Paper No. 428. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

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FRAME RETROFIT BY USING VISCOUS AND VISCOELASTIC DAMPERS

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ABSTRACT

This paper compares the properties and the seismic behavior of a system having viscous (VS-) damper with that having viscoelastic (VE-) damper. The consistent expressions for added stiffness and damping caused by VS-damper or VE-damper are derived, by including the important effect of the member supporting the damper. Two fundamental parameters, brace stiffness ratio and damper loss stiffness ratio, are used to conduct the parameter analysis to investigate the stiffness and damping property of the two systems. Under some condition, the system having VS-damper can gain substantial added stiffness, becoming similar to the system having VE-damper. The closed-form expressions for the variations, caused by added stiffness and damping, in peak response quantities of the two systems are derived by using a compatible combination rule. Such expressions are applied to the drift-control-based retrofit of a 10-story frame by using either VS- and VE-dampers. The seismic global and local response are comparatively discussed.

KEYWORDS

Frame Retrofit; Viscous Damper; Viscoelastic Damper; Added Stiffness; Seismic Response Reduction.

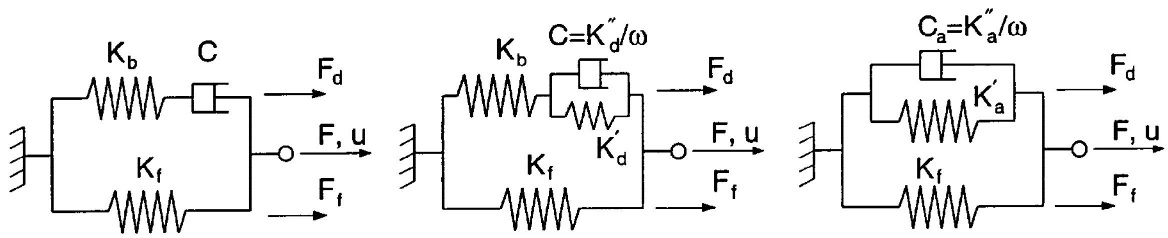
INTRODUCTION

Use of supplemental energy dissipation devices is considered as an effective way to reduce structural seismic response. Among of such devices, viscous(VS-) damper and viscoelastic(VE) damper are the two which dissipate seismic energy through velocity dependant resistance. VS-damper such as fluid damper can be designed to provide resistance force out-of-phase with harmonic deformation (Constantinou 1993). VE-damper such as polymer damper usually provides resistance force containing components both in-phase and out-of-phase with harmonic deformation (Kasai *et al.* 1993, Chang *et al.* 1992). When used in a frame, either VS- or VE-damper requires a support component such as a brace. Thus, typically the frame contains an *added subsystem* consisting of damper itself and a brace. Earlier studies indicated a considerable effect of brace stiffness on the stiffness and damping of the subsystem (Kasai *et al.* 1994, Sause *et al.* 1994). When subject to harmonic deformation, the added subsystem typically provides the resistance force containing the components both in-phase and out-of-phase with the deformation. The in-phase component is an elastic force that depends on the storage stiffness of the subsystem. The out-of-phase component is a viscous force and corresponding to the energy dissipation that depends on the loss stiffness of the subsystem. Although VS-damper and VE-damper have different mechanical property, the harmonic response of the subsystem containing either dampers can be consistently simulated by an equivalent model and such a model is convenient to compare the performance of the two subsystem on a consistent basis and will be presented herein.

This paper also discusses the comparative seismic performance of a frame having either VS- and VE-subsystem. BY simplifying the frame as a single-degree-of-freedom (SDOF) system, system responses are investigated through closed-form expressions, by using harmonic excitation as well as idealized seismic excitation. The results are also applied to drift-control-based retrofit of a flexible steel moment resistance frame by using either VS- or VE-dampers. Dynamic analysis results for the frame are explained in order to compare the performance of the two types of dampers and to demonstrate the reasonableness of the SDOF predictions.

PROPERTIES OF SDOF SYSTEMS WITH VS- AND VE-DAMPERS

Model. The SDOF model with VS-damper, VE-damper and bracing members are shown in Fig. 1 respectively. There K_f may stand for lateral stiffness of a one story frame segment, and K_b lateral stiffness of a brace supporting the damper. The VS-damper (Fig. 1(a)) is assumed to have linear behavior and is represented by a dashpot with a damping coefficient C. In a typical VS-damper, C is known to be almost constant over a certain excitation frequency range as well as temperature range. However, depending on design of the damper, the C may strongly depend on the deformation amplitude. This aspect is not the scope of the present discussion. The VE-damper (Fig. 1(b)) is simulated by two parallel linear elements, a spring with stiffness K_d' and an equivalent dashpot with damping coefficient $C = K_d''/\omega$, where ω is the circular frequency of the harmonic excitation. K_d' and K_d'' are the storage and loss stiffness of the VE-damper respectively and they are frequency and temperature dependent. Because in frequency domain the ωC of VS-damper and K_d'' of VE-damper govern represent the velocity-dependent resistance forces, the parameter K_d'' ($= \omega C$) will be used to establish the connection between VS-damper and VE-damper.



