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A STATIC EQUIVALENT FORCE DESIGN METHOD FOR A DAMAGE-FREE BUILDING WITH VISCOELASTIC DAMPERS

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ABSTRACT

This paper presents a recently developed step-by-step design method for viscoelastically (VE-) damped frames, by satisfying specific displacement and/or stress limit imposed by the building owner or designers. The method uses a static equivalent force to estimate the highly dynamical force associated with VE-damper response. This simplified method is consistent with current practice for design of conventional frames, and still considers complex interactions between members and dampers. A relatively stiff 10-story VE-frame is designed by using the proposed design method. It is designed to be "damage-free" even against a major earthquake, using beams and columns of small size. For performance comparison purpose, a relatively flexible VE-frame as well as conventional special moment resisting frame (SMRF) are also designed. The three frames are analyzed dynamically by using various earthquakes. The result indicates a variety of significant advantages of both the stiff VE-frame and flexible VE-frame over the SMRF.

KEYWORDS

Viscoelastic Dampers; Seismic Design; Equivalent Static Force; Steel Frame

INTRODUCTION

Use of viscoelastic (VE-) dampers for seismic damage mitigation of structures has received substantial interest lately. The VE-dampers substantially reduce building drifts and member forces by adding both damping and stiffness to the structure. Although past studies demonstrated such effectiveness of VE-damper, there is a considerable lack of study regarding design methodology of a VE-frame. Pursuant to this, the writers have been conducting extensive analysis, experiment, and design studies in order to provide a comprehensive guideline for VE-damper applications. This paper proposes a step-by-step method to analyze and design frame members and VE-dampers by satisfying the displacement and stress limits imposed by building owner or designer. The method uses a static equivalent force and yet simulates the complex dynamic interactions among member forces and deformations caused primarily by the time lag of the VE-damper response. It also provides control of temperature sensitivity of a VE-frame. The method is made to be consistent with the current practice of conventional frame design. A relatively stiff 10-story VE-frame is designed by using the proposed method. It is designed to remain "damage-free" even against a major earthquake. For performance comparison, a relatively flexible 10-story VE-frame as well as conventional special moment resisting frame (SMRF) are also designed. These frames are analyzed dynamically by using large earthquakes. The results indicate superior seismic performance and economy of the VE-frames compared with the SMRF.

EQUIVALENT STATIC FORCE DESIGN CONCEPT FOR VE-FRAME

NEHRP Seismic Design Force. Typical building seismic design for conventional frame types has been based on the concept of equivalent lateral static force. The design base shear spectrum defined by the U.S. National Earthquake Hazard Reduction Program (NEHRP 1991) is as follows:

$$V_{DBE} = (1.2\ A_v\ S\ /\ T^{2/3})\ W \leq\ 2.5\ A_a\ W \tag{1}$$

where V_{DBE} = base shear force of elastic building subjected to design basis earthquake (DBE), W = building total weight, T = fundamental vibration period of building, and S = soil factor. A_a and A_v = coefficients for effective peak acceleration (EPA) and effective peak velocity-related acceleration (EPV) of the DBE, respectively with a return period of 475 years (NEHRP 1991). The NEHRP required yield base shear strength V_{YLD} is:

$$V_{YLD} = V_{DBE} / R \tag{2}$$

where R = strength reduction factor specified for conventional frame types. For instance, R = 8 for a special moment resisting frame (SMRF). Note, however, that design of typical SMRF in U.S. is often governed by the requirement for stiffness to control the story drifts rather than the strength aforementioned. Consequently, a typical SMRF tends to have the yield overstrength of about 1.5 to 3 times the V_{YLD}.

High Damping Design. The writers propose damage-free design that ensures elastic response, rather than inelastic response, of a building against the DBE. A conventional frame to remain elastic under the DBE would require very large V_{YLD}, since R must be set to 1.0, and it requires significantly large members and connections, making the design economically unfeasible. In contrast, the proposed economical damage-free design takes advantage of supplemental damping devices and aggressively reduces the overall building responses. Here, we define a parameter D_ξ to indicate the damping effect, and corresponding required V_{YLD} becomes:

$$V_{YLD} = D_\xi\ V_{DBE}\ , \qquad D_\xi = 1.5\ /\ (\ 1 + 25\ \xi\ )^{1/2} \tag{3}$$

where ξ = damping ratio of the building. The writers obtained somewhat conservative D_ξ expression by modifying an equation in AIJ (1993) as well as the table given in the new version of NEHRP (1994). Note that the original DBE spectrum (Eq. 1) is given for ξ = 5%, for which D_ξ = 1.0 (Eq. 3). It is also possible to combine Eq. 2 and Eq. 3, allowing significant yielding of the supplementary damped building under the DBE, but such design is not a scope of the present paper.

