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State University of New York at Buffalo

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NOTICE

Inelastic Response of Reinforced Concrete Structures with Viscoelastic Braces

by

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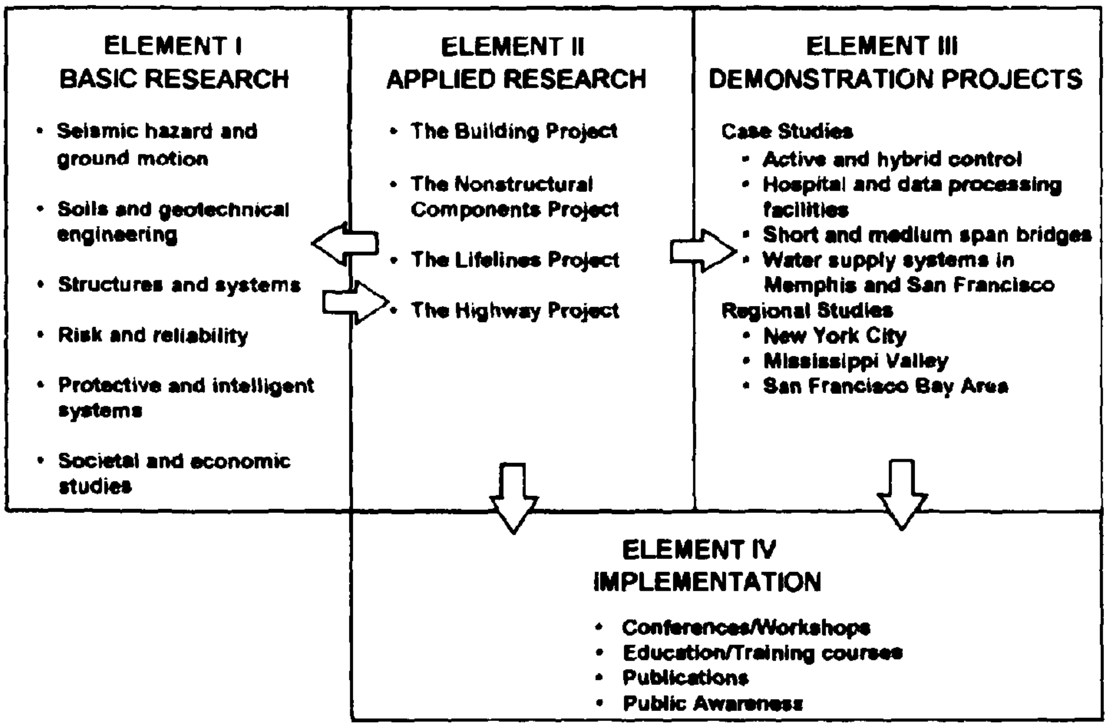
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

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Two of the short-term products of the **Building Project** will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The **protective and intelligent systems program** constitutes one of the important areas of research in the **Building Project**. Current tasks include the following:

1. Evaluate the performance of full-scale active bracing and active mass dampers already in place in terms of performance, power requirements, maintenance, reliability and cost.
2. Compare passive and active control strategies in terms of structural type, degree of effectiveness, cost and long-term reliability.
3. Perform fundamental studies of hybrid control.
4. Develop and test hybrid control systems.

Research at NCEER on seismic applications of viscoelastic dampers to retrofit nonductile concrete frames is being carried out as a collaborative effort among researchers at the University of Illinois, U.S. Army Corps of Engineers, the 3M Company, and the State University of New York at Buffalo. Presented in this report are results related to viscous and stiffness effects due to addition of the dampers on the inelastic response of reinforced concrete frames. Verification of these results was performed based on shaking table tests conducted on a one-third scaled model of a three-story lightly reinforced concrete frame.

ABSTRACT

The addition of viscoelastic braces in structures for vibration reduction was proposed and implemented in the past decade in metal models or full-scale structures. Viscoelastic braces provide energy dissipation, while the structures remains by-and-large elastic. In reinforced concrete structures, the seismic response is by-and-large inelastic, which is often accompanied by permanent deformations and damage. The addition of viscoelastic dampers can dissipate energy at the early stages of cracking of the concrete elements and reduce the development of damage. With proper selection of dampers, this damage can be substantially reduced or even eliminated. However the addition of viscoelastic dampers may stiffen the structure unnecessarily producing increased inertial forces and base shears when subjected to seismic motion. The quantification of the influence of viscous and elastic stiffness properties of dampers during the inelastic response of reinforced concrete structures is the subject of this investigation. Models for analysis of inelastic response with damage indexing for reinforced concrete structures that include viscoelastic braces are developed and calibrated using experimental data produced by shaking table tests. These models are then used to determine the variation of expected damage in the presence of damping and quantify the hysteretic energy dissipation along with the damping energy.

