Seismic retrofit of a hospital building with supplementary damping devices

Authors:

Kit Miyamoto, Miyamoto International, 1450 Halyard Drive, West Sacramento
kmiyamoto@miyamotointernational.com
Amir S.J. Gilani, Miyamoto International, 1450 Halyard Drive, West Sacramento
agilani@miyamotointernational.com
Robert S. Glasgow, Miyamoto International, 1450 Halyard Drive, West Sacramento
bglasgow@miyamotointernational.com

ABSTRACT

Performance-based earthquake engineering was utilized to ascertain the seismic performance of a group of hospital buildings located in the regions of high seismicity in Southern California. The structures were designed and constructed in early 1970's per the applicable building codes at the time of construction. Seismic evaluation of the structures, per current version of California Building Code [3] or FEMA's seismic evaluation procedures [1,2], indicated that buildings had severe structural deficiencies and would not survive the type of ground shaking anticipated at the sites. The design team developed an evaluation procedure and acceptance criteria for the buildings. The hospital complex consisted of a large four-story structure and two adjacent one-story buildings. Different framing systems were used to provide resistance to lateral loading for the three buildings. The four-story structure used Pre-Northridge connections and small wide flange column sections. It was supported on a one-story reinforced concrete basement. The owner initially investigated adding full-length cover plates to the majority of the columns. The retrofit cost and loss of functionality proved prohibitive. However, nonlinear response history analysis showed that when supplementary damper elements were strategically added to the building, the building response would markedly improve, and the story drifts and member flexural rotations were significantly reduced. For this structure, three performance objectives were developed. They included limiting: story drifts, steel member plastic rotations, and the lateral displacement of the first floor. One of the single story structures had masonry walls placed asymmetrically along the perimeter and showed large torsional response. The torsional response and story drifts were controlled by adding stiffness elements along the perimeter. The rehabilitated structures would meet the current seismic codes.

INTRODUCTION

Overview

Linear dynamic and nonlinear static procedures were used to assess the seismic performance of the four-story hospital facility in Southern, CA. This structure is the main building in a group of four units that consists of a tower, a four-story and two adjacent single story buildings. Design criteria were developed that limited the extent of member plastic hinge rotations, story drifts, and first story displacement. Analysis showed that the demand on the building was large. Large story drifts would require extensive ductile yielding of the steel members. However, the existing Pre-Northridge connections would not be able to provide these rotations. For seismic retrofit, it is proposed to add Fluid Viscous Dampers (FVDs) to the building. The analytical evaluation showed that only minor yielding would be expected when such retrofitted building is implemented and that the response would nearly be elastic.

Description of the building

The hospital structure is a four-story hospital located in a seismic active zone in Southern California. The 40-ft tall structure is comprise of steel framing above grade and is supported on concrete-framed basement.

Figures 1 and 2 present a photograph of the building and the typical floor framing. The building is 282 ft long and 75 ft wide. Steel Moment Resisting Frames (SMRFs) were used to resist lateral loading [6]. In the longitudinal direction, full-length SMRFs were provided along the perimeter only, whereas, in the transverse direction, all lines incorporated SMRFs. The beam-to-column connections for the SMRFs used typical 1970 approved details.

- Beam-to-column flange (strong direction) connection: Pre-Northridge non-ductile details are used, continuity plates were not provided. Panel zones would likely yield during a seismic event, and are classified as weak panel zones.
- Beam-to-column web (weak direction) connection: the connection is assumed not to transfer moment (pinned).

