STATE OF THE ART DESIGN OF STEEL MOMENT FRAME BUILDINGS WITH DAMPERS

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ABSTRACT :

Steel Special Moment Resisting Frames (SMRFs) with Viscous Damping Devices (VDDs) have been used for design of many buildings, resulting in a reliable and cost effective solution, with a high confidence level. The dampers serve to reduce the seismic demand and damage to structures. It is also anticipated that the design will have lower repair cost and shorter downtime following an earthquake. The cost-effectiveness and the anticipated superior performance of this system present an opportunity for a more widespread application. However, no comprehensive and rigorous analysis has been conducted to address several outstanding issues: the probabilistic assessment of performance, the realistic confidence levels, and correlation between the engineering data and hazard evaluation parameters including probable maximum loss (PML) and business interruptions (BI). It is proposed to address such issues in an upcoming research program.

KEYWORDS: Special moment frames, viscous dampers, reduced beam section, nonlinear analysis, confidence level, case study

1. INTRODUCTION

1.1. Overview

Steel special moment resisting frames (SMRFs) are one of the preferred options for seismic design in regions of high seismicity. The 1994 Northridge and 1995 Kobe earthquakes demonstrated that the standard assumptions and construction detail (complete penetration welding of beam flanges to column flanges and bolted/welded shear tab) exhibited sudden and brittle failure. To address this issue, extensive testing and evaluations were conducted and pre-qualified connections have been developed. Reduced beam section (RBS); see Figure 1, is a connection that is qualified for any size member. By reducing the beam flexural capacity, nonlinearity is concentrated in the reduced region and away from the potentially vulnerable beam-to-column connection.

The combination of viscous damping devices (VDDs), and steel SMRFs presents an attractive design option. The result is a highly damped, low-frequency building that limits seismic demand on structural and nonstructural components. VDDs are an ideal option due to their high damping because they are velocity dependent, and hence, do not significantly increase demand on foundations or columns. VDDs were 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 orifices; see Figure 2. During seismic events, the devices become active and the seismic input energy is converted to heat and is thus dissipated. In the past several years, the authors applied the design methodology discussed here for a number of steel SMRF buildings. Sample structures are listed in 0.

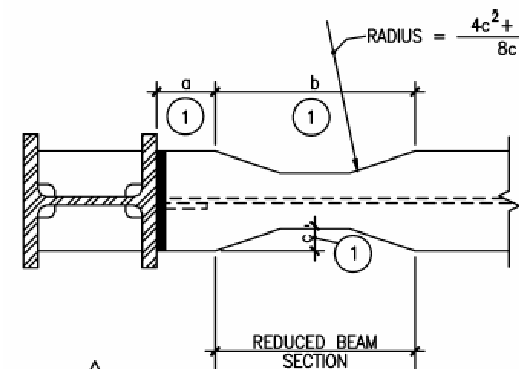


Figure 1. Details of RBS (AISC 2005b)

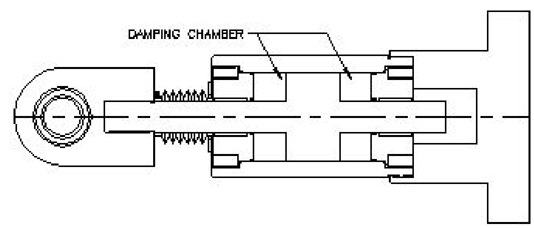


Figure 2. Schematic of VDD (Taylor 2007)

Table 1. Sample of newly designed/constructed steel SMRF with dampers

Structure	Stories	Area, m^2
Town Square	4	8,000
Sutter Gold, Modesto	5	13,000
CSU Sacramento AIRC Building	4	10,000
Vacaville Police Station	2	4,000
Ziggurat building	11	30,000

The additional cost of the dampers is typically offset by the savings in steel tonnage and foundation concrete volume. Hence, the conventionally designed and the damped buildings have similar initial costs. Sample data is presented in Table 2.

