Enhancing Sustainability of Structures: Retrofit of a Historic Building Using Energy Dissipation

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Abstract

Energy dissipation was used to retrofit a historic high-rise reinforced concrete building in California. The building was constructed in the 1920's and was the first high-rise structure in the state capital. It has 14 stories and a basement. The building is a California registered historic structure, and has many important architectural features such as terra cotta tile finish. Concrete moment frames with limited ductility comprise the lateral-load resisting system. As part of the city's downtown revitalization, this historic structure is being transformed from an old office building into a premier boutique hotel. Due to this change of occupancy, the building needs to satisfy California Building Code performance objectives. Performance Based Engineering revealed that the building would experience excessive story drift at its middle stories. This drift ratio could cause damage to both structural and especially to the historic architectural components. Energy Dissipaters were added to these critical stories to alleviate this deficiency and to satisfy the performance objectives. Sustainability of this structure was enhanced by using the innovative technology.

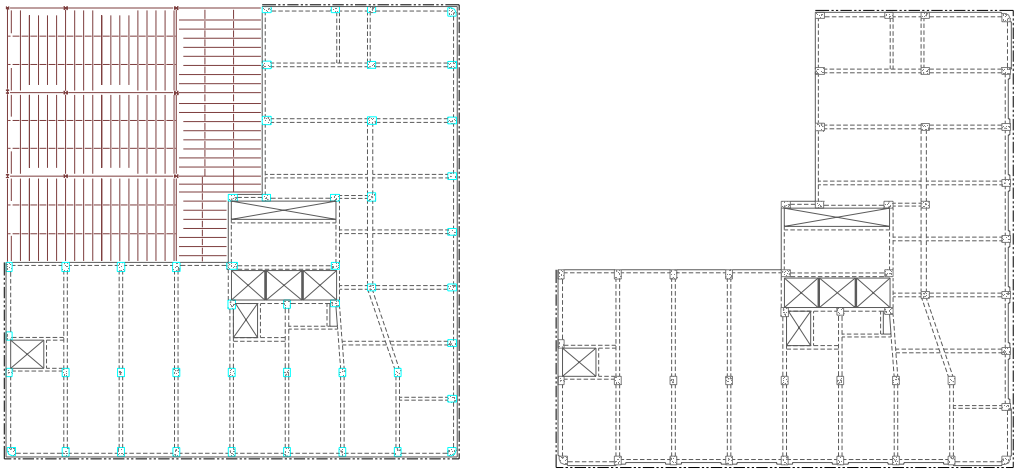
Description of the Structure

The building is 200 ft (61 m) tall and has 14 stories and a full basement. It is located at the corners of 10th and J Streets in downtown Sacramento, California. It was constructed in 1922 as the first high-rise building constructed in the city. Full-length perimeter walls span between the ground floors and the basement. The footprint for the original construction was L-shaped, measuring 80 x 120 ft (24 x 36 m) with a total area of approximately 70,000 ft^2 (6,500 m^2). Two later additions to the buildings were a 6-story, 23,000 ft^2 (2,100 m^2) annex in 1932, and a 2-story L-shaped annex in 1950. Figure 3 presents a photograph of the building. As seen in the figure, the building has many unique architectural features, including a terra cotta skin on the perimeter. Because of its many unique features, this building is considered a historic structure that must be preserved.

The gravity load-resisting system consists of 5- and 6-in. (125- and 150-mm) thick reinforced concrete one-way slabs supported on reinforced concrete beams and columns. Reinforced concrete moment frames resist lateral loading; see Figure 2. Additionally, between first and second floors, 8-in. (200-mm) thick reinforced concrete walls span the entire length on two faces. Reinforced concrete walls extended approximately half of length on two faces, between the second and third floors. At the lower levels, the concrete columns measure 24 in. (600 mm) square, and are reinforced with up to twenty 1-1/8 in. (29 mm) square bars. The transverse reinforcement for columns is 3/8 in. (9 mm) diameter bars at 4-in. (10-mm) spacing. Smaller column sizes and reinforcement are used at upper floors. A variety of beam sizes were used in the building. Typical beams measure 10 x 24 in. (250 x 600 mm) and are reinforced with two ¾-in. (19 mm) square bars at top and bottom. ½-in. (13 mm) diameter stirrups are used as beam ties. Deep spandrel beams up to 43 in. (1,100 mm) deep occur along the perimeter at the lower levels. Figure 3 presents the typical floor plans for the building.