FIGURE 1

PHOTOGRAPH OF THE MEDICAL FACILITY

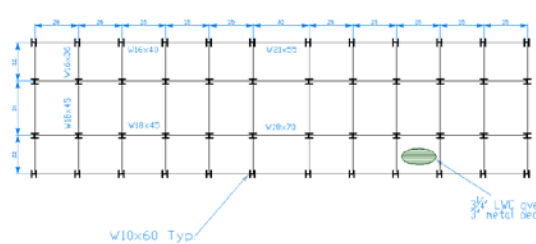


FIGURE 2

FLOOR FRAMING FOR THE HOSPITAL BUILDING

Performance objectives

The performance objectives for this structure are two fold: meet life safety (LS) requirements at the design basis earthquake (DBE), and meet collapse prevention (CP) requirements at the maximum credible event (MCE). The scope of this paper is limited to the first objective. To satisfy the seismic requirement for this essential facility, three design criteria were used. These limits are:

- Story drift ratios of less than 1%,
- Plastic hinge rotations of less than 0.5% radian in steel members. A limit more stringent than FEMA 356 [1] or FEMA 351 [2] requirements was chosen based on the building occupancy and limited ductility of existing connections,
- First floor displacement of 1.5 in. to ensures that the SRSS displacement [1 and 3] is less than the existing 2-in. wide seismic gap to prevent pounding.

Site seismic hazard

The building site is classified as Type D soil. The site short and 1-sec spectral accelerations equal 0.8 and 0.3g, respectively. The procedure of FEMA 356 [1] is used to construct DBE acceleration spectrum for the site. The target response spectrum and the 5%-damped acceleration spectrum used in analyses are shown in Figure 3.

Mathematical model of the building

Computer program ETABS [5] was used to prepare several mathematical models for the building. Only the lateral load-resisting members were modeled. Figure 4 presents isometric view of the model. The structure is regular and has symmetric distribution of mass and stiffness.

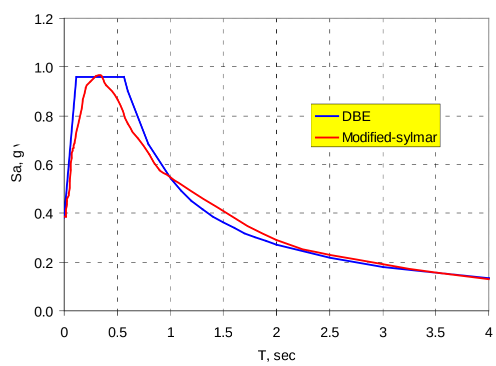


FIGURE 3
DBE SITE RESPONSE SPECTRUM (5% DAMPED)

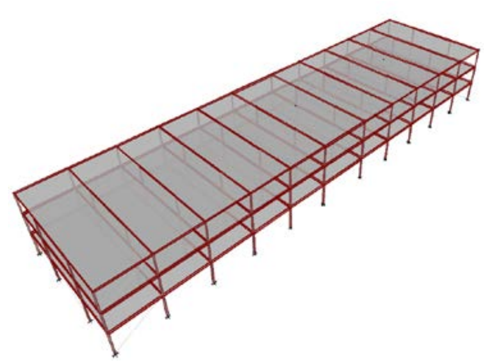


FIGURE 4
THREE-DIMENSIONAL MODEL OF THE BUILDING

Member centerline dimensions were used in analysis. The geometry of the building was obtained from the contract documents [6]. Nominal member sizes and material properties were used. Code-recommended [3] values for live load and additional non-structural dead load and seismic mass were applied to the building. P-Δ was included in analysis. The concrete flooring at second and third floors were modeled as rigid (concrete over metal deck). Rigid and semi-rigid diaphragms were used at the roof level. Since the columns were supported on grade beams and basement walls, the base boundary condition is between fully fixed and pinned. Two models were prepared to envelope the results. In one model the base of the columns were fixed; in the other model, pinned connections were provided.

ANALYSIS OF THE EXISTING STRUCTURE

Modal analysis

The three-dimensional models of the existing building were used to compute the dynamic properties (mode shapes and periods) of the buildings and to compute the story drifts for the building in the existing condition. The inertial weight of the building was estimated at 3,100 kips. Table 1 presents the dynamic properties of the first six modes. Examination of the entries of Table 1 shows the following.

- The building periods in the x- and y- directions are similar.
- No torsional coupling or coupling between lateral directions exists.
- The fundamental periods for the buildings are large enough to allow for the application of principle of equal displacement.
- The generalized mass for the first mode is approximately 80% of total mass.

