Seismic Retrofit of a Hospital Building Using Fluid Viscous Dampers

Amir Gilani[1], Kit Miyamoto[1], and Nahib Youssef[2]
1. Miyamoto International, Inc. West Sacramento CA/USA
2. Nahib Youssef & Associates, Los Angeles, CA/USA

ABSTRACT

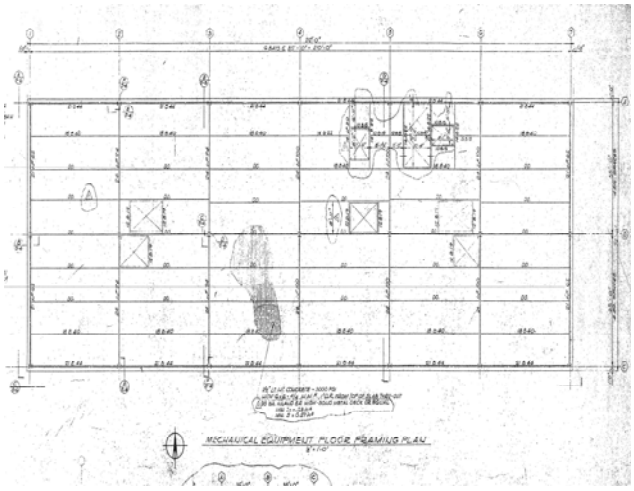
Seismic performance of a multi-story hospital structure located in Southern California was evaluated using performance-based earthquake engineering. The uppermost eight stories of the superstructure use steel framed construction, whereas reinforced concrete framing is used for the lowest floor (L2) and the four sub-grade parking stories. The building is rectangular in plan, has a typical story height of 3.6 m (11.8 ft) and a total floor area of 12,000 m^2 (130,000 ft^2). In both lateral directions, steel moment-resisting frames using Pre-Northridge connection details were used to resist lateral loading. Reinforced concrete framing comprise the lateral load resisting system for L2 and parking levels. Provisions of FEMA 273 [FEMA 1997], FEMA 351 [FEMA 2000], and OSHPD-approved design procedure [NYA&MI 2003] were used to evaluate the expected seismic performance of the structure in the existing configuration subjected to a 475-year seismic event. A comprehensive three-dimensional mathematical model of the structure was prepared and subjected to acceleration histories generated to match the site-specific response spectrum. Response was evaluated using nonlinear analysis. Structural performance did not meet the design requirements for story drifts and nonlinear flexural rotations. To enhance the building response, the analytical model was modified by adding fluid viscous dampers attached to new exterior frames. Analysis of the structure in the upgraded configuration indicated that the performance requirements were satisfied. Parametric studies were conducted to investigate the effects of variation in key parameters, such as damper coefficients, nonlinear hinge properties and soil-structure interaction, on the response of the retrofitted structure.

INTRODUCTION

OVERVIEW This paper summarizes the analytical studies conducted to evaluate the seismic performance of a multi-story hospital building located in Southern California. For analysis and evaluation, provisions of the 2001 edition of California Building Code [CBC 2001] and FEMA 273 [FEMA 1997], and FEMA 351 [FEMA 2000] were utilized. Project-specific design guidelines [NYA & MI 2003] were constructed in accordance with the OSHPDS'specific requirements of the project. Analysis of the structure indicated that the story drifts and flexural hinge rotations exceeded FEMA 1997 limits and design guidelines. To mitigate these problems, fluid viscous dampers (FVDs) were sized and added to the analytical model. Nonlinear acceleration history analyses indicated that the upgraded structure complied with the design guideline limits for story drift and member nonlinear flexural rotations.

DESCRIPTION OF THE BUILDING The Hospital building is a nine-story building constructed over four levels of underground parking located in Southern California. The underground footprint exceeds that of the hospital building and supports an additional multi-story office structure. Only the seismic performance of the hospital building is addressed in this paper. The uppermost eight stories of the superstructure (levels L3 through roof) use steel framed construction, whereas reinforced concrete framing is used for the lowest story level (level L2). The building is rectangular and has plan dimensions of 21 m (70 ft) wide (in y- or NS direction) by 63 m (210 ft) long (in x- or EW direction). Figure 1 presents a photograph of the structure and typical floor plan.

a. Photograph of building



b. Typical floor plan

Figure 1. The hospital building.

