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DESIGN OF A NEW STEEL MOMENT FRAME BUILDING INCORPORATING VISCOUS DAMPERS FOLLOWING THE GUIDELINES OF THE 1999 SEAOC BLUE BOOK

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Abstract

The project consists of a two-story 150,000 s.f. data storage facility located in Santa Clara, CA. The site for the project, which lies within a region of high seismicity, is located approximately 16 km from the San Andreas and Hayward faults. The building owner desires an enhanced performance state for the building, exceeding that of the "essential services" criteria of the 1997 Uniform Building Code. The owner also desires reduced accelerations and displacements to protect sensitive equipment housed on the second floor. The mass of the building is approximately three times that of a typical office building with a similar floor plate. The lateral strength and stiffness of the structure are provided by 52 bays of special moment resisting frames (SMRF's). The building's displacement is controlled by 96 – 400 kip nonlinear fluid viscous dampers (FVD's) installed within typical two story "X" braced frames.

In accordance with the Structural Engineers Association of California's 1999 Recommended Lateral Force Requirements and Commentary (Blue Book), when a time history analysis is performed and dampers are incorporated to limit the story drift to an acceptable level, the building's lateral force resisting system (LFRS) is designed for strength requirements only. This design philosophy leads to a relatively flexible building, thus resulting in lower building shear. The dampers reduce drifts to levels that would typically only be attainable with a much stiffer LFRS.

There are several benefits with using this design methodology, including, a decrease in structural steel cost and a decrease in forces on foundation elements, while additionally providing an enhanced performance state 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's are unable to achieve both low floor displacements and accelerations. A truly 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 which relies highly on nonlinear behavior of the LFRS. By concentrating the nonlinearity in the FVD's, there is a drastic reduction in the inelastic behavior of the LFRS. Additionally, the incorporation of FVD's allow for low floor displacements and accelerations during a major seismic event. The Blue Book provides guidelines for the engineer to design a building to capture this design philosophy.

In addition to the above design requirements, many high-tech facilities house a large amount of heavy equipment, which results in a very high building seismic mass. Additionally, 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's can be a very efficient way of countering these design obstacles. The Blue Book 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's. This case study shows that this design philosophy is capable of producing buildings that are able to achieve an immediate occupancy status after a MCE event.

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Recommendations of the 1999 SEAOC Blue Book

The authors of the "Passive Energy Dissipation Systems" section of the 1999 SEAOC Blue Book had a goal of developing a set of design guidelines that would allow a practicing engineer to effectively incorporate passive energy dissipation devices into new and existing structures. The authors were successful in creating guidelines that allow the design engineer to capture a realistic building response through the use of a time history analysis, while avoiding the application of unrealistic safety factors which could tend to limit the effectiveness of the energy dissipation system (EDS). Various building departments have accepted the guidelines, paving the way for a more widespread incorporation of EDS's. The subject project is believed to be the first building with an EDS that was designed and approved using the recommendations of the Blue Book.

A traditional LFRS relies on yielding of elements in order to dissipate the seismic energy. Many LFRS's are designed to support gravity loads in addition to resisting lateral forces. Yielding of gravity load-carrying elements, in addition to being costly to repair, can lead to an unsafe condition within the building. The Blue Book recognizes the benefit of concentrating the inelastic behavior within discrete elements that are capable of experiencing this nonlinearity, while dissipating a relatively large amount of seismic energy. Additionally, these discrete damper elements could easily be removed and tested following a seismic event to determine if any damage had occurred. This design philosophy is a further attempt to uncouple the gravity system from the LFRS thereby limiting damage of the gravity system. (Blue Book, 1999)

The Blue Book allows the engineer, in certain circumstances, to use a static force procedure to design a structure with EDS. The structure must meet a set of eight requirements in order to be designed using the static force procedure. In addition to limits on building geometry and damper properties, one of the more limiting of these requirements is the necessity to design the LFRS to meet the strength and drift criteria of the UBC, independent of the EDS. The design engineer often opts for the time history analysis to avoid this requirement. Use of a time history analysis allows the engineer to design the LFRS for the strength requirements of the UBC, while incorporating the EDS to meet the drift requirements. Since the weight, and therefore the cost, of most moment frame systems is governed by drift limitations, significant savings in the cost of the LFRS can be achieved by utilizing this design methodology.

Dampers properties are determined based on the force encountered during each time step of the time history analyses. The force in the damper is expressed as:

$$F_D = Cv^{\alpha} \tag{1}$$

where:

C = damping coefficient
v = velocity in damper
α = velocity exponent for nonlinear dampers

Based on UBC requirements the damper design must include several time history analyses, described in depth below. The Blue Book further requires the time histories to be scaled based on amplitude and frequency, such that the ordinates of the SRSS spectrum do not fall below 130 percent of the ordinates of the target spectrum, similar to the requirements in the UBC for base isolated buildings. (Blue Book, 1999)

The components of the EDS system, including the dampers, braces and connections, shall be designed to resist the maximum force expected in the element:

$$F_C = \rho_D M_M F_D \tag{2}$$

where:

ρ_D = reliability and redundancy factor

M_M = Maximum capable earthquake response coefficient

The reliability and redundancy factor is based on the amount of damping elements in a given story and is intended to encourage the engineer to utilize a larger number of small force capacity dampers, instead of a few large force capacity dampers. The maximum capable earthquake response coefficient is intended to approximate the maximum force that the component will experience. The EDS shall also be designed to accommodate multi-axis movement based on the geometry of the damper and in-plane and out-of-plane building displacement.

Collectors shall be designed to resist the force resulting from the viscous forces in the FVDs. Viscous forces are maximum at the time of peak velocity, which corresponds approximately to the point of zero displacement. The magnitude of the force will depend on the size, quantity and location of dampers within the building.

The Blue Book recognizes that many design engineers and building officials are not experienced with design of structures that incorporate EDS's. As a result, the analysis, design and construction of the EDS is required to be reviewed by an independent engineering panel that is experienced in the analysis, theory and application of energy dissipation devices.

Testing of energy dissipation devices is specified in the Blue Book. In general, testing is performed to confirm the force-displacement-velocity properties of the dampers and to demonstrate the ability of the damper to withstand extreme seismic excitation. Prototype testing should be performed on two full-size devices of each type and size used in the design. Additionally, prior to installation, production testing of each damper is required to ensure adequate force-displacement-velocity characteristics. (Blue Book, 1999)

Building Description

The building is a two-story, 165,000 square foot (15,325 m^2) data storage facility in Santa Clara, California. The footprint is relatively regular, with one re-entrant corner at the northeast portion [Figure 1]. 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 LFRS consists of 56 total bays of two-story special moment resisting frames (SMRF), utilizing Reduced Beam Sections (RBS) at the end of the beams. 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 [Figure 2]. Due to potential liquefaction at the site, continuous heavily reinforced grade beams were required along each column line. This structural system produced a fundamental period of 1.45 seconds, resulting in a code prescribed base shear of 0.052g.

More traditional LFRS's were not chosen for a number of reasons. A concrete shear wall system was not selected because high floor acceleration and the associated potential for 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. Additionally, it is extremely difficult, if not impossible, to achieve the enhanced performance levels desired by the client, with these traditional systems which rely heavily on inelastic behavior. Base isolation, though theoretically a viable option, was not selected due to the relatively high cost associated with designing the system to meet the stringent requirements of the UBC.

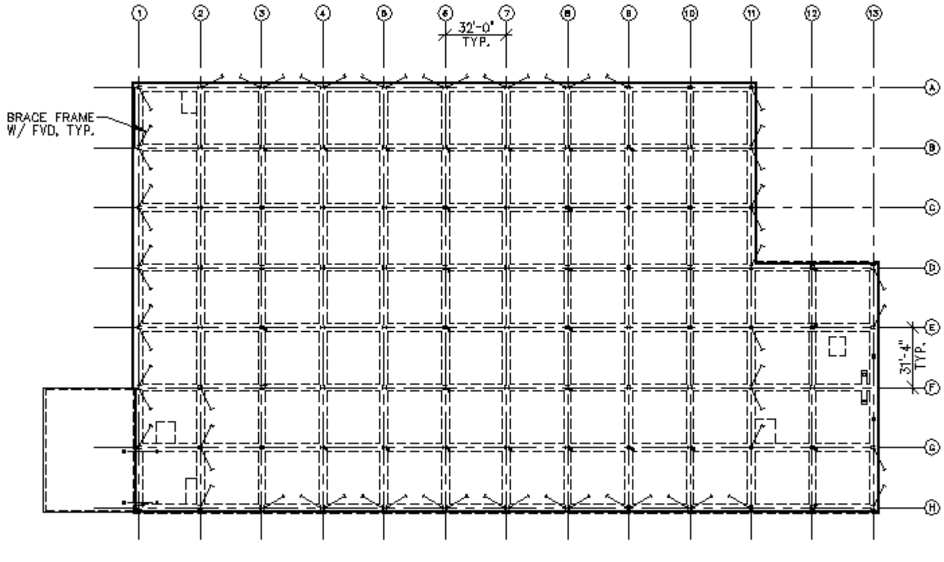

Figure 1. Building foot plan showing location of FVD's.

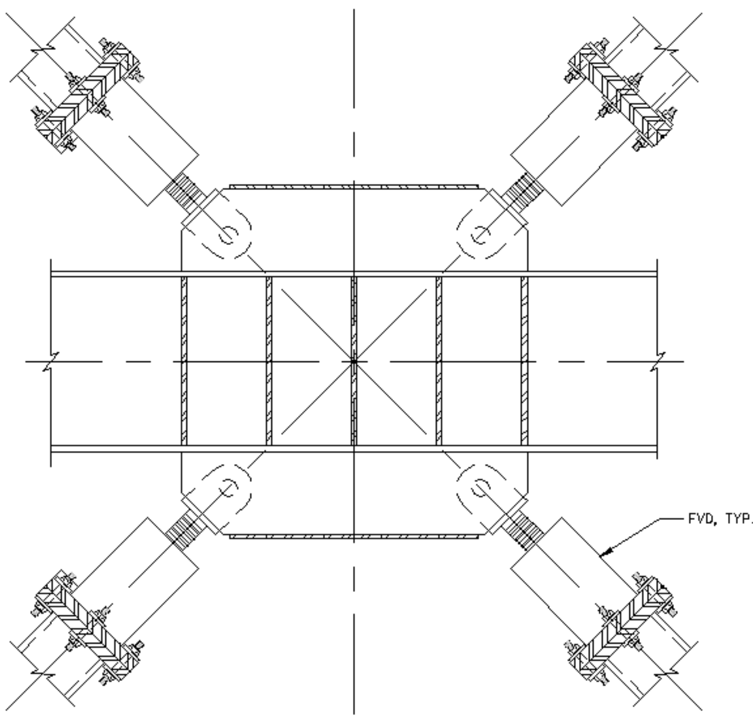

Figure 2. Connection of dampers to wide flange beam.

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 San Francisco 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 3, along with the 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), per the requirements of the UBC. The worst case results for acceleration, velocity and displacement were used in design of the LFRS and EDS. The time history values are shown in Table 1.

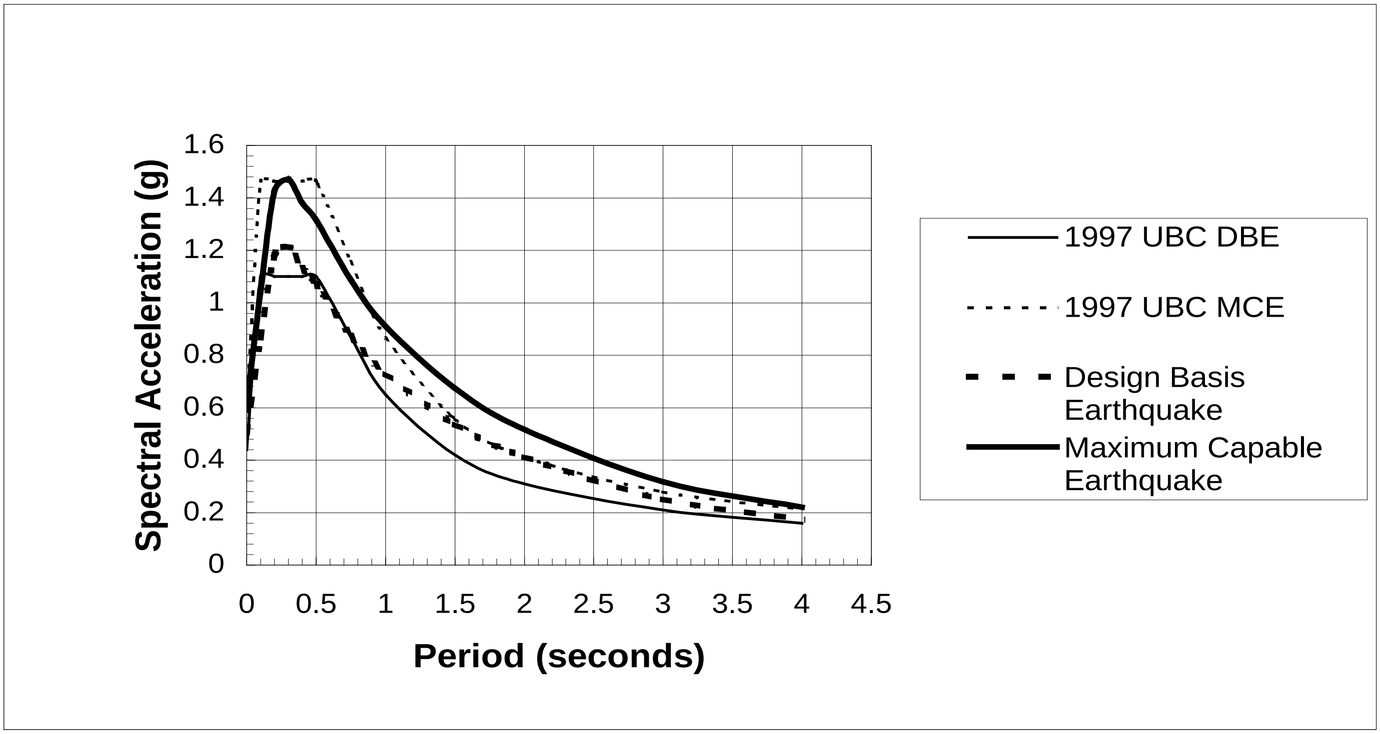

Figure 3. Response spectrum graphs for 5% damped building.

Design and Analysis Procedure

The design of the LFRS and EDS was based on the guidelines of the Blue Book. A full nonlinear dynamic analysis was performed to allow for the benefit of designing the LFRS for strength requirements only. Below is a detailed summary of full procedure, from the initial analysis to the final design. After determining the overall building layout, a preliminary location for the SMRF's was determined. The

Table 1. Time history values

Earthquake	*Time History*	*Magnitude*	*Hazard Level*	*Epicentral Distance [km]*	*Peak Accel. [g.]*	*Peak Velocity [in./sec.]*	*Peak Displ. [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

SMRF's were designed by using conventional computer software, performing a code level design. The SMRF's were designed based on strength requirements only, as indicated above.

A two-dimensional model, based on the story stiffness of the SMRF's and the building mass, was first developed to obtain an initial estimate of the required critical damper properties, including maximum force, damping coefficient and nonlinear damping exponent. Time history analyses 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 nonlinear viscous elements with elastic bars representing the steel braced frames. 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 the 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. 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 Tables 2 and 3 for a summary of the analysis results.

Table 2. Summary of Analysis – Force Controlled Components

Force Controlled Components	*Force Level*	*Demand/Capacity (DCR)*	*Performance Level*
Beam (not at RBS)	DBE	0.65	I.O.*
	MCE	0.74	I.O.
Column above base	DBE	0.74	I.O.
	MCE	0.8	I.O.
Panel zones	DBE	0.87	I.O.
	MCE	1.09	I.O.
Viscous dampers	DBE	0.67	I.O.
	MCE	0.82	I.O.

*I.O. = Immediate occupancy.

Table 3. Summary of Analysis – Deformation Controlled Components

Deformation Controlled Components	*Force Level*	*Rotation (rad)*	*Performance Level*
RBS "hinge"	DBE	0.0071	I.O.
	MCE	0.0088	I.O.
Column base	DBE	0.0077	I.O.
	MCE	0.0089	I.O.

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. The model consisted of the same building geometry and seismic mass. Due to the drift requirements, the columns required by the UBC were substantially larger than those of the building with FVD, which was expected. 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-designed building due to the decrease in period of 30% resulting from the stiffer LFRS.

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 4. Comparison of UBC-Designed Bldg. without EDS to Blue Book-Designed Bldg. with EDS

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

Conclusions

Following the guidelines of the Blue Book, a well-performing building with a reasonable construction cost was achieved. For a cost of approximately $1.90/s.f., an immediate occupancy level at an MCE event was produced, as opposed to a collapse prevention state for the UBC-designed 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. 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 could be feasible and cost effective.

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