# LIGHTLY DAMPED MOMENT RESISTING STEEL FRAMES

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**ABSTRACT :**

The current U.S. seismic design provisions for steel moment resisting frames generally result in structures for which stiffness is the controlling factor in the design. The design for stiffness often provides considerable overstrength, which reduces rotational ductility demand on the plastic hinges in the structure. Even though the reduction in ductility demand may be considerable, the design provisions do not allow the detailing rules to be waived, resulting in designs which are not economically optimum. This paper presents the results of a study in which a variety of steel frames were designed for strength, and which used added energy dissipation in the form of linear viscous dampers to control the drift. The goal of the study was to provide only enough damping to control the drift, and to this end, it was found that total system damping of 10% critical was sufficient. As shown in the paper, the added damping provided the required drift control, and had the added advantage of minimizing the dispersion which typically occurs in response history analyses carried out under several appropriately scaled ground motions. Such dispersion control is illustrated through Incremental Dynamic Analysis of damped three-story and 9-story buildings in Seattle, Washington.

**KEYWORDS: Seismic Design, Viscous Fluid Dampers, Incremental Dynamic Analysis, Structural Steel**

## 1. INTRODUCTION

Properly designed moment resisting steel frames are generally very effective in resisting strong earthquakes. However, due to the low lateral stiffness of such systems, it is often necessary to increase the lateral stiffness to meet drift or stability limits. Increasing the stiffness increases the strength, and theoretically, the increased strength would reduce the ductility demands. If the ductility demands were reduced enough, it would seem feasible to relax the detailing requirements, and possibly, enhance the economy of the system. Current U.S. design provisions (ASCE, 2006), AISC Seismic Design Specifications (AISC, 2005) do not allow such an approach, however.

Another approach would be to simply ignore the drift and stability limits, and design the system for strength alone. Experience has shown that this approach is not feasible because of the potential for developing large residual displacements, or complete dynamic instability. The tendency towards dynamic instability is exacerbated by the low amount of inherent damping that is present in steel systems. It has been recognized that the inherent damping is not likely to be in excess of 2% critical (the almost universal practice of modeling such systems with 5% damping is unconservative).

If more damping could be justified, say a total of 10% inherent viscous damping, the excessive residual deformations and dynamic instabilities might be avoided, and the systems could be designed for strength alone. Exactly such a concept is the focus of the research reported in this paper. It is noted, however, that unlike other papers that concentrate on the effects of added damping that produce total system damping of 25 to 30% critical, this paper concentrates on adding the minimum amount of damping that is required to obtain an acceptable response. As shown in the remainder of the paper, systems with a total of only 10% damping have the desired performance, with the added benefit of increasing the reliability of the structural system.

## 2. ANALYSIS OF A NINE STORY MOMENT RESISTING STEEL FRAME WITH ADDED DAMPERS

The effect of added viscous fluid dampers was investigated on a five-bay nine-story special steel moment frame building, located near Seattle, Washington. The purpose of this study is to design a steel moment frame for only strength and then control the drift by using supplemental dampers. However, since the codes permit designing a flexible building in Seattle and checking the elastic drift limits by using the lateral forces that are calculated by using the computed period of the structure (instead of $C_uT_a$), the strength design satisfied the drift requirements of ASCE 7. Although the strength controlled design meets the drift requirements, the stability checks of both ASCE 7 and the AISC Seismic Design Manual Commentary were not satisfied. Then, another moment frame was designed, by increasing the member sizes at the lower levels, which was controlled by the stability checks of ASCE 7 and the Seismic Manual Commentary. Thus, two different nine story Special Moment Frames (SMF) were designed in Seattle, called the “Stability” and “Strength” controlled designs. Besides the inherently damped frames, viscous fluid dampers were added to only strength controlled design and therefore two more frames were obtained which are called the 5% and 10% total damped strength designs. The ASCE 7 design parameters used for the designs are summarized in Table 1.

