
U.S. DESIGN OF STRUCTURES WITH DAMPING SYSTEMS

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ABSTRACT:

Many applications of damper devices in both new and existing buildings in both United States and Japan have resulted from extensive damper device development efforts. The increased usage of this technology has created a demand for design guidance and building codes to specify their use in the United States. This paper provides a case study using code type design procedures.

A two-story police head quarters in Vacaville, California with an area of 3,716m² (40,000ft²) is summarized. This new building was designed with damping system following the 2000 NEHRP procedures as described in a companion paper.

1. Introduction

This paper presents an earthquake design procedure and a case study of the Vacaville Police Headquarters. The earthquake design “goal” of this essential facility is to provide an immediate occupancy performance for a 475-year return seismic event. However, the project requirement is to keep the construction cost within typical code conformed buildings. The combination of Special Moment Resisting Frames (SMRF) and Fluid Viscous Dampers (FVDs) are used as the lateral force resistance system. This system as described by Gimmel, Lindorfer, and Miyamoto, (2002) results in cost efficiency and superior seismic performance. The 2000 NEHRP (FEMA, 2000) guideline was used to design the project, since it is considered to be a state-of-the-art procedure for seismic damping devices. This project becomes the first structure in the United States to use this advanced procedure.

2. Building Description

The project is located in Vacaville, California, which is within a region of many active faults and high seismic activity. The structure is a 2-story, 40,000ft² (3,716m²) steel framed structure. The roof is composed of metal deck and WF beams, and the floor is composed of 2¹/₂-inch (6.4cm) lightweight concrete over 3-inch (7.6cm) metal deck and steel composite beams. The exterior finish is lightweight architectural finish over nonstructural metal stud walls. See figure 1 for architectural rendering. See figure 2 for the second floor structural plan. A perimeter SMRF is provided along the longitudinal direction. For the transverse direction, one-bay SMRF is provided at each column line. The location and quantity of SMRF is the same for the roof plan. A 24-inch (61.0cm) deep pad foundation is provided at WF columns.

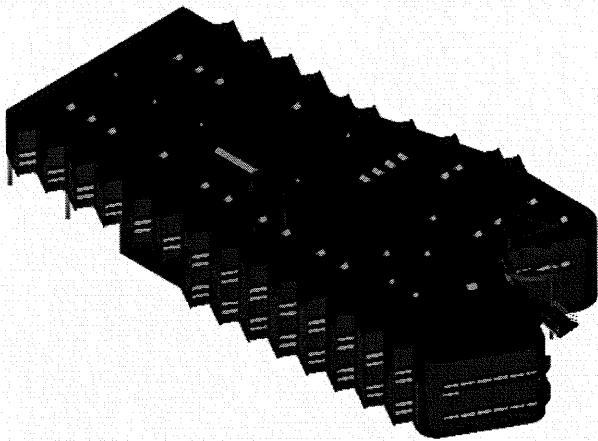


Figure 1: Architectural Rendering
(Courtesy of Indigo Architecture)

