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RETROFIT OF CONCRETE STRUCTURES USING SUPPLEMENTAL DAMPING DEVICES

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ABSTRACT

This paper presents the results of a shaking table study done on a reinforced concrete frame structure at 1:3 scale retrofitted with several alternative damping devices, i.e. viscoelastic, fluid, and friction dampers assembled as braces and viscous walls assembled as partial infills. The studies show that the dampers are efficient in reducing the deformation demands and the demand for energy dissipation by the structural members, if the structure responds either elastically or inelastically. However, the accelerations and the shear forces in the structural system may not be reduced and sometimes even increased. The paper shows a simplified procedure that can be used for preliminary design purposes based on composite spectrum and a capacity analysis.

KEYWORDS

Retrofit; Damping devices; Inelastic structures; Experiments; Shaking table; Composite spectra; Capacity analysis.

EXPERIMENTAL STUDY OF RETROFITTED STRUCTURE

A three story 1:3 scale model structure with lightly reinforced concrete frames, damaged by prior testing with moderate and severe earthquake was retrofitted by conventional concrete jacketing of interior columns and joint beam enhancements and was damaged again by several severe earthquakes (Bracci et al. 1992c). The same structure was further used to assess the possibility of retrofit of damaged frames with supplemental dampers installed in braces attached to the concrete joints. The study was developed to assess efficiency and structural interaction of various types of dampers, i.e.:(a) *viscoelastic dampers* of 3M Company (Lobo et al. 1993) (b) *fluid viscous* damper of Taylor Devices Inc. (Reinhorn et al., 1995) (c) *friction dampers* of Tekton Co. and Sumitomo Co. (Li et al., 1995). (d) *viscous walls* of Sumitomo Co. (Reinhorn et al. 1995).

<u>Structure Model for Shaking Table Study.</u> The structure was a three story 1:3 scale reinforced concrete frame structure original only for gravity loads without any special seismic provisions. The model was scaled from a prototype using mass simulation (Bracci et al. 1992a). The structural model (see Fig. 1) had a floor weight of 120 kN (27,000 lbs). The structure had 50.8 mm (2 in) thick slabs supported by 76.2x172.4 mm (3x6 in) beams supported by 101.6x101.6 mm (4x4 in) columns before retrofit. After the conventional retrofit the interior columns were increased to 152.4x152.4 mm (6x6 in) by concrete jacketing with longitudinal post-tensioned reinforcement and with a column capital at each floor obtained by a fillet of joint connection. The columns were symmetrically reinforced using 1.2%, total reinforcement ratio, and the beams had 0.8% positive reinforcement along entire beam and 0.8% negative reinforcement ratio above the supports. Details of reinforcement, material properties and construction can be found in Bracci et al. 1992a. The structure was subjected to earthquake simulated motion using the shaking table at University of Buffalo. Moderate (peak ground acceleration PGA 0.2g) and severe episodes (PGA=0.3g) were used to verify the seismic behavior and the efficiency of structure suffered damage near collapse (90%, based on a damage index normalized to a unit which means collapse), the conventionally retrofitted structure suffered less damage, in repairable range.

