Seismic Rehabilitation of a Nine-Story Hospital Building Using Fluid Viscous Dampers

Authors:

Amir S.J. Gilani, Miyamoto International, Inc., West Sacramento, CA,
agilani@miyamotointernational.com
H. Kit Miyamoto, Miyamoto International, Inc. West Sacramento, CA,
kit@miyamotointernational.com
Todd Kohagura, Miyamoto International, Inc., West Sacramento, CA,
tkohagura@miyamotointernational.com

ABSTRACT

Analytical investigation were undertaken to assess the seismic performance of a hospital building located in Southern California in seismic zone 4. The structure consists of a eight-story steel, and a bottom story reinforced concrete superstructure constructed over four levels of sub-grade parking stories. The building is rectangular, is 125 ft tall, and has total floor area of 130,000 ft^2. Steel and concrete moment-resisting frame along the grid lines of the building provide resistance to lateral loading. Project-specific design guidelines and FEMA, and SAC guidelines were used for evaluation. A comprehensive three-dimensional mathematical model of the structure was prepared. Nonlinear response history analysis of the existing building revealed that the performance was inadequate. In particular, story drifts and member nonlinear flexural rotations exceeded the limits specified in the design guidelines. The building rehabilitation consists of adding fluid viscous damper frames to the exterior faces of the building. The structure was then re-analyzed in the new configuration, and its performance was satisfactory.

OVERVIEW

Analytical studies were conducted to assess the seismic performance of a nine-story hospital building located in Southern California. Provisions of FEMA 273 [5], and FEMA 351 [6] were used to develop the mathematical model of the building and to interpret findings. In addition, project-specific design guidelines [11] were constructed in accordance with the OSHPD's specific seismic requirements for this project. Analysis of the existing hospital structure indicated that the building response was unsatisfactory and in particular story drifts were excessive and flexural hinge rotations exceeded the acceptable limits. To mitigate these problems, supplementary damping systems were selected as the retrofit strategy. Fluid Viscous Dampers (FVDs)—independently supported on new exterior frames—were added to the analytical model. The retrofitted structure was subjected to nonlinear response history analyses. The analyses of the rehabilitated structure indicated that the story drifts and level of member nonlinear

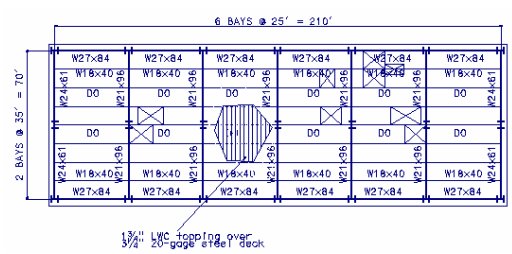
rotations were significantly reduced and that the upgraded structure complied with the design guideline limits for both story drift and member nonlinear flexural rotations limits.

DESCRIPTION OF THE BUILDING

The hospital is a nine-story building constructed over four levels of underground parking. 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 hospital (levels L3 through roof) use steel-framed construction, whereas reinforced concrete framing is used for the lowest story level of the hospital (level L2) and the parking floors. The building is approximately 120 ft tall above ground, is rectangular in plan, and measures 70 ft wide (in y- or NS direction) by 210 ft long (in x- or EW direction). Figure 1a presents a photograph of the existing structure.

Typical flooring for the superstructure consists of 1 ¾ in. lightweight concrete topping slab on 3 ¼ in. 20 gage steel decking. The floors are supported by W18x40 steel beams, in turn supported by W24 or W27 main steel girders, spaced at 35 ft on center, connected to W14 steel columns. Steel column are spliced 3 ft above the fifth and seventh floors using either full penetration welding for column flanges and web, or a bolted web connection. At level L2, 18-in. deep concrete joists span to 32-in. deep concrete girders supported on 24- or 28-in. Square concrete columns. The upper three levels of parking are partially exposed and the lowest level is underground. Concrete waffle slab and 14-in. wide concrete girders reinforced concrete columns and a 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 steel moment-resisting frames along the building perimeter and girder lines. Un-reinforced Pre-Northridge details were used for all steel moment resisting connections. The lateral load resisting system for the first story level of the superstructure (L2) is comprised of concrete moment-resisting frames. For concrete members, the reinforcement had adequate development length and allowed beams and columns to develop their nominal flexural capacity. Figure 1b depicts the structural drawings for level L5.