	Fixed base				Pinned base			
	Period, sec	Participating mass, %			Period, sec	Participating mass, %		
Mode		x-	y-	θ-		x-	y-	θ-
1	1.28	0	82	0	1.94	96	0	0
2	1.25	81	0	0	1.93	0	96	0
3	1.23	0	0	85	1.80	0	0	96
4	0.55	12	0	0	0.64	3	0	0
5	0.54	0	11	0	0.62	0	3	0
6	0.51	0	0	10	0.58	0	0	3

TABLE 1
MODAL PROPERTIES OF THE BUILDING

Linear Dynamic Analysis

Table 2 presents the story drifts computed from dynamic response history analysis. Both column base articulations must be investigated since the pinned base connection governs response at the bottom floor and the fixed base case produces larger drifts for upper stories. For the pinned based model, the large drifts at the first floor are indicative of potentially undesirable soft-story response. The story drifts for the existing building exceed the recommendations of FEMA 351 [2]. These large drifts can only be produced if the steel members and connections can undergo large plastic rotations. Since the existing connections have limited ductility, they cannot sustain these large rotations. The extent of member nonlinearity required to obtain such large deformations will be investigated next.

	Fixed base				Pinned base			
	Displacement, in		Drift, % radian		Displacement, in		Drift, % radian	
Floor	x-	y-	x-	y-	x-	y-	x-	y-
4th	12.8	12.2	3.5	3.1	15.4	15.6	1.9	1.7
3rd	7.8	8.1	2.9	3.0	13.1	13.1	2.4	2.6
2nd	3.5	3.7	2.1	2.2	9.4	9.2	5.6	5.4

TABLE 2
STORY DISPLACEMENTS AND DRIFTS FROM TIME HISTORY ANALYSIS, EXISTING BUILDING

SEISMIC EVALUATION OF THE EXISTING STRUCTURE

The seismic response of the existing building was evaluated using nonlinear static (pushover) analyses. Since the building period is large, the principle of equal linear and nonlinear displacements is used and the pushover performance is directly correlated to the response obtained from dynamic response history analysis. Since the structure has uniform and symmetric mass and stiffness distributions, two-dimensional models in the longitudinal and transverse directions were used in evaluation. Since the two- and three-dimensional models have identical dynamic properties, they are dynamically equivalent and such would have similar seismic performances.

Flexural and flexural-axial plastic hinges, based on FEMA 356 [1] guidelines, were used for beams and columns, respectively. However, the Immediate Occupancy (IO) level for the beam hinges was selected at 0.5% radian, consistent with the design guidelines. Therefore, as long as the member hinges were at or below this level, the expected response would be satisfactory. The panel zones for this building are classified as weak as computed from AISC 1997 [4]. FEMA 351 [2] specifies two methods for modeling of weak panel zones: either the rotational flexibility of the panel zone must be modeled explicitly, or alternatively the beam plastic hinges has to be placed at the centerline of the beam-to-column connections. The latter approach was used here. The column hinges were also conservatively placed at the centerline of the joints, since no continuity plates were used.

The models were preloaded with FEMA 356 [1] gravity loading prior to incremental application of lateral loading whose profile was either proportional to the first modal response or proportional to mass at each floor. The control node was selected at the roof and the target displacement was chosen to equal the values of Table 2.

Figure 5 presents the normalized pushover curves. The roof displacement and base shear were normalized with respect to the building height and mass, respectively. Table 3 summarizes the results from the pushover analyses. Figure 6 shows the displaced shape of the frames at the target displacements of Table 2. Note the extensive yielding of columns, large plastic hinge rotations, and the soft-story response for pinned columns. As such, the response of the existing structure is not satisfactory.

