Rehabilitation of a 1985 Steel Moment-Frame Building

Gregg Haskell,[a] M.EERI

A 1985 steel moment frame is seismically upgraded using passive energy dissipation, without adding stiffness to the system. The design and analysis techniques for sizing the Velocity Braces™ and their impact on the demand capacity ratios are reviewed. The structure was built in the San Francisco Bay Area in compliance with the 1985 *Uniform Building Code (UBC)*. The moment frame contains the classic pre-Northridge nonductile moment connection, complete with weld backup bars left attached. Nonlinear time-history analysis procedures were implemented to verify the demand capacity ratios at the critical beam-column connections. Flexural demand capacity ratios of .6 achieve elastic behavior in the design basis earthquake with R=1.0. The response spectra of the time history chosen for design exceed the requirements of the 1997 *UBC* Zone 4. Torsional response to earthquake excitation is minimized by strategic placement of nonlinear viscous dampers. Nonlinear dampers that reduce the flexural demand on joints and control interstory drift without inelastic excursions of the beam flanges are achieved. Floor spectral accelerations and maximum drift limits are reduced to be consistent with immediate occupancy performance. The damper driver mechanism, being velocity driven, reduces moment frame demands and allows flexibility in configuration. [DOI: 10.1193/1.1572170]

INTRODUCTION

The 1994 Northridge earthquake failed many steel beam-column joints in a brittle manner—just as predicted by research done in 1988 by Tsai and Popov. The structure being upgraded was a well-configured two-story steel moment-frame structure with the beam-column joint design, constructed following the accepted detailing in use circa 1985 (Figure 1). Further, the structure met drift limitations and strong-column/weak-beam provisions of the 1985 *Uniform Building Code (UBC)* (ICBO 1985), which was assumed to deliver a life-safety performance in its day.

The new owners were changing the use of this structure to a web network communications center and investing $180/sq. ft. in new computer systems with battery backup to stay online. The owner required a 1997 *UBC* compliance, and thus the stage was set for a voluntary seismic upgrade effort engineered and constructed in 1999. This request was generated by the fact that approximately $100/sq. ft. of mechanical systems had been installed in the structure, based on compliance with the current SMACNA provisions (1998)—which were based on the 1997 *UBC*; consequently, the *FEMA 273* guide-

[a] 17554 Main Street, Knights Ferry, CA 95361

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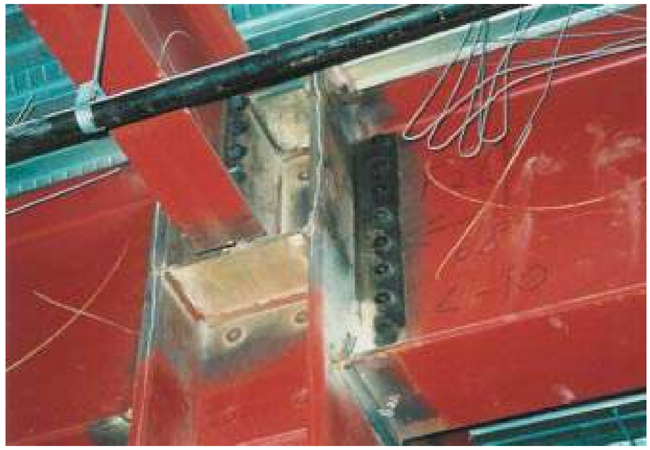

Figure 1. Typical existing moment-frame connection.

lines (ATC 1997) were not offered as an option. The seismic upgrade was requested when the owner was informed that his network communications center could be damaged and possibly closed without seismic structural renovation.

There are many approaches to performance enhancement. The first two are added strength approaches (*FEMA 273*): (1) Add concrete shear walls or braced frames, usually accompanied by new or strengthened foundations, or (2) strengthen individual beam-column joints. The second two options are (3) base isolation and (4) passive energy dissipation (added damping). The fourth option of adding viscous damping was chosen for this project to meet the performance demand within the shortest schedule and producing the least amount of disturbance.