A. PHOTOGRAPH OF THE BUILDING



B. BUILDING FLOOR PLAN, LEVEL L5

FIGURE 1
THE HOSPITAL BUILDING

SEISMIC DEMAND

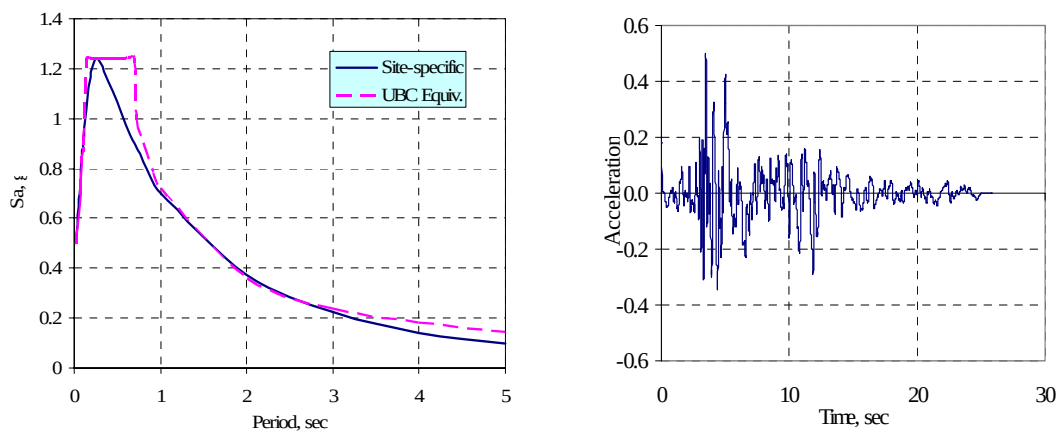
Seven pairs of acceleration histories were used for analysis and evaluation. The Design Basis Earthquake (DBE) was used for design and evaluation. GeoPentech [7] synthesized site-specific response spectra, and seven pairs of spectrum-compatible acceleration histories. The records were independently peer reviewed. Typical acceleration histories had total durations of 30 to 40 seconds and strong motion duration of up to 20 sec. Table 1 lists the site-specific spectrum-compatible suite of acceleration records used in analysis.

Analysis ID	Source record	x- & y-components *
HIST01 & 02	Imperial Valley 1979	IV-FN & FP
HIST03 & 04	Sylmar 1994	FN & FP-syl032
HIST05 & 06	Newhall 1994	FN & FP-nwh032
HIST07 & 08	Duzcee 1999	FN & FP-dzc265
HIST09 & 10	Kobe 1995	FN & FP-taz320
HIST11 & 12	Superstitions Hill 1987	FN & FP-b-pts217
HIST13 & 14	Imperial Valley 1979	FN & FP-h-emo270

* FN = Fault Normal and FP = Fault parallel

TABLE 1
ACCELERATION HISTORIES USED IN ANALYSIS

Figures 2a presents the site-specific acceleration spectrum. The response spectrum is anchored at a PGA of 0.5 g and has peak amplitude of 1.25g. For comparison, the standard code [14] spectrum is also depicted in this figure. The CBC spectrum with C_a=0.5 and C_v=0.72 closely resembles the site-specific spectrum. Figure 2b presents one of the fault-normal components of acceleration histories used in analysis.