SIMPLIFIED STIFFNESS AND DAMPING ANALYSIS OF LINEAR VE-SYSTEM

Combined VE-Damper and Brace. In general, the VE-damper is attached to a steel brace (Fig. 1), and it is important to understand interactions of these elements. From know on, the combined damper and brace will be called "*added component*". The stiffness and loss factor of the added component is (Kasai and Fu 1995):

$$K'_a = K_b K'_d\ /\ (K_b/\Gamma + K'_d), \quad \eta_a = \eta_d\ /\ \{1 + (1 + \eta_d^2)\ K'_d/K_b\}, \qquad \Gamma = 1 + \eta_d^2\ /\ (\ 1 + K_b/K'_d) \tag{4}$$

where K_a = added component stiffness, and K_b = brace stiffness. K'_d and η_d are the stiffness and loss factor of VE-damper, respectively (Kasai *et al.* 1993). All these stiffnesses are defined in horizontal direction. The stiffness K'_{story}, damping ratio ξ_{story}, and loss factor η_{story} of a typical story model are (Kasai and Fu 1995):

$$K'_{story} = K_f + K'_a\ , \qquad \xi_{story} = \eta_{story}/2 = \eta_a\ /2/\ (1 + K_f/K'_a) \tag{5}$$

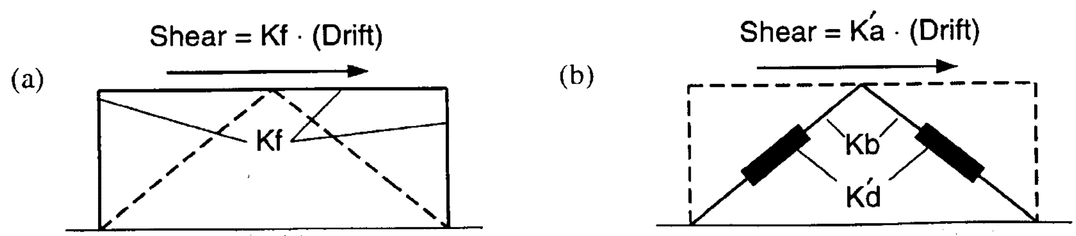

Figure 1. Typical VE-Frame Story (a) Frame Stiffness, and (b) Brace and Damper Stiffness

Multistory VE-Frame. A multistory VE-frame involves more complicated interactions of stiffness and damping of structural components than discussed above. Hereby a so-called *Modified Modal Strain Energy (MSE) Method* is presented. The VE-frame is modeled by attaching to the original frame the equivalent braces having stiffness of the added components K'_a. The expressions for the overall stiffness K'_{tot} and damping ratio ξ_{tot} of the VE-frame become:

$$\underline{K}'_{tot} \cong \underline{K}_f + \underline{K}'_a \ , \quad \xi_{tot} \cong \frac{\pi \underline{\phi}^T \eta_a \underline{K}'_a \underline{\phi}}{4\pi \underline{\phi}^T \ \underline{K}_{tot} \underline{\phi}/2} = \frac{\underline{\phi}^T \eta_a \underline{K}'_a \underline{\phi}}{2 \ \underline{\phi}^T \ \underline{K}_{tot} \underline{\phi}} \cong \frac{\Sigma \eta_a \cdot F_a \cdot u_a}{2 \ \Sigma F \cdot u} \tag{6}$$

Eq. 6 is an MDOF version of Eq. 4, and uses matrix form. $\underline{\phi}$ is the mode shape. The current study (Kasai and Fu 1994) indicates reasonableness of using 1st mode frequency and mode shape, ω_1 and $\underline{\phi}_1$, respectively. The last expression in Eq. 6 alternatively uses static elastic analysis to estimate ξ. The F_a and u_a are respectively the force and deformation of the equivalent brace at each story, when frame is subjected to the static story lateral force F, derived from Eq. 3, and develops displacement u at each story. This static analysis approach is attractive, since it is compatible with the current code design method using static forces.

Local Peak Responses of VE-Frame. The obtained member peak elastic force Q′ of a dynamically excited elastic structure can be obtained approximately by using static analysis. Note, however, that peak member force Q is the peak of combined elastic force and viscous force generated by the damper. The Q′ and Q occur at different instances. The Q can be predicted by considering idealized steady state excitation and corresponding equilibrium for the frame (Fig. 2) (Kasai and Fu 1995). i.e.:

$$Q = Q' \ (1 + \eta^2)^{1/2} \tag{7}$$

Where the η value depends on the type of force Q. For bending moments of beam and column, $\eta = 0$. For axial forces of brace and beam $\eta = \eta_a$. For axial force of column $\eta = \eta_{p.col} = \eta_{story}\{1 - (H/L)(V'_{col}/P'_{col}\}$, where L and H = span length and height of the story. V'_{col} and P'_{col} are the static column shear and axial force, respectively (Fig. 2).