**Table 1**
***ASCE 7-05* Design Parameters**

| Design Parameter | Value | Design Parameter | Value |
|---|---|---|---|
| 0.2 second spectral acceleration $S_s$ | 1.25 g | Seismic Design Category | D |
| 1.0 second spectral acceleration $S_1$ | 0.5 g | Effective Seismic Weight W | 10,500 kips |
| Site Class | D | Base Shear | 358 kips |
| 0.2 second design acceleration $S_{ds}$ | 0.83 g | Response Modification Factor, R | 8 |
| 1.0 second design acceleration $S_{d1}$ | 0.5 g | Deflection Amplification Factor, $C_d$ | 5.5 |
| Seismic Use Group | II | Seismic Response Coefficient, $C_s$ | 0.034 |
| Importance Factor | 1.0 | Max. Fundamental Period, $C_uT_a$ | 1.83 sec |

All structural analysis was conducted using Perform-3D (CSI, 2006), using a planar model consisting of one of the two perimeter frames that are parallel to the design ground motion. To move the plastic hinges away from the column face, reduced beam sections were used and the moment rotation properties of each hinge, forming at the reduced sections, were calculated explicitly (Charney, 2006; Atlayan, 2008). Panel zones were explicitly represented by use of Krawinkler's model (Charney and Marshall, 2006).

P-Delta effects were included in all analysis, using a special linear "ghost frame" which captures the entire gravity load tributary to the leaning columns. The inherent damping was determined by setting the critical damping ratio to 2% at the natural period of the structure and at a period of 0.2 sec as it was done in the SAC Report (FEMA, 2000). Both P-Delta and inherent damping frames are laterally constrained to the main frame as can be seen in Figure 1. The added damping coefficients were found by using the tools of NonlinPro (Charney and Barngrover, 2006). The added damper coefficients were updated until the total damping of the strength design reaches 5% and 10% of critical. The added dampers were distributed equally at each story and a chevron brace configuration was used to support the dampers (see Figure 1). This chevron brace configuration was used to provide complete control over the modeling of inherent damping, and thereby avoid the potentially adverse consequences of modeling inherent damping as a viscous mechanism (Charney, 2008).

Two types of analysis were performed for each frame; nonlinear static pushover analysis (NSP) and incremental dynamic analysis (IDA). Figure 2 displays the nonlinear static pushover curves with highlighted target displacements, first significant yields and the design base shear for both of the designs. If the stability ratios of the stories are more than the limit of ASCE 7, it is necessary to check the system at 1.5 times the target displacement to have more confidence that story mechanisms will not occur at the MCE level of shaking. Note from Figure 2 that the tangent stiffness of the pushover curve is continuously increasing at 1.5 times the target displacement for the stability controlled design, whereas the tangent stiffness of the strength controlled design becomes negative prior to reaching 1.5 times the target displacement. See Atlayan (2008) for a much more detailed description of the design procedures and differences between the stability and strength designs.

