APPLICATION OF ENERGY DISSIPATION TECHNOLOGY FOR RETROFITTING STEEL STRUCTURES WITH VULNERABLE PRE-NORTHRIDGE CONNECTIONS

Omar Waqfi, Ph.D., S.E.
Ronald O. Hamburger, S.E.
ABS Consulting
Oakland, CA

Ravi Kanitkar, P.E.
The Crosby Group
Redwood City, CA

Abstract

Prior to the 1994 Northridge earthquake, modern welded moment-resisting steel frame structures were regarded as highly resistant to earthquake-induced damage and few engineers regarded earthquake-induced collapse of such structures as credible. This paradigm changed following the 1994 Northridge, California and 1995 Kobe, Japan earthquakes, creating a new class of potentially hazardous structures. In response to this new information, the Federal Emergency Management Agency retained a consortium of the Structural Engineers Association of California, the Applied Technology Council and the California Universities for Research in Earthquake Engineering, known as the SAC Joint Venture, to research the cause of the unexpected poor performance of these buildings and develop recommended design criteria. The resulting *FEMA-351* publication provides performance-based design criteria for the evaluation and upgrade of these structures. This paper presents the application of the *FEMA-351* criteria to the design of structural upgrades employing energy dissipation technology to an existing 10-story steel structure. The building was completed in 1991, employing the standard vulnerable moment connections. Alternative upgrade strategies were investigated, including modification of individual connections and application of energy dissipation criteria. The latter approach, using viscous dampers, was selected. Initial linear dynamic analysis showed significant plastic behavior in the beam-column connections for the MCE ground motion. A 3-D non-linear time history analysis was embarked upon with the joints modeled utilizing *FEMA-351* guidelines for fracture mechanisms. Non-linear analyses demonstrated that while connection fracture had little impact on the response, inelastic behavior of the frame significantly reduced the effective damping obtained from the energy dissipation devices.

Building Description

The East Bay Municipal Utility District (EBMUD) administration building is nine stories tall, with three below-grade basement levels and a rooftop, mechanical penthouse. The building is rectangular in plan, with setbacks in elevation and an overall length that is approximately 2.1 times its width for floors 1 through 3 and aspect ratio of approximately 3.2 for floors 4 through 9. At the first story, overall dimensions are approximately 145 feet north to south by 300 feet east to west. At the upper level, overall dimensions are approximately 94 feet north to south by 300 feet east to west. Above the third level, the structure is relatively regular in plan shape. It employs a distributed moment-resisting steel frame with nearly all beam-column connections detailed for moment resistance. The floors and roof are comprised of concrete fill on metal deck, spanning to structural steel beams and girders. The exterior curtain wall is a combination of precast concrete panels and glass. It was constructed between 1989-1991, to the provisions of the 1982 Uniform Building Code. Figure 1 is an exterior view of the building.

Figure 1. Exterior view of new administration building

The building shell and core were constructed nearly complete at the time of the 1989 Loma Prieta earthquake. This magnitude 7.1 event, with an epicentral distance approximately 60 miles to the south-southwest of the building is estimated to have produced ground shaking with a peak acceleration of approximately 0.2g and a duration of approximately 20 seconds at the building. This ground shaking caused some damage to exterior wall panels in the building and several nearby, modern steel frame buildings, with construction characteristics similar to this building are known to have experienced damage to framing connections. Based on this, in 2000, the building owner elected to perform an assessment of the building's probable performance in future earthquakes and when this was predicted to be unreliable, a decision was made to upgrade the structure.

Beam-Column Joint Investigation

The building has a total of 1500 joints that are considered vulnerable to earthquake induced damage. Approximately 600 of these connections are configured such that the beam joins with the strong axis of the column and the remaining 900 connections are configured such that the beams frame to the weak axis of the column. Following procedures suggested by *FEMA-352*, random samples of 85 welded joints were inspected. No connection damage was found in any of these connections. On the basis of these inspections, a confidence level of 95% that fewer than 5% of the connections in the building are damaged is established. The sampling and inspection criteria contained in FEMA-352, tolerates undiscovered damage to as many as 20% of the connections in a building. Thus, this building is well within the criteria suggested by FEMA-352 for acceptance of the building as not significantly damaged and no further inspections of building connections are recommended.