Typical flooring consists of 80-mm (3-in.) lightweight concrete slab supported by gravity steel beams, and in turn are supported by the main steel girders spaced at 10.5 m (35 ft). Column sections are spliced 1 m (3 ft) above the fifth and seventh floors using either full penetration (FP) welding for both column flanges and column web, or bolted web connection. At level L2, the floor framing is a 115 mm (4.5 in.) thick concrete slab supported on 150 x 470 mm (6 x 18 in.) deep concrete floor joist in each direction. The joists span to 815-mm (32-in.) deep concrete girders supported on 600 to 690-mm (24 to 28-in.) square concrete columns. The upper three levels of parking are partially underground and the lowest level (P4) is completely covered. The typical floor framing of the parking levels consist of a 115-mm (4.5-in.) thick concrete waffle slab and 350-mm (14 in.) wide concrete girders. Reinforced concrete columns and a 250-mm (10-in.) perimeter wall support these floors.

The lateral load resisting system for the upper eight stories (L3 through roof) of the superstructure consists of a steel moment-resisting frame along all the perimeter and girder lines of the building. The steel moment frame connections in both the strong and weak-axis use Pre-Northridge (PN) details. The panel zones for the beam-to-column moment connections are unreinforced—no doubler plates were used. The lateral load resisting system for the first story level of the superstructure (L2) consists of a concrete moment-resisting frame along the perimeter and all girder lines of the floor. For columns, longitudinal reinforcement was either #36 (#11) or #44 (#14), and transverse reinforcement was #13 ties (#4) spaced at 100 mm (4 in.). For beams, top and bottom main longitudinal reinforcement was #32 (#10) bars. Beam main reinforcements were hooked into the column. The reinforcement had adequate development length and allowed beams and columns to develop their full moment capacity.

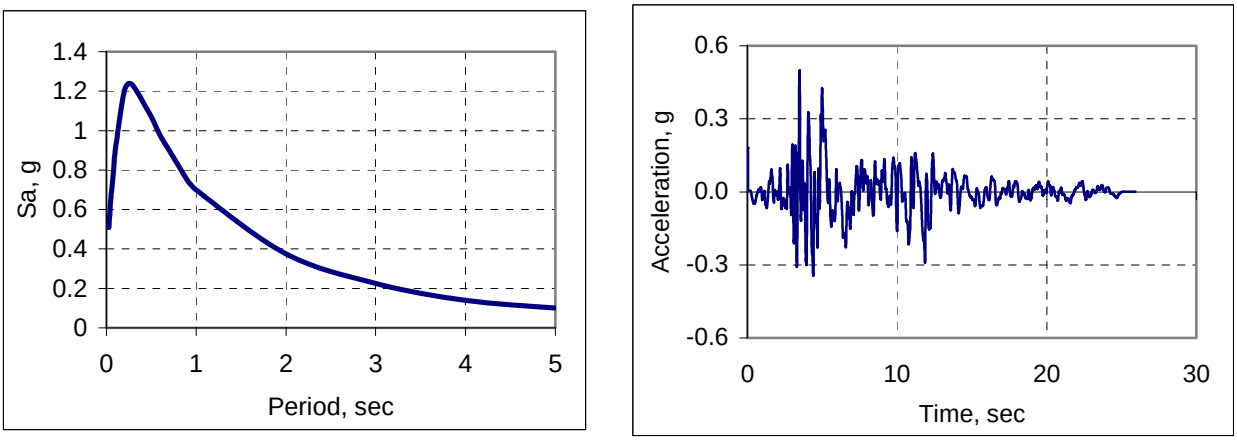
SEISMIC DEMAND The Design Basis Earthquake (DBE) was used for design and evaluation. GeoPentech [GeoPentech 2003-1] synthesized site-specific response spectra and seven pairs of spectrum-compatible acceleration histories were developed and peer-reviewed. The input histories had a typical duration of 30 to 40 seconds. To speedup analysis time without the loss of accuracy, the original records were shortened to have duration of ten to fifteen seconds for design process. The reduction in duration was done in such a way that all critical acceleration pulses were retained. The effect of conducting analyses with these shorter-duration acceleration histories is that the residual force and displacements in members might not be correctly estimated. The critical parameters for design process are the maximum responses in members. These maximum responses will be accurately captured with the shorter duration records. A full-length acceleration history was used at the end of the design process to verify the results. Table 1 lists

the site-specific spectrum-compatible suite of acceleration records used in analysis; Figure 2 presents the site-specific spectrum and trace of a typical record.