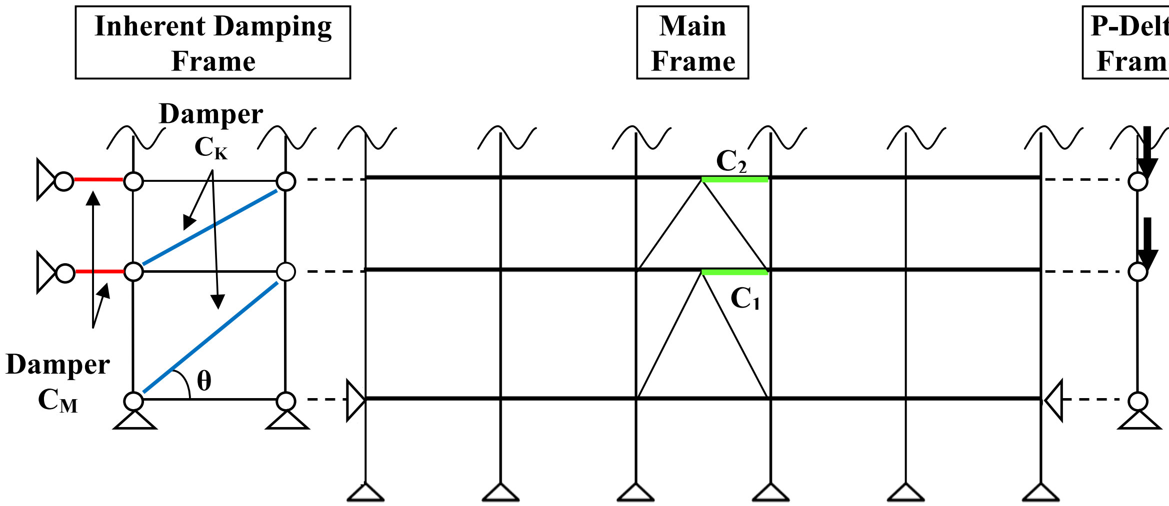



**Figure 1. Inherent Damping and P-Delta Frames Constrained to Main Frame**

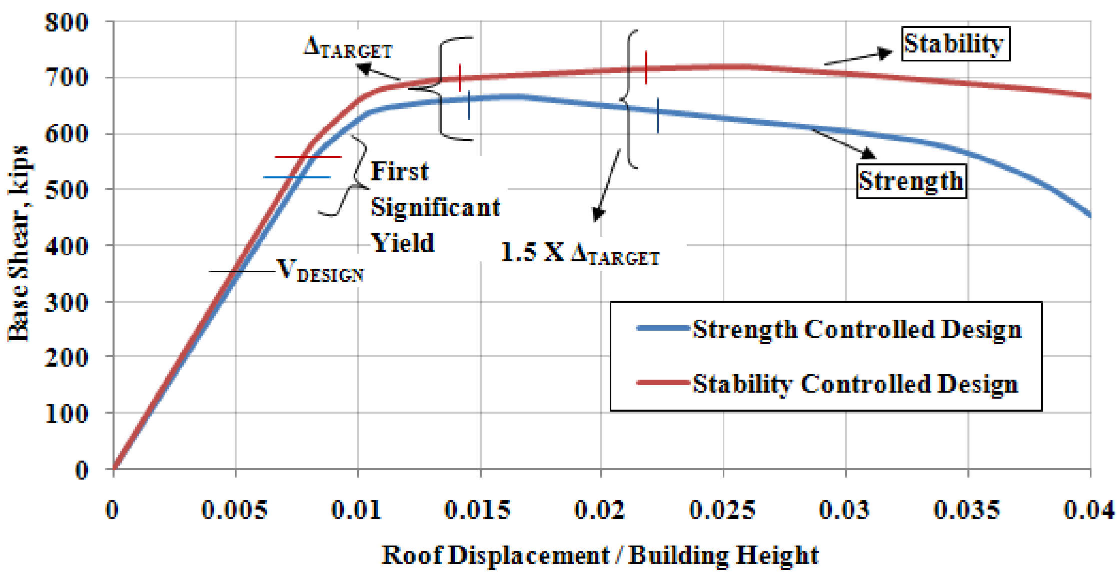



**Figure 2. Nonlinear static pushover curves for strength and stability controlled designs**

In this study, incremental dynamic analysis (Vamvatsikos, 2002) was conducted for the structure subjected to ten different earthquake records, and at intensities of 0.2 to 2.0 times the ground motion scaled to match the design basis earthquake. Thus, the scale factor of 1.0 corresponds to the Design Basis Earthquake (DBE) and the scale factor of 1.5 corresponds to the Maximum Considered Earthquake (MCE). The ground motions were scaled to match the ASCE-7 spectrum at the structure's fundamental period. This scaling procedure is recommended for IDA analysis by Shome and Cornell (1998). It is noted that the ground motions used in the analysis were the same as those used in the original SAC research (FEMA, 2000). The scale factor of the ground motions was used as the intensity measure and the interstory drift, base shear, maximum and residual roof displacements, and IDA dispersion were used as the damage measures in this study.

As discussed before, the elastic drift limits of ASCE 7 were satisfied for both of the inherently damped strength and stability designs. In addition to the elastic drift limit check, according to Chapter 16 of ASCE 7, the results of nonlinear response history analysis (NRHA) shall not exceed 1.25 times the allowable drift limit (2% of story height in this study). However, the results of NRHA exceeded 1.25 times the allowable drift limit for both of the inherently damped designs and for six out of ten earthquakes used in this study. The added dampers play a crucial role here. Table 2 shows the maximum drifts of all the designs subject to two different ground motions at the design level. For this study, the 5% totally damped strength controlled design satisfied the drift requirements under all 10 different earthquakes used. As expected, a further decrease in drift values occurred when the total damping increased to 10% of critical. Thus, a structure, that satisfies the elastic drift limits of the codes, may not satisfy the drift limits by NRHA, but these limits can be met by using supplemental damping.

