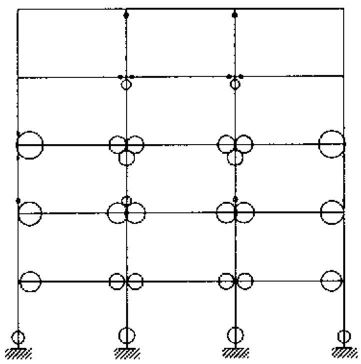
Newhall Station, (T_1=1.4 SEC)

Figure 5



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

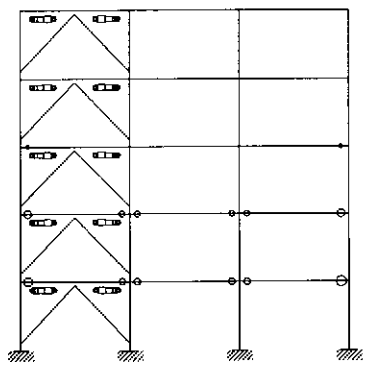
Sylmar Station, (T_1=1.8 SEC)



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

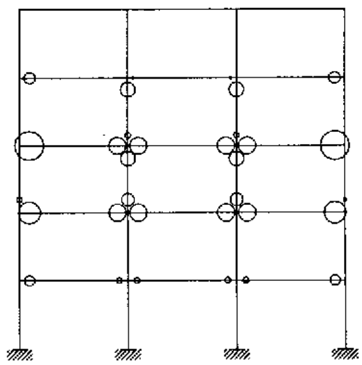
Sylmar Station, (T_1=1.4 SEC)

Figure 6



V MAX. = 0.27w
MAX. DRIFT = 0.016
MAX. PLASTIC ROTATION = 0.67%

Redwood City Synthesized, (T_1=1.8 SEC)



V MAX. = 0.38w
MAX. DRIFT = 0.026
MAX. PLASTIC ROTATION = 2.0%

Redwood City Synthesized, (T_1=1.4 SEC)

Figure 7