(a) With Viscous Damper (b) With Viscoelastic Damper (c) Equivalent System

Figure 1. SDOF System Model

Storage stiffness and Loss Stiffness. In Fig. 1(b) the added subsystem consisting of VE-damper and brace, is characterized by three parameters, K_d', C ($= K_d''/\omega$) and K_b. The added subsystem (Fig. 1(a)) having VS-damper can be treated as the special situation with $K_d' = 0$. The force (F_d) - deformation (u) relationship of the subsystem is:

$$(K_b+K_d')F_d(t)+C\frac{dF_d(t)}{dt}=K_bK_d'u(t)+K_bC\frac{du(t)}{dt} \tag{1}$$

When subject to the harmonic displacement $u(t) = u_{max}\sin\omega t$, where ω is a given circular frequency, the damper force $F_d(t)$ has the components both in-phase and out-of-phase with the displacement:

$$F_d(t)=K_a'u_{max}\sin\omega t+K_a''u_{max}\cos\omega t \tag{2}$$

where K_a' and K_a'' are, respectively, the storage stiffness and loss stiffness of the added subsystem, and they can be determined by substituting Eq. 2 into Eq. 1:

$$K_a'=\frac{(K_b+K_d')K_bK_d'+K_bK_d''^2}{(K_b+K_d')^2+K_d''^2} \qquad K_a''=\frac{K_b^2K_d''}{(K_b+K_d')^2+K_d''^2} \tag{3}$$

Note that for the subsystem having VS-damper, $K_d' = 0$, meaning that the damper has only loss stiffness in the form of ωC. But the subsystem has both storage and loss stiffness (i.e. K_a' and K_a'' are not zero even if $K_d' = 0$, see Eq. 3). Eq. 2 also shows that the added subsystem can be alternatively simulated by a Kelvin model with two frequency dependant parameters K_a' and K_a'' (Fig. 1(c)).

Added Stiffness and Damping. The frame force is in-phase with displacement, $F_f(t) = K_fu(t)$. Based on this

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and using Eq. 2, the total resistance force F(t) of the whole system is :

$$F(t) = F_f(t) + F_d(t) = (K_f+K_a')u_{max}\sin\omega t + K_a''u_{max}\cos\omega t \tag{4}$$

The term involving $\sin\omega t$ in Eq. 4 indicates the elastic force of the whole system (i.e. in-phase with displacement u(t)). Thus the whole system's stiffness becomes K_f+K_a', and K_a' is named as *added stiffness.* The term involving $\cos\omega t$ in Eq. 4 indicates the viscous force of the whole system (i.e. out-of-phase with displacement u(t)), and it is proportional to K_a''. Thus K_a'' reflects the *added energy dissipation capability* of the system. By defining added stiffness ratio $\alpha_a = K_a'/K_f$ and by using the concept of modal strain energy, the added damping ratio (Kasai and Munshi 1994, Munshi and Kasai 1994) $\xi_a= K_a''/[2(K_f+K_a')]$, Eq. 4 can be expressed as follows:

$$F(t) = u_{max}K_f(1+\alpha_a)[\sin\omega t+2\xi_a\cos\omega t] \tag{5}$$

$$\alpha_a=\frac{\alpha_b\alpha_d''^2}{\alpha_b^2+\alpha_d''^2} \qquad \xi_a=\frac{\alpha_b^2\alpha_d''}{2[\alpha_b^2+(1+\alpha_b)\alpha_d''^2]} \qquad \text{(VS-system)} \tag{6}$$

$$\alpha_a=\alpha_b\alpha_d''\frac{\alpha_b\eta_d+(1+\eta_d^2)\alpha_d''}{(\alpha_b\eta_d+\alpha_d'')^2+\eta_d^2\alpha_d''^2} \qquad \xi_a=\frac{1}{2}\frac{\alpha_b^2\eta_d^2\alpha_d''}{(\alpha_b\eta_d+\alpha_d'')(\alpha_b\eta_d+\alpha_b\alpha_d''+\alpha_d'')+(1+\alpha_b)\eta_d^2\alpha_d''^2} \qquad \text{(VE-system)} \tag{7}$$

where $\eta_d=K_d''/K_d'$ is the loss factor of VE-damper, $\alpha_b = K_b/K_f$ is the brace stiffness ratio, and $\alpha_d'' = K_d''/K_f$ is the damper loss stiffness ratio which represents the energy dissipation capability of VS- or VE-damper. The Eq. 6 for VS-damper case can be alternatively derived from Eq. 7 for the VE-damper case by setting $\eta_d \rightarrow \infty$, since $K_d'=0$. For the VE-damper case, Kasai *et al.* (1994) have derived the similar expressions by using a so-called complex stiffness method.