ACKNOWLEDGEMENTS

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SECTION 1

INTRODUCTION

The addition of viscoelastic braces in structures for vibration reduction was thoroughly investigated in the past decade using metal scaled models or full-scaled structures. While the viscoelastic braces provide energy dissipation through non-load bearing elements, the load bearing structure remains by-and-large elastic. Reinforced concrete structures are designed to resist earthquakes by dissipating the input energy transmitted to the structure through inelastic deformations of the load bearing components. The seismic response is therefore accompanied by permanent inelastic deformations and damage. Proper selection of additional viscoelastic dampers can contribute to the energy dissipation in the early stages of cracking and limit the development of damage or completely eliminate it.

Various damping devices were suggested for use in structures to limit damage to the load bearing structural elements. Of these devices the two more popularly used are: (i) the direct shear seismic damper (DSSD) (Mahmoodi, 1969) and (ii) the steel plate added damping and stiffness (ADAS) damper Scholl ,1990). Mahmoodi (1969) showed that viscoelastic dampers at appropriate locations within the structure are effective in reducing the vibrations in tall buildings. These dampers have proved successful as adequate damping devices with stable engineering properties with regards to aging in the World Trade Center Buildings (New York) and the Columbia Center Building (Seattle), (Keel et al. 1986). A number of experimental studies have also been conducted to show the effectiveness of these dampers in reducing the story displacements, accelerations, shear forces, and damage to structures. Lin et al. (1991) tested a 1/4-scale three story steel framed model building, Chang et al. (1992) tested a 2/5-scale five story steel framed model building, to name a few. These studies show conclusive evidence that mechanical dampers, acting as non-load bearing elements, effectively damp the vibrations in buildings caused by wind, seismic, or other forms of transient lateral loadings. These dampers effectively dissipate the input energy to the structure by increasing

SECTION 2

INELASTIC DAMAGE ANALYSIS OF REINFORCED CONCRETE STRUCTURES WITH VISCOELASTIC BRACES

Inelastic analysis of reinforced concrete structures to seismic or wind loadings has been the subject of several previous developments for planar systems, such as DRAIN-2D (Kanaan and Powell, 1973), SARCF (Rodriguez-Gomez et al., 1990) and a family of analytical developments, IDARC (Park et al., 1987 and Kunnath et al., 1992). A recent development of the two dimensional version of IDARC (Kunnath et al., 1992) was extended to a full three dimensional analysis of reinforced concrete structures including space torsional behavior and biaxial bending interaction in the structural elements, IDARC-3D (Lobo et al., 1992). The salient features of the above analytical model for reinforced concrete structures are:

(i) An extensive hysteretic model governed by several parameters to simulate inelastic behavior of beams, columns, shear-walls, and braces.

(ii) A distributed flexibility model that accounts for the nonsymmetric distribution of plasticity along the members.

(iii) A variety of loading conditions including simultaneous action of static, cyclic, and random forces and base excitations.

(iv) Evaluation of damage progression and energy balances. The hysteretic model has the capability of reproducing a variety of hysteretic curves by selection of three independent parameters which control stiffness degradation, strength deterioration and pinching usually generated by bond slip of the reinforcement during cracking (Kunnath et al., 1992).

The above analytical platform was verified using extensive simulations and comparisons with experimental data from laboratory tests of components and structures (Kunnath et al., 1992, Bracci et al., 1992a, 1992b, 1992c, and El-Attar et al., 1991). The simulations obtained are suitable to either duplicate or predict actual measured behavior. Thus the analytical model IDARC-3D was chosen as a base to develop the new models for analysis of reinforced concrete buildings with viscoelastic dampers.

2.1 Numerical Solution for Dynamic Analysis

The inelastic analysis of structures with viscoelastic braces is done using numerical models and direct integration techniques. The fundamental equation of motion for numerical integration is expressed in matrix form as:

$$\mathbf{M\ddot{u}} + \mathbf{C\dot{u}} + \mathbf{Ku} = -\mathbf{MI}\ddot{u}_g + \mathbf{F}_w \tag{2.1}$$

where $\mathbf{M}$ = mass matrix, $\mathbf{C}$ = mass proportional damping matrix, $\mathbf{K}$ = instantaneous overall stiffness matrix, $\mathbf{I}$ = vector of ones or zeros indicating excitation in any degree of freedom. $\mathbf{u}$, $\mathbf{\dot{u}}$, and $\mathbf{\ddot{u}}$ = displacement, velocity, and acceleration vectors, respectively, $\ddot{u}_g$ = ground acceleration vector, and $\mathbf{F}_w$ = wind forces. Equation (2.1) can be solved by a linear step-by-step dynamic analysis procedure using the Newmark Beta constant average acceleration method, which gives an unconditionally stable solution. It can by expressed in a generalized form in terms of the incremental forces and displacements after the inclusion of the additional stiffness and damping from the viscoelastic braces as:

$$\Delta\mathbf{F}^* = \mathbf{K}^*\Delta\mathbf{u} \tag{2.2}$$

where

$$\Delta\mathbf{F}^* = -\mathbf{MI}\Delta\ddot{u}_g + \Delta\mathbf{F}_w + \mathbf{M}[\frac{4}{\Delta t}\mathbf{\dot{u}} + 2\mathbf{\ddot{u}}] + 2[\mathbf{C} + \Delta\mathbf{C}]\mathbf{\dot{u}}$$

