DESIGN OF STEEL PYRAMID USING
FLUID VISCOUS DAMPERS WITH MOMENT FRAMES

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Abstract

This 11-story 450,000 ft^2 pyramid-shaped office building became one of the first new buildings in the United States to use Seismic Dampers. This National Headquarters for a financial institution is located in West Sacramento, CA.

The basic lateral force resisting system of the building consists of steel moment frames. In addition, approximately 15% of critical damping was provided using Fluid Viscous Dampers (FVD) in order to reduce displacement and acceleration. The steel moment frames were designed to remain well below the yield strength, and the story drift ratio was limited to 0.005 to protect the welded moment connections for the Design Basis Earthquake (DBE).

Earthquake performance, cost effectiveness, and architectural requirements were the primary concerns in designing this building. Site-Specific Response Spectra and Time Histories were synthesized for the DBE response spectrum. Design procedures and criteria were created to limit disruption to business operations after a major seismic event.

A cost study shows that using FVDs in conjunction with elastic steel moment frames proves to be cost effective. Specifically: 1) the natural period of the structure was kept out of high acceleration spectra range, yet displacement was controlled by FVDs, and 2) FVDs eliminated the uncertainty involved with the non-linear behavior of the structure.

Introduction

The National Headquarters for this financial institution, located in West Sacramento, California, became one of the first new buildings in the United States to employ seismic dampers to control a building's response during a seismic event.

This 11-story pyramid-shaped steel moment frame building occupies approximately 450,000 total square feet, including a partial basement and exterior deck areas. The ground floor footprint is 300 by 300 feet. There is a 10 foot set-back at each story, and the uppermost floor footprint is 90 feet by 120 feet. The typical story height is 14 feet, with the exception of the

height between the first and second level, which is 16 feet. The total height of the structure is 156 feet (see Figure 1).

The typical floor system consists of structural steel beams with 3 1/4" light weight, cast-in-place concrete over 3"x20 ga. metal deck. The structure's lateral system consists of elastic steel moment frames with fluid viscous dampers (FVDs) (see Figure 2).

The structure is supported on approximately 580 12"- square prestressed, precast piles, embedded approximately 75 feet into the ground. The geotechnical engineers determined that a site coefficient of "S2" was appropriate for this stiff sandy soil site.

Performance-Based Design Criteria

Earthquake performance, cost effectiveness, and architectural requirements were the primary considerations in designing this building. The owner's requirements were as follows: 1) disruption of business operations should be minimal after a major seismic event, and 2) the construction cost should not exceed that of a minimum code-conforming building.

Faced with the above requirements, the use of the current Uniform Building Code (ICBO, 1994) was insufficient, since it did not address the performance objectives in quantifiable ways. Furthermore, its design philosophy relies heavily on plastic hinge formations within the structural elements to absorb the seismic energy. This increases the uncertainty of the structure's non-linear behavior (McNamara, 1994). Therefore, the design team and the owner agreed upon the following criteria to satisfy the first requirement:
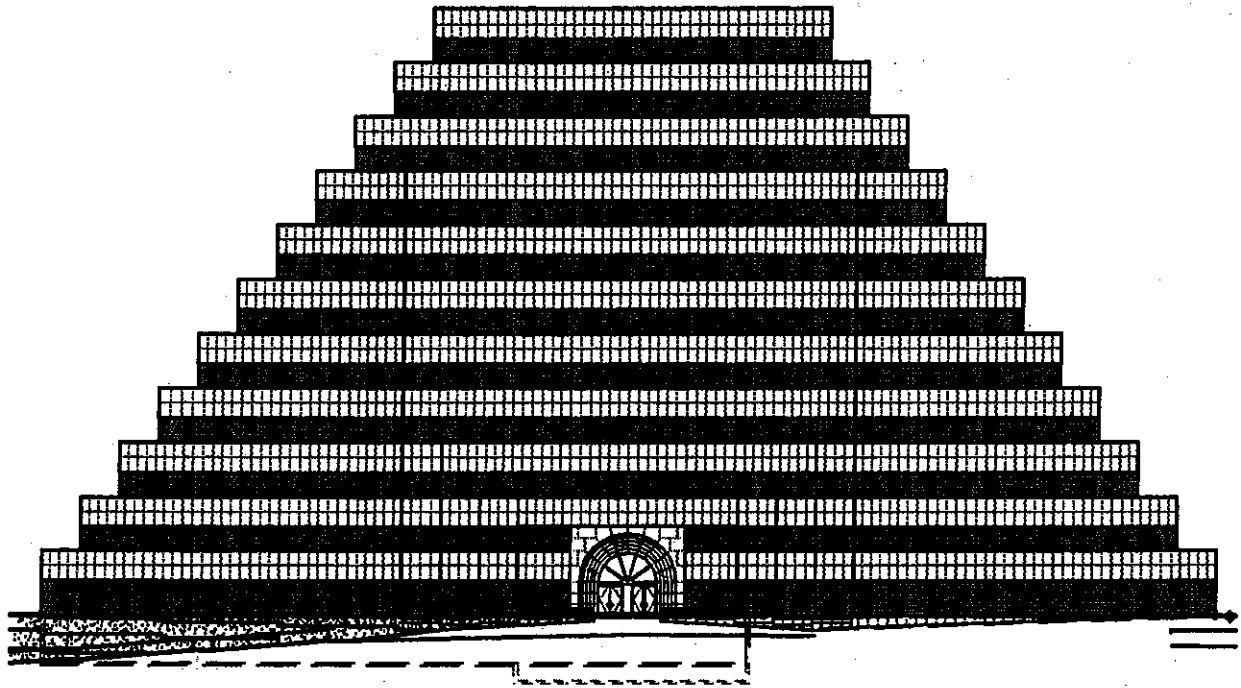
1. All structural members and connections shall remain below yield levels for the Design Basis Earthquake (DBE).
2. The maximum inter-story drift ratio shall be less than 0.005 at the DBE to protect non-structural elements.
3. An unreduced Time History Analysis shall be utilized to study the actual behavior of the structure during the DBE.

