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**SEISMIC REHABILITATION OF A HISTORIC  
NON-DUCTILE SOFT STORY CONCRETE STRUCTURE  
USING FLUID VISCOUS DAMPERS**

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by

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# **SEISMIC REHABILITATION OF A HISTORIC NON-DUCTILE SOFT STORY CONCRETE STRUCTURE USING FLUID 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 (VDs). 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: first obscuring the historical appearance of the building and second, 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.

This paper presents detailed results of a case study illustrating the processes and decisions regarding retrofit criteria and design procedure for earthquake demand, building response performance, historical interests, and economic considerations.

## **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.

In the aftermath of the Northridge Earthquake of 1994, the fundamental concept of earthquake resistant design, which relies heavily on the ductile behavior of the material, was put into question. Code-confirmed concrete and steel buildings were damaged beyond engineers' and the public's expectations. Providing supplemental damping may be one of the solutions. Dampers can eliminate or reduce plastic deformation of members, therefore decreasing the uncertainty involved with non-linear behavior of the structure.

This paper presents results of a case study illustrating the processes and decisions regarding the seismic retrofit design of a 4-story non-ductile reinforced concrete building.

## **DESCRIPTION OF THE STRUCTURE**

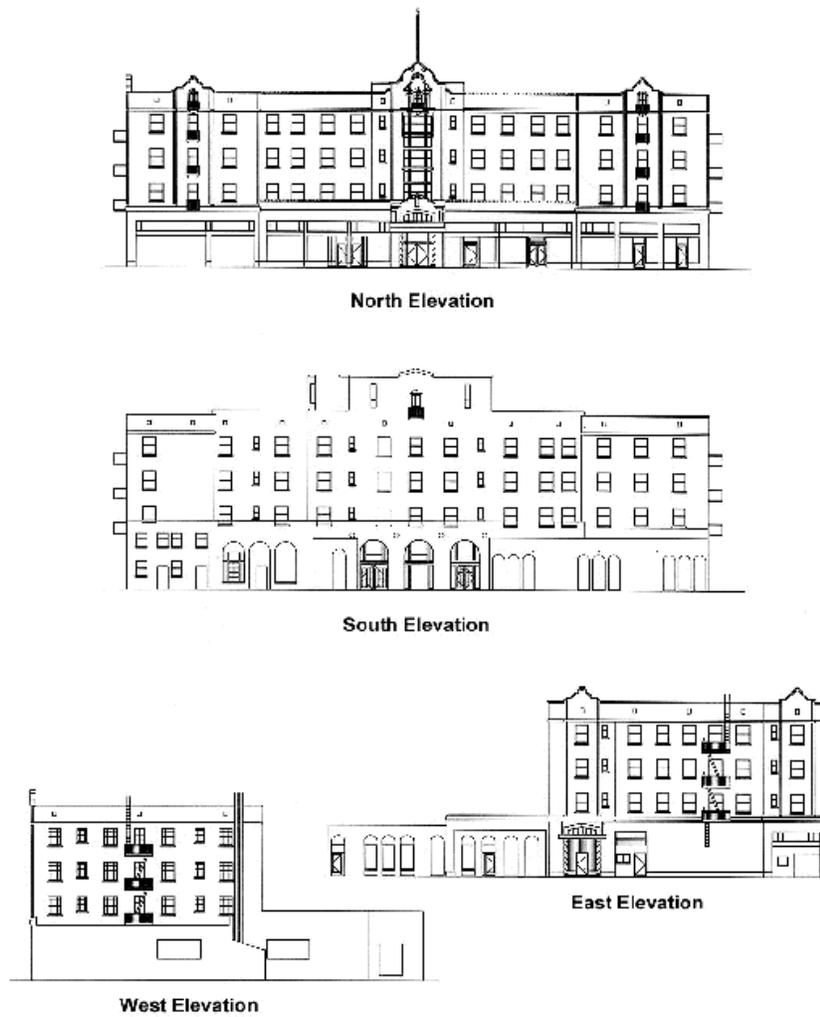
Hotel Woodland is a 4-story reinforced concrete building constructed during the latter part of 1927. The building is a National Historic Registered building. The ground level footprint is approximately 168 feet by 95 feet, the upper three levels have a footprint of 168 feet by 50 feet, and the total square footage is approximately 50,000 square feet. The total height of the structure is about 53 feet, not including the basement that 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 Figure 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 2.5 inch concrete slab. Typical columns are 16 inch square concrete, reinforced with 4.75 inch square grade 40 reinforcing bars with .31 inch square ties at 12 inches. A typical exterior frame consists of 48 inch deep by 10.75 inch thick concrete spandrel beams and 48 inch wide by 6.75 inch thick concrete piers. The concrete wall pier-spandrel beam construction is terminated at the 2nd floor. In addition, 6.75 inch 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 inch 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 pounds per square inch.