$$\mathbf{K}^* = \left[\frac{4}{\Delta t^2}\mathbf{M} + \frac{2}{\Delta t}[\mathbf{C} + \Delta\mathbf{C}] + [\mathbf{K} + \Delta\mathbf{K}]\right]$$

$$\Delta\mathbf{\ddot{u}}_g = \mathbf{\ddot{u}}_g^{(t)} - \mathbf{\ddot{u}}_g^{(t-1)};\ \Delta\mathbf{F}_w = \mathbf{F}_w^{(t)} - \mathbf{F}_w^{(t-1)}$$

where $\Delta\mathbf{u}$ = the vector of incremental displacements, $\Delta\ddot{\mathbf{u}}_g$ = the increment of ground acceleration, ΔF_w = the vector of incremental wind forces, $\dot{\mathbf{u}}$ and $\ddot{\mathbf{u}}$ = the velocity and acceleration at the beginning of the time step, and $\Delta\mathbf{K}$ and $\Delta\mathbf{C}$ = the matrices corresponding to the additional stiffness and additional damping provided by the viscoelastic braces. These matrices can be obtained by linearization of frequency dependent viscoelastic models models of complex formulation as shown in the following.

The global equivalent viscous damping in reinforced concrete buildings seems to play an important role for the elastic behavior, usually in the non-damaged state. When structures enter the inelastic range much energy is dissipated by hysteretic behavior and therefore the influence of this viscous damping effects to the total apparent damping diminishes. A proportional damping representation used in IDARC-3D, (Lobo, 1993), accounts for the global viscous damping and produces acceptable results both in the elastic as well as the inelastic range. When more control on the damping in the various modes is required, in the elastic range, the proportional damping matrix can be expressed as:

$$\mathbf{C} = \alpha_0\mathbf{M}(\mathbf{M}^{-1}\mathbf{K})^0 + \alpha_1\mathbf{M}(\mathbf{M}^{-1}\mathbf{K})^1 + \alpha_2\mathbf{M}(\mathbf{M}^{-1}\mathbf{K})^2 + \ldots \qquad (2.3)$$

where $\alpha_0, \alpha_1, \alpha_2$ are proportionally factors that lead to real modes and frequencies. The first two terms correspond to the mass and stiffness proportional damping respectively. Using an effective critical damping ratio, ξ_s, corresponding to mass or stiffness proportional damping matrix, could yield adequate results if the number of dampers were located uniformly throughout the structure. This procedure provides, however, only an approximation of the damping produced by addition of supplemental damping such as provided by viscoelastic braces, which is non proportional. In the further modeling it is assumed that, only the lateral degrees of freedom are affected, without influencing the damping to the rotational degrees of freedom. Non proportional viscous damping for multi degree of freedom systems, produces free vibration response of the structure, that is exponentially damped at the same frequency, but at different phase angles, resulting in non stationary modes. This is well represented by complex eigen values and eigen vectors. Thus the use of an equivalent critical damping ratio ξ to represent damping is only an approximation limited to structures with evenly distributed supplemental damping.

Various attempts were made to emphasize more realistically the influence of added damping. Instead of the equivalent damping approach, Caravani and Thomson (1974) suggested to define a damping matrix that included the influence of story damping in an implicit way. Modeling of viscoelastic braces was successfully attempted by Hanson et al., (1987). Su and Hanson (1990) modeled the structural and hysteretic damping of ADAS devices using the Ramberg - Osgood hysteresis model in DRAIN-2D (Kanaan and Powell, 1973). Pall et.al (1982) modelled the response of structures with diagonal cross friction bracing using a non symmetric bilinear model, also using DRAIN-2D. Recently Liang and Lee (1991) have expressed the damping matrix similar to the previous authors, however the influence of modal frequencies and structural brace configurations was also included. The model of viscoelastic braces used in IDARC-3D and detailed in the next section is an extension of the models proposed by Ashour et al. (1987) and Liang et al. (1991).

2.2 Determination of Damper Properties

Viscoelastic damping material as the name implies, has two components, the viscous part or energy absorbing part, and the elastic part or energy restoring part. Figure 2.1 shows a typical structural model with supplemental viscoelastic braces which serves as a test case for modeling and analysis using IDARC-3D. For a single viscoelastic damper (see Fig. 2-2) subjected to a steady state harmonic excitation, the damping force can be described by the complex relation:

$$\mathbf{F}(\omega)_d = [(k_s(\omega) + ik_l(\omega))]x(\omega) \tag{2.4}$$

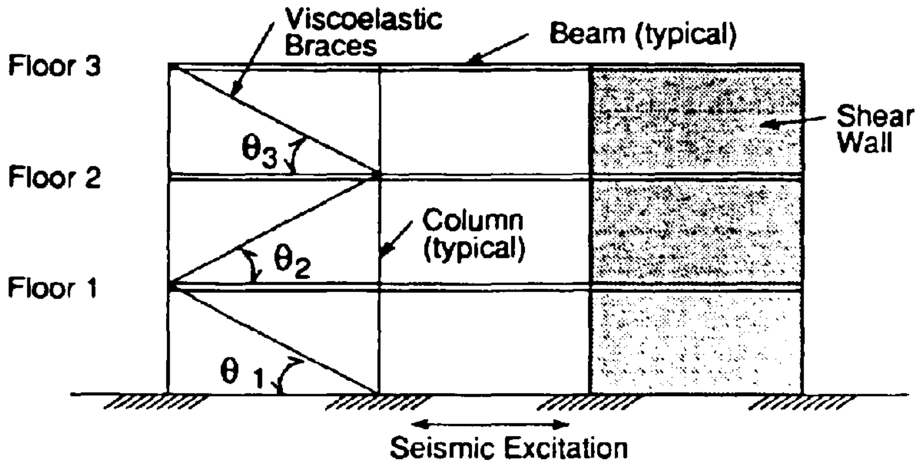


FIGURE 2-1 Structural Model With Viscoelastic Braces

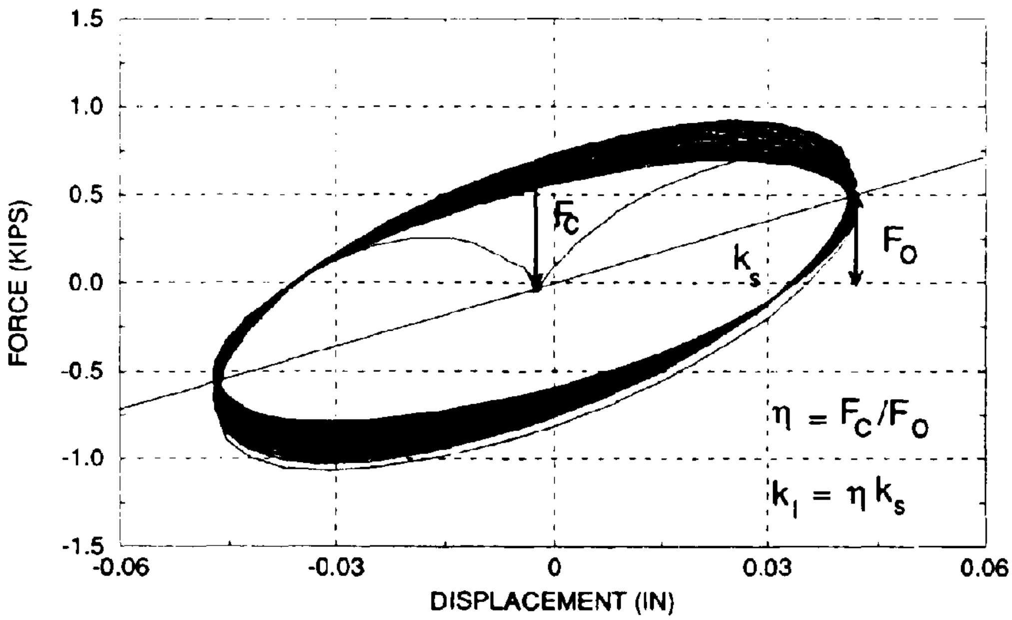


FIGURE 2-2 Typical Force Deformation Loop at 3Hz (Experimental)

where $\mathbf{F}_d$ = the force in the brace, $k_s(\omega)$ = the shear storage stiffness, $k_l(\omega)$ = the shear loss stiffness, and x = the displacement in the damper. Since the damping coefficient force formulation is dependent on frequency (Liang, 1991), Eq. (2.4) can be generalized as:

$$\mathbf{F}_d(\omega) = k_s(\omega)(1 + i\eta(\omega))x(\omega) \tag{2.5}$$

where $\eta(\omega)$ = the loss factor and defined as the ratio of $k_l(\omega)/k_s(\omega)$. For a process governed by a narrow band excitation the coefficients $k_s(\omega)$ and $\eta(\omega)$ may be considered constant.

With this assumption and after some, manipulations of Eq. (2.5), using the definition of viscous damping coefficient, c, as:

$$c = \frac{\eta k_s}{\omega} \tag{2.6}$$

The force in the damper can be defined:

$$\mathbf{F}_d(\omega) = (k_s + ic\omega)x(\omega) \tag{2.7}$$

An inverse Fourier transform applied to (2.7) produces

$$\mathbf{F}_d(t) = k_s x(t) + c\dot{x}(t) \tag{2.8}$$

which indicates that the shear storage stiffness ($k_l = \eta k_s$) influences the stiffness of the brace and the shear loss stiffness (k_l) influences the damping of the brace. Although the structure shows vibrations in various modes, the first mode of vibration is dominant and therefore the properties of the damper $k_s(\omega)$ and $\eta(\omega)$ can be selected based on the significant mode without appreciable loss of accuracy.