The Design Basis Earthquake is defined as a seismic event with a 10% probability of occurrence in a 50 year duration. This event is consistent with the 'Blue Book' of Structural Engineers Association of California.
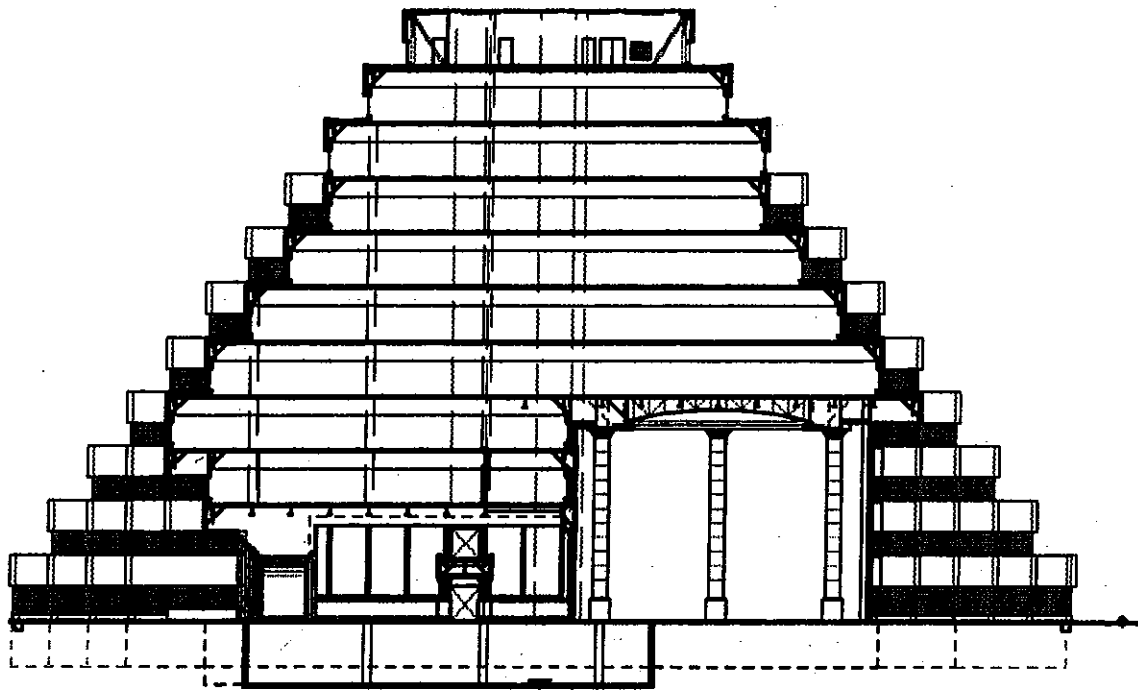
Site-Specific Seismic Hazard Analysis

The geotechnical consultant (Wallace-Kuhl) performed a site-specific ground motion study for this site. Historically, the most significant ground shaking activity occured during the Vacaville-Winters event in 1892. The Richter magnitude of the earthquake was 6.75, and the epicenter was located approximately 22 miles west of the site.

Deterministic and probabilistic analyses were performed to estimate the Peak Ground Acceleration (PGA). These analyses revealed that a 0.17g DBE rock site acceleration would occur from an event of 6.5 magnitude on the Dunnigan Hills fault. Since the maximum credible
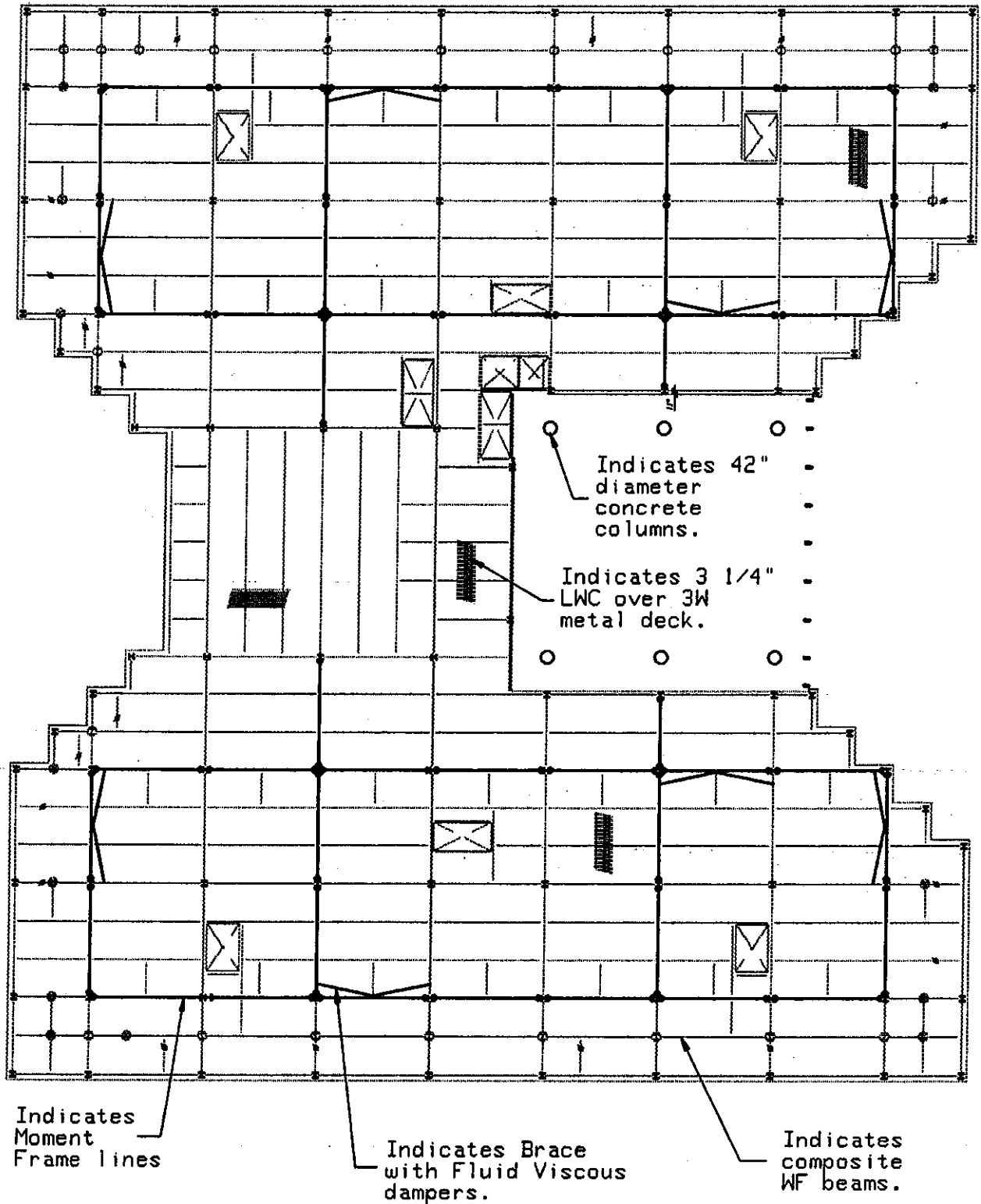
WEST EXTERIOR ELEVATION

