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SEISMIC RETROFIT OF A STEEL MOMENT FRAME STRUCTURE USING VISCOELASTIC DAMPERS

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ABSTRACT

This paper describes the analysis and design process for the seismic retrofit using viscoelastic dampers of a four-story steel moment frame structure in the Los Angeles area. An extensive array of linear and non-linear analyses were conducted during the retrofit design process using site specific earthquake demands. Based on the analyses performed, the retrofitted building is expected to perform in conformance with the performance criteria established for this project.

KEYWORDS: Viscoelastic (VE) Dampers, Global Damping, Push-Over Analysis

INTRODUCTION

The building under consideration (from the 1980s) is approximately 100 m by 66 m in plan on the ground floor, with offset floors above. The lateral force resisting system is composed of perimeter steel moment frames with a horizontal offset at the fourth floor.

Analysis of the as-built structure showed that the building did not meet the stress and drift criteria of the 1994 Uniform Building Code even for the equivalent static lateral force method. Studies using site specific ground motions showed very high inter-story drifts with structural members over-stressed to a point where catastrophic failure in the form of a story collapse was deemed certain at the Maximum Credible Earthquake (MCE).

The primary goal of the project was to keep the structure essentially elastic during a Design Based Earthquake (DBE) and stable (without collapse) during a Maximum Credible Earthquake (MCE). Dames & Moore, Inc. developed the probabilistic site-specific DBE and MCE ground accelerations for the retrofit. A peer review panel, in conjunction with the design team, developed detailed structural performance criteria to evaluate the performance of the retrofitted structure.

Viscoelastic dampers were selected for their dual benefit of adding stiffness to the structure and achieving passive energy dissipation. The initial damper design was developed through response spectra analyses and linear time-history analyses using the computer software ETABS. The design was finalized using a detailed 3D non-linear model developed in ANSYS. Non-linear pushover and time history analyses were performed to evaluate the building and damper performance.

As an added level of detail the constitutive properties of the viscoelastic material (VEM) were programmed into an ANSYS element. This element simulates the frequency and temperature dependencies of the material, capturing the effects of higher modes and of softening of the material due to temperature rise as the VEM is strained. This project represents the first design application of this detailed non-linear element.

The final retrofit design involved placing 8 dampers on each floor of the building (4 in each principal direction). It was observed that adding dampers to the structure significantly improved its seismic performance and enabled the retrofit goals to be achieved.

RETROFIT PERFORMANCE CRITERIA

The objective of the retrofit design was to obtain a structure that would comply with an extensive array of performance criteria. These criteria were developed to ensure that the upgraded building would perform satisfactorily during the design ground motions. The various criteria are briefly described below:

1. **Global Performance Objectives**

- Global Structural Damping:

  A minimum of 15 % and a maximum of 30 % of critical damping shall be provided for the fundamental modes of vibration of the structure.

- Load Transfer Considerations:

  Ensure that a clear load path is achievable to transfer the load from the damper locations to the existing structure and finally to the foundation. Verify that the retrofit will not adversely affect the capacity of the structure to resist all possible loads, at a local level (due to added dampers) and on a global level.

- For Design Based Earthquake (DBE)

  I. Global Structural Performance: Under the Design Based Earthquake (with a 10 % probability of exceedence in 50 years), the structure shall remain stable with significant reserve capacity. Non-structural damage will be reduced to non-hazardous levels.

  II. Inter-Story Drift: The maximum inter-story drift will be limited to 1 % of the story height under the Design Based Earthquake.

- For Maximum Credible Earthquake (MCE)

  I. Global Structural Performance: Under the Maximum Credible Earthquake (with a 2 % probability of exceedence in 50 years), the building shall remain stable without collapse.

  II. Inter-Story Drift: Under the Maximum Credible Earthquake, the maximum inter-story drift shall be limited to 1.5 % of the story height.

2. **Component/Subassemblies Performance Objectives**

- For Design Based Earthquake (DBE)

  I. Rotation Demands on Joints: The in-elastic rotation demand on moment connections will be restricted to 0.005 radians under the DBE loading.

  II. Member Performance: The structural elements will be checked for over-stress within the upgraded structure. AISC LRFD (American Institute of Steel Construction, Load Resistance Factor Design) 2nd Edition will be the code used to check the members. Under DBE loading, the members will stay essentially elastic.

- For Maximum Credible Earthquake (MCE)

  I. Member Performance: Under MCE loading, inelastic behavior is expected in some structural members. Some limited local inelastic behavior shall be permitted such that the structure is not compromised.

3. **Site Specific Ground Motions**

All the analyses for the structure were performed using site specific ground motions developed for this project. Site specific probabilistic response spectra and the corresponding three separate pairs of time histories were provided for both the DBE and MCE level earthquakes. Some site specific ground motions and the target spectra are shown in Figure 1.

LINEAR VISCOELASTIC THEORY

In structural applications, a Viscoelastic (VE) damper typically consists of VEM slabs sandwiched between relatively rigid steel plates.