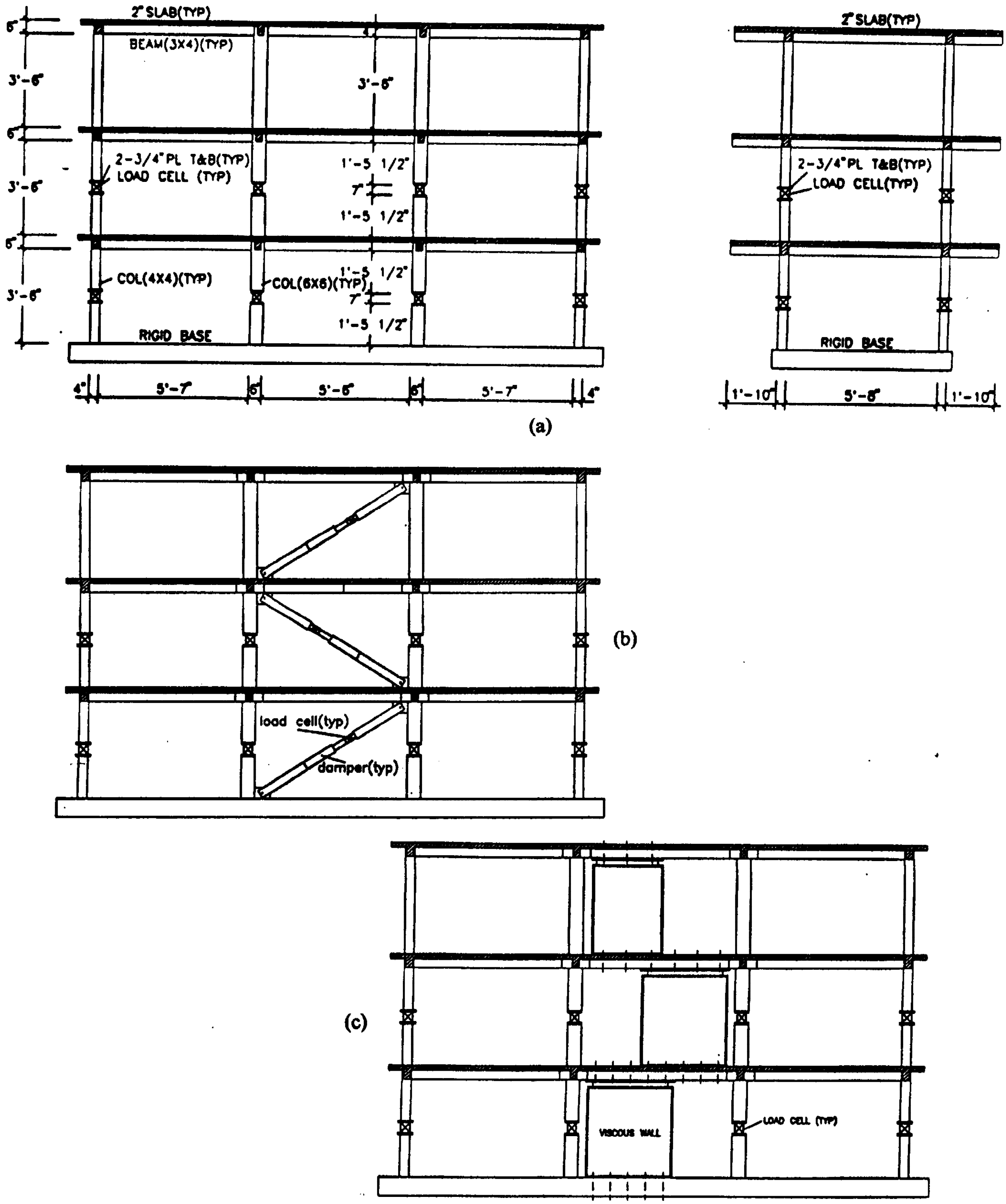

Fig. 1 - Test structure; (a) dimensions; (b) retrofit with braces; (c) retrofit with walls

Retrofit with Supplemental Damping Devices. The structure was retrofitted with additional damping devices (Taylor fluid viscous dampers, Sumitomo friction dampers, Tekton friction dampers and Sumitomo viscous damping walls) in the middle bay of each frame at all floors as shown in Figs.1(b) and 1(c). The braces of fluid viscous damper, friction dampers (of two types) were connected to the floor at base and top of column and transferred loads to the joint through the beams and a fillet joint. Each brace consists of an A36 L6x6x1/2 steel angle connected through 1/2 in diameter bolts to allow for a pinned connection at its ends. The loads were transferred through bending moment on beams and slabs between the two floors that the walls connected to. The damping devices had the following characteristics: (i) *Fluid Viscous Dampers:* installed in the brace were selected from the catalog of Taylor Devices Inc. Model 3x4, rated to 10,000 lbs (44.6 kN). The dampers was connected to the brace using a load cell with a capacity of 30,000 lbs. Detailed description of dampers properties are presented by Reinhorn et al. (1995) (ii) *Friction Dampers:.* Two types of friction dampers installed in the brace were specially designed by Tekton Company and Sumitomo Company respectively. The friction force of the dampers were calibrated to about 3.2 kips. The damper was connected to the brace using a load cell with a capacity of 30,000 lbs. Tekton Co. friction dampers were installed in the structure in a similar arrangement of Sumitomo friction dampers can be seen in Li et al.1995. (iii) *Viscous Damping Walls:* The viscous damping walls installed in the mid-bays of the frame were specially designed by Sumitomo Company, Japan. Larger viscous damping walls were installed at the first floor and smaller viscous damping walls were installed at the second and third floors. Details are presented in Reinhorn et al., 1995.