DAMPER OPTIONS

There are many options to select from when considering dampers to implement a passive energy dissipation strategy for reduction of seismic response in buildings. The dampers currently thought of are viscoelastic, friction, hysteretic, lead extrusion, shape memory, and fluid viscous.

The viscoelastic family of dampers depends on a material that behaves elastically at low strains and viscously at high strain demands. Extensive research has been conducted at several universities across the country using this damper. The initial material tested had a temperature-dependent stiffness characteristic that made design with this product interesting. A new formulation of viscoelastic material has been generated that eliminates the temperature dependency; however, the single manufacturer that offers this product is no longer supplying the old formulation and has not begun manufacturing the new formulation. Thus, this option is not available at this time.

The family of friction dampers offers a very cost-effective damper to the market. The dampers work by having two plates of a known material slide across each other. Heat is generated and energy dissipated. Unfortunately, the friction devices have the least amount of successful research data as backup for performance. Discussion with Professor Michael Constantinou (1998) indicates that various materials have been tested over the last ten years and none have been found to deliver a long-term constant friction coefficient. Further, research by Canadian engineers has indicated that the friction dampers may not break loose under near-field earthquake excitation (Filiatrault and Kremmidas 1999). The friction damper has not developed to the point where it can be used with confidence in building structures where near-field effects must be considered.

Hysteretic dampers or plate-yielding devices have been in existence for more than twenty years worldwide. Only one building in the United States has these devices as part of the lateral energy dissipation system. Actually, *all* code-compliant lateral systems (besides base isolation) are technically hysteretically damped systems. The Northridge earthquake broke more than one hundred hysteretically damped steel moment frames, casting serious doubt on our ability to engineer and build *dependable* hysteretic systems.

Lead extrusion and shape memory alloy damping show good promise for future use. While lead extrusion dampers are cost-effective, the research database for these elements needs more development. There appear to be no buildings in the United States with lead-extrusion dampers installed. Shape memory alloys are currently not cost-effective, though they are intriguing.

The family of dampers with the largest research and testing database is the fluid viscous damper. The first fluid viscous damper was patented in France in 1897 and used to damp the shock from a 75 mm cannon. Every automobile and truck on the road today has fluid viscous shock absorbers installed. Dozens of civilian structures (buildings and bridges) in the United States are currently protected from seismic events with viscous dampers. Eight buildings in California have viscous dampers in their lateral systems. Caltrans has run extensive tests on prototype viscous dampers at the U.C. Berkeley Earthquake Engineering Research Center. Caltrans will be using large-capacity viscous dampers to protect the Golden Gate Bridge from future earthquakes. More than 950 fluid viscous dampers and variants are currently in civilian buildings and bridges in the United States. Based on the above information, the fluid viscous damper is the best family of dampers for use in lateral systems.

Adding viscous dampers using a damper driver mechanism known as a Velocity Brace™ (Patent No. 5,845,438) resulted in no stiffness change in the lateral system, while reducing drift to elastic limits (.0067h). The Velocity-Braced structure qualified for SP1 (Operational) per 1999 Blue Book criteria (DBE drifts less than initial yield) (SEAOC 1999). The time history chosen for design had a response spectrum that exceeded the 1997 code spectra for the site.

The design process was one of successive approximation. The first-order approximation hand calculated as a rough estimate for damper size and velocity. Due to the symmetry of the lateral framing and the floor plan, the second-order approximation using nonlinear time-history analysis on a plane frame model was sufficient for design. Finally, a three-dimensional space frame was analyzed to check torsional response.