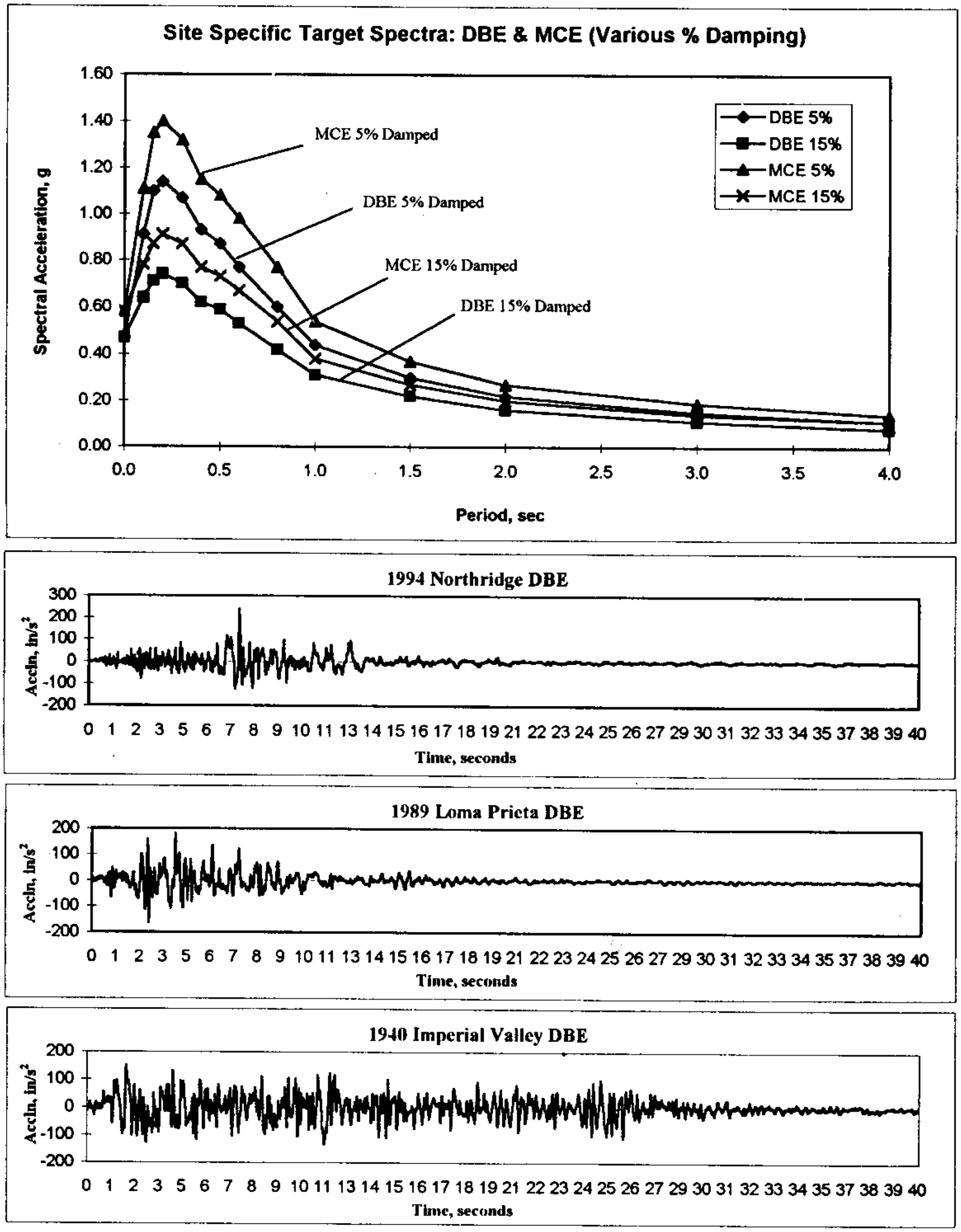


Fig. 1 Site specific response spectra and sample ground motion

The VE damper can be characterized by the storage, K', and loss, K'', stiffnesses which are related to G' and G'' (moduli) as

$$K' = \frac{G' A}{h}, \qquad K'' = \frac{G'' A}{h} \qquad \text{and} \qquad \eta = \frac{K''}{K'} \tag{1}$$

where A is the total shear area and h is the thickness of the VEM slab (Kasai et al., 1993). An important advantage of the VE damper is that the damper is linearly scaleable as shown in Equation (1). K'' can be further related to the viscous damping constant, c, as

$$c = \frac{K''}{\omega} = \eta \frac{K'}{\omega} \tag{2}$$

where ω is the damper operating frequency. To use this linear theory it is necessary to know the fundamental frequency of the structure, the initial temperature of the VEM, and the anticipated strain level in the VEM. From this information the values of G' and G'' can be determined as discussed in Lai et al. (1996). The damper can then be linearly modeled using a spring for the stiffness term (K') and a dash-pot for the viscous term (c).

DEVELOPMENT OF THE VISCOELASTIC DAMPER RETROFIT

Once the concept of using Viscoelastic damping was decided upon, the development of the final retrofit scheme progressed through several steps. Given the global damping and the building performance requirements, the first step was to develop preliminary damper sizes. This was required to determine the total number of dampers to be placed within the building, which has a direct impact on the cost. Once the number of dampers and their approximate sizes were determined, more detailed analyses were conducted to finalize the design. This section describes the retrofit development process.

1. Preliminary Damper Design

The objective of the preliminary damper design was to determine how much global damping would be required to improve the structural performance so as to comply with the performance criteria. Once the global damping requirement was determined, the next step was to compute the total number of dampers required and their approximate sizes. Linear response spectra analyses were used to perform the preliminary design of the VE dampers. Viscoelastic dampers add both stiffness and global damping to the structure. The added stiffness was incorporated into the linear model by introducing equivalent bracing in the structure. Appropriately damped response spectra were used to represent the global damping added by the VE dampers. In this way, both the stiffness and damping characteristics of the dampers were included in the response spectrum model. The spectrum analyses demonstrated that adding 15-20 % global damping would yield the required performance improvements.

Using the story drift demands and the target global damping, preliminary damper sizes were developed. The global damping provided by these preliminary sizes was determined using the Modal Strain Energy (MSE) method (Soong and Lai 1991 and Chang et al. 1995). The sizes were determined by an iterative process during which the results of one analysis were used to modify the parameters for the next analysis and so on. This process was iterated until a good correlation was obtained between the damper sizes, the target global damping and the structural performance. This process eventually yielded the total number of dampers and their approximate size requirements.