A. SITE-SPECIFIC ACCELERATION SPECTRUM B. SPECTRUM-MATCHED ACCELERATION HISTORY

FIGURE 2
ACCELERATION RESPONSE SPECTRUM AND HISTORY

PERFORMANCE OBJECTIVE

The performance level for this building is Life Safety (LS) for a 475-year return (DBE) seismic event. The existing building did not meet this level of performance. Supplemental damping was added to the hospital building to upgrade its response to this level. The critical acceptance criteria for seismic rehabilitation were enumerated in the design guidelines as:

- Limit story drift ratios to less than 1.0 percent
- Limit the extent of flexural rotations in nonlinear elements to approximately 0.5% radian.

For steel beams, member nonlinear flexural rotations were limited to 0.5 percent radian. This low value was selected in order to prevent the type of non-ductile failure observed during the Northridge earthquake [12]. Note that the limits of Table 2 are more conservative that those of FEMA 273 [5] and FEMA 351 [6], because a more restrictive design guideline was selected by the design team [11] for this structure.

	Allowable PH rotation, % 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

TABLE 2
ACCEPTABLE PERFORMANCE LIMITS FOR FLEXURAL HINGES

MATHEMATICAL MODELING OF THE BUILDING

To evaluate the seismic performance of the existing and rehabilitated structures, three-dimensional mathematical models were prepared using program SAP2000 [4]. The models incorporated geometric (P-Δ), material (member flexural yielding), and FVD (axial force-axial displacement hysteresis) nonlinearity. A benchmark model was developed for evaluation. This model was used to conduct a complete suite of nonlinear response history analysis. The results were used to investigate the response of the building, and to determine the extent of nonlinear response in members. Additional analytical models were then prepared for the purpose of conducting Parametric (sensitivity) studies. The varaibles included in these studies included variations in: the placement of quadrant of center of mass, gravity preloads, flexural hinge properties, and modeling of the parking levels.

All existing steel members used ASTM Grade A36 steel. For the new damper frames, ASTM A992 steel was specified. Three types of concrete compressive strengths were identified in the structural plans: 5 ksi for columns, 4 ksi for beams, and 3 ksi for walls and floor slabs. Independent material testing of selected steel members was conducted [13] and indicated that the as-built material properties closely matched the values shown in the plans and specifications. Steel member sizes were specified per AISC manuals [1] and [2]. Most steel beam and column sizes satisfied the flange and web compactness requirements of AISC seismic provisions [3]. FEMA 351 [6] permits panel zones to be modeled using one of the following approaches. The panel zone is explicitly included in the model by specifying a rotational spring at the beam-to-column connection. In this

approach, steel and beams extend to the face of the columns. Alternatively, the panel zone flexibility is implicitly accounted for by modeling the beams to extend to the centerline of the columns. This latter approach was used in analysis.

For the baseline analysis, the building columns extend one floor below the plaza level. This approach was used in lieu of modeling the entire four-story garage structure. The lateral stiffness of the 10-in perimeter walls in these levels, were computed and modeled as linear springs attached to structural nodes. Similarly, the soil damping effect was modeled as linear dashpots. Soil-structure spring stiffness and dashpot damping properties were provided by GeoPentech [8]. The accuracy of this approach was verified as part of parametric studies. Figure 3 depicts the mathematical model of the building.