Table 1. Acceleration records used in analysis

Analysis	Record	Δt, sec	Time interval[2], sec	x-component [1]	y-component [1]
HIST01	Imperial Valley 1979	0.005	0-10	IV-FN	IV-FP
HIST02				IV-FP	IV-FN
HIST03	Sylmar 1994	0.02	0-12	FN-syl032	FP-syl032
HIST04				FP-syl032	FN-syl032
HIST05	Newhall 1994	0.02	0-12	FN-nwh032	FP-nwh032
HIST06				FP-nwh032	FN-nwh032
HIST07	Duzcee 1999	0.005	2-14	FN-dzc265	FP-dzc265
HIST08				FP-dzc265	FN-dzc265
HIST09	Kobe 1995	0.01	0-10	FN-taz320	FP-taz320
HIST10				FP-taz320	FN-taz320
HIST11	Superstitions Hill 1987	0.01	5-20	FN-b-pts217	FP-b-pts217
HIST12				FP-b-pts217	FN-b-pts217
HIST13	Imperial Valley 1979	0.005	2-12	FN-h-emo270	FP-h-emo000
HIST14				FP-h-emo000	FN-h-emo270

1. FN = Fault Normal and FP = Fault parallel
2. Time window from original record.



a. Site-specific response spectrum b. Acceleration history

Figure 2. Response spectrum and typical record

PERFORMANCE OBJECTIVE. The performance level for this building is the Life Safety (LS) performance level for a 475-year return seismic event. To reach this performance level, adequate supplemental damping (in the form of Fluid Viscous Dampers) was added to the existing nine-story superstructure to ensure that the existing steel and concrete moment-resisting frames would remain *essentially* elastic when subjected to site-specific input histories. The two main design objectives were to:

- Limit story drift ratios to less than 1.0 percent, and
- Limit the extent of flexural rotations in nonlinear elements as shown in Table 2.

For steel beams, member nonlinear flexural rotations were limited. This low limit was selected in order to prevent the type of non-ductile failure observed in the Northridge earthquake [SAC 1997]. Note that the limits of Table 2 are more conservative that those of FEMA 273 [FEMA 1997]and FEMA 351 [FEMA 2000], because a more restrictive design guideline was selected by the design team [NYA&MI 2003] for this hospital building that is located in a region of high seismicity in California.

Table 2. Acceptable performance limits for flexural hinge demands

	Allowable PH rotation, percent radian	
Component	*Design guideline*	*FEMA 273/351*
Steel beam	0.5	1.0
Steel column	0.6	1.0
Concrete beam	0.5	0.5-1.0
Concrete column	0.5	0.5-1.0

ANALYSIS PROGRAM

To evaluate the seismic performance of the upgraded structure, a three-dimensional mathematical model of the structure was prepared using program SAP2000 [CSI 2004]. The model incorporated geometric (P-Δ), material (flexural yielding), and FVD (axial force-axial displacement hysteresis) nonlinearity. A benchmark model was developed to simulate the response of the existing building and assess the efficacy of FVD implementation. This model was used to conduct full nonlinear time history analysis, check the response of existing members, and determine the extent of nonlinear response in existing members. Parametric studies were also undertaken to investigate effects of directionality of torsional response, FEMA 273 (FEMA 1997) gravity preloads, effect of lower garage levels, and definitional properties of nonlinear flexural hinges.

All existing steel members were specified as ASTM A36 steel. For the new damper frames, ASTM grade 50 A992 steel is specified. The as-built plans identified three types of concrete compressive strengths: 35 MPa (5 ksi) for columns, 28 MPa (4 ksi) for beams, and 21 MPa (3 ksi) for walls. Grade 40 reinforcement was specified for all members, except for column main reinforcement which used Grade 60 steel. Independent material testing of selected steel members [Twining 2004] has shown that the as-built material properties for these elements did not deviate substantially from the values shown in the plans and specifications. AISC manuals [AISC 1980 and AISC 2001] were used to determine the nominal beam and column member sizes for modeling. A majority of steel beams and columns satisfied the flange and web compactness requirements of AISC seismic provisions [AISC 2002]. All dimensions were specified as centerline-to-centerline. FEMA 351 [FEMA 2000] allows two approaches to model the panel zone: either the panel zone is explicitly model by specifying a rotational spring at the beam-to-column connection and beams are modeled to span to the face of the column, or panel zone flexibility is implicitly accounted for by modeling the beams to span to the centerline of the column. The latter approach was used in the analysis program described hereafter. At the roof level, some beam-to-column connections were modeled as pin connections, as indicated on the as-built plans.

