Seismic Evaluation and Upgrade of Historic Multi-Story Buildings in Sacramento, California

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ABSTRACT

Structural performances of two historic high-rise buildings, constructed in the early 1920s and located in seismic zone 3 in downtown Sacramento, CA were investigated. The objectives of this evaluation were to determine whether these structures met the current seismic criteria and to propose strategies to upgrade such performance to an acceptable level. Both structures, have fourteen main floors, are 200 ft tall, have an approximate floor area of 90,000-ft^2. Performance-based procedures based on the Life Safety (LS) provisions of FEMA 356 for the Deign Basis earthquake (DBE). Detailed three-dimensional mathematical models of the structures were prepared. Both structures performed better than expected when subject to seismic loading. A voluntary seismic upgrade using fluid viscous dampers (FVDs) and steel struts was utilized for one structure, whereas, for the second structure, structural modifications were undertaken to address some of the occupancy needs.

INTRODUCTION

The concept of performance-based engineering was applied to two multi-story structures located in downtown Sacramento. These structures were similar in size and height; however, had different lateral-load resisting framing. One used semi-rigid steel moment frames and the other concrete columns and beams. Both buildings employed infill masonry panels. The are located in seismic zone 3 as defined by the California Building Code [4]. The main objective of this paper is to assess their seismic performance, provide a comparison of the anticipated performance, and investigate the efficacy of possible seismic upgrades.

DESCRIPTION OF BUILDINGS AND THEIR MATHEMATICAL MODELS

The historic Elks Lodge building has seventeen full floors and two mezzanines located above the second and third floor. The structure is currently used for both commercial and office occupancy. The footprint of the building is 160 x 100 ft for the first five floors and typical floor bays measure approximately 14.5 ft in each direction. Above fifth floor, the floor dimensions are reduced to 75 x 45 ft. Typical story heights vary from 10 to 14 ft. The first three floors have a height of 20 ft. 3-in. thick concrete slab and Steel Moment-Resisting Frames provide vertical and lateral load resistance. Heavy W14 column sections were used for the 14-story tall part of the building and smaller W14 columns

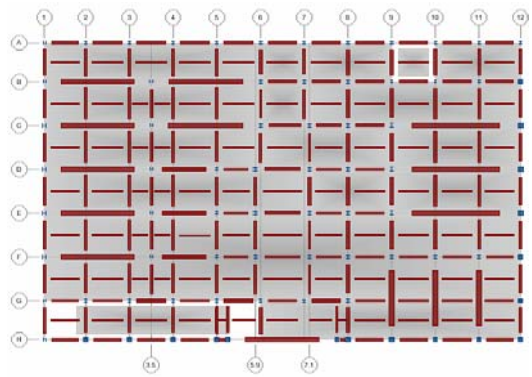
were utilized for the 5-story segment. Bethlehem or Carnegie steel channel or S sections were used for steel beams and had a typical depth of 10 to 18 in. To allow for the ballroom and dining area and the rooftop terrace on fourth floor, columns are omitted at some locations of the building between third and fifth floors. Large built-up steel beams, rooftop trusses, and plate girders were used to span between columns at these locations. To fireproof members, a number of members and connections were encased in either plaster or concrete. A 13-in. thick unreinforced masonry (URM) perimeter wall spans the entire width of the five-story segment of the building on one side in each direction. A 13-in thick concrete wall encases the perimeter of the building below the first floor. Riveted details were used for all steel beam-to-column connections. Wind bracing was provided for some of the connections; whereas simple clip angle riveted connections were used for the remainder. Typical riveted clip angle connections used. ¾ in. rivets and L4 x 6 x 5/8 in. angles for top and bottom flanges of beams. A photograph of the building is depicted in Figure 1a. Details of structural plan, elevation, and connections are shown in Figure 2.