As a final step of the preliminary damper design, the damper sizes developed were used to evaluate the structural response to all the site-specific spectra provided. Site-specific spectrum analyses were performed in both the principal building directions for the DBE and MCE levels. Based on the building performance results (e.g., story drifts, member demands, etc.), it was concluded that the damper sizes developed in the preliminary design would be adequate.

2. Design of Discrete Dampers

The analysis performed thus far addressed the global design issues. The next step was to determine if the dampers designed in the preliminary stage would yield the target global damping and improve the structural performance when installed as discrete units within the building. The seismic response of each VE damper is a function of its location, the stiffness of the structure supporting it, modal and torsional characteristics of the structure, etc. All further analyses were aimed at finalizing the damper requirements on a local level. Linear and non-linear time history analyses were used for this purpose.

2.1 Linear Time History Analyses

The dampers were modeled in the ETABS linear analyses using parallel stiffness and viscous elements (Kelvin model). This is as per FEMA 273 (ATC 33, Ballot Version, September 1996), Section 9.3.3.2(A). The damper properties were based on a design temperature of about 70 - 75°F. To evaluate the damper performance at different temperatures, several bounding analyses were performed (these are described later in this paper).

The linear analyses were performed using all three pairs of time histories provided. To capture directional effects, the ground motions were used 100 % in one principal direction and 30 % in the perpendicular direction. From the time history analyses building displacements, inter-story drifts, maximum damper shear strains, damper shear force, and member demand were monitored. It was observed that the damped structure inter-story drifts were well within the criteria established for the retrofit (i.e., 1 % of story height at DBE and 1.5 % of story height at MCE). A comparison of the as-built and damped drifts is shown in Figures 2a and 2b. The damper shear strains were seen to be approximately 100 % at DBE and 125 % at MCE. The member demands were used to compute the AISC LRFD member utilizations. At the DBE, the structure was found to be essentially elastic. The only over-stress was found to be in the first floor columns and to the order of 1.0 to 1.10. The axial capacity of these columns was checked and was found to be adequate (the over-stress was a combination of the flexural and axial demands).

2.2 Linear Bounding Analyses

It is well known that viscoelastic materials are temperature sensitive. At low temperatures, the VE material is stiffer, while at higher temperatures it is more flexible. As a VE damper undergoes cyclic shear deformation during a seismic event, the temperature within the material increases. Although it is assumed that dampers are within a controlled environment inside the building, variations can occur in the ambient temperatures due to power curtailment, breakdown of air-conditioning systems, or shut-down during holidays. In addition to temperature effects, long term effects on the damper material have to be considered, e.g., stiffening of the material with age. These changes in the damper stiffness affect the damper performance, which in turn affects the structural performance of the building. In order to capture scenarios like those mentioned above, several bounding analyses were performed. These bounding scenarios were incorporated within a set of linear time history analyses.

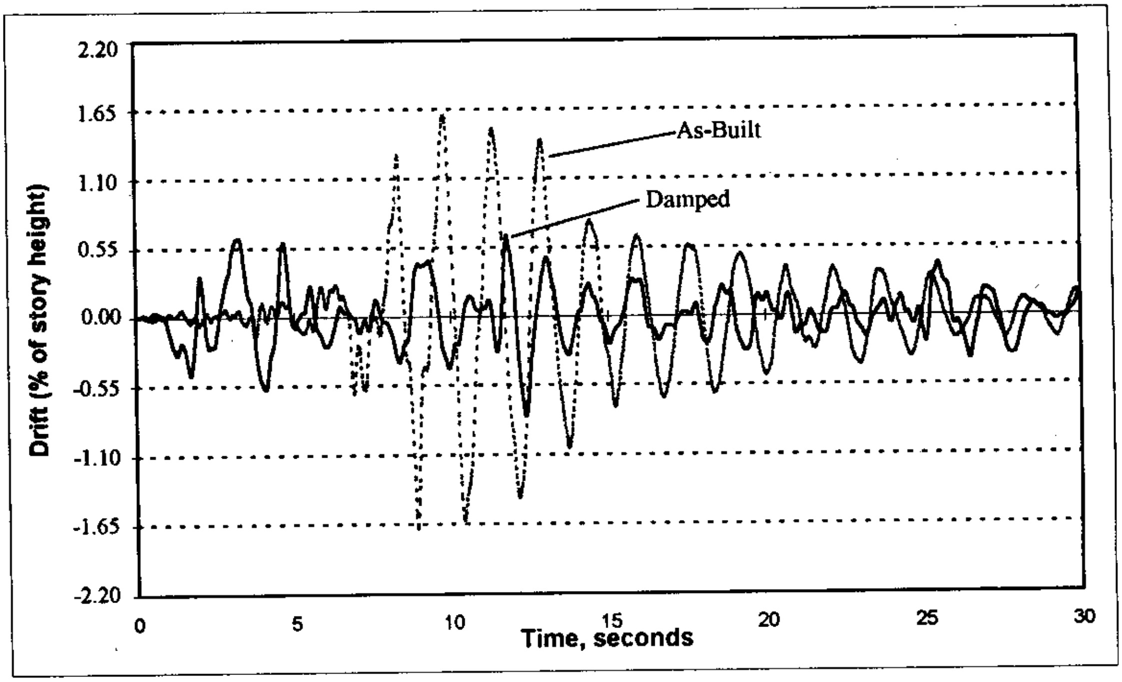
A lower bound damper ambient temperature of 60°F was defined. For the upper bound, a damper ambient temperature of about 80°F, with a temperature increase of about 10°F was considered appropriate. Time history analyses were performed with the VE damper properties based on the above temperature bounds.

