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Seismic Retrofit of Moment Resisting Frame with Viscoelastic Dampers

Consolidaton parasismique d'un cadre rigide au moyen d'amortisseurs viscoélastiques

Erdbebenertüchtigung eines biegesteifen Rahmens mittels viskoelastischem Dämpfer

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SUMMARY

This paper discusses seismic retrofit of a weak moment-resisting steel frame by viscoelastic dampers. The viscoelastic damper behavior is accurately simulated by using a nonlinear element which takes into account the frequency and temperature dependency of the damper's material. The performance of the frame with and without dampers is investigated under four different earthquake excitations. Both elastic and inelastic member responses are investigated. The elastic response is simulated through a modal method. A simple method based on drift control is proposed for retrofit of weak frames.

RÉSUMÉ

L'article traite de la consolidation parasismique d'un cadre rigide sous-dimensionné au moyen d'amortisseurs viscoélastiques. Le comportement de l'amortisseur viscoélastique est simulé de façon précise à l'aide d'un élément non linéaire qui courante de la fréquence et de la relation de la fréquence et de la température avec les matériaux de l'amortisseur. La performance du cadre est étudiée pour quatre cas de charges sismiques, avec et sans amortisseurs. Le comportement élastique et inélastique des membrures est étudié. Le comportement élastique est également simulé au moyen d'une méthode modale. Une méthode simple basée est proposée pour la consolidation de cadres faibles.

ZUSAMMENFASSUNG

Der Beitrag behandelt die Erdbebenertüchtigung eines unterdimensionierten Stahlrahmens durch viskoelastische Dämpfer. Deren frequenz- und temperaturabhängiges Verhalten wird mittels eines nichtlinearen Elements rechnerisch simuliert. Das Verhalten des Rahmens mit und ohne Dämpfer wird für vier verschiedene Erdbebenanregungen untersucht. Neben dem inelastischen wird auch das elastische Verhalten, in modaler Analyse, simuliert. Für die Nachrüstung der Rahmen wird eine einfache Methode auf der Basis der Stockwerksauslastung als Steuerparameter vorgeschlagen.

INTRODUCTION

The Northridge earthquake of January 1994, by causing fracture damage of numerous moment resisting connections, indicated serious safety and cost implications of steel moment resisting frames. The safety concern of such frames calls for a retrofit scheme, which could ensure safety of both the occupants and the structure under a major earthquake, fulfil the serviceability, and be economical in the long run. Retrofit using VE-dampers result in well controlled response such as significantly small drift and member forces, and relative invariance of response to earthquakes of different characteristics. This paper will focus on the effectiveness of VE-dampers for seismic retrofit of MRFs and address the various issues associated with it.

STIFFNESS AND DAMPING

The key of the retrofit is to limit the story drift ratios to well within 1% so that strength demand of the members is well controlled. In order to estimate the amount of stiffness and damping required for this, consider a single degree-of-freedom (SDOF) system and its acceleration spectrum LMNO for nominal damping (e.g. 3%) as shown in Fig. 1(a). Locate the equivalent elastic force (point A) corresponding to the original building period T_o which is assumed to lie in the constant velocity region of the spectrum. Now locate the corrected elastic force (point B') for the shifted period T of the VE-damped building on a high damping spectrum L'M'N'O'. Points A, A', and B' are also shown in Fig. 1(b) representing the approximate elastic strain energies at peak deformation of the building. Point A' corresponds to force F' and deformation Δ', a situation when only stiffness due to VE-dampers is added to the structure. From Fig. 1(b) the approximate energy contributed by VE-dampers at peak structure deformation is the area under line A'B'. Thus,

$$1/2K\Delta'^2 - 1/2K\Delta^2 = \pi F_d\Delta \tag{1}$$

where $K = K_o + K_d'$, i.e stiffness of original MRF plus storage stiffness of the added dampers. The right hand side is the area of ellipse with equivalent energy, F_d = peak damper force, and Δ_d = peak damper deformation. Here $\Delta_d \approx \Delta$ is assumed by neglecting the brace and column axial deformation. Under the constant velocity, $\Delta' = \sqrt{K_o/K}\,\Delta_o$. Using $F_d = \eta K_d'\Delta$, where $\eta_d = K_d''/K_d'$ = loss factor of VE-material (Kasai et al, 1993), the following equations result:

$$\frac{K_d'}{K_o} = \frac{(\Delta_o^2/\Delta^2 - 1)}{1 + 2\pi\eta_d}, \quad \text{and} \quad \xi = \frac{\eta_d}{2}\frac{K_d'}{K} = \frac{\eta_d}{2}\left[\frac{1}{1 + K_o/K_d'}\right] \tag{2}$$