Seismic Hazard. The building site is located in Oakland, California, approximately three miles west southwest of the Hayward fault (Figure 2). Based on a recent study by the United States Geologic Survey, the Hayward fault is believed capable of producing earthquakes with a magnitude approximating 7 once every three hundred to five hundred years. Events with magnitudes as large as 7.25 may occur every thousand years or so. The last major event on this fault, was the 1868 Hayward earthquake, which ruptured the southern segment of the fault, extending from San Leandro south to Fremont. The United States Geologic Survey has recently estimated a 16% chance that the Hayward fault will generate a large magnitude earthquake along the northern segment, running through Oakland, sometime in the next 30 years and a 20% chance of such an event on the southern segment, during this same period. In total this yields a 32% chance of a large magnitude earthquake on the Hayward fault in the next 30 years.

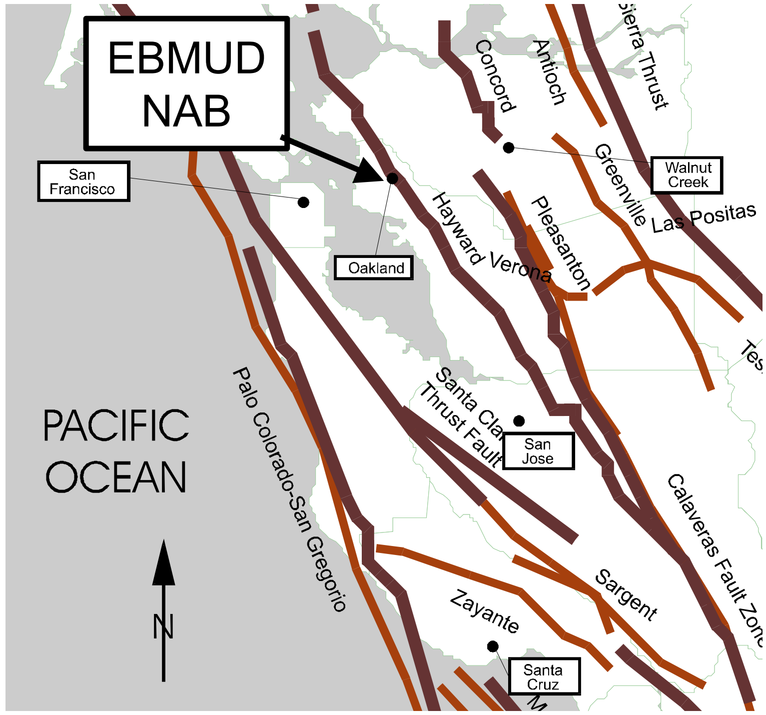

Figure 2. Fault map for San Francisco Bay Area

As part of the evaluation of the building, a site-specific response spectra and time history accelerograms for ground shaking with both the 475- and 975-year mean return periods were developed. These may be thought of as the levels of shaking that may be anticipated at the building site in the event of magnitude 7 and magnitude 7.25 events on the adjacent Hayward fault, respectively. Figure 3 displays representative response spectra for these events. The building is located sufficiently close to the northern segment of the Hayward fault that ground motion directionality effects are potentially significant. Accordingly, a suite of seven ground motion accelerograms developed from the 1995 Kobe, 1999 Kocaeli and 1992 Landers earthquakes were selected and scaled to the site-specific spectra, considering near field and directional effects.

