VIRTUAL BASE ISOLATION BY BUILDING SOFTENING WITH DRIFT CONTROL PROVIDED BY FLUID VISCOUS DAMPERS

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ABSTRACT

Large mass and high story heights are common requirements for many data storage and collocation facilities. These building characteristics, which are typically considered design obstacles, actually provide a unique opportunity for reducing seismic response through behavior similar to that of a base isolated building.

In accordance with the 1999 SEAOC Blue Book (SEAOC, 1999) recommendations for passive energy dissipation, the building's Lateral Force Resisting System (LFRS) is designed for strength requirements only, resulting in a relatively flexible LFRS, while Fluid Viscous Dampers (FVD) are incorporated to limit story drifts to acceptable levels. Due to the high building mass, large story heights of 18'-0" (5.5m), and long period LFRS, the building exhibits a fundamental period of 1.45 seconds, compared to 0.5 seconds for a typical two-story moment frame building. The long period LFRS emulates the response of a traditional base isolated system by reducing the acceleration on the building and its contents, while story drifts are controlled by FVD.

There are many benefits to this "virtual isolation" system and incorporation of the SEAOC Blue Book recommendations. With the elimination of the maximum drift requirements, the moment frames are substantially lighter than a traditionally framed building, thus lowering the structural steel cost of the LFRS. The long period structure also produces significantly reduced forces in the foundation elements. Velocity and displacement are reduced significantly through the use of the FVDs, which protects the sensitive contents of the building. These benefits lead to a reduced response resulting in an enhanced performance level during a major seismic event.

Introduction

Several common structural design requirements for a facility housing high tech equipment are low floor displacements, low floor accelerations and immediate occupancy status after a major seismic event. When subjected to high ground accelerations, traditional LFRS are unable to achieve both low floor displacements and accelerations. A true enhanced performance level, such as an immediate occupancy state following a Maximum Capable Earthquake (MCE), is also very difficult to obtain with a traditional structural system since it relies highly on nonlinear behavior of the LFRS. The incorporation of FVD into the LFRS provide a drastic reduction in nonlinear behavior while maintaining low floor displacements and accelerations during a major seismic event.

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Many high-tech facilities house a large amount of heavy equipment, which result in a very high building seismic mass. Large story heights are also often required to allow for the installation of mechanical and electrical support systems. A large mass building with large story heights can be a major design obstacle for a traditional LFRS in a region of high seismicity. The use of FVD can be a very efficient way of countering these design obstacles. The 1999 SEAOC Blue Book states that when incorporating FVD, the LFRS shall be designed for strength requirements only, with no limitation on drift demands. The FVD are in turn added to control drifts to acceptable levels. This approach is intended to allow for a higher-period building, with corresponding lower force demands, while maintaining acceptable drift limits by the addition of dynamic stiffness through the use of FVD. This case study showed that this design philosophy is very effective in producing buildings that are capable of achieving an immediate occupancy status after a MCE event.

Building Description

The building is a two-story, 165,000 square foot (15,325 m^2) data storage facility in Santa Clara, California. The building is intended to house sensitive computer equipment, large numbers of batteries, typically near the perimeter, weighing in excess of 400psf., a large number of mechanical units resting atop a roof mechanical platform, and extensive electrical conduits. The seismic mass of the building is approximately three times that of a typical office building. The building has relatively high story heights of 18'-0". The developer required a seismic performance level of an immediate occupancy state following a Design Basis Earthquake (DBE), which exceeds the requirements of the 1997 Uniform Building Code (UBC) (ICBO, 1997), for an "essential services" facility. In addition, low floor displacements and accelerations were required to protect the sensitive equipment housed on the second floor and roof mechanical platform.

The LFRS consists of 56 total bays of two-story special moment resisting frames (SMRF), utilizing Reduced Beam Sections (RBS) at the connections. A total of 104 nonlinear FVD with a 400 kip. capacity were incorporated along the perimeter of the building, within two-story "X" braced frames. The fundamental period of the building is 1.45 seconds, resulting in a code prescribed base shear of 0.052g.