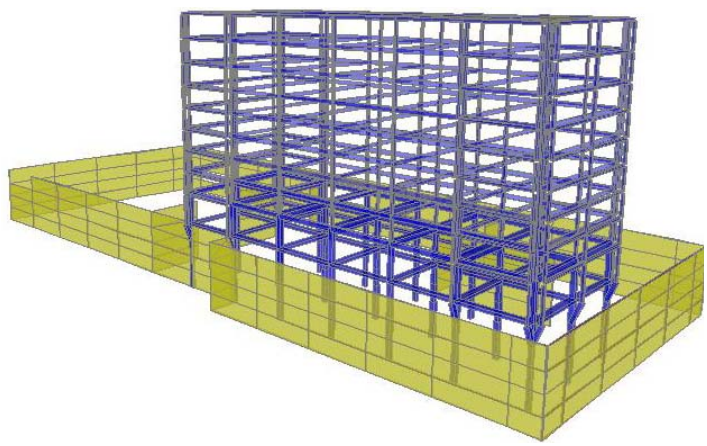


FIGURE 3
MATHEMATICAL MODEL OF THE HOSPITAL BUILDING

To account for the code-mandated accidental torsion, the center of mass was placed at five percent eccentricity at each level. At each floor, translational and rotational masses were computed and placed at this center of mass. The total mass of the building, including the basement, was estimated at slightly over 57,000 kips. Multi-linear axial force-flexural hinges were used to represent concentrated nonlinearity of frame members. To verify the accuracy of such modeling, plastic hinge for individual members were computed using the recommendations of FEMA 273 [5], as part of sensitivity analyses. The two analyses yielded similar

Nonlinear flexural hinges were placed along the length of the members at the:

- Connection centerline for columns with weak panel zones
- Intersection of the beam flange to column flange for beams framing into column web
- 1/3 depth of member from the centerline of support for all other members

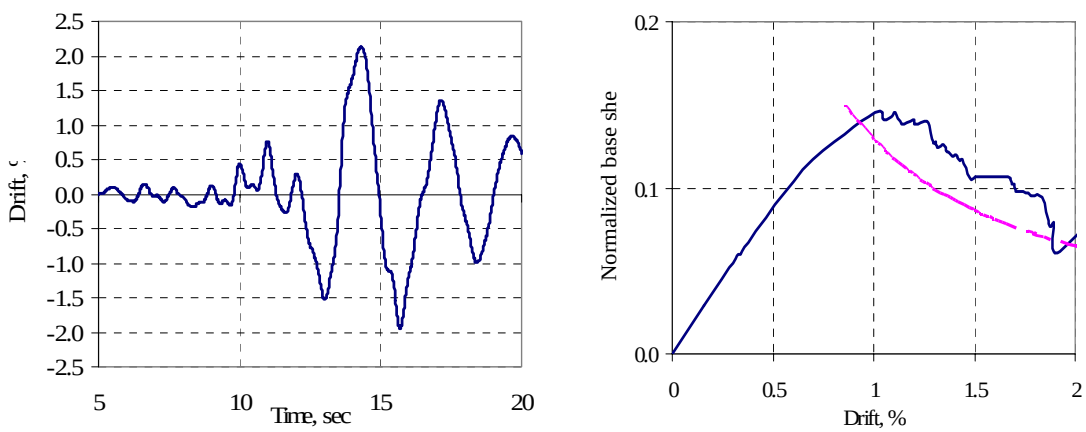
For nonlinear static and dynamic analyses, the structure was initially loaded with the gravity load (either1.2D+0.25 L or 0.9 D) prior to application of seismic forces.

SEISMIC EVALUATION OF THE EXISTING STRUCTURE

Modal analysis, static nonlinear, and dynamic linear procedures were used to assess the response of the existing structure. The fundamental periods of the existing building were

approximately 2.8 and 2.5 sec in the x- and y-directions, respectively. The building mode shapes were un-coupled and approximately 70 percent of the building mass participated in the first mode response in each translational direction. Figure 4a presents the story drift history response obtained from linear dynamic analysis. The maximum story drifts were approximately 2%. For this lomng-period structure, similar drift ratios are expecteded from dynamic nonlinear analysis of this building using equal displacement concept.

Staic nonlinear (pushover) analysis was conducted to determine the post-yield response of the structure. Conventional FEMA 273 [5] pushover analysis was adequate, because the building is symmteric (little coupling between torsional and translational response) and since the first modes in each direction govern response.In each direction, one load pattern proportional to the fundemental mode and one pattern proprotioanl to the deformed shape obtained from response spectrum analyses were included in pushover analysis. Figure 4b depicts the static pushover curve for the existing building. Note that at the anticipated 2% drift of Figure 4a, the lateral-load resisting capacity of the structure would be less than one-third of its nominal value and the building would be unstable. Nonlinear flexural rotations in steel beams and columns were approximately 2% radians. Unreinforced Pre-Northridge connections would not be able to sustain this level of plastic rotations. Since both story drifts and member nonlinear flexural hinge rotations exceeded the limiting values of Table 2, the seismic response of the existing hospital building was unsatisfactory.