The force and deformation reduction from F' and Δ' to F and Δ, respectively (Fig. 1(b)) become:

$$\frac{\Delta}{\Delta'} = \frac{1}{\sqrt{(1 + 4\pi\xi)}} = \frac{F}{F'} \tag{3}$$

Note that in Eq. 3 the reduction factor is determined based on the shifted structure period T. Eq. 3 well predicts the reduction of response when compared to those proposed by Newmark & Hall (1973), Kawashima & Aizawa (1986) and also that recommended by NEHERP for passive dissipation system as shown in Fig. 2.

RETROFIT METHOD

Based on the above simplifications, the following method is proposed as a general retrofit scheme of MRF using linear analysis approach:

1. Determine the story drifts (Δ_o) of the unretrofitted MRF under the design earthquake assuming elastic behavior by either modal analysis or equivalent lateral force analysis method.
2. By using ratio of the drift limit (Δ) and the original drift (Δ_o) as well as Eq. 2, determine

required stiffness ratio K_d'/K (Eq.2).
3. Determine the period T of the retrofitted building and its total global damping ratio by static lateral force method (Kasai et. al, 1994).
4. Obtain story drifts of the VE-damped structure for the design earthquake with the damping ratio and compare the drifts with the design/desired drifts. If the drifts are not within limits, revise added stiffness and damping locally if needed (Eq. 2), and go back to step 2.
5. Determine the forces in the members and ensure strength.

UNRETROFITTED MRF

The study considers the 10-story MRF as shown in Fig. 3(a), designed by Anderson and Bertero (1969), satisfying the code minimum strength requirement but intentionally disregarding the code drift limit. The frame has small sections and does not have overstrength. It is very flexible having a long fundamental vibration period of 2.44 sec. El Centro (1940) scaled 1.5 times (peak ground acceleration of 0.52g), Artificial earthquake with spectrum characteristics compatible with NEHERP design spectrum (0.4g), Hachinohe earthquake (Japan), scaled 2 times (0.44g), and Parkfield earthquake (0.49g) are used for the analysis. The analysis assumes rigid floor diaphragms. The steel frame is assumed to have 3% viscous damping.

Elastic Building Case. Fig. 5(a) shows that building develops maximum displacement of 20 to 30 inches and very large story drift ratios of 2% to 3% under the 4 stipulated earthquakes.

Inelastic Building Case. For 5% strain hardening stiffness of steel members and yield stress of 36 ksi, the frame develops large plastic hinge rotations of over 0.015 rad. in columns and beams (Fig. 3). The displacements reach 30 in. with drift concentration of about 3% at several floors (Fig. 6(a). The responses indicate a biased motion and chances of incremental collapse for the building. The column moment and axial forces reach their limiting capacities and indicte a severe ductility demand as understood by comparing with elastic analysis (Fig. 5(a)).

RETROFITTED BUILDING

For limiting the drift ratio to under 1%, against the drift ratio of 2.9% (El Centro), a damper stiffness $K_d' = 0.8\, K_o$ is supplemented as per Eq. 2 at each floor (Fig. 4(a)). The loss factor η is not very sensitive to vibration period (3M ISD110 material). Table 1 lists the additional stiffness and the corresponding damper area required at each story for 1 inch thickness of layer and 1.9 second vibration period of VE-frame (24°C). The braces are sized to be at least 10 times stiffer than the VE-damper to ensure $\Delta_d \approx \Delta$ [Munshi & Kasai, 1994]. Nonlinear analyses of the VE-frame is carried out using writers' VE-finite element which accounts step-by-step for the frequency and temperature induced nonlinearity of response [Kasai, et al, 1993].

Elastic Building Case. Fig. 5(b) shows peak displacement of only 10 to 12 inches, with uniform drifts of less than 1% through the height of buildings. About 20% reduction of column axial forces and 70% reduction of column moments (Fig. 5(b)) significantly reduces the strength demand on the members and connections of the moment frame, which could enhance fatigue and fracture performance of MRFs.

