

Performance-Based Design Examples for Steel Moment-Resisting Frames with Supplemental Damping

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ABSTRACT

The goal of this paper is to present a design procedure for steel moment-resisting frames with supplemental damping with performance-based design constraints. The first design example is based on a 4-story steel moment frame building with velocity dependent dampers; the second design example is based on the same building with displacement dependent dampers. The primary seismic-resistant frames, without dampers, were designed to meet the Uniform Building Code of 1997. The amount of supplemental damping for velocity-dependent devices was proportioned based on the first mode of the structural motion. The design of supplemental damping for displacement dependent devices was based on the stiffness of the structure. The seismic resistant frames, with dampers, were designed to remain elastic under a Design Basis Earthquake. The design goal is to limit the lateral drift to 1% of the story height and the Demand-to-Capacity ratio of the moment connections within 1 for a Design Basis Earthquake event. In the event of a larger earthquake, such as Maximum Credible Earthquake, plastic hinge formations and some structural damage would be expected. In that case, the post-Northridge moment connections will provide the ductility to minimize the damage. Lessons learned and suggestions for design guidelines are presented.

KEY WORDS

Performance-based, Steel; Moment-Resisting Frames; Supplemental Damping; Displacement-dependent; Velocity-dependent.

1. Introduction

Performance-based design for structures under strong ground motion recently has been gaining great attention. Many researchers and engineers have proposed various methodologies, which incorporated performance-based concepts and criteria, to improve structural performance against earthquakes [1-4]. The FEMA-273 (1997), NEHRP Guidelines for the Seismic Rehabilitation of Buildings, provides nation-wide acceptable guidelines for the seismic rehabilitation of buildings [5]. One of the significant new features is that the document identifies the methods and design criteria to achieve different levels and ranges of seismic performance. The four building performance levels are collapse prevention, life safety, immediate occupancy and operational. These levels are discrete points on a continuous scale describing the structure's expected seismic behaviors and measuring the amount of structural capacity loss due to earthquake demands. Furthermore, estimates of potential economic loss and business disruption can be made.

Probabilistic hazards are frequently used in the FEMA guidelines and are defined in terms of probability that more severe demands will be experienced in a 50-year period. The Guidelines make frequent reference to two levels of earthquake hazard that are particularly useful for the

formation of Rehabilitation Objectives. They are defined as a Basic Safety Earthquake (BSE-1) and Basis Safety Earthquake (BSE-2). The BSE-1 has a 10 per cent probability of being exceeded within 50 years, which equals a mean return period of 500 years. The BSE-2 has a 2 per cent probability of being in exceeded in 50 years, which equals a mean return period of 2500 years. The Guidelines define that the basis-safety objective can be achieved if the rehabilitated structure will meet life/safety performance under BSE-1 and will meet collapse prevention performance under BSE-2. Any objective intended to provide performance superior to that of BSO is termed an Enhanced Objective. Furthermore, FEMA-276 (1999), example applications based on the NEHRP Guidelines, was published in April 1999 [6]. This report illustrates the process for applying the FEMA-273 through the use of the real building examples taken from around the United States. Unfortunately, the FEMA-273 Guidelines aimed at the seismic rehabilitation of the existing buildings rather than at new constructions. In addition, the use of supplemental damping for enhancing new building performance was not recognized in most of the design provisions. Although such an approach may not be appropriate design strategy for most buildings, it has been used for a number of buildings for which enhanced objectives are required. This paper addresses designing such new buildings, incorporating performance-based design criteria derived from the FEMA Guidelines. Design procedures for steel moment-resisting frames with supplemental damping are proposed.

2. Supplemental Damping

Passive energy dissipation is an emerging technology that enhances the performance of a building by adding damping — and, in some cases, stiffness — to the building. The primary benefit of energy-dissipation devices is to reduce earthquake demands. Therefore, the structural deformations and forces providing the structure respond elastically. Such devices have been adopted for many buildings during the past decade [7-9]. The energy-dissipation devices are classified as displacement-dependent and velocity-dependent depending upon their mechanical characteristics. Velocity-dependent devices exhibit a force-displacement response, the function of which is related to the relative velocity between each end of the device. Fluid viscous dampers are typical examples. On the other hand, displacement-dependent devices exhibit a force-displacement response, the function of which is related to the relative displacement between each end of the device. For example, metallic dampers are force-displacement devices.

3. Modeling of energy dissipation devices

Many mathematical models for both velocity-dependent and displacement-dependent have been proposed. Two-surface model [10] was used for displacement-dependent devices and Maxwell model [11] was used for velocity-dependent devices, respectively. In order to evaluate the response of these devices, FEMA-273 suggests the following model for displacement-dependent device:

$$F = K_{eff} D \quad (1)$$

where the effective stiffness K_{eff} of the device is calculated as:

$$K_{eff} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} \quad (2)$$

and where forces in devices, F^+ and F^- , are evaluated at displacements D^+ and D^- , respectively.

The force in the fluid viscous device may be modeled as shown in the following, in the absence of stiffness in the frequency range $0.5 f_1$ to $2f_1$:

$$F = C_0 |\dot{D}|^\alpha \text{sgn}(D) \quad (3)$$

where α is the velocity exponent for the device, C_0 is the damping coefficient for the device, \dot{D} is the relative velocity between each end of the device, and sgn is the signum function that, in this case, defines the sign of the relative velocity term.

4. Design Parameters for Structures with added dampers

Design parameters are defined so that simplified procedures can be followed. Design parameter for displacement-dependent devices are defined as Stiffness Ratio (SR).