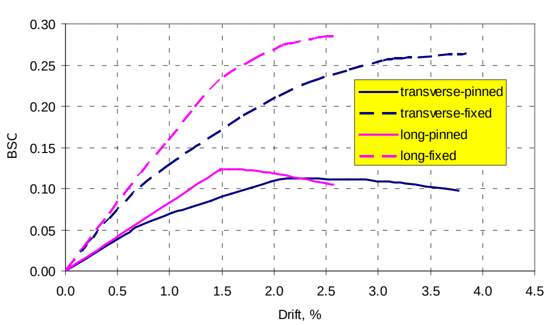
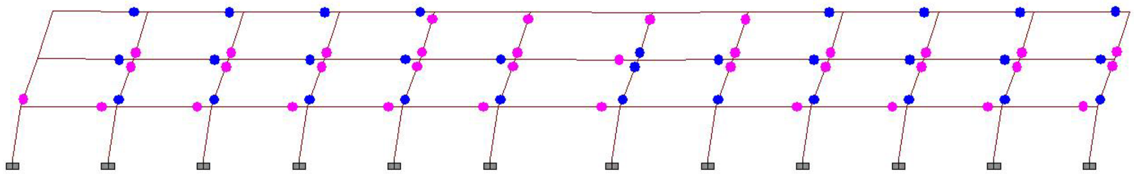


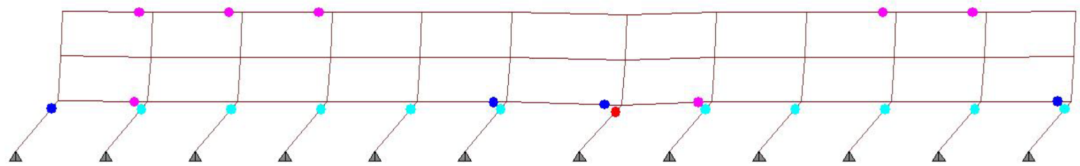
FIGURE 5
STATIC PUSHOVER CURVES

	Roof displacement, in.		
Analysis	1st beam hinge	Hinge past IO	1st column hinge
Trans., pinned	1.8	4.6	9.2
Trans., fixed	1.0	3.4	-
Long., pinned	1.2	5.6	7
Long., fixed	0.7	3.2	6.5

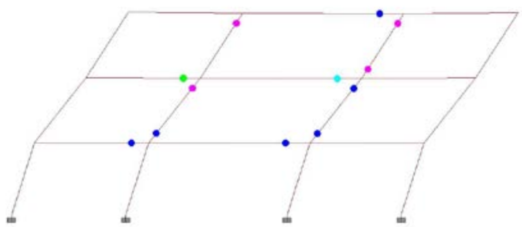
TABLE 3
STATIC PUSHOVER RESULTS



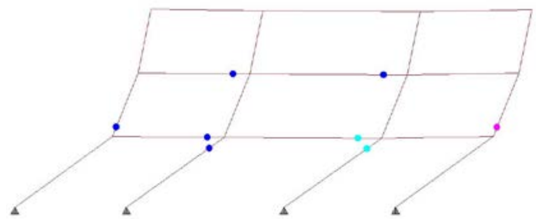
a. Longitudinal fixed base



b. Longitudinal pinned base



c. Transverse fixed base



d. Transverse pinned base

FIGURE 6
NONLINEAR DISPLACED SHAPE OF THE MOMENT-RESISTING FRAMES

SEISMIC RETROFIT

Overview

Static pushover analysis, see Table 3, showed that the seismic retrofit needs to limit the roof displacement to approximately 3 in. Conventional seismic retrofit would be expensive and would necessitate welding full-height flange plates to columns. Instead, supplementary damping devices can be used to reduce the seismic demand and produce a a nearly elastic response. This approach is one methodology recommended by FEMA 351 [2]. Fluid viscous dampers (FVDs) were selected as the retrofit choice. The authors [7 and 8] have previously used this methodology for new and retrofit construction. The choice of FVD as the damping device was based on the following.

- FVD force is primarily proportional to velocity and out-of-phase with elastic forces.
- They do not increase the building stiffness, nor attract more seismic force to the frames.
- Dampers are readily installed, do not reduce the available floor space significantly, and do not interfere with the building's architecture.
- FVDs are robust, low-maintenance, and cost-effective.

