CASE STUDY: SEISMIC REHABILITATION OF A NON-DUCTILE SOFT STORY CONCRETE STRUCTURE USING VISCOUS DAMPERS

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ABSTRACT

Hotel Woodland is one of the first structures in North America to be seismically retrofitted using viscous dampers (VD's). This 4-story 1927 vintage Historical Landmark reinforced concrete building is located in Woodland, California. Maintaining the historical appearance of the building, the earthquake response performance of the building, and cost were the primary considerations in establishing the retrofit design.

The building is essentially a non-ductile reinforced concrete (RC) frame at the first level, and RC shear wall at levels 2, 3, and 4. Options considered for the retrofit included both adding conventional shear walls or braces at the first level, and using VDs and steel moment frames at the first level. Adding shear walls or braces at the first level caused 2 ancillary problems: 1) obscuring the historical appearance of the building, and 2) limiting commercial development. Both stick and 3D model analyses revealed that installing VDs and moment frames at the first level reduced drifts at all levels (1, 2, 3, and 4) to the desired performance. Using VDs proved to be the most cost effective method for seismically retrofitting the building. In addition, using VDs economically facilitated maintaining the historical appearance of the building.

KEYWORDS

Seismic Rehabilitation; Non-Ductile Concrete; Soft Story; Viscous Dampers.

INTRODUCTION

Seismic rehabilitation of existing buildings is one of the most challenging tasks that structural engineers face today. Historical buildings in particular increase the complexity of the rehabilitation task, because there are stricter architectural requirements. Every building is unique in its own way, and there is no easy cookbook approach. Conventional code and method may not be applicable to the project, therefore engineers must be creative as they tackle retrofitting problems. Adequate communication between the design team and the owner of a building is critical, much more so than with projects involving new buildings. Performance objectives, cost limitations, and future commercial development are very important issues that everybody must understand and agree upon.

DESCRIPTION OF THE STRUCTURE

Hotel Woodland is a 4-story reinforced concrete building constructed during the latter part of 1927. The building is a National Historic Registered building. The ground level footprint is approximately 168 feet by 95 feet, the upper three levels have a footprint of 168 feet by 50 feet, and the total square footage is approximately 50,000 ft^2. The total height of the structure is about 53 feet, not including the basement which is under only part of the building. The ground floor is used as commercial/retail space, and the 2nd floor and above are single-occupancy apartments presently occupied (see fig. 1).

None of the original plans were available at the time of analysis, therefore destructive investigation was conducted to determine material properties. The 2nd, 3rd, and 4th floors are cast-in-place concrete joist-beam construction with 2 1/2" concrete slab. Typical columns are 16" square concrete, reinforced with 4- 3/4" square grade 40 reinforcing bars with 5/16" square ties at 12". Typical exterior frame consist of 48" deep by 10 3/4" thick concrete spandrel beams and 48" wide by 6 3/4" thick concrete piers. The concrete wall pier-spandrel beam construction is terminated at the 2nd floor. In addition, 6 3/4" thick concrete bearing-shear walls exist at the East and West end of the structure at the ground floor.

No lateral resisting elements are found at the North and South elevation of the building at the ground floor, except for 16" square lightly reinforced concrete columns. This type of structure in the East-West direction is often defined as a non-ductile soft/weak story structure. The total reactive weight of the structure is approximately 5100 kip and the destructive testing indicated that the average compression strength of the concrete is approximately 3000 psi.

SEISMIC RETROFIT CRITERIA

This was an owner-option seismic retrofit, therefore maintaining the historical appearance of the building, the earthquake response performance of the building, and cost were the primary considerations in establishing the retrofit design.