Table 2. Cost comparison for typical supplementary damped steel SMRF

Item	Conventional	Damped	Differential cost
Moment Frames	274 Ton	223 Ton	- $150,000
Foundation	Concrete grade beams, reinforcement, excavation & backfill	No grade beam required for pinned foundations	- $200,000
Dampers	None	$200,000	+ $200,000
Net			-$150,000

1.2. US Code Provisions

In structural engineering practice, performance based engineering based on codes such as ASCE 7 (AISC 2005a) has been used to design steel SMRF buildings. Chapter 18 of ASCE 7 details the seismic design requirements for structures with supplementary damping. The pertinent code requirements are summarized below.

- When using the equivalent lateral load procedure, the base shear can be reduced to 75%.
- Elastic analysis procedures are allowed with certain limitations. When such analysis is allowed, a response reduction factor, B, is used to account for additional damping for static and response spectrum procedures.

- 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. DCR is defined as the ratio of applied seismic demand to the member capacity and is obtained from stress check calculations.
- Strength reduction factor, Ω, and redundancy factor, ρ, of unity are used to evaluate the response of members.
- Prior to installation, prototype or production tests are required to ensure that the constitutive relation for dampers fall in the acceptable range.

2. CASE STUDY

Provisions of ASCE 7 were used to design a new steel framed multi-story building in the Los Angeles area. The steel members were sized using conventional code (CBC 2001) design procedures. VDDs were sized to control the story drifts. The dampers were placed only at the ground floor with pinned column bases where the maximum velocity is expected to occur. A parallel design was carried out using the conventional design methodology. This model was designed following the conventional code procedure for both strength and drift.

The four-story commercial building is 18.5 m tall and has a total floor space of 8,000 m^2. Architectural rendering of the building is presented in Figure 3. Computer program SAP (CSI 2007) was used to prepare three-dimensional mathematical models of the damped and conventional designs. The SMRF steel beams and columns were modeled using the program's beam-column elements. Nominal spans and member sizes (AISC 2005) were used. For the damped model, the bases of all columns were modeled as pinned. For conventional design model, the fixity, provided by the grade beams, was assumed at the base of all columns. Figure 4 depicts the mathematical model of the building. Sixteen nonlinear VDDs were used to control story drifts at the first floor. The seismic mass of the building was approximately 9 MN.

Figure 3. Architectural rendition

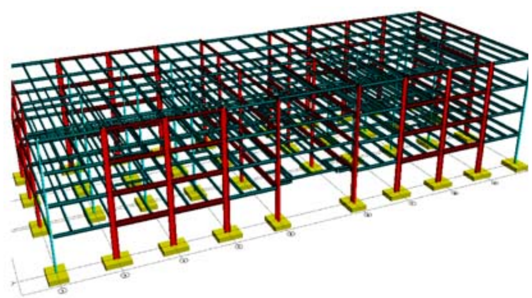


Figure 4. Mathematical model

Two levels of seismic hazard were investigated in design: the maximum considered earthquake (MCE) with a 2,500-year recurrence interval, and the design basis earthquake (DBE) with a return period of 475 years, or 2/3 of MCE. The response spectra for the two sites are shown in Figure 5. Spectrum-compatible records were synthesized using seeds from past earthquake records and having response spectra closely matching the target. The records have a typical duration of 40 seconds. Two performance levels were used in evaluation of building, life safety (LS) at DBE and collapse prevention (CP) at MCE.

Nonlinear response history analysis was performed to evaluate performance. The models were first preloaded with gravity load combinations and then subjected to the three pairs of accelerations at the DBE level and three pairs at the MCE level. The components of the ground motion were aligned with building principal axes. Maximum response quantities, such as, building floor displacement and accelerations, story shears, VDD forces, and member stresses, were extracted. The extreme values from all analyses were then used for evaluation.

The maximum computed story drift was approximately 1.4%, which meets the code requirement. Both damped and conventional structures had similar drifts. Base fixity and larger member sizes control drift for the conventional model. VDDs provide such control for the damped model. The damped model has smaller base shear (Figure 6) and floor accelerations because it has a larger period and damping. Limiting acceleration will protect acceleration-sensitive nonstructural components such as piping and ceilings. Therefore, the application of the VDDs seismically protects both the structural and nonstructural components.

