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DESIGN, ANALYSIS AND TESTING OF A VISCOELASTICALLY DAMPED STEEL FRAME

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SUMMARY

This paper presents the design, analysis, and test results of a viscoelastically (VE-) damped steel frame. A design approach using high-performance criteria is presented for the VE-building. Nonlinear time-history analyses using a fractional derivative model for the VE-dampers were conducted with results indicating the specified design goals were achieved and the proposed design approach provided a practical method of sizing frame members and dampers. Following design and analysis, a unique methodology was developed to test the full-scale lower three-story portion of the 10-story prototype VE-frame. The test frame was subjected to combined large earthquake and gravity forces and tests were conducted in real-time due to the velocity sensitive VE-material properties. Tests included realistic sized frame members, connections, and damper brace assemblies that have not been modeled previously. Experimental results including: overall frame response, local member response, and damper-member interaction under various earthquakes are described. Test results provided detailed response data for overall frame and local member behaviors that were previously unavailable. Measured frame, member, and damper responses for the earthquake motions indicated significant energy dissipation is provided by the VE-dampers and that the steel members behaved elastically. Moments and shear forces in frame members were in-phase with frame displacement, while axial forces had significant out-of-phase components due to VE-damper forces. The phase lag between moment and axial force in the beams and columns results in peak forces occurring at different instances of time and the interaction between these forces can be predicted with the author's method.

INTRODUCTION

Viscoelastic dampers (VE-) for building structures continue to attract the interest of the earthquake engineering community due to the dramatic changes in dynamic structural response resulting from their additional stiffness and damping (Aiken and Kelly 1991, Chang *et al.* 1991, Kasai and Fu 1995). VE-dampers offer the potential for economical earthquake resistant structures, which can behave elastically and maintain small drifts even when subjected to a major earthquake, thereby protecting both structural and non-structural components. To validate VE-frame analysis and design methods, a full-scale three-story portion of a steel frame with VE-dampers has recently been tested under simulated seismic loading. This paper presents a design methodology for a multi-story VE-frame, results of time-history analysis, and full-scale and real-time experimental response of the VE-frame to simulated earthquake ground motions. The key response parameters are correlated among design, analysis, and experiments.

DESIGN

A prototype 10-story steel frame building was designed to provide high-performance seismic resistance including: the structure should remain elastic against the design based earthquake (DBE) (return period of about 500 years) at the reference temperature of 24 $^{\circ}$C, and the building should protect non-structural components for the DBE by limiting the drift to 0.0075 radians at 24 $^{\circ}$C. To economically achieve this level of seismic performance, the structure requires some type of supplemental damping device. For this study, VE-dampers were considered. The prototype structure was an ordinary office building located in San Francisco, California on very dense soil (NEHRP soil profile B/C: A_v=0.4, A_a=0.4, C_v =0.48, and C_a =0.40). The plan and elevation of the selected prototype VE-building are shown in Fig. 1. The total dead weight of the building was 40.65 MN (9139.2 kips). Building response was considered in the east-west direction only and all VE-frames were identical. Additional framing members support gravity loads only and do not contribute to lateral force resistance.

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Design Forces

Design of the VE-frame was conducted using a NEHRP (1994) based equivalent static force design procedure developed by Kasai and Fu (1995). The procedure uses elastic static analysis and permits: determination of dynamic properties of the VE-frame such as vibration period and damping ratio; estimation of high damping global responses; and estimation of local responses. The base shear spectrum from the NEHRP provisions (1994) was used for design. The design intent is to maintain elastic response of the structural members even against the DBE (i.e. R=1). For such a case, a conventional structure requires large base shear and thus large members and connections. The VE-frame uses VE-dampers to reduce the earthquake forces by providing high damping. To include the beneficial effects of higher damping, the yield base shear strength is reduced by a factor $D_\xi = 1.5/\sqrt{1+25\xi}$ (Kasai and Fu, 1995, Kasai *et al.* 1998) where ξ is the damping ratio. As the damping ratio increases, the DBE acceleration decreases, resulting in lower required base shear. Building displacements are also reduced with increasing damping ratio. A target damping ratio of 30% was selected for design of the prototype building to minimize the DBE base shear.