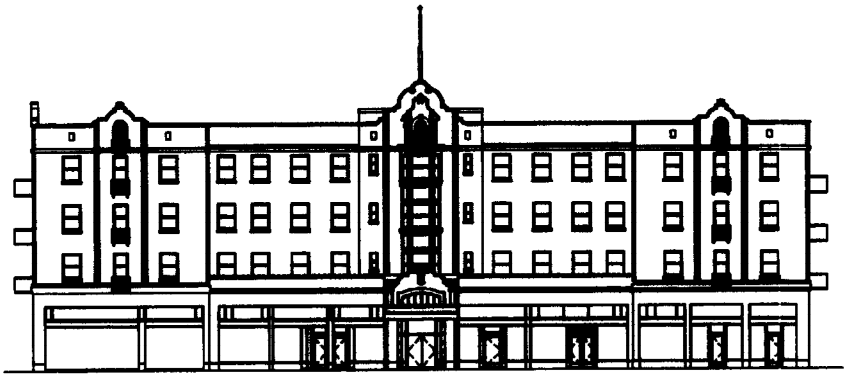
Performance criteria is based on the Uniform Code for Building Conservation, which defines a performance objective as "promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes" (ICBO, 1994). Considering the cost, the design team and owner decided that the seismic retrofit objective should be limited to preventing the collapse of the four-story super structure, since it would present a major threat to life safety. Some damage to the super structure was allowed.

The Design Basis Earthquake (DBE) is a 20% probability of occurrence in a 50 year duration. This event is consistent with the California Seismic Safety Commission Recommendations for the "Acceptable Seismic Risk for State Buildings" report. The Maximum Capable Earthquake (MCE) selected for the retrofit is a 10% probability of occurrence in a 100 year duration. Three pairs of Time Histories for each DBE and MCE event were constructed to analyze the structure. A detailed discussion regarding the design usage of DBE and MCE is described in the 'Design Criteria' section of this paper.

SEISMIC HAZARD

Deterministic and probabilistic analyses were performed to estimate the Peak Ground Acceleration (PGA). These analyses revealed that a .17g site acceleration would occur from a magnitude 4.5 DBE event on the Dunnigan Hills fault and .26g acceleration would occur from a magnitude 5.5 MCE event on the Dunnigan Hills fault (Gius, 1994).

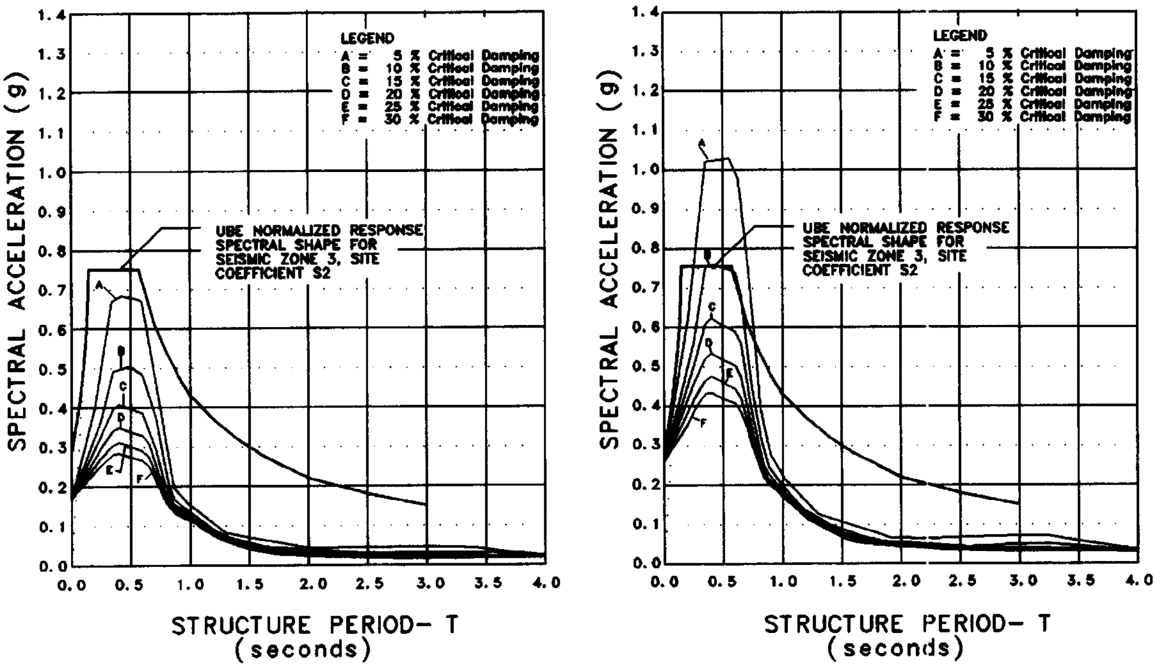
Actual California earthquake time histories were utilized to develop site-specific ground response spectra. Horizontal ground accelerations were selected based on events 15 to 30 miles from the recording station, at an alluvium underlain station, and from an event of a similar fault mechanism to faults expected to affect the site (thrust faults). The selected horizontal ground time histories were scaled to DBE and MCE maximum horizontal accelerations considering potential amplification effects of the soil by a computer program 'Shake' (Gius, 1994), (see fig. 2) .