The 17-story, 200-ft tall structure at the corner of 9th and J street built in 1922, was the first high rise building constructed in Sacramento, California. It has twelve full floors up to an attic and five smaller floors above, including a mechanical room and a water tank room. The footprint for the original construction was L-shaped, measuring 80 x 120 ft and had a total area of approximately 70,000 SF. Two later additions to the buildings were a 7-story, 23,000 SF annex in 1932, and a 2-story L-shaped annex in 1950. In Analyses described hereafter, the annexes were modeled to contribute mass but not stiffness to the main structure. The gravity load-resisting system consists of 5- and 6-in. thick reinforced concrete slabs supported on reinforced concrete beams and columns. Reinforced concrete moment frames resist lateral loading. Reinforcement for concrete beams and columns had adequate splice to ensure yielding, and no shear failure of members or joints was anticipated. Several full-bay URM infill walls extend up to the fourth floor on two perpendicular faces of the building. Figure 2 presents a photograph of the building, whereas, plan and elevation structural details are shown in Figure 3. Table 1 summarizes the pertienet geometric informations for the buildings.

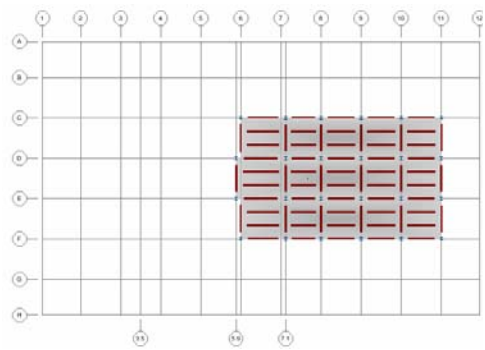
A. ELKS BUILDING

B. 926 J STREET BUILDING

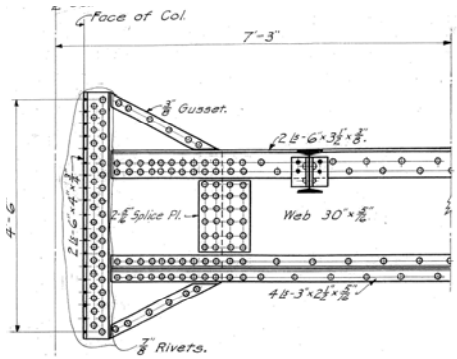
FIGURE 1
BUILDINGS UNDER CONSIDERATION



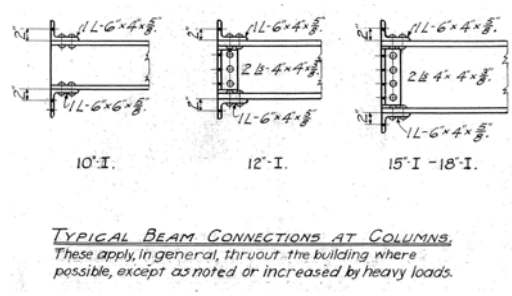
A. LOWER LEVEL TYPICAL PLAN (2ND FLOOR)