Building period for initial design was estimated using the NEHRP approximate formula for the fundamental period T_a (NEHRP 1994). The coefficient used for the VE-frame with stiff dampers was 0.03 (same as that specified for an eccentrically braced frame). The approximate building period is typically lower than the actual period, which is generally conservative for strength requirements. However, it tends to be unconservative for estimating lateral drift. For the prototype frame, T_a was 1.06 seconds which required $S_{pa}(T_a=1.06, \xi=30\%)=0.284$ g. To limit frame drift below 0.0075 rad., the actual building period T must be below 1.94 seconds for $\xi = 30\%$. Limits on actual building period were estimated by noting that when the fundamental mode shape of a multi-degree of freedom (MDOF) system is a straight-line, the maximum roof displacement is 3/2 times $S_d(T, \xi)$. Highly damped VE-structures tend to exhibit straight-line fundamental mode shapes (Kasai *et al.* 1993). Traditionally, building code specifications compute the required base shear as $S_{pa}M_{tot}$. For the MDOF VE-frame, which exhibits a straight-line fundamental mode shape, the theoretical base shear is $S_{pa}M_{tot}/1.33$. Using this theoretical base shear, the required yield strength was computed as 8670 kN (1949.2 kips) for the prototype frame. The total base shear was shared equally among the six VE-frames in the east-west direction (Fig. 1) and was distributed vertically to each story level according to the NEHRP formula.

Design of Frame Members

Dampers and frame members were sized for the design lateral forces considering their relative stiffnesses. For preliminary design, the ratio between the horizontal stiffness of the damper K'_d and frame K_f is considered for a single-degree-of-freedom (SDOF) system. For an assumed $\eta_d=1.32$ at the reference temperature of 24 °C, a loading frequency of 0.5 hz, and strain of 50%, a ξ of 33% is achieved when the damper to frame stiffness ratio $K'_d/K_f=3$ and brace to frame stiffness ratio $K_b/K_f>20$. As the damping ratio for a SDOF system is generally higher than a MDOF system, the SDOF damping ratio should be slightly higher than the required level of damping for the MDOF system. A stiff damper was selected because it attracts force and a stiff brace was also chosen because it permits the force attracted by the damper to be dissipated by enabling deformation to take place in the VE-material instead of in elastic deformation of the brace. By providing a stiff damper and a stiff brace relative to the frame stiffness without dampers, the damping ratio remains relatively stable even as the relative stiffnesses vary due to VE-material temperature changes.

The portal method, modified to consider effects of the viscous force created by the VE-dampers, was used to obtain the maximum member forces from the computed lateral loads and the required stiffness ratios. The method combines the force in-phase with the frame drift (elastic force) and that 90° out-of-phase (viscous force) to compute the maximum member forces (Kasai and Fu 1995, Fu and Kasai 1998). In addition to seismic forces, gravity forces were included using the AISC LRFD (1993) load combination. Live load on the floors was 2.4 kPa (50 psf) and live load reduction factors were used as permitted by ASCE 7-95. Beams and columns were designed using A572 grade 50 steel for combined axial force and moment using the AISC LRFD (1993) interaction equations. Due to viscous force contributions from the VE-dampers, maximum axial load and moment occur at different instances of time (Kasai and Fu 1995, Fu and Kasai 1998), however for conservative design, peak forces are assumed to occur simultaneously. This point will be discussed further, using the experimental results. Square tube sections corresponding to A500 Grade B (F_y=317 MPa (46 ksi)) were used for the braces, which attach the VE-dampers to the frame. For additional conservatism, the axial force in the brace was amplified by a factor of 1.5 to prevent possible brace buckling (Kasai and Popov 1986). Connections were detailed using A36 steel. Frame member sizes are shown in Table 1.