A supplementary damping of approximately 50% of critical would be required. Figures 7 and 8 present schematic of a typical FVD and the demand and capacity spectra curves, respectively. The demand curves are shown for the 5% of critical (existing) condition and with supplementary damping. Note that once FVDs are added, the story displacements and drifts would be significantly reduced.

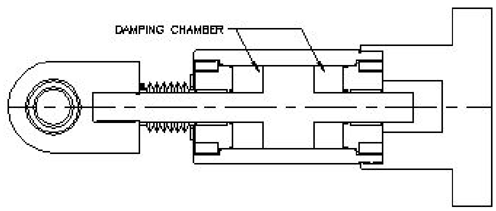


FIGURE 7
SCHEMATIC OF FVD

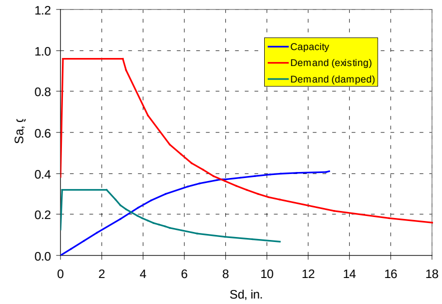


FIGURE 8
DEMAND AND CAPACITY SPECTRA

Verification studies

FVDs were added diagonally to the three-dimensional model of the building. Eight units were used at each floor. Figure 9 depicts the analytical model of the damped structure. Response (time) history analysis of the three-dimensional models was conducted. The story displacements and drifts are presented in Table 4. Note that story drifts are less than 1% radian; and that the first floor displacement, in the transverse direction, is not greater than 1.5 in. For reference, Figure 10 presents the roof displacement in the x-direction for the model with pinned column bases.

	Fixed base				Pinned base			
	Displacement, in		Drift, % radian		Displacement, in		Drift, % radian	
Floor	x-	y-	x-	y-	x-	y-	x-	y-
4th	2.1	1.7	0.2	0.2	2.6	2.2	0.2	0.1
3rd	1.8	1.5	0.6	0.5	2.3	2.0	0.5	0.4
2nd	0.9	0.8	0.6	0.5	1.5	1.3	0.9	0.8

TABLE 4
STORY DISPLACEMENTS AND DRIFTS FROM TIME HISTORY ANALYSIS; FVDS ADDED

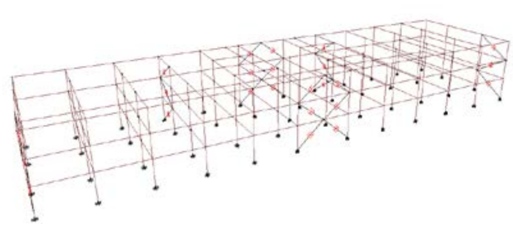


FIGURE 9
ISOMETRIC VIEW OF THE BUILDING WITH FVDS

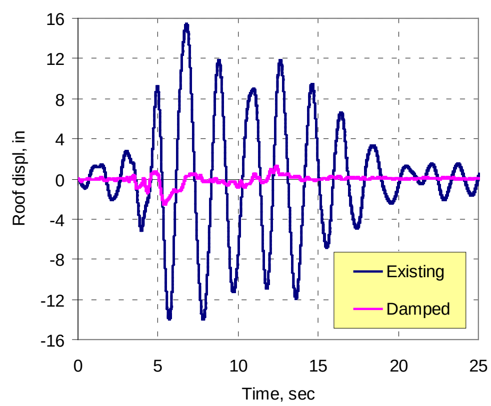
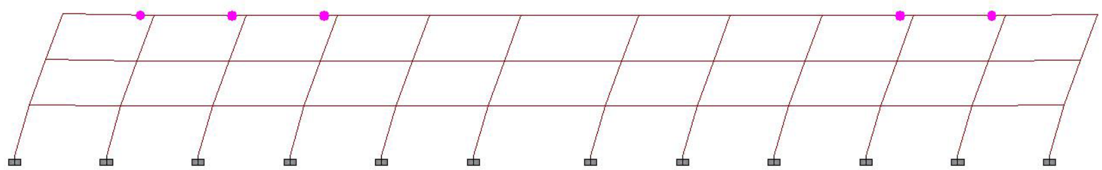
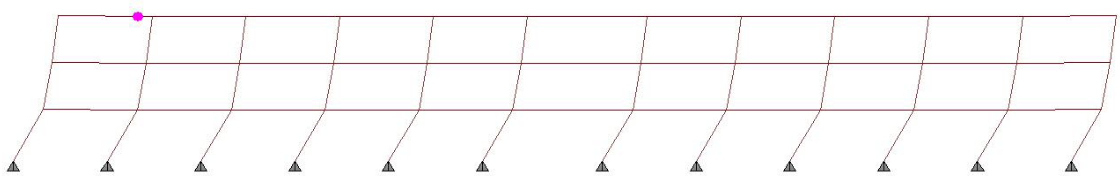