NORTH ELEVATION

ARCHITECTURAL ELEVATION

FIG. 1



SITE SPECIFIC DBE SPECTRA

SITE SPECIFIC MCE SPECTRA

FIG. 2

EXPECTED PERFORMANCE OF THE EXISTING BUILDING

As part of the study, 3-dimensional Time History analysis of the original building was performed. An analytical model was subjected to non-reduced DBE Time Histories. Due to the lack of ductility details, hysteric energy absorption capacity of the existing material was discounted. Following are the results of the study:

The fundamental period of the building in the East-West direction was approximately .7 second. Effective mass factor of a the fundamental mode was 98%. Maximum displacement and story shear are shown in table 1.

Table 1

Story	Story Displacement (inch)	Story Shear (kips)
Roof	0.01	290
4th	0.02	890
3rd	0.01	1484
2nd	2.00	2200 (.43G)

The concrete columns at the ground floor level were overstressed in bending and shear due to excessive deflection and the lack of ductility detailing and strength. Most of the non-linear behavior of this building was concentrated at the ground floor level columns. This type of adverse behavior could cause total collapse of the superstructure. An example can be seen at Olive View Hospital after the 1971 San Fernando earthquake. Under earthquake-induced loading, excessive lateral displacement caused plastic hinges to form in the ground floor columns (Moehle, 1994).