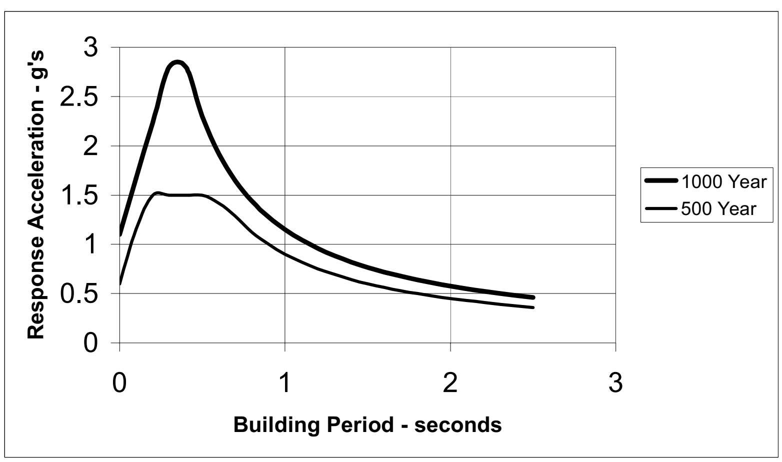

Figure 3. Response spectra for the building site

The building has three subgrade levels and is located on a site with moderately firm soils. On such sites, the interaction between a massive structure and the soils can tend to reduce the effective level of ground shaking felt by the building. Soil-structure interaction analyses were performed. The analyses determined that the reduction in ground motion due to soil-structure interaction effects is negligible. This may be attributed to the fact that the building has a somewhat long period of vibration that is significantly in excess of the fundamental period of motion of the site soils.

Performance Criteria. *FEMA-351* provides criteria for evaluating the ability of a building to meet two alternative performance levels under a specified level of ground shaking hazard; namely, the Immediate Occupancy and Collapse Prevention levels. *FEMA-351* also provides a method for estimating a level of confidence that a building would meet either of these performance levels, when subjected to a certain level of seismic hazard. A level of confidence on the order of 90% or greater indicates that the building is highly likely to provide the indicated performance. A level of confidence lower than 50% indicates that the building most likely will not provide the desired performance.

The level of confidence associated with a building performance evaluation is a function of several things including: the seismicity of the site, that is, the frequency at which earthquakes of given intensity are likely to occur, the strength, stiffness and deformation capacity of the building, the type of connections present in the building and their ability to withstand earthquake shaking without developing brittle fractures, the level of knowledge and understanding the evaluating engineer has with regard to the construction of the building, and the level of sophistication of the structural analysis performed to predict the interstory drift and column loads produced in the building by the earthquake.

FEMA-351 does not provide criteria for a third level of performance, termed the Life Safety level, that is often of interest to building owners. However, in a parallel document, *FEMA-273*, the Life Safety level is defined as occurring at building response levels that are 75% of those corresponding to the Collapse Prevention level. This definition was adopted for this project.

For existing buildings, no specific guidelines are available to determine the appropriate levels of confidence for a building to meet specific performance criteria. However, a companion publication to *FEMA-351*, *FEMA-350,* describes the confidence expected of new buildings designed to current code and the FEMA/SAC recommendations. Specifically, such a building should be able to provide:

- a 90% level of confidence that the building will not experience global collapse, and,
- a 50% level of confidence that the building will not experience local damage that could cause partial collapse,

Table 1 indicates the performance criteria adopted for the building.

Table 1. Seismic Performance Criteria

Earthquake	*Performance*	*Confidence*	*Performance*
1,000 Year Return Period	No Global Collapse	90% Confidence	No Global Collapse
	No Local Collapse	50% Confidence	No Local Collapse
500 Year Return Period	Global Life Safety	90% Confidence	Global Life Safety
	Local Life Safety	50% Confidence	Local Life Safety

FEMA-351 uses three important response quantities to evaluate the probable performance of the building. Namely; the interstory drift, axial compressive force in the individual columns and axial tension force in the individual columns.

The acceptable drift demand is defined by the following relationship:

$$D \leq \lambda\phi C / \gamma\gamma_a \quad (1)$$

Where, D is factored demand,
C is capacity

λ is a factored D/C limiting value for a given confidence level and Uncertainty Factor, β_{UT}
γ & γ_a are uncertainty factors associated with demand prediction and analytical procedures.