FIGURE 10
ROOF DISPLACEMENT IN X-DIRECTION (PINNED BASE)

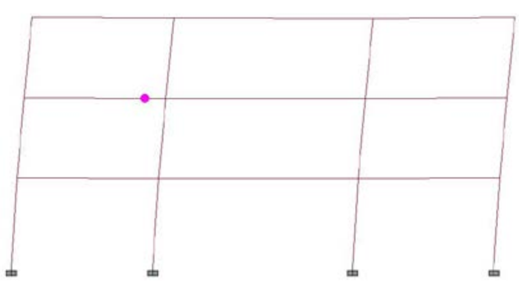
Figure 11 presents the nonlinear displaced shape of the retrofitted building at the target displacements of Table 4. For the retrofitted structure, only minimal beam yielding occurs at the target displacement and the response is essentially elastic. A comparison of Figures 6 and 11 shows that by adding FVDs the nonlinear seismic demand on steel members is significantly reduced.



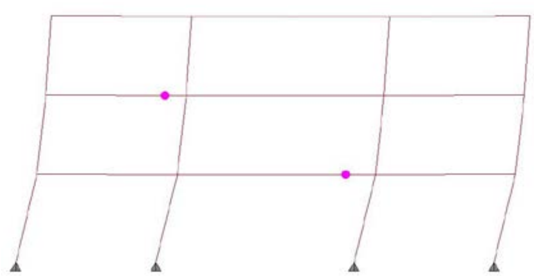
a. Longitudinal fixed base



a. Longitudinal pinned base



c. Transverse fixed base



d. Transverse pinned base

FIGURE 11
NONLINEAR DISPLACED SHAPE OF THE MOMENT-RESISTING FRAMES

Figure 12 presents the force-displacement hysteresis response of dampers. Note that significant energy is dissipated by the dampers. This energy dissipation increases the system equivalent viscous damping to a such level that story drifts are limited to 1%. In the absence of dampers, the non-ductile existing connections would be asked to dissipate this energy. Figure 13 presents the components of input seismic energy. FVDs dissipate most of the seismic input energy.

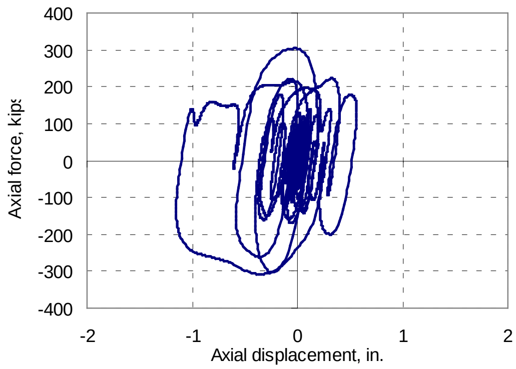


FIGURE 12
DAMPER FORCE-DISPLACEMENT HYSTERESIS

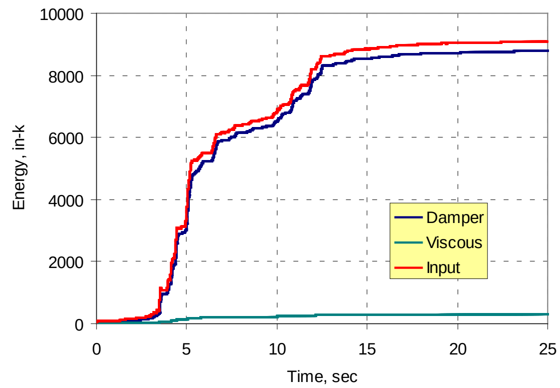


FIGURE 13
COMPONENTS OF SEISMIC ENERGY

Retrofit methodology

Figures 14 and 15 present the proposed details for connection of a FVD to the existing steel framing. To upgrade the seismic performance of the existing hospital building the following measures were proposed.