The subsurface investigation revealed that the structure is supported on isolated spread foundations extending five feet below the slab on grade. The typical dimensions are 11 feet by 11 feet by 1 foot deep. The geotechnical consultant determined that a site coefficient of S2 was appropriate for the site.



**FIGURE 1**  
**ARCHITECTURAL ELEVATIONS**

## **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.

When seismically rehabilitating existing structures, use of the Uniform Building Code is insufficient since it does not address performance objective other than life safety in quantifiable ways (Hart & Elhassan, 1994). Therefore, the design team and owner defined the design seismic event and the performance criteria for the structure.

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 superstructure, since it would present a major threat to life safety. Some damage to the superstructure was allowed.

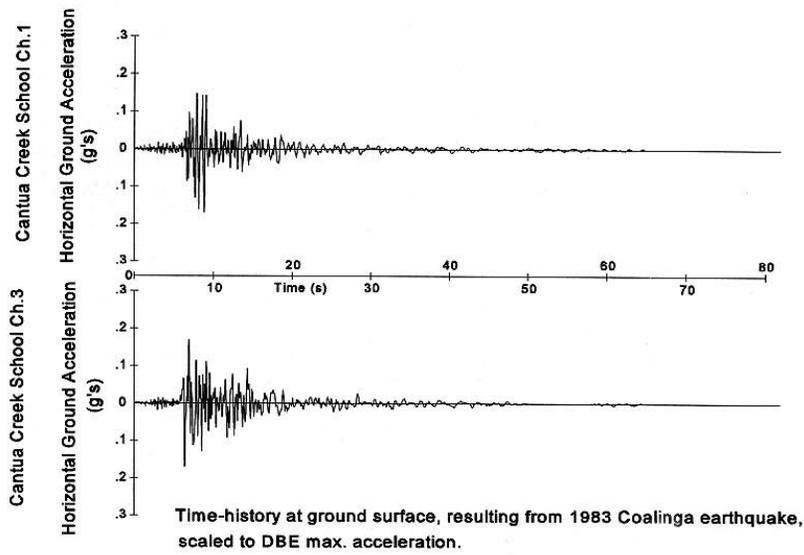
The Design Basis Earthquake (DBE) is a 20% probability of occurrence in a 50 year duration. This event is consistent with the California Seismic Safety Commission Recommendations for the “Acceptable Seismic Risk for State Buildings” report. The maximum Capable Earthquake (MCE) selected for the retrofit 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**

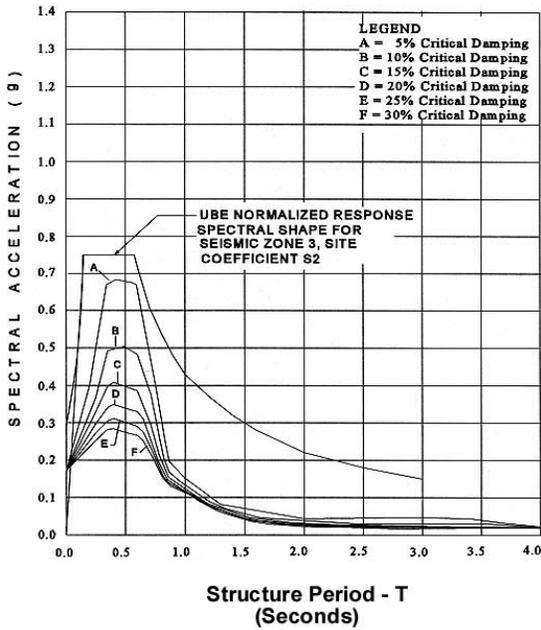
The geotechnical consultant (Wallace-Kuhl & Associates) performed a site-specific ground motion study for the hotel site. Historically, the largest earthquake event to influence the site was the Richter Magnitude 6.75 Vacaville-Winters event of 1892. This event and its aftershocks were estimated to have produced an attenuated site acceleration of approximately 0.14g (Gius, 1994).