Tables 3 and 4 show detail calculations for the acceptable drift levels to achieve the confidence levels shown in Table 1. These factors were determined according to *FEMA-350* guidelines.
The acceptable drift limits and column axial loads are summarized below:

500- year Demands
Inter-story Drift Ratio at 2nd and 3rd floors (maximum beam depth is 36") < 0.0197
Inter-story Drift Ratio at 4th floor and above (maximum beam depth is 30") < 0.02191
Column Axial Compression < 0.56 C, where C = LRFD nominal compressive strength with k=1.0.
Column Splice Tension < 0.45 T, where T – LRFD nominal tensile strength

1,000 year (MCE) Demands
Inter-story Drift Ratio at 2nd and 3rd floors < 0.0271
Inter-story Drift Ratio at 4th floor and above < 0.0302
Column Axial Compression < 0.74 C
Column Splice Tension < 0.60 T

Table 3. Evaluation of Acceptable Drift Demand Levels (30" Deep Beam)

	Collapse Prevention		*Life Safety*	
	90% Global	*50% Local*	*90% Global*	*50% Local*
β_{UT}	0.4	0.35	0.35	0.3
λ	0.76	1.2	0.77	1.14
γ_a	1.06	1.06	1.04	1.04
γ	1.05	1.05	1.05	1.05
C	0.08	0.035	0.06	0.026
ϕ	0.7	0.8	0.8	0.8
$\gamma\ \gamma_a$	1.113	1.113	1.092	1.092
$\lambda\ \phi$ C	0.043	0.034	0.037	0.024
D<(%)	3.82	**3.02**	3.38	**2.19**

Table 4. Life Safety, 50% Confidence level for Local Collapse criteria with FEMA 350 referenced

β_{UT}	0.3	Table 3-12. Type 2 connection for mid-rise. Interpolated between Collapse Prevention and Immediate occupancy. Reduced by 0.05
λ	1.14	Table 3-6
γ_a	1.04	Table 3-8 for Type 2 connection for mid-rise. Interpolated between 1.06 and 1.02
γ	1.05	ANSYS results
C	0.026	0.75* 0.035 (Table 6-1)
ϕ	0.8	Table 6-1.
$\gamma\ \gamma_a$	1.092	1.05*1.04
$\lambda\ \phi$C	0.024	1.14*0.8*0.026
D (%)	2.19	0.024*100/1.092

Existing Building Investigation and Analyses. A linear dynamic analysis of the building, in its existing configuration was performed following the guidelines of *FEMA-351* using SAP2000 computer software. The floor drifts were evaluated and are shown in Figure 4 for the 500-year ground motions. The figure shows the floor drifts for the existing structure and the response of the retrofitted structure. The building does not meet the target confidence levels for Collapse Prevention performance at the lower two stories, either for 500 or 1,000-year ground motion, although for the 500-year level, it nearly meets this performance, providing an 85% level of confidence relative to avoidance of global collapse and a 25% level of confidence relative to local collapse.

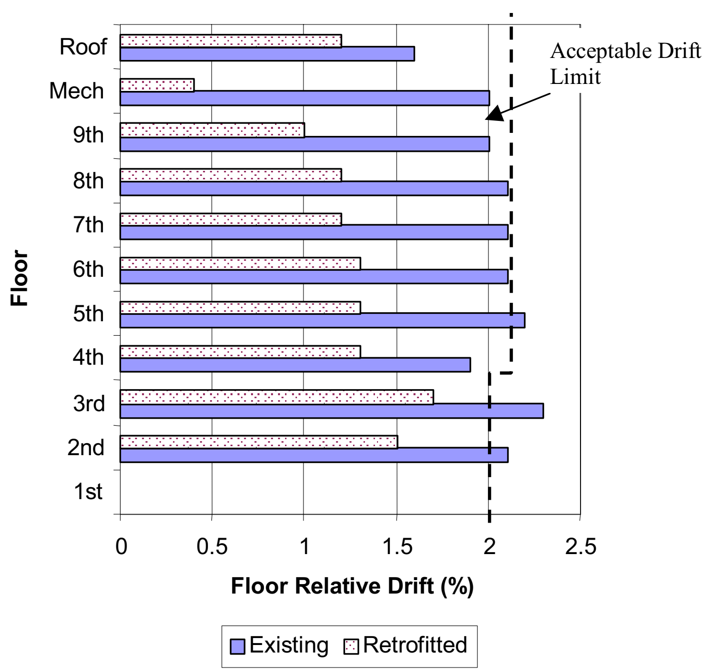

Figure 4. Projected drift demands-Linear Analyses– 500 year ground shaking.

