Design of a New Steel-Framed Building Using ASCE 7 Damper Provisions

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ABSTRACT

Performance Based Design (PBD) and a system of steel Special Moment Resisting Frames (SMRFs) with Viscous Damping Devices (VDDs) were used for the seismic design of a new multi-story medical building in California. The five-story, 132,000 ft^2 office building is one of the first structures in the United States to apply 2005 ASCE 7 procedure to design with VDDs. In accordance with ASCE 7, the steel frames were sized and designed with strength requirements of the code level force. VDDs were provided to control displacement of the structure. Earthquake performance and cost effectiveness were the primary concerns in designing this building. Site-specific response spectra and spectrum-compatible time histories, synthesized for 500-year and 2,500-year return events, were used for nonlinear response history analysis. Comparison analysis of the PBD design and conventional design (CD) showed that the PBD building had superior seismic performance. PBD lead to a long period, low frequency, structure with low acceleration. VDDs reduced the displacement level to less than a 1% story drift ratio. A cost study shows that much of the VDDs expense is offset by decrease in the weight of the steel members and reduction in foundation costs while providing a far superior performance.

INTRODUCTION

Provisions of ASCE 7 [1] were used to design a new steel framed multi-story building. The $26 million building is part of the expansion of a medical facility located in California. The existing building will be demolished and the new facility built on the site. The facility involves a increase in size intended to accommodate patients and staff. The construction started in 2007 and is expected to span over the next two years. Figure 1 [2] presents an architectural rendering of the building. The building's lateral loading system is comprised of SMRFs, using ductile and laboratory tested beam-to-column connections, and VDDs. SMRFs were designed to provide strength requirements. The drift limitations were met by adding VDDs to the structure. ASCE 7 allows PBD to be used to optimize such design.

ASCE 7 PROVISIONS

Chapter 18 of ASCE 7 details the seismic design requirements for structures with supplementary damping. When using the equivalent lateral load procedure, the base shear can be reduced to 75% size member strength. Site-specific ground motions are permitted to be used to determine the seismic demand. ASCE 7 nonlinear procedure requires preparing a detailed mathematical model of the building that incorporates the damping devices. Nonlinear response history analysis procedure accounting for damper behavior is permitted. The inherent damping in the structure is limited to 5% of critical. When the demand to capacity ratio (DCR) in a member is below 1.5, that member is allowed to be modeled as linear. The response is based on the maxima obtained from a minimum of three pairs of input histories. In such analysis, a strength reduction factor, ϕ, of unity is used to evaluate the response of members. Prior to installation, production tests are required to ensure that the constitutive relation for dampers fall in the acceptable range.

DESCRIPTION OF THE BUILDING

The medical office building under investigation is a five-story structure. The building height measured from the ground to the high roof is approximately 69 ft (21 m); typical story height is 14 ft. (4.3 m). The building floor plan varies from story to story. The total building area is approximately 132,000 square ft. Typical bays measure 30 x 30 ft. describe floor plan. Figure 2 presents schematic of the second floor framing. Gravity loading is resisted by filled lightweight concrete steel decks and steel beams and columns. The steel decking is 3 in. deep and has a topping of 3.25 and 2.5 in. at the floor and roof areas.

FIGURE 1. ARCHITECTURAL RENDERING OF THE BUILDING [2]