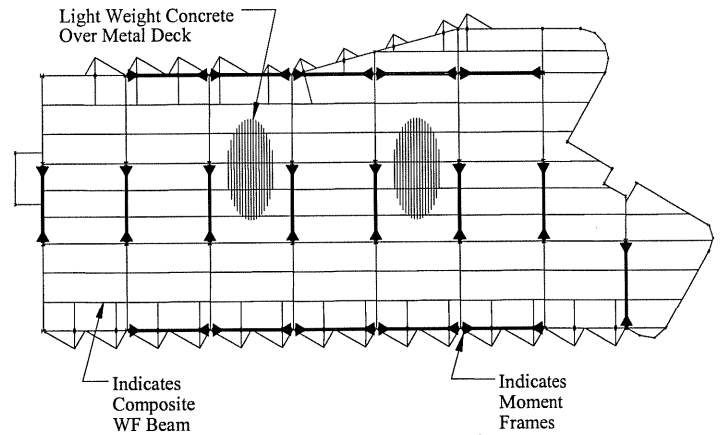


Figure 2: Second Floor Structural Plan

3. Seismic Risk

The site soil consists of 65 feet (19.8m) of alluvium overlaying siltstone. The site is considered to be Sd soil per 1997 UBC. The Vaca fault, which is not considered to be active, is closest to the site at a distance of 0.2 km. The next closest faults are segments 4 and 5 of the Great Valley Seismic Source Zone located at distance of 6.6 and 9.8 km, respectively. These faults are considered to be blind thrust faults. The closest fault considered capable of surface rupture is the Green Valley-Concord fault located at 18 km from the site. The significant nearby earthquake was 1892 Vacaville/Winters (M 6.5), which was attributed to the Great Valley Fault. The 1889 Antioch (M 6.3) earthquake is attributed to Greenville fault (Singh, 2002). The 1997 Uniform Building Code (ICBO, 1997) ignores the near fault effects from blind thrust faults such as the Great Valley Source, therefore, the site specific response spectra were created for this project (Singh, 2002). See figure 3 for a 475-year return.

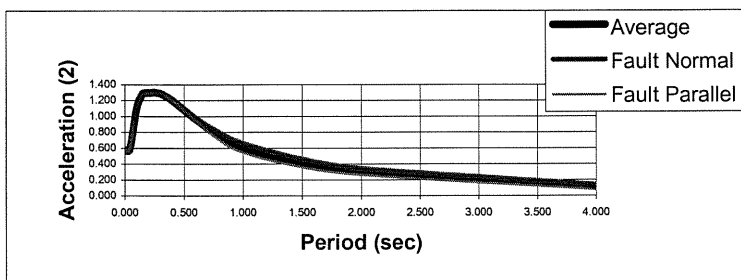


Figure 3: 475-year Return Response Spectra (Singh, 2002)

4. Conventional Structural Design

The structure was first designed as a conventional SMRF to provide a benchmark for cost and seismic performance comparison with a high-tech system. The 2000 NEHRP was used to design SMRF. The following are design parameters for the Equivalent Lateral Force Procedure.

Seismic Use Group III I = 1.5
 $S_{MS} = 1.95g$ at 0.3sec. (site specific)

$$S_{ML} = 1.05g \text{ at } 1.0\text{sec. (site specific)}$$

$$S_{DS} = 2/3 \times 1.95g = 1.3g$$

$$S_{DL} = 2/3 \times 1.05g = 0.7g$$

Seismic Design Category D

SMRF: $R = 8, C_d = 5.5$

$$C_s = \frac{S_{DS}}{R} \quad I = 0.24g$$

$$C_s = \frac{S_{DL}}{R T_a} \quad I = 0.32g$$

$$T_a = 0.4$$

0.24g should be used for seismic shear.

The above value is compared with 1997 UBC.

$$C_a = 0.44 \times N_a = 0.44$$

$$C_v = 0.64 \times N_v = 0.64$$

Near field factors are 1.0, since blind thrust faults are ignored by 1997 UBC.

$$R = 8.5, I = 1.25$$

$$V = \frac{2.5 C_a I}{R} = 0.16g$$

$$V = \frac{C_v I}{R T} = 0.24g$$

0.16g should be used for seismic shear. This value is lower than 2000 NEHRP value. It is affected by the magnitude of R, I, and near field factors. The 2000 NEHRP allows 75% of seismic shear to be used for the damped frame if the total effective damping is 14% or greater. Therefore, $0.75 \times 0.24g = 0.18g$. SMRF is designed for both strength and drift criteria using 0.18g base shear. The 0.18g value is larger than the 0.16g value required by the 1997 UBC; therefore, it will be used to design this frame to compare with the damped frame described later. The drift criteria is the controlling criteria of the design rather than the strength criteria. Allowable story drift ratio is 0.015 and computed maximum drift is multiplied by C_d .

Figure 4 shows the longitude frame elevation. For the transverse direction, SMRFs and dampers are provided to approximate equivalent stiffness and strength as the longitudinal direction. Therefore, for the following discussion, only the longitudinal frame is considered. Tributary weight of the roof is 380kip (1,690 KN) and of the floor is 924kip (4,110 KN). Table 1 shows the results of the modal analysis.