The figure provides insight into the projected physical behavior of the building. Under strong ground motion in the longitudinal (east-west) direction, the building is likely to develop fractures in the moment resisting connections at the first and second stories at levels of ground motion slightly less than the 500-year motion. As a result of this, at larger levels of ground shaking, the building is likely to develop a soft/weak-story condition at these levels and thereafter, a significant portion of the building displacement occurs in these two stories, leading to potential development of instability.

Retrofit Alternatives

Following the determination that the building is not capable of reliably providing either Life Safety performance for 500-year ground shaking, representative of a magnitude 7 earthquake on the Hayward fault, or Collapse Prevention performance for 1,000-year ground shaking, representative of a magnitude 7.25 earthquake on the Hayward fault, two alternative concepts were evaluated for improving the probable building performance. The first concept consists of upgrade of selected moment-resisting connections to make them less susceptible to earthquake-induced fracture. The purpose of this upgrade approach is to inhibit the development of weak/soft stories in the building, and allow it to withstand more intense ground shaking before developing instability and potential collapse. The second concept is the installation of energy dissipation devices within the building. In this alternative, dampers are distributed in the building at various levels and locations as shown in Table 2. The purpose of this upgrade approach is to reduce the amount of lateral displacement experienced by the building in a given intensity of ground shaking, through benign dissipation of the earthquake's energy in the form of heat, that builds up in the energy dissipation devices. Under this alternative, two different options were investigated. The first option used viscoelastic (wall dampers) dampers and the second option utilizes viscous (piston) dampers.

Table 2. Distribution of Damper Locations

	Option 2b	
Story	*East-West*	*North-South*
1	3	3
2	3	3
3	3	3
4	2	2
5	2	2
6	2	2
7	2	2
8	2	1
9	2	1
Total	21	19

Note: Dampers at top three floors were eliminated subsequent to results of the non-linear analyses.

Retrofitted Building Analyses

A series of analyses were performed to determine the effectiveness and required number of dampers to achieve the performance criteria. Both linear dynamic and nonlinear dynamic analyses were performed. Linear dynamic analyses indicated that adequate building performance could be obtained by installing approximately 72 viscous dampers (at 38 to 40 building locations), ranging in capacity between 300 kips and 600 kips. In these analyses it was assumed that damper force would be proportional to the square of the damper stroke velocity. Figure 4 compares the predicted drift response for this option as compared to the response of the existing building. The drifts are shown for response of the building in the longitudinal direction, which was the controlling direction.

The linear analysis showed that although drift demands on the building would be substantially reduced by the damper upgrade, the lower stories would continue to be subjected to high drift demands and rotation demands on the beam-column moment connections in these stories would be in excess of the yield capacity. As a result, the structure would develop significant damping through hysteretic behavior. In order to account for this affect in the design of the viscous dampers, non-linear time history analysis were performed.

The non-linear time history analysis was performed using ANSYS software. Columns were modeled as plastic beam elements with bilinear axial stress-strain properties including isotropic strain hardening. Expected steel yield stress was used per *FEMA-351* with 2% post yield strain hardening. Rigid end offsets, corresponding to 50% of the beam depths, were included in the model. P-Δ effects were activated in the non-linear analysis and thus taken into account to capture the additional moment in addition to the axial load and the bi-axial bending on the columns.

The moment connection hysteretic behavior is modeled through the beams Each beam is modeled as two parallel plastic beam elements each possessing 50% of the total flexural stiffness and strength of the actual floor beam. Yielding is allowed to occur in both beams at the same yield end rotation. However, once the plastic rotation demand reaches the *FEMA-350* defined fracture rotation, one of the two beams is deactivated. This results in a loss of the beam stiffness and strength (reflecting a sudden joint fracture) with the remaining beam element providing only 50% of the original capacity. This 50% value was chosen to model the condition that when fracture occurs at one of the beam flanges (typically the bottom flange), the beam still retains some capacity due to the integrity of the remaining flange.

Since ANSYS does not support modeling of non-linear viscous force versus velocity relationships (i.e. $F = CV^{\alpha}$), the viscous dampers were modeled as linear damper elements (i.e. F=CV). The effective linear damper properties were based on calibrations to achieve the same maximum damper force at the peak velocities expected. Figure 5 shows one such calibration for a 300k viscous damper. The effect of this modeling compromise is to underestimate the damping energy dissipated at velocities lower than the maximum value. In addition to the hysteretic and the viscous piston dampers, viscous damping equivalent to 4% of critical is added to the structure to represent damping due to non-structural elements.