NORTH — SOUTH SECTION

FIG. 1

Indicates 42"
diameter
concrete
columns.

Indicates 3 1/4"
LWC over 3W
metal deck.

Indicates
Moment
Frame lines

Indicates Brace
with Fluid Viscous
dampers.

Indicates
composite
WF beams.

THIRD FLOOR FRAMING PLAN

FIG. 2

earthquake (MCE) for this fault has been estimated to have a magnitude of 6.5 to 6.75 by the California Division of Mines and Geology, the DBE and MCE are approximately equal in magnitude.

Actual California earthquake time histories (Coalinga, 1983; Whittier, 1987; Northridge, 1994) were utilized to develop site-specific response spectra. These ground accelerations were selected based on events 15 to 25 miles from the recording station, at an alluvium underlain station, and from an event with a similar fault mechanism (thrust faults).

Each orthogonal pair of time histories were scaled to reflect a PGA, and each of the six time histories were then processed using the computer program SHAKE. The SHAKE program computes the response of horizontally layered soil deposits subjected to vertically propagating shear waves. The analysis revealed a 0.38g PGA for the site-specific response spectrum. Finally, the synthesized time history was calculated from this response spectrum for design purposes (Gius, 1995) (see figure3).

Lateral Force Resisting System

Numerous lateral systems were considered to satisfy the design requirements set forth by the owner and the design team. Conventional shear wall, as well as eccentric and concentric braced frames were rejected because they caused two problems: 1) the extent of the structural and non-structural damage after a seismic event was potentially great, since stiffening the building caused the natural period of the structure to shift to the high acceleration spectra range, and 2) the amount and location of shear walls and braces would interfere with tenant improvements. Base isolation was also rejected as a potential solution because 1) the additional cost was greater than 5% of the total construction cost, and 2) the effectiveness of the isolation was found to be not substantial for this long period building.

Steel moment frames with Fluid Viscous Dampers (FVDs) were selected as the best alternative for the following reasons (see Figure 4):
1) The additional cost for FVDs was less than 1% of the total construction cost.
2) The natural period of the structure was kept out of the high acceleration spectra range, yet displacement was controlled by FVDs.
3) The choice of location for mounting the FVD braces was more flexible than conventional braces, since the force in FVDs is out of phase with the frame forces.
4) Plastic hinge formation within structural elements and connections were prevented.