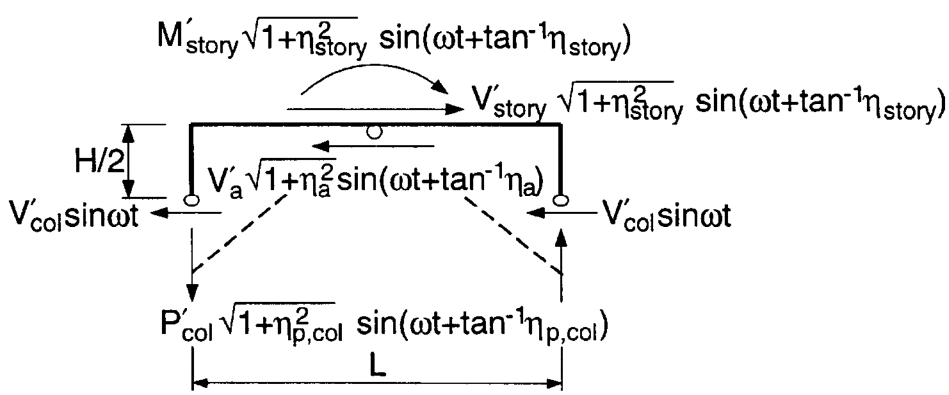

Figure 2. Free Body Diagram of a Typical VE-Frame Story at Steady State

STIFF VE-FRAME EXAMPLE

10-Story Building and Design Criteria. A 10-story office building located in California is to be designed. The building is 90 feet x 90 feet in plan, and has three bays at each side (Fig. 3). The middle bay is equipped with VE-dampers and is a steel MRF, whereas other bays have virtually gravity framing with simple connections at the beam-column junctions. Accordingly, seismic force tributary area of a VE-frame bay is 4,050 ft^2, large enough to qualify as an efficient lateral force resisting frame (Fig. 3). Dead loads of roof and typical floor are 67 psf and 75 psf, respectively, and curtain wall dead load is 15 psf. Total dead weight of the building is 6,620 kips. Live loads for the roof and floor are 20 psf and 50 psf, respectively. The steel used is A572 Grade 50 (i.e., yield stress = 50 ksi) for wide flange beams and columns, and is A500 Grade B (i.e., yield stress = 46 ksi) for square cross section tube braces. The writers consider the following design criteria for a damage-free 10-story building:

(1) The building will have no structural damage (no yielding or buckling of structural members) against the DBE having EPA = EPV = 0.4g, under the temperature between 16°C to 32°C.

(2) The building will protect non-structural component against the DBE by limiting the story drift angle to about 0.005 radian under the temperature of 24°C, requiring a relatively stiff VE-frame.

A step-by-step design using the static force is explained below. The procedure treats in a simple manner the complex effects of frequency and temperature sensitivities of VE-material. The example first assumes a standard room temperature of 24°C, and then refers to the cases of 16°C and 32°C.