If the shear storage modulus (G') is known, the stiffness k_s can be obtained directly according to the relation

$$k_s = G'A/t \tag{2.9}$$

where A is the total shear area of the viscoelastic material and t is the thickness of the viscoelastic material. Similarly k_l can be obtained as

$$k_l = G''A/t \tag{2.10}$$

when the shear loss modulus (G'') is known. The same stiffnesses k_s and k_l can also be obtained from the cyclic test hysteresis results as shown in Fig. 2-2.

2.3 Influence of Individual Damper's Properties on the Structure Properties

The properties of each brace using identical damping devices are incorporated in the structural model as increments of the stiffness, ΔK, and of the damping, ΔC, matrices:

$$\Delta\mathbf{K} = k_s\mathbf{B} \text{ and } \Delta\mathbf{C} = c\mathbf{B} \tag{2.11}$$

where $\mathbf{B}$ is a non dimensional brace location matrix that takes the following form:

$$\mathbf{B} = \begin{bmatrix} N_j\cos^2\theta_j & -N_j\cos^2\theta_j & & \cdot & \cdot \\ -N_j\cos^2\theta_j & N_j\cos^2\theta_j + N_{j-1}\cos^2\theta_{j-1} & -N_{j-1}\cos^2\theta_{j-1} & \cdot & \cdot \\ & & \cdot & \cdot & \cdot \\ \cdot & \cdot & N_i\cos^2\theta_i & N_i\cos^2\theta_i + N_2\cos^2\theta_2 & -N_2\cos^2\theta_2 \\ & \cdot & \cdot & -N_2\cos^2\theta_2 & N_2\cos^2\theta_2 + N_1\cos^2\theta_1 \end{bmatrix} \tag{2.12}$$

where N_k = the number of dampers at the k-th story such that $N_k c$ is the total damping coefficient of all the dampers at k-th story and $\cos\theta$ = the inclination of each brace from the horizontal. For unequal dampers, the value of N_k may be a noninteger $\mathbf{B}$ can be therefore suitably modified to reflect a variable number of braces at each floor, the variable damper properties, and the inclination of the dampers.

The incremental matrices $\Delta\mathbf{K}$ and $\Delta\mathbf{C}$ are added to the dynamic equations of motion, Eq. (2.2), within IDARC-3D. The validity of the above formulation is verified with experimental data and used for further parametric analysis as described in the subsequent sections.

2.4 Determination of Damping Ratios

2.4.1 Equivalent Formulation of Damping Ratio

The contribution of identical viscoelastic devices to the critical damping in each mode can be obtained using modal characteristics as :

$$\xi_{\Delta i} = \frac{1}{2\omega_i}\frac{\Phi_i^T(\Delta\mathbf{C})\Phi_i}{\Phi_i^T\mathbf{M}\Phi_i} = \frac{c}{2\omega_i}\frac{\Phi_i^T\mathbf{B}\Phi_i}{\Phi_i^T\mathbf{M}\Phi_i} \tag{2.13}$$

where Φ_i is the i-th modal shape and ω_i is the i-th modal frequency. For very simple structures such as in Fig. 2-1 the i-th modal damping ratio can be obtained from Eq. (2.13) or as:

$$\xi_{\Delta i} = c \cdot \left[\mathbf{N}_1{*}\cos^2\theta_1\Phi_{i1}^2 + \sum_{j=2}^{J}\mathbf{N}_j\cos^2\theta_j(\Phi_{ij} - \Phi_{i(j-1)})^2\right] / \; 2\omega_i\Sigma m_j\Phi_{ij}^2 \tag{2.14}$$

where m_j is the j-th story mass and J is the total number of stories. Eq. (2.13) or (2.14) can be used in design process for estimating the required damping property, c, of a typical brace such that a desired supplemental modal damping ratio ξ_i can be obtained. The total damping can be further obtained including the contribution from the inherent viscous damping already existing in the structure as:

$$\xi_{TOTi} = \frac{\omega_i}{2}\frac{\Phi_i^T(C + \Delta C)\Phi_i}{\Phi_i^T(K + \Delta K)\Phi_i} \tag{2.15}$$

Equation (2.15) can be expressed in terms of the individual damping ratio contributions as:

$$\xi_{TOT_i} = \xi_{\Delta i} + \xi_{si}(1 - \alpha_i + \alpha_i^2 - \alpha_i^3 + \ldots) \tag{2.16}$$

Where ξ_{si} is the original structural modal damping ratio $\omega_i\,(\boldsymbol{\Phi}_i^T\mathbf{C}\boldsymbol{\Phi}_i)/2(\boldsymbol{\Phi}_i^T\mathbf{K}\boldsymbol{\Phi}_i)$ and α_i is $\boldsymbol{\Phi}_i^T\Delta\mathbf{K}\boldsymbol{\Phi}_i/\boldsymbol{\Phi}_i^T\mathbf{K}\boldsymbol{\Phi}_i$. Note that for a small stiffness increase $\Delta\mathbf{K}$ the resultant damping is the sum of the added damping and the original one.