EXPERIMENTAL PROGRAM

The study was performed using simulated ground motion of various levels of simulated historical earthquakes scaled to produce elastic and inelastic response in the structure. A total of 77 earthquake simulation tests were performed for the structure model with various damping devices and different configurations. The simulated ground motion included Taft N21E 1952, El-Centro S00E 1940, Hachinohe 1964, Pacoima Dam S16E 1971, and Mexico City N90E 1985. The simulated requirements for a 1:3 scale structure using artificial mass simulation dictated a reduction of the time interval for the horizontal accelerogram of $1:\sqrt{3}$. The periods and mode shapes were determined through white noise excitations experiments (PGA 0.025g, 0.1g and 0.15g) of narrow band (0-25) before and after each severe shaking (Reinhorn et al. 1995, Li et al. 1995). The equivalent modal damping was estimated for a mode k according to Lobo et al. (1993)

The approximated values calculated according to the above and the experimental values are listed in Table 1

. Table 1 - Dynamic characteristics of tested structure

Ground motion (1)	PGA(g's) (2)	Damping (% of critical): Low amplitude testing (3)	Damping (% of critical): Strong motion testing (4)	Damping (% of critical): Approximated analytically[3] (5)	Fundamental period / Frequency; (second / Hz): Low amplitude testing [1] (6)	Fundamental period / Frequency; (second / Hz): Strong motion testing (7)
Without dampers						
El-Centro S00E	0.3	3	6	6	0.62 / 1.61	0.76 / 1.31
Taft N21E	0.2	3	5	5	0.62 / 1.61	0.76 / 1.31
With fluid dampers (Taylor)						
El-Centro S00E	0.3	16	28	26	0.53 / 1.88	0.62 / 1.61
Taft N21E	0.2	16	26	25	0.50 / 2.00	0.55 /1.81
With friction dampers (Tekton (Sumitomo))						
El-Centro S00E	0.3	7 (7)	25 (23)	28 (28)	0.25/4.00 (0.31/3.22)	0.29/3.49 (0.42 / 2.38)
Taft N21E	0.2	7 (7)	26 (26)	22 (22)	0.25/4.00 (0.31/3.22)	0.29/3.49 (0.50 / 2.00)
With Vicous Damping Walls (Sumitomo)						
El-Centro S00E	0.3	50	46	44	0.25 / 4.08	0.27 / 3.70
Taft N21E	0.2	49	47	44	0.25 / 4.08	0.27 / 3.70

[1] Low amplitude testing - white noise testing before and after simulated ground motion testing.
[2] Strong motion testing - simulated ground motion testing indicated in column (2).
[3] Approximated analytically - according to Lobo et al., 1993.

SEISMIC RESPONSE OF SHAKING TABLE TESTS

The peak response at various levels of shaking is summarized in Fig.2. It should be noted that while the deformations are substantially reduced, the total base shear is only minimally influenced. However, while the total base shear is increased, the maximum column shear force is somewhat reduced (see Fig. 2(c)). The forces in the structural components are shown in Fig. 3. The columns develop a maximum shear of only 14.0 kips with dampers, vs. 19 kips without dampers. The story drift is reduced from 1.45% to 0.83%. The energy is dissipated by the fluid or friction dampers, without much demand on the structural columns. It should be noted that while the maximum damper shear is 7.0 kips, the total base shear is only 15 kips which indicated that the maximum and the column shear are close to a 90° phase and do not influence the total peak responses simultaneously. The internal energy is redistributed such that 80% to 90% is taken by the supplemental dampers and dissipated, while hysteretic energy dissipation demand is reduced 85% to 95% in presence of dampers. The reduction of the demand for hysteretic energy dissipation is particularly important since it is preventing further deterioration of columns.. The experimental study shows: (i) For the structure with *fluid viscous* dampers, the experiment indicates that the dampers show a small stiffness increase (depending on the intensity of the earthquake) and control deformation through damping. However, the forces transmitted to the foundation are only minimally reduced and in some cases minimally increased. The main benefit of the dampers in such inelastic structures consists in transferring of the needs for energy dissipation from the original columns to the dampers, while controlling the lateral drifts and deformations. (ii) For the structures with *friction* dampers, the initial stiffness of the structure is increased The energy dissipation capacity increase with the increase of intensity of the earthquake and the period of the structure varies with the intensity of the earthquake which will prevent possible resonance. (iii) The experiment indicates that the structure with viscous damping walls show significant stiffness increase and reduce deformation through both damping and stiffness. The forces transmitted to the foundation and the structure's accelerations are increased substantially. The benefit of moment reduction in columns is counteracted, however, by increase of column axial forces. The foundation has to be designed considering the increase of base shear force due to added viscous damping walls.