Parameter Analysis. The dynamic behavior of the damped system basically depends on the above two parameters α_a and ξ_a, that express the added stiffness and added damping effect. Further it can be seen from Eqs. 6 and 7, both α_a and ξ_a are the function of α_b (brace stiffness ratio) and α_d''(damper loss stiffness ratio).

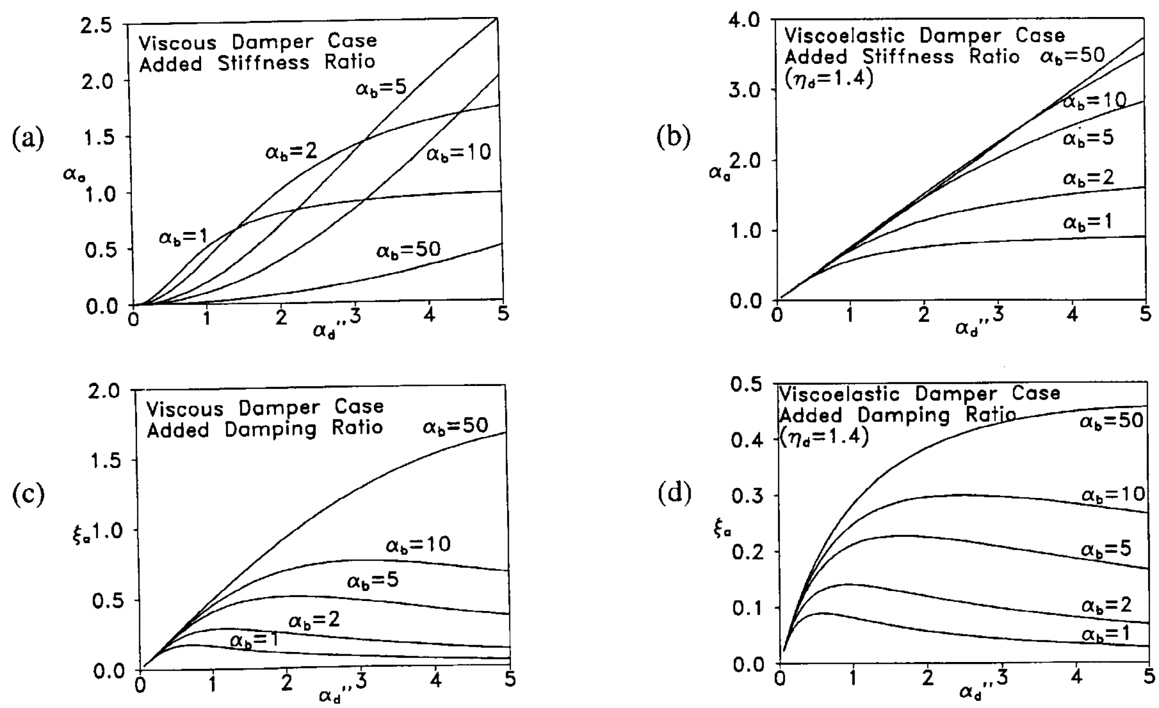

Figure 2. Added Stiffness Ratio and Damping Ratio vs. Brace Stiffness ratio and Loss Stiffness Ratio

Fig. 2(a) and (b) show α_a versus α_b and α_d'' for VS-case and VE-cases respectively. Generally the added stiffness ratio is a double curvature curve (in VE-case this occurs when α_d'' is very large, not shown in Fig. 2(b)). When α_d'' increases, α_a increases and reaches the peak value (equal to α_b), which is confirmed by Eqs. 6 and 7 for VS-case and VE-case respectively. This is because when the damper loss stiffness K_d'' becomes very large, the damper becomes difficult to deform and the total deformation of the added subsystem concentrates on the brace only, and both VS- and VE-added subsystems can provide the added stiffness K_a' up to magnitude of brace stiffness K_b. This is also occurs when the system is subjected to high frequency excitation, since $K_d'' = \omega C$. With the increase of K_d'', VE-case has faster increase of K_a' than VS-case at the beginning especially with a large brace stiffness. Thus, generally VE added subsystem gives larger K_a' than VS added subsystem when both α_b and α_d'' are the same between the two subsystems. It is interesting to note that the K_a' of VS-system increases faster with a softer brace when α_d'' increases, indicating the sensitivity of VS-case against brace stiffness. However, if a very stiff brace is used, the added stiffness of VS-subsystem can be neglected in the typical range of α_d'' as shown in Fig. 2(a).