Figure 1. Photograph of the building

Figure 2. Reinforced concrete frames



a. Third floor

b. Eighth floor

Figure 2. Building typical floor plans

Performance Objectives

The building was originally designed as a mixed-use facility. In 2004, the owner decided to evaluate and undertake seismic evaluation of the building for use of California state government tenancy. This implied a one-level occupancy upgrade and the building had to meet the Life Safety (LS) performance for the Design Basis Earthquake (DBE). In 2006, it was decided to convert the building to a hotel. Since this meant a two-step upgrade in occupancy, the building now also had to meet Collapse Prevention (CP) for the Maximum Credible Event (MCE). These two performance objectives—LS at DBE level and CP at MCE level—were used in evaluations presented hereafter.

Seismic Demand

Geotechnical investigation [Singh, 2007] was conducted to determine the site-specific acceleration spectra for this building. DBE and MCE spectra (475-year and 2,500-year return periods, respectively) were developed based on the site geological evaluation, proximity of active faults, and historic seismic activities. Three sets of time histories were prepared to match each of the two spectra. The acceleration

seeds were taken from the 1989 Loma Prieta earthquake. The DBE and MCE spectra are anchored at 0.2 and 0.25 g and have maximum spectral accelerations of 0.44 and 0.57 g, respectively.

Response of the Existing Building

Computer programs ETABS [CSI 2007a] and SAP 2000 [CSI-2007b] were used to prepare mathematical models of the building. All pertinent stiffness and mass components were included in the model. Compressive strengths of 5.2 and 4.6 ksi (39 and 32 MPa) were obtained for concrete columns and beams, respectively from material samples tested by Wallace-Kuhl [WK 2005]. Nominal properties of Grade 40 (275 MPa) steel were used for reinforcement. Nominal dimensions, as shown in contract plans were used in analysis. Member sizes and spans were verified during site visits. Beam-to-column connections were modeled as rigid. Concrete floor slabs were modeled as shell elements and rigid diaphragms were applied to them. Member plastic hinge properties were derived using the FEMA 356 [NHERP, 2000] recommendations. Conservatively, the stiffness contribution from the unreinforced masonry (URM) infills was ignored—and equivalent struts were not included in the model because multiple windows perforate these infills; see Figure 4. The inertial weight of the building is estimated at 23,000 kips (100 MN).

Table 1 presents the modal properties of the building for the first six modes. In analysis, 18 modes were used to ensure that over 90% mass participation of the structure was accounted for. Note that the fundamental mode has a period of approximately 2.1 to 2.2 sec, the modes are uncoupled, and nearly 60% of the total mass participates in the first mode.

Table 1. Modal Properties

		Mass participation, %		
Mode	Period, sec	x-	y-	θ-
1	2.2	59	4	6
2	2.1	4	58	3
3	1.9	1	2	54
4	0.8	12	3	1
5	0.8	2	11	0
6	0.7	0	0	11

The building was subjected to the response spectrum loading and its response was evaluated. Analysis indicated that the building did not exhibit any irregular response; there was neither soft story response, nor, amplified torsional behavior. Figure 3 presents the displaced shape of the exterior frame subjected to response spectral loading in the x- (N-S) direction. Note that the maximum story drifts occur near the mid-height of the building, between fourth and eighth floors. Although, the story drifts are not excessive at these levels, they could potentially cause damage to the existing nonstructural elements, including the terra cotta and URM infill at these levels.

For regular structures with a dominant first mode response, such as the building under investigation, static nonlinear (pushover) analysis can be used to accurately estimate the seismic response of the building. The control node was selected at approximately the center of mass at the attic level. In each lateral direction, two loading patterns were considered: one resembling the deformed shape obtained from the response spectrum analysis, and the other proportional to the seismic mass at the floors (uniform acceleration). The structure was preloaded with gravity effects, then incrementally displaced to the target displacement computed from nonlinear response history analyses. The concept of equal displacement was utilized. Figure 4 presents the displaced shape of an exterior frame at target displacement of approximately 8 in. (200 mm) at the control node—a value close to the anticipated roof displacement during the DBE event.