EVALUATION OF INELASTIC RESPONSE OF DAMPED STRUCTURES

Response of an inelastic system The response of an inelastic system can be determined using the monotonic capacity diagram and the inelastic "composite response spectra" defined below. The response is obtained by intersecting the capacity diagram with the composite response spectra diagram.

Capacity of inelastic structures Structural systems composed of multiple components with inelastic characteristics can also be represented by a force-displacement relation. This relation can be obtained by applying monotonically increasing loads, distributed similarly with the inertial forces that will lead finally to structural failure (push-over analysis). If the structure is assumed elastic with well-defined mode shapes ϕ_j, (mass normalized, such that $\phi_j^T \mathbf{M} \phi_j = 1$, where T represents transposition and **M** is the mass matrix), the modal base shear (BS = Q(u)), indicative to the structure's resistance, and the displacement response are:

$$BS_j = Q_{Bj} = \Gamma_j^2\, S_a\left(\omega_j \xi_j\right) = Q\,(u) \quad \text{and} \qquad u_{ij} = \phi_{ij} \Gamma_j\, S_d\left(\omega_j \xi_j\right) \tag{1}$$

where, $\Gamma_j = \phi_j^T\ \mathbf{M}\ \mathbf{r}$ is the modal participation factor and **r** is a vector of units $(\mathbf{r}^T = \{1,1,...1\})$. To obtain the force-displacement capacity relation, Q(u), a monotonic inelastic analysis should be performed using increasing base shear, BS_j, and a force distribution, $F_i = m_i \phi_{ij} (BS_j / \Gamma_j)$. Simultaneously, the modal characteristics Γ_j and ϕ_{ij} *should be adjusted during the analysis as the system yields and the restoring and dynamic properties change.* The updated dynamic properties can be obtained through appropriate eigenvalue analyses. For m.d.o.f. with multiple mode contribution the capacity diagram is modified according to the contribution of higher modes from ratios of participation factors, γ, mode shapes, f, and spectra, s (Reinhorn et al, 1996):

$$Q^* = Q/\Gamma_1^2 \bullet \underset{j=1}{\overset{M}{\mathrm{srss}}}\,(\gamma_j^2 s_{aj}); \qquad u_i^* = u_i / \phi_{i1} \Gamma_1 \bullet \underset{j=1}{\overset{M}{\mathrm{srss}}}\,(f_{ij} \gamma_j s_{dj}) \tag{2}$$

An approximation of the spectral ratios can be made successfully using a building code approach. This type of incremental monotonic analysis with adjustment of loads is defined here-in as the "adaptable push-over" technique. This approach was integrated in an inelastic analysis computational platform, IDARC2D, Ver. 4.0 (Valles et al., 1996). Fig 4 shows the monotonic capacity of structure before and after retrofit.