From the lower bound analyses, peak damper forces were extracted for evaluating structural performance. At the higher bound, displacement response was checked. It was found that the peak inter-story drift at the fourth floor exceeded the drift limit at the upper bound temperature. However, the exceedence was small and was judged to be acceptable. The bounding analyses demonstrated that the damper design would be adequate over a significant realistic range of ambient temperatures within the building.

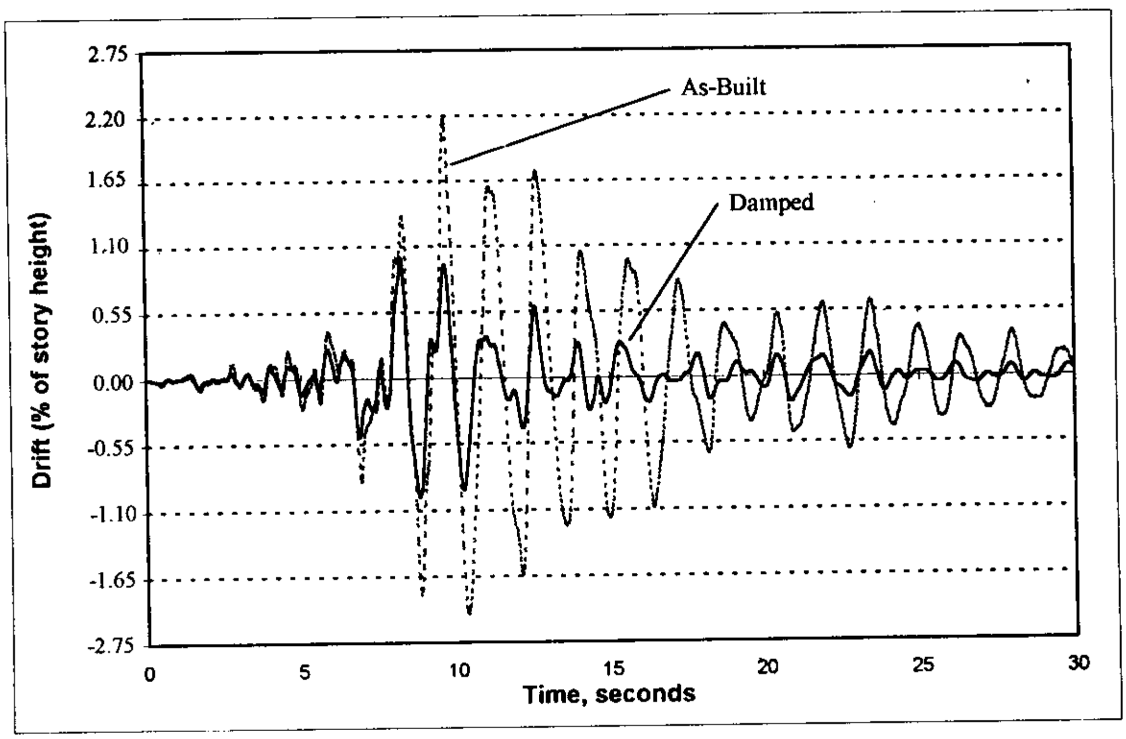
Since the stiffness of the VE material changes with temperature, the temperature bounding analyses were used to evaluate damper behaviour for the corresponding stiffness bounds. The damper stiffness varied from about +120 % to –75 % of the stiffness at the assumed ambient temperature. This extensive range of damper stiffness variation was considered sufficient to envelop any stiffness changes in the damper material due to long term effects such as aging.

2.3 Non-Linear Time History Analyses

Two types of non-linear analyses were performed using the ANSYS finite element code. In the first type of analyses the structure was kept linear while the VE dampers were modeled with the ANSYS non-linear damper element. In the second type, the dampers were modeled with equivalent linear properties while the structure was modeled with non-linear elements.



3rd Floor Y Drifts, 1994 Northridge DBE 30% X + 100% Y



3rd Floor X Drifts, 1994 Northridge MCE 100% X + 30% Y

Fig. 2 Comparison of as-built and damped story drifts (linear TH analyses)

2.3.1 Analyses with Non-Linear Damper Element

The objective of this portion of the analysis was to determine the building's response to the site-specific loads as a function of the temperature and frequency dependencies of the VEM. For the design of a viscoelastic damper using a linear model, the VEM properties are taken at a fixed ambient temperature and a fixed frequency, which is based on the fundamental frequency of the structure. This is an approximation of the damper performance since the temperature of the VEM rises during a seismic event and the operating frequency changes continuously. In addition, the VEM is known to exhibit nonlinear behavior when subjected to high strains (Kasai et al., 1993 and Lai et al., 1996). This is due to the temperature rise in the VEM, which has the effect of lowering the moduli of the material.

To accurately address this non-linear behavior of the VEM, an ANSYS element was developed using the fractional derivative method of relating the stress to strain (K. Kasai, Lehigh University). Calibration of the element was conducted by comparing the analytical results to several experimental test results. The extremely close agreement between the two sets of results demonstrated the validity of the new element. Time history analysis using the non-linear element allowed the design team to evaluate the damper performance with respect to the temperature and frequency variations. Temperature rise in the dampers was closely monitored to validate the assumptions made in the linear bounding analyses. The response of the structure to the variations in the damper properties was also recorded. Along with determining the response of the structure, these analyses were also used to check the equivalent properties used for the VEM in the linear analysis.

Analyses with the non-linear viscoelastic element lead to the conclusion that the damper design was adequate to ensure satisfactory structural performance even with the various property changes expected in the VEM. Figure 3 is a plot of the damper temperature rise on a per floor basis. Note that the temperature of the dampers has increased approximately ten degrees thirteen seconds into the earthquake, which is when the structure's response is maximum.