Deterministic and probabilistic analyses were performed to estimate the Peak Ground Acceleration (PGA). These analyses revealed that a 0.17g site acceleration would occur from a magnitude 4.5 DBE event on the Dunnigan Hills fault and 0.26g acceleration would occur from a magnitude 5.5 MCE event on the Dunnigan Hills fault (Gius, 1994).

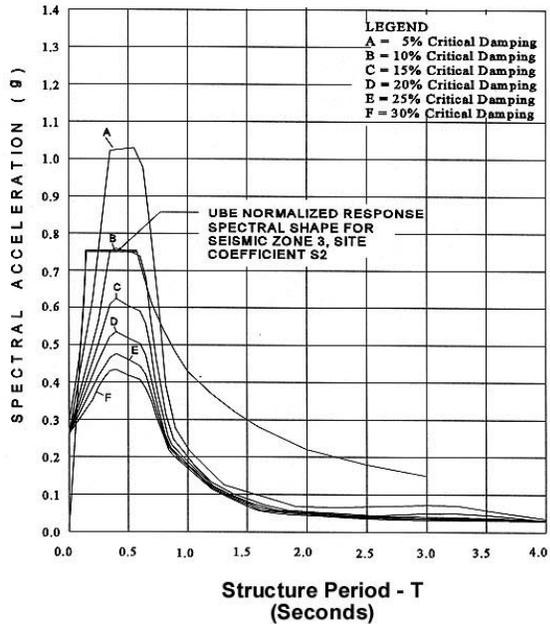
Actual California earthquake time histories were utilized to develop site-specific ground response spectra. Horizontal ground accelerations were selected based on events 15 to 30 miles from the recording station, at an alluvium underlain station, and from an event of a similar fault mechanism to faults 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 Figure 2).



### DBE Modified Time Histories



Site Specific DBE Spectra



Site Specific MCE Spectra

FIGURE 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, hysteretic energy absorption capacity of the existing material was discounted. Following are the results of the study:

### *East-West (Longitudinal) Direction:*

The fundamental period of the building in the East-West direction was approximately 0.9 second. Effective mass factor of the fundamental mode was 98%. Maximum displacement and story shear are shown in Table 1.

**TABLE 1**  
**Maximum Displacement and Story Shear**

<b>STORY</b>	<b>STORY DISPLACEMENT (INCH)</b>	<b>STORY SHEAR (KIPS)</b>
Roof	0.01	118
4th	0.02	363
3rd	0.01	606
2nd	1.40	896

The concrete columns at the ground floor level were over-stressed 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).

### *North-South (Transverse) Direction:*

Fundamental period of the building in the North-South direction was approximately 0.16 second. Effective mass factor of the fundamental mode was 84.1%.

The wall piers and shear walls at the East and West ends of the building were over stressed in shear. This type of failure could cause total collapse of the structure. Also torsional excitation of the building was excessive due to the substantially longer shear wall at the West end.