The stiffness ratio is defined as the ratio of the horizontal stiffness of the devices to the bare-frame building story stiffness. The relationship is expressed as:

$$SR = \frac{K_L}{K_B} \quad (4)$$

where K_L is horizontal stiffness of the displacement-dependent device and K_B is the building stiffness without the devices in place.

The design parameter for velocity-dependent devices is C , which is related to the critical damping ratio. It is commonly accepted by practicing engineers that the maximum damping will not exceed 30% of the critical damping. Therefore, proportional damping from Rayleigh's assumption that damping is proportional to the mass and/or stiffness can be used. Rayleigh's damping for Single Degree Of Freedom (SDOF) system is expressed as:

$$C = 2\mathbf{x}(mk)^{1/2} = 2\frac{\mathbf{x}}{\mathbf{w}}k \quad (5)$$

where \mathbf{x} is damping ratio, \mathbf{w} is frequency, m is mass and k is stiffness of the SDOF systems. Once the bare-frame building was designed, the stiffness of each story can be obtained. With the targeted supplemental damping ratio, the design parameter, C , for velocity-dependent devices can be derived.

5. Steel building frame

A 4-story 11,150 m² office building was designed in accordance with Uniform Building Code (UBC) 97 provision requirements based upon static analysis using the criteria that follow. The frame elevation is shown below in Fig. 1.

Seismic Zone Factor, Z	0.4
Important Factor, I	1.0
Site Coefficient	S_d
Seismic Source	A
Distance from Fault	12 km

The height of the frame is 18.6 m (4.3 m for the roof and 4th levels, and 4.7 m and 5.3 m, respectively, for the 3rd and 2nd levels). The width of the building frame is 29.2 m, equally divided into six bays. Typical floor mass is 16,400 kN. The total of the story weights for the entire building is nearly 57,500 kN. An elevation view of the building frame, including beam and column sizes is shown in Fig. 1.

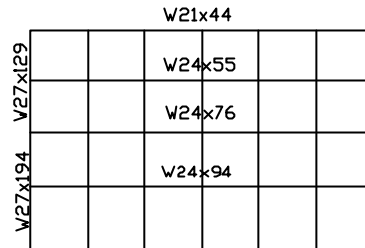


Fig. 1 - Frame Elevation

Dampers were installed in the building in conjunction with chevron-bracing, so that the damper connects the brace at its apex to the floor beam of the next story. It was assumed that the bare frame includes 5% inherent damping.

With the design of the building complete, requirements of UBC 97 were applied to determine the vertical distribution of force through the frame. The building period, T_a , was estimated using the following formula, included as Method A in UBC 97 Section 1630.2.2:

$$T = C_t (h_n)^{3/4} \quad (6)$$

C_t is a coefficient equal to 0.0853 for SMRF, and h_n is the total building height. The value obtained by Method A was 0.764 second. The value, T_b , was found by multiplying T_a by 1.3 (the maximum permissible per UBC 97, Method B). The calculated value of T_b was 0.993 second. This value of T_b was rounded up to one second for simplicity and used in subsequent calculations. In accordance with UBC 97 Section 1630.2, design base shear for the building was calculated using the following formula:

$$V = \frac{C_v I}{RT} W = 0.084W \quad (7)$$

where C_v (taken as 0.72) is the seismic coefficient determined per UBC 97 Table 16-R from the above listed criteria, I is the importance factor (taken as 1), W is the building weight, R is a factor based on ductility of structure type ($R=8.5$), and T is the building period (1 second for this frame).

Once the building was designed, the story stiffness was obtained as shown in Table 1.

Table 1 - Story Stiffness		
Story	Stiffness Label	Stiffness (kN/cm)
Roof	K1	860
4 th Floor	K2	928
3 rd Floor	K3	1060
2 nd Floor	K4	1470

6. Design Examples for damper-added structure

The first step was to identify the target roof displacement during the various levels of ground motions. Roof displacement is a proper locator in deciding the level of damage to the structures. Table 2 shows the allowable transient drift on each structural performance levels for steel moment frames based on FEMA document.

Table 2 - Story Drift Ratio Limit of Steel Moment Frames

Structural Performance Levels		
Collapse Prevention	Life safety	Immediate Occupancy
5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent

In order to meet immediate occupancy performance level, structural damage is limited to the following situations which include (1) minor local yielding at a few places, (2) no fractures, and (3) minor buckling or observable permanent distortion of members. The use of energy-dissipating devices in the structures, or the isolation of a structure from earthquakes are commonly used to achieve that performance level, because application of such seismic protective systems can effectively reduce the demand on structures. In order to achieve the enhanced structural performance level, the steel building frame with added damping was designed so that the story drift is limited to 1% of the story height during the BSE-1 ground motions. It is believed that the structure damage is limited when the drift ratio is less than 1% during BSE-1 earthquake.

The second step was to request the site-specific three levels of ground motion. It should be noted that the occurrence of a strong earthquake in contrast to that of a moderate earthquake in the zones of frequent occurrence of strong ground shaking, such as, California is much smaller than in other parts of the country. Therefore, a rare strong BSE-2 earthquake in other parts of the country may not be applicable in the zones of frequent occurrence of strong ground shaking. A shorter return period of strong ground motion, Maximum Credible Earthquake (MCE), was selected instead of BSE-2 in this design. Three levels of ground motion used in this study are listed in Table 3. The Design Basis Earthquake (DBE) is actually the BSE-1 of FEMA document.