A. STORY DRIFT RESPONSE

B. STATIC PUSHOVER CURVE

FIGURE 4
RESPONSE OF THE EXISTING BUILDING

SEISMIC REHABILITATION STRATEGY

FEMA 351 [6] lists several techniques for seismic rehabilitation of buildings exhibiting the type of inadequate response observed for this structure. Either the rotational capacity of the connections should be increased or the seismic demand must be reduced. Reinforcing connections would increase their rotational capacity. However, this is an intrusive and rather expensive repair. Alternatively, seismic demand can be reduced by adding supplementary damping to the building. This latter approach was used here.

The 40-percent demand spectrum for the building with the addition of the supplementary damping is also shown in Figure 4b. It is noted that the performance point—anticipated response point for the building during seismic event—has a drift of close to 1% and as such, it would have a satisfactory seismic performance.

Fluid Viscous dampers (FVDs) were selected to provide supplementary damping to the hospital building. FVDs provide a cost-effective, non-intrusice, and reliable method for upgrading the seismic performancre of existing structures. Since, the damper forces are primarily out-of-face with displacement response, they do not significantly place additional demand on existing columns. essfully utilized in upgrading the seismic performance of structures. This rehabilitation methodology is one of the recommended practices advocated by the SAC Joint Venture [6] and has been successfully implemented by the authors in both new construction [10] and in seismic rehabilitation of existing structures [9]. For the hospital building, FVD frames would be constructed on the exterior of the building. Figure 5 presents rendering of the building after rehabilitation. The exterior FVD frames were detailed to provide rotational fixity at the beam-column joints. The FVDs were placed along the diagonals. The beams in these exterior frames were attached to the existing perimeter beams by horizontal steel trusses that transfer the seismic forces from the existing diaphragm to the new exterior frame. The basement concrete columns at the first level would be increased in size to allow the exterior new exterior columns to bear on them. The exterior frame reaction would then be transferred to the existing column for the remaining levels of the parking structure.

FIGURE 5
ARRANGEMENT OF FVDS ON THE EXTERIOR OF THE BUILDING

To account for the manufacturing tolerances and variations of operating temperatures, a $\pm 10\%$ variation—consistent with the manufacturer data and past performance of in-service dampers—in the nominal effective damping coefficients of dampers was considered. The nominal damping coeffient of the FVDs was conservatively designed—to control story drifts and member nonlinear rotations—in the analysis were damping coefficient was set at ten percent *below* nominal. On the other hand, exterior frame member were conservativelyby choosing the largest damper force which is derived from analysis in which FVD damping coefficients were ten percent *above* nominal.