**Table 2**
**Maximum Drifts for Design Basis Miyagi Oki and Valpariso-2 Earthquakes (IDA scaling = 1.0)**

| | | Miyagi Oki | | | | Valpariso 2 | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Level** | **Drift Limit (%125) (in.)** | **Strength Inherent Damping (in.)** | **Stability Inherent Damping (in.)** | **Strength 5% Damping (in.)** | **Strength 10% Damping (in.)** | **Strength Inherent Damping (in.)** | **Stability Inherent Damping (in.)** | **Strength 5% Damping (in.)** | **Strength 10% Damping (in.)** |
| 9th Level | 3.90 | 3.13 | 3.37 | 1.94 | 1.02 | 5.00 | 5.41 | 2.21 | 1.04 |
| 8th Level | 3.90 | 3.59 | 4.16 | 2.63 | 1.52 | 6.64 | 7.61 | 3.15 | 1.64 |
| 7th Level | 3.90 | 4.53 | 4.96 | 2.97 | 1.99 | 4.97 | 5.46 | 3.37 | 1.98 |
| 6th Level | 3.90 | 4.46 | 4.81 | 3.33 | 2.40 | 4.06 | 5.28 | 2.90 | 2.07 |
| 5th Level | 3.90 | 4.08 | 4.75 | 3.37 | 2.69 | 4.24 | 5.49 | 2.73 | 2.02 |
| 4th Level | 3.90 | 4.36 | 4.74 | 3.47 | 2.99 | 3.22 | 3.67 | 3.01 | 2.07 |
| 3rd Level | 3.90 | 4.64 | 4.63 | 3.49 | 3.16 | 2.84 | 3.42 | 3.09 | 2.18 |
| 2nd Level | 3.90 | 4.30 | 3.53 | 3.54 | 3.19 | 2.77 | 3.11 | 2.92 | 2.35 |
| 1st Level | 5.40 | 5.40 | 4.04 | 4.12 | 3.69 | 4.63 | 4.37 | 3.56 | 3.01 |

Fig. 3(a) and 3(b) illustrate the maximum and residual roof displacement IDA plots by using Valpariso-1 and Seattle earthquakes, respectively. All the designs resist Valpariso-1 earthquake. However, the inherently damped strength design collapses after the scale factor of 1.4 times the DBE, and the stability design collapses after the scale factor of 1.8 times the DBE when frames are subjected to Seattle earthquake. The frames with added dampers resist the collapses for Seattle earthquake and reduce the maximum and residual roof displacements significantly.