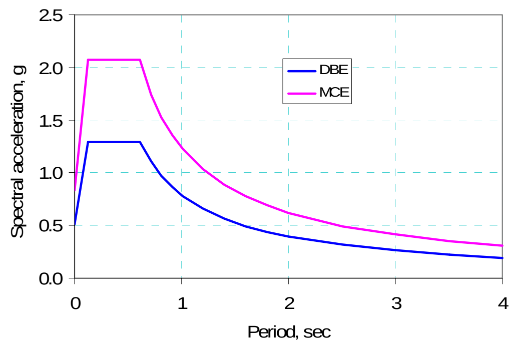


Figure 5. Seismic demand

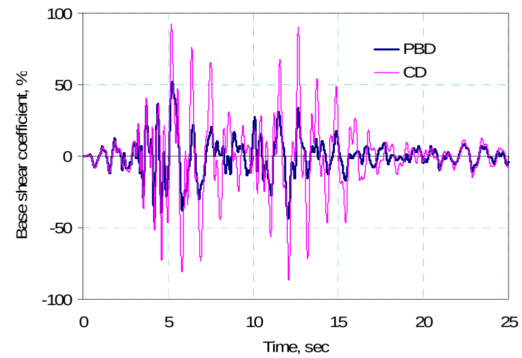
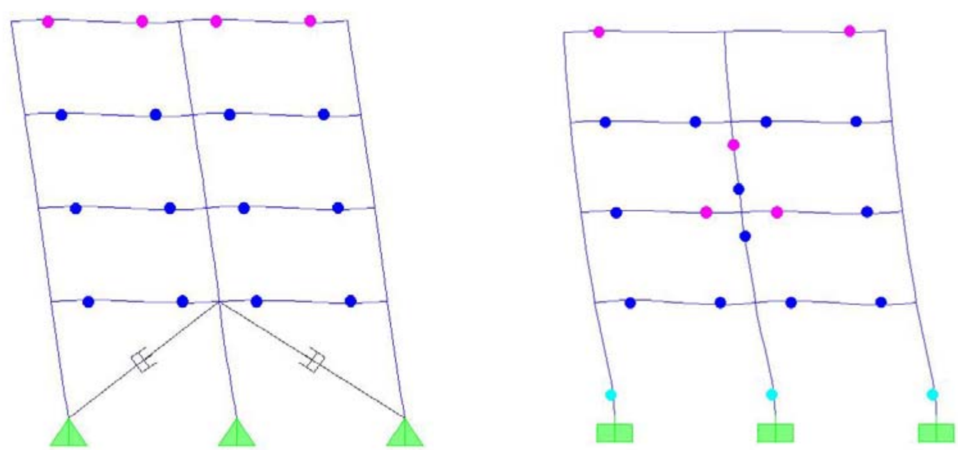


Figure 6. Base shear coefficients

Figure 7 shows the snap shot of the damped and conventional models at maximum deformation for the MCE event. Both models meet their performance goal of collapse prevention for this event. However, the damped model meets the higher LS performance goal. Furthermore, the columns of the PBD model remain elastic and, as listed in Table 3, the plastic rotations are smaller for the damped model.



a. Model with dampers

b. Conventional design

Figure 7. MCE plastic hinge rotations

Table 3. Maximum MCE plastic hinge rotations, % radian

	Conventional	Damped
Beam	1.7	1.3
Column	2.6	0

Figure 8 presents the damper hysteresis loop and the components of seismic energy computed from analysis. In the absence of dampers, yielding in ductile beam members would substitute for such energy dissipation. Prior to installation, production tests of the dampers were conducted. The test data is used to ensure that each unit had adequate capacity and to verify the force-velocity relations for dampers. Sample laboratory hysteretic data for is shown in Figure 9 (Taylor 2007). The damper constitutive force-displacement relation closely correlated to the theoretical values used in analysis.