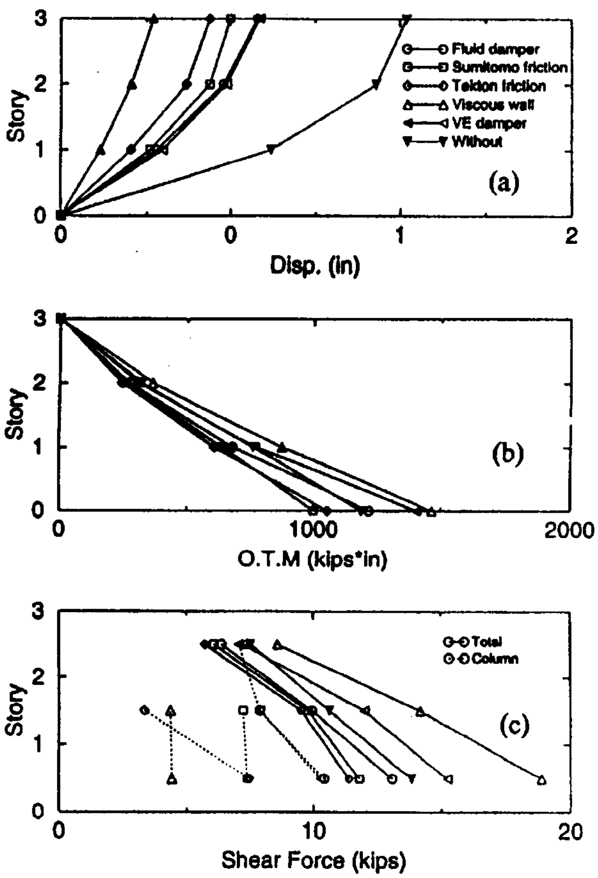

Fig.2 - Peak response during seismic test
Taft earthquake pga=0.2g (a) displacements
(b) overturning moments; (c) shear forces