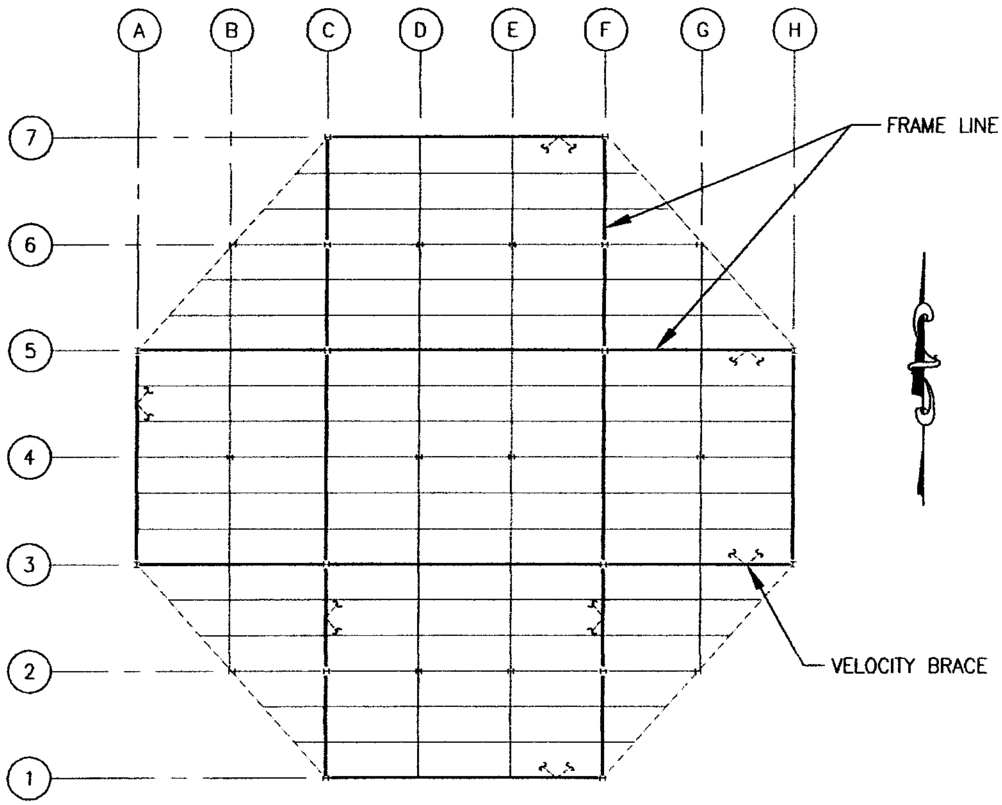

Figure 2. Roof framing plan.

STRUCTURE

The flat roof vertical load system consists of 1-1/2-inch 18 ga. roof deck spanning 10 feet to W16×26 roof beams. The roof beams span 30 feet to W18×40 girders, which span 30 feet to W14×76 columns bearing on spread footings (see Figure 2).

The second-floor vertical load system is 2-1/2-inch normal concrete on 3-inch 18 ga Formlok metal deck spanning 10 feet to W18×35 floor beams. The floor beams span 30 feet to W24×76 girders spanning 30 feet to W14×76 columns bearing on spread footings (see Figure 3).

The east-west lateral system consists of three bay moment frames on Lines 1 and 7, utilizing W27×94 floor frame girders and W21×44 roof frame girders. Lines 3 and 5 are seven bay moment frames with W21×44 roof girders and W30×99 floor girders and W14×176 columns. The columns at each frame are W14×109 at second floor to roof and W14×176 grade to second floor.

Four Velocity Brace bays with twin 50 kip viscous dampers (Figure 4) are positioned along the frame lines, in each direction.