2.4.2 Complex Formulation for Damping Ratio

The ξ_{TOT_i} computed by this process is only an approximate value of the critical damping ratio, because of the non proportional characteristics of the damping matrix. The natural frequencies ω_i and corresponding damping ratios ξ_{TOT_i} for each mode can be computed more accurately from the set of homogeneous equations (Frazer et. al. 1946) using the total complex damping C^*, and stiffness K^* matrices based on the state equation:

$$\begin{bmatrix} \mathbf{0} & \mathbf{M} \\ \mathbf{M} & \mathbf{C}^* \end{bmatrix}\begin{bmatrix} \ddot{\mathbf{u}} \\ \dot{\mathbf{u}} \end{bmatrix} + \begin{bmatrix} -\mathbf{M} & \mathbf{0} \\ \mathbf{0} & \mathbf{K}^* \end{bmatrix}\begin{bmatrix} \dot{\mathbf{u}} \\ \mathbf{u} \end{bmatrix} = \begin{bmatrix} \mathbf{0} \\ \mathbf{0} \end{bmatrix} \tag{2.17a}$$

or:

$$\mathbf{A}\dot{\mathbf{y}} + \mathbf{B}y = \mathbf{0} \tag{2.17b}$$

where

$$\mathbf{A} = \begin{bmatrix} \mathbf{0} & \mathbf{M} \\ \mathbf{M} & \mathbf{C}^* \end{bmatrix};\quad \mathbf{B} = \begin{bmatrix} -\mathbf{M} & \mathbf{0} \\ \mathbf{0} & \mathbf{K}^* \end{bmatrix};\quad \mathbf{y} = \begin{bmatrix} \dot{\mathbf{u}} \\ \mathbf{u} \end{bmatrix} \tag{2.17b}$$

The eign solution can therefore be obtained from:

$$\dot{\mathbf{y}} = -\lambda\mathbf{y} \tag{2.18}$$

or:

$$\mathbf{A}\mathbf{y} = \frac{1}{\lambda}\mathbf{B}\mathbf{y} \tag{2.19}$$

Equation (2.19) has complex roots that can be obtained as:

$$\lambda_i = \mu_i + i\nu_i \tag{2.20}$$

where *lamda*$_i$ are calculated from the characteristic equation;

$$\lambda_i^2 + 2\xi_i\omega_i\lambda_i + \omega_i^2 = 0 \tag{2.21}$$

that yields the characteristic values:

$$\mu_i = \xi_i\omega_i \tag{2.22a}$$

$$\nu_i = \omega_i\sqrt{1-\xi_i^2} \tag{2.22b}$$

The free vibration response is obtained from:

$$y_i(t) = e^{-\xi_i\omega_i t}\left[\frac{y_i(0) + y_i(0)\xi_i\omega_i}{\omega_d}\sin\omega_d t + y_i(0)\cos\omega_d t\right] \tag{2.23}$$

with the natural frequencies and the equivalent damping ratios for the respective modes computed as:

$$\omega_{d_i} = \nu_i \tag{2.24}$$

$$\omega_i = \sqrt{\mu_i^2 + \nu_i^2} \tag{2.25}$$

$$\xi_i = \frac{\mu_i}{\sqrt{\nu_i^2 + \mu_i^2}} \tag{2.26}$$

A comparison of the analytical predictions of equivalent damping ratios and of complex ratios to the values obtained in experiments are given in the next section.

SECTION 3

PERFORMANCE VERIFICATION OF ANALYTICAL MODEL IN 1:3 SCALE STRUCTURE TESTING

An experimental study of a 1:3 scale R/C frame structure retrofitted with viscoelastic braces using 3M[TM] materials was carried out at NCEER [Shen, Soong, Bracci/1993]. The purpose of this experimental study is

(i) To observe the performance of viscoelastic dampers

(ii) To validate the analytical (computational) model that make use of several simplified assumptions.

(iii) To determine the influence of dampers on the structural components and overall structural system.

The results of this study are used here to validate the analytical model described in the proceeding section.

3.1 Experimental Program

A one-third scale model of a three story lightly reinforced concrete frame building (Figs. 3.1, 3.2, and 3.3) was previously tested under simulated base motions using the shaking table in the Seismic Simulation Laboratory at the State University of New York at Buffalo (Bracci et al., 1992a and 1992b). The structure was tested using a series of simulated motions obtained from the scaled 1952 Taft earthquake, N21E component, normalized for peak ground accelerations (PGA) of 0.05g, 0.20g, and 0.30g representing minor, moderate, and severe ground motions. The structure was also tested with a uniform random noise (white noise) after each episode for identification purposes. The severe base motions induced large inter-story drifts and internal damage to the columns such that an incipient column-sidesway mechanism was apparent and leading towards a collapse situation