For the baseline analysis, the columns extend one floor below the plaza level. In lieu of modeling the entire four-story garage structure below the superstructure, this simplified approach was used. Since the 250-mm perimeter shear walls provide lateral stiffness in these levels, the lateral stiffness in x- and y-directions were independently computed and modeled as linear spring. Similarly, the soil damping effect was modeled as linear dashpots at the lower levels. Soil-structure spring stiffness and dashpot damping properties were provided by GeoPentech [GeoPentech 2003-2]. Translational and rotational mass was placed at five percent eccentricity to the center of mass. Equivalent stiffness and damping for lower

parking levels was modeled as links at the center of mass of level P1 and Plaza. The accuracy of this approach was verified by analysis, in which all garage floors were explicitly included; see Figure 3.

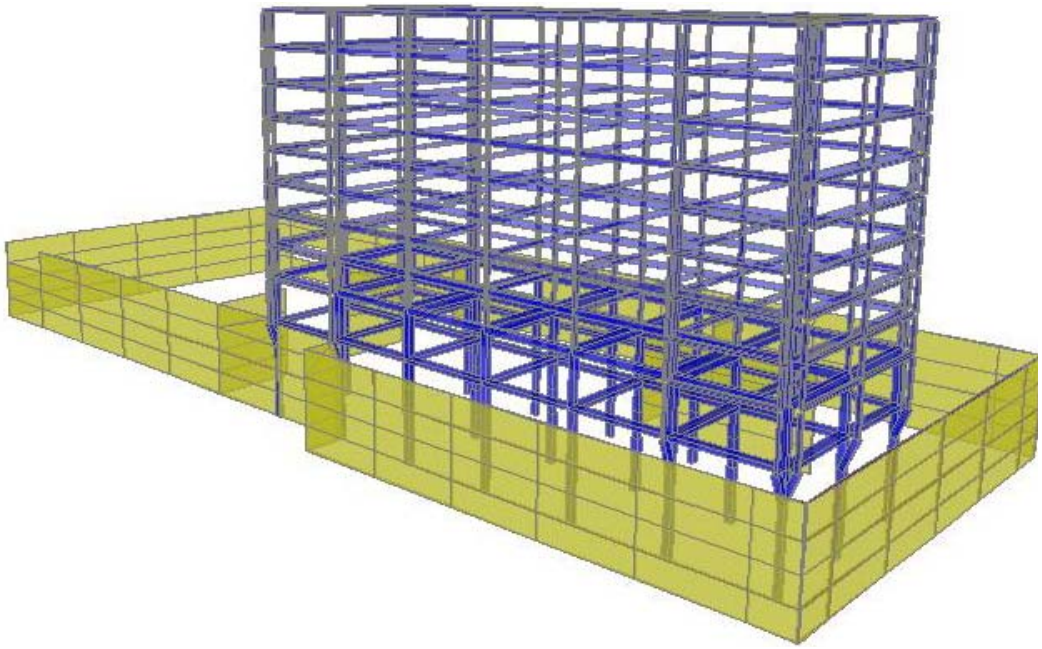


Figure 3. Mathematical model of the hospital building.