More traditional LFRS were not chosen for various reasons. A concrete shear wall system was not chosen because high floor acceleration and associated potential equipment damage was a concern of the client. A traditional braced frame system was eliminated as an option due to the inherent high floor accelerations, as well as the need for deep foundation elements to resist overturning forces. In addition, it is extremely difficult, if not impossible, to achieve the enhanced performance levels desired by the client, with these traditional systems which rely on inelastic behavior. Base isolation, though theoretically a viable option, was not selected due to the high cost associated with designing the system to the stringent requirements of the UBC.

Input Time Histories

The building is located 16km from the Hayward fault and 17km from the San Andreas fault, within a region of very high seismic activity. A site-specific probabilistic seismic hazard analysis (PSHA) was performed to estimate the magnitude of ground acceleration at the site. The PSHA modeled the faults in the Bay Area as linear sources and assigned earthquake activities to the faults. Site-specific spectra at the ground surface were estimated using stiff soil attenuation relationships consistent with the subsurface conditions encountered at the site. (Gouchon, 2000) DBE is defined as a 500 year return event, and MCE is defined as a 1000 year return event. Spectral matching was performed to provide appropriate time histories for both DBE and MCE levels. Site specific response spectra for a 5% damping are shown in Figure 1, along with corresponding UBC response spectra graphs. Time histories were chosen based on similarities in magnitude and distance to the target spectra. Three earthquakes were incorporated for each level of seismic hazard (6 total). The worst case results for acceleration, velocity and displacement were used in design of the LFRS. The time history values are shown in Table 1.

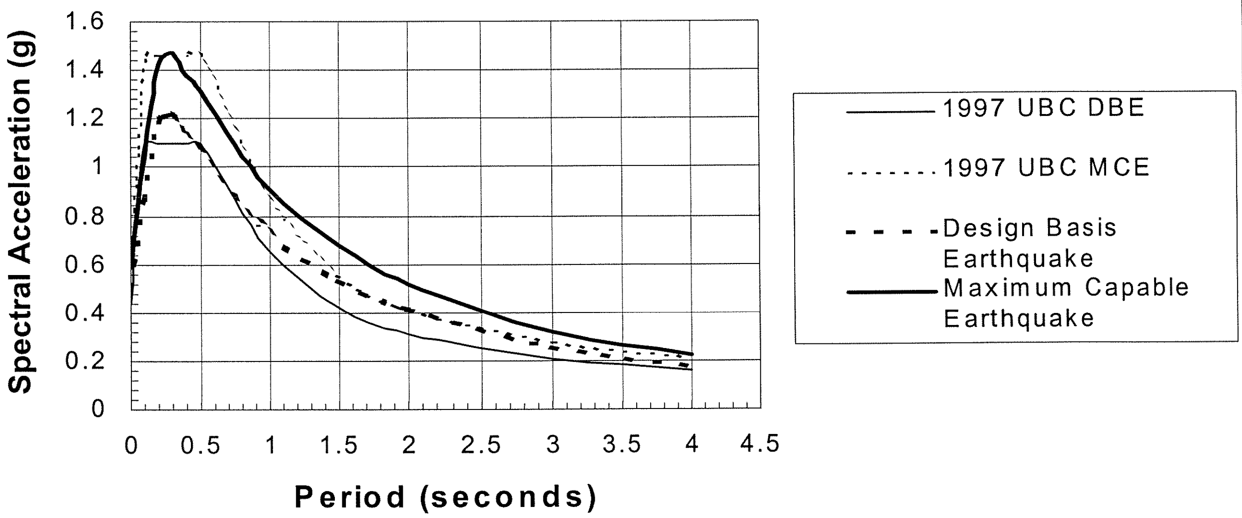

Figure 1. Response spectra graphs for 5% damped building

Table 1. Time history values

Earthquake	Time History	Magnitude	Hazard Level	Epicentral Distance (km)	Peak Acceleration [g.]	Peak Velocity [in./sec.]	Peak Displacement [in.]
Landers	Joshua Tree	7.4	DBE	15	0.504	28.20	9.12
Loma Prieta	Los Gatos	6.9	DBE	23	0.464	29.64	10.08
Imperial Valley (1979)	Differential Array	6.6	DBE	26	0.492	34.56	14.76
Kocaeli	Duzce	7.4	MCE	90	0.606	36.36	14.28
Loma Prieta	Los Gatos	6.9	MCE	23	0.575	37.08	12.96
Landers	Yermo	7.4	MCE	84	0.556	29.64	13.32