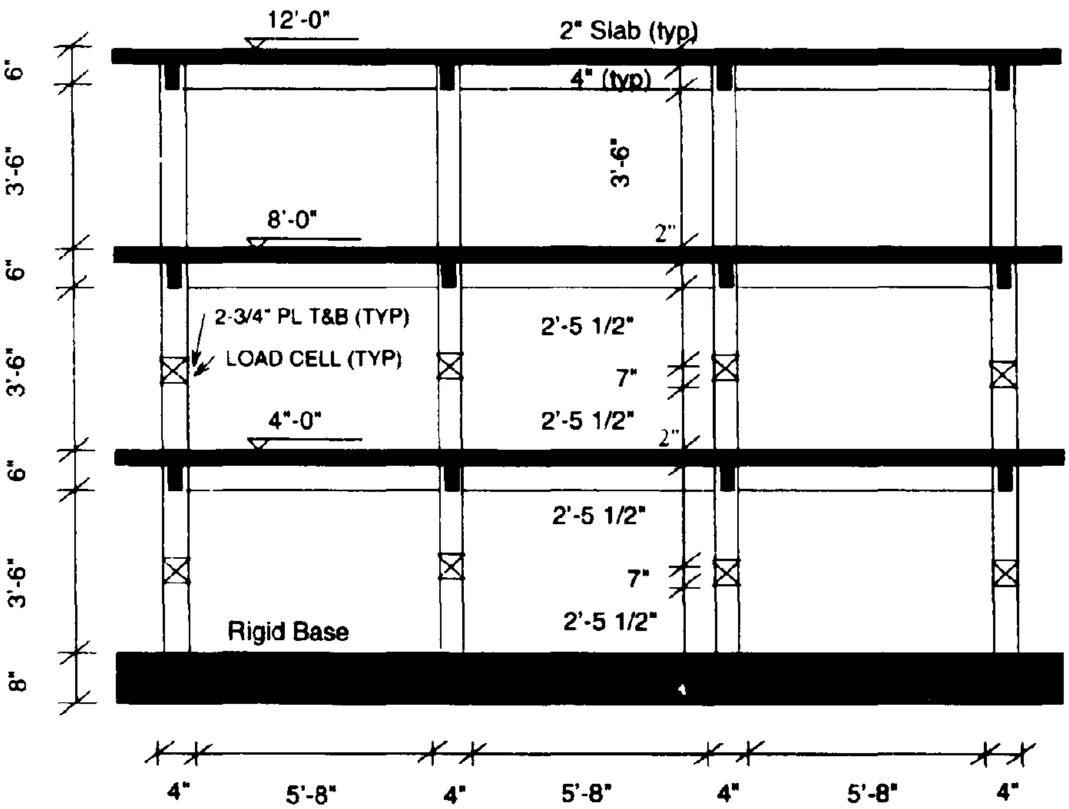


FIGURE 3-2 Front Elevation of Test Structure Before