The additional damping provided by the viscous dampers resulted in approximately 15% reduction in the maximum drifts. Figure 6 shows the floor drifts comparisons between the original and the retrofitted building for the 500-year ground motions. As expected from the original linear analyses, a soft-story condition was observed between the second and third floors. A thorough study of the drift at all floors resulted in a reduction of the number of dampers. This was a direct result of the reduction of the drift demand at the higher floors due to softening of the second and third floor and the reduction in drift demands based on lower variability factor γ due to lower uncertainty in the seismic demand. In addition, the rotation capacity of the beam-column joints at the upper floor is higher due to the shallower beams.

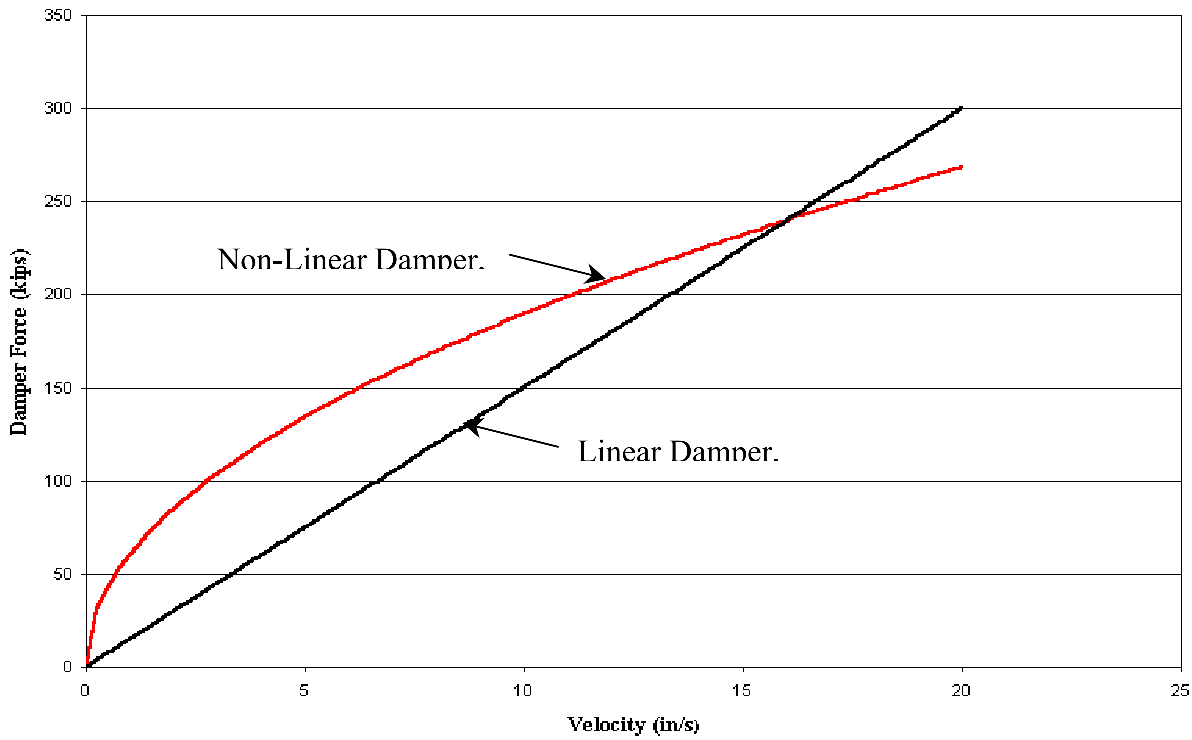

Figure 5. Calibration of linear versus non-linear damper

One of the important parameters monitored in this analysis effort was the performance of the welded moment connections. It was observed that significant joint fractures occurred within the as-built structure, especially at the MCE level ground shaking. The incorporation of viscous dampers partially mitigated this phenomenon. Within the upgraded structure some joint fractures were observed, mostly at the lower floors. Although the bottom flange rotation demands were exceeded in several locations, the top flange always remained intact. Thus the beams did not lose all the their flexural capacity and also maintained their gravity shear load carrying capacity.