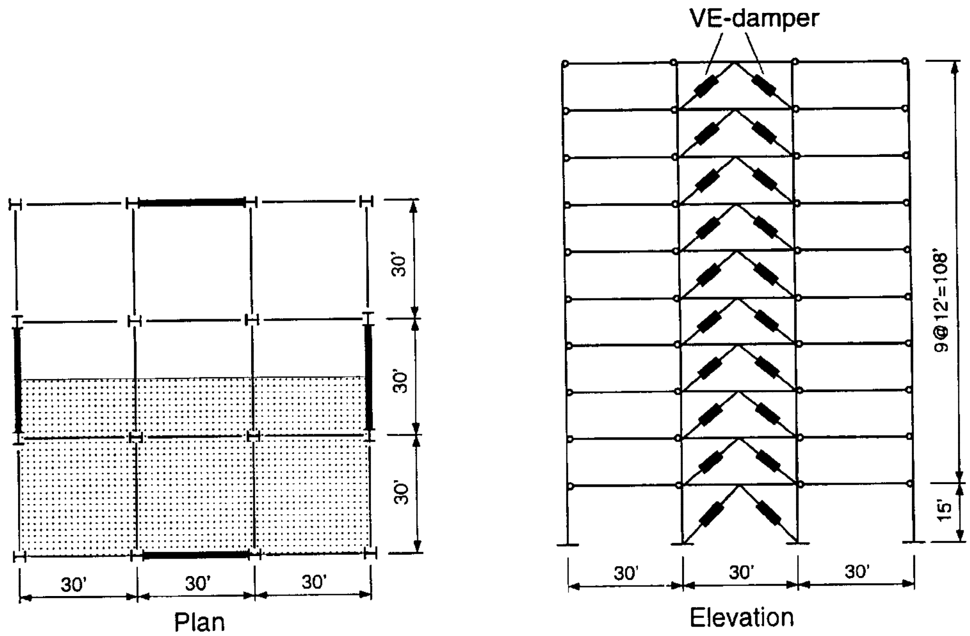

Figure 3. Frame Configuration

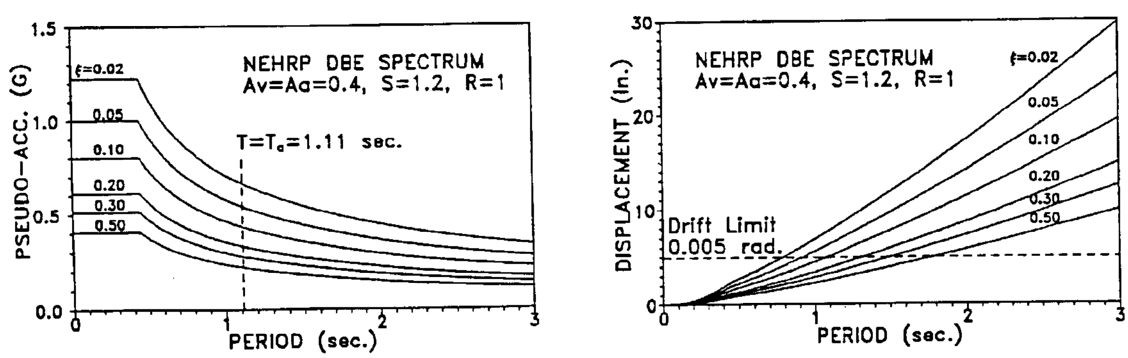

Figure 4. Design Basis Earthquake Spectra Considering Various Damping

<u>Vibration Period, Damping, and Base Shear.</u> The first step is to determine pseudo-acceleration spectrum S_{pa}. The plots of S_{pa} under various damping ratios are shown in Fig. 4 by using the following formula (see also Eq. 3):

$$S_{pa}(T,\xi) = D_\xi \; S_{pa}(T,5\%) \tag{8}$$

and also, S_d (= S_{pa}/ω^2) are shown in Fig. 4. Various combinations of the period T and damping ξ can be considered for design. The writers will initially restrict T, by adopting the approximate period formula (NEHRP 1991) for an eccentrically braced steel frame, i.e., $T_a = 0.03\ h_n^{3/4}$, where T_a = approximate and empirical code period, and h_n (feet) = frame height. For the present 10-story VE-frame, this leads to T_a = 1.11 sec. Note that T_a-value is typically smaller than the actual building period T. Thus, required base shear based on T_a is generally on a conservative side. T_a = 1.11 sec. gives the strength requirement S_{pa} = 0.54g, 0.43g, 0.33g, and 0.28g for the damping ratio ξ = 5%, 10%, 20%, and 30%, respectively. Also, criterion (2) is applied by using S_d (Fig. 4): The S_d reflects the maximum displacement of a multistory building at its 2/3-height when building's deformed shape is a straight line, which actually is a trend of a typical VE-frame (Kasai *et al.* 1994). Accordingly, the criterion (2) and Fig. 4 give the upper bound for the actual T, which are 0.91, 1.07, 1.31, and 1.49 sec. for ξ of 5%, 10%, 20%, and 30%, respectively. From these, the use of high damping appears to be a good choice, since it allows the use of relatively long period structure that will enjoy less inertia force, thereby having less stress demand. Based on these, target ξ = 30% and actual T < 1.5 sec. under 24°C are considered for further design calculation.

Member Force Calculation. For a system like a VE-frame that tends to develop a straight deformed shape. approximately $S_{pa}M_{TOT}/1.33$ would be the theoretical base shear by considering participation factor for the MDOF system. Instead, $S_{pa}M_{TOT}$ is used by adopting the conservatism of the traditional static design concept. Accordingly, required yield strength V_{YLD} under criteria (1) is expressed as follows:

$$V_{YLD} = M_{TOT} \cdot S_{pa}(T{=}T_a{=}1.11 \text{ sec.}, \xi{=}30\%) \tag{9}$$

As indicated by the writers (Kasai et al. 1994), K'_d/K_f and K_b/K_f are a key for controlling the temperature sensitivity of VE-frame as well as forces in the beams and columns. Before knowing the member sizes, the writers consider target values of $K'_d/K_f = 3$ for each story level, as well as the average (throughout the building height) of $K_b/K_f \geq 20$. According to Eq. 5, these ratios lead to $\xi \approx 37\%$. However, as will be mentioned, the ξ of a multistory VE-frame is typically lower than that of SDOF system. This is because VE-frame does not develop only shear drifts considered in the SDOF calculation but also develops bending drifts that do not necessarily promote energy dissipation by the VE-damper. Based on the aspect ratio of the VE-frame and past experience, it was felt that the actual ξ of the VE-frame would be somewhat lower and yet would be close to the target damping of 30%. Using these parameters as well as the static design force determined in Eq. 3, the maximum member forces are obtained considering the effect of viscous forces (Eq. 7). Further, gravitational load effect was superposed to this seismic load effect.