Table 3 - Three levels of Design Ground Motions

Earthquake Definition	Earthquake Having Probability of Exceedance	Mean Return Period (years)
Lower Level Earthquake (LLP)	50%/50 year	72
Design Basis Earthquake (DBE)	10%/50 year	474
Maximum Credible Earthquake (MCE)	10%/100 year	950

Site-specific ground motions were required due to the face that the structural dynamic response is very sensitive to the site and to the characteristics of the input ground motions. In order to catch the various frequency contents of the input ground motions, it is suggested to have at least 3-pair of ground motions for dynamic analyses if the design values of seismic resisting members are taken from the maximum values of the three analyses. Three levels of 3 pairs of ground motion are listed in Table 4. Seven ground motions used in this design are listed in Fig. 2-8.

Table 4 - Site-Specific Ground Motions

Design Earthquake	Earthquake	Magnitude	Time History	Dist. (km)
LLE	Loma Prieta	7.1	Santa Teresa	18
DBE	Imperial Valley	6.7	El Centro	12
	Landers	7.4	Joshua Tree	15
	Loma Prieta	7.1	Saratoga	3
MCE	Imperial Valley	6.7	El Centro	12
	Landers	7.4	Joshua Tree	15
	Kern County	7.6	Taft	56

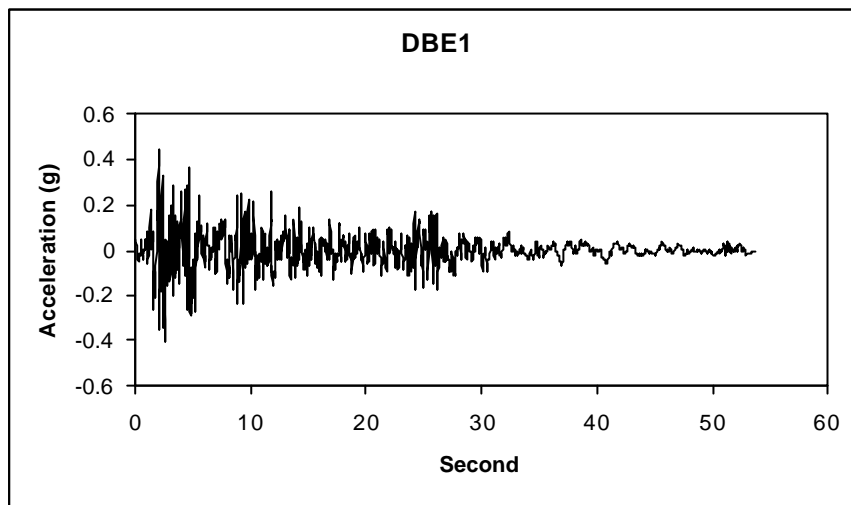


Fig. 2 - DBE1 Ground Motion Record

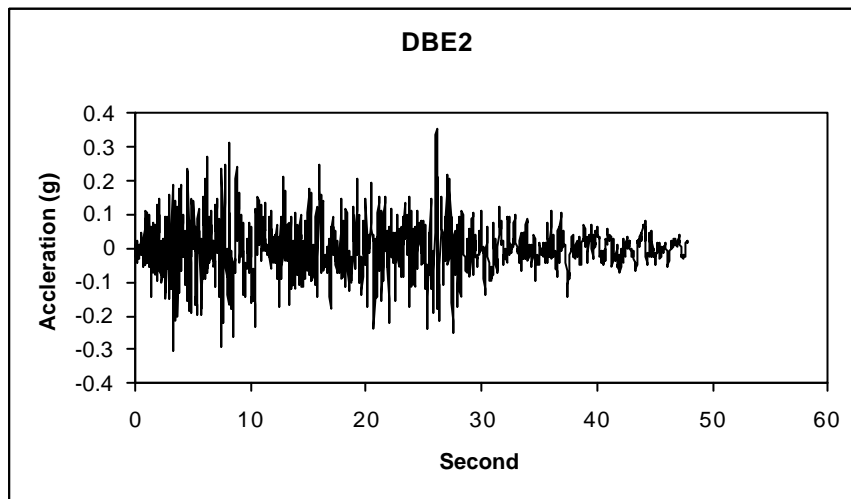


Fig. 3 - DBE2 Ground Motion Record

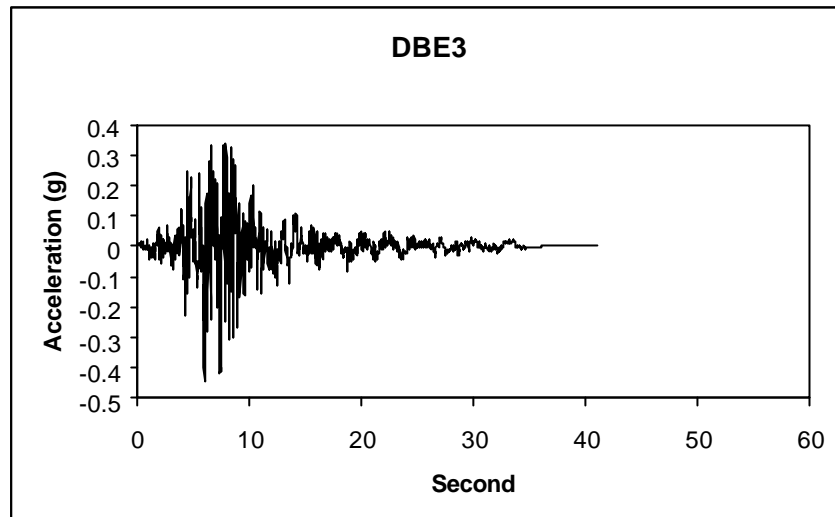


Fig. 4 - DBE3 Ground Motion Record

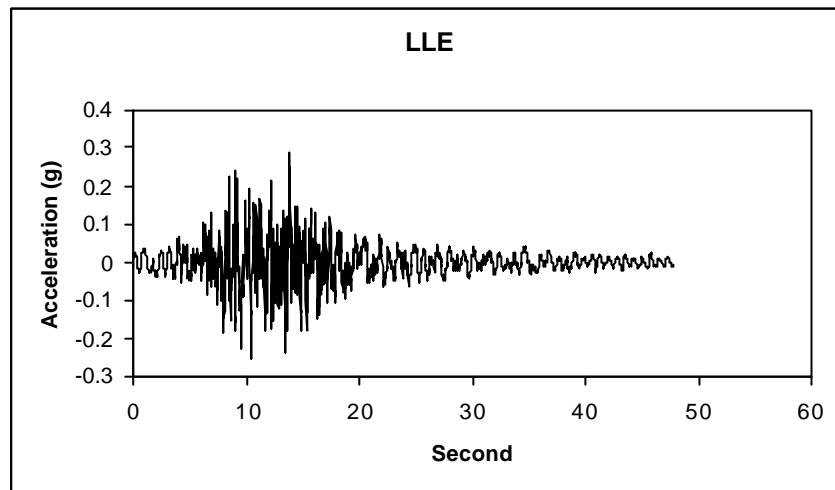


Fig. 5 - LLE Ground Motion Record

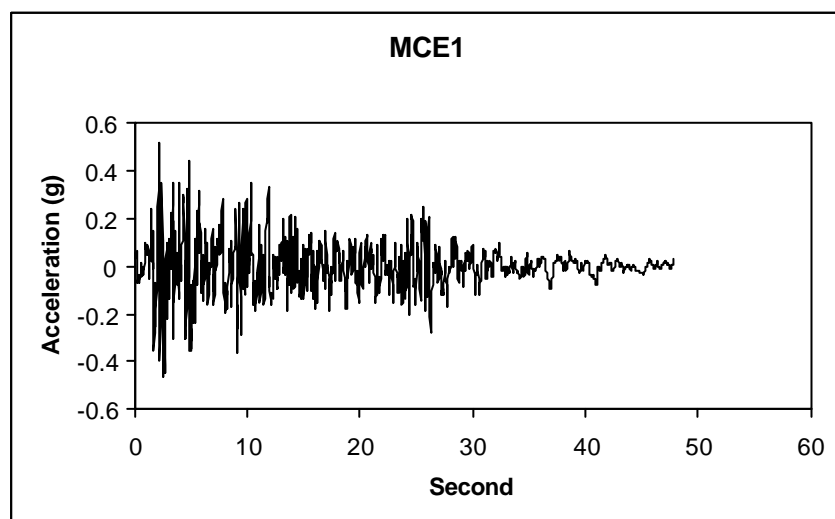


Fig. 6 - MCE1 Ground Motion Record

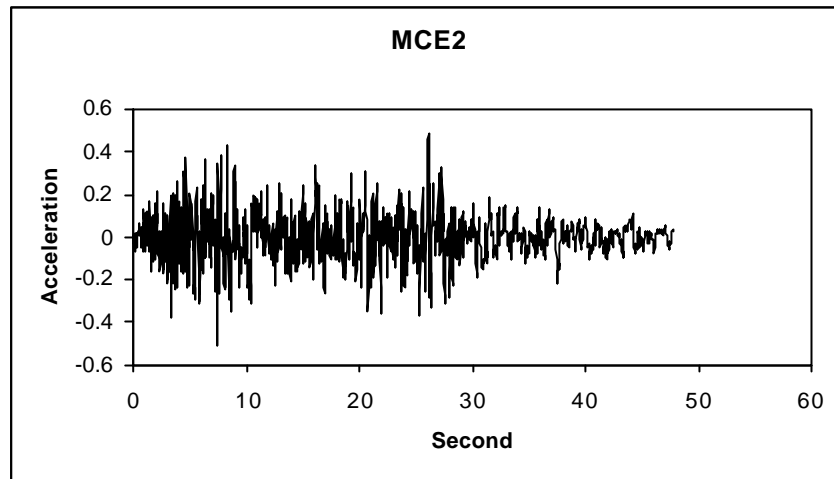


Fig. 7 - MCE2 Ground Motion Record

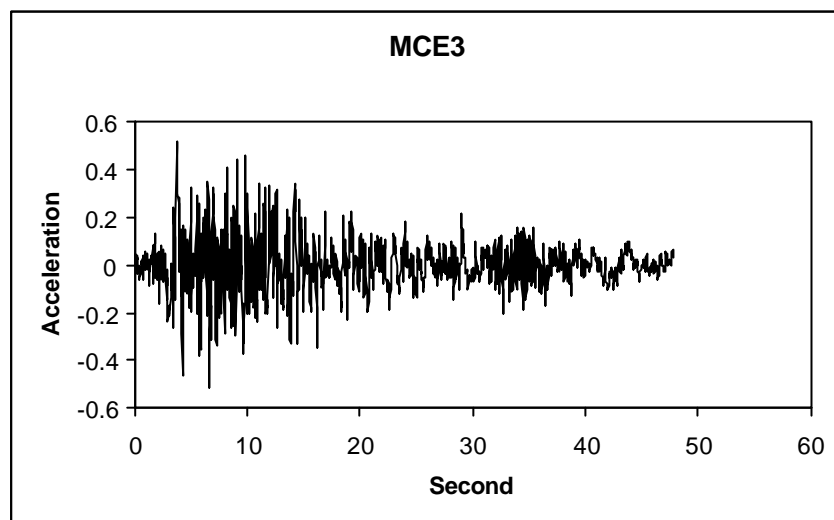


Fig. 8 - MCE3 Ground Motion Record

The third step was to decide the amount of supplemental damping, so that the optimal design can be achieved. The design parameter of fluid viscous damper is C , which can be obtained as function of mass and stiffness if Rayleigh's damping is used. In the light of equation 6, the C factor at each story can be obtained. The design parameters of displacement-dependent devices are SR and DR. The SR factor of 2.5 was selected. Table 5 shows the design parameters for each story.