	Period (sec)	Mass Participation %
Mode 1	0.69	84
Mode 2	0.27	16

Table 1: Results of Modal Analysis (Conventional Design)

Nonlinear static pushover was conducted to gauge an earthquake performance of this frame. Figure 5 shows capacity/demand spectra with a site-specific 475-year return event. Please note that figure 5 is for a single degree of freedom system.

The following are results of the pushover for a 475-year return event. The results are converted to the multi degree of freedom system.

$$\text{Maximum roof displacement} = 5.6 \text{ inch (14.2cm)}$$

$$\text{Base shear} = 0.80g$$

Effective period = 0.71 sec.
 Effective damping = 8.4%
 Max drift ratio = 0.016

Some yielding events were observed at the bottom of the first floor columns and second floor beams. The drift ratio is reasonable, but the base shear of 0.8g may cause nonstructural damage to the second floor equipment and roof HVAC units. This is the limitation of the conventional design. This fairly strong SMRF provides near elastic response. However this system also produces high roof and floor accelerations. For this ground motion, the high frequency system such as shear walls and steel brace systems would produce an even higher acceleration and increase seismic demands on nonstructural components. The base isolation may be an ideal solution for this case, yet, the cost increase was not allowed by the project requirement.

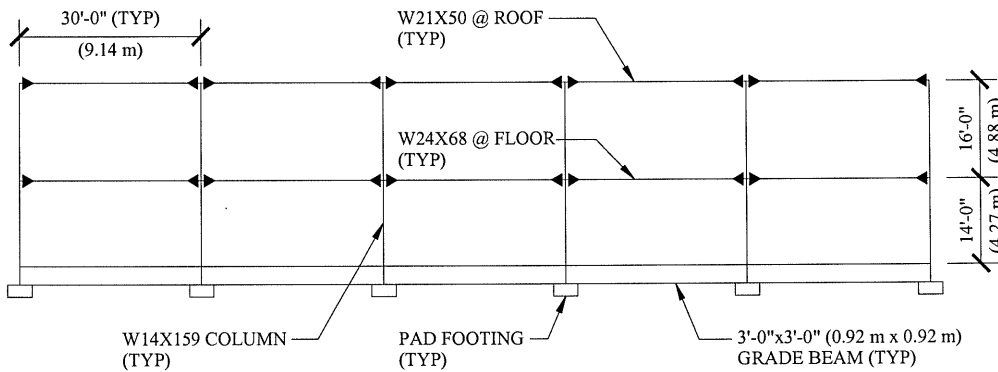


Figure 4: Longitudinal Frame Elevation

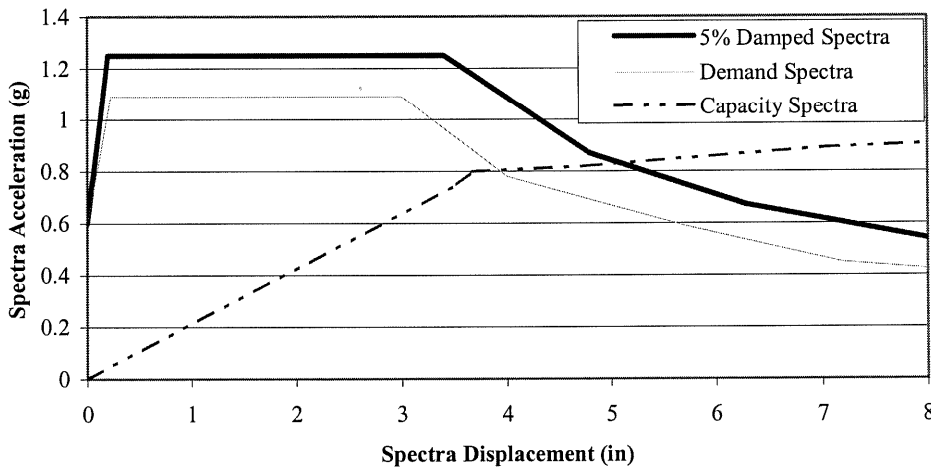


Figure 5: Capacity/Demand Spectra for Conventional Frame

5. Hi-tech Systems Design

The structure was then redesigned using SMRF with FVDs per 2000 NEHRP. The base shear of 0.18g as described above was used to resize the frame members. The 2000 NEHRP describes that the frame members are sized with strength requirements of the code level (0.18g), and FVDs are provided to control displacement of the structure. See figure 6 for the new frame elevation. The difference from figure 4 is the “pinned” foundation condition and roof beam sizes. See table 2 for FVD properties.