## 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.
4. Managing cost effectiveness.

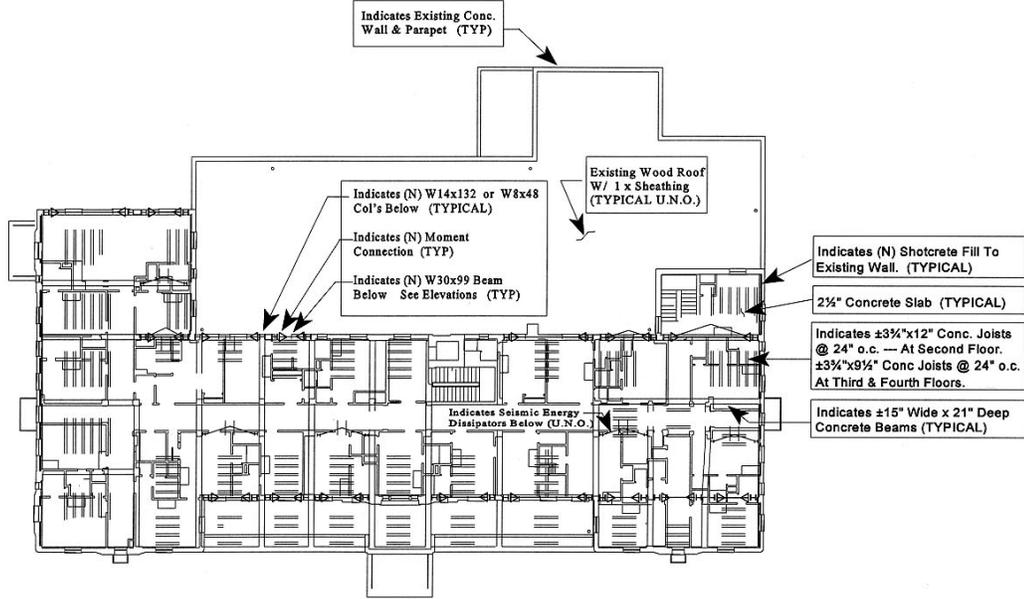
### *East-West (Longitudinal) Direction:*

Concrete shear walls were rejected because they caused two ancillary problems:

1. Damage of historical appearance.
2. Limitation of commercial development.

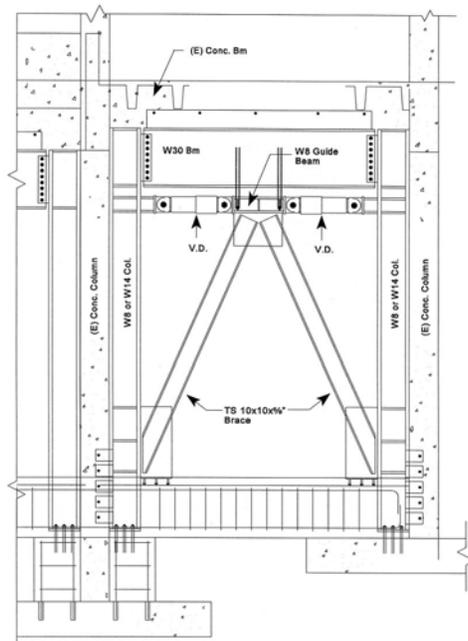
Conventional steel brace frames were considered and rejected because practical brace locations were very limited. Also the steel brace frame and shear wall construction required major foundation modifications. Concrete jacketing of existing ground floor columns was considered and rejected because of 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 Figure 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 Figure 4) and the VDs-Chevron Braces were strategically located to meet the above requirements.



**Second Floor Framing Plan**  
G. Parker

**FIGURE 3**



**Typical VD Assembly Elevation**  
G. Parker

**FIGURE 4**

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.
3. The long history of military application proves system reliability.

***North-South (Transverse) Direction:***

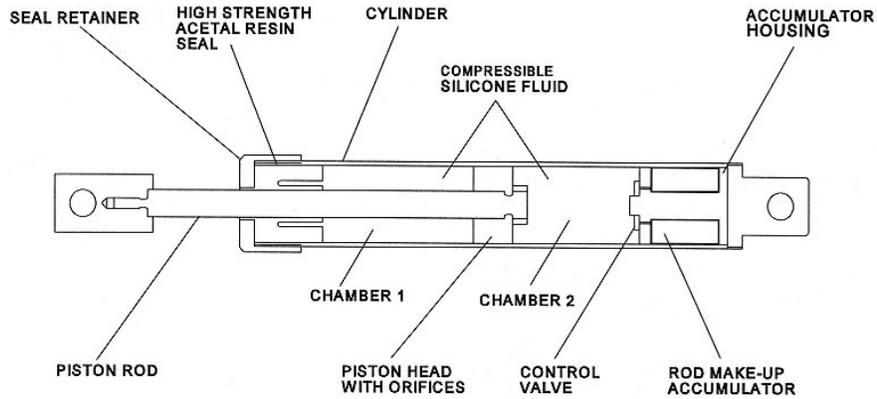
Displacement and velocity of the existing concrete shear walls were not large enough to activate VDs, therefore conventional shotcrete and new shear walls were provided at the ground floor level. The shotcrete and the new shear wall did not damage the historical appearance of the structure in the North-South direction.