Table 5 - Design parameter at each story

		C (kN sec/cm)	Stiffness (kN/cm)
Story	Stiffness (kN/cm)	$\xi=20\%$	SR=2.5
Roof	860	2160	2150
4th Floor	928	2332	2320
3 rd Floor	1060	2665	2650
2 nd Floor	1470	3670	3675

Steel moment frames with dampers were designed to remain elastic under Design Basis Earthquake (DBE). The goal was to limit the lateral drift to 1% of the story height and, to keep Demand-to-Capacity ratio (DCR) of the moment connections within an upper limit of 1 under the DBE events. In case of a larger earthquake such as Maximum Credible Earthquake (MCE), the plastic hinge formations and some structural damage would be expected. Therefore, the post-Northridge moment frames connections will provide the ductility to minimize the structural and nonstructural damage in the events of an MCE. The reduced beam-section connections were selected to produce an intended plastic rotation hinge zone.

Steel-moment frames with supplemental damping were designed to have DCR remain 1 using the Load and Resistance Factor Design (LRFD) method under site-specific time histories dynamic analyses. The time-history analyses with three pairs of DBE were required and the maximum response of the parameter of interest was used for the final structural member design. Each pair of site-specific time histories was applied simultaneously to the computer model, considering the most disadvantageous location of mass eccentricity. Story drift ratios were limited to less than 1% under DBE forces. Braces were designed to have a DCR remain within 2 so that the braces would have higher safety factors. This ensures that the damper units function properly under higher earthquake demands. The demands on the foundation were significantly increased due to the addition of the dampers. More piles and larger pile caps were designed in order to have the DCR be less than 1 at strength level.

7. Dynamic response for the steel building frame with Velocity-dependent devices

The structural seismic response of steel building frames, with 20% supplemental damping from velocity-dependent devices and without dampers, under LLE, DBE, and MCE, respectively, are shown in the Tables 6-10. The results also show the significant reduction on seismic demands due to the addition of velocity-dependent devices under various levels of ground shaking. When the undamped structure was subjected to the LLE ground motion, the DCR of beam and column members are within 1 and the story drift ratio is much smaller than 1%. This scenario agrees with the description of the FEMA-302 document that the allowable story drift ratio is limited to 1% for new building Seismic Use Group III [12]. Such story drift also corresponds to the description of FEMA-273 document for immediate occupancy performance level for steel moment frames. The story drift ratio was limited to 0.75% for the structure with velocity-dependent devices so that the building will meet the immediate occupancy performance level under 475-year-return-period earthquake events. The DCR of beam and column members is within 1 so that the structure remains elastic response under DBE. The enhanced structural performance then will be achieved. Some of beam and column members will experience some fractural failure during the strong earthquake events, such as MCE. For example, a DCR of 1.14 at the second floor beam and a DCR of 1.15 at the column base during MCE were shown in Table 10. It indicated some inelastic behavior will occur during the MCE. However, the Post-Northridge connections as well as the limited story drift requirement will reduce enough ductility demands so that enhanced structural performance still will be met during MCE. Fig. 9-12 show typical connection details designed for this example.

Table 6 - Story Drift Ratio for Frames with and without Velocity-Dependent Devices

	Without Damper			With Damper		
	LLE	DBE	MCE	LLE	DBE	MCE
ROOF	0.78%	1.26%	1.47%	0.42%	0.74%	0.78%
4 th Floor	0.78%	1.24%	1.46%	0.44%	0.77%	0.80%
3 rd Floor	0.73%	1.16%	1.36%	0.43%	0.72%	0.75%
2 nd Floor	0.63%	1.00%	1.17%	0.38%	0.64%	0.68%

Table 7 - Story Lateral Inertial Acceleration (g) for Frames with and without Velocity-Dependent Devices

	Without Damper			With Damper		
	LLE	DBE	MCE	LLE	DBE	MCE
ROOF	0.533	0.833	0.999	0.225	0.420	0.480
4 th Floor	0.371	0.648	0.750	0.237	0.322	0.396
3 rd Floor	0.405	0.563	0.630	0.223	0.296	0.350
2 nd Floor	0.375	0.463	0.566	0.186	0.338	0.390

Table 8 - Story Drift (cm) for Frames with and without Velocity-Dependent Devices under Various DBE

	Without Damper			With Damper		
	DBE1	DBE2	DBE3	DBE1	DBE2	DBE3
ROOF	1.26%	1.32%	1.28%	0.74%	0.61%	0.57%
4 th Floor	1.24%	1.34%	1.28%	0.77%	0.64%	0.57%
3 rd Floor	1.16%	1.27%	1.22%	0.72%	0.63%	0.54%
2 nd Floor	1.0%	1.14%	1.1%	0.64%	0.56%	0.48%

Table 9 - Story Acceleration (g) for Frames with and without Velocity-Dependent Devices under Various DBE

	Without Damper			With Damper		
	DBE1	DBE2	DBE3	DBE1	DBE2	DBE3
ROOF	0.833	0.875	0.758	0.420	0.345	0.422
4 th Floor	0.648	0.747	0.659	0.322	0.268	0.329
3 rd Floor	0.563	0.647	0.506	0.296	0.291	0.336
2 nd Floor	0.463	0.571	0.500	0.338	0.337	0.363

Table 10 - DCR for Beams & Columns with Velocity-Dependent Devices

	Beam			Column		
	LLE	DBE	MCE	LLE	DBE	MCE
ROOF	0.580	0.780	0.930	0.444	0.650	0.750
4 th Floor	0.660	0.920	1.002	0.710	0.920	1.110
3 rd Floor	0.740	0.940	1.095	0.520	0.670	0.780
2 nd Floor	0.750	0.990	1.140	0.800	0.987	1.150