Inelastic Building Case. Negligible inelastic activity of the building is seen with incorporation of this damper configuration as shown in Fig. 4(b). The building behaves almost in elastic manner under the 4 earthquake excitations, and response envelops of the building accordingly are very similar to those of the elastic building as shown in Fig. 6(b). The member forces are well within their corresponding capacities setting aside the chances of damage.

Linear Response Simulation. Though stiffness and damping contribution of VE-dampers will actually vary during an earthquake excitation of the VE-building, reasonable prediction of added stiffness and damping can be made based on its fundamental vibration period. The equivalent viscous damping ratio of the VE-damped building can be determined by using static lateral force method assuming single mode approximation [Kasai et al, 1994]. The VE-damped structure is simulated through Modal Strain Energy (MSE) approach [Munshi and Kasai, 1994], with 16% damping for all vibration modes. Fig. 7 shows that displacement, drift, and column axial forces, are well predicted through the linear approach when compared with nonlinear analysis using the VE-hysteretic element. Note, however, that the damper forces and brace forces predicted through the linear approach are significantly underestimated (Fig. 7) and need to be corrected (see proposed correction method, Munshi and Kasai, 1994).

Effect of Temperature. In the present analysis (24°C), an average maximum temperature rise of 3 to 4°C does not alter global building responses significantly [Munshi and Kasai, 1994]. Analyses of the VE-damped building at 16°C and 32°C shows different responses which are well predicted by modifying the 24°C responses by damper stiffness correction for that temperature.

CONCLUSIONS

Viscoelastic dampers significantly reduce strength and deformation demand on the members and connections and are effective for retrofitting the existing weak and vulnerable moment frames for enhanced performance under strong earthquakes. The amount of stiffness and damping needed can be reasonably estimated based on drift control strategy proposed herein. A simplified linear approach is proposed as a practical analytical method for VE-damped frame.

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Table 1. Retrofit Scheme of Moment Resisting Frame.

Story No. i	Story Stiffness K_o (Kip/in)	Story Drift Ratio Δ_o/h (El Cent) %	Target Drift Ratio Δ/h $\leq .01$ %	Required Damper Stiffness $K_D'=.8K_o$ (Eq. 2) (Kip/in)	Required Damper Area (in²) $A=K_D't/G'$	Damper Size Provided (in²)	Damper Stiffness Provided K_D' (Kip/in)	Story Drift Ratio (Linear Analysis) %	Actual Story Drift Ratio (Nonlin) %
10	21.3	2.25	≤1.0	17.2	111	100	15.0	0.62	0.54
9	27.0	2.88	1.0	21.8	140	100	15.0	0.91	0.82
8	35.8	2.94	1.0	28.9	187	200	30.0	0.92	0.82
7	40.9	2.91	1.0	33.0	213	300	45.0	0.89	0.80
6	48.9	2.62	≤1.0	39.5	255	300	45.0	0.84	0.81
5	55.0	2.15	≤1.0	44.5	287	400	60.0	0.75	0.75
4	62.4	1.97	≤1.0	50.4	325	400	60.0	0.76	0.73
3	92.2	2.26	≤1.0	74.5	481	500	75.0	0.73	0.66
2	77.3	2.63	≤1.0	62.5	403	500	75.0	0.77	0.68
1	82.9	2.07	≤1.0	67.0	432	500	75.0	0.60	0.52

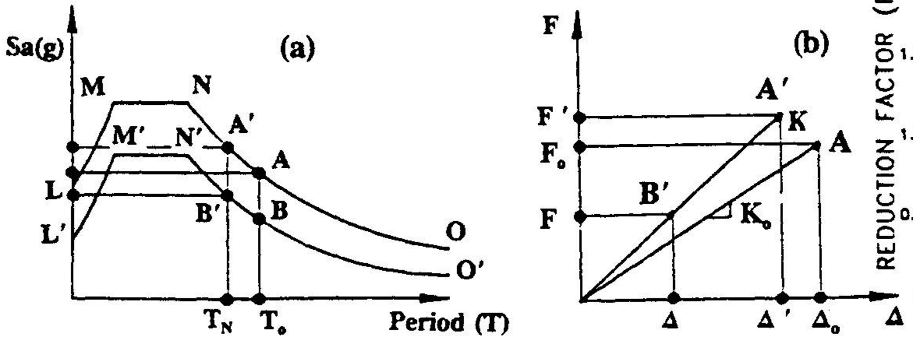

Fig. 1 (a) Schematic Spectrum for VE-System, and (b) Energy Idealization.