Fundamental Period and Damping Estimate

Elastic static analysis was performed for the frame without dampers to determine the shear stiffness of the frame K_f subjected to the design lateral forces. Beam-column and column base connections were assumed rigid, center-line frame dimensions were used for the model, and column axial deformations were suppressed to isolate the frame shear stiffness. Subsequent laboratory tests of the bare steel frame indicated the beam-column connections behaved as rigid connections (Kasai and Higgins 1997). The computed frame shear stiffness was ≈ 3 kN/mm (17.2 kips/in) in a typical lower story. Using the previously selected target stiffness ratios ($K'_d/K_f = 3$ and $K_b/K_f > 20$) the required damper stiffness and brace stiffness were determined. The stiffness of the combined VE-damper and brace assembly K'_a, termed 'added component', was computed according to Kasai and Fu (1995) assuming a loss factor $\eta_d = 1.32$ (for the required vibration period, as described earlier) and using K'_d and K_b at each story.

Following analysis of the unbraced frame, an analytical model of the VE-frame was developed. The added components were modeled with linear elastic truss elements with horizontal stiffness K'_a and the fundamental building period T was calculated using Rayleigh's method. The computed period was 2.0 seconds, which is larger than the $T_a = 1.06$ used in preliminary design and is also very close to the required period of 1.94 seconds for drift control at $\xi = 30\%$. Equivalent viscous damping for the VE-frame ξ_{tot} was calculated using an energy approach and static analysis (Kasai *et al.* 1993, Fu and Kasai 1998). The damping is made up of the inherent damping of the bare frame as well as that contributed by the VE-dampers. The unbraced frame damping ratio was selected as 2%, a typically assumed value. The computed damping ratio ξ_{tot} was 30.2%, which is very close to the 30% assumed during initial design.

VE-Damper Design

The VE-damper proportions were determined from structural deformations computed from static analysis of the vertically distributed design base shear. Thickness of the VE-layer h=16 mm (0.625 in.) was selected so as to limit the maximum shear strain γ to below 100%. An average γ of 0.7 times the maximum shear strain was considered as an average VE-material strain during a random earthquake event. For the selected damper thickness, reference temperature of 24 °C, VE-frame building period of 2.0 seconds, average strain of 70%, and storage modulus G'= 786 kPa (114 psi) determined from the manufacturer's data, the required damper area A was determined as $A = ((K'_d/\cos^2\theta)\, h)/G'$ where θ is the inclination angle of the damper. The required VE-material area was 805,210 mm^2 (1248 in^2) for each damper. The four faces of the steel tube permit a large amount of VE-material to be placed in a relatively small volume of space.

ANALYSIS

Nonlinear time-history analyses were performed on the prototype 10-story VE-frame using nine different earthquake ground motions (Table 2). The selected earthquake records reflect many different characteristics including duration, intensity, frequency content, and soil conditions. An additional criteria for selection required that the records produced structural responses which could be simulated in the laboratory. While the soil conditions varied for the different earthquakes, the highly damped response spectra have the same general trend as the design spectra near the natural frequency of the prototype structure. Acceleration and displacement response spectra for the nine records considering equivalent damping ratio of 30% are shown in Figs. 2 and 3, respectively. Three of the records (1.5xEl Centro, 1.5xHachinohe, and 2.824xTaft) are considered design level 2 earthquakes for Japanese seismic design practice (AIJ 1990).

Nonlinear dynamic time history analysis of the prototype VE-frame analytical model was carried out using PC-ANSR (Maison 1992). Nonlinear analyses were performed to assure any nonlinear behavior could be accurately captured due to VE-material temperature rise, large amplitude VE-material strain, and possible frame member yielding. Mass of the structure due to dead load only, as well as stiffness and damping of the complete 10-story structure were included in the analysis. Steel frame members were modeled with beam-column elements and the dampers were modeled using a special VE-damper element with an initial VE-material temperature of 24°C. Nonlinear VE-material behavior was incorporated using a fractional derivative stress-strain relationship (Kasai *et al.*, 1993). Parameters used in the constitutive rule were obtained using experimentally determined values for the material storage modulus and loss factor. The material constitutive rule is integrated in a step-by-step analysis procedure to determine the dynamic response of the VE-damper. At each time step, the amount of energy dissipated and the temperature rise are calculated using thermo-mechanics principles and heat transfer theory. Based on the temperature rise and satisfying the VE temperature-frequency equivalence property, parameters for the constitutive rule are updated at each time step. Continuous updating of the parameters results in a nonlinear constitutive rule. These features have been incorporated into a finite element, which can accurately simulate the nonlinear cyclic behavior of a VE-damper including temperature and excitation frequency effects.