B. UPPER LEVEL TYPICAL PLAN (7TH FLOOR)

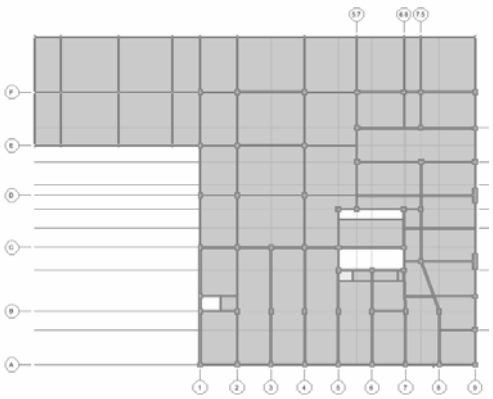


C. WIND BRACING (TYPICAL)

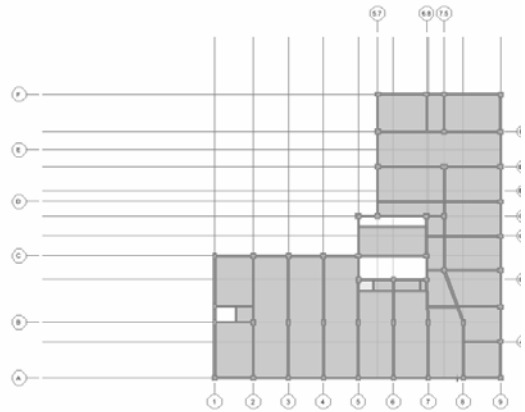


D. SEMI RIGID CONNECTION (TYPICAL)

FIGURE 2
DETAILS FOR THE ELKS BUILDING



A. FLOOR PLAN AT 2ND FLOOR



B. FLOOR PLAN AT 8TH FLOOR

FIGURE 3
FLOOR PLANS FOR THE 926 J STREET BUILDING

	Elks Building	926-J Building
Stories	17	17
Height ft	225	200
Plan shape	Rectangular	L-shaped
Lower floors, ft	160x100	80x120
Upper floors, ft	75x45	
Area, ft^2	90,000	70,000
Lateral load system	S-MRF	RC-MRF
Year constructed	1926	1922
Inertial weight, kips	20,000	22,000

TABLE 1
PROPERTIES OF THE TWO BUILDINGS

MATHEMATICAL MODELS OF THE STRCUTURES

The computer program ETABS [6] was used to prepare mathematical models of the three buildings. The recommendations of FEMA 356 were followed to prepare comprehensive three-dimensional mathematical models of the stcrutues. All members contributing mass or stiffness to the structures were accounted for. The cross sectional dimensions were obtained from the first edition of the AISC Manual of Steel Construction [1]. For steel members that were encased in structural concrete, equivalent, transformed sectional properties were calculated. Similarly, for built-up sections, equivalent wide flange sections were prepared. Nominal dimensions were used for concrete beams and columns, using a reduced flexural stiffness to account for cracking. Member self-weight and additional dead and live load were applied using code recommendations. The code-mandated 5% eccentricity of mass was included in the model. P-Δ and material nonlinearity was included in the models. Independent material evaluations for structures were performed by taking coupon and core samples following FEMA 356 recommendations. When field data was not readily avialble, the recommended [3] lower bound strength values for concrete, reinforcement, strcutural steel, and rivets were used.

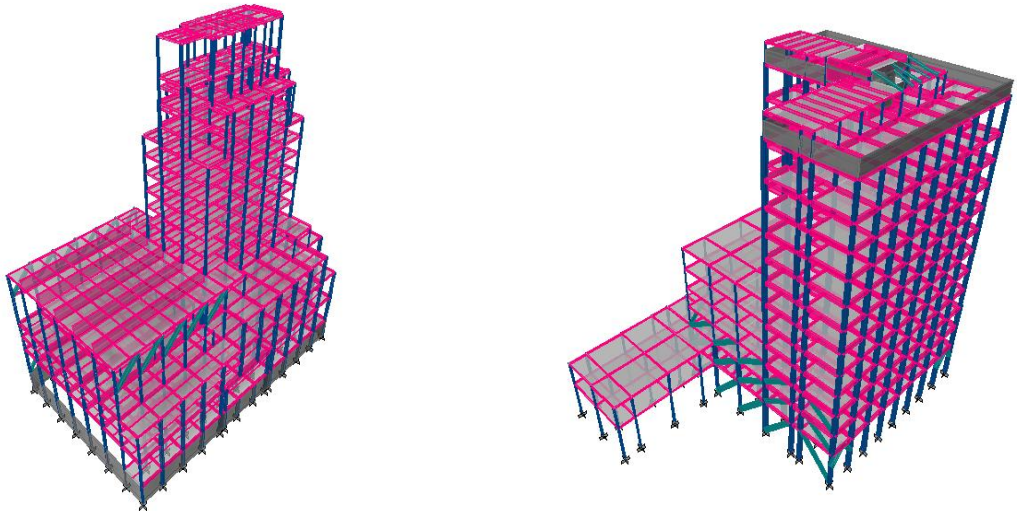
URM infill panels presented an important part of lateral-load resisting system. To accurately model these members, equivalent compression struts were used in analysis. The width of the equivalent compression struts was computed using provisions of FEMA 356 [3] and the procedure presented by Reinhorn et al [12]. The width of compression struts depends on geometry and stiffness of infill panel and supporting frame. Rather than specifying an intersecting pair of diagonal compression-only struts per bay, a single diagonal tension-compression strut was developed to model the infill panel. The approximate width of compression struts were 20 to 30 in. for the buildings. The perforated (partial-width) infill (bays with infill and window openings) were not included in the model, since these members would have smaller stiffness and strength properties compared to full URM bays; see Bennet et al [2]. Figure 5 presents a graphical representation of an equivalent compression strut.