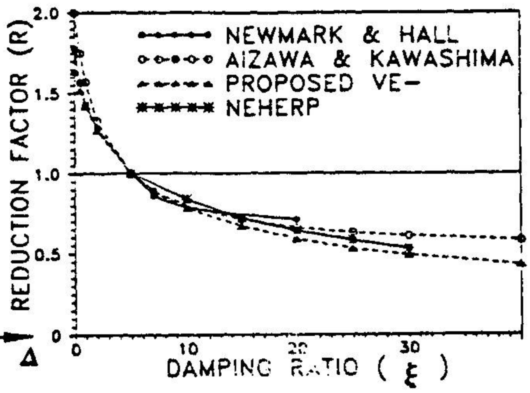

Fig. 2 Response Modification Factor for VE-System.

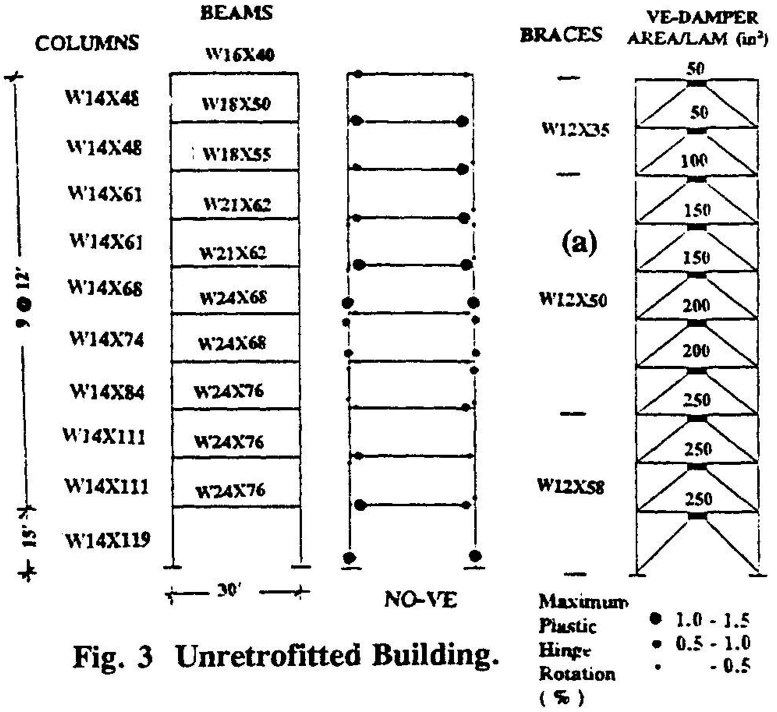

Fig. 3 Unretrofitted Building.

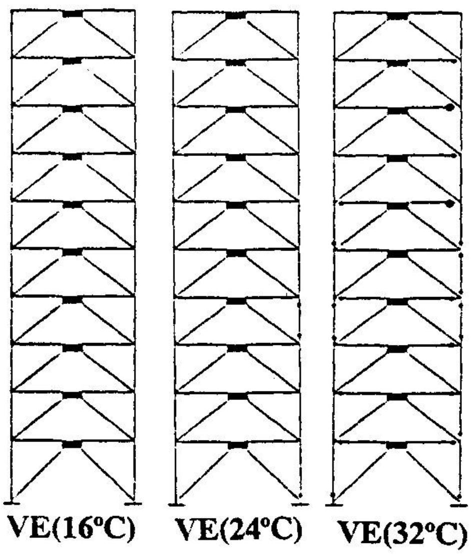

Fig. 4 Retrofitted Building.

