US Design Procedure of Steel Moment Frames with Viscous Dampers

Kit Miyamoto, M.S., S.E., President & CEO
Marr Shaffer & Miyamoto, Inc., West Sacramento, CA

Robert D. Hanson, PhD., P.E.
University of Michigan, Walnut Creek, CA

ABSTRACT: This new 2-story, 40,000 ft^2 police headquarters will be the first building in the United States to apply 2000 NEHRP procedure to design an essential facility with Fluid Viscous Dampers (FVDs). The structure is located in Vacaville, California, which is in a region of high seismic activity and classified as zone 4 per 1997 Uniform Building Code. The lateral force resisting system (LFRS) consists of special steel moment frames with FVDs. In accordance with 2000 NEHRP, the LFRS is sized and designed with strength requirements of the code level force. FVDs are provided to control displacement of the structure. This design philosophy leads to a low frequency structure with low acceleration. FVDs reduce the displacement level to less than 0.01 story drift ratio. Earthquake performance and cost effectiveness are the primary concerns in designing this building. Site specific response spectra and time histories are synthesized for a 500-year and a 2,500-year return event. Performance Based Design using both linear and nonlinear time history analyses is conducted to ensure “immediate occupancy” performance. A cost study shows that much of the FVD’s cost is offset by reducing the weight of the LFRS while providing a far superior performance than the “code-compliant” structures.

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 will be the first structure in the United States to use this advanced procedure.

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, 40,000 ft^2 (3,716 m^2) steel framed structure. The roof is composed of metal deck and WF beams, and the floor is composed of $2^1/_2$ inch (6.4 cm) lightweight concrete over 3 inch (7.6 cm) 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 (61.0 cm) deep pad foundation is provided at WF columns.

Antioch (M 6.3) earthquake is attributed to the Greenville fault (Singh 2002). The 1997 Uniform Building Code (ICBO, 1997) ignores the near fault effects from the 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 475-year return and figure 4 for a 2,500-year return response spectra.

Figure 1: Architectural Rendering (Courtesy of Indigo Architecture)

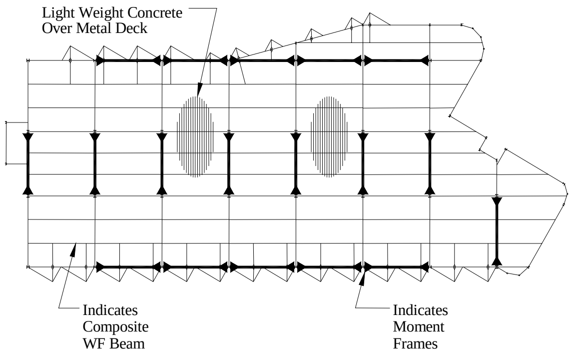

Figure 2: Second Floor Structural Plan

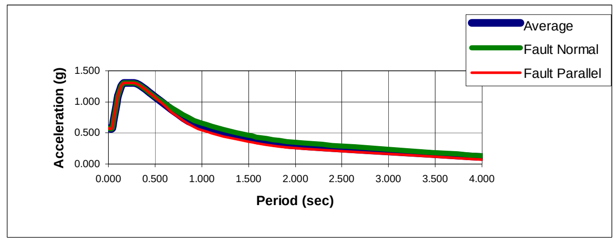

Figure 3: 475-year Return Response Spectra (Singh, 2002)

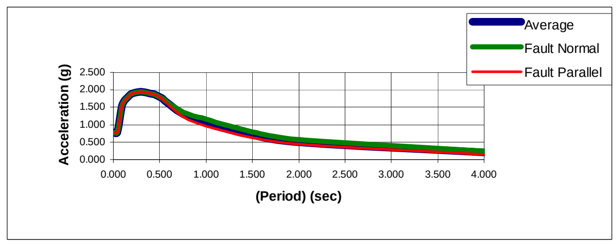

Figure 4: 2,500-year Return Response Spectra (Singh, 2002)

SEISMIC RISK

The site soil consists of 65 feet (19.8 m) 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 6.6 and 9.8 km, respectively. These faults are considered capable of surface rupture is the Green Valley-Concord fault located at 18 km from the site. The significant nearby earthquake was the 1892 Vacaville/Winters (M 6.5), which was attributed to the Great Valley Fault. The 1889

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 $\quad I = 1.5$

$S_{MS} = 1.95$g at 0.3sec. (site specific)

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

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

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

Seismic Design Category D

SMRF: $R = 8$, $Cd = 5.5$

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

$$Cs = \frac{S_{DL}}{RTa} I = 0.32\ g$$

$Ta = 0.4$

0.24g should be used for seismic shear.