Computed displacement envelopes indicated approximately a straight-line deflected shape for all earthquakes except Northridge-Castaic. Inter-story drift envelopes for each of the nine earthquakes, specified design drifts, and the NEHRP drift limits are shown in Fig. 4. The drifts were all less than the NEHRP 0.01 rad. specified drift limit and all but one (Northridge-Castaic) were below the required 0.0075 rad. design drift limit. In general, the building exhibited fairly uniform drift for all earthquakes except Northridge-Castaic. Story shear forces were computed by combining the horizontal component of the damper forces and column shear forces to find the maximum at each time-step. Maximum damper force and column shear forces do not occur at the same time and threfore simply adding the peak member forces generally results in an over-estimation of the true story shear. Maximum cumulative story shears for each of the nine earthquakes and the design values are shown in Fig. 5. As seen in this figure, all but one of the base shears (Northridge-Castaic) were below the design limit. By comparing computed forces in the frame members with their corresponding yield surfaces, it was determined that the frame members remained elastic against all nine earthquakes.

The nine earthquakes used for analysis were all less than the NEHRP DBE at the 2 second period of the prototype VE-frame (Figs. 2 and 3). Several of the earthquakes would more closely match the specified NEHRP design level earthquake if they were scaled by a factor of approximately 1.3. Peak displacements of the VE-frame subjected to the earthquakes scaled by 1.3 can be estimated from Fig. 4 assuming linear behavior of the VE-system. Estimated building drifts for the scaled earthquakes would also be less than or approximately equal to the 0.0075 rad. design drift limit. However, for some of the scaled earthquakes, the story shears may exceed the design limit (estimated from Fig. 5). It is important to note that linear scaling is appropriate as the frame members remained elastic and peak VE-material strain amplitudes did not result in significant nonlinear VE-material behavior. This was verified by subsequent nonlinear time-history analysis for the scaled earthquakes. However, if the VE-dampers are not well proportioned or if earthquake induced motions are very large, temperature rise and large strain amplitudes can produce significant nonlinear response and linear scaling would not be applicable.

Comparison of analysis results with the design intent indicates the design approach is a practical means of proportioning a VE-structure, which can closely correspond to the strength and drift requirements of the DBE. The design story shear is conservative due to the use of T_a instead of the actual building period T (Eq. 8). However, the design base shear may be unconservative by using steady state elliptical response assumption, which may not hold for a pulse type earthquake and large VE-damping. Additionally, assuming only first mode response through use of a 1.33 reduction factor and ignoring higher mode response on the design base shear is unconservative. This was particularly evident for Castaic, a Northridge earthquake, which exhibited higher mode response as shown by the nonuniform drift in Fig. 4. Due to these uncertainties, it would appear reasonable to use T_a instead of T for determining the design base shear.

TESTING

A unique methodology was developed to test the full-scale, lower 3-story portion, of the prototype 10-story VE-frame (Fig. 1). The key feature of the experimental program was the use of the displacement response from time-history analysis of the 10-story prototype VE-frame as the input motion to the test structure. Analytically predicted frame response at the third story level was imposed to the laboratory test frame. Experiments were performed dynamically, in real-time, due to the velocity sensitive properties of the VE-material. The test set-up is shown schematically in Fig. 6b and the three-story portion of the prototype building tested in the laboratory is shown in Fig. 6a. The experiment incorporated over 90% of the tall reaction wall in the ATLSS Laboratory at Lehigh University, one of the largest in the North America. Cumulative axial forces from gravity loads from the complete 10-story structure were applied to the column tops of the frame test specimen by 76 mm (3 in.) diameter wire ropes attached to two actuators (Fig. 6a and 6b). Vertical forces up to 4.45 MN (1000 kips) develop in each column due to gravity and lateral load effects during some tests. Large earthquake loads accumulated from the 10-story structure were applied at the third story level through two additional hydraulic actuators. Second order or P-Δ effects were simulated with a fifth actuator, which ensures that the simulated gravity force remains vertical despite tilting of the frame (Fig. 6c). During each test, data were acquired from 196 sensors including: strain gages, position sensors, load cells, and thermocouples. Initial damper temperatures were controlled by placing insulated enclosures around each damper. Forced air, either heated or cooled, was blown into the enclosures until the desired initial VE-material temperature was achieved.