Program-default hinges were used, because plastic hinge rotations are limited to small amount for the upgraded structure. For verification, provisions of FEMA 273 [FEMA 1997] were used to separate a second model. The comparison of building responses validated that the program-default flexural hinges were adequate. Nonlinear flexural hinges were placed along the length of the member according to:

- At connection centerline for columns with weak panel zones (not expected to remain elastic)
- At the intersection of the beam flange to column flange for beams framing into column web
- At 1/3 depth of member from the centerline of support for all other members

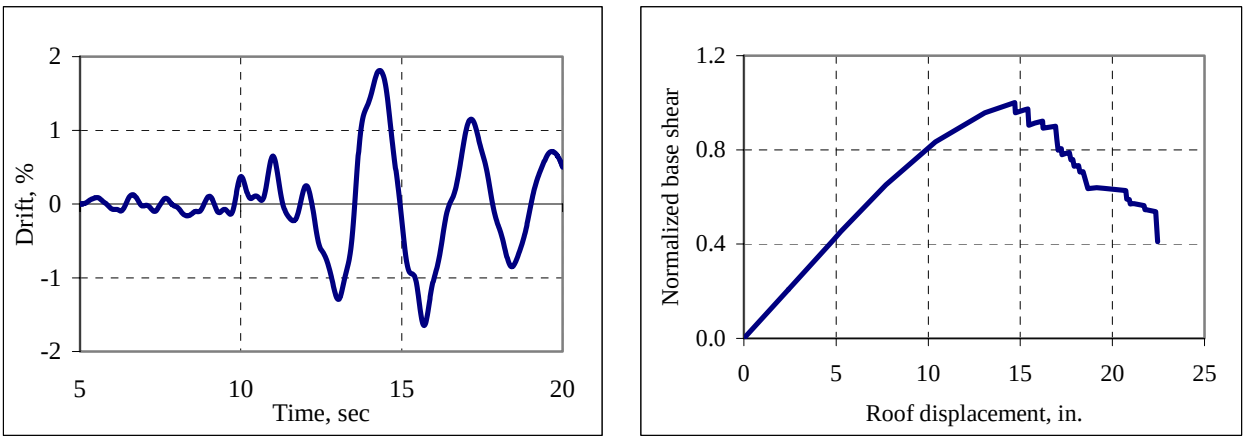
FEMA 273[FEMA 1997] gravity loads of 1.2D+0.25 L (un-reduced) and 0.9 D were used. The inertial mass for each floor was modeled as a mass placed at five-percent eccentricity from center of mass in the x- and y-directions. The directionality of this eccentricity was investigated. Both lateral and rotational mass were included in the model. The total mass of the building, including the basement, was estimated at slightly over 26,000 Mg (57,000 kips).

ANALYSIS RESULTS

Global and local responses were used in evaluation. Analysis was conducted for 7 pairs aligned at 0 and 90 degrees. Response output for all 14 histories were extracted and averaged to obtain the desired seismic demand. Program Matlab [Mathwork 2001] was used to process the output data.

Response of the existing hospital building was evaluated using modal analysis, static nonlinear and linear and nonlinear dynamic procedures. The fundamental vibration periods of the existing building were approximately 2.8 and 2.5 sec in the x- and y-directions. Figure 4 presents the drift history response obtained from acceleration history analysis and the static pushover curve obtained from static nonlinear

analysis. The maximum story drifts were close to 2%. At this level of story drift, roof displacement is approximately 24 in. and as such, the structure has lost most of its lateral capacity. Nonlinear hinge rotations in steel beams and columns exceeded 2% radian, and thus, both story drifts and member nonlinear flexural hinge rotations exceeded the design guideline limits. In addition, potential for soft story response at the first superstructure level (L3) existed. *As such, the seismic response of the existing hospital building was unsatisfactory.*



a. Story drift b. Pushover curve

Figure 4. Response of the existing building

SEISMIC REHABILITATION PROGRAM

FVDs have been successfully utilized in upgrading the seismic performance of structures. This methodology is one of the recommended practices [FEMA 2000] and has been successfully implemented in both new construction [Miyamoto et. al, 2003-2] and in seismic rehabilitation [Miyamoto et. al. 2003-1] by the authors. The proposed seismic rehabilitation scheme is to construct FVD frames on the exterior of the building. FVD frames would be detailed to provide fixity at the beam-column joints for these exterior frames. The FVDs—sized to control the drift—will be placed along the diagonals. The beams in the FVD frames will be attached to the existing perimeter beams by horizontal steel trusses that transfer the seismic forces from the existing diaphragm to the exterior frame. The basement concrete columns shall be increased in plan size for the first level to allow the exterior new FVD frame columns to bear on them. The FVD frame reaction will then be transferred to the existing column for the remaining levels of the parking structure. Figure 5 presents schematics of the building after rehabilitation.