	Damping Constant 'C' Per a FVD Unit
1 st Floor FVDs	60 kip-sec ² /in (105kN-sec ² /cm)
2 nd Floor FVDs	30 kip-sec ² /in (52.5kN-sec ² /cm)

Table 2: FVD Property

The damping force is defined as

$$F = CV^\alpha$$

V = Velocity

$$\alpha = 0.6$$

These damping properties were selected based on an optimal displacement reduction and FVD force output. Table 3 shows the results of the modal analysis. The results show that the predominant period shifted from 0.69sec of the conventional SMRF to 1.2sec. This frequency shift effectively brings the dynamic response to a lower acceleration range in the site-specific response spectra.

	Period (sec)	Mass Participation (%)
Mode 1	1.20	96.5
Mode 2	0.08	03.5

Table 3: Results of modal Analysis (Hi-tech Frame)

The nonlinear computer model with discrete damping elements were created using ETABS 7. Three sets of time history ground motions compatible to a 475-year return event were synthesized by Singh (2002). Nonlinear time history analyses using step-by-step linear acceleration procedure were conducted.

	SMRF w/ FVD	Conventional SMRF
Max. roof displacement	2.5in (6.4cm)	5.6in (14.2cm)
Max. base shear	0.29g	0.8g
Max. story drift ratio	0.010	0.016

Table 4: Performance Comparison for 475-year Record

Nonlinear time history analysis showed that all SMRF elements remained elastic. The maximum roof displacement is reduced by 55% from the conventional design; the base shear is reduced by 65%; and the maximum story drift ratio is reduced by 38%. The maximum FVD force per unit is 206kip (916kN) at the first level. These results show that the structural damage is eliminated and nonstructural damage is significantly reduced by adding FVD. Figure 7 shows FVD force vs. FVD displacement for one of the first floor FVD units. It shows the effect of the damping exponent 0.6. The shape of the hysteresis loop is between the oval ($\alpha = 1.0$) and the rectangular ($\alpha < 0.1$). Figure 8 shows FVD force vs. 2nd floor velocity for one of the first floor units. It shows nonlinear response of FVD unit. Figure 9 shows the base shear of SMRF with FVD and the conventional SMRF for the first 20 seconds. It shows a substantial reduction of the base shear. A linear time history analysis was conducted on the elastic frame of the conventional SMRF. The elastic frame was used

since the push over results show near elastic response of the conventional frame. Results of linear time history and pushover analyses are slightly varied. Figure 10 shows the roof displacement of SMRF with FVD and the conventional SMRF for the first 20 seconds. It shows a substantial reduction of the displacement. Figure 11 shows energy balance. FVD energy dissipates the majority of the input energy.