For steel beam-to-column connections, two types of end conditions were considered. When wind bracing (see Fig. 2c) was utilized, the connection was modeled as FR. However, at the connections where beams were attached to the column flanges using top and bottom clip angle riveted connections (see Fig. 2d), the connection was modeled as Partially Rigid (PR). For PR connections, rotational springs were placed at the beam-to-column connection to represent the flexibility of the connection. The rotational rigidities

were computed from FEMA 356 [3] equation: $K_\theta = \frac{M_{CE}}{0.005}$. For reinforced concrete members, calculations demonstrated that the shear capacity of beams and columns exceeded the plastic shear demand and that the reinforced concrete joints had adequate shear capacity. For steel PR connections, plastic hinge capacities were computed based on the four limit states of clip angle riveted connections [3]. Axial hinges for the compression struts were computed based on FEMA 356 procedure. Figure 5 presents mathematical models of the structures.

A. PHYSICAL URM INFILL PANEL B.EQUIVALENT STRUT REPRESENTATION

FIGURE 4
MODELING OF INFILL STRUTS



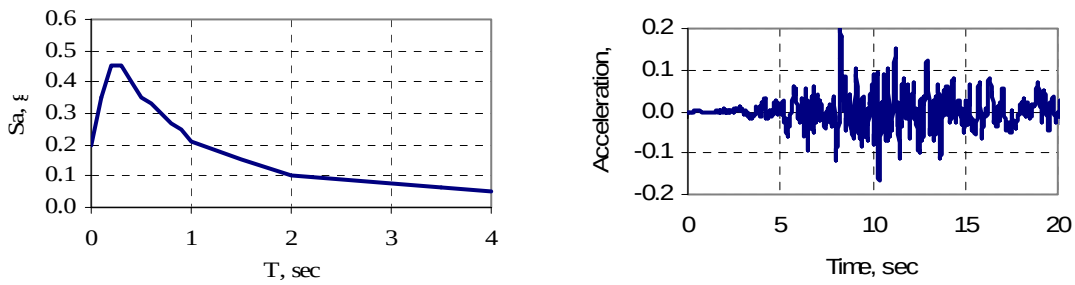
A. ELKS BUILDING B. 926 J STREET BUILDING

FIGURE 5
MATHEMATICAL MODELS OF BUILDINGS

SITE-SPECIFIC SEISMIC LOADING

For both structures, the Life Safety (LS) performance levels of the Design Basis Earthquake (DBE), which has a return period of 475 years, were selected as the target performance. Using available geotechnical data, site-specific acceleration response

spectra were developed [13]; see Fig. 6a. Three sets of spectrum compatible acceleration histories were also prepared. Figure 6b presents of the acceleration histories.