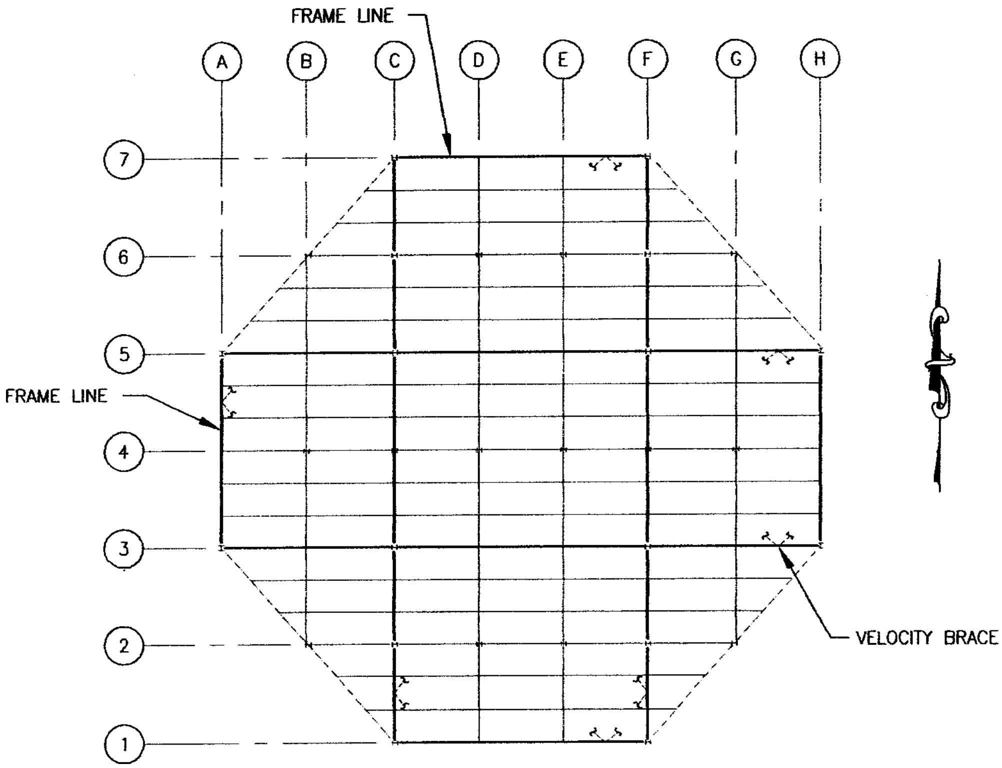

Figure 3. Second-floor framing plan.

The north-south lateral system is similar to the east-west frame. Since the east-west frame lines carry beam loading only, the flexural demands do not control and thus are not presented (see Figures 2 and 3).

The story height is 16 feet for each level. The weight of the second floor is 2,180 kips and that of the roof level is 1,000 kips.

GROUND MOTION

The site is between the Hayward Fault (Class A) and the Livermore Fault (Class C) and just over 10 km from the Hayward Fault. The soil type is S_d. From the 1997 *UBC*:

$$C_v = .64 \ \text{g}$$

$$C_a = .44 \ \text{g}$$

$$T_s = C_v/2.5 \times C_a = .58 \ \text{s}$$

$$T_o = .2 \times T_s = .12 \ \text{s}$$

Using these factors, a design response spectrum was constructed (see Figure 5).