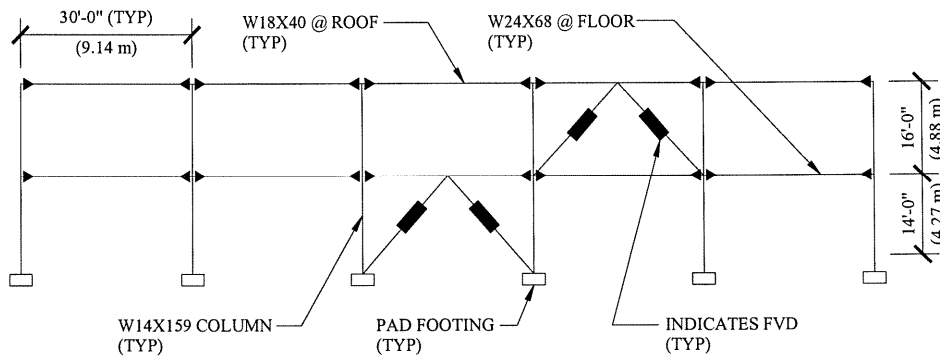


Figure 6: Longitudinal Frame Elevation with FVDs

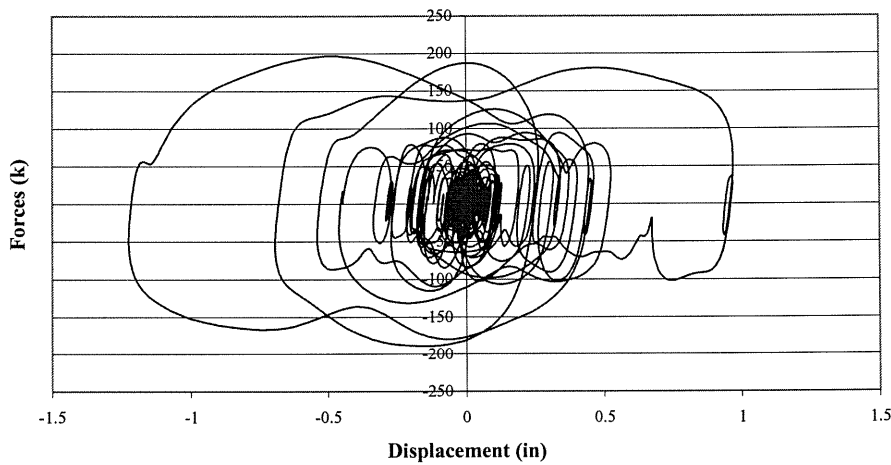


Figure 7: FVD Force vs. FVD Displacement for a 475-year Record

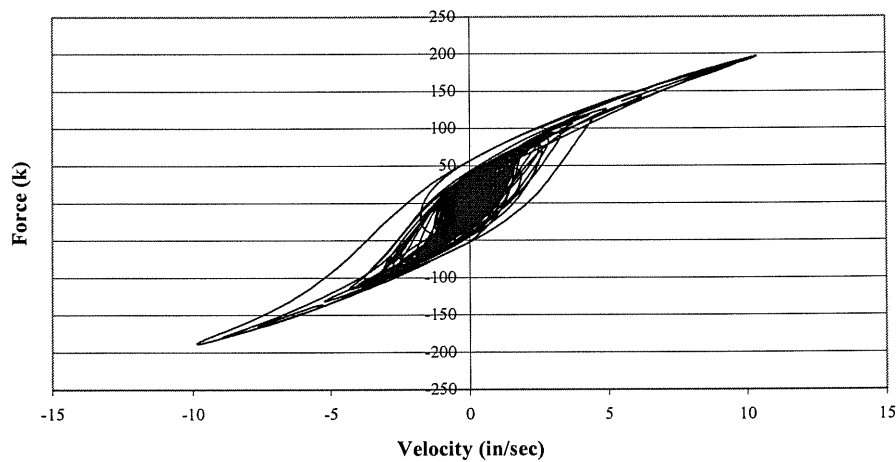


Figure 8: FVD Force vs. 2nd Floor Velocity for a 475-year Record

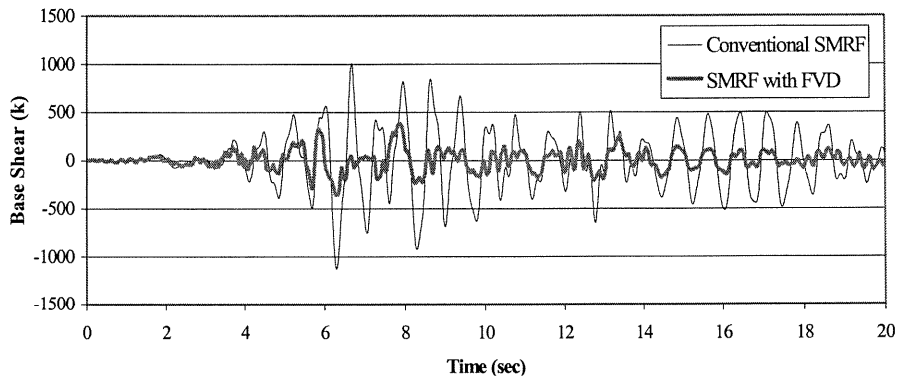


Figure 9: Base Shear of SMRF with FVD vs. Conventional SMRF for a 475-year Record