More than 500 welded moment connections are distributed throughout this structure for increased redundancy. The design team and the owner agreed to reinforce the 6th and 7th level connections with cover plates to provide ductility, while the other welded connections remained non-reinforced. This decision was based on the following:
1) The stress level of the connections are less than 50% of yield, and the story drift ratio is limited to less than 0.005 for the DBE/MCE events. According to the latest research data, damage to welded connections would not take place for this stress and drift level (FEMA 267, 1995).
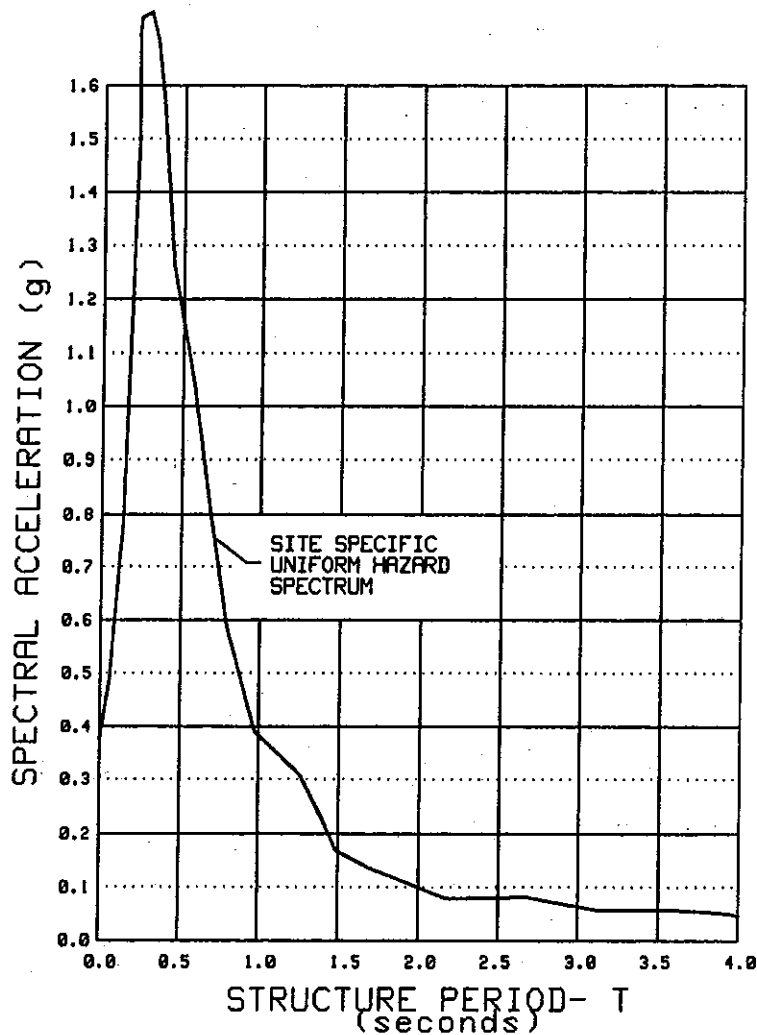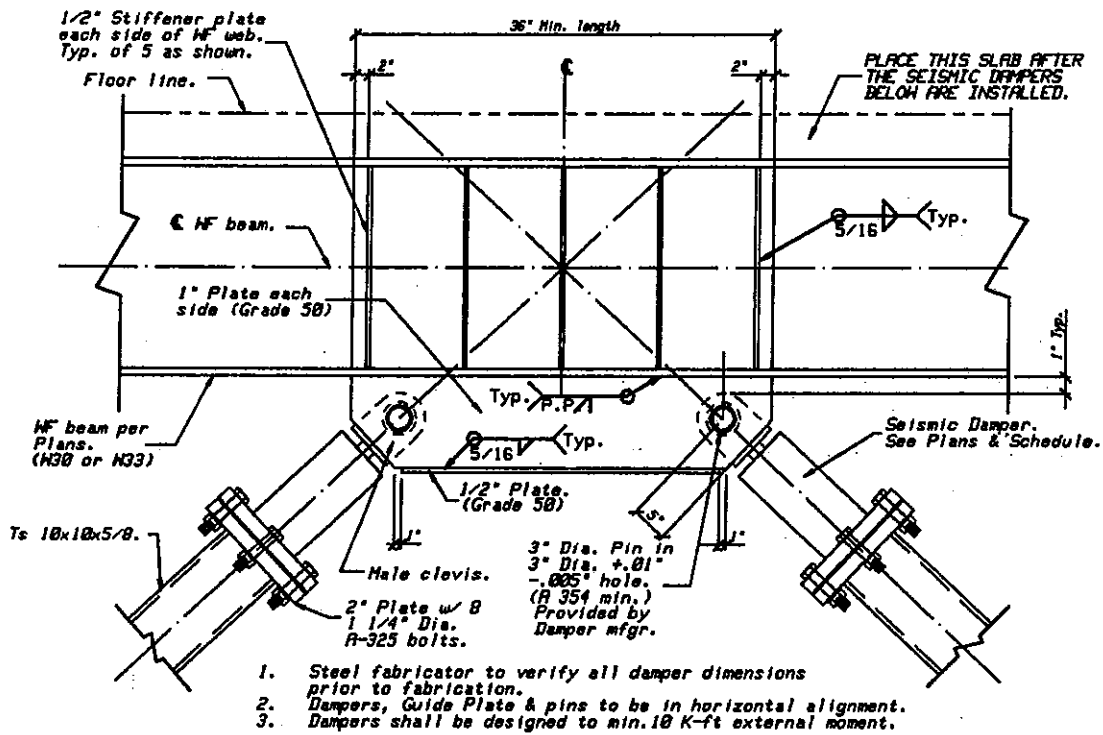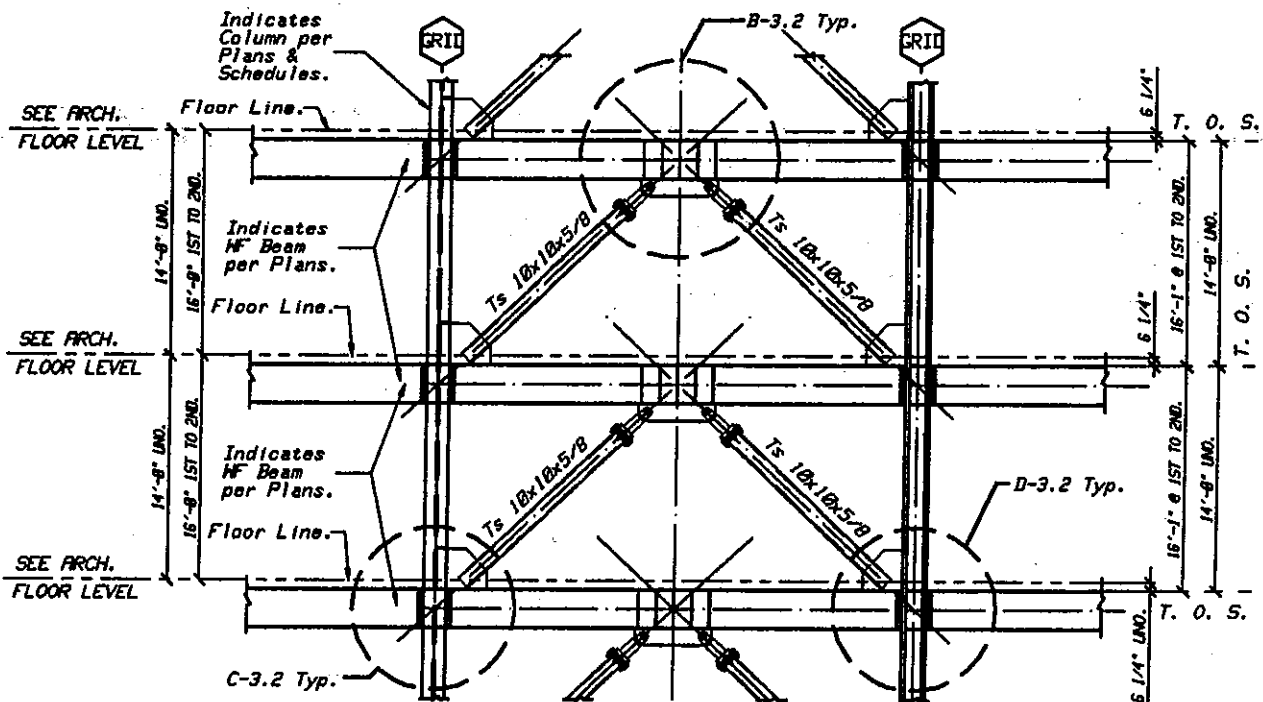
DBE/MCE Site Specific Spectra. (5% Damped).
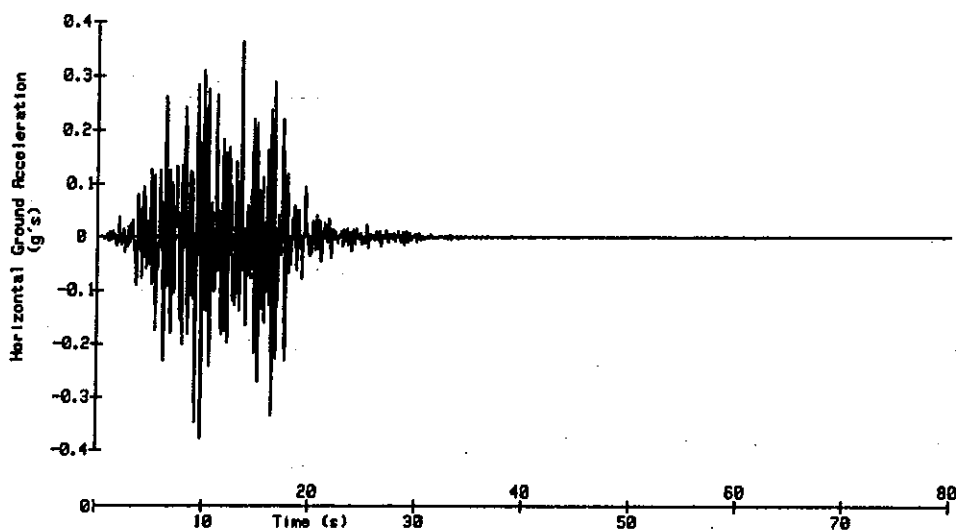
DBE/MCE Synthetic Time History for Site Specific Spectra.