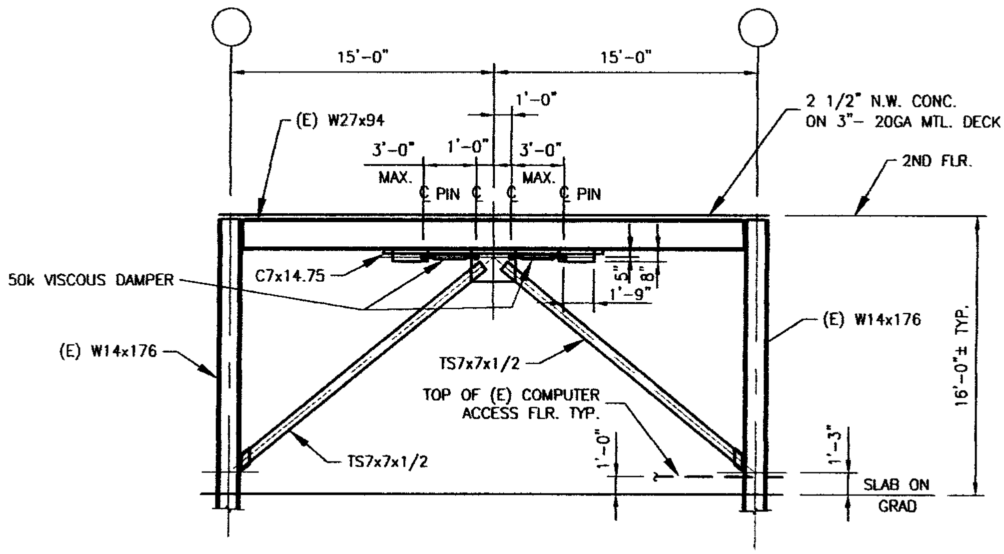

Figure 4. Typical bay.

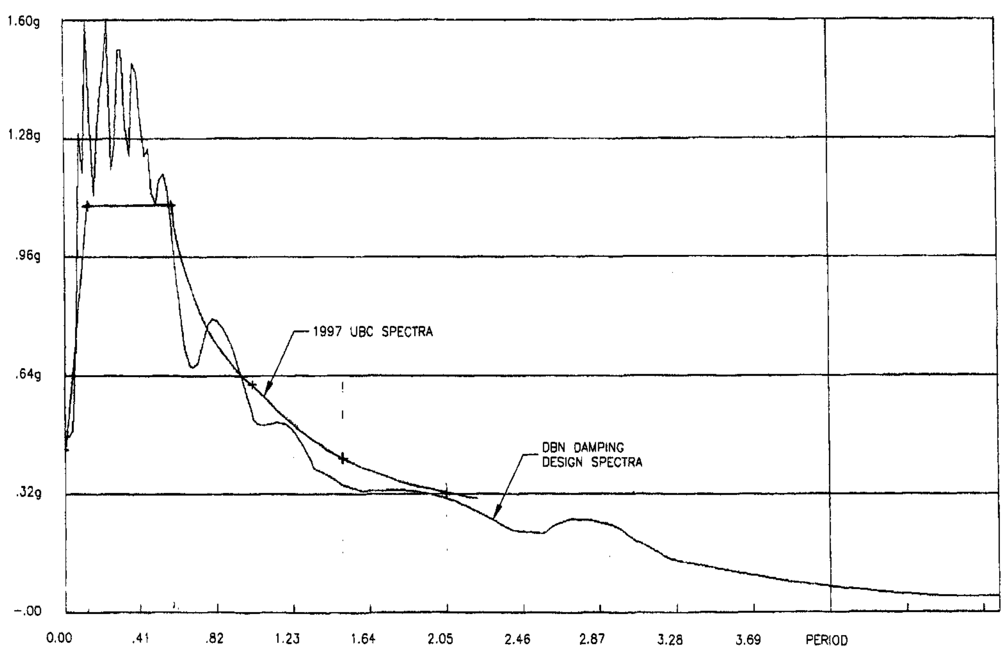

Figure 5. Controlling earthquake response spectra.