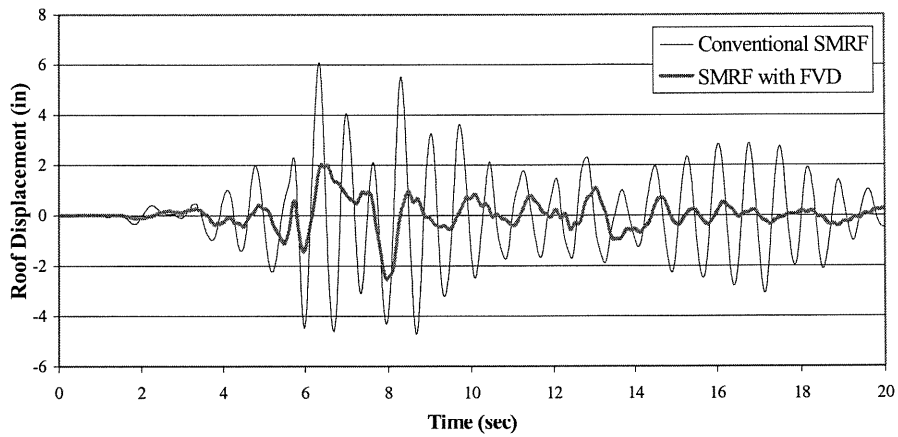


Figure 10: Conventional SMRF for 475-year Record vs. Roof Displacement of SMRF with FVD

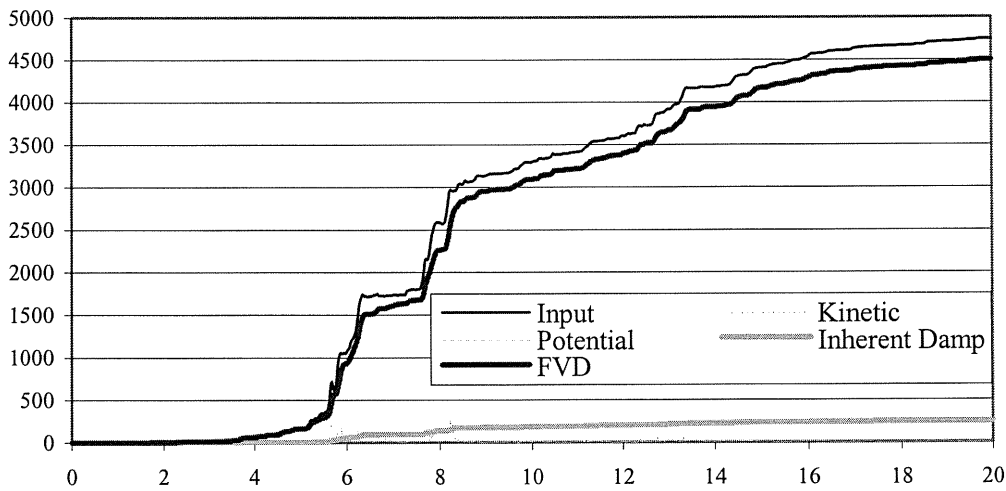


Figure 11: Energy Balance for a 475-year Record

6. Discussion and Conclusions

The results of this case study show that the 2000 NEHRP procedure is a very effective way to design a damped structure. The elastic frequency of the structure is shifted to a low frequency that generates lower floor and roof accelerations while the story displacements are controlled by dampers. The maximum story drift was limited to less than 1.0%, while all members remained elastic. The maximum base shear is 0.29g for a 475-year return event. The final study will include results of a 2,500-year return. These parameters indicate that structural and non-structural damages are significantly reduced when compared to conventional lateral system. The cost of FVDs are effectively offset by the reduction in costs of the foundation system and the structural steel of the roof beams.

8. References

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