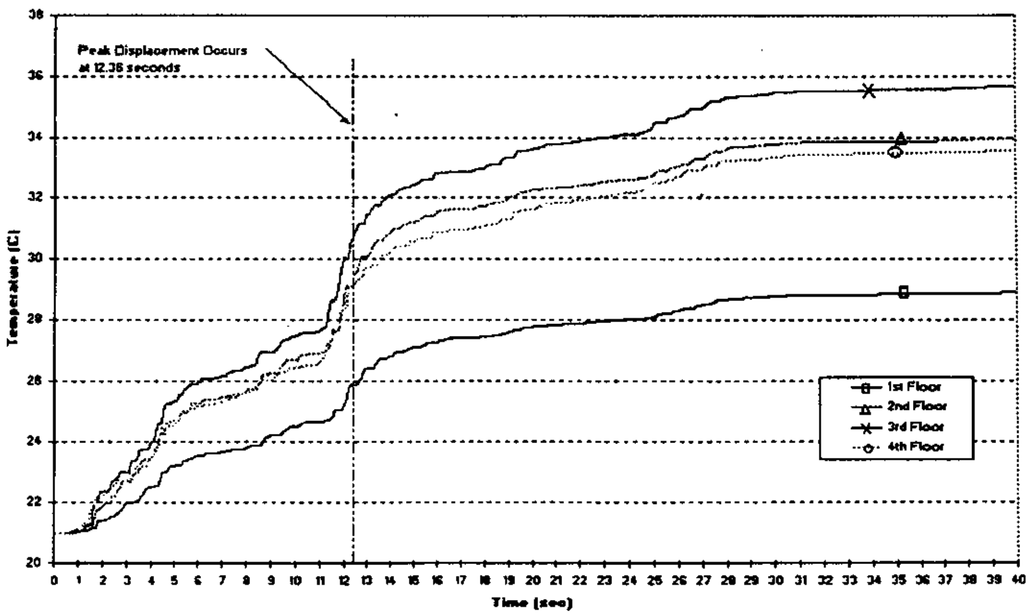


Fig. 3 Temperature rise in VEM due to MCE Imperial Valley loading

2.3.2 Analyses with Non-linear Structure

This part of the analysis was aimed at evaluating the stress levels in the structure during the seismic events. The results were used to comment on the structural stability at the MCE ground motions. Steel members with bilinear stress-strain material properties were used in the 3-D non-linear model. Dynamic time histories analyses were conducted using the Imperial Valley earthquake (DBE and MCE). The Imperial Valley earthquake was selected because it produced the largest response in the linear analysis. In the MCE analysis, some yielding was observed at the base of some first, second, and third floor columns and some second floor beams. Figure 4 is a plot of the stresses in a typical end bay showing some yielding in first and second floor columns. It is important to note that although some yielding occurred at the MCE level, the structure is still stable.

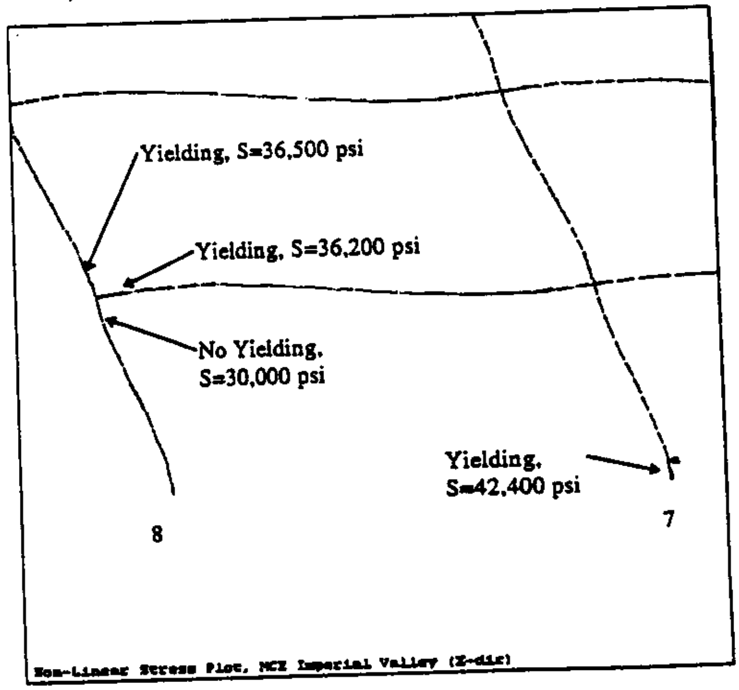


Fig. 4 Non-linear stress plot for members in typical end bay (MCE loading)

In the 1994 Northridge earthquake, welded moment connections performed poorly. This poor performance was attributed to the high inelastic joint rotation demands on such joints. To address this concern, total joint rotation demands were monitored in the retrofitted structure at the DBE ground motions. The peak total joint rotation demands were found to be approximately half a percent of a radian (0.005 radian). Based on results from the SAC Joint Venture (FEMA 267, 1995) on welded moment connections, it has been observed that pre-Northridge moment connections can be considered adequate up to total rotations of about 0.01 radian.

2.4 Non-Linear 3-D Push-Over Analyses

As a complement to the non-linear time history analyses, a non-linear static incremental pushover analysis of the structure was performed in one of the principal directions of the structure. The objective of this analysis was to determine the structural stability at the maximum displacement demands determined from the non-linear time history analyses.

Horizontal loads, based on the first fundamental mode, were applied to the structure at each floor. In addition to the horizontal loads, gravity loads were applied at each floor of the structure to capture second order (P-Δ) effects. The static load was incrementally increased and the structural response (in terms of member stresses) was monitored. Non-linear events, such as hinging in the steel members, were recorded.