Example measured overall force-deformation response for the frame is shown in Fig. 7 for the Taft earthquake. Analytically predicted overall force-deformation response for the laboratory imposed motion of Taft is shown in Fig. 8. As shown in these figures, the measured and predicted responses are similar and the methodology reasonably reflects the seismic response of the prototype building. Overall responses indicated the dampers provided significant energy dissipation for all earthquakes. The energy dissipated by the dampers during the earthquakes results in temperature rise within the VE-material. The largest observed temperature rise (3.4 °C) was recorded during the Mexico City: Central de Abastos-Frigorifico earthquake, which was 60 seconds in duration and contained many significant excursions. Temperature rise in the dampers can be predicted according to Kasai *et al.*'s method (1993) as the total area or dissipated energy density obtained from the stress-strain loops of the damper divided by the product of specific heat and density of the VE-material (=1958.1 kPa/°C (0.284 ksi/°C) for the given VE-material).

In addition to overall frame response, data was collected which enabled assessment of local member responses. Data and observations indicated that the frame members remained elastic during all tests with the exception of minor local yielding near connections. Measured column and beam moments (and shear forces) tended to be in-phase with frame displacement, while axial forces tended to be in-phase with the VE-damper forces. As an example, the normalized measured time-history response for moment and axial force in the second story beam are shown in Fig. 9. There is a phase-lag between the axial force and moment due to the viscous force component transferred to the beam by the VE-dampers. Maximum axial force and bending moment do not occur at the same instant of time due to the phase-lag and thus members exhibit moment-axial force interaction as shown in Fig. 10. The out-of-phase member forces generated by the dampers can be predicted by the author's method (Kasai and Fu 1995) and although the peak moment and axial forces do not occur at the same time, it is conservative to design these members assuming they occur simultaneously.

CONCLUSIONS

A prototype VE-frame has been designed, analyzed and tested. Design was performed using an equivalent lateral force procedure with specified high-performance design criteria which included elastic performance and small drift (<0.0075 rad.) at the reference temperature of 24°C under the California type DBE. Subsequent analysis and testing indicated the design methodology provides a practical means of designing VE-damped steel frames which can closely correspond to the strength and drift requirements of the DBE. However, for some near field pulse type earthquakes or when significant higher mode effects are present, the methodology may not be conservative. Following design of the prototype frame, a unique methodology was developed to enable testing of the full-scale lower three-story portion of the 10-story prototype VE-damped frame subjected to significant seismic and gravity forces. Measured overall frame, member, and damper responses for the earthquake motions indicated significant energy dissipation is provided by the VE-dampers and that the steel members behaved elastically. Analytically predicted response and measured experimental response were well correlated indicating the testing methodology is reasonable. Measured moments and shear forces in frame members were in-phase with frame displacement, while the axial forces had significant out-of-phase components due to VE-damper forces. The phase lag between moment and axial force in the beams and columns results in peak forces occurring at different instances of time and an interaction between the forces. A proposed simplification would conservatively design members assuming the peaks occur simultaneously.

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Table 1 – Member sizes for prototype VE-frame.

Story Level	Beam Size	Column Size	Brace Size
1	W18x50	W14x159	TS8x8x1/2
2	W18x40	W14x159	TS8x8x1/2
3	W18x40	W14x159	TS8x8x1/2
4	W18x40	W14x145	TS8x8x3/8
5	W18x35	W14x145	TS8x8x3/8
6	W18x35	W14x145	TS8x8x3/8
7	W16x31	W14x82	TS8x8x1/4
8	W16x31	W14x82	TS8x8x1/4
9	W10x19	W14x48	TS8x8x3/16
10	W10x19	W14x48	TS8x8x3/16

Table 2 – Earthquake records used for analysis and testing of VE-frame. –(Seed *et al.* 1974) +(Miranda 1993)