Sizing of Beams, Columns, and Braces. Based on the calculations in the previous section, the beams and columns are selected by considering the combined effect of moment and axial forces and using the AISC LRFD equations (1994). Note that the peaks of the axial force and bending moment of the member occur at different instances. But they are assumed to occur simultaneously in using Eq. 7, resulting in some conservatism. For the brace design, only axial force is considered. Axial force obtained from the static analysis is amplified 1.5 times in order to prevent hazardous brace buckling, and the effective length factor of 0.8 is used. The selected sizes of these members are indicated in Table 1.

Table 1. Comparison of Member Sizes and Steel Weights among Three Frames

	VE-frame (0.5% drift limit)			Flexible VE-frame (1.0% drift limit			SMRF	
Story level	Beam section	Column section	Brace section	Beam section	Column section	Brace section	Beam section	Column section
10	W18x35	W14x61	ST 6x5/16	W18x35	W14x53	ST 6x3/16	W30x116	W14x211
9	W24x62	W14x61	ST 7x1/2	W21x50	W14x53	ST 8x1/4	W36x135	W14x211
8	W27x84	W14x132	ST 8x5/8	W24x62	W14x90	ST 8x3/8	W36x170	W14x257
7	W30x99	W14x132	ST10x1/2	W24x76	W14x90	ST 8x1/2	W36x194	W14x257
6	W30x108	W14x257	ST10x5/8	W24x76	W14x132	ST 8x1/2	W36x194	W14x398
5	W33x118	W14x257	ST14x1/2	W24x76	W14x132	ST 8x5/8	W36x194	W14x398
4	W33x130	W14x257	ST14x1/2	W24x84	W14x132	ST 8x5/8	W36x230	W14x500
3	W33x130	W14x426	ST14x1/2	W24x84	W14x193	ST10x1/2	W36x210	W14x500
2	W33x130	W14x426	ST14x1/2	W24x84	W14x193	ST10x1/2	W36x210	W14x605
1	W33x141	W14x426	ST16x1/2	W24x94	W14x193	ST10x1/2	W36x210	W14x605
	Total Beam and Column Weight = 92 kips			Total Beam and Column Weight = 53 kips			Total Beam and Column Weight = 154 kips	

Static Elastic Analysis for Period and Damping Estimates. Using the member sizes obtained in the previous section, elastic static analysis is conducted for the frame without dampers. The purpose is to obtain frame shear stiffness K_f at each story level, thus, the column axial deformation was suppressed in this analysis only. Then, required damper stiffness K'_d at each story level is estimated based on the obtained K_f and target ratio of $K'_d/K_f = 3$. Using thus-determined K'_d, the K'_a is estimated (Eq. 4) by assuming $\eta_d = 1.4$. Then, a computer model of the VE-frame is formulated. The added component (i.e., combined damper and brace) are modeled as an elastic strut brace having the stiffness of K'_a. The building period T is calculated using static elastic deflection due to the design lateral force. Calculated period T = 1.48 sec. is larger than T_a = 1.11 sec. used in the preliminary design. Also, T = 1.48 sec. just satisfies the upper bound limit defined in the above section for the drift control under the assumption of ξ= 30%. Calculated ξ using the modified MSE method (Eq. 6) is 27%. By adding 2% initial damping assumed for the steel frame, ξ = 29% that is close to the target ξ = 30%.

Sizing of VE-Dampers. Deformation of the added component calculated in the previous section is used as a conservative estimate for the damper deformation. Then, thickness of VE-layer is determined such that maximum shear strain is within 100%. For K'_d used in the previous section, the shear area of the VE-material is determined based on manufacture's data for shear modules at 0.5 times the maximum shear strain.

Static Elastic Analysis for Various Temperatures. Using the above determined members and dampers, static elastic analysis is repeated for the temperature of 16°C and 32°C. This is done iteratively by using Eq. 6, since the damper stiffness K'_d and vibration period are related. Obtaining convergence after a few iterations, the member forces as well as damper deformations are checked. The magnitude of the static load used is based on the calculated T, ξ, and D_ξ (Eq. 3) under the temperature used.

NONLINEAR DYNAMIC ANALYSES AND COMPARISON

Nonlinear Analyses with Major Earthquakes. The designed stiff VE-frame is analyzed by using the writers' nonlinear VE-damper element (Kasai *et al.* 1993) and beam-column elasto-plastic element. Two major earthquakes are used. They are: artificial earthquake (0.4g) whose spectrum characteristics are compatible with the DBE spectrum of NEHRP, and 1.5 times El Centro Earthquake (0.52g). For each earthquake, 3 temperatures of 16°C, 24°C, and 32°C are used. For the members of the frame, Rayleigh damping equivalent to 2% damping ratio is used. Fig. 5 shows the peak displacements and drifts of the VE-frame. The stiff VE-frame develops average drifts of 0.005 rad. or less under the temperature of 16°C and 24°C, and the increase of the drifts at 32° is marginal. Fig. 6 shows moment-axial force interaction of the beam and column that are located at 5th story level and 1st level, respectively. The member forces are well inside the yield surface. In general, the members will have higher axial forces and lower moments under the lower temperature, and vice versa. This is because the lower temperature makes VE-damper stiff and the VE-frame behaves like a concentrically braced frame (CBF). Similarly, under higher temperature VE-frame behaves like an MRF since the stiffness contribution from the added component becomes smaller. Note however that the responses of the VE-frame should be still much less than those of the MRF due to the supplemental damping effect (Fig. 5). Fig. 7 shows typical VE-damper hysteresis having high stiffness at 16°C and much lower stiffness at 32°C. In spite of this sensitivity, other member responses (Fig. 6) and overall responses (Fig. 5) of the VE-frame have been kept less sensitive to the temperature.