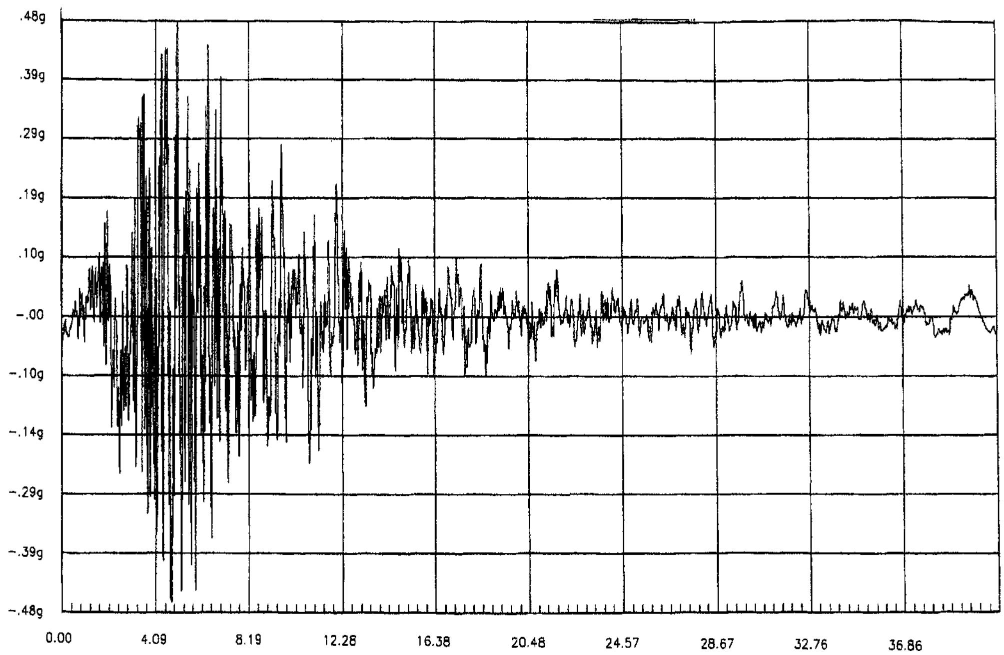

Figure 6. Controlling time history.

Three time histories from a similar site anchored a .44 g EPGA were run through the simplified plane frame linear model without added damping to determine the maximum roof displacement. The response spectra for this motion is shown on Figure 5 and the time history is plotted on Figure 6.

Preliminary sizing of the dampers placed in steel moment frames is done using results of dozens of analysis and designs. Simply stated, the force in the viscous dampers placed in the horizontal position is approximately 10% of the building weight above the level of damping. For this building, grade to second floor requires approximately 318 kips to damping force. The velocity in the viscous damper is a function of the interstory relative velocity. The dampers placed below the second floor experience the difference between the second-floor velocity response history and the foundation velocity time history. From experience this damper velocity is given by

$$V_i = S_v/4$$

$$V_i = \{T_a \times S_a \times g/2\pi\}/4$$

$$T_a = \text{Actual First-Mode Period} = .93 \text{ s}$$

$$S_a = 2.5(\text{EPGA}) = 1.1 \text{ g}$$

$$g = 386.4 \text{ in/s}^2$$

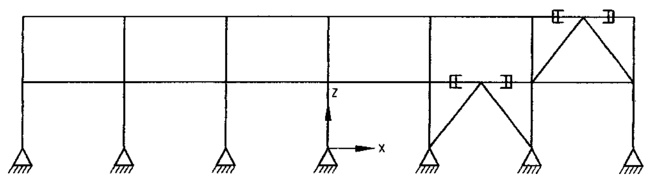

Figure 7. Typical frame elevation.

For this building $V_i = 16$ in/s.

For preliminary design the damper force is assumed to be linear on the velocity:

$$F_i = C_i V_i$$

$$C_i = F_i / V_i = 318/16 = 20 \text{ k-s/in}$$

Using the above preliminary estimates for the damper coefficient, a simplified plane frame was modeled. Using the ground motion and SADSAP nonlinear analysis program, the following velocities were attained: The second-floor horizontal velocity was at 12 in/s and the damper force for the structure for grade to second floor is 440 kips. Also, the horizontal displacement of the second floor is 1.2 inches. Actually the design could be frozen right here because the first-floor story drift was .006 times the story height.

FINAL DESIGN

The existing symmetry of the lateral system and mass distribution allows use of a single plane frame for final sizing of the dampers (see Figure 7). The damping coefficient from preliminary design and linear dampers generated a first-level damping force of 376 k when the time history analysis was conducted.