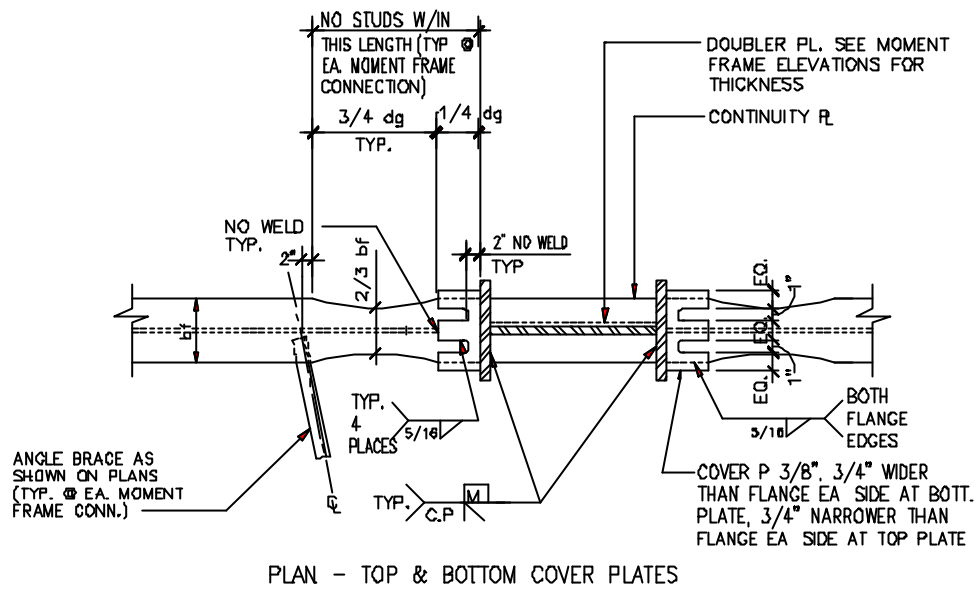


Fig. 9 - Cover Plates Details

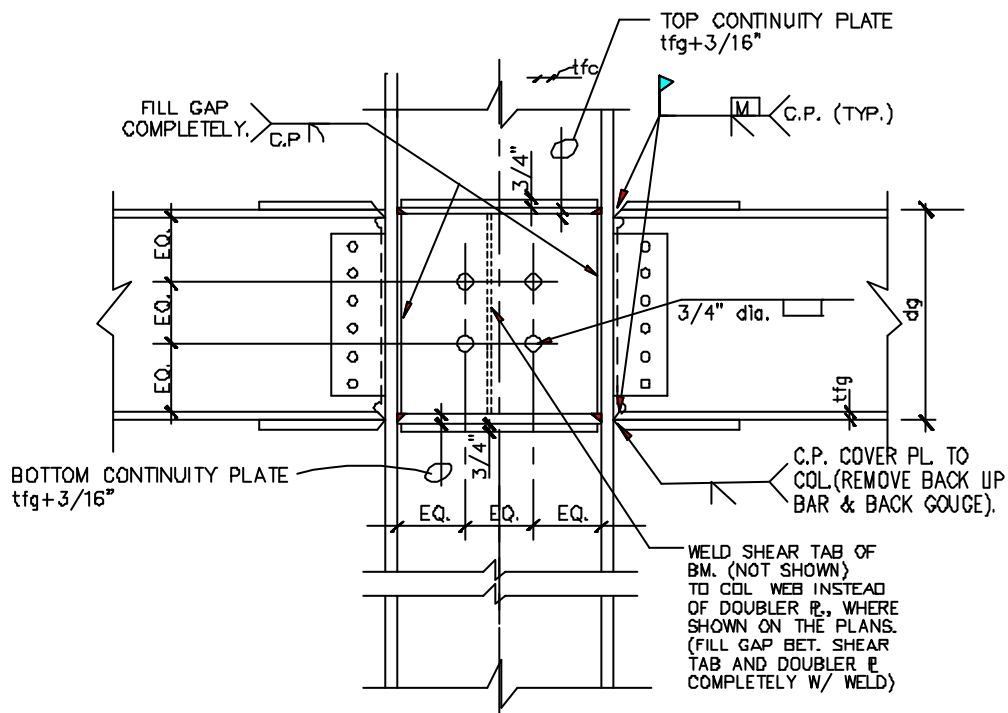


Fig. 10 - Post-Northridge Moment Connection

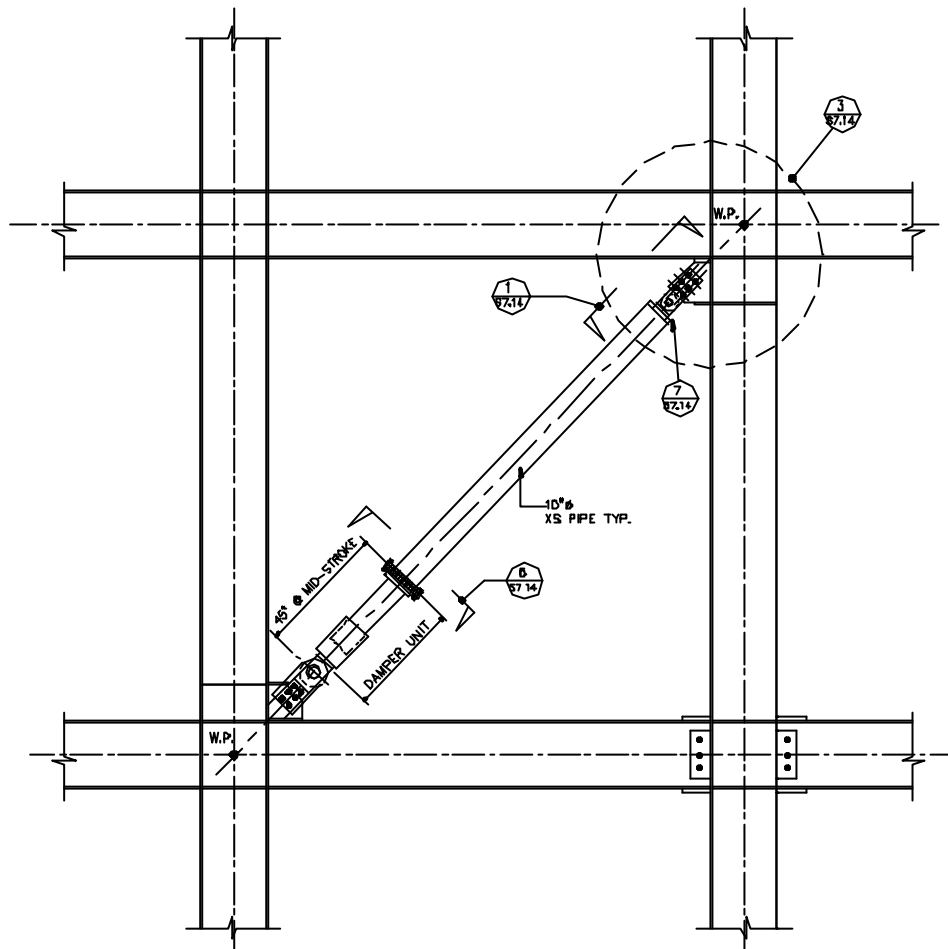


Fig.11- Brace with a Damper

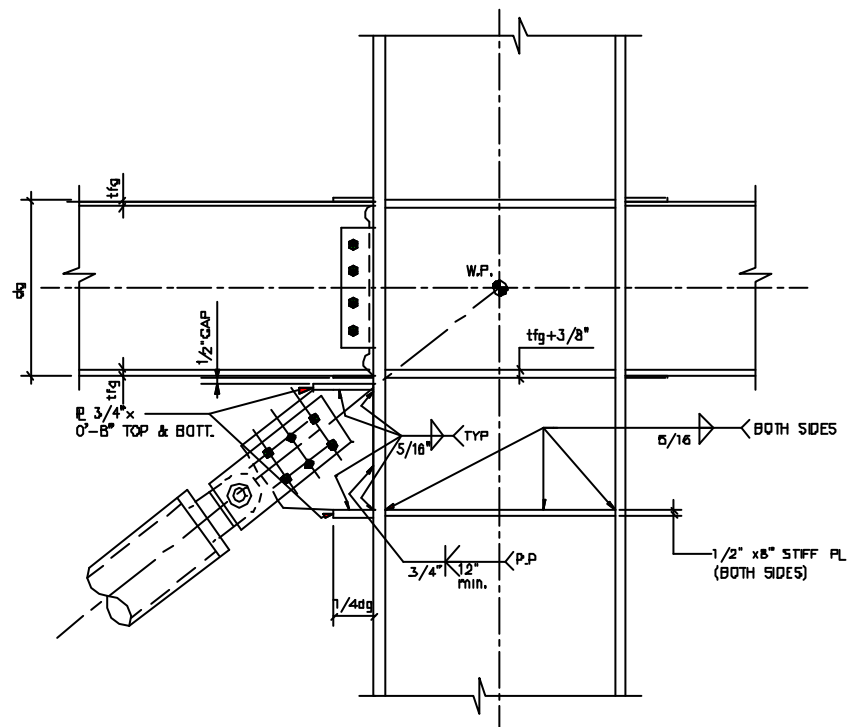


Fig. 12 - Brace Connection

8. Dynamic response for the steel building frame with Displacement-dependent devices

Similarly, the structural seismic response of steel building frames, with 20% supplemental damping from displacement-dependent devices and without devices, under LLE, DBE, and MCE, respectively, are shown in the Tables 11-13. The results show the significant reduction on seismic demands due to the addition of displacement-dependent devices under various levels of ground shaking. When the undamped structure was subjected to the LLE ground motion, the DCR of beam and column members are within 1 and the story drift ratio is much smaller than 1%. This scenario agrees with the description of the FEMA-302 document that the allowable story drift ratio is limited to 1% for new building Seismic Use Group III [12]. However, the story-drift ratios are much smaller compared to those of structure with velocity-dependent devices. On the other hand, DCR ratios for structure with displacement-dependent devices are much larger compared to those of structures with velocity-dependent devices. The addition of the displacement-dependent devices increases the total structural stiffness, therefore, reducing the story drift. As the result of the structure remaining in an elastic range under LLE, the demands on beam and column members increased. Such story drift also corresponds to the description of FEMA-273 document's immediate occupancy performance level for steel moment frames. The story drift ratio was limited to 0.75% for the structure with displacement-dependent devices so that the building will meet the immediate occupancy performance level under even 475-year-return-period earthquake events. Some of the beam and column members will experience some inelastic failure during DBE and MCE. For example, a DCR of 1.759 at the second floor beam and a DCR of 1.031 at the column base during DBE are shown in Table 13. It indicates some inelastic behavior will occur during the DBE. However, the Post-Northridge connections as well as the limited story drift requirement will reduce ductility demands enough so that enhanced structural performance still will be met during DBE and MCE. FEMA document suggests that DCR within 2 can be acceptable if Post-Northridge connections design is applicable.