Nonlinear Analyses with Catastrophic Earthquakes. Additional analyses are conducted to determine the yield strength of the designed stiff VE-frame. The artificial earthquake is amplified to 0.6g, 0.8g, and 1.0g, respectively. For the 0.6g earthquake, temperatures of 16°C, 24°C, and 32°C are used, and for the 0.8g and 1.0g earthquakes, only 24°C is used. Under 0.6g earthquake and at 16°C and 24°C, the VE-frame remains elastic and its average drift angle is 0.0066 rad. or less. At 32°C, the drift angle is 0.011 rad., and slight flexure yielding of only one beam occurs. Average of peak shear strain of all the dampers is 160% at 32°C. Under 0.8g earthquake and temperature of 24°C, all members remain elastic, but some are close to yielding, and average drift angle is 0.010 rad. and average damper shear strain 143%. Finally, under the 1.0g earthquake, 80% of the beams yielded and a few columns yielded. The average drift is 0.016 rad. and average damper shear strain 230%. The average of plastic rotations in the yielded members is small and 0.0035 rad. It appears that under 1.0g, the damper strain becomes more critical than other members, and this can be resolved if damper thickness were made larger, which however requires increasing the area of the damper layer in order to retain the damper stiffness. There are a few reasons for the strong performance of the stiff VE-frame: the vibration period of the VE-frame appears to be longer than T_a, and receive less forces than assumed in the code calculation; the D_ξ-expressions (Eq. 3) involves some conservatism; static force design approach involves conservatism, and; strength reduction factor are used for sizing the members.