Thirteen sizes of FVDs were used in analysis. To account for the manufacturing tolerances and variations in operating temperature the effective nominal damping coefficient (C) values were increased or decreased by 10-percent [Taylor 2004]. In one model, FVD damping coefficient was set to be ten percent *below* nominal. In a second simulation, FVD damping coefficient was set to be ten percent *above* nominal.

RESPONSE OF THE RETROFITTED BUILDING

The fundamental vibration periods of the retrofitted building were approximately 2.4 and 2.2 sec in the x- and y-directions. The retrofitted building is stiffer than the existing structure, since exterior moment frames were added to the building. Table 3 presents, the maximum roof displacement and base shear responses. These quantities were computed by obtaining the absolute maximum response of each fault-normal record and then averaging. The computed seismic base shear coefficient is approximately 5.5

percent of total weight. Typical roof displacement and base shear traces are depicted in Figure 6. Response of story drifts (Figures 4 and 6) show that the story drift is reduced by a factor of nearly two.

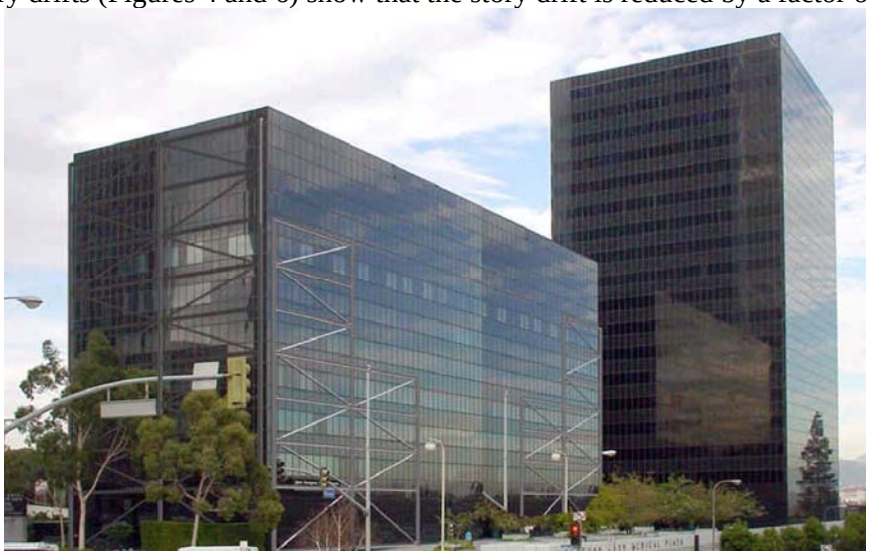
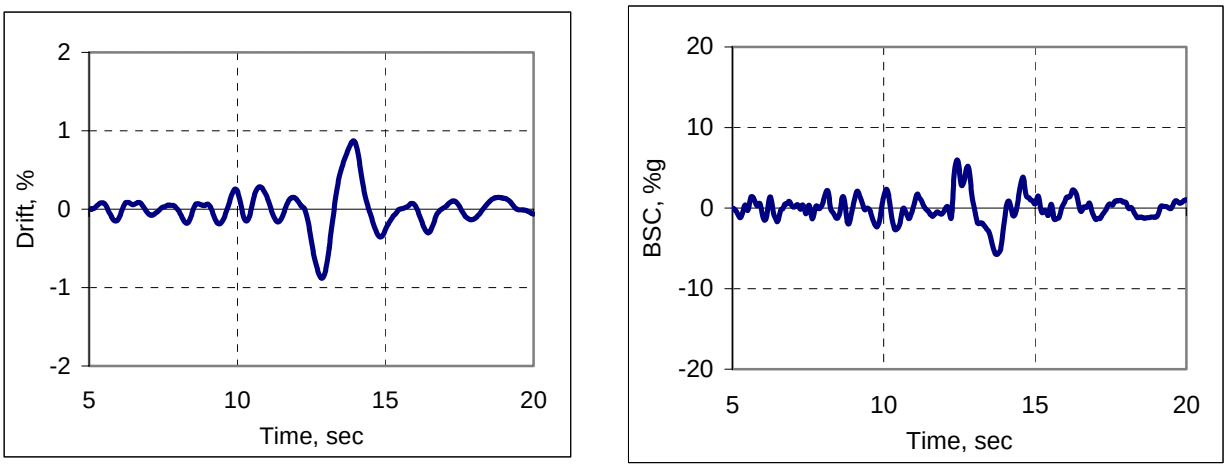