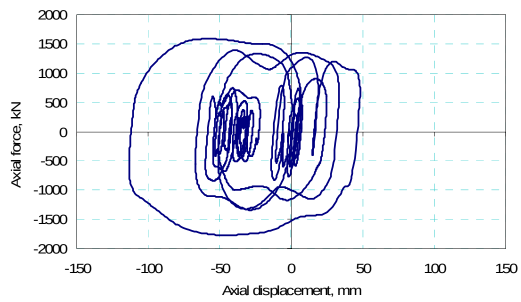


Figure 8. Analytical hysteresis

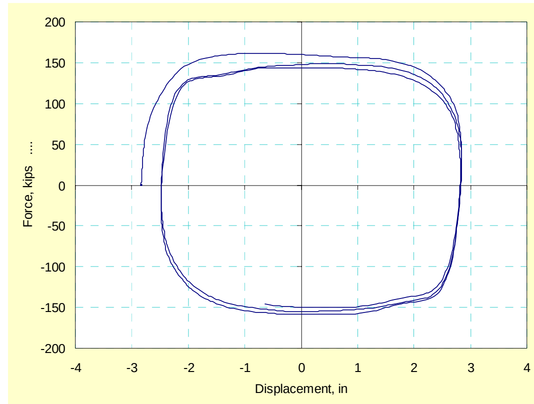


Figure 9. Experimental hysteresis

3. COST-BENEFIT ANALYSIS

The damped structure has superior long-term performance and lower maintenance costs. Following a design earthquake, the conventional building should provide life safety, but will sustain significant damage because of significant ductile yielding of the gravity carrying elements and higher accelerations.

The long-term performance of damped and conventional buildings is qualitatively illustrated in Figure 10. The buildings have similar performances at construction time. Sometime later, a seismic event occurs. This reduces the quality level of the buildings. The degradation for conventional building is greater, resulting in larger repair cost and downtime. It is anticipated that building would sustain significant structural and nonstructural damage. For the damped building, the damage level is lower. This results in a lower repair cost, less loss of occupancy, shortened business interruptions (BI), and a reduced amount of nonstructural damage. This also translates to shorted repair time. Hence, the damped building will more readily retain its pre-earthquake performance level.

The long-term relative efficacy of the seismic design is inversely proportional to the areas under the curves of Figure 10. This area approximates lost time or repair cost times loss of quality. In other words, the damped structure is more robust and has a higher seismic resiliency.

4. CONFIDENCE LEVEL CALCULATIONS

For the damped building, the column plastic hinge rotations are zero. Using the FEMA 350 (NEHRP 2000) methodology, nominal column yield rotation, and FEMA 350 default values for variability coefficient, the approximate confidence levels of Figure 11 are computed for the two design approaches.

The figure indicates that for the damped design there is a very high probability that the performance goals would be reached. It is worth remembering that the results of Figure 11 are for an idealized dampened structure. Three factors contribute to differentiate between the *idealized* model assumed here and the real-life behavior. 1) Not 100% of dampers will meet a performance goal. For a sample of *n* dampers laboratory tested to a target performance, there is a probability *p1* that one damper will not meet its performance goal. 2) When a number of dampers all meeting their performance goals are installed in a building, the introduction of the braced connector and the damper connection hardware introduce performance reduction variables to these units. Thus, there is a probability *p2* that the idealized installed dampers will not meet their performance goal. 3) Finally, the dampers are designed and sized for a specified force and displacement capacity derived from analysis at a performance level. When the units are subjected to motions larger than anticipated from analysis, there is a probability *p3* that the units would experience thermal effects grater than design or reach their stroke or force capacity and thus become ineffective. As such, the *realistic* confidence level attainable for the damped building is somewhat lower than the idealized case and is shown by the dashed line in Figure 11.