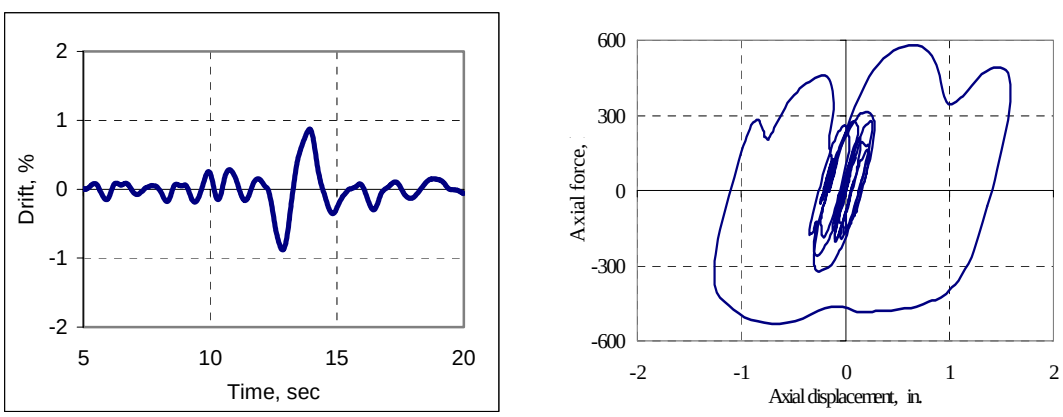
RESPONSE OF THE RETROFITTED BUILDING

The fundamental periods of the retrofitted building were approximately 2.4 and 2.2 sec in the x- and y-directions, respectively. The retrofitted building is stiffer than the existing structure, since exterior moment frames add lateral stiffness to the building. For the retrofitted structure, the averaged maximum story drifts were approximately 1.0%, and the base shear coefficient was close to 5.5 percent. Typical story drift response is depicted in Figure 6a. Comparison of story drifts of the existing (Figure 4a) and rehabilitated (Figure 6a) structures show that the story drift is reduced by a factor of nearly two after addition of FVDs. Table 3 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 records of Table 1. For each record, the story drift response at level *n* was computed using the *exact* formulation. As shown in Table 3, the computed drift demands are either below or slightly above the 1% limiting value. As such, the upgrade satisfies the design requirements for drift limits.

Floor	Roof	L9	L8	L7	L6	L5	L4	L3	L2	Plaza
x-	0.5	0.7	0.9	1.1	1.1	1.0	1.0	1.1	0.7	0
y-	0.4	0.6	0.8	0.9	1.0	1.0	0.9	1.0	0.7	0

TABLE 3
AVERAGED MAXIMUM STORY DRIFTS IN PERCENT

Figure 6b presents the axial-force-axial displacement hysteresis response for a typical damper. Significant seismic energy is dissipated by FVDs. In the absence of dampers, this seismic energy would have been dissipated by yielding of steel frame members, resulting in unacceptably large flexural rotations in these members.



A. STORY DRIFT

B. FVD ENERGY DISSIPATION

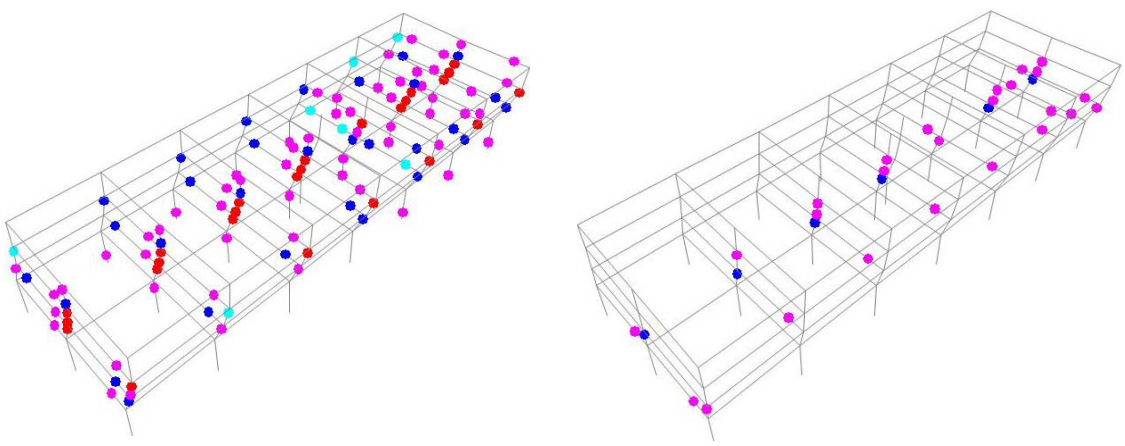
FIGURE 6
RESPONSE OF RETROFITTED STRUCTURE

One of the objectives of seismic upgrade was to limit the level of nonlinear flexural rotations in the members. Table 4 presents the maximum computed nonlinear flexural hinge rotation obtained from analysis. Also shown are the design guidelines' limiting values of Table 2. It is noted that in all members, the magnitude of nonlinear flexural rotations are below the acceptable values and as such, the retrofitted structure satisfies the design requirement for member nonlinear rotations.