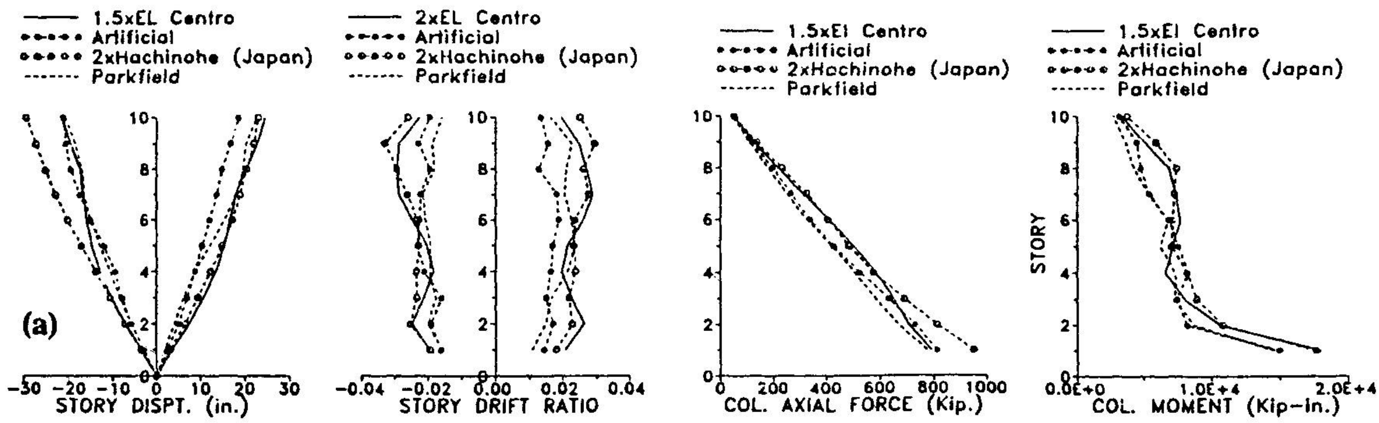

Fig. 5 Elastic Building: Global and Local Responses of (a) Unretrofitted and, (b) Retrofitted Building (Ambient Temperature 24°C).

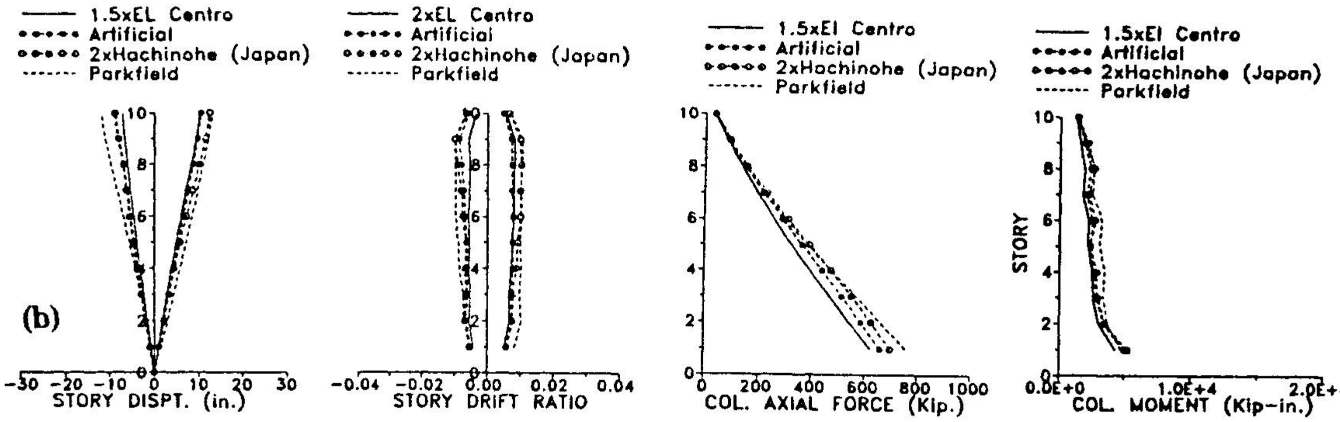

Fig. 5 Elastic Building: Global and Local Responses of (a) Unretrofitted and, (b) Retrofitted Building (Ambient Temperature 24°C).