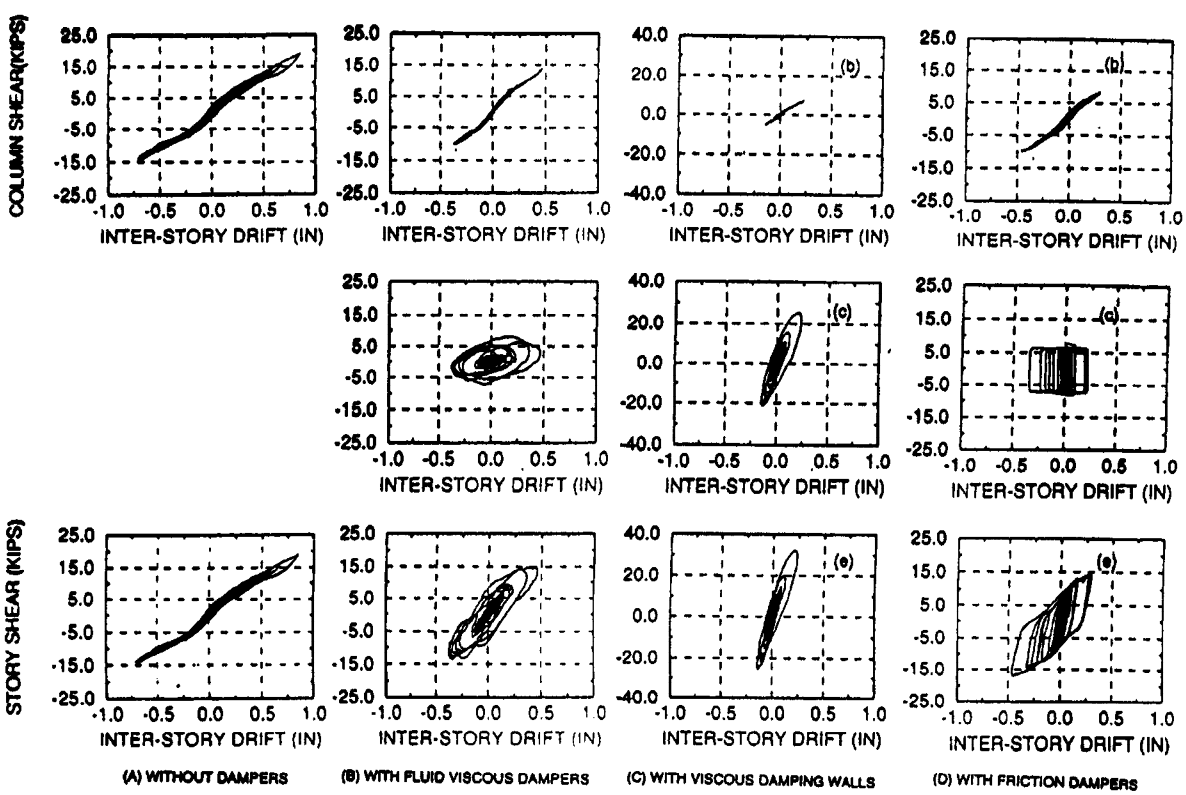

Fig. 3 - Internal force distribution in columns and dampers

Definition of composite response spectra. The maximum response of a family of single-degree-of-freedom (s.d.o.f.) systems to a well defined ground motion $\ddot{u}_g(t)$ is the well known response spectrum $S(\omega_o\ \xi_o)$ function. The response spectrum can therefore be obtained for displacement, $S_d(\omega_o\ \xi_o) = \max(u)$, for absolute acceleration, $S_a(\omega_o\ \xi_o) = \max(\ddot{u} + \ddot{u}_g)$. A combination of the response spectra for the same system characteristics $(\omega_o\ \xi_o)$ will form a new function defined here as the "composite spectra," as shown in Fig. 5. The composite spectra provide simultaneous information on response displacement, S_d, and response acceleration, S_a. With a simple dimensional transformation the composite spectrum may be adjusted to provide a direct relation between displacement and force spectra.

EVALUATION OF RETROFITTED STRUCTURE WITH COMPOSITE SPECTRA

The effect of rehabilitation on the response displacements and accelerations can be obtained by intersecting the capacity curves, by the demand composite spectra, representing the seismic motion as given by following relations:

$$\left[\mathbf{M}\left(\ddot{u}+\ddot{u}_g\right)+\mathbf{C}\dot{u}\right]_{max} = -\mathbf{K}u_{max} = Q\,(u) \quad \text{or} \quad \mathbf{M}S_{\ddot{u}}\left(T_0,x_0\right) = \mathbf{K}S_u\left(T_0,x_0\right) = Q\,(u) \tag{4}$$

where on the left hand side shows the demand (given by the composite spectra) and, on the right, the capacity.

Response with Supplemental Viscoelastic (Fluid or Solid) Damping Devices. Assuming that the damping force linearized model for supplemental viscoelastic (fluid or solid) dampers (Kelvin model) is: $F_d = \Delta K u + \Delta C \dot{u}$, when added to Eq.5 becomes:

$$\left(\mathrm{M}\left(\ddot{u}+\ddot{u}_g\right)+(\mathrm{C}+\Delta\mathrm{C})\dot{u}\right)_{max} = (\mathrm{K}+\Delta\mathrm{K})u_{max} = Q\,(u) \tag{6}$$

which indicates a change of slope in the stiffness-mass line to (K+ΔK)/W and a shift in the original spectral line from ξ_0 to $\xi_0 + \Delta\xi$ characteristics to the increase from C to C + ΔC.

Response with Supplemental Friction Devices. The friction in damper force can be represented by:

$$\left[\mathbf{M}\left(\ddot{u}+\ddot{u}_g\right)+\mathbf{C}\dot{u}\right]_{max} = -(\mathbf{K}+\Delta\mathbf{K})u_{max}, \quad for\ |F_D| \le \mu_{break-away}N$$