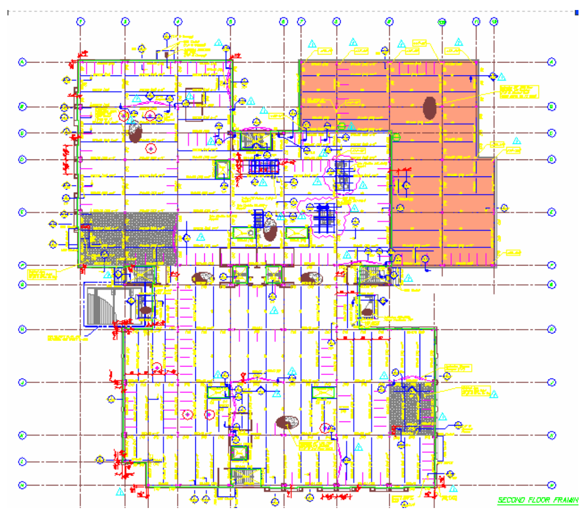


FIGURE 2. PLAN VIEW, SECOND FLOOR

LATERAL LOAD RESISTING COMPONENTS

ASTM Grade 50 SMRFs and VDDs are used for seismic design. The beam-to-column connections for SMRFs use the ductile slotted-web beam design. VDDs are devices, originally developed for the defense and aerospace industries. They are activated by the transfer of incompressible silicone fluids between chambers at opposite ends of the unit through small orifices; see Figure 3 [5]. During seismic events, the devices become active and the seismic input energy is used to heat the fluid and is thus dissipated. The application of VDDs for seismic design of steel SMRFs is one of the recommended practices of the SAC Joint Venture [6] and has been successfully implemented by the authors in both new construction [7] and in seismic rehabilitation [8]. Slotted web connections [9] are proprietary products developed to ensure ductile flexure behavior away from the face of the connection. The webs are slotted to make sure that the flanges only carry normal stresses; the shear force and part of the bending moment is resisted by the web. This eliminates the triaxial state of stress, common to Pre-Northridge connections. Additionally, the separation of beam flanges and web eliminates lateral torsional buckling. Figure 4 shows the details for a typical slotted web connection.

Forty nonlinear VDDs, comprised of ten units in x- and y-directions for the first two floors were used. The VDDs were arranged in the inverted V (Chevron) configuration; see Figure 5. For the first mode, the equivalent-damping ratio produced by the FVDs is approximately 35% of critical.

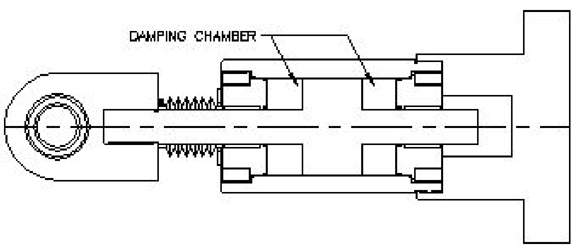


FIGURE 3. SCHEMATIC OF VDD [5]

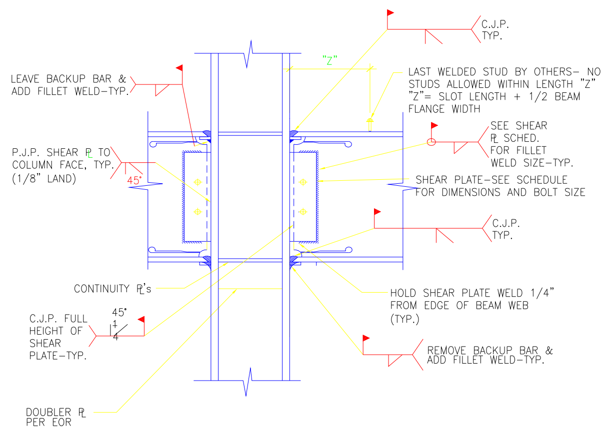


FIGURE 4. TYPICAL SLOTTED-WEB CONNECTION DETAIL [9]

MATHEMATICAL MODEL OF THE BUILDING

Computer program ETABS [10] was used to prepare a three-dimensional mathematical model of the building. The steel beams and columns were modeled using the programs one-dimensional beam-column element. Nominal spans and member sizes as defined in AISC [4] and as specified in the contract, plans were used in analysis. Centerline dimensions were used and no rigid end offsets were specified, and rigid joints were assumed. Two-dimensional shell elements were used to model floor decking. P-Δ effect will be included in the analysis. Figure 6 presents the three-dimensional mathematical model of the building.

Gravity loading on the building consisted of selfweight of members, a uniformity distributed nonstructural load of 20 psf, perimeter wall uniform dead load of 200 plf, mechanical area load of 200 psf distributed over a specific roof area to account for the HVAC and air conditioning units, and uniformity distributed reducible live loading.

In the PBD model, the bases of all columns were modeled as pinned to represent the expected boundary condition. A similar model without the VDDs was prepared to simulate the conventional design. In this CD model, fixed boundary conditions were used.