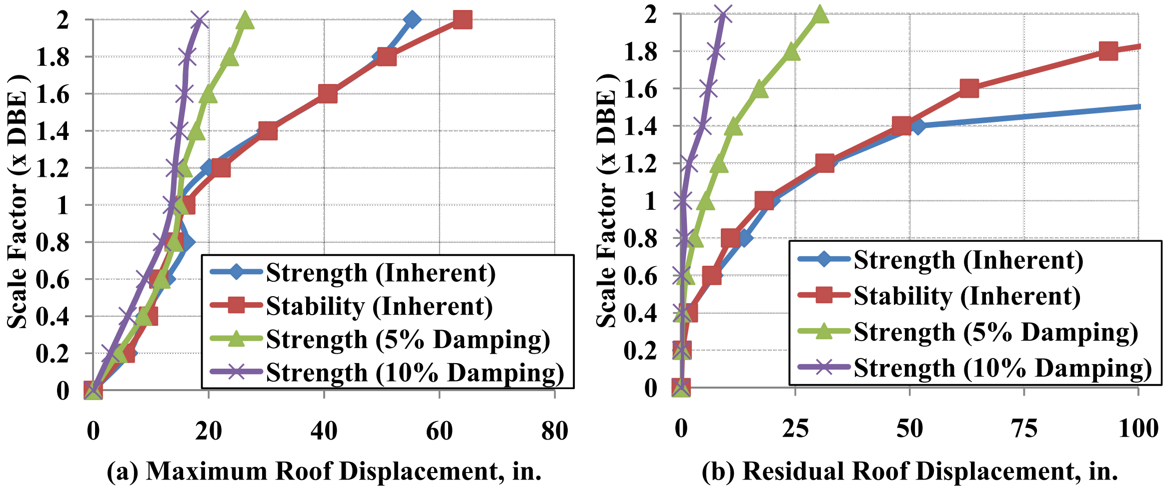



**Figure 3. IDA plots for Maximum and Residual Roof Displacements using Valpariso-1 and Seattle Eq.**

Figures 4(a) and 4(b) display base shear IDA plots for Erzincan and Miyagi Oki earthquakes. At low scale factors, where the structure behaves elastically, base shear decreases as damping increases. The IDA curves generally intersect before displaying an increase in base shear with damping in the nonlinear region. The 5% damped strength controlled design and the inherently damped stability controlled design behave very similar in base shear IDA plots after the structures yield. Especially, around the design and maximum considered earthquakes (scale factors 1.00 and 1.50), the base shear is very close for them (See Figure 4). The same behavior was observed for the other 8 earthquakes used in this study as well. Note that the difference between the base shear IDA plots of stability and 5% damped strength designs at scale factors more than 1.60 in Figure 4(b) is due to collapses. The inherently damped stability design collapses when the scale factor reaches 1.80, however 5% damped strength design resists the scale factor 1.80 and collapses at 2.00 when the frames are subjected to Miyagi Oki earthquake. The main drawback of linear viscous dampers is the high base shears in the inelastic region of the IDA plots. This study concludes that 5% damping with linear viscous dampers doesn't cause base shear problems. The 10% total damping increases the base shear between 10% and 30%, depending on the earthquake. The use of nonlinear viscous dampers, with the velocity exponent of about 0.5, would likely reduce the increase in base shear associated with added viscous damping, but this was not investigated in this study.