FIG. 3

B-3.2   SEISMIC DAMPER DETAILS

A-3.2   TYPICAL MOMENT FRAME & FVD ELEVATION

FIG. 4

2) The 6th and 7th level connections reached 80% of yield at DBE/ MCE events. The cover plates are provided for extra safety factor.
3) Based on the design team's recommendations, the owner decided that reinforcing all connections would increase the cost substantially without any benefit to the structure's behavior and performance during the DBE/MCE events.

Fluid Viscous Dampers

Fluid Viscous Dampers were selected over other damping devices for several reasons. Since FVDs are velocity-dependent systems, the forces are out of phase with the axial loading of the columns, and the change in the natural period of the building is insignificant. Additionally, the long history of military application proved the system's reliability.

Fluid Viscous Dampers operate on the principle of fluid flowing through orifices (see Figure 5). A stainless steel piston travels through chambers filled with silicone oil, which flows through an orifice in/around the piston head. During a seismic event, the seismic energy is transformed into heat, which dissipates into the atmosphere. The orifice construction utilized in FVDs is similar to that of classified applications for the U.S. Armed Forces, and is considered state-of-the-art (Constantinore & Symans, 1992).

Design Procedures

Step 1:
In order to expedite the plan approval process and determine a design starting point, steel moment frames were designed to conform to ordinary steel moment frames, as specified by the 1994 UBC, discounting the effect of FVDs.