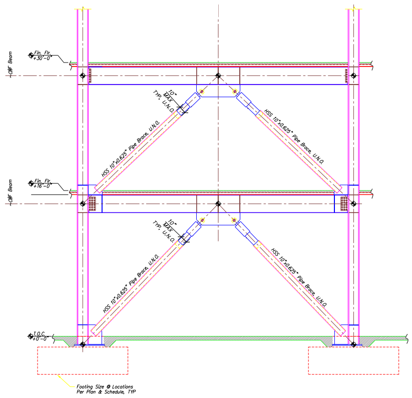


FIGURE 5. DAMPER ELEVATION

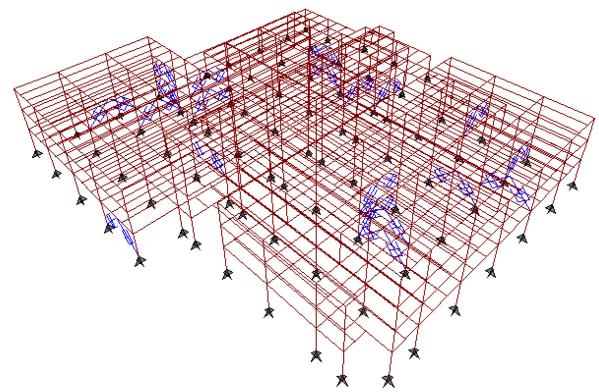


FIGURE 6. MATHEMATICAL MODEL

SEISMIC HAZARD AT THE SITE

Geotechnical investigations [11] were undertaken to prepare the seismic input for the site. Site-specific response spectra for the design basis earthquake (DBE) and maximum considered earthquake (MCE)—475-year and 2500-year return events, respectively, were developed. The DBE event has an intensity that is approximately 2/3 of the MCE event. For each spectrum, three-pairs, fault normal and fault parallel components, of spectrum-compatible records were synthesized using seeds from past earthquake recorded accelerations. These records have a typical duration of 60 sec with 30 sec of strong shaking at a minimum.

PERFORMANCE OBJECTIVES

The steel members were sized using conventional code design procedures [3]. VDDs were sized to control the story drift. Two performance levels were considered. These criteria are consistent with ASCE 07 [1] recommendations: 1) DBE: ensure all steel members of the SMRF remain elastic and limit story drifts to 1%, 2) MCE: keep the DCRs for all SMRF members to 1.5 or less.

MODAL ANALYSIS

Table 1 presents the modal data for the PBD and CD models. For both structures, the fundamental modes were dominant with large mass participation. The translational and torsional modes are uncoupled. The fundamental modes of the PBD structure have longer periods than that of the CD models. Hence, the PBD model will be subjected to smaller seismic demands. Eighteen modes were used for further analyses. This ensured that more than 95% of the total building mass participated in response. The seismic weight of the structure is estimated to equal approximately 12,400 kips.

	Direction	CD	PBD
1	y-	1.50	2.27
2	x-	1.48	2.22
3	θ-	1.06	1.65

TABLE 1. MODAL PROPERTIES OF THE MODELS

RESPONSE HISTORY ANALYSIS

Nonlinear response history analysis was performed to evaluate the response of the building. The VDDs nonlinear force-deformation response was included in the model. The models were first preloaded with ASCE 7 gravity load combinations. For each combination, six DBE and six MCE analyses were performed, two analyses per three records and for different components of the ground motioned aligned with building principal directions. Maximum response quantities such as building floor displacement and drifts, story shears, VDD forces, and member stresses for each analysis was extracted. The maxima of these values were then used for evaluation.

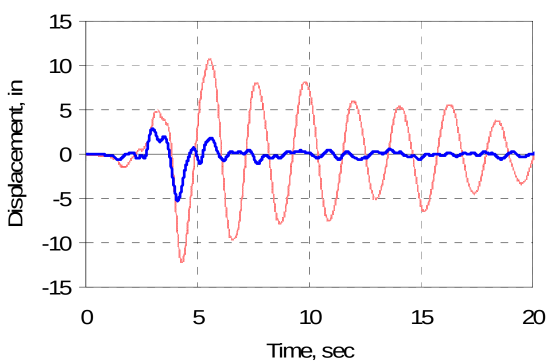
Story Displacements and Drift Ratios

The CD model was designed to satisfy the CBC drift requirements. For the PBD model, dampers are used to control story drifts. Table 2 lists the computed story displacements and drift ratios in %, at various floors. Note that the computed story drifts satisfy the CBC limits. Furthermore, the computed drifts at DBE and MCE levels are below 1.0- and 1.5-percent, respectively. Thus, the drift targets at these performance goals are satisfied.