For the as-built structure, the first non-linearity occurred around 50 % of the maximum expected roof displacement. At 75 % of the maximum expected roof displacement, the non-linearity started to spread to the tops of some of the 1st floor columns. The P-Δ effects became significant and the load carrying capacity of the structure diminished rapidly. The pushover was continued up to a point after which the program did not converge due to excessive displacement. This indicated that the structure was unstable, due to formation of a 'collapse' mechanism.

The retrofitted structure pushover curve shows the effect of the added stiffness due to the dampers (approximately 35 % increase in the initial stiffness was seen). The first deviation from linearity was seen at about 60 % of the maximum roof displacement expected. At the maximum roof displacement the capacity curve still showed an upward slope with reserve capacity.

Using the push-over results, a capacity spectrum was developed. This capacity curve was plotted on top of the damped demand curves (Figure 5 shows the pushover curves at the MCE level), to evaluate the effectiveness of the added damping. It was found that the added damping of 15 %-20 % would significantly reduce the displacement demands on the structure and ensure stability of the structure at the peak MCE demands.

MEMBER EVALUATION AND RETROFIT DESIGN

Based on the analyses described above, realistic member force demands were determined. Since the damper induces a force in the structural elements to which it is attached, extensive member checks were performed. The peak damper forces from the time history analyses were used for these member evaluations. The peak damper forces were further scaled up for an added factor of safety.

Existing members were checked for global effects due to eccentric brace loading at connections and local effects due to concentrated local loads (web crippling, local web yielding, flange local buckling etc.). The existing members in the moment frames were checked to ensure that they could resist the demands induced within the damped structure. In addition, the load transfer mechanism between the damper and the beams, columns and deck was reviewed. The damper assembly components, damper connections to existing structure, out-of-plane kickers for the dampers, and the foundation retrofit were designed for 125 % of the expected peak damper force. Figure 6 shows an elevation of a typical damped bay with a detailed view of the damper assembly. The dampers were manufactured in modules for ease of installation. Figure 7 shows typical sections at the damper locations.

PROTOTYPE TESTING OF VISCOELASTIC DAMPERS

As part of the retrofit design, an extensive testing program was conducted to verify that the dampers performed as assumed in the design. The tests were performed at the University of California in Berkeley, California. The results of the testing demonstrated that the behavior of the VE dampers (with temperature and frequency dependency) can be very accurately predicted with the use of the non-linear damper constitutive model described earlier in the paper. This conclusion validated the design assumptions. The description of the test program and the results is beyond the scope of this paper.

CONCLUSIONS

The four-story steel moment frame structure was designed with a viscoelastic damper retrofit to reduce the inter-story drifts, joint rotation demands, and global force demands on the structure. The as-built structure was deemed to be seismically deficient because of the excessive inter-story drifts and member over-stress. Extensive analyses of the retrofitted structure have demonstrated that the retrofit will significantly improve the seismic performance of the structure. At the DBE, the structure will stay essentially elastic with reserve load carrying capacity. At the MCE, the structure will be stable with some local inelasticity. The inter-story drifts in the damped structure comply with the performance criteria established for the design. The inelastic joint rotation demands at the welded moment connections will be reduced and should allow ductile behavior of the joints. The construction of the retrofit was completed in January 1998.