$$\left[\mathbf{M}\left(\ddot{u}+\ddot{u}_g\right)+\mathbf{C}\dot{u}\right]_{max} = -\left(N\mu_{min}+\Delta\mathbf{K}u_{max}\right), \quad for\ \mu_{min}N \le |F_D| \le \mu_{break-away}N$$

which indicates a change of initial slope in the stiffness/mass line in Fig. 4(b) and 5(b) to (K+ΔK)/W and a constant strength increase after slip. A shift in the original spectral line from ξ_0 to $\xi_0 + \Delta\xi$ characteristics to the increase from C to C + ΔC occurs.

The hysteretic energy dissipation can be interpreted as an increase in the "viscous" damping. In such case the response is obtained at the intersection of the elastic strength function Q(u) with the composite spectral lines for an increased damping ratio $\xi_2 = \xi_1 + \Delta\xi$. The equivalent damping increase can be estimated (see Reinhorn et al., 1995a). The equivalent damping increase was measured from experiments using the equivalent frequency response for the structure (see Table 1). The intersection of the composite spectra and the strength capacity function, Q(u) in Fig.5 are very close to the experimental results. This indicates that the approach can determine the response of forces and displacements with an acceptable approximation The dampers influences are as follows: (i) *Influence of Damping Increase*. If the damping devices have only damping characteristics, without or with minimal stiffness increase, the structure resistance (capacity) remains as before retrofit (see Fig. 5a). The displacement response is reduced primarily with some reduction of base shear. Devices which can control the damping increase without stiffening effects, such as fluid viscous devices specially designed, can produce such effect. (ii) *Influence of Stiffening due to Supplemental Dampers* The dampers have a substantial contribution to stiffening either in a "static" form, or "dynamic" form (see also Fig. 5a). The influence of stiffening contributes to a further reduction of displacement response and increase in the base shear demand (although minor). A substantial stiffening will increase the base shear demand substantially. (iii) *Influence of Friction Damping Increase* If the damping devices have only friction damping characteristics, neglecting the initial stiffness increase, the structure resistance (capacity) increases by a constant amount (see Fig 5b). The dampers have a substantial contribution to stiffening at initial stage (see Fig. 5b).