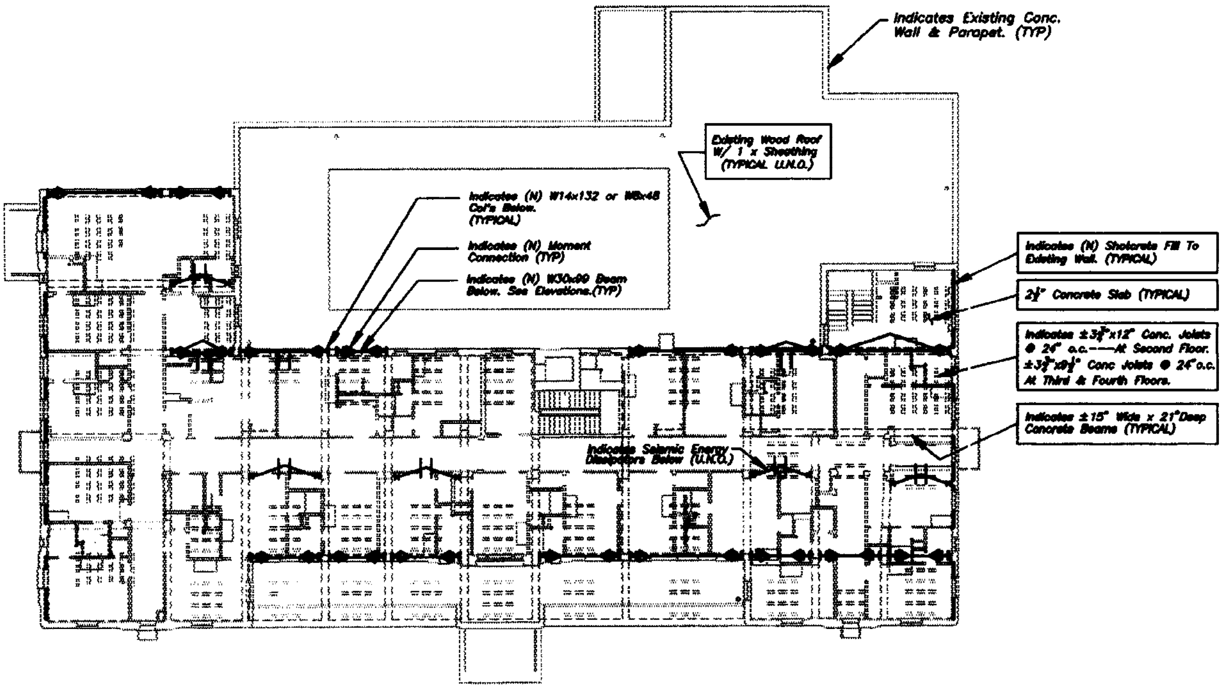
SEISMIC RETROFIT SCHEMES

After the design team and the owner defined and agreed on the above retrofit criteria, numerous seismic retrofit schemes were considered. Since this building is a National Historical Registered building, there were unique challenges that the design team had to meet, including 1) keeping the historical appearance of the landmark hotel, 2) maximizing the retail/commercial area at the ground floor, 3) avoiding disturbance to tenants living in apartments on the 2nd floor and above, and 4) managing cost effectiveness.

Concrete shear walls were rejected because they caused two ancillary problems: 1) damage of historical appearance, and 2) limitation of commercial development. Conventional steel brace frames were considered and rejected because practical brace locations were very limited. Also the steel brace frame and shear wall construction required major foundation modification. Concrete jacketing of existing ground floor columns was considered and rejected because of the uncertainty of the existing construction and cost limitation. Finally, using steel moment frames with fluid viscous dampers (VDs) at the ground floor was chosen (see fig. 3).

The steel moment frames were designed to provide stiffness, strength, and redundancy, which the existing lightly-reinforced concrete columns lacked. VDs were provided to control drift at the 1st floor and to keep steel moment frames in the elastic range. Elastic rotation of the moment frame connection was limited to acceptable level. VDs were attached to the top of the steel Chevron Braces (see fig. 4) and VDs-Chevron Braces were strategically located to meet the above requirements.

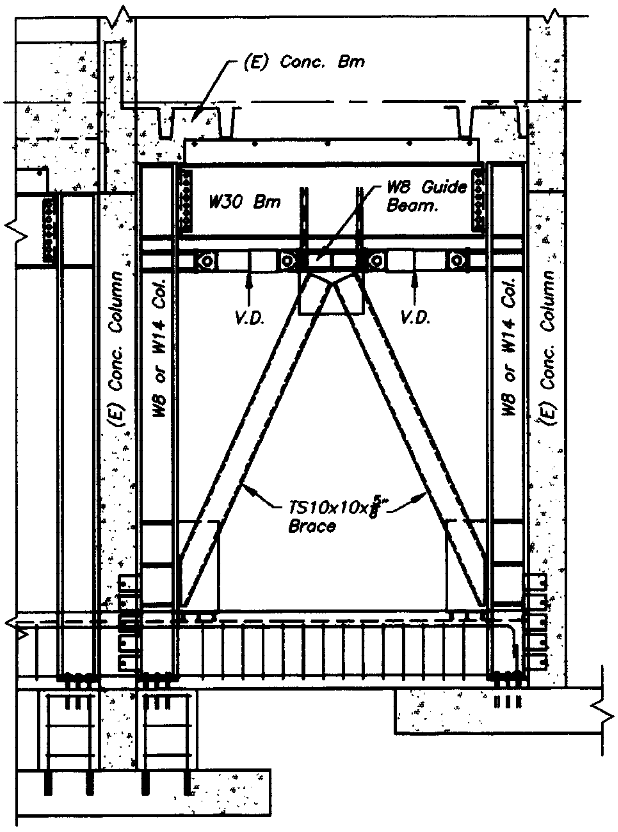
Fluid viscous dampers were selected over other supplemental dampers for the following reasons: 1) since it is a velocity-dependent system, large energy dissipation would be activated with small displacement, 2) the forces in VDs are out of phase with axial loading of the columns, and 3) the long history of military application proves system reliability.