Table 11 - Story Drift Ratio for Frames with and without Displacement-Dependent Devices

	Without Damper			With Damper		
	LLE	DBE	MCE	LLE	DBE	MCE
ROOF	0.78%	1.26%	1.47%	0.35%	0.65%	0.65%
4 th Floor	0.78%	1.24%	1.46%	0.36%	0.67%	0.67%
3 rd Floor	0.73%	1.16%	1.36%	0.37%	0.63%	0.65%
2 nd Floor	0.63%	1.00%	1.17%	0.33%	0.48%	0.58%

Table 12 - Story Lateral Inertial Acceleration (g) for Frames with and without Displacement-Dependent Devices

	Without Damper			With Damper		
	LLE	DBE	MCE	LLE	DBE	MCE
ROOF	0.533	0.833	0.999	0.550	0.788	0.648
4 th Floor	0.371	0.648	0.750	0.420	0.632	0.671
3 rd Floor	0.405	0.563	0.630	0.472	0.581	0.597
2 nd Floor	0.375	0.463	0.566	0.298	0.446	0.547

Table 13 - DCR for Beams & Columns with Displacement-Dependent Devices

	Beam			Column		
	LLE	DBE	MCE	LLE	DBE	MCE
ROOF	0.645	0.986	0.885	0.497	0.548	0.598
4 th Floor	0.915	1.495	1.340	0.694	1.057	0.899
3 rd Floor	0.856	1.759	1.552	0.606	1.031	0.968
2 nd Floor	0.870	1.397	1.511	0.806	1.073	1.267

9. Summary and Conclusions

Seismic performance and cost effectiveness were the primary issues during the design development phase of this project. Special steel moment frames with post-Northridge moment connections were the basic seismic frames designed to meet the Uniform Building Code of 1997. Since the story drift ratio is limited to less than 1% during DBE, the structural and nonstructural damage will be minimal. In an event of MCE, the post-Northridge moment connections will yield to provide additional structural damping to reduce the demand on the primary structural frames. Purchase of dampers along with foundations constitutes an additional cost. More piles with increasing length and larger pile caps were designed to meet higher performance demands. In addition, collector forces were found to be much higher than originally expected. Therefore, larger connections and beam collector members were also redesigned to meet this requirement.

The structural behavior during lower demand earthquakes such as LLE and DBE is not much different, since the structure remains elastic under those events. However, structural behavior is quite different under MCE because demands on the joints increase significantly. Table 8 shows that DCR is larger than 1 in some columns and beams under a MCE. The primary structural members contribute larger energy dissipation, which reduces the role of the dampers as a major mechanism for energy dissipation under MCE events. Due to the uncertainty of the input ground motion's characteristics, the conclusions for structural inelastic behavior are hard to make. Therefore, a nonlinear structural analysis for this kind of structural system is needed under MCE.

Energy dissipation devices should be designed with consideration of environmental conditions including wind, fatigue, ambient temperature, operating temperature, and other damaging substances. A good quality control program needs to be implemented to ensure that dampers will consistently function properly after installation. A well-established maintenance document is a useful tool to ensure better long-run service.

Supplemental damping devices are still new to many professionals. The seismic behavior of structures with dampers requires more research and testing to ensure reliability. Therefore, research efforts and professional education and training are urgently needed in this field.

REFERENCES

1. Krawinkler H. New Trends in seismic design methodology. Proceedings of 10th European Conference on Earthquake Engineering, vol. 2. A.A. Balkema: Rotterdam, 1995, pp. 821-830.
2. Ganzerli, G., Pantelides, CP. and Reaveley, L.D., Performance-based design using structural

- optimization. *Earthquake Engineering and Structural Dynamics* 2000; Vol. 29 No. 11, pp. 1677-1690.
3. Mahoney M, Hanson RD. An action plan for performance-based design. *Proceedings of the 6th US National Conference on Earthquake Engineering*, EERI: Oakland, CA, 1998.
 4. Hamburger, RO. A framework for performance-based earthquake resistive design. UBC/EERC-97/05, The EERC-CUREe Symposium in honor of Vitelmo V. Bertero, Berkeley, CA, 1997, pp. 101-108.
 5. Federal Emergency Management Agency, NEHRP Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Report No. 273 and 274, Washington, D.C., 1997.
 6. Federal Emergency Management Agency, Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Report No. 276, Washington, D.C., 1999.
 7. Miyamoto, HK. And Scholl, RE. Performance-based Design of steel Pyramid Using Viscous Dampers. *Proceedings of 65th Structural Engineers Association of California Annual Convention*, October 1-6, 1996, Maui, Hawaii, pp. 135-146.
 8. Jokerst, MS. And Soyer C. San Francisco Civic Center Complex: Performance Based Design with Passive Dampers and Welded Steel Frames *Proceedings of 65th Structural Engineers Association of California Annual Convention*, October 1-6, 1996, Maui, Hawaii, pp. 119-134.
 9. Soong, TT. Structural Control: Impact on Structural Research in General *Proceedings of the 2nd World Conference on Structural Control*, June, 1998, Kyoto, Japan, Vol. 1, pp. 5-14.
 10. Tseng, NT. and Lee, GC. Simple plasticity model of two-surface. *Journal of Engineering Mechanics*, ASCE, 1983, Vol. 109, No. 3, pp. 798-810.
 11. Bird, RB., Armstrong, RC. and Hassager, O. *Dynamics of Polymeric Liquids*, 2nd edition, Wiley and Sons, New York, 1987, Vol. 1, pp. 258-260.
 12. Federal Emergency Management Agency, NEHRP Recommended Provisions for seismic Regulations for New Buildings and Other Structures. Federal Emergency Management Agency, Report No. 302, Washington, D.C., 1999.