Virtual Isolation

The relatively flexible moment frames emulate a traditional base isolation system. In a traditional base-isolated structure,

> The system decouples the building or structure from the horizontal components of the ground motion by interposing structural elements with low horizontal stiffness between the structure and the foundation. This gives the structure a fundamental frequency that is much lower than both its fixed-base frequency and the predominant frequencies of the ground motion." (Naeim and Kelly, 1999),

The two fundamental advantages of a base isolated system are low acceleration and drift for the superstructure. Both of these qualities are captured in the low frequency moment frame with FVD system. In addition, the performance of a base-isolated system rises with an increase in mass of the ground floor. The large mass of the 2nd floor of the subject project produces an analogous effect on the performance of the LFRS.

Design and Analysis Procedure

A stick model, based on story stiffness and mass, was first developed in order to obtain an initial estimate of the required critical damper properties, including maximum force, damping coefficient and nonlinear damping exponent. Time History Analyses (THA) were used to determine the seismic demand on the structure. The analyses were performed on a trial and error basis, with a final result commensurate with our performance requirements of maximum allowable building drift.

The stick model was then transformed into a two-dimensional model and analyzed in ETABS (CSI, 1999) using the damper properties from the previous model. The number of dampers was selected based on the damping coefficient required to produce an acceptable drift. Additionally, the quantity of dampers was chosen such that a maximum damper force of 400kips was produced under a DBE level event. The 400kip level was deemed an acceptable and economical maximum force level for this structure. The model was analyzed using three DBE time histories [Table 1.], which produced a slight variation of damping properties of the FVDs from the preliminary stick model. The beams and columns were modeled as linear elements, and their demand-to-capacity ratios (DCR) were determined. A maximum DCR of approximately 0.9 was observed. A DCR of less than two for ductile elements is generally considered an acceptable level for immediate occupancy, per FEMA 273 (FEMA, 1997) when using an unreduced ground acceleration on a linear structure.

A true nonlinear model was built using RAM Perform (RAM International, 2000) software to verify the results in the ETABS model. This software has advanced nonlinear modeling capabilities for the nonlinear beam and column elements. The beam elements consisted of rigid end zones from the centerline of the column to the column face, an elastic beam segment from the column face to the centerline of the RBS, zero length plastic moment hinges at the centerline of the RBS and an elastic beam segment between the hinges. The column elements consisted of a rigid end zone at the base, a zero length moment hinge above the rigid end zone, and an elastic element above the hinge. The beam-column joint was modeled as an elastic panel zone element, comprised of four pin-connected rigid links with a rotational spring. The dampers were modeled as a nonlinear viscous element with an elastic bar representing the steel driver brace. LRFD was used for design using a load combination of 1.2D + 1.0L + 1.0E. The model was subjected to both DBE and MCE level time history events. Beam rotations and demands, column rotations and demands, panel zone DCR, damper DCR, and interstory drifts were recorded for worst case DBE and MCE events.

Analysis Results

All elements that had a possibility of experiencing inelastic response, including RBS and column bases, were modeled as deformation controlled elements using nonlinear components. Inelastic limits were checked for these elements based on FEMA 273 requirements. All elements that were expected to remain elastic, including panel zones and portions of beams and columns, were modeled as force controlled elements using linear components. Force levels were checked for these elements based on standard steel design equations with a stress reduction factor of 1.0.