**DESCRIPTION OF THE FLUID VISCOUS DAMPERS**

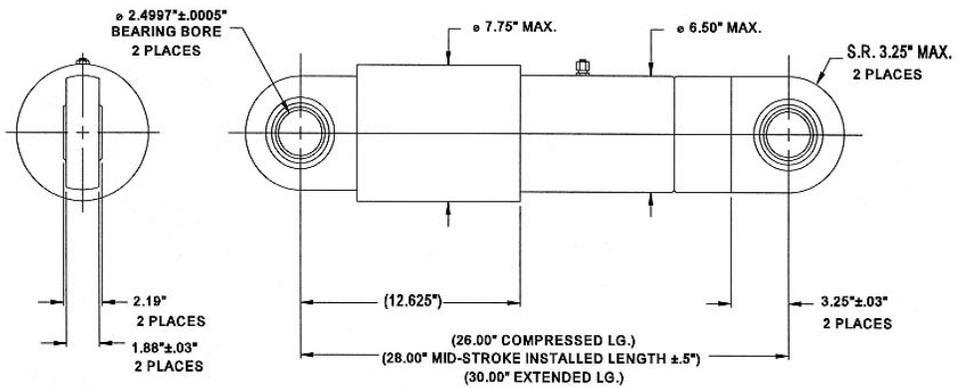
Fluid viscous dampers (VDs) operate on the principle of fluid flow through orifices. A stainless steel piston travels through chambers that are filled with silicone oil. The pressure difference between the two chambers cause silicone oil to flow through an orifice in the piston head, and seismic energy is transformed into heat, which dissipates into the atmosphere (see Figure 5). VDs can operate over temperature fluctuations ranging from -40 degrees F to +160 degrees 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**



**NOTE:**  
 PART NUMBER 67DP-15573-01, REV. A  
 BY TAYLOR DEVICES, INC. OR EQUIVALENT

- SPECIFICATIONS:**
1. UNIT TYPE: DOUBLE ACTING LINEAR FLUID VISCIOUS DAMPER.
  2. ALL PARTS THAT SLIDE RELATIVE TO ANY SEALS SHALL BE MADE FROM 17-4PH STAINLESS STEEL.
  3. APPROXIMATE WEIGHT OF THE UNIT = 275 LBS.
  4. UNITS TO BE CONSTRUCTED FROM CORROSION PROTECTED MATERIALS.
  5. OPERATING FLUID IS INERT SILICONE, PER U.S. FEDERAL STANDARD W-D-1078.
  6. OPERATING AMBIENT TEMPERATURE RANGE:  $-20^{\circ}\text{F}$  TO  $+130^{\circ}\text{F}$  WITH MINIMAL CHANGE IN PERFORMANCE CHARACTERISTICS.
  7. DAMPER STROKE  $\pm 2$  INCHES NOMINALLY WITH IDENTICAL CHARACTERISTICS IN EITHER DIRECTION OF MOTION.
  8. NOMINAL DAMPING FORCE = 50 KIP @ 5 IN/SEC.
  9. MAXIMUM DAMPER DESIGN LOAD = 100 KIPS.