Fig. 2(c) and (d) show added damping ratio ξ_a versus α_b and α_d'' for VS-case and VE-case respectively. In both two cases the trend of ξ_a is similar. The more stiffer the brace is, the higher damping ratio can be achieved. This is also mentioned by Sause *et al.* (1994) for the viscoelastically damped system. ξ_a has a peak value which depends on the given α_b. With respect to α_d'', ξ_a varies significantly at the left side of the peak, and become insignificantly at the right side. With the same α_b and α_d'', the VS-case has much larger ξ_a than VE-case when α_b and α_d'' are the same between the two cases. This is because due to its larger added stiffness K_a', VE-system stores more strain energy than VS-system and yet with same damper loss stiffness K_d'' it dissipates the same amount of energy as VS-system.

Harmonic Response. From Eqs. 2 and 5 the peak magnitudes of the force of a whole system (F_{max}), frame member force ($F_{f,max}$), and damper force ($F_{d,max}$) are:

$$F_{max}=u_{max}K_f(1+\alpha_a)\sqrt{1+4\xi^2} \qquad F_{f,max}=u_{max}K_f \qquad F_{d,max}=u_{max}\sqrt{K_a'^2+K_a''^2} \tag{8}$$

Considering a mass M is attached at the right end of the system shown in Fig. 1, the peak steady state excitation force $F_{max} = MS_a$, where S_a is spectral acceleration due to the harmonic ground motion. Likewise, the peak steady state displacement $u_{max} = S_d$, where and S_d is spectral displacement. Then the S_a-S_d relationship can be derived from Eq. 8 as follows:

$$S_a(\xi,\omega) = \sqrt{1+4\xi^2}\,\omega^2 S_d(\xi,\omega) \tag{9}$$

where $\omega = [(K_f+K_a')/M]^{1/2}$ is the natural vibration frequency of the system.

RESPONSE VARIATION DUE TO ADDED STIFFNESS AND DAMPING

Displacement Reduction Caused by Damping. The effect of damping on the reduction of spectral displacement can be expressed by a *damping reduction function* D_ξ. The D_ξ reflects statistical trend of the reduction over various types of earthquakes. Kasai and Fu (1995) obtained a simplified expression of D_ξ that matches well with the expression given by AIJ (1993) as well as with the values specified by NEHRP (1994), i.e.:

$$D_\xi=\frac{S_d(\xi,T)}{S_d(\xi_0,T)}=\sqrt{\frac{1+25\xi_0}{1+25\xi}} \tag{10}$$

where S_d is the earthquake displacement spectrum. T is the vibration period of the structural system. ξ_0 is the damping ratio of the frame ($T = 2\pi/\omega$). ξ is the total damping ratio of the system including added damping ratio ξ_a and frame damping ratio ξ_0 (i.e., $\xi = \xi_0 + \xi_a$).

Variation due to Period Shift. According to NEHRP's assumption (1991), the seismic coefficient C_s is proportional to $1/T^{2/3}$. Because C_s is intended to reflect the trend of pseud-acceleration spectrum $S_{pa}(\xi,T) = (2\pi/T)^2 S_d(\xi,T)$, the S_d-variation due to shifting of the period caused by the added subsystem is:

$$\frac{S_d(\xi,T)}{S_d(\xi,T_0)} = \frac{S_{pa}(\xi,T)/(2\pi/T)^2}{S_{pa}(\xi,T_0)/(2\pi/T_0)^2} = \left(\frac{T}{T_0}\right)^{4/3} \tag{11}$$

where and T_0 is the and period of the original system without dampers. Note that when T is small C_s has a constant value (NEHRP 1991). In such a case, the above equation will overestimate $S_d(T)$, and give a conservative result when it is used to predict the displacement restriction.