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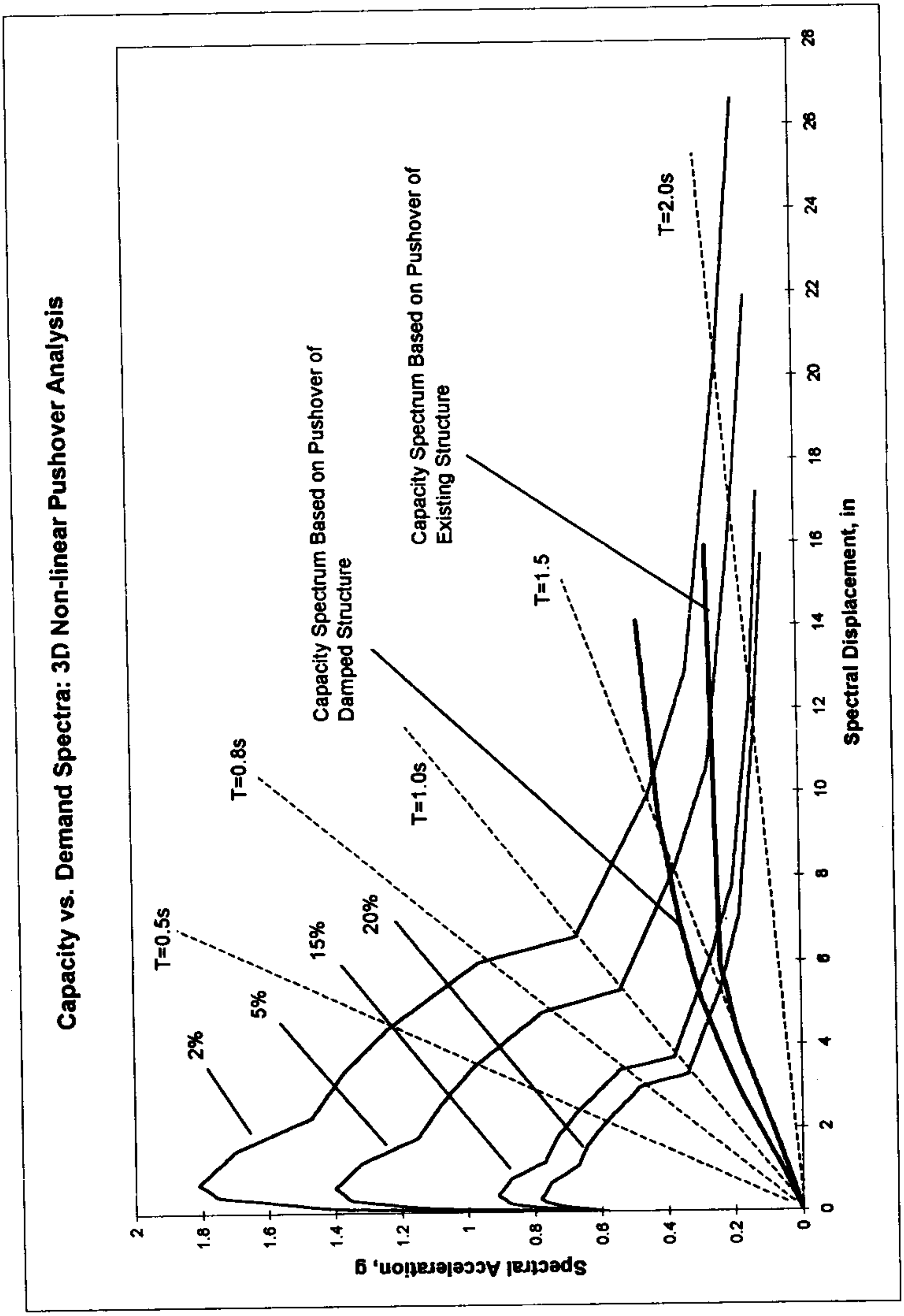
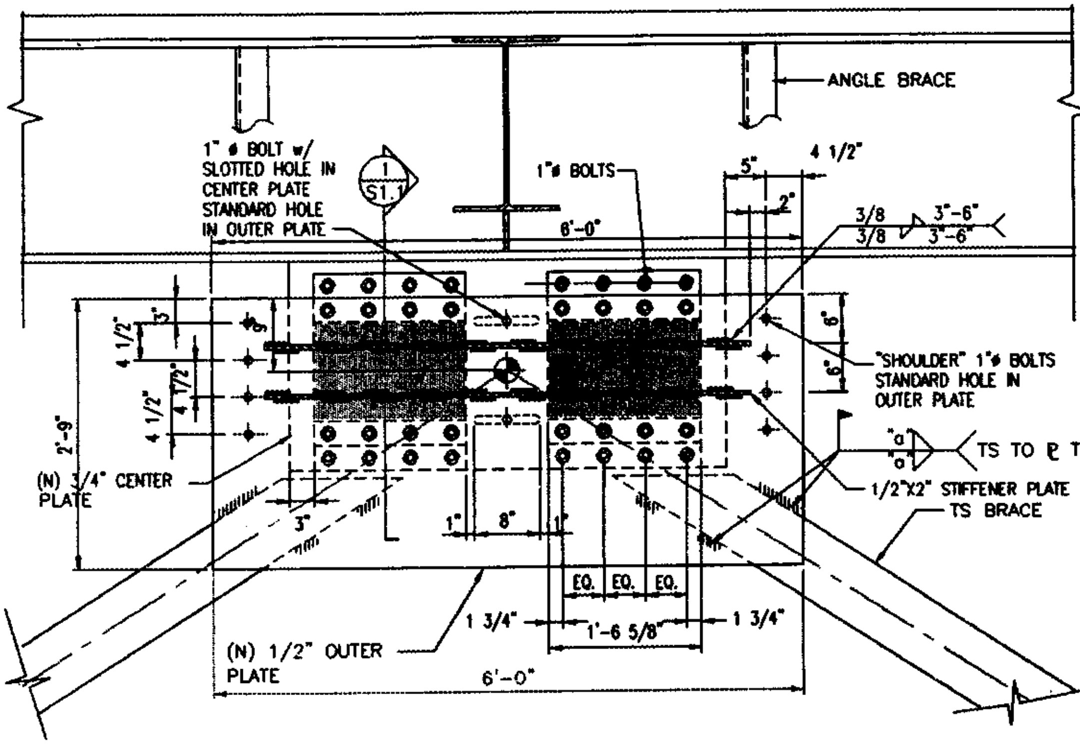