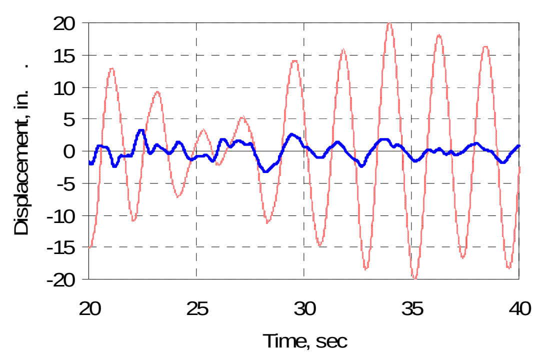
To assess the efficacy of VDDs in controlling story drifts, an additional analysis was performed for which, the VDDs were removed, however, the base of the columns were left as pinned. For this theoretical model, the computed maximum displacements and drift ratios were approximately three times that of the PBD model. Figure 7 depicts a 20-sec trace of the computed displacements for the models. The addition of VDDs reduces the floor displacements significantly, resulting in lower demand on the SMRF members and connections.

	Story displacements, in.				Story drift ratios, %			
	DBE		MCE		DBE		MCE	
Story	x-	y-	x-	y-	x-	y-	x-	y-
Hi roof	5.23	5.59	8.55	9.24	0.7	0.8	1.0	1.4
Roof	4.63	4.80	7.63	7.95	0.7	0.9	1.1	1.4
Fourth	3.25	3.07	5.50	5.50	0.5	0.6	0.9	0.9
Third	2.33	2.28	4.21	4.10	0.5	0.5	0.9	0.9
Second	1.48	1.44	2.68	2.54	0.8	0.7	1.4	1.3

TABLE 2. COMPUTED STORY DISPLACEMENTS AND DRIFT RATIOS, PBD MODEL



a. DBE event (PBD: ____, no VDD: -----)



a. MCE event (PBD: ____, no VDD: -----)

FIGURE 7. DISPLACEMENT RESPONSE OF PBD AND THEORETICAL MODEL WITHOUT VDDS

Story Shears

Table 3 present the computed story shears for the CD and PBD models. The PBD model has much smaller story shears. Figure 8 presents the computed base shear in the x-direction at the MCE level for the two models. The reduction in the base shear for the PBD model is attributed to two factors. First, removing the rotational fixity at the base of columns elongates the period and hence, reduces seismic demand. This is visualized by traveling on the 5%-damped spectra from left to right and then to a region of lower accelerations. Second, the addition of VDDs increases the equivalent damping of the structure and hence the seismic demand. This is visualized by traveling down, at a given period, from the 5%-damped to a highly-damped spectrum with smaller ordinates at all periods.

	CD				PBD			
	DBE		MCE		DBE		MCE	
Story	x-	y-	x-	y-	x-	y-	x-	y-
Hi roof	78	84	121	153	36	37	48	51
Roof	684	710	1080	1270	270	305	466	493
Fourth	1500	1550	2990	3070	640	654	1000	1020
Third	1970	1960	3560	3600	920	920	1340	1330
Second	2540	2510	4000	4470	1270	1260	1830	1720

TABLE 3. COMPUTED STORY SHEARS

Floor Accelerations

The absolute story accelerations at the roof level for the CD and PBD models are presented in Table 4. The PBD accelerations are less than 40% of the CD values. High floor acceleration can damage acceleration-sensitive nonstructural components such as piping and ceilings. Therefore, the application of the VDDs seismically protects both the structural and nonstructural components. The acceleration trace at the center of the roof for the DBE level in the x-direction is presented in Figure 9.