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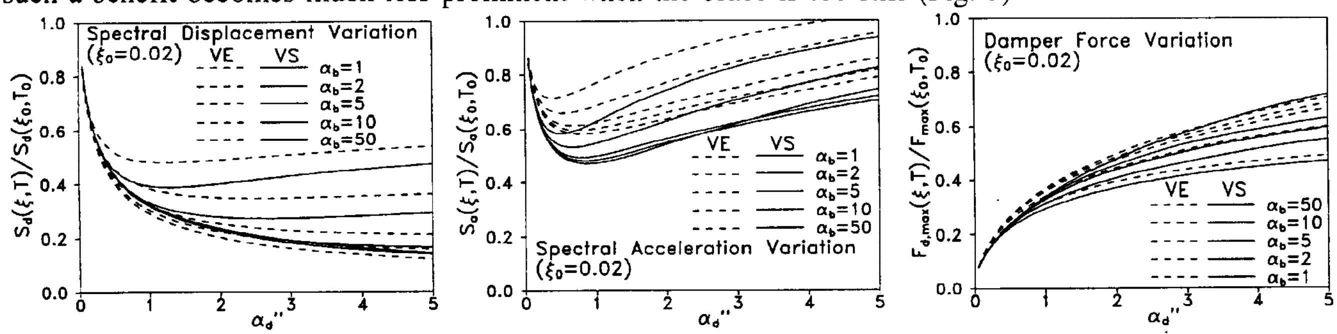
Spectrum Reduction Function. As described before, When the original frame and added subsystem are combined, both vibration period and damping ratio of the whole system change. Based on Eq. 9-11, the reduction function for S_d, namely R_{sd}, and the reduction function for S_a, namely R_{sa} can be obtained as follows:

$$R_{sd}=\frac{S_d(\xi,T)}{S_d(\xi_0,T_0)}=\sqrt{\frac{1+25\xi_0}{1+25\xi}}\left(\frac{T}{T_0}\right)^{4/3}=\sqrt{\frac{1+25\xi_0}{1+25\xi}}(1+\alpha_a)^{-2/3} \tag{12}$$

$$R_{sa}=\frac{S_a(\xi,T)}{S_a(\xi_0,T_0)}=\sqrt{\frac{(1+25\xi_0)(1+4\xi^2)}{(1+25\xi)(1+4\xi_0^2)}}\left(\frac{T}{T_0}\right)^{-2/3}=\sqrt{\frac{(1+25\xi_0)(1+4\xi^2)}{(1+25\xi)(1+4\xi_0^2)}}(1+\alpha_a)^{1/3} \tag{13}$$

where α_a is defined by Eq. 6 and Eq. 7 for VS-case and VE-case respectively. Note also that $1+\alpha_a = (T_0/T)^2$. Note that Eq. 13 assumes that Eq. 9 derived from the harmonic steady state is applicable to the random case.

R_{sd} and R_{sa} can be further expressed by α_b and α_d'' by using Eqs. 6 and 7 and $\xi = \xi_0 + \xi_a$. The results are plotted in Fig. 3(a) and (b) respectively. Note that damper loss stiffness increases (i.e. energy dissipating capability increases), the peak displacement decrease effectively when $\alpha_d'' < 1$ and less effectively after α_d'' exceeds 1, especially when a soft brace is used. It is possible by using VS- or VE-dampers to achieve very small displacement reduction ratio R_{sd} of about 0.15. Also the peak acceleration reaches a minimum value at α_d'' of about 0.5. Because of the larger added stiffness, VE-system has about 20% larger spectral acceleration than VS-system. It appears stiff brace can give more reduction in both S_d and S_a. However, such a benefit becomes much less prominent when the brace is too stiff (Fig. 3).



(a) (b) (c)

Figure 3. Variations in Spectral Displacement, Acceleration and Damper Force

Variations in Forces. Since the whole system force F_{max} (base shear) and frame member force $F_{f,max}$ are proportional to S_a and S_d respectively, their variation caused by the added damping and added stiffness can be found in Fig. 3(b) and (a). By manipulating Eq. 8, the ratio of peak damper force to peak whole system force ($F_{d,max}(\xi,T)/F_{max}(\xi,T)$) can be derived. Then using this ratio, Eq. 13, and $F_{max} = MS_a$, the peak damper force can be normalized to the peak shear of the original system without damper, i.e.:

$$\frac{F_{d,max}(\xi,T)}{F_{max}(\xi_0,T_0)}=R_{sa}\frac{F_{d,max}(\xi,T)}{F_{max}(\xi,T)}=R_{sa}\sqrt{\left[1-\frac{1}{(1+\alpha_a)(1+4\xi^2)}\right]^2+\left[\frac{2\xi}{(1+\alpha_a)(1+4\xi^2)}\right]^2} \tag{14}$$

Figure 3(c) shows the variation of the above ratio with respect to the two fundamental parameters α_b and α_d'' in both VS- and VE-system cases. In general, with the increase of loss stiffness ratio α_d'', the damper force will increase but becomes almost unchanged when loss stiffness is large. Usually the VS-system has slightly smaller damper force than VE-system.