The associated second-floor drift is 1.3 inches or .0067 h. The design could be stopped here, however, damper manufacturers prefer nonlinear dampers to ease orificing design. Using velocity exponent of .4 with a damping coefficient of 30 k-in/s and a total of 400 kips damping from grade to second floor and 400 kips damping from second floor to roof, the second-floor displacement is 1.34 inches, or {.007 h} (Figure 8). The demand capacity ratios for beams and columns are given in Table 1. Figure 4 shows the typical Velocity Brace, and Figure 9 shows a photograph of the installed Velocity Brace, damper, and deadman. A detail of the Velocity Brace is given in Figure 10.

CONSTRUCTION ISSUES

The cost for adding shear walls to this structure was estimated at \$15/sq. ft. Also the delay associated with construction of these elements with attendant foundation system

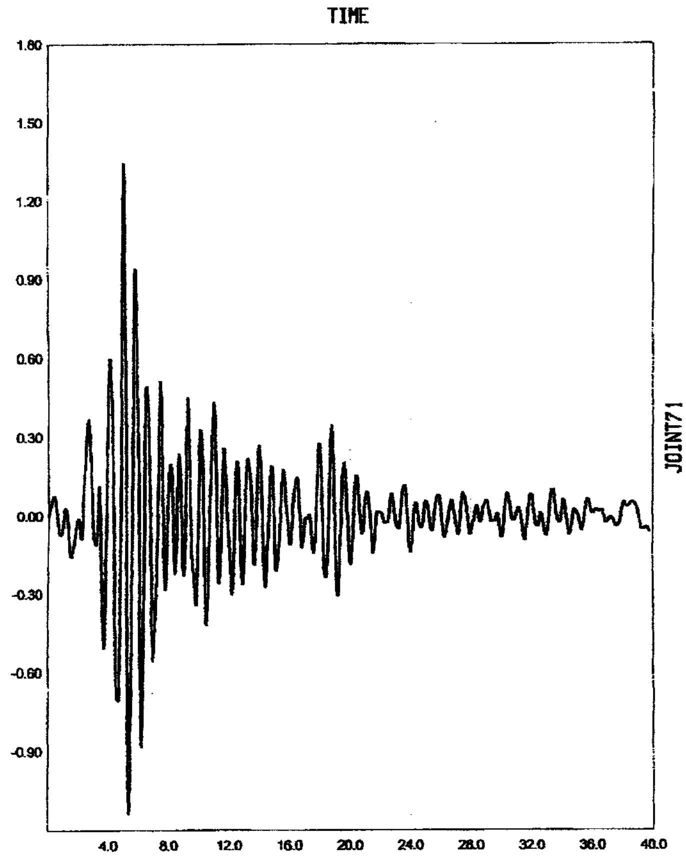

Figure 8. Second-floor displacement.

was unacceptable to the owner. Adding brace frames with the associated foundation concrete priced out at $12/sq. ft. for this structure. Delays caused by foundation construction effort were unacceptable to the owner. Strengthening of the individual beam-column joints would have cost $20/sq. ft. and again the construction time did not fit the owner's schedule. The construction time associated with base isolating this structure eliminated

Table 1. Frame element demand capacity ratios

Element	Demand/Capacity Ratio
Roof frame girders W21×44	.39
Roof frame columns W14×109	.23
Floor frame columns weak axis	.34
Floor frame girders W30×99	.58
1st floor column W14×176	.44

Figure 9. Typical damper installation.

this option. Passive energy dissipation, that is, the addition of Velocity Braces using viscous dampers, priced out at $4.20/sq. ft., was the least costly, and had the shortest installation time of the reviewed seismic upgrade options. The structural steel associated with the Velocity Braces was installed prior to the construction of the computer floor