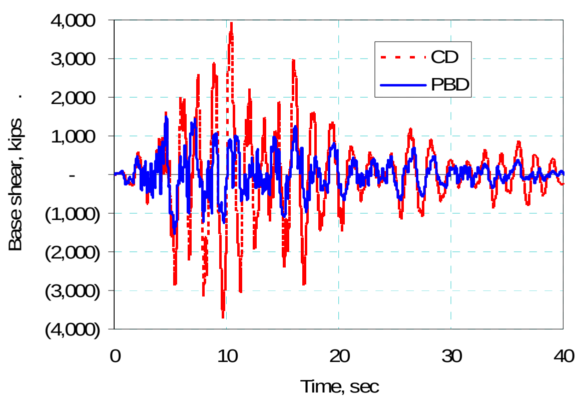


FIGURE 8. BASE SHEAR RESPONSE, MCE

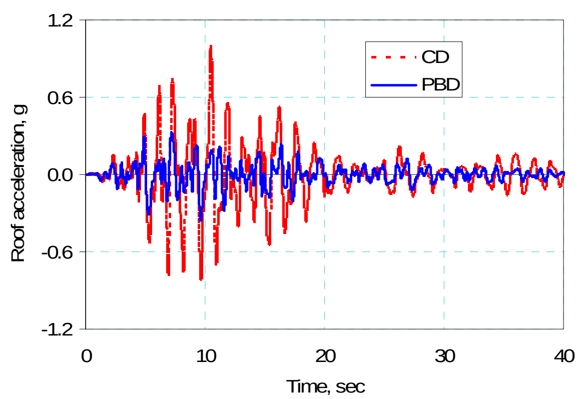


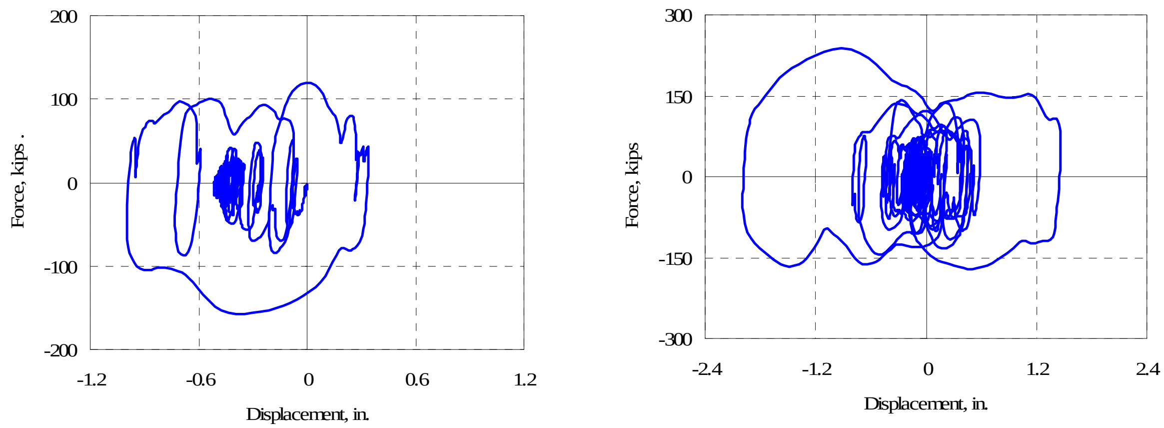
FIGURE 9. ROOF ACCELERATION, MCE

Acceleration, g		
Level	CD	PBD
DBE	0.62	0.23
MCE	1.00	0.36

TABLE 4. STORY ABSOLUTE ACCELERATION

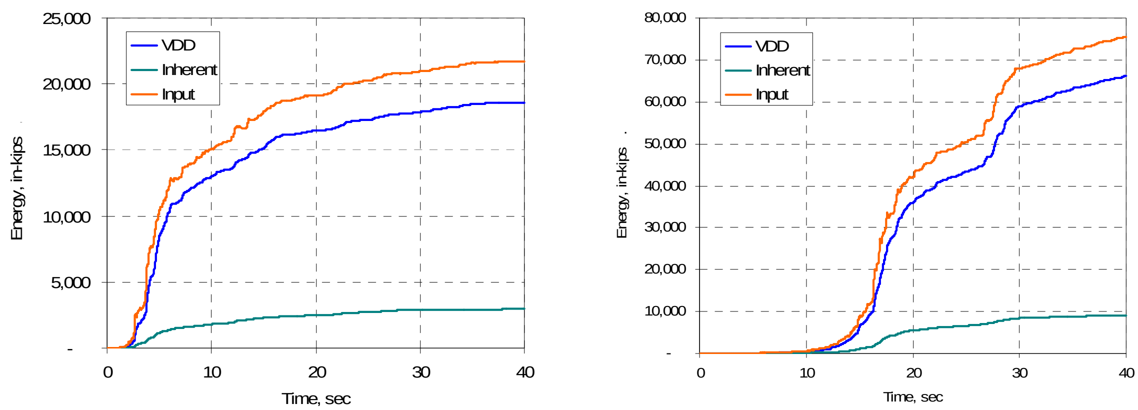
Energy Dissipation Evaluation

Figure 10 presents the axial force-axial displacement hysteresis response of the first floor VDD with the largest response. Significant seismic energy is dissipated by the dampers. As shown in Figure 11, the dampers are effective in resisting the largest portion of this energy. Table 5 lists the maximum force in the FVDs at each floor at DBE and MCE level. The first floor damper design force (DBE) is approximately 160 kips.