It is notable that the accuracies of Eqs. 12 to 14 depend on the accuracies of Eq. 10 (for the effect of damping) and Eq. 11 (for the effect of period shift). The damping reduction function D_ξ in Eq. 10 appears to be close to the upper bound equation given by Hanson and Jeong (1994). Hence it conservatively expresses the effect of damping. Thus when the above expressions are used to estimate the required added damping as well as accompanying period shift, a conservative result will be obtained especially in VS-case because variation of VS-system's response mainly depends on its damping effect.

MULTI-STORY FRAME RESPONSE

10-story Steel Frame Retrofit. A 10-story flexible and weak steel moment resistant frame (MRF) is selected in order to demonstrate applications of the above findings and to show the effectiveness of damper in the frame retrofit situation (Fig. 4). The MRF is designed by Anderson and Bertero as the minimum weight steel frame. More details about the frame can be found elsewhere (Anderson and Bertero 1969, Kasai and Munshi 1994). The control of the story drift is considered as the key for the frame retrofit. Because as mentioned before, the VS- or VE-added subsystem can provide very large stiffness under the high frequency excitation, higher mode responses of the damped frame are largely suppressed and the first mode response dominates the system displacement. Therefore only 1st mode response analysis is necessary when Eq. 12 is used to calculate the required reduction of spectral displacement. For the comparison purpose three retrofit cases are considered. Also, since too stiff brace is not necessary as mentioned, $K_b/K_f=10$ is selected for all three cases. In Case-1 VE-dampers are used to limit the maximum story drift angle to 0.8%. Based on the 1st mode displacement calculation by using spectrum method, the required $R_{sd}=0.28$. According to Eq. 12 and Eq.7, $K_d''/K_f=1.2$ is needed to satisfy the required R_{sd}. In Case-2 VS-dampers are used and they have the same K_d''/K_f (=1.2) ratio as VE-frame, that means the two frames have same energy dissipation capability. In Case-3 VS-dampers are used again but $K_d''/K_f=0.5$ is selected to make the frame has the same damping ratio as the VE-frame. The frame's damping ratio caused by added subsystems is calculated by using the concept of modal strain energy method (Clough and Penzien 1975, Chang *et al.* 1991, Kasai and Fu 1995). The original MRF's damping ratio is assumed as 0.02. With the same damper loss stiffness (K_d''), the damping ratio of VS-frame (Case-2, 0.50)) is twice as that of VE-frame (0.25). This is because VE added subsystem has larger storage stiffness which will induce more strain energy stored than that stored by VS-system.

Dynamic time history analysis are conducted in order to verify the design and to compare the frames' responses. An artificial earthquake record having peak ground acceleration of 0.4g is used. It simulates NEHRP spectrum (1991) with parameters Av=Aa=0.4, S=1.2 and R=1 at damping ratio of 5%. The analytical viscoelastic element (Kasai *et al.* 1993) is used to model both VS- and VE-dampers. For VE-dampers the ambient temperatures are set to 24°C and the nonlinear response due to temperature rise is suppressed. Likewise, the possible nonlinear response caused by VS-damper is not considered. The effect of temperature rise is not very prominent under the typical duration of the earthquake (Kasai *et al.* 1993). The members are also assumed to be elastic. For highly damped building, Kasai and Munshi (1994) as well as Hanson and Jeong (1994) indicated the similarity of global response between elastic and inelastic frames.

Comparative Performance. Fig. 5(a) shows the envelop of story shear force of the original MRF, VE-frame, and VS-frames. Because of shorter period caused by larger added stiffness and less damping ratio, VE-frame has lager seismic load than VS-frames. The shear force reduction can be expressed through the S_a reduction. For Cases-1, 2 and 3, Eq. 13 (Fig. 3(b)) gives the estimation of elastic base shear reduction ratios of 0.62, 0.49 and 0.51, respectively, with respect to the original MRF. For the 10-story frames, the actual story shear reduction ratios are averaged throughout the frame height, which are 0.51, 0.38 and 0.39 for Case-1, 2 and 3 respectively. Compared with these, Eq. 13 gives 20-30% conservativeness.

Fig. 5(b) and (c) show the envelops of story displacements and drift angles. They also show that seismic response can be significantly reduced by the dampers. Compared with the target value (0.8%), the drift angles are controlled very well in VE-frame (less than 0.73%) as well as VS-frame (Case-2, less than 0.62%). Although VE-frame has smaller damping ratio than VS-frame (Case-2), it can achieve almost same maximum displacement response as VS-frame. This is because VE-frame has both added stiffness and damping, while VS-frame mainly has added damping. Accordingly when these frame have the same damping ratio (Case-1 and Case-3), the VS-frame develops larger story displacements and drifts than VE-frame.