Figure 5. Arrangement of FVD frames on the exterior of the building

Table 3. Average maximum response of the retrofitted building

Quantity	*Roof displacement, mm (in.)*	*Base shear, MN (kips)*
x-direction	310 (12)	13.4 (3,000)
y-direction	270 (11)	14.3 (3,200)



a. Story drift b. Base shear coefficient

Figure 6. Response of the retrofitted building

Table 4 presents the story drifts in percent obtained from analysis. The entries in this table in each direction were computed by taking the average of the story drifts from the normal-fault components of motions. For example, the average drift in x- direction was computed by averaging values of records 1,2,3,5,7,11, and 13. Drift in the y-direction was computed by averaging drifts from records 2,4,6,7,10,12, and 14. For each record, the story drift response at level *n* was computed using the *exact* formulation. As shown in Table 4, the computed drift demands are below or slightly above 1% radian and as such the design satisfies the requirements for drift limits. The entries of this table correspond to the computed drift at the center of mass of the structure.

The maximum seismic demand (force, velocity, and deformation) on FVDs was evaluated. Figure 7 presents the axial-force-axial displacement hysteresis response for a typical FVD obtained from analysis. Also shown in the figure is the hysteretic energy dissipated by this damper. This energy was obtained by integrating the area under the force-displacement curve. Note that significant seismic energy is dissipated by a single damper. Similar results were noted for all dampers. In the absence of dampers, the seismic energy absorbed by FVDs would have to be dissipated by the yielding of steel frame members, resulting in unacceptable large flexural rotations in these members.

Table 4. Computed story drifts (% radian).

	Floor									
Direction	*Roof*	*Mech.*	*L8*	*L7*	*L6*	*L5*	*L4*	*L3*	*L2*	*Plaza*
x-	0.45	0.67	0.92	1.08	1.1	1.04	1.04	1.17	0.73	0.01
y-	0.36	0.59	0.84	0.94	1.01	0.97	0.92	1	0.75	0.02

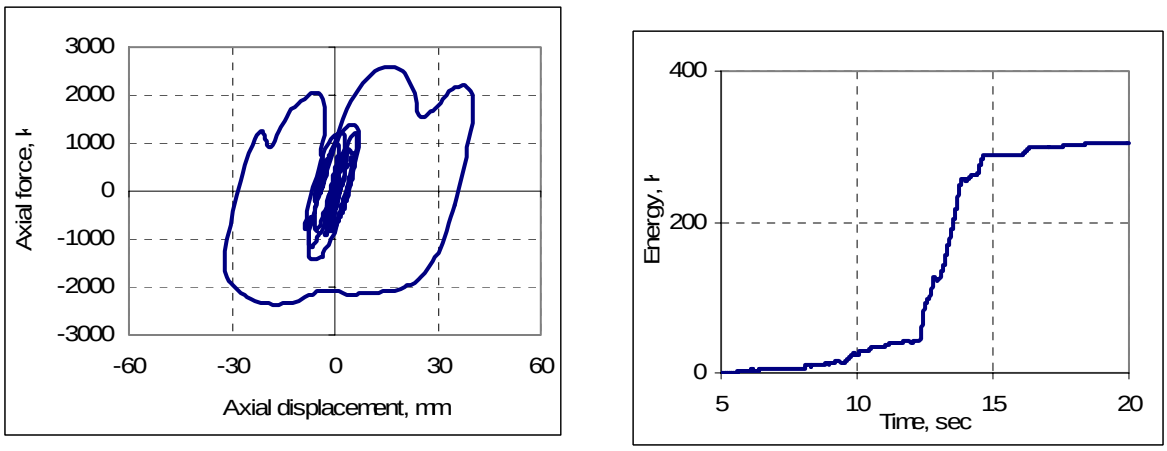
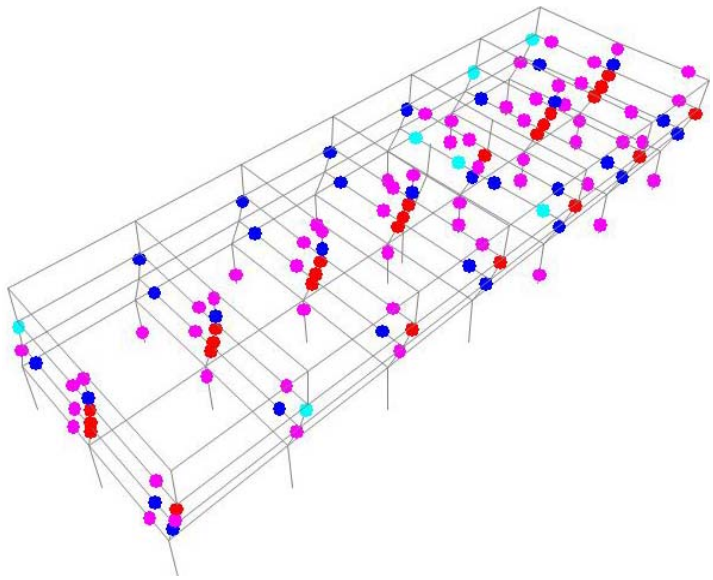