The analysis captured the combined demands of bi-axial moments and axial forces on the columns. A majority of the columns in the building remained elastic during the non-linear analyses. Inelastic behavior was observed for some columns on the lower floors. The maximum inelastic strain in any column was about twice the yield strain, i.e. a ductility demand of about three, which was deemed acceptable. The column axial compressive and tensile stresses were within the limits set by the acceptance criteria.

Conclusions

It is confirmed in this study that a significant reduction in the floor displacement can be achieved by using energy dissipation devices. However, it is of interest to recognize that the linear analyses showed a more significant reduction in floor drifts than the non-linear analyses. This is likely attributed to the significant hysteretic damping present in the non-linear model of the original structure. This damping was not captured in the linear analyses. It was also concluded that the formation of a soft-story condition at a particular (between the second and third floors) created a pseudo base isolated (partially) effect on the floors above the third floor. It was observed that the displacement demands on the upper floors from the non-linear analysis were lower than those obtained from the linear analysis. As a result of the revised calculations of the variability of the seismic demands and the reduced drift demands on the upper floors, the number of dampers to be installed is reduced by 30%.

It is also concluded that fracture of a significant number of the beam-column connections at a particular floor would not result in global collapse mechanism.

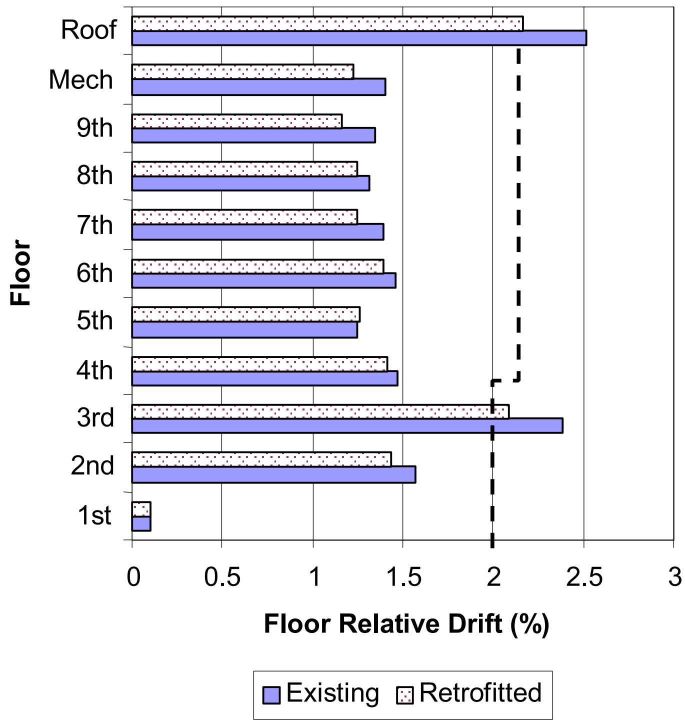

Figure 6. Projected drift demands- nonlinear analyses– 500 year ground shaking

References

International Conference of Building Officials, 1982, *Uniform Building Code*, Whittier, California.

SAC Joint Venture, 2000, *Recommended Seismic Evaluation and Upgrade Criteria for Welded Steel Moment Frame Buildings Report No. FEMA-351*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture, 2000, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment Frame Buildings Report No. FEMA-352*, Federal Emergency Management Agency, Washington, D.C.

Applied Technology Council, 1997, *NEHRP Guidelines for Seismic Rehabilitation of Buildings Report No. FEMA-273*, Federal Emergency Management Agency, Washington, D.C.

Applied Technology Council, 1997, *Commentary to NEHRP Guidelines for Seismic Rehabilitation of Buildings Report No. FEMA-274*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture, 1995, *Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures Report no. FEMA-267*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture, 1996, *Interim Guidelines Advisory No. 1, Supplement to FEMA-267 Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures Report No. FEMA-267A*, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture, 2000, Recommended Seismic Design Criteria for New Steel Moment Frame Buildings, Report No. FEMA-350, Federal Emergency Management Agency, Washington, D.C.