SECOND FLOOR FRAMING PLAN

G.PARKER

FIG. 3



TYP VD ASSEMBLY ELEVATION

G.PARKER

FIG. 4

DESCRIPTION OF THE FLUID VISCOUS DAMPERS

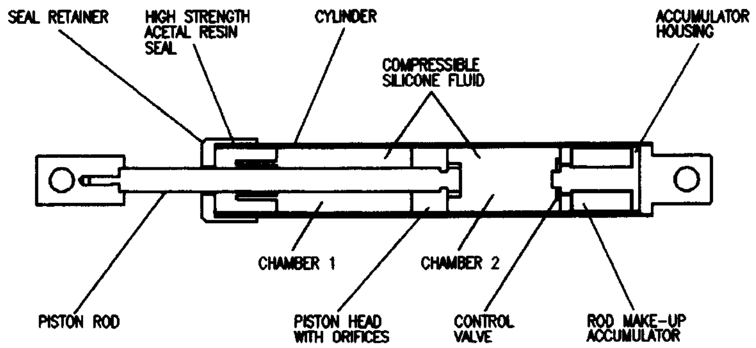
Fluid Viscous Dampers (VD's) operate on the principle of fluid flow through orifices. A stainless steel piston travels through chambers that are filled with silicone oil. The pressure difference between the two chambers cause silicon oil to flow through an orifice in the piston head, and seismic energy is transformed into heat, which dissipates into the atmosphere (see fig. 5). VD's can operate over temperature fluctuations ranging from -40 F to 160 F. The orifice construction utilized is similar to that in the classified application for the U.S. Air Force B-2 Stealth Bomber and is considered state of the art (Constantinou & Symans, 1992).

Sixty six earthquake simulation tests were performed on 1 and 3 story model structures at the State University of New York, Buffalo by Constantinou and Symans. Following are the major conclusions of the tests:

1) VDs exhibited essentially linear viscous behavior for a range of frequencies below 4 Hz, therefore if the natural frequencies of the dominant modes of the structure are below 4 Hz, the VDs may be modeled as linear viscous dampers.

2) The addition of VDs in the tested structure resulted in 30-70% reduction in story drifts and 40-70% reduction in story shear forces.

3) VDs introduced additional column axial forces, however these are out-of-phase with beam-column action forces (Constantinou & Symans, 1992).



CONSTRUCTION OF FLUID VISCOUS DAMPER

FIG.5

ANALYTICAL PROCEDURE

Two different mathematical models of the building were constructed to study retrofit schemes. One was a simple 2 dimensional stick model and the other was a complex 3-dimensional finite element model.

The stick model was utilized to 1) determine the most effective combination of mass, stiffness and damping, and 2) identify the most demanding time histories.

Linear Time History analyses were performed using a computer program 'RESP' (R.E. Scholl, 1994), which includes a step-by-step response solution procedure. Story mass, stiffness, and damping values were lumped at each story level to form a 4 degree of freedom model.

The 3-dimensional model was used to 1) verify 2-D stick model results, 2) obtain member forces, and 3) study cross coupling between stiffness of braces and VDs.
Time history analyses were performed using the computer program ETABS 6.04 (CSI, 1994) which also utilized the Step-by-Step Linear Acceleration Method. The computer model had 146 column lines, 115 beam bays, 16 brace elements, 22 panel elements, and 8 link elements. The link elements represented the VDs. A

total of 20 mode shapes were extracted. Twelve mode shapes belonged to the structural stiffness and mass matrix, and 8 mode shapes belonged to the link elements. Each floor had X, Y translation and Z rotation, which constitute a rigid diaphragm. The top of the Chevron Braces were disconnected from the 2nd floor rigid diaphragm, and the link elements connected the top of the Chevron Braces to the 2nd floor to emulate VDs.