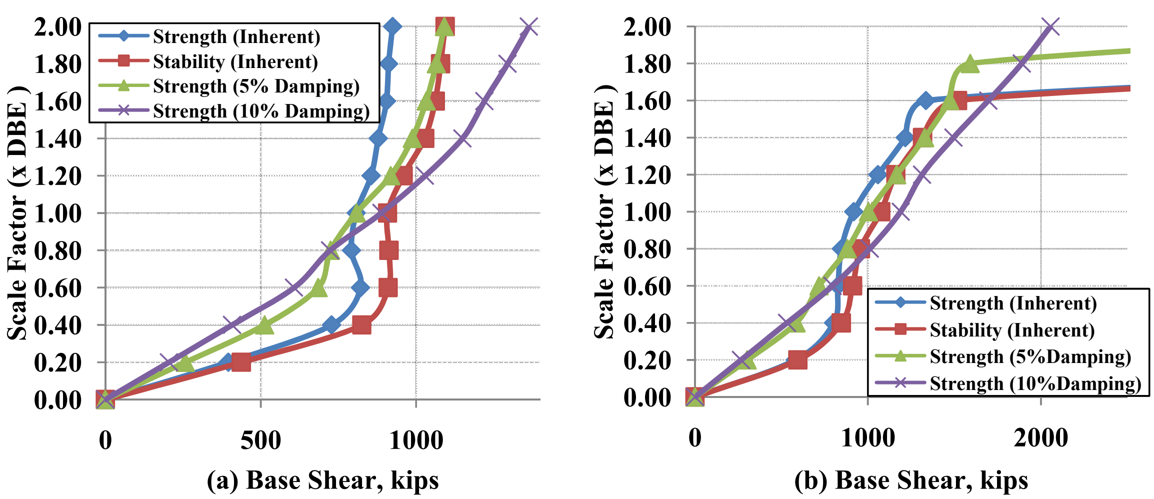



**Figure 4 Base Shear IDA Plots for Erzincan (a) and Miyagi Oki (b) Earthquakes**

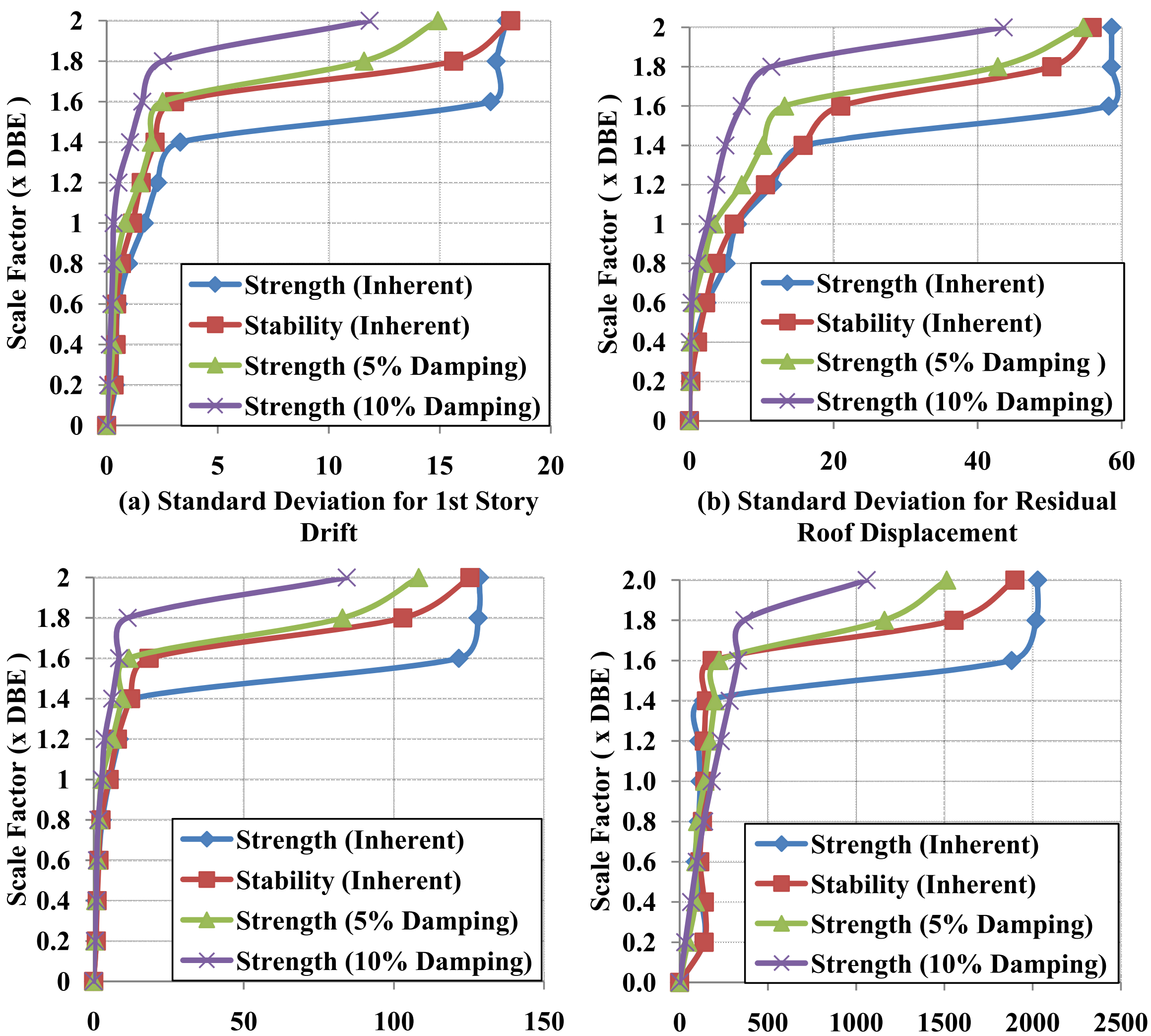
One of the main purposes of this study is to investigate the effect of added dampers on dynamic instability (collapses). Table 3 shows the collapse check of four structures subject to six earthquakes, from scale factor 1.0 to 2.0, which caused collapses in the IDA studies. Both the strength and the stability controlled designs don't collapse until the MCE. However, the strength design collapses just after the MCE for three earthquakes, when the IDA scale factor reaches 1.6. The effect of damping on dynamic instability is obviously significant. 5% damping prevents the collapses of four out of six earthquakes that occurred in the strength controlled design. The 10% total damped structure resists all the earthquakes except EQ09 with scale factor of 2.0. The amount of damping necessary to prevent the collapse is dependent on the design of the structure. Regarding the strength controlled design investigated in this study, 10% total damping is adequate for providing dynamic stability.