Fig. 5(d), (e) and (f) show the maximum seismic elastic member forces. Compared with the original MRF, all VE- and VS-frames have significant reduced member forces. Since beam moments are approximately proportional to the story drift, the drift-controlled VE-frame and VS-frame (Case-2) have almost same beam moments due to their similar maximum displacements. But the VS-frame (Case-3) has larger beam moments than VE-frame because of its larger displacements as explained above. Corresponding to the story shear force (Fig. 5(a)), VE-frame has the largest column axial forces and the VS-frame (Case-2) has the smallest. Due to the added stiffness effect and larger story shear VE-frame has the larger brace forces (damper force) than VS-frame. Comparing two VS-frame cases (Fig. 3 (c)), with more damper loss stiffness Case-2 has larger brace forces than Case-3. The normalized brace forces are calculated by using Eq. 15 (Fig. 3(c)) as

0.40, 0.36 and 0.24 for Case-1, 2 and 3 respectively. Compared with the actual 10-story-average values of 0.33, 0.30 and 0.20, Eq. 15 also gives close and conservative results in brace force estimation.

Fig. 6 show the moment (M_{col})-axial force (P_{col}) interactions of the 1st story columns for the three frames respectively. The P_{col} mainly depends on overturning moment having components both in-phase and out-of-phase with the displacement whereas M_{col} tends to be in-phase with displacement and drift as explained. Thus time lag exists between the M_{col} and P_{col} and the M_{col}-P_{col} curve has an inclined ellipse shape. The slope and fatness of such ellipse depend on the frame's global behavior, such as period and damping of the frame. Although VE- and VS-added subsystems locally affect the original frame in different ways, their influence to the global behavior is similar as explained. Accordingly the M_{col}-P_{col} curves in the frames become similar. Due to the larger seismic load (Fig. 5(a)), VE-frame's M_{col}-P_{col} curves slightly exceeds the yield surface while VS-frame's (Case-2) reaches to the surface. But when VS-frame (Case-3) has same damping ratio as VE-frame, Its M_{col}-P_{col} curves exceed the yield surface due to larger moment.

These analysis results were obtained by assuming elastic behavior of the frame members. It is to be noted that against the 0.4g artificial earthquake representing the design base earthquake (i.e. major earthquake), very few members of the retrofited frame indicated the forces exceeding the yield limit. In contrast, the original weak MRF indicates significant member forces exceeding yield limit at numerous locations. These indicate superior performance of both VE-damper and VS-damper in seismic retrofit of steel building.