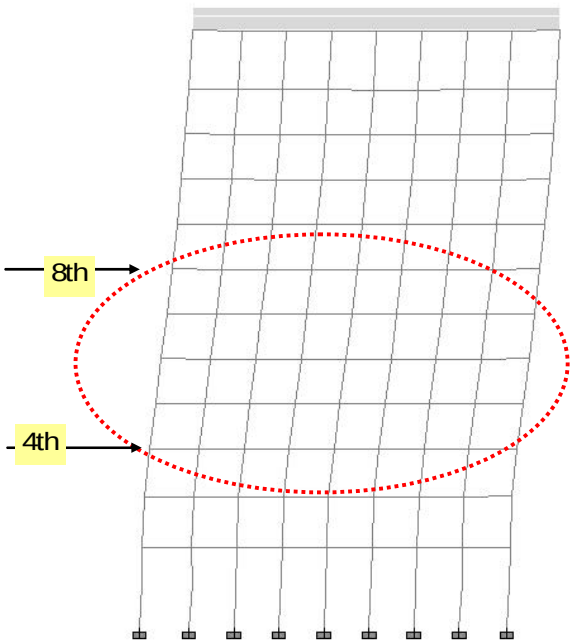


Figure 3. Response spectrum deformation

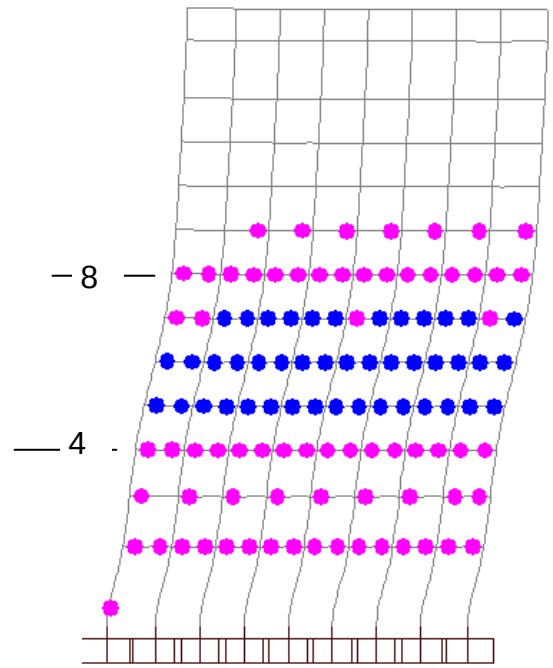


Figure 4. Displaced shape

The building global response is adequate as it meets the LS requirements. However, the largest drifts and plastic hinge rotations occur at the mid-height of the building; see Table 2 results of two-dimensional analysis at the DBE level. The mid-level floors experience large deformations and seismic demand on the concrete beams.

Table 2. Story Drifts and Plastic Hinge Rotations, DBE Level

Floor	Drift, %	PH rotation, % radian
Attic	0.18	0
12	0.26	0
11	0.38	0
10	0.54	0
9	0.67	0.03
8	0.73	0.25
7	0.65	0.58
6	0.53	0.79
5	0.47	0.77
4	0.37	0.37
3	0.12	0.05
2	0.13	0.08

Seismic Retrofit

Fluid Viscous Dampers (FVDs) were used at mid-story levels to reduce the story drifts at these locations. The two main objectives of the retrofit were: 1) reduce plastic hinge rotations, and 2) protect the historic architectural components. FVDs provide an effective and economical method for retrofit of reinforced

concrete structures. They are external devices, originally developed for shock and vibration control in the defense and aerospace industries. In the past decade, they have been used for seismic protection in both retrofit and new construction for many structures including reinforced concrete buildings. FVDs consist of a cylinder and a stainless steel piston. The cylinder is filled with incompressible silicone fluid that has stable properties over a wide range of operating temperatures. FVDs are activated by the transfer of the silicone fluid between chambers at opposite ends of the unit through small orifices. The mechanical construction and orifice properties can be varied to obtain the desirable damper properties.

Figure 5 shows the connection details for the FVDs. As shown, the damper forces are transferred to the existing concrete members. FRP collectors were then provided to transfer this force to the rest of frames. At each floor, two dampers were provided in each direction; see Figure 6. The damper properties were specified as required by FEMA 356 [NEHRP, 2000]. Figure 7 shows a photograph of an installed damper.