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

TABLE 4
MAXIMUM AVERAGE FLEXURAL ROTATIONS (RETROFITTED STRUCTURE)

Figure 7 presents the distribution of nonlinear rotations for the existing and retrofitted structures, respectively. For clarity, only the building elevations to level L5 are shown. For the existing structure, many nonlinear hinges were formed and the level of rotations in a number of the members exceeded the limits of Table 2. By contrast, for the retrofitted building, fewer plastic hinges formed, and the rotation levels were below the values of Table 2. For the upgraded structure, only deformation-controlled ($P_u / P_c \leq 0.5$) steel columns experienced nonlinearity, acceptable per FEMA 273 [5] recommendations.



A. EXISTING STRUCTURE B. RETROFITTED STRUCTURE

FIGURE 7
DISTRIBUTION OF MEMBER PLASTIC HINGES

SUMMARY AND CONCLUSIONS

Seismic response of a multi-story hospital building was evaluated by analysis. From analyses reported herein, it is seen that:

- Performance-based analysis can be effectively used to assess the seismic response of steel moment-resisting frame structures and readily identify the building deficiencies and strategies to alleviate these deficiencies.
- The performance of the building was inadequate. The flexural demands on the conections exceeded tha design values and values obtained from previous experimental observations. This could potentially cause brittle failure of beam-to-column connections.
- Fluid viscous dampers provide a cost-effective, efficient, and non-intrusive method for rehabilitation The seismic demand is significantly reduced by increasing the total system damping.
- For the retrofitted building, the story drifts and member nonlinear rotational demands were below the target values specified in the design criteria. As such, the upgraded structure met the design criteria, and it is anticipated to perform well in seismic events.

REFERENCES

[1] AISC 1980, Manual of Steel Construction, Allowable Stress Design, Vol. 1, 9th Ed., American Institute of Steel Construction, Inc.

[2] AISC 2001, Manual of Steel Construction, Load & Resistance Factor Design, Vol. 1, 3rd Ed., American Institute of Steel Construction, Inc.

[3] AISC 2002, Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc.

[4] CSI 2004, SAP 2000, Three-dimensional static and dynamic finite element analysis and design of structures. Version 9, Computer and Structures Inc., Berkeley, CA.

[5] FEMA 1997, FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, D.C.

[6] FEMA 2000, FEMA 351: Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings, Federal Emergency Management Agency, Washington, D.C., 2000.

[7] Geopentech 2003-01, Transmittal of seven pairs of adjusted time histories. GeoPentech.

[8] Geopentech- 2003-02, Transmittal of spring and dashpot parameters. GeoPentech.

[9] Miyamoto 2003-1, Design of an Essential Facility with Steel Moment Frames and Viscous Dampers Using US NEHRP Procedure, Kit Miyamoto, Amir Gilani, Lon M. Determan, and Robert Hansen, proceedings of 13th World Conference on Earthquake Engineering, Vancouver, B.C.

[10] Miyamoto 2003-2, US Practice – Design of Structures with Dampers, Kit Miyamoto, Robert D. Hanson, Todd Kohagura, and Amir Gilani, proceedings of 13th World Conference on Earthquake Engineering, Vancouver, B.C.

[11] NYA & MI 2003, Personal Communications, Nabih Youssef & Associates, and Miyamoto International.

[12] SAC 1997, Connection test summaries. SAC 96-02 / FEMA 289 SAC Joint Venture.

[13] Twining 2004, Report on Testing/Inspection on Structural Steel Members, Twining Laboratories.

[14] Uniform Building Code, 1997 Edition, International Conference of Building Officials, Whittier, CA