**Double Acting Linear Fluid Viscous Damper**

**FIGURE 5**  
**Double Acting Linear Fluid Viscous Damper**

## 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:

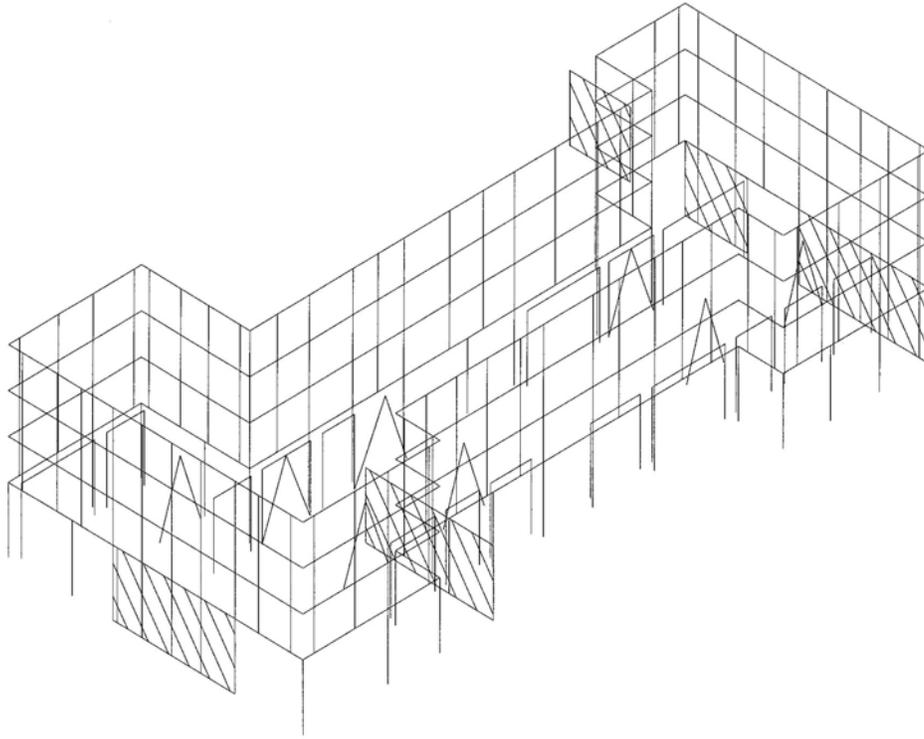
1. Determine the most effective combination of mass, stiffness, and damping.
2. Identify the most demanding time histories.

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.

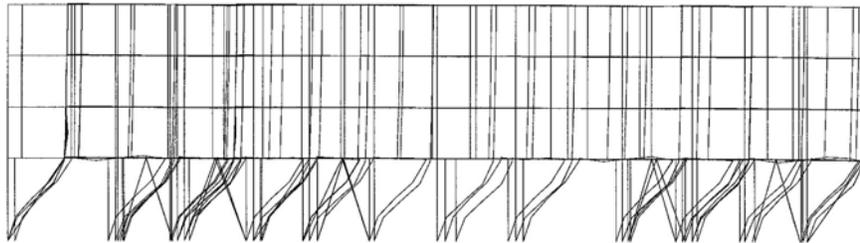
The 3-dimensional model was used to:

1. Verify 2-D stick model results.
2. Obtain member forces.
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 (see Figure 6). 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.



**ETABS6.0 - 3D Computer Model**



**ETABS6.0 - Deformed Shape Elevation (Mode 1)**

**FIGURE 6**

## 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.
2. Coalinga, 1983, channel 3.
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 stress to protect the welded connections at the DBE event. Joint rotation was limited to less than .001 radians.
6. VDs were designed for MCE events. All VD connections and Chevron Brace assemblies were also designed to remain elastic for MCE events.

## EXPECTED PERFORMANCE OF THE RETROFITTED STRUCTURE

### *East-West (Longitudinal) Direction:*

The fundamental period of the retrofitted building in the East-West Direction was 0.46 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 14 x 132 grade 50 columns, and W 30 x 99 grade 50 beams were provided. A total of 8-VD assemblies with 16-50 kip output dampers were provided (see Figure 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 a Table 3 for 40% of critical damping at the ground level with VDs.