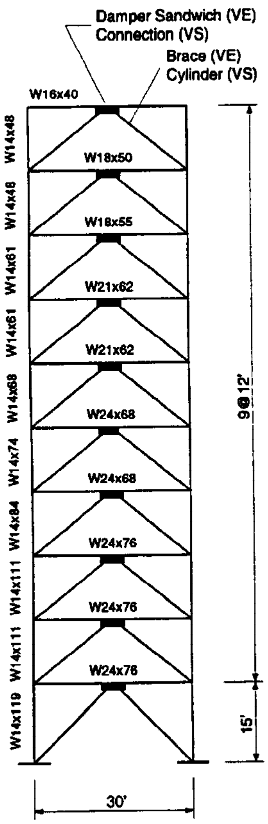


Figure 4.Retrofited Frame

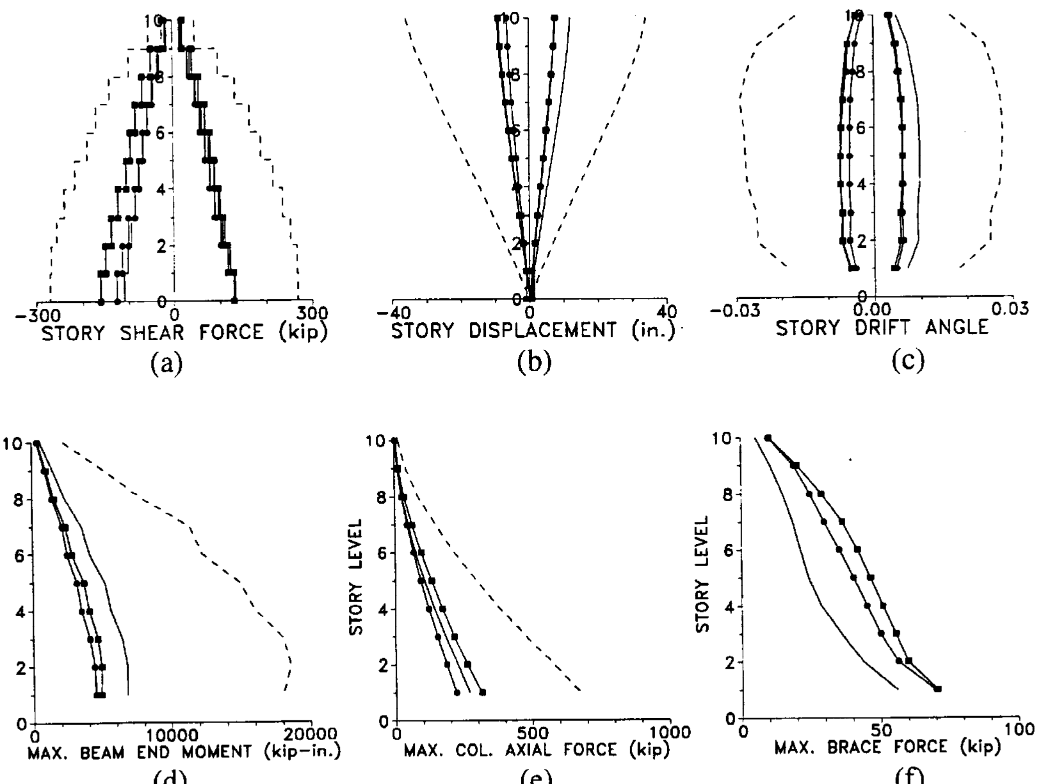


Figure 5. Frame Response

Artificial EQ. (PGA=0.4g)
VE-Frame (ξ=0.25, T=1.80)
VS-Frame (ξ=0.50, T=2.28)
VS-Frame (ξ=0.25, T=2.40)
MRF-Frame (ξ=0.02, T=2.43)

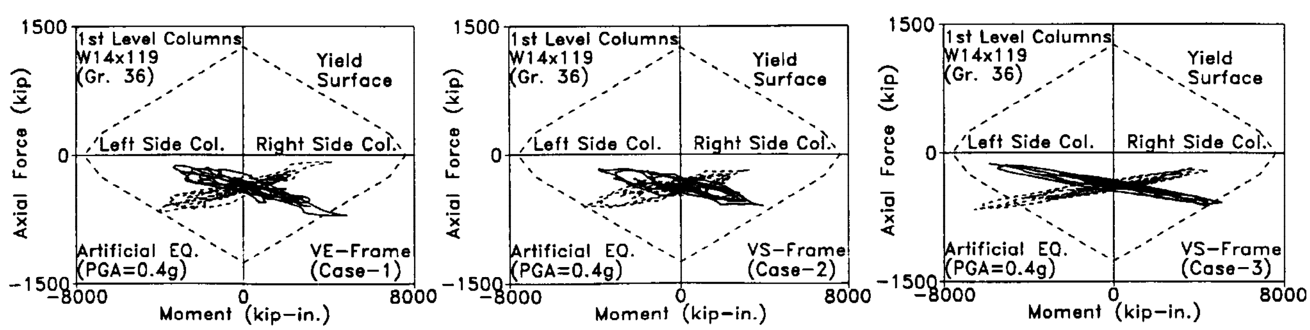


Figure 6. Moment-Axial Force Interaction of 1st Story Column

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CONCLUSION

Both viscous (VS-) dampers and viscoelastic (VE-) dampers can effectively improve seismic performance of a frame originally designed as a conventional moment frame. Both VS-frame and VE-frame will have added stiffness and damping. VS-frame has more damping ratio and less stiffness than VE-frame when their dampers are designed to have the same energy dissipation capability. The added stiffness of VS-frame can be neglected only if stiff brace is used and low frequency excitation is applied to the damper. Under the same ground motion and with the same energy dissipation capability of the damper, VS-frame and VE-frame have similar global as well as local responses, although the former develops slightly smaller response than the latter. Note, however, the very large damping ratio of the VS-frame that required to achieve the similar behavior as VE-frame may be difficult to realize in the actual construction. Moreover, VS-damper of a commonly considered piston type may be deformation dependent, and its damping coefficient or loss stiffness can vary with respect to the vibration amplitude. In contrast, VE damper's property seems to be more stable with respective to the response amplitude. Further the VE-damper is easy to install and can have a variety of configuration suited for actual use. These as well as the cost of the VS-damper and VE-damper may play a key role for actual selection of either damper. Although not explained in this paper, both VS-damper and VE-damper studied herein have moderate size and commercially available. For added subsystem, brace stiffness ratio α_b = 10 to 20, and damper loss stiffness ratio α_d''= 0.5 to 1.5 are recommended to achieve the effective results. It is not necessary to use very stiff brace.

ACKNOWLEDGMENT

The support provided by Nippon Steel Corporation, Japan is gratefully acknowledged. However, all opinions expressed in this paper do not necessarily reflect the views of the sponsor. The writer thanks Dr. Kazuhiko Kasai for his great contribution to this paper. Due to the conference's limitation for the number of publishable papers per person, his name could not be listed as an author.

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