The studies previously conducted by the authors have indicated that the required frame member sizes with 15-20% critical damping were approximately equal to UBC member sizes designed for an Rw = 6 to Rw = 12. Code design criteria is as follows:
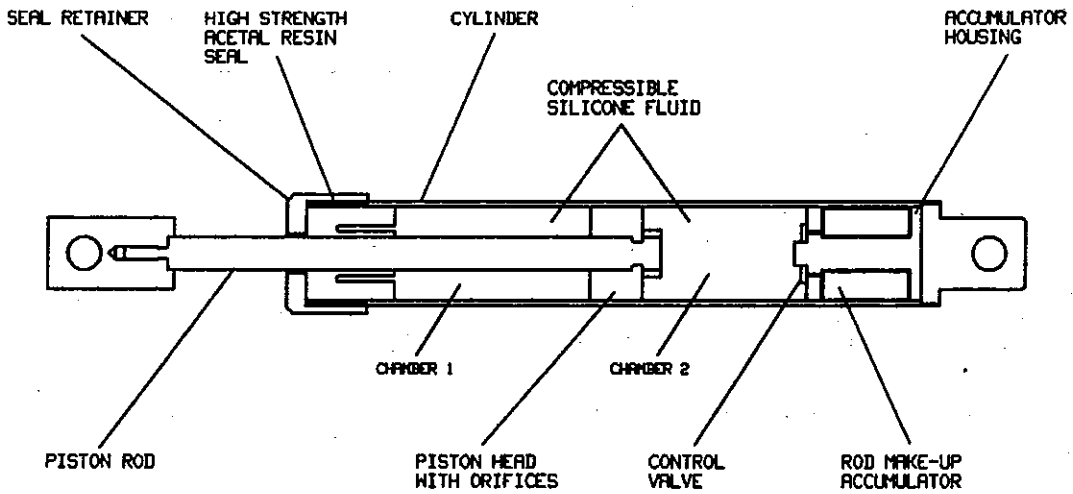1) Seismic zone = 3
2) Rw = 6
3) Three dimensional dynamic analysis
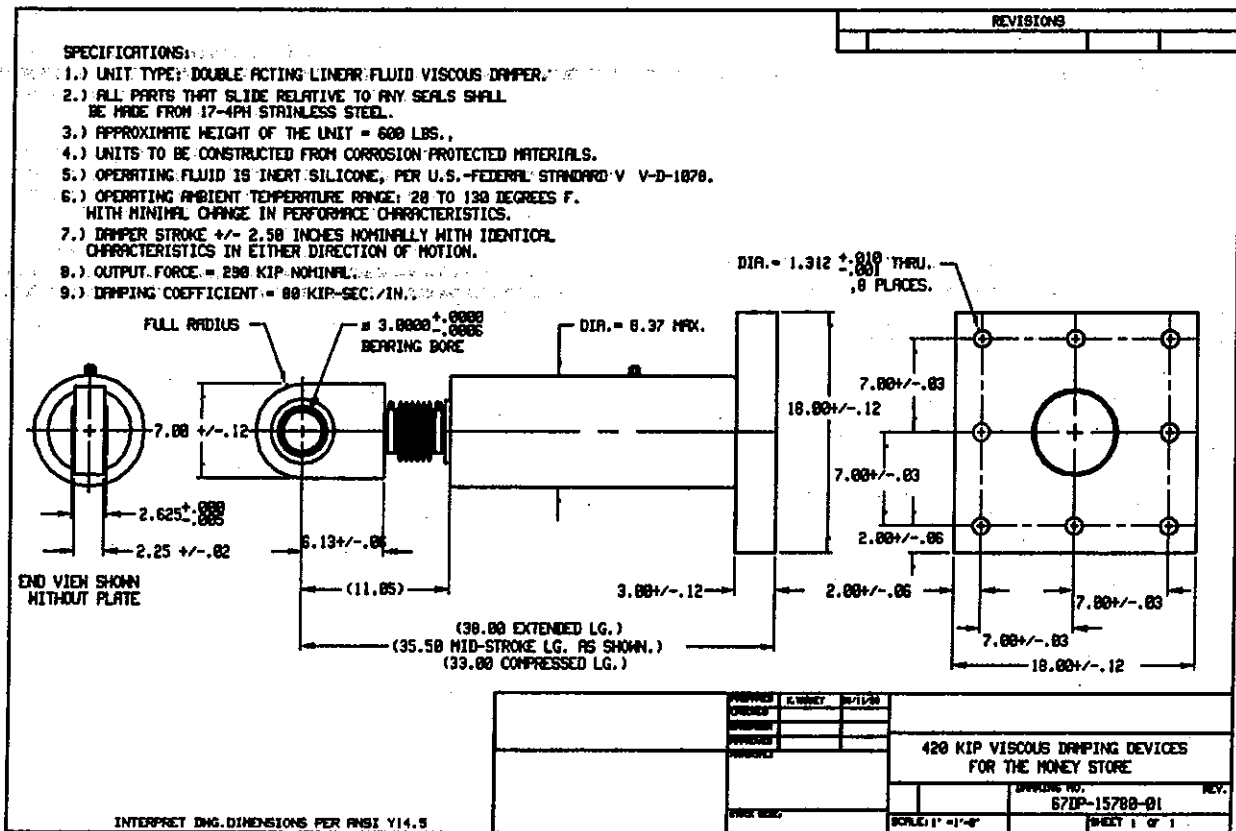4) Maximum allowable drift ratio = 0.004

Step 2:
Using member sizes determined in step 1, time history analyses were performed to study the effects of FVDs.