A. ACCELERATION RESPONSE SPECTRUM B. ACCELERATION HISTORY

FIGURE 6
SITE-SPECIFIC ACCELERATION SPECTRA, DBE

RESPONSE OF BUILDINGS

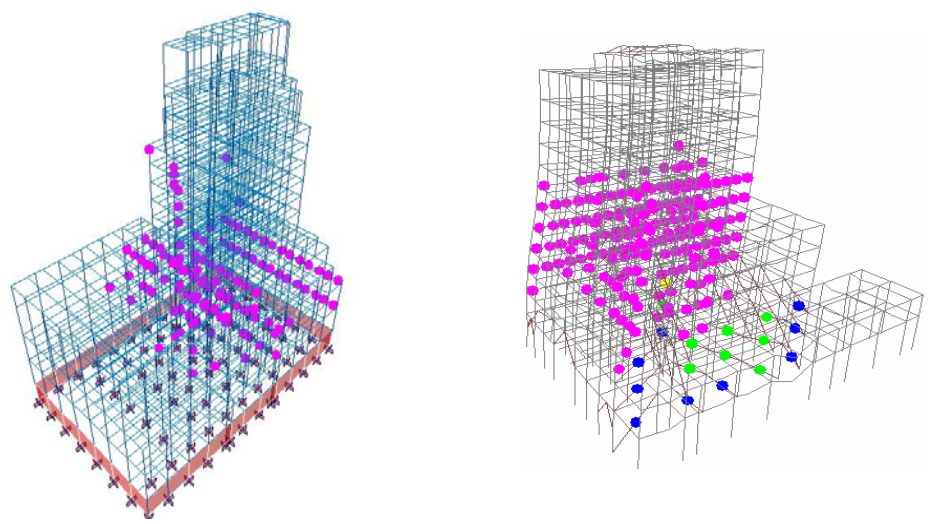
Dynamic analyses were conducted to obtain the modal properties of the structures. For the Elks building, there was some coupling of translational and torsional response. Although the building has nearly rectangular plans at each floor, due to offset above the fifth floor, there is an eccentricity between center of mass and rigidity below and above this level. .For the 926-J street building, the modes in each direction were uncoupled and there was insignificant translational-torsional coupling. Although, this structure is L-shaped, the URM infill panels were constructed on the short sides of the *L*, and as such, they reduce torsional response. There was no soft-story response for either building. However, relative deformations were largest above the fourth floor for the 926-J building, where infill panels were terminated. Table 2 presents pertinent modal information for the first six major structural modes. The Elks building has longer periods because semi-rigid connections were used for this structure. Response-spectrum analyses showed that the base shear was approximately 0.08W for each building.

	Elks Building				926-J Building			
Mode	T, sec	M participation, %			T, sec	M participation, %		
		x-	y-	θ–		x-	y-	θ–
1	3.26	0	46	0	2.24	65	0	3
2	3.04	30	0	4	2.05	0	62	1
3	2.30	0	0	19	1.74	0	0	49
4	1.87	8	14	21	0.85	19	0	0
5	1.49	9	14	2	0.79	0	19	4
6	0.7	18	0	10	0.69	0	0	19

TABLE 2
DYNAMIC PROPERTIES OF THE STRUCTURES

Displacement patterns obtained from the response spectrum Analyses were used for the FEMA 356 static nonlinear analyses. For the 926-J building, due to lack of torsional response, this procedure was adequate in lieu of the more sophisticated modal pushover

Analyses, recommended by Chopra and Goel [5] for asymmetrical multi-story buildings. For the Elks building, this method is also appropriate for the following reason. Results of nonlinear static analysis showed that at the performance point, the steel members did experience only limited nonlinear rotations. In addition, elastic acceleration history analysis resulted in displacements similar to the of nonlinear static analysis. Fig. 7 presents the displaced shape of the structures at the target displacement. For the Elks Building, nonlinear hinges are well distributed and nonlinear flexural demands on the hinges does not exceed the LS limit. At lower stories, wheree steel beams and a number of columns are encased in concrete and have larger flexural capacity, no flexural hinges were formed at this level of deformation. For the 926-J Building, significant flexural hinges formed at the mid height of the building. At the lower floors and near the roof, few members experienced yielding. All nonlinear response was confined to beams and struts; no concrete columns yielded. Due to the presence of large flexural hinges in the beams at the middle floors, the structure did not meet the LS performance level.