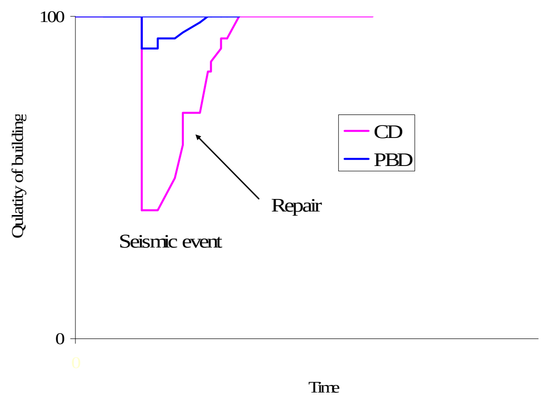


Figure 10. Qualitative resiliency curves

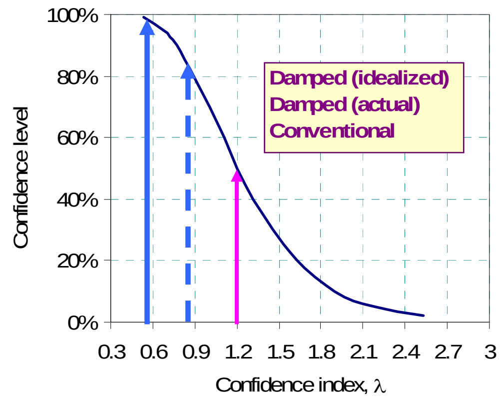


Figure 11. Sample confidence levels

5. UPCOMING RESEARCH

The proposed research is intended to expand the knowledge base for steel SMRF buildings with dampers. In particular, the scope of the work would include confidence level computations accounting for realistic installation and imperfection concerns, development of fragility curves for such construction, and establishing structural- and non-structural-based PML and BI data for the design. The research would only consist of a detailed analysis component and will closely follow the guidelines and procedures established by ATC 63 (NEHRP 2008).

The proposed analysis will be two-dimensional and would attempt to envelop the response of low- to high-rise buildings. Tentatively, a total of 9 models are currently under consideration. The basic geometry and distribution of dampers for these models are summarized in Table 4. The selected building models will be regular in plan and elevation with a dominant first mode response.

Table 4. Proposed analysis models

Building ID	No of stories	VDD configuration 1	VDD configuration 2
A	5	Half height	Ground floor only
B	10	Half height	2/3 height
C	20	Half height	2/3 height
D	40	Half height	Bottom 1/4 and mid 1/4 corresponding to the second mode

All the buildings will be designed per the requirement of the current code (IBC 2006). Dampers will be selected to limit the drift ratio to the code mandated limits. The damping constant, C, will be selected to provide an approximate damping ratio of 20% critical in the first mode. Steel SMRFs will be assumed to be present only along the perimeter. Only wide flange members with the beam flanges framing onto column flanges will be investigated.

The nonlinear response of steel frames will be concentrated at the reduced beam sections; columns and panel zones will be designed to remain elastic. Nonlinearity in steel beams will be represented by concentrated plastic hinges. The constitutive moment-relation relation for the plastic hinges will be derived using the available experimental data. The base of all columns will be modeled as pinned due to the absence of grade beams or deep pile foundations. Frame bays will be 9.1 m wide and typical story height will be 3.8 m. Linear dampers, with a velocity coefficient, α, of 1, will be used. The internal damper flexibility due to the oil column, attachment hardware, and piston will be ignored. Maxwell idealization will be used to model the attaching damper brace, which will b modeled as stiffer than the damper. The dampers will be placed one per bay in the diagonal configuration. For selected analysis, the damper coefficient will be varied by +/- 15% to account for thermal effect resulting from the seismic demand. For large events, nonlinear elements will be placed in line with the dampers to ensure dampers become ineffective when they reach their stroke or force capacity. Idealized connection tolerances without gaps will be assumed.

The seismic demand will be based on a structure located in the Los Angeles basin, a region of high seismicity. Default soil (SD) will be used. The site spectrum will be based on the ASCE 7 design values. The PEER NGA motions will be used as the seeds. The ATC 63 records, incorporating both ordinary and near field large velocity pulses will be examined.

Incremental dynamic analysis (IDA) will be used for evaluation. The ground motions will be scaled at the initial fundamental period of the building. Analysis will be conducted to collapse. The normalized roof displacement will be selected as the evaluation parameter. Fragility curves will be developed for system incorporating uncertainties in analysis, ground motion, and damper properties. The realistic confidence levels will be established and will be used to develop the PML and BI data.

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