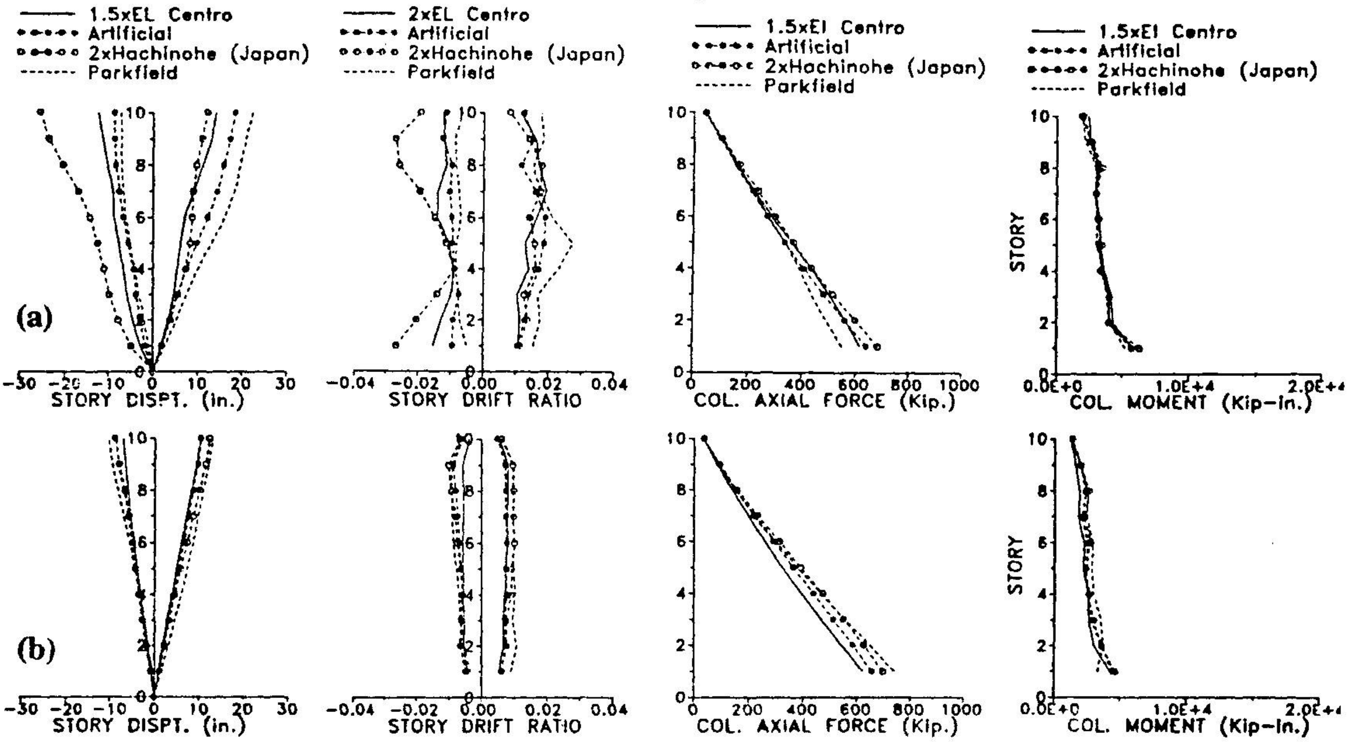

Fig. 6 Inelastic Building: Global and Local Responses of (a) Unretrofitted and, (b) Retrofitted Building (Ambient Temperature 24°C).

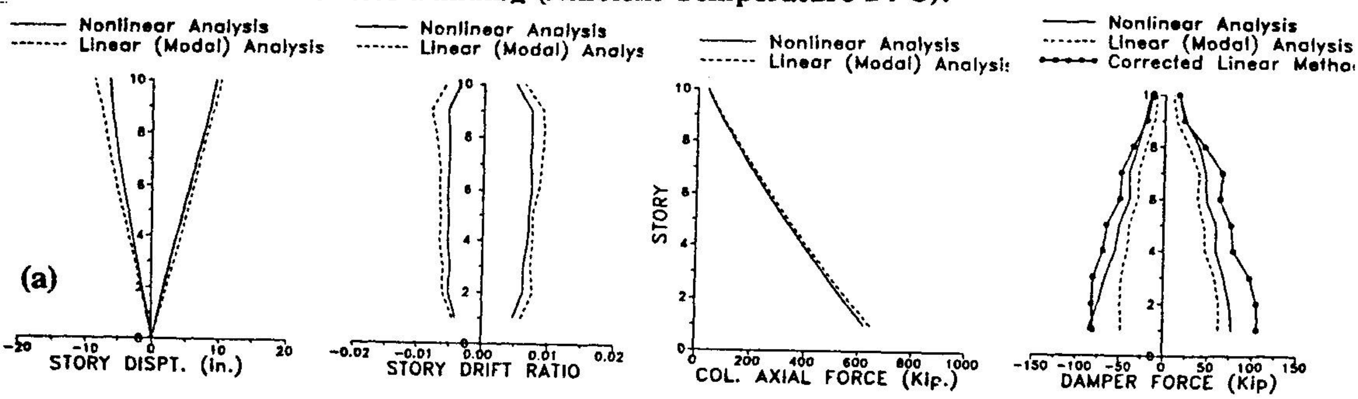

Fig. 7 Linear Response Simulation for El Centro earthquake (24°C).