Fig. 5 Demand & capacity spectra: 3D pushover analysis for MCE demands



Damper Assembly Detail

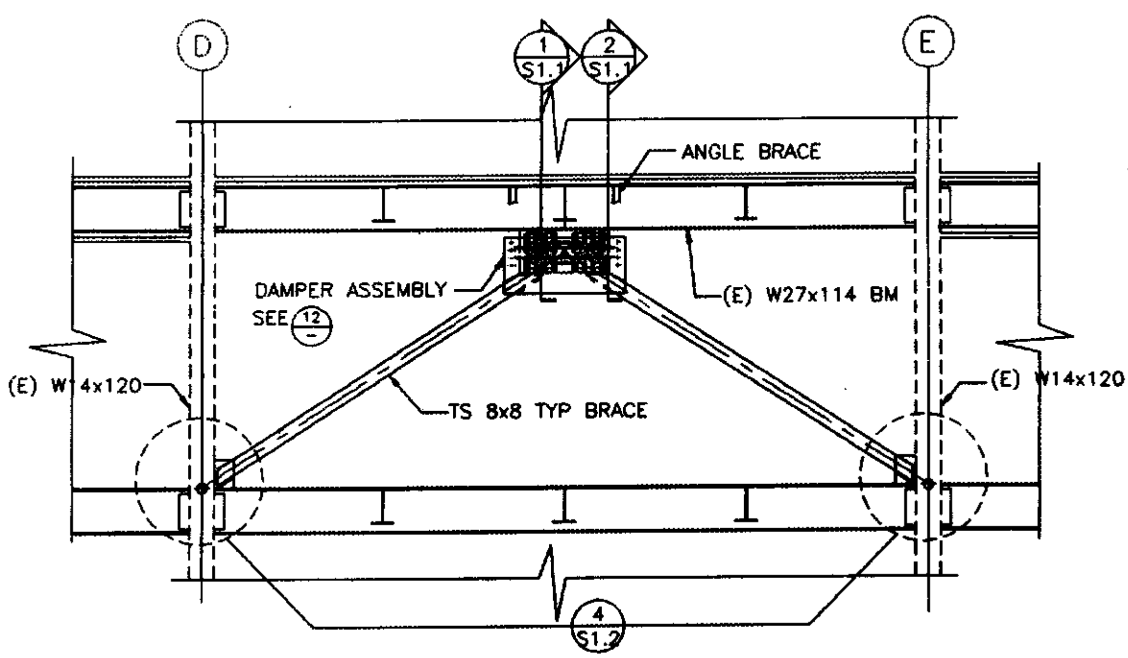
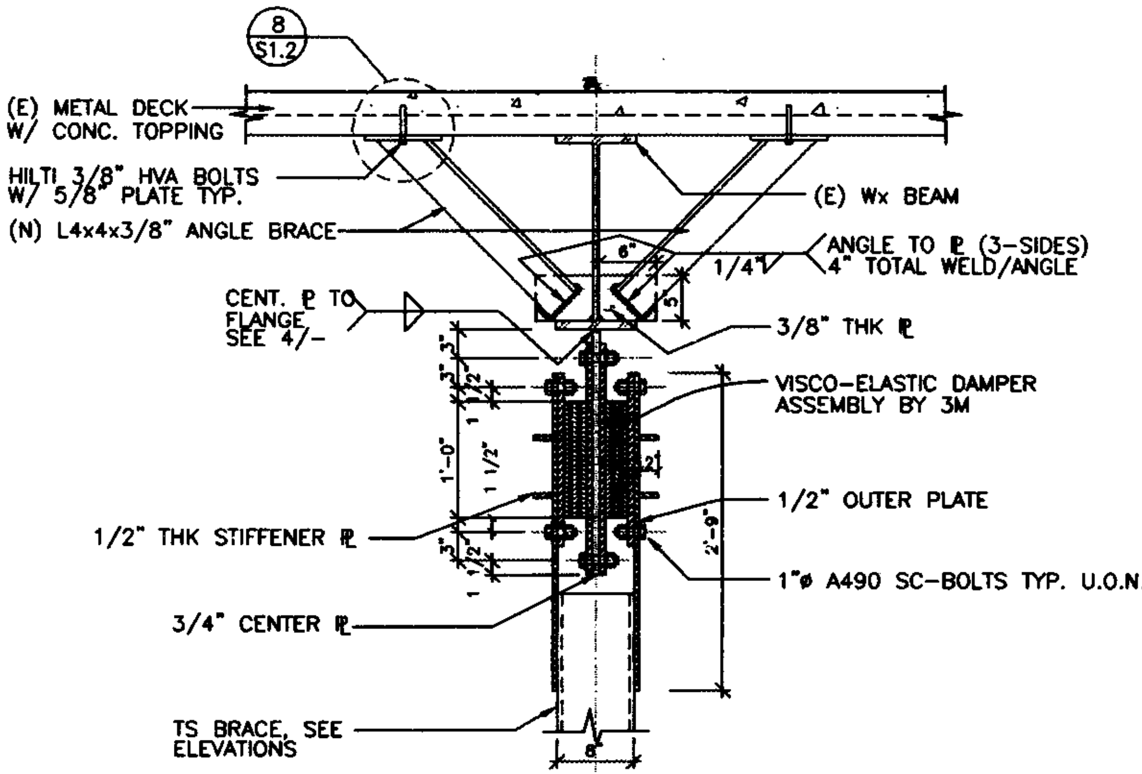


Fig. 6 Typical damper bay elevation



Damper Below Floor Beam

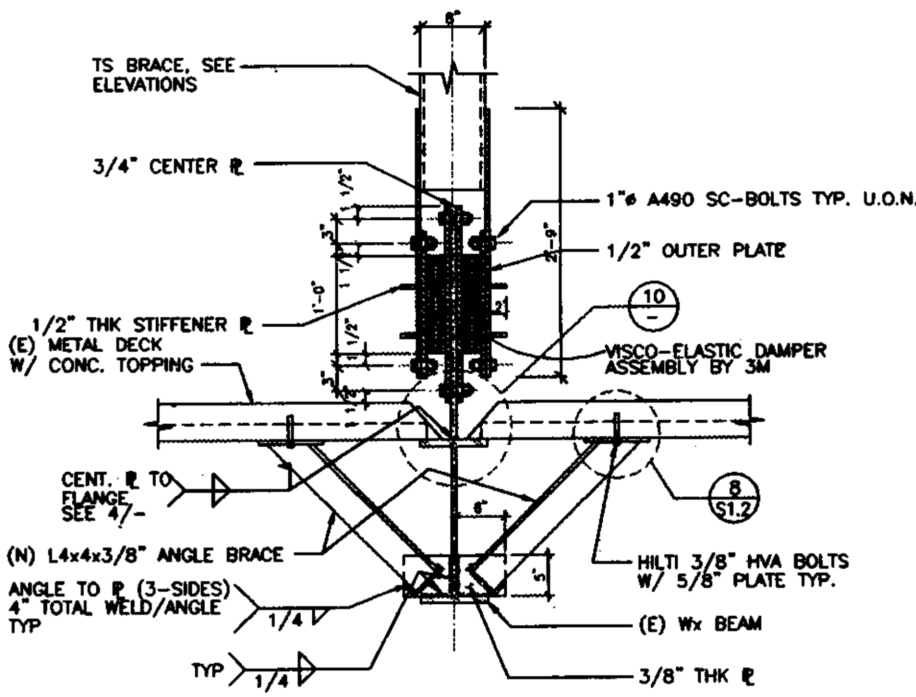


Fig. 7 Typical sections – damper above floor beam

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