FVD design criteria is as follows:
1) Maximum allowable drift ratio = 0.005
2) All members, connections, and foundations are to remain elastic using the following L.R.F.D. strength factors:
1.2 DL + 0.5 LL +/- 1.0 EQ
0.9 DL +/- 1.0 EQ

CONSTRUCTION OF FLUID VISCOUS DAMPER

Labels for top diagram:
SEAL RETAINER
HIGH STRENGTH ACETAL RESIN SEAL
CYLINDER
COMPRESSIBLE SILICONE FLUID
ACCUMULATOR HOUSING
PISTON ROD
CHAMBER 1
CHAMBER 2
PISTON HEAD WITH ORIFICES
CONTROL VALVE
ROD MAKE-UP ACCUMULATOR

SPECIFICATIONS:
1.) UNIT TYPE: DOUBLE ACTING LINEAR FLUID VISCOUS DAMPER.
2.) ALL PARTS THAT SLIDE RELATIVE TO ANY SEALS SHALL BE MADE FROM 17-4PH STAINLESS STEEL.
3.) APPROXIMATE WEIGHT OF THE UNIT = 600 LBS.,
4.) UNITS TO BE CONSTRUCTED FROM CORROSION PROTECTED MATERIALS.
5.) OPERATING FLUID IS INERT SILICONE, PER U.S.-FEDERAL STANDARD V V-D-1078.
6.) OPERATING AMBIENT TEMPERATURE RANGE: 20 TO 130 DEGREES F. WITH MINIMAL CHANGE IN PERFORMANCE CHARACTERISTICS.
7.) DAMPER STROKE +/- 2.50 INCHES NOMINALLY WITH IDENTICAL CHARACTERISTICS IN EITHER DIRECTION OF MOTION.
8.) OUTPUT FORCE = 290 KIP NOMINAL.
9.) DAMPING COEFFICIENT = 80 KIP-SEC./IN.

420 KIP VISCOUS DAMPING DEVICES
FOR THE MONEY STORE
DRAWING NO.
670P-15788-01

INTERPRET DWG.DIMENSIONS PER ANSI Y14.5

FLUID VISCOUS DAMPER

FIG. 5

3) The FVD capacity and its connections are designed for 1.5 times the design force.
4) The allowable damper displacement is 2.0 times the design displacement.

Two different mathematical models of the building were constructed to study the effect of FVDs. One was a simple 2-dimensional stick model, and the other was a complex 3-dimensional finite element model (see Figure 6).

The stick model was used to determine the most effective amount of damping, and to check the 3-dimensional model results. The 3-dimensional model was used to: 1) determine the most effective location of the FVDs, 2) obtain member forces, and 3) study cross coupling between stiffness of braces and FVDs.

Time History analyses were performed using ETABS 6.04 (CSI, 1994) which utilizes the step-by-step linear acceleration method. All FVDs were modeled as discrete link elements in order to study the interaction between the moment frames and the FVDs.

Response Performance of the Structure

The fundamental period of the building in both directions was 2.2 seconds. Approximately 15% of the critical damping was provided by FVDs at each story. The typical column sizes were W14x370 and W14x398. The typical beam sizes were W30x116, 124, 132, and W33x141. A total of 60 FVD assemblies (bays) with 120 FVD units were distributed throughout the stories. The damping constant for each FVD was 40, 60, and 80 kip second/inch. The design force was 160 kip and 290 kips. The exponential constant was set as a unity, which produced perfect linear viscous behavior.

The maximum story displacement and story shear for the DBE/MCE are shown graphically below for 2% critical damping and 15% critical damping by FVDs.
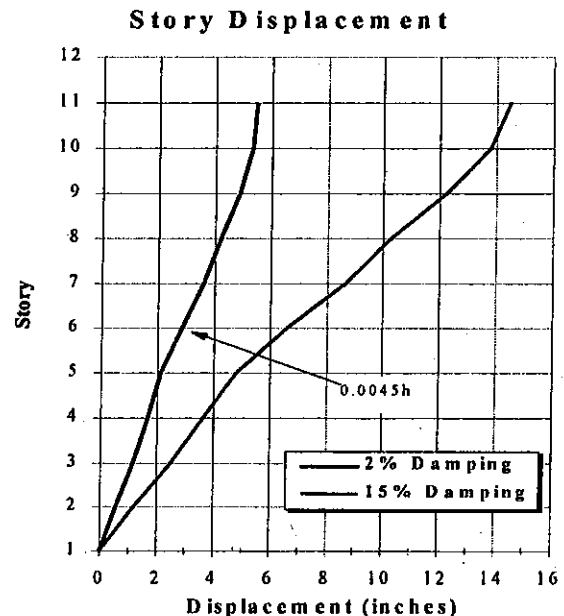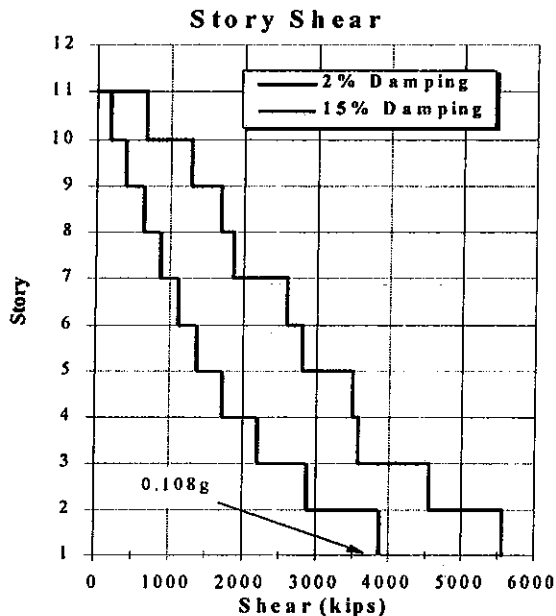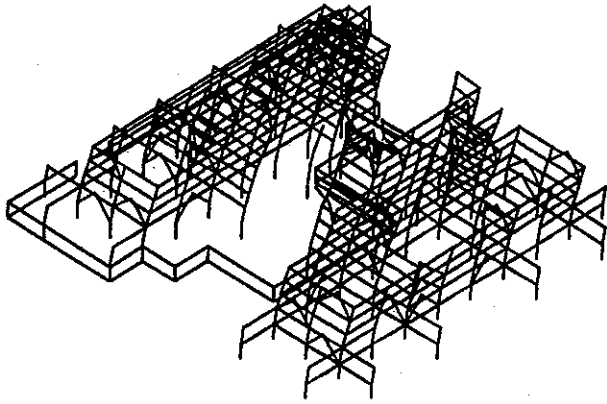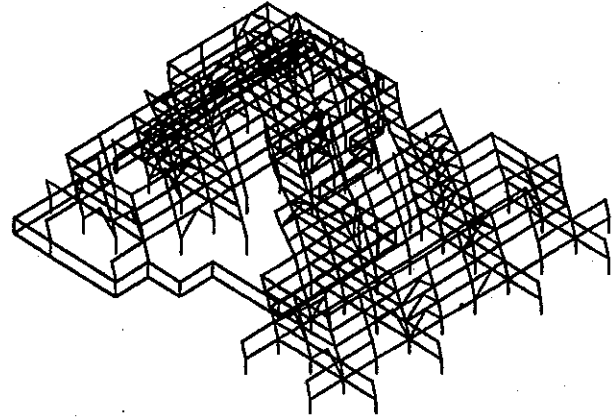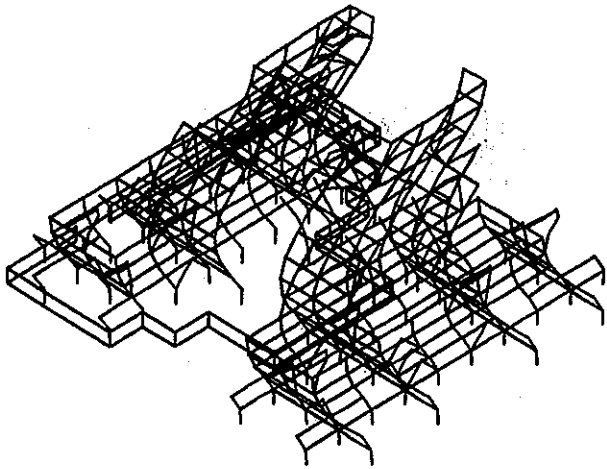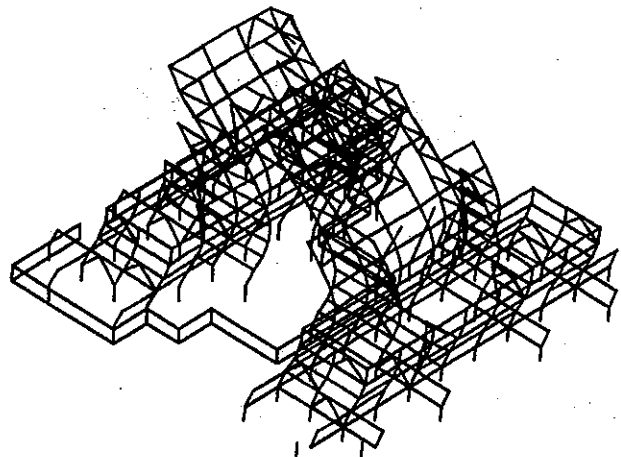
MODE 1 (T_1 = 2.2 Sec.)   MODE 2 (T_2 = 2.2 Sec.)