The above value is compared with 1997 UBC.

$Ca = 0.44xNa = 0.44$

$Cv = 0.64xNv = 0.64$

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

$R = 8.5$, $I = 1.25$

$$V = \frac{2.5CaI}{R} = 0.16 \text{ g}$$

$$V = \frac{CvI}{RT} = 0.24 \text{ g}$$

0.16 g 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, 0.75x0.24 g = 0.18 g. SMRF is designed for both strength and drift criteria using 0.18 g base shear. The 0.18 g value is larger than the 0.16 g 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 $\frac{Cd}{I}$.

Figure 5 shows the longitudinal 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 (1,690kN) and of the floor is 924 kip (4,110kN). Table 1 shows the results of the modal analysis.

Table 1: Results of Modal Analysis (Conventional Design)

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

Nonlinear static pushover was conducted to gauge an earthquake performance of this frame. Figure 6 shows capacity/demand spectra with a site-specific 475-year return event. Please note that figure 6 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.

Maximum roof displacement	= 5.6 inch (14.2cm)
Base shear	= 0.80 g
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 0.8 g 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.

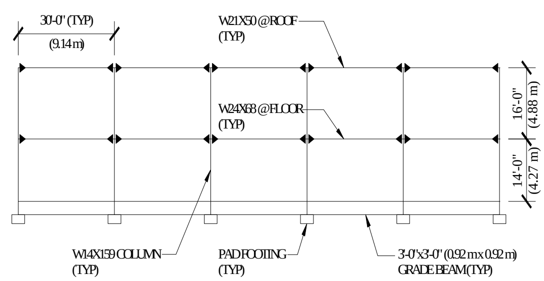

Figure 5: Longitudinal Frame Elevation

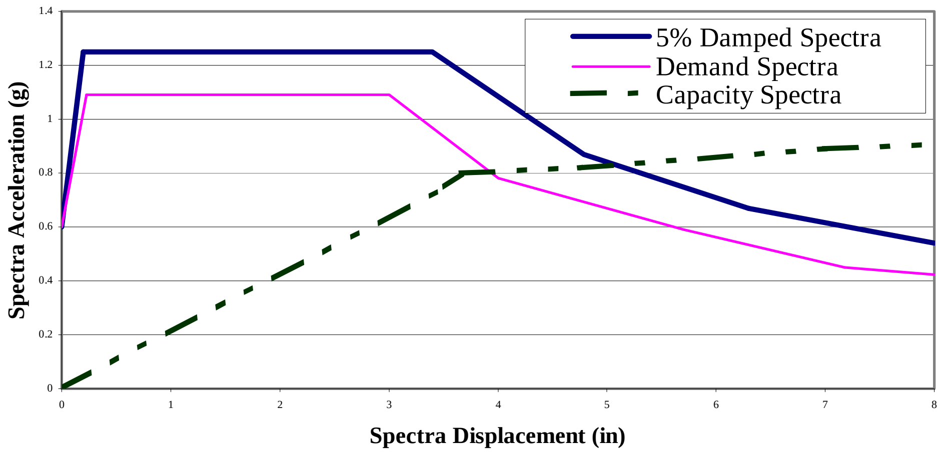

Figure 6: Capacity/Demand Spectra for Conventional Frame

HI-TECH SYSTEMS DESIGN

The structure was then redesigned using SMRF with FVDs per 2000 NEHRP. The base shear of 0.18 g 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 (0.18 g), and FVDs are provided to control displacement of the structure. See figure 7 for the new frame elevation. The difference from figure 5 is the "pinned" foundation condition and roof beam sizes. See table 2 for FVD properties.

Table 2: FVD Property

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

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 0.69 sec of the conventional SMRF to 1.2 sec. This frequency shift effectively brings the dynamic response to a lower acceleration range in the site-specific response spectra.