Earthquake Record	Year	Location	Soil Type	PGA (g)	Duration(sec)
1.5xEl Centro (N-S)	1940	El Centro, CA	Alluvium+	0.52	30
1.5x Tokachi-Oki (N-S)	1968	Hachinohe, Japan	Deep Cohesionless-	0.34	30
2.824xTaft (S69E)	1952	Kern County, CA	Alluvium+	0.51	30
Oakland (CSMIP 58224 Ch.6)	1989	Loma Prieta, CA	Alluvium+	0.25	20
Treasure Island (90)	1989	Loma Prieta, CA	Fill+	0.16	15
Castaic Old Ridge Route (360)	1994	Northridge, CA	Unknown	0.51	20
Santa Monica City Hall (90)	1994	Northridge, CA	Unknown	0.88	15
Cent. Abastos-Frigorifio (E-W)	1985	Mexico City	Soft Clay+	0.10	60
Cent. Abastos-Oficina (N-S)	1985	Mexico City	Soft Clay+	0.07	120

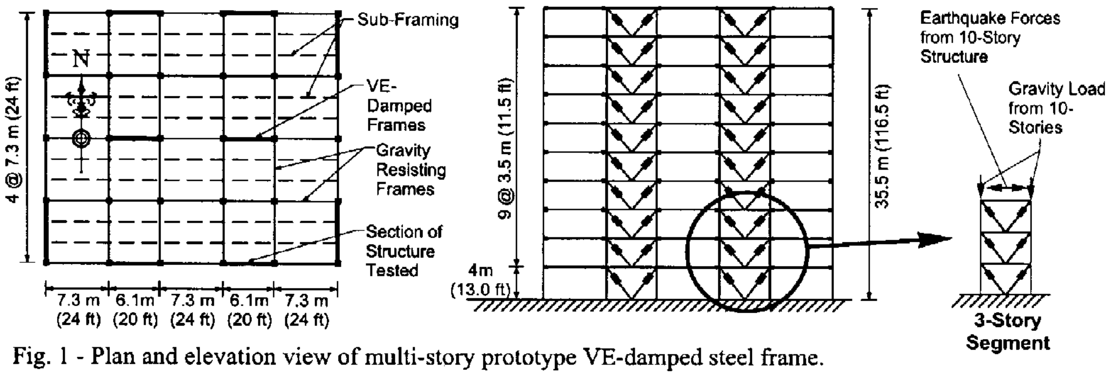

Fig. 1 - Plan and elevation view of multi-story prototype VE-damped steel frame.

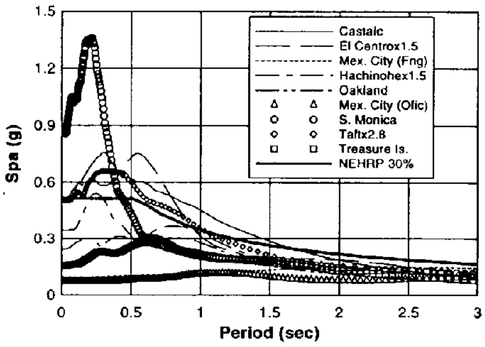

Fig. 2 - Spectral accelerations for selected earthquakes ξ=30%.

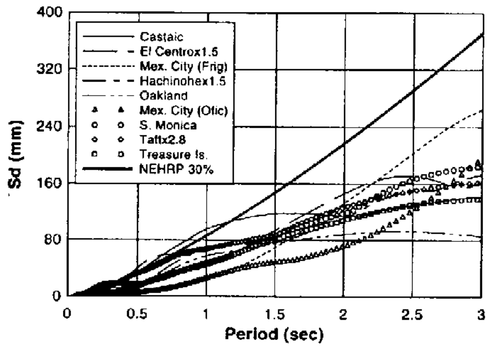

Fig. 3 - Spectral displacements for selected earthquakes ξ=30%

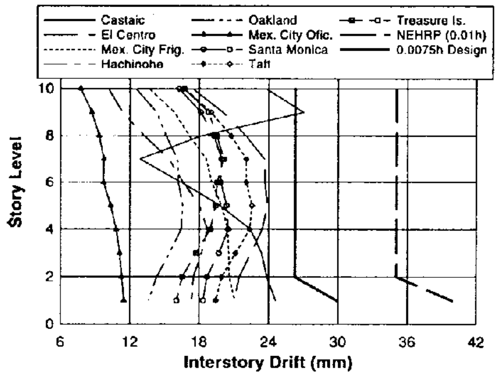

Fig. 4 - Drift envelopes from time-history analysis.