Conventional Retrofit

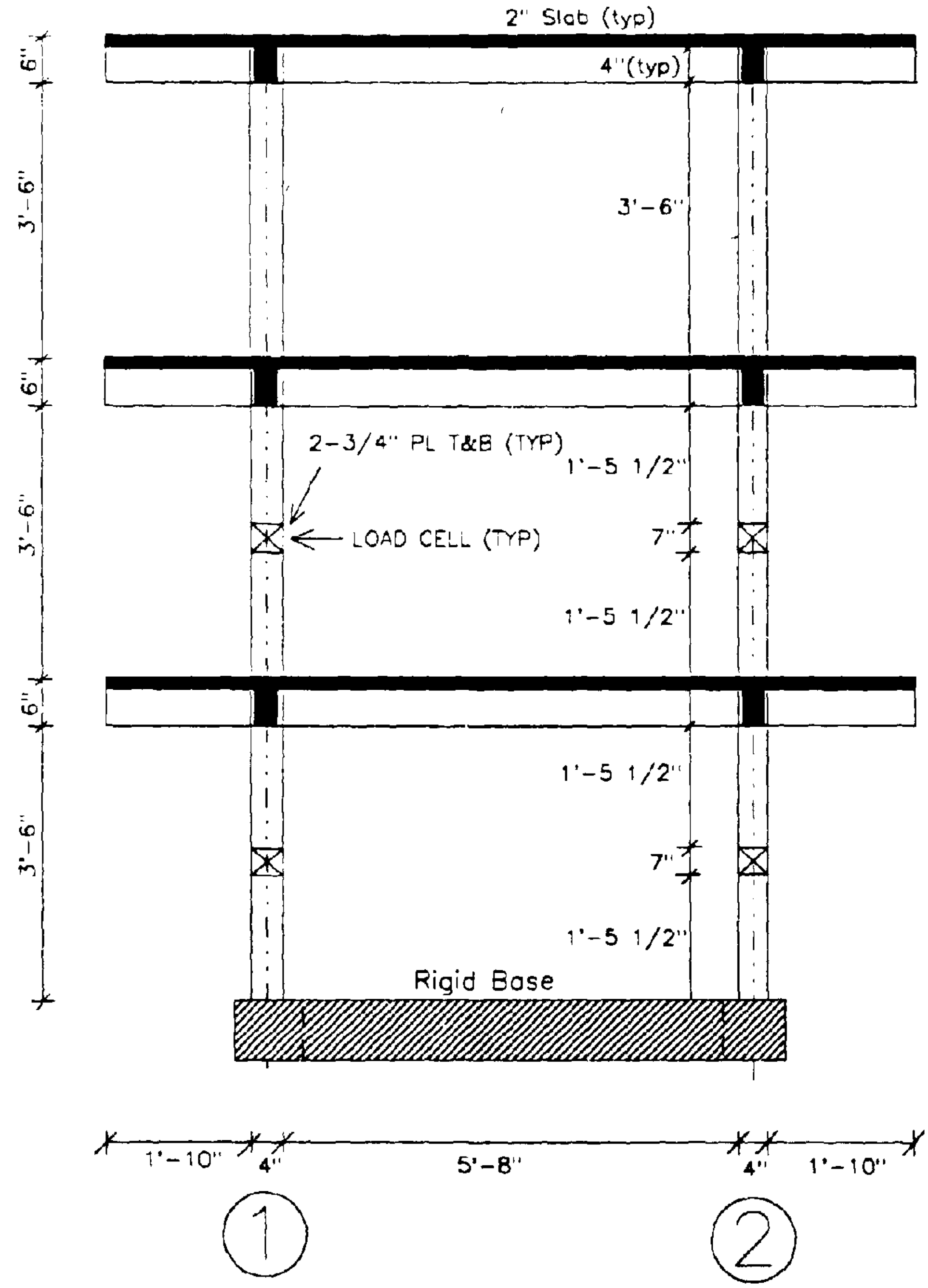
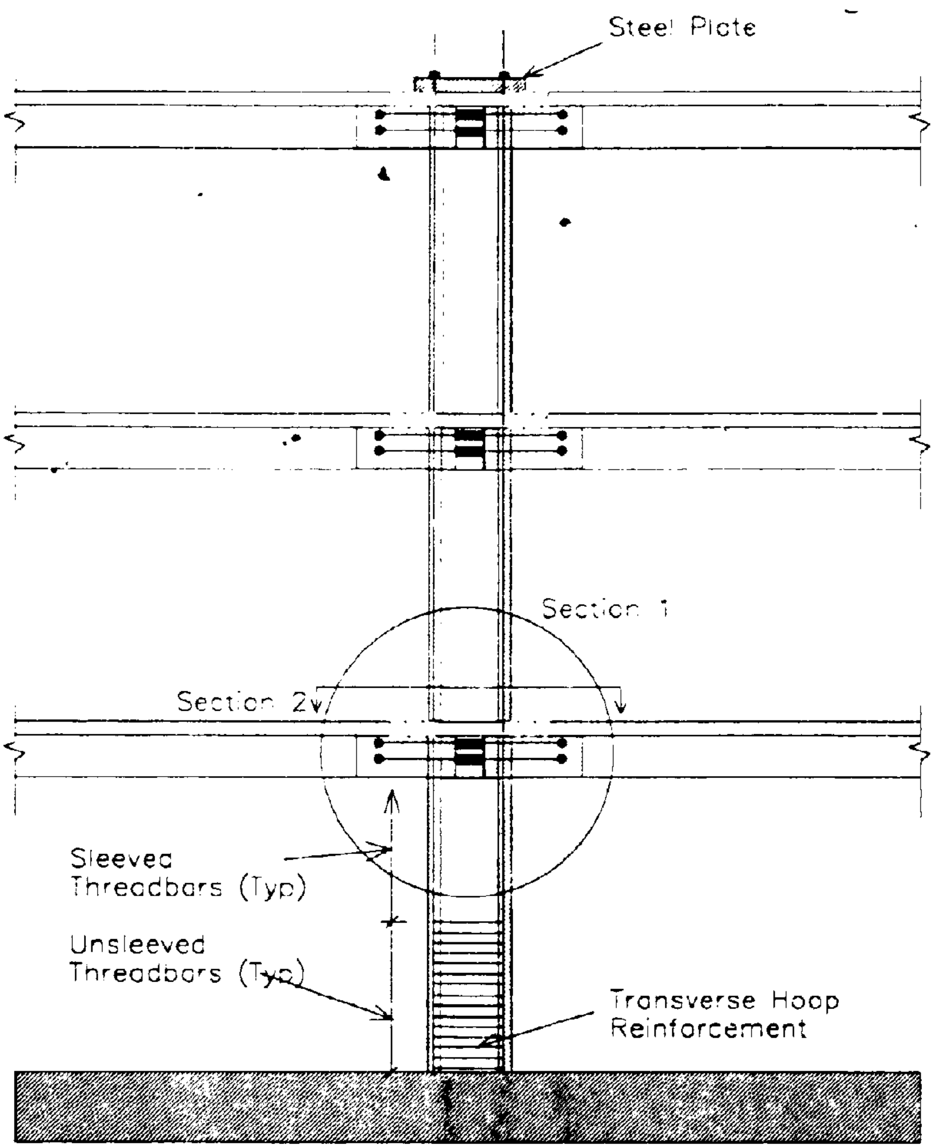