A. ELKS BUILDING B. 926 J STREET BUILDING

FIGURE 7
NONLINEAR STATIC ANALYSIS

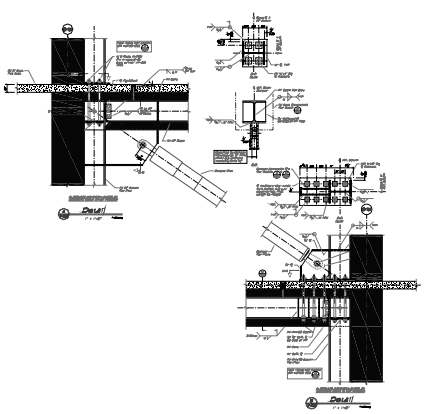
SEISMIC UPGRADE

For the Elks Building, the only shortcoming was the pounding against the adjacent low-rise building. In order to mitigate this problem, the owner increased the seismic gap between the two structures. This building met the FEMA 356 [3] LS requirements. No seismic requirement of this building was required and this concludes the discussion regarding this structures. FVDs, steel braces, and fiber-reinforced polymer (FRP) composites were used in the seismic upgrade of 926 J Building. FVDs connect diagonally in bays and serve to increse the damping in the structure and thus reduce the seismic demand. The motion of silicone oil in between chambers serves this purpose. The input kintetic energy to the damper is converted to heat and is dissipated. FVDs have been extensively researched, see for example Constantinou and Symans, [7] and Reinhorn et al [11] and implemented by Miyamoto International in seismic retrofit of the historic Hotel

Woodland [8], seismic retrofit of an essential facility [10], and seismic retrofit of historic Hotel Stockton [9]. Sixteen FVDs were added between the fifth and eighth floors on the building's perimeter to reduce story drift ratios and seismic demand on the reinforced concrete members. Diagonal steel braces were added to the lower levels to complement the URM infill panels and add lateral stiffness to the lower levels. FRP was added to the floor slabs to serve as drag struts. The damper had an exponent (α) of 0.5 and a damping constant (c) of 300 kip-sec/in.

VERIFICATION STUDIES

Nonlinear acceleration history analyses were performed to verify the performance of the proposed seismic upgrade. Even with addition of dampers, some minor yielding of concrete members is anticipated. The maximum story displacement and drift at the corner of the sixth floor was reduced from 3.7 to 3.3 in. and from 0.6% to 0.4%, respectively. Fig. 8 depicts the construction detail for FVDs and summarizes the seismic response of the upgraded structure. FVDs reduce drift at the critical mid-height stories, the levels for which the existing structure experienced the largest floor displacements, seismic demand, and flexural hinging. FVDs dissipate significant seismic energy as indicated in the force-displacement hysteresis of a typical damper in Fig. 9. FVD and inherent structural damping comprise the major components resisting seismic input energy. As shown, FVDs dissipate close to 75% of the seismic energy for this structure. In the absence of dampers, the yielding of concrete members would have absorbed this energy. As such, the dampers precipitously reduce the nonlinear response of concrete beams.

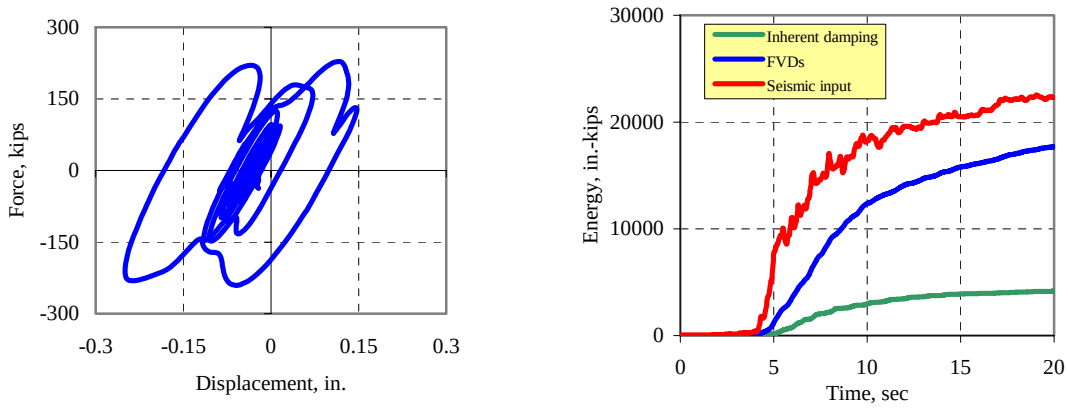


A. CONSTRUCTION DETAILS FOR FVDS

Floor	Displacement, in.		Drift reduction, %
	Existing	retrofitted	
7th	4.4	3.9	21
6th	3.7	3.3	22
5th	2.9	2.7	22