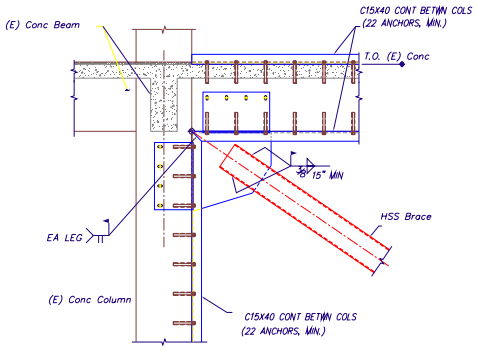


Figure 5. Connection detail (elevation)

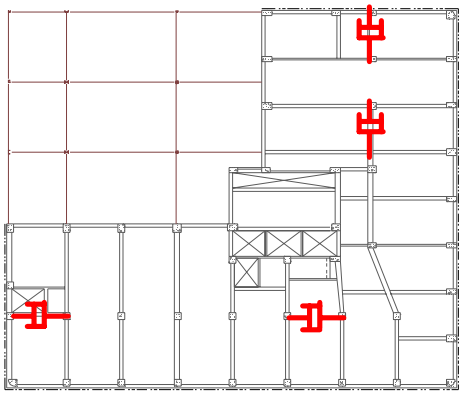


Figure 6. Typical location (plan)

Figure 7. Installed damper

Response of the Retrofitted Building

FVDs, steel braces, and fiber-reinforced polymer (FRP) composites were used in the seismic upgrade. Sixteen FVDs were added between the fourth and eighth floors to reduce story drift ratios and seismic demand on the reinforced concrete members at these levels. Diagonal steel HSS braces were added between the first and second floors. These braces were placed opposite the existing 8-in. (200 mm) reinforced concrete wall and were intended to add lateral stiffness and reduce torsional response at this level. FRP was added to the floor slabs to serve as drag struts (collectors) and served to transmit and distribute the seismic forces between the damper bays and the floor diaphragms. The dampers have a velocity exponent (α) of 0.5 and a damping constant (C) equal to 300 kip-sec/in. (8.5 MN-sec/m). The steel connections at the ends of dampers and the components transferring the damper forces to the existing members were designed for the MCE level forces. Since these forces are substantially out-of-phase with elastic forces, they do not significantly increase loading on these existing members.

Nonlinear analyses were performed to assess the seismic response of the retrofitted building at the MCE and MCE levels. Figure 8 presents the x-component of the computed displacements at the attic-level control node. The displaced shape of the building at this level of displacement is shown in Figure 9. Note that the building met its design objectives.

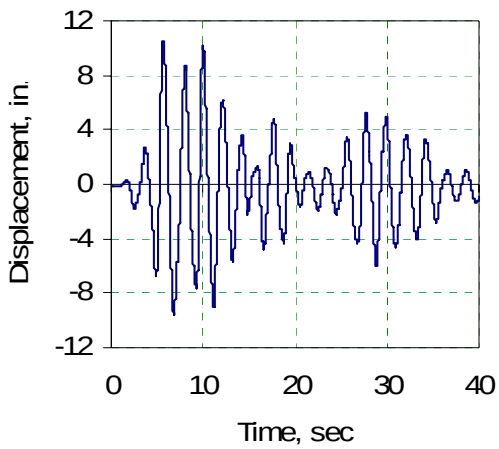


Figure 8. Roof displacement

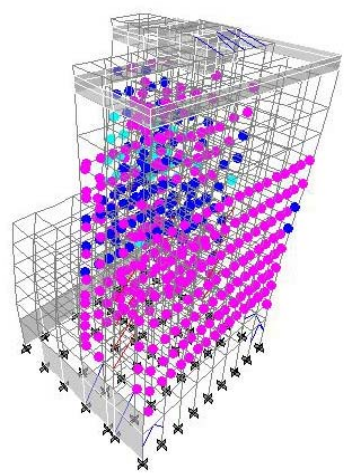


Figure 9. Displaced shape

Figure 10 shows the normalized pushover curve for the building. The ordinates of roof displacement and base shear were normalized with respect to the building height and mass, respectively. The horizontal axis corresponds to an average drift ratio for the building and the vertical axis is its base shear coefficient (BSC). The DBE and MCE performance points are marked in the figure. The structure has substantial reserve capacity beyond the MCE demand. The analysis was carried up to a roof displacement of approximately 16 in. (400 mm)—significantly greater than the MCE target displacement. Although some of the plastic hinges in the concrete beams exceeded their CP level, the overall building response remained stable.