All deformation and force level results corresponded to an immediate occupancy level for both the DBE and MCE level events, surpassing the client's performance requirements. The LFRS remained fully elastic throughout the MCE event, except for onset of yielding that was experienced in several panel zones. Based on FEMA 273 requirements for this LFRS, an immediate occupancy level is achieved if the maximum beam rotation is less than 1.7% and the maximum column rotation is less than 1.6%. A maximum rotation of approximately 0.9% for both the RBS and column bases, occurred during the MCE level event, which indicates an immediate occupancy level was achieved. The dampers were designed to possess a nonlinear exponent of 0.4. This nonlinearity limits the increase in axial force above the design value resulting from the MCE level event. Thus, a maximum DCR of 0.82 occurred in the damper elements. Several of the panel zones had a DCR of approximately 1.1 under the MCE level event. This slight overstress was considered acceptable, since no overstrength was considered when determining the capacity of the panel zones. P-delta effects were checked for gravity columns and proved negligible due to the displacement control provided by the FVD. Refer to Table 2 for a summary of the analysis results.

Comparison to UBC-Designed Building

A three-dimensional model without dampers was built using ETABS and designed solely based on the 1997 UBC, including drift requirements. Due to the drift requirements, the columns required by the UBC were substantially larger than those of the building with FVD. Table 3 shows a comparison of the weight of the LFRS for the building without dampers to the weight of the LFRS for the building with dampers. In addition to the increase in structural steel, the elastic forces on the foundation elements are higher on the UBC building due to the decrease in period of 30%.

Assuming a cost of structural steel to be approximately $1/lb. ($2,000/ton), the total cost of the LFRS for the UBC building is $1,765,000. The dampers for the project cost $929,000. The total cost of the SEAOC building w/ FVD is $2,078,000. Therefore, the increase in the cost of the SEAOC building with dampers is $313,000 or about $1.90/s.f.. This is approximately a 1% increase of total construction cost.

Table 2. Summary of nonlinear analysis results

Deformation Controlled Components	**Force Level**	**Rotation (rad)**	**Performance Level**
RBS "hinge"	DBE	0.0071	Immediate Occupancy
	MCE	0.0088	Immediate Occupancy
Column base	DBE	0.0077	Immediate Occupancy
	MCE	0.0089	Immediate Occupancy
Force Controlled Components	**Force Level**	**Demand/Capacity (DCR)**	**Performance Level**
Beam (not at RBS)	DBE	0.65	Immediate Occupancy
	MCE	0.74	Immediate Occupancy
Column above base	DBE	0.74	Immediate Occupancy
	MCE	0.8	Immediate Occupancy
Panel zones	DBE	0.87	Immediate Occupancy
	MCE	1.09	Immediate Occupancy
Viscous dampers	DBE	0.67	Immediate Occupancy
	MCE	0.82	Immediate Occupancy

Conclusions

For a cost of approximately $1.90/s.f., an immediate occupancy level at an MCE level event was achieved, as opposed to a collapse prevention state for the UBC building. This is a relatively small cost increase for the drastic increase in building performance. Life cycle analyses would show that over the life of the building, the structure with dampers would be less expensive. Therefore, the incorporation of FVD and the associated increase in seismic performance can be very cost effective. This design philosophy could be incorporated into more traditional buildings such as office buildings, commercial buildings, schools and hospitals. For these buildings, the enhanced seismic performance would be very feasible and cost effective.

Table 3. Comparison of UBC building without Energy Dissipation System (EDS) to SEAOC Building with EDS

	UBC Building w/out EDS	**SEAOC Building w/ EDS**
Lateral columns	W14x550	W14x211
Total weight (kips)	*1452*	*557*
Lateral floor beams	W33x116	W33x116
Total weight (kips)	*204*	*204*
Lateral roof beams	W21x62	W21x62
Total weight (kips)	*109*	*109*
Tube braces	-	TS10x10x5/8
Total weight (kips)	-	*175*
Damper gussets	-	1-1/2" Plates
Total weight (kips)	-	*104*
Total weight of struct. steel for LFRS (kips)/(tons)	***1765/883***	***1149/575***

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