a. DBE event a. MCE event

FIGURE 10. VDD HYSTERIC BEHAVIOR, FIRST FLOOR UNITS



a. DBE event a. MCE event

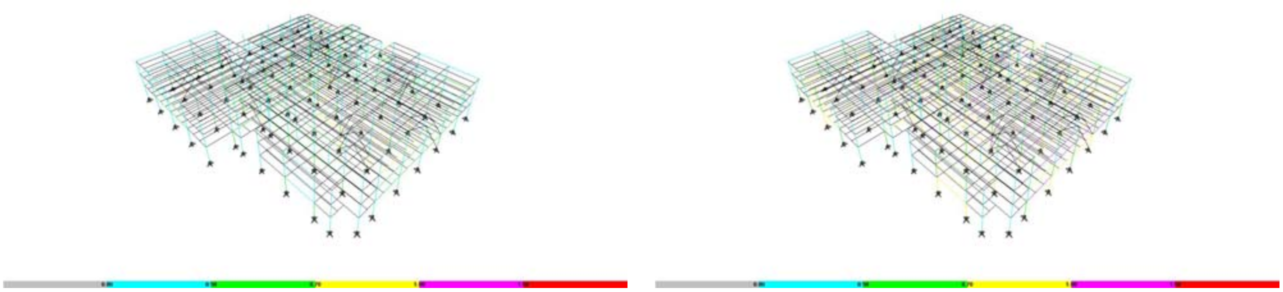
FIGURE 11. COMPONENTS OF INPUT SEISMIC ENERGY

	DBE		MCE	
Story	x-	y-	x-	y-
Second	125	124	185	181
First	158	157	234	228

TABLE 5. VDD FORCES (KIPS)

Steel Member DCR Checks

ETABS steel design utility was used to compute the DCR values for the steel members that form part of the SMRFs and for the gravity columns. For evaluation, DBE and MCE design combinations were un-scaled to account for strength reduction factor, ϕ, of unity. Figure 12 presents the DCR values. At the DBE event, all members have a DCR of less than unity. As such, at this level, the building response is linear and no member yielding occurs. Thus, the first design criterion is satisfied. At the MCE event, all member stresses (DCR) are below the target value of 1.5. Most members have a DCR of less than unity and only a few have a DCR of between 1.0 and 1.5. Thus, the second design criterion is satisfied.



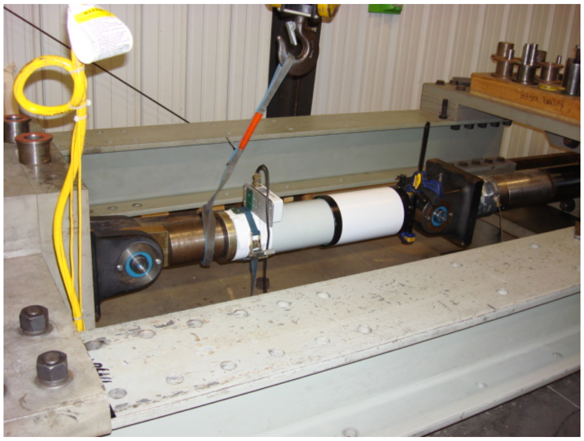
a. DBE event

a. MCE event

FIGURE 12. MEMBER STRESS CHECK

PROTOTYPE TESTS

Prior to construction, prototype tests of the dampers is required to ensure that they have adequate capacity and stroke, to verify the force-velocity relations, and to check the endurance of units for seismic loading. The prototype tests of one damper of each size were conducted by the manufacturer [12]. Figure 13 shows testing of one of those dampers. Laboratory data—see Figure 14—shows that the damper constitutive force-displacement relation closely correlates to the theoretical values used in analysis. The dampers properties are listed in Table 6.