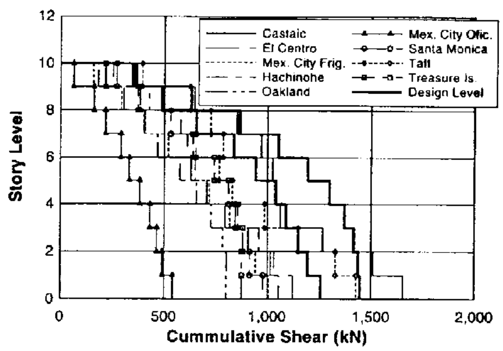

Fig. 5 - Shear envelopes from time-history analysis.

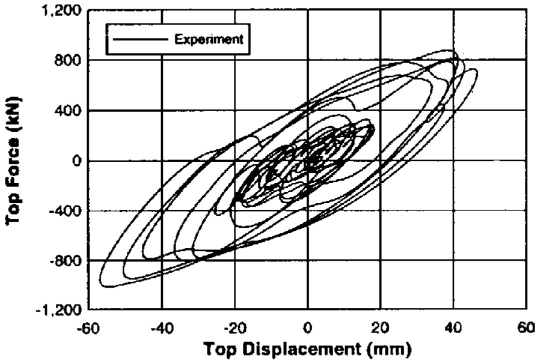

Fig. 7 - Experimentally measured overall VE-frame response.

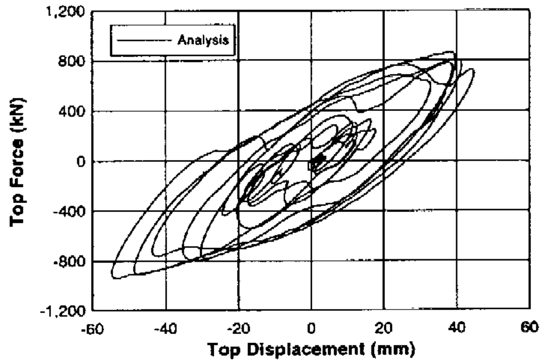

Fig. 8 - Analytically predicted overall VE-frame response.

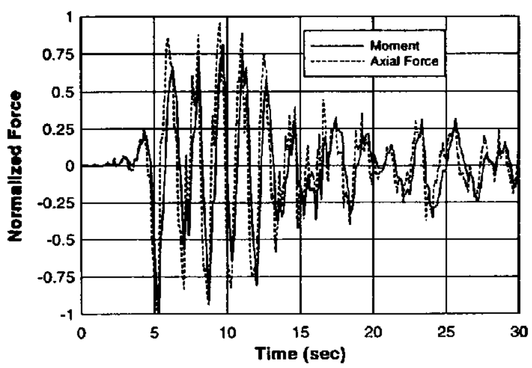

Fig. 9 - Normalized experimentally measured moment and axial force in second story beam.

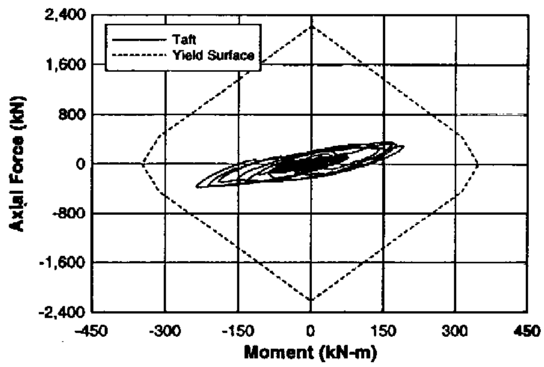

Fig. 10 - Moment-axial force interaction for second story beam.