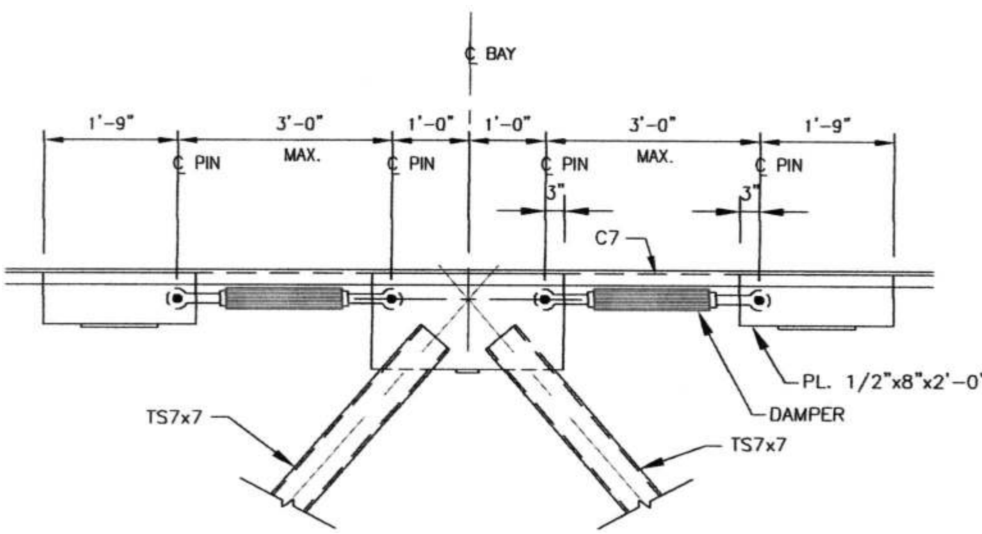

Figure 10. Typical Velocity Brace detail.

Figure 11. Typical interior clean room.

system. The viscous dampers were installed after the computer floors and acoustical tile ceilings were in place, due to the lead time for delivery of dampers. There was no interruption to the owner's schedule for the installation of the dampers. Figure 11 shows the finished clean room with viscous dampers ready to be installed. Figure 12 shows the exterior of the finished building.

One of the interesting characteristics of the Velocity Brace is that the demand in the brace is limited to the velocity across the dampers at each site. Offsetting the Velocity Braces in plan view from floor to floor creates no significant engineering problem.

Since the forces in the braces are limited and the Velocity Braces are placed in an existing moment frame with large W14 columns the connection of the brace to the column can be raised above the work point of the beam-column joint. In this building, the braces intersect the column above the 12-inch computer access floor generating a clean, easily installed detail at each braced column intersection (see detail, Figure 11).

Figure 12. Exterior elevation.

RESULTS

As shown in Table 1, in the north-south direction the W21×44 roof frame girders have M_u/M_p equal to .39. The roof columns have demand capacity ratio of .23. Second-floor frame girders (W30×99) operate at .58 times their flexural capacity. First-level columns (W14×176) are pushed to 44% of their capacity. Horizontal drift at the second floor is .007 h and causes less than 1% rotation demand on girders.

The amplification of the input ground motion is minimized. The second-floor horizontal acceleration is .51 g, while the roof acceleration is .48 g.

DESIGN CHECK

A 3-D SAP2000 nonlinear model of the structure was constructed. The results of the SAP2000 nonlinear time-history analysis showed that the demands resulting from the 3-D frame were within 10% of the space frame results.

CONCLUSIONS

1. Simplified models are sufficiently accurate for design when compared to detailed 3-D model results for this building.

2. Interstory drift can be kept to less than .007 h with use of horizontally placed viscous dampers for design basis earthquake for this steel moment-frame building.
3. Demand capacity ratios of frame girders in this structure can be kept to less than .5 in the DBE.
4. With horizontal drift at .007 h, rotation demands on this structure's girders are less than 1%.
5. The amplification of the input ground acceleration is limited to a 16% increase, from .44 g to .51 g at the second floor for this building.
6. For this structure, Velocity Braces with viscous dampers is the least-cost seismic upgrade option.

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