**TABLE 2**  
**Steel Moment Frame without VDs (DBE)**  
**5% modal damping**

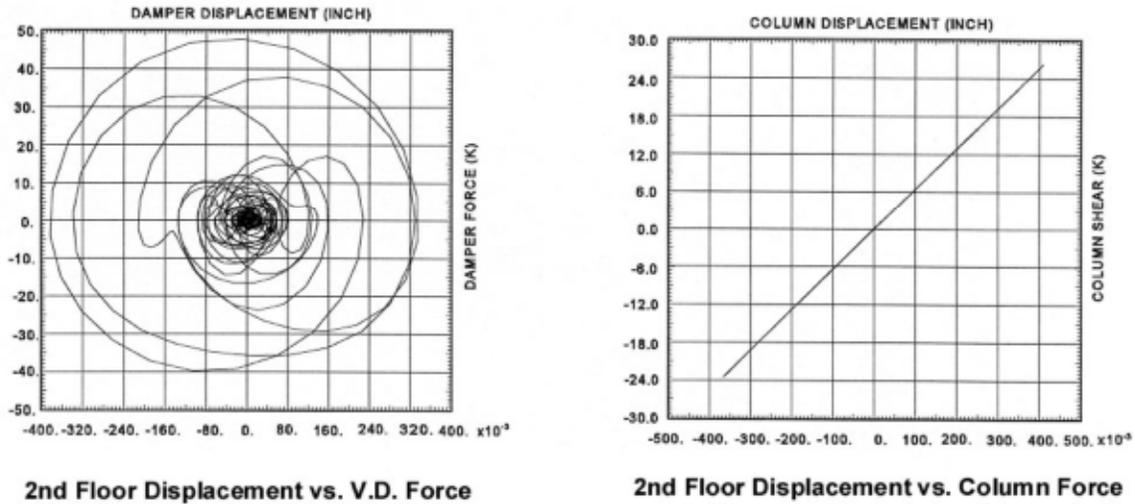
STORY	STORY DISPLACEMENT (INCH)	STORY SHEAR (KIPS)	STORY VELOCITY (INCH/SEC)
Roof	0.01	410	0.09
4th	0.03	1256	0.25
3rd	0.03	2090	0.41
2nd	1.31	3087 (.60G)	16.80

**TABLE 3**  
**Steel Moment Frame with VDs (DBE) - Final Design**  
**40% modal damping**

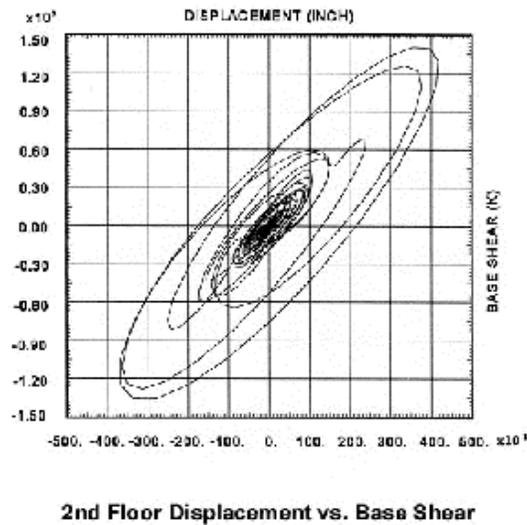
STORY	STORY DISPLACEMENT (INCH)	STORY SHEAR (KIPS)	STORY VELOCITY (INCH/SEC)
Roof	0.01	183	0.05
4th	0.01	558	0.13
3rd	0.01	927	0.22
2nd	0.41	1374 (.26G)	5.50

The above tables show that by providing VD's, 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 VD's.

Figure 7 shows Displacement vs. Damper force and steel column shear force for the BE. It clearly indicates the out-of-phase character of VD's. Figure 8 shows the ground floor displacement vs. base shear for DBE, which illustrates the energy absorption character of the structure.



**FIGURE 7**



**FIGURE 8**

### ***North-South (Transverse) Direction:***

Fundamental period of the retrofitted building in the North-South direction was 0.16 second. The effective mass factor of the fundamental mode in the N-S direction was 82%. Five inch shotcrete was added to the existing 7 inch wall at the full height of the East-West elevation. Also a new 12 inch shear wall was added at the ground floor to reduce torsional excitability of the structure.

### **REHABILITATION COST**

Total construction cost of the seismic strengthening was approximately \$500,000 U.S. (1995 present) which equates to \$10.0 per square foot. The above figure satisfied the construction cost requirement of the project. Construction began in August 1995.

### **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.

### **ACKNOWLEDGMENTS**

Owner and Developer: Engstrom, Morrow & Nagler Property  
Architect: Wirth & McCandless Architects  
Contractor: Unger Construction  
Damper Manufacturer: Taylor Devices, Inc.

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