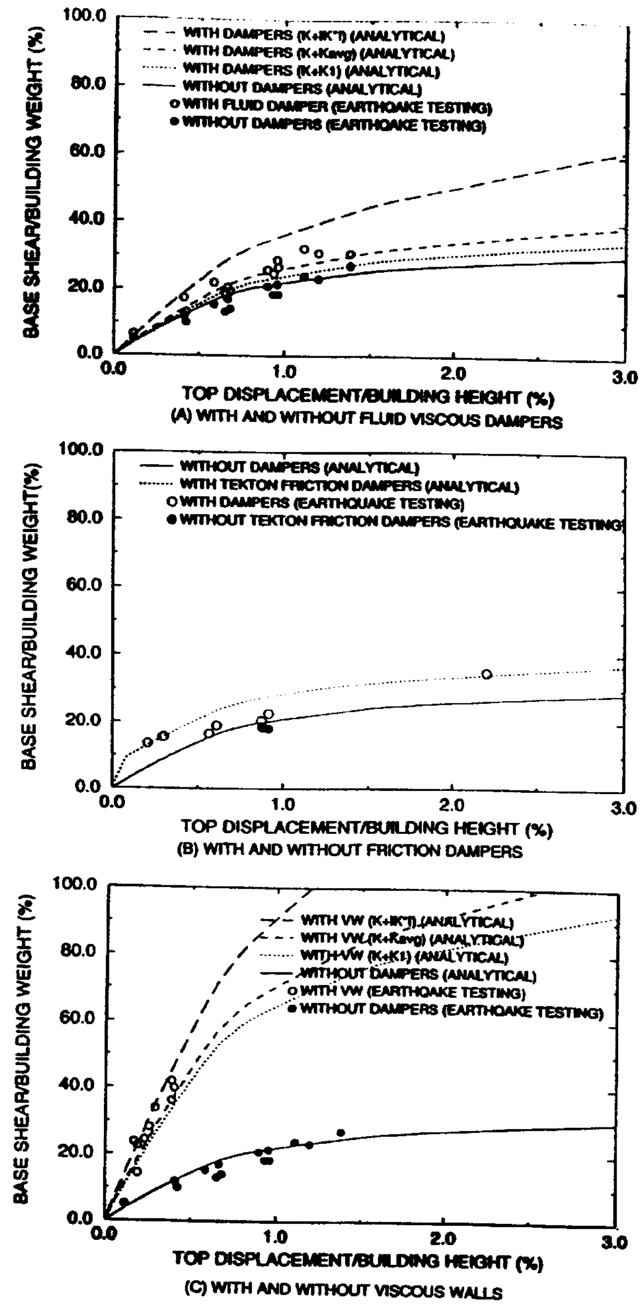

Fig. 4 - Capacity diagrams of structure without and with dampers

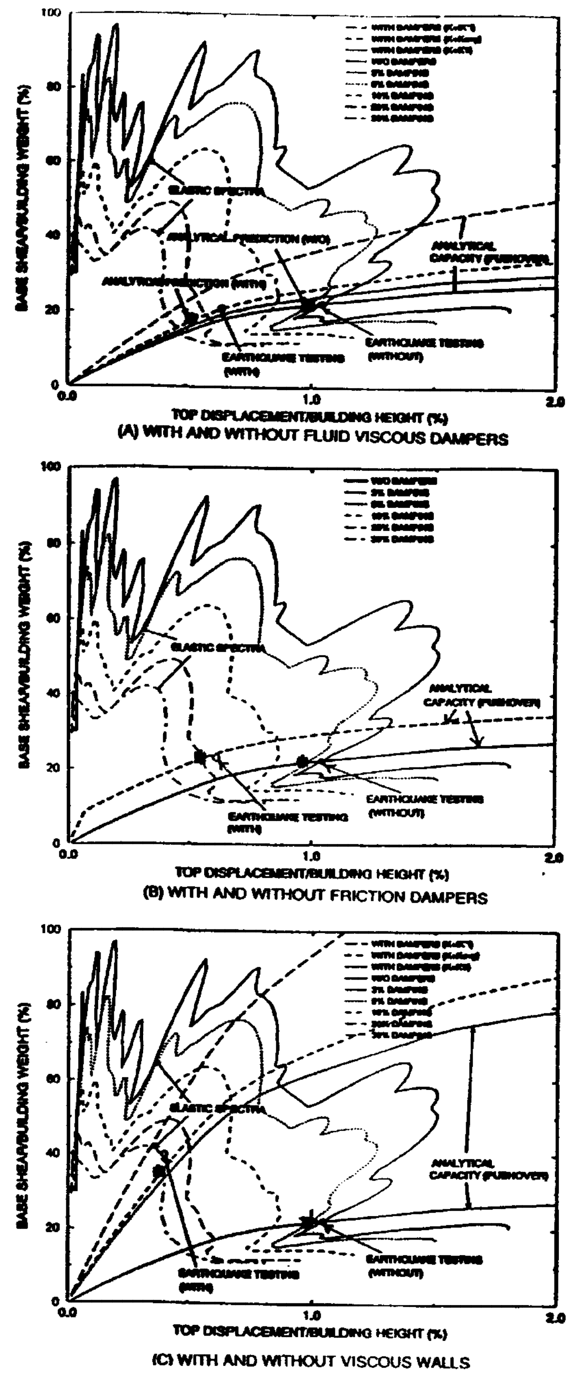

Fig 5 - Composite spectra and response demands

CONCLUDING REMARKS

The major findings from the comparison of the response shown in Fig 2 for all devices and from the simplified composite-spectrum approach are :

(a) The response related to displacements or drifts shows substantial reductions, from 30% to 50% for fluid viscous dampers ,30% to 60% for friction dampers, and 60% to 80% for viscous damping walls. This can be derived from the simplified composite spectra.

(b) For the structure with fluid viscous dampers, the response related to overturning moments (Fig. 2(b)), or story forces (Fig 2(c)) show very little change, some reduced and some increased. The composite spectrum approach indicates that the forces are increased minimally since the fluid viscous dampers have minimal stiffness increase. For the structure with friction dampers, similar conclusion is obtained. For the structure with viscous walls, the response related to overturning moments or total story forces is increased significantly due to substantial stiffening (see Figs 2 and 5(c)).

(c) The internal shear force (measured during the experiments) in the columns of the structure retrofitted with fluid viscous dampers are smaller than the forces in the unretrofitted structure, by a small amount (Figs. 2(c) and 3) Although the total shear force is reduced insignificantly, the forces in the columns alone are reduced more substantially 20% - 40% for fluid viscous dampers, 20% to 50% for friction dampers and 30% to 60% for viscous damping walls. This reduction is expected in view of the composite spectra and capacity curves as shown by the original curves in Fig. 5.

(d) The forces in columns and in the fluid viscous dampers reach their maximum out of phase. The maximum resultant of such forces is near zero column shears, at maximum dampers force. This indicates that the connections and columns can be designed independently for maximum forces resulting from either dampers or from internal column stresses. The total forces that are transmitted to the foundations (through suitable connections) will be therefore the larger between the damping forces or the column forces. The total forces from dampers are larger than those from columns, therefore, the forces from dampers play a key role in retrofit design of connections and foundations. The forces in the friction dampers reach their maximum before the forces in columns do, and then keep a constant value for larger deformation. The connections and columns should be designed for combination of maximum flexural moment and friction damper slip force and the same should be done for the foundation.

Note: The fluid viscous dampers, friction devices, viscoelastic devices and special viscous walls were sized to fit the desired retrofit scheme. Although the designs were similar, due to construction constrains the resulting devices were different in damping capacity and stiffening characteristics, such that their influence can not be directly compared. However, the trends of their influence can be evaluated and quantified using the composite spectrum approach.

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