Table 3: Results of Modal Analysis

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

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. Table 4 compares the results of this analysis with the results of the push over analysis of conventional SMRF.

Table 4: Performance Comparison for 475-year Record

	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

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 206 kip (916kN) at the first level. These results show that the structural damage is eliminated and nonstructural damage is significantly reduced by adding FVD. Figure 8 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 (α = 1.0) and the rectangular (α < 0.1). Figure 9 shows FVD force vs. 2nd floor velocity for one of the first floor units. It shows nonlinear response of FVD unit. Figure 10 shows FVD force at one of the first floor FVD units and first floor column bending moment of the first 20 seconds of this 40 second record. It shows maximum FVD force is out-of-phase from maximum bending moment. Figure 11 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 12 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 13 shows energy balance. FVD energy dissipates the majority of the input.

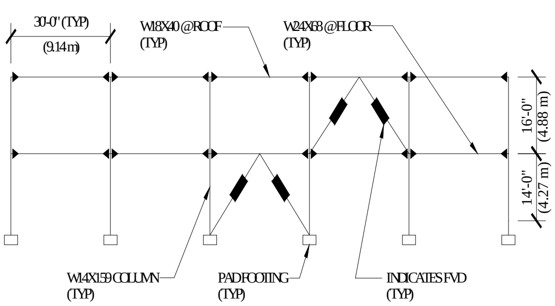

Figure 7: Longitudinal Frame Elevation with FVDs