**Table 3 Collapse Check**

| Design | Scale Factor | Seattle EQ | Valpariso-2 EQ | Deep Interpolate EQ | Miyagi Oki EQ | Shallow Interpolate-1 EQ | Shallow Interpolate-2 EQ |
|---|---|---|---|---|---|---|---|
| Strength Controlled Design with Inherent Damping | 1.0 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.2 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.4 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.6 | Collapse | Collapse | ✓ | ✓ | ✓ | Collapse |
| | 1.8 | Collapse | Collapse | ✓ | Collapse | ✓ | Collapse |
| | 2.0 | Collapse | Collapse | Collapse | Collapse | Collapse | Collapse |
| Stability Controlled Design with Inherent Damping | 1.0 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.2 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.4 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.6 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.8 | ✓ | ✓ | ✓ | Collapse | ✓ | Collapse |
| | 2.0 | Collapse | Collapse | ✓ | Collapse | ✓ | Collapse |
| Strength Controlled Design with 5% Total Damping | 1.0 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.2 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.4 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.6 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.8 | ✓ | ✓ | ✓ | ✓ | ✓ | Collapse |
| | 2.0 | ✓ | ✓ | ✓ | Collapse | ✓ | Collapse |
| Strength Controlled Design with 10% Total Damping | 1.0 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.2 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.4 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.6 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 1.8 | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | 2.0 | ✓ | ✓ | ✓ | ✓ | ✓ | Collapse |

The dispersion of IDA curves can be used to see the effect of dampers in terms of giving a reliable estimate of the performance of the buildings. To measure the IDA dispersion, the standard deviation of the responses, produced at each intensity level of ten different earthquakes, was calculated for each structure with different amounts of damping. The standard deviation increases as the amount of dispersion increases. When the standard deviation IDA curves for the different levels of damping are displayed together, the curve that has the steepest slope will correspond to the system that is best at reducing the IDA dispersion.

Figures 5(a) through 5(d) illustrate the IDA dispersion for different damage parameters. While the building response is elastic, there is not a significant dispersion. As the earthquake intensity increases, dispersion increases as well, and when the buildings collapse, dispersion increases because of the collapse measures used for the dispersion study. In all of the dispersion plots, the gentle sloped (flat) lines occurring at high intensity measures display the collapses. When IDA dispersion plots are analyzed, it can be concluded that the 10% total damped strength controlled design has a drastically improved performance which is much better than the other designs. The 10% damped strength design gives high dispersion or uncertainty only for the base shear damage measure between scale factors 0.8 and 1.5 (See Figure 5(d)). This is an expected result if the drawback of the linear viscous dampers at high damping ratios is considered. After the scale factor of 1.5, 10% damped strength design gives better results in terms of base shear IDA dispersion as well because the other designs collapse at high earthquake intensities and the use of collapse measures increases the dispersion.




(a) Standard Deviation for 1st Story Drift

(b) Standard Deviation for Residual Roof Displacement

(c) Standard Deviation for Maximum Roof Displacement

(d) Standard Deviation for Maximum Base Shear

**Figure 5. Standard Deviation IDA Plots for Different Damage Measures**

## 3. SUMMARY AND CONCLUSIONS

The supplemental dampers have a remarkable effect on reducing the inelastic response of the elements of steel moment frames. The effects of added dampers were studied through Incremental Dynamic Analysis by using various damage measures. After adding the linear viscous dampers to the strength designed frame, a significant performance improvement was achieved. The IDA responses of the roof displacement, residual displacement and interstory drifts decreased drastically as a result of added dampers. Although the inherently damped strength design satisfied the elastic drift requirements of ASCE 7, it didn't meet the drift requirements for the nonlinear response history procedure where the allowable drift limits are increased by 25%. However, after adding the dampers, 5% total damping was adequate to meet the drift criteria of ASCE 7. Thus, linear viscous dampers can be used in the design of new moment frames without changing the sections of the strength controlled design if it doesn't meet the stiffness or stability requirements.

A collapse check study and an IDA dispersion study were implemented to investigate the effect of damping on dynamic instability. Regarding the optimum level of damping, 5% total damping was almost always adequate to prevent collapse. As the damping increased, the IDA dispersion plots got steeper, which indicates a more reliable performance of the structure under various earthquakes. The 10% total damped strength design gave the best results, followed by 5% damped strength and stability designs, in terms of IDA dispersion.

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