Figure 7. Hysteretic response of a typical FVD unit

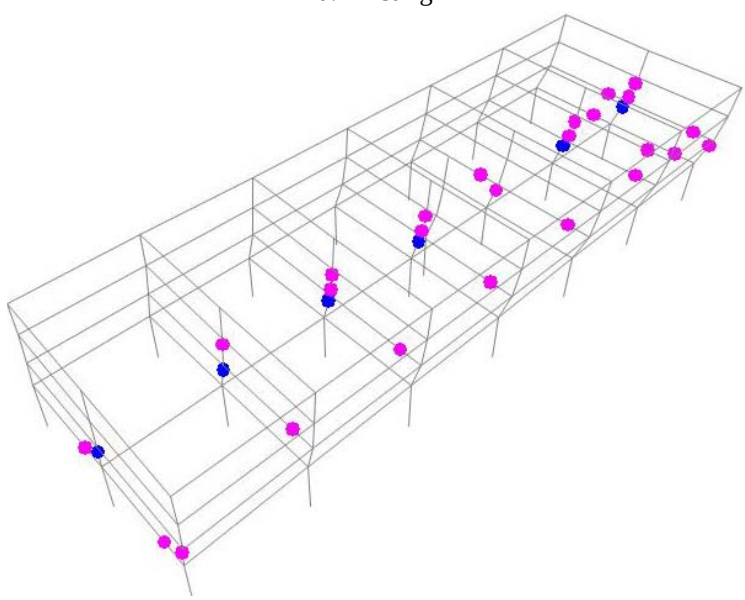
One of the primary objectives of analysis was to determine the pattern of plastic hinge formations and the level of plastic flexural rotations of the yielded members. Table 5 presents the maximum computed nonlinear flexural hinge rotation obtained from analysis and the limiting values set by the design guidelines. The retrofitted structure satisfies the design criteria requirements. Figure 7 presents the distribution of plastic rotations for the existing and retrofitted structures. For clarity, only the building elevation to level L5 is shown. Many hinges for in the existing building and a number of them exceed the life safety limits. In contrast, for the retrofitted building, fewer plastic hinges form and they were within the life safety limits. For the upgraded structure, steel columns that might yield, are *deformation-controlled* (P_u / P_c <= 0.5), [FEMA 1997]. As such, formation of nonlinear hinges in these members is allowed.

Table 5. Maximum of average flexural rotations (% radian) in various components

Component	*Plastic hinge rotations,% rad*	*Allowable rotation, % rad*
Steel beam	0.42	0.5
Steel column	0.11	0.6
Concrete beam	0.27	0.5
Concrete column	0.19	0.5



a. Existing



b. Retrofitted

Figure 7. Distribution of nonlinear hinges in members

SUMMARY AND CONCLUSIONS

Seismic response of a multi-story hospital building using unreinforced steel beam-to-column connections was evaluated by analysis. It was seen that the structure did not meet the Life Safety requirements of FEMA 1997. The structure was then modified by the addition of a limited number of FVDs. In the upgraded structure, the story drifts and member nonlinear rotational demands were below the target values specified in the design criteria. As such the upgraded structure meets the design criteria and is anticipated to perform well in seismic events.

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