MODE 3 (T_3 = 0.97 Sec.)   MODE 4 (T_4 = 0.94 Sec.)

3D COMPUTER MODEL OUTPUTS

FIG. 6

Figures 1 and 2 shows that by providing FVDs, base shear was reduced by 30%, and displacement was reduced by 60%. Maximum inter story drift was limited to 0.0045, and all structural members remained well below yield level for the DBE/MCE event.

Cost Analysis

Total structural construction cost, including structural steel, FVDs, metal deck, concrete, and foundations was approximately $10.3 million (1996, present) which equals $23.00 per square foot. The above figure satisfied the construction cost requirement, which was not to exceed that of a minimum code-conforming building. Constuction was set to start in April, 1996.

Time History Analysis for UBC Zone 4 and Northridge Event

In addition, the structure was subjected to the following ground motions:
1) A synthesized time history that matched the UBC Zone 4 , S2, response spectra.
2) Recorded time histories from the 1994 Northridge event.

Using the previously described design procedure, moment frames were redesigned to conform to special steel moment frames of UBC Zone 4. Typical beam sizes are W33x118 and 152. All columns are W14x426. Approximately 20% critical damping was provided at each story by FVDs.

The following are the results of the time history analyses:

Table 2
Maximum Values (T_1 = 2.0 sec)

Time History Station	Story Drift Ratio	Base Shear Coefficient (G)	Column Stress Ratio	Beam Stress Ratio
UBC Spectra	0.0079	0.20	0.79	1.00
Downey, 360°	0.0015	0.05	0.31	0.38
Moorpark, 180°	0.0022	0.08	0.37	0.39
Newhall, 90°	0.0072	0.21	0.72	0.98
Sylmar, 360°	0.0150	0.35	1.44	1.88
Santa Monica, 90°	0.0072	0.16	0.74	1.00
UCLA, 360°	0.0047	0.09	0.49	0.69
Tarzana, 90°	0.0130	0.35	1.20	1.87

Table 2 shows that plastic hinge formations and structural damage were expected for the Sylmar and Tarzana stations. This particular study showed that pure elastic behavior of the building may be difficult for Zone 4 regions. Therefore, the ductile connections are recommended for steel moment frames located in Zone 4. The interaction between plastic hinge formations and FVDs is being studied by the authors at this time.

Conclusion

Using a combination of steel moment frames and FVDs proved to be the most cost effective way of resisting the seismic forces. It eliminated the plastic hinge formations in the structure, thereby reducing the uncertainty in the structure's behavior. Using seismic energy dissipator units, the seismic design process should become more logical and scientific, which is the basis of performance-based design.

Acknowledgements

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