B. STORY DRIFT RATIOS

FIGURE 8
CONSTRUCTION DETAILS FOR FVDS AND REDUCTION IN STORY DRIFT RATIOS



A. FVD RESPONSE

B. COMPONENTS OF SEISMIC ENERGY

FIGURE 9
ENERGY DISSIPATED BY FVDS, FOR A TYPICAL ACCELERATION HISTORY

SUMMARY AND CONCLUSION

Performance-based earthquake engineering showed that the seismic performance of two multi-story moment-resisting frame buildings. One building met the Life Safty performance goal, whereas the other structure was rehabilitated using dampers. Time history analyses showed that the rehabilitated structure meets its performance goals. From analyses reported herein, it is seen that:

- Performance-based analysis can be effectively used to assess the seismic response of reinforced concrete and steel moment-resisting frame structures and readily identify the building deficiencies and strategies to alleviate these deficiencies.
- FEMA 356 recommendations provide a convenient method of incorporating complex structural features such as masonry infill panels and partially rigid connections.
- The performance of the building using steel framing was adequate and better than similar structure using concrete framing with limited ductility. Similar observations have been made during site investigations after major earthquakes affecting older buildings.
- Fluid viscous dampers provide a cost-effective, efficient, and non-intrusive method for rehabilitating historic buildings. The seismic demand on sensitive and non-ductile members is significantly reduced by increasing the total system damping. Since these dampers are primarily out-of-phase with displacement response, the added demand on building columns is minimal.
- A combination of dampers and braces or viscoelastic dampers can be used to increase system damping and lateral stiffness. This application would mitigate soft-story response, coupled torsional response, and reduce story drift ratios and nonlinear flexural demand on concrete members.

REFERENCES

[1] American Institute of Steel Construction Inc.*: AISC Manual of Steel Construction*, 1928 Edition,, Dec. 1928.

[2] Bennet, R. M, Flanagan, R. D., Adham, S., Fischer, W. L., and Tenbus, M. A.: *Evaluation and Analysis of the Performance of Masonry Infills during the Northridge Earthquake*, Report to the National Science Foundation, March 1996.

[3] Building Seismic Safety Council: *FEMA 356, Prestandard and Commentary for the Seismic rehabilitation of Buildings*, Washington, D.C. November 2000.

[4] California Building Standards Commission: California Building Code, 2001 Edition, , Sacramento, CA, 2001.

[5] Chopra, A.K. and Goel, R.K.: *A Modal Pushover Analysis Procedure for Estimating Seismic Demands for Buildings*, Earthquake Engineering and Structural Dynamics, Vol. 31, No. 3, pp. 561-582, March 2002.

[6] Computer and Structures, Inc.: *ETABS 8.5.2, Linear and nonlinear static and dynamic analysis and design of building systems*, Berkeley, CA, 2005.

[7] Constantinou, M. and Symans, M.: *Experimental & Analytical Investigation of Seismic Response of Structures with Supplemental Fluid Viscous Dampers*," State University Of New York at Buffalo, Buffalo, New York, NCEER-92-0032, 1992.

[8] Miyamoto, H.K. and Scholl, 2002: *Seismic Rehabilitation of an Historic Non-Ductile Soft-Story Concrete Structure using Fluid Viscous Dampers*, Proceedings of the 11th, World Conference on Earthquake Engineering, 2002.

[9] Miyamoto, Kit, Determan, L., Gilani, A. and Hanson R.:, *Design of an Essential Facility with Steel Moment Frames and Viscous Dampers Using NEHRP Procedure*, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 2004.

[10] Miyamoto, Kit, Hanson R., , and Kohagura T.: *US Practice for Buildings with Added damping*, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 2004.

[11] Reinhorn, A.M., Madan, A., Valles, R.E., Reichmann, Y., Mander, J.B.: *Modeling of Masonry Infill Panels for Structural Analysis*, Report No. NCEER-95-0018, State University of New York, Buffalo, December 1995.

[11] Reinhorn, A.M., Li, C., Constantinou, M.C.: *Experimental & Analytical Investigation of Seismic Retrofit of Structures with Supplemental Damping, Part 1: Fluid Viscous Damping Devices*, Report No. NCEER-95-0018, State University of New York, Buffalo, January 1995.

[13] Singh J.P.: Personal communications, 2004.