DESIGN CRITERIA

Three Time Histories for each DBE and MCE event were chosen and scaled based on the criteria described in the 'Seismic Criteria' section of this paper. These time histories include: 1) Coalinga 1983 Cantua Creek School 1, 2) Coalinga, 1983, channel 3, and 3) El Centro 1940 (N-5 Record).

Each event was applied simultaneously to the mathematical model in X and Y directions. The analysis indicated that Coalinga 1983, channel 1, produced the worst case scenario, therefore it was chosen as the design earthquake motion.

Critical elements in the structure were designed to sustain limited damage for the MCE event, and the other elements were designed for the DBE event. The following is a summary of design criteria:

1) Critical ground floor concrete columns were analyzed with the MCE event, considering cracked sections, and p-delta effect.

2) The ground story drift was limited to .002 at DBE and .003 at MCE to protect the existing brittle structure.

3) All existing and new shear wall responses were limited to elastic range only at the DBE event.

4) The foundation stability was analyzed using the DBE event.

5) The stress ratio of the new Steel Moment Frames were limited to approximately 20% of the yield to protect the welded connections at the DBE event.

6) VDs were designed for MCE events. All VD connections and Chevron Braces assembly were also designed to remain elastic for MCE events.

EXPECTED PERFORMANCE OF THE RETROFITTED STRUCTURE

The fundamental period of the retrofitted building in the East-West Direction was .46 second. The effective mass factor of the fundamental mode in the East-West Direction was 97%. Approximately 40% of critical damping was provided by VDs at the ground floor where maximum inter-story drift and velocity occurred. A total of 16 steel moment frames with W 14x132 grade 50 columns, and W 30x99 grade 50 beams were provided. A total of 8-VD assemblies with 16-50 kip output dampers were provided (see fig. 4). The damping constant for each VD was 9.4 kip-second/inch. The exponential constant was set as a unit, which produced perfect linear viscous behavior. The maximum design axial force of the VDs was 100 kip with a safety factor of 2.0. The maximum displacement, velocity, and story shear for DBE are shown on Table 2 for 5% of critical damping without VDs and on Table 3 for 40% of critical damping at the ground level with VDs.

Table 2. Steel Moment Frame without VDs (DBE), 5% modal damping

Story	Story Displacement (inch)	Story Shear (kips)	Story Velocity (inch/sec)
Roof	0.01	410	0.09
4th	0.03	1256	0.25
3rd	0.03	2090	0.41
2nd	1.31	3087 (.60G)	16.80

Table 3. Steel Moment Frames with VDs (DBE): Final Design, 40% modal damping

Story	Story Displacement (inch)	Story Shear (kips)	Story Velocity (inch/sec)
Roof	0.01	183	0.05
4th	0.01	558	0.13
3rd	0.01	927	0.22
2nd	0.41	1374 (.26G)	5.50

The above tables show that by providing VDs, both base shear and 2nd floor displacement were reduced by approximately 60%. Plastic deformation of both existing concrete and new steel moment frames were precluded, and the majority of the seismic energy was absorbed by VDs.

CONCLUSION

Using a combination of steel moment frames and VDs proved to be the most cost effective method for seismically retrofitting the building. It also accommodates the historical appearance and commercial utilization requirements. In addition, limiting plastic deformation of the structural material reduced the uncertainty in the structural behavior in the case of a seismic event.

Using supplemental damping can be a very effective method to resist seismic force for many buildings. The authors strongly believe that supplemental dampers will be one of the 'star' solutions to protect structures from the destructive forces of earthquakes in the 21st century.

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