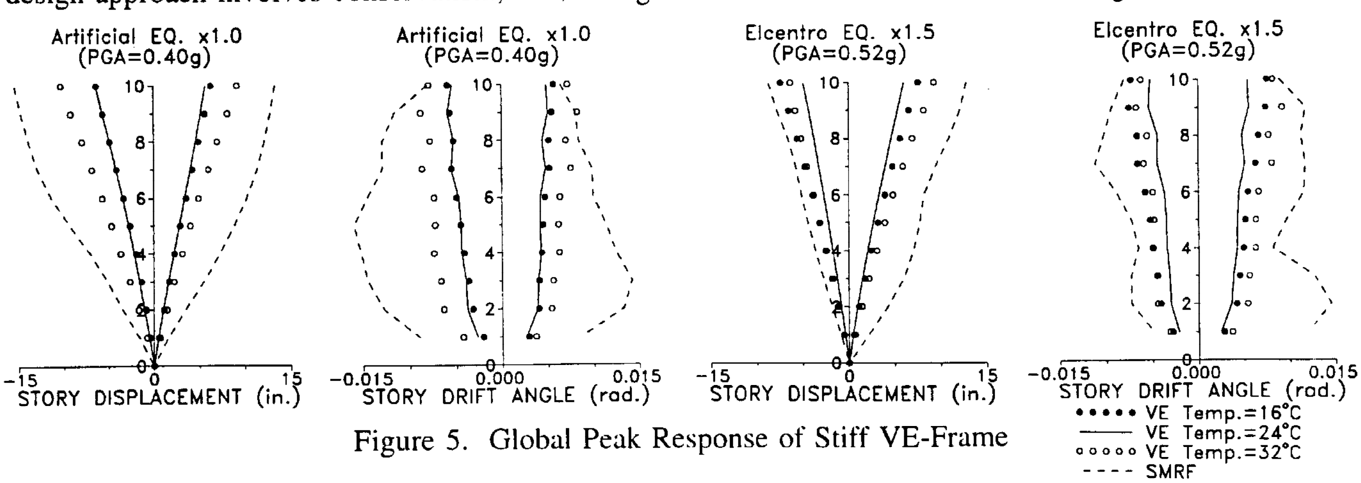

Figure 5. Global Peak Response of Stiff VE-Frame

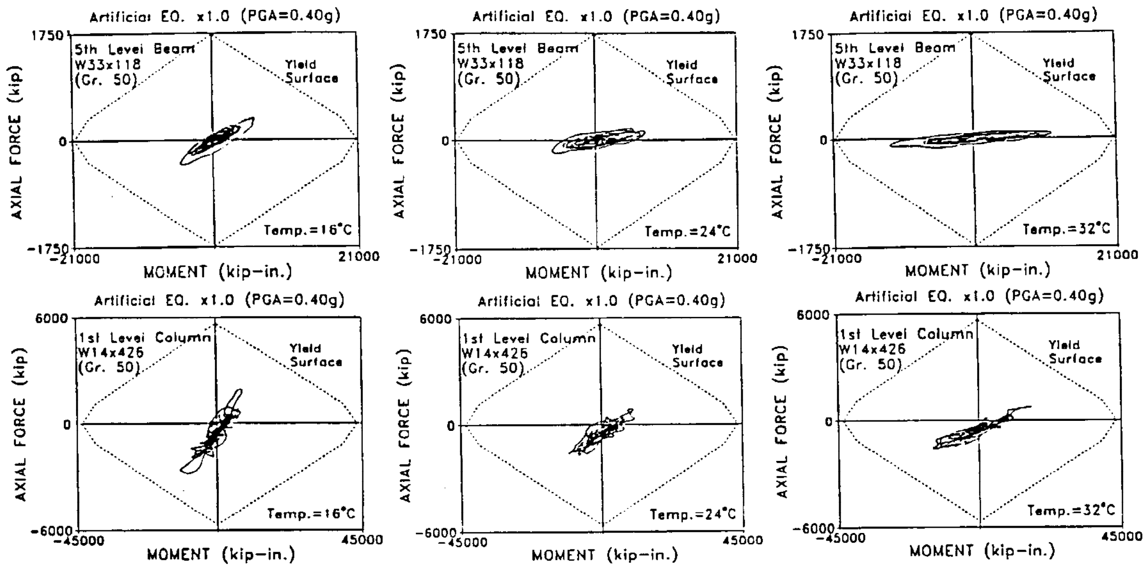

Figure 6. Member Forces of Typical Beam and Column under Various Temperature

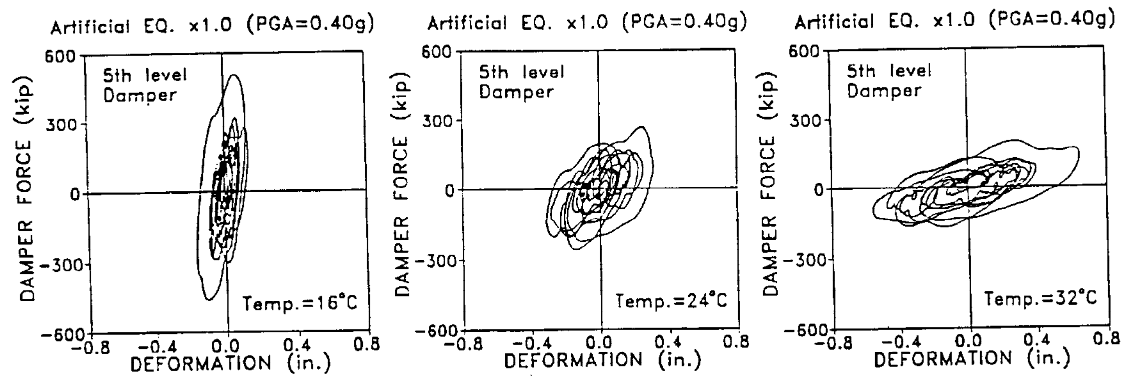

Figure 7. Damper Hysteresis under Various Temperature

Special MRF. Under the same load condition, a so-called special MRF (SMRF) is designed using the NEHRP code. As the name indicates, SMRF has the most ductile characteristics among MRFs. Like the VE-frame, only the middle bay has lateral force capacity. Table 1 indicates that the weight of steel used is about 67% more than that of the stiff VE-frame. The beams, columns, and connections of SMRF are much heavier than those of the stiff VE-frame. This design was governed by the stiffness requirement rather than the strength requirement, as aforementioned. Both 0.4g artificial earthquake and 1.5 times El Centro earthquake are used to conduct the dynamic analysis. Figs. 5 shows that the roof displacement of SMRF is 2 to 2.5 times that of the stiff VE-frame depending on the earthquake. More strikingly, the drift angles of some story levels are more than 4 times those of the stiff VE-frame, and exceed 0.015 rad. The drifts are also more irregularly distributed compared with the case of the stiff VE-frame. Yielding takes place in 80% of the beam ends, but the maximum plastic rotations are of moderate magnitude (0.01 rad. or less).