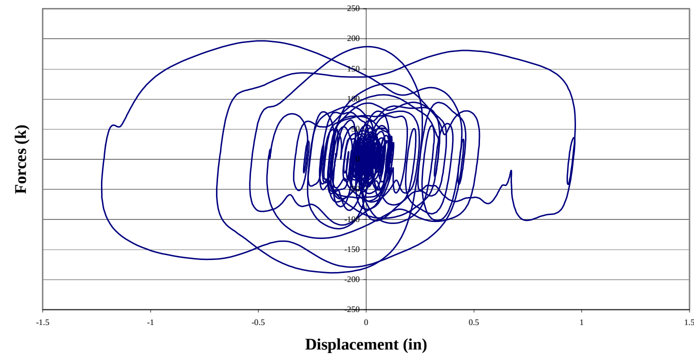

Figure 8: FVD Force vs. FVD Displacement for 475-year Record

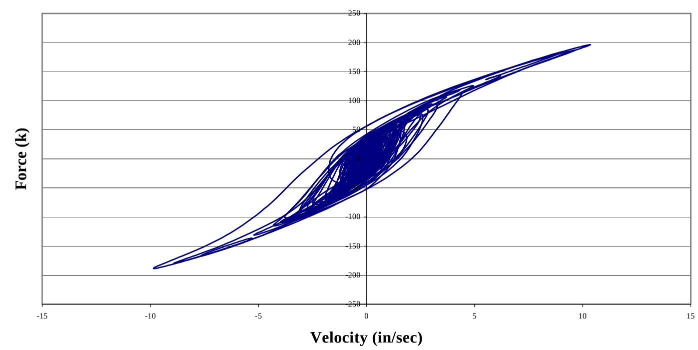

Figure 9: FVD Force vs. 2nd Floor Velocity for 475-year Record

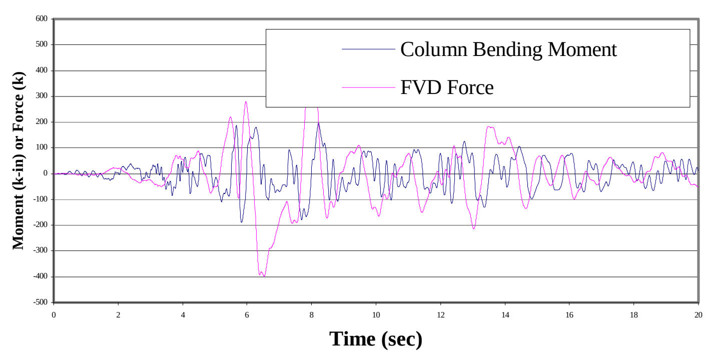

Figure 10: FVD Force vs. Column Moment for 475-year Record

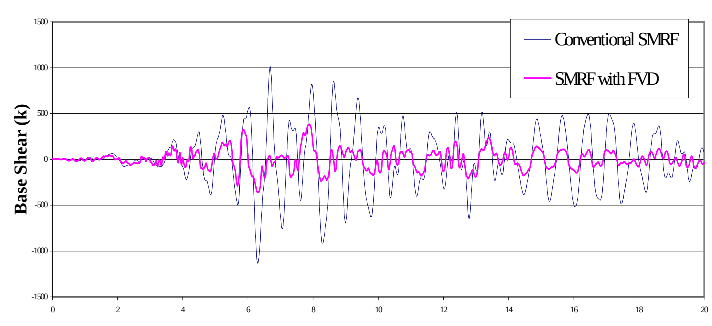

Figure 11: Base Shear of SMRF with FVD vs. Conventional SMRF for 475-year Record

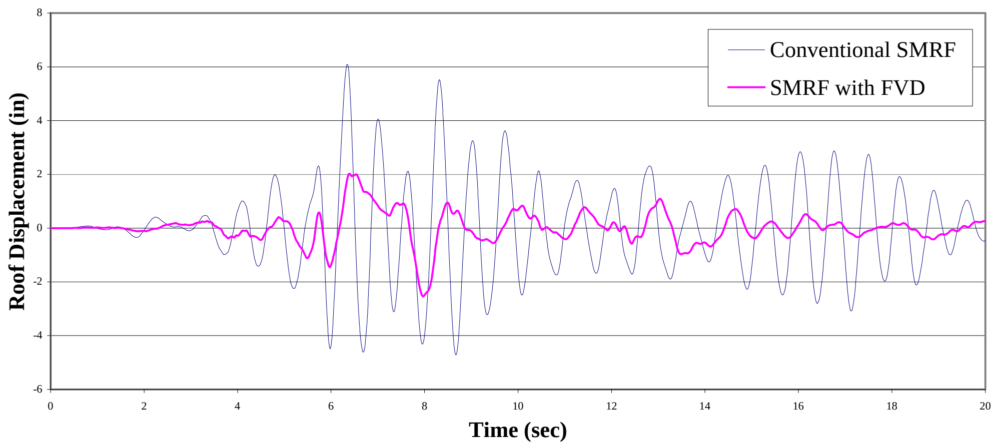

Figure 12: Roof Displacement of SMRF with FVD vs. Conventional SMRF for a 475-year Record

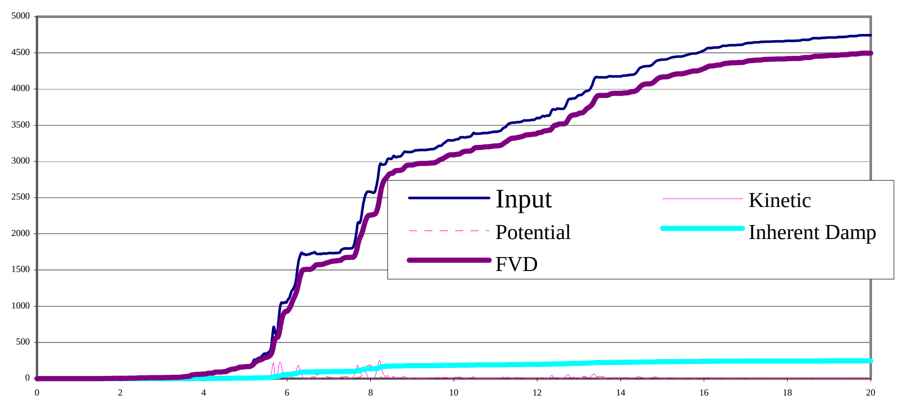

Figure 13: Energy Balance for 475-year Record

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 0.29 g 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.

REFERENCES

Gimmel, Lindorfar, and Miyamoto, “Design of a New Moment Frame Building Incorporating Viscous Dampers Following the Guidelines of the 1999 SEAOC Blue Book,” *Proceedings of ATC17-2*, May 2002, Redwood City, California.

NEHRP, 2000, “Recommended Guidelines for the Seismic Design of Buildings and Other Structures,” FEMA 368, Washington, DC

ICBO, 1997, Uniform Building Code, International Council of Building Officials, Whittier, California.

J.P. Singh and Associates, 2002, “Geotechnical and Seismological Findings and Recommendations,” Richmond, California