Table 3 presents the computed MCE drift ratios for the existing and retrofitted building obtained from the three-dimensional analyses. Only the drifts at the critical middle stories are shown. Note that the addition of dampers has reduced the story drifts by over 20%.

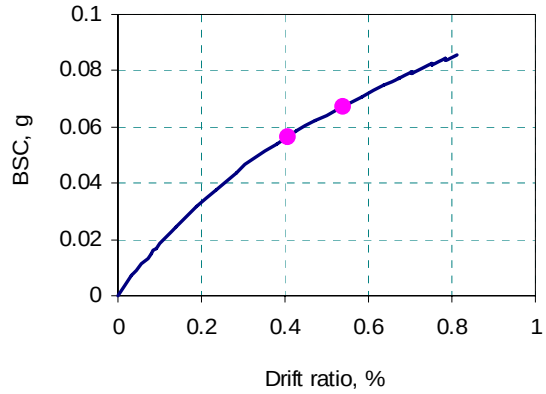
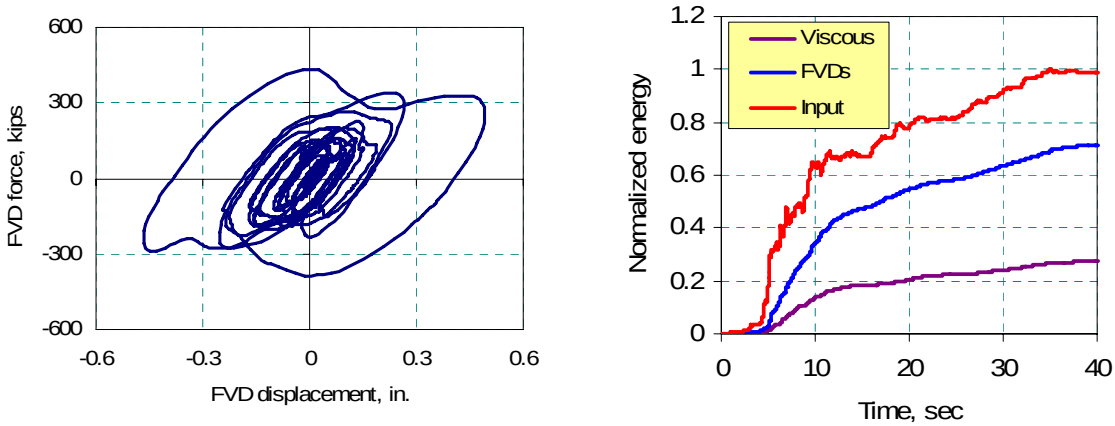


Figure 10. Normalized pushover curve

Table 3. Story Drift Reductions

Floor	Existing	Retrofitted	Reduction, %
8	0.92	0.84	9
7	0.87	0.72	18
6	0.74	0.59	24
5	0.59	0.51	14

FVDs increase the equivalent damping of the building by dissipating the seismic energy. Figure 11a presents the nonlinear force-displacement hysteresis for one of the FVDs during one of the MCE event. The FVD absorbs significant energy. Figure 11b shows the component of the seismic energy for the same event. For clarity, the data is normalized. The dampers dissipate approximately 75% of input seismic energy. In the absence of the FVDs, the yielding of the existing concrete members would have been subject to this energy demand.



a. FVD response

b. Components of seismic energy

Figure 11. Energy dissipated by FVDs for a typical acceleration history

Conclusions

1. State-of-the-art analysis of a historic reinforced concrete high-rise retrofitted with FVDs showed that the structure met its performance goals. Story drifts and member nonlinear actions were kept within the acceptable limits.
2. Performance-based earthquake engineering was effectively used to assess the seismic response of a historic reinforced concrete structure. This method readily identified a building weakness and the method to mitigate it.
3. FVDs provide an efficacious, cost-effective, and non-intrusive retrofit method. They act to increase the effective damping, do not stiffen the building, and reduce the seismic demand. The force in FVDs is primarily out-of-phase with elastic forces.

References

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