Flexible VE-frame. Based on the above results for the damage-free VE-frame, another VE-frame is designed by relaxing the drift limit. The criterion for this flexible VE-frame is that the building has drift angle less than 0.01 rad at 24°C, instead of 0.05 rad. This means that the building may have same nonstructural damage, but will have essentially no structural damage under the DBE. The design procedure is similar to the one for the stiff VE-frame. The upper bound of the structural period is 2.5 sec. to satisfy the target drift limit (Fig. 4) based on 30% damping ratio. In order to achieve this high 30% damping ratio, the ratios of $K'_d/K_f = 3$ and $K_b/K_f \geq 20$ are still used as before. The brace design forces are calculated under the load based on the actual period T (Eq. 3), rather than the T_a used before. The obtained frame has a period of 2.3 sec. with 32% damping ratio including structural damping 2%. The selected member size of beam, column and brace are listed in Table 1. It can be seen that the beam and column weight of this VE-frame is only 34% of that of SMRF. This VE-frame is analyzed by using the aforementioned two major earthquakes. Fig. 8 shows the drifts of this VE-frame are well controlled at 16°C and 24°C, and exceed the target value slightly at 32°C. All beam and column remain in elastic at 16°C and 24°C, and only three beams and one bottom column have minor yielding at 32°C under the artificial earthquake (the plastic rotation less than 0.0026 rad.). Note that this flexible VE-frame has the smallest steel weight among the three frames (Table 1), and its performance is between those of stiff VE-frame and SMRF.

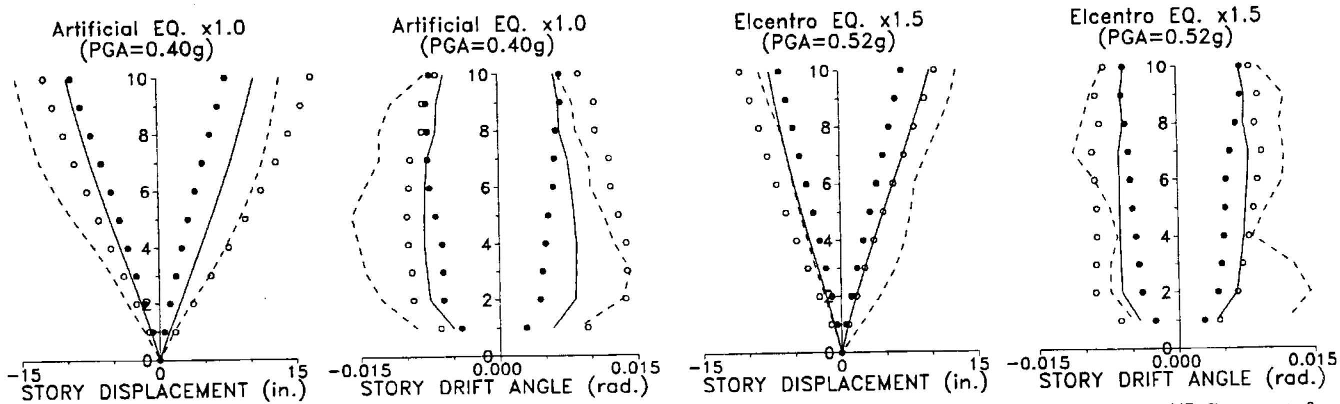

Figure 8. Global Peak Response of Flexible VE-Frame

CONCLUSIONS

The VE-frames designed under the proposed procedure has shown superior seismic performance. The stiff VE-frame has demonstrated essentially a damage-free performance, developing very small drifts against not only a major earthquake considered in the code but also the earthquake of extraordinary magnitude. It also provides saving of beam and column steel by 40% compared to the conventional SMRF. The flexible VE-frame has shown larger drifts than the stiff VE-frame, but still smaller drifts than the SMRF. It has substantial economy of 66% saving of the beam and column steel compared to the SMRF. It's small and flexible members have behaved mostly elastically against the major earthquakes. The analytical conclusions gained should be experimentally confirmed, by realistically simulating the actual details, sizes, and magnitudes of the forces and deformations of the VE-frame. Pursuant to this, a full scale and real-time testing of a VE-damper as well as the bottom 3-story of a 10-story VE-frame have started at Lehigh University. The frame test employs seismic load as well as gravity load of a large magnitude on a real-time basis. These tests are hoped to offer valuable guidance for further advancement of VE-damper application technology.

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REFERENCES

AIJ. (1993). *Recommendations for the Design of Base Isolated Buildings*. Architectural Institute of Japan.

AISC. (1994). *Load and Resistance Factor Design Specification, Seismic Provisions for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, Illinois, USA.

Kasai, K., Fu, Y. (1995). Seismic Analysis and Design Using Viscoelastic Damper. *Proc. of Symposium on a New Direction in Seismic Design*. Tokyo, Japan.

Kasai, K., Fu, Y. and Lai, M. L. (1994). Finding of A Temperature Insensitive Viscoelastic Frame. *Proc. of 1st World Conference on Structural Control*. Irvine, California, USA.

Kasai, K., Munshi, J. A., Lai, M. L. and Maison, B. F. (1993). Viscoelastic Damper Hysteretic Model: Theory, Experiment, and Application. *Proc., ATC 17-1: Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*. ATC, San Francisco, CA. USA. March 11-12, 1993, 521-532.

NEHRP. (1991). *Recommended Provisions for the Development of Seismic Regulations for New Buildings*. Federal Emergency Management Agency, Building Seismic Safety Council. Washington, D.C. USA.