- Add FVDs to reduce: the story drifts to 1%, plastic hinge rotations to 0.5% radian, and SRSS first floor displacement to 2 in.
- Add steel plates and HSS sections at the floors to transfer the lateral components of the seismic forces to the damper bays.
- Add column cover plates at a few strategic beam-to-column connections.
- Minor retrofit of foundations supporting dampers.

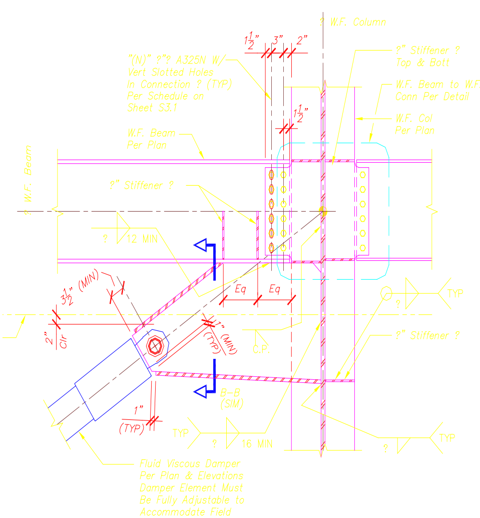


FIGURE 14
FVD CONNECTION DETAILS

FIGURE 15
PROPOSED FVD CONNECTION TO EXISTING FRAMING

SUMMARY AND CONCLUSIONS

Seismic evaluation of a four-story health facility indicated that the structure would undergo large member nonlinearity and story drifts. Since the building used Pre-Northridge connections that have limited ductility, the response was unacceptable. The owner considered both a conventional retrofit consisting of adding column full height cover plates and an innovative retrofit consisting of fluid viscous dampers. The dampers were chosen due to the large construction cost and time savings and because they minimized business interruption.

- FVDs provide a cost-effective retrofit technique that does not increase the seismic demand on buildings.
- The addition of dampers reduces the seismic demand on steel members to the level that the response of these members is essentially elastic. This is archived by dissipating the seismic energy.
- By limiting the story drifts to less than 1%, the non-ductile Pre-Northridge connections will be protected against unexpected brittle failure.

REFERENCES

[1] Federal Emergency Management Administration, “FEMA 356: Prestandard and commentary for the seismic rehabilitation of buildings,” Washington, D.C, 2000.

[2] Federal Emergency Management Administration, “FEMA 351: Steel moment frame buildings: evaluation and upgrade criteria for existing buildings,“ Washington, D.C., 2000.

[3] California Building Standards Commission, “CBC 2001: California Building Code, Volume 2,” Whittier, CA, 2001.

[4] American Institute of Steel Construction, “Manual of steel construction, Load and resistance factor design,” Chicago, IL, 1997.

[5] Computers and Structures, “ETABS: Extended three-dimensional analysis of building structures,” Berkeley, CA, 2006.

[6] Structural drawings, personal communications, 2006.

[7] Kohagura, T.T., Glasgow, B.S., and Miyamoto, H.K., “design of a four story educational building with steel moment frames and viscous dampers,” Proceeding of 2003 SEAOC Convention, Squaw Creek, CA, 2005

[8] Gilani, A.S., Miyamoto, K., and Kohagura, T, “Seismic Rehabilitation of a Nine-Story Hospital Building Using Fluid Viscous Dampers”, Proceeding of the 2006 SEI ASCE Structures Congress, St. Louis, MO, 2006.