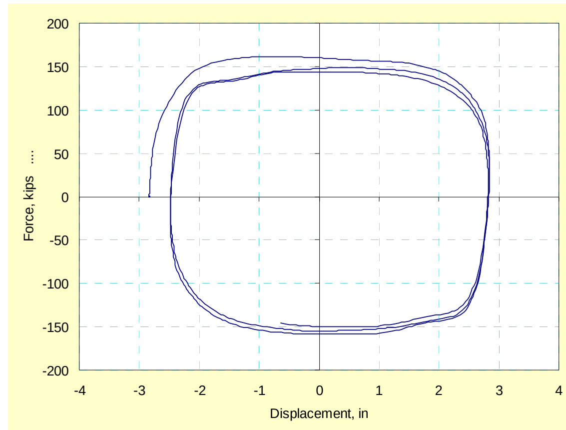


FIGURE 13. LABORATORY TESTING

FIGURE 14. EXPERIMENTAL HYSTERESIS

Floor	No.	Capacity, kips , (DBE)	Stroke, in.	α	C, k-sec/in.
Second	20	135	± 3	0.5	75
First	20	160	± 3	0.5	75

TABLE 6. DAMPER PROPERTIES

PERFORMANCE EVALUATION AND COST COMPARISONS

Table 7 presents relative cost comparison for the CD and PBD buildings. The additional cost of the dampers is offset by the savings in steel tonnage and foundation concrete volume. Hence, the two buildings have similar initial costs. However, the PBD building has superior performance and lower long-term costs. Following a design earthquake, the CD building will provide life safety, but may sustain significant damage. This entails loss of operation and large repair cost. By comparison, the PBD building is expected to be operational and will sustain little damage.

This is qualitatively illustrated in Figure 15. The buildings have similar performances at construction time. Sometime later, a seismic event occurs. This reduces the performance level of the buildings. The degradation for CD building is greater, resulting in larger repair cost and downtime. The long-term relative efficacy of the seismic design is inversely proportional to the shaded areas of Figure 15, which accounts for severity of damage and repair time, i.e., cost and loss of operation. The PBD structure is a more robust design or it has a higher seismic resiliency.

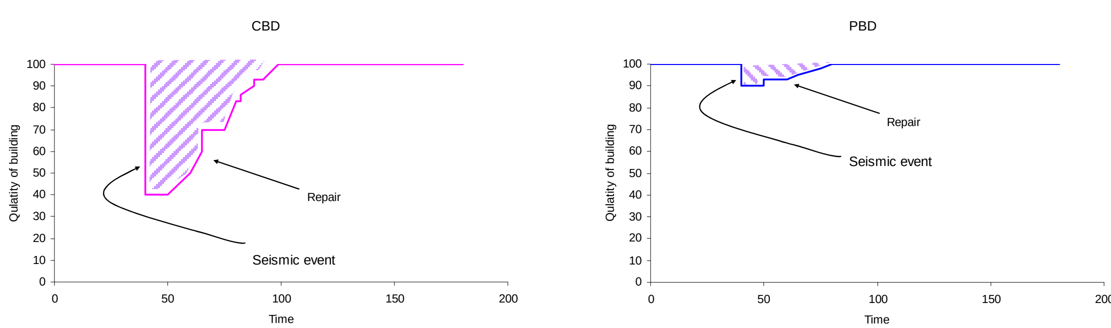


FIGURE 15. QUALITATIVE RESILIENCY CURVES

	CD	PBD	Comments
SMRF members, lbs	547,000	447,000	Saving of 100,000 lbs
Grade beams	240 CY	-	Saving of 240 CY of concrete and 36,000 lb of reinforcement
VDDs	--	$170,000	Additional cost of dampers

TABLE 7. INITIAL COST COMPARISON

SUMMARY AND CONCLUSION

A new steel building was designed using PBD and provisions of ASCE 7. SMRFs were used to provide strength; VDDs were used to control story drifts. Analysis showed that this building had a superior seismic performance to that of a traditionally designed structure. The expense of VDDs was offset by the reduction in cost of steel members and foundation.

- ASCE 7 provisions are readily applicable to seismic design; PBD approach can be used to produce a successful structure.
- PBD design using VDDs is superior to the conventional design. The demand on both structural and nonstructural components is reduced.
- VDDs provide non-intrusive and reliable toll for seismic design. The VDD force demand is controlled by using nonlinear damping properties. These forces are out-of-face with elastic forces and do not increase the demand on members.
- The additional cost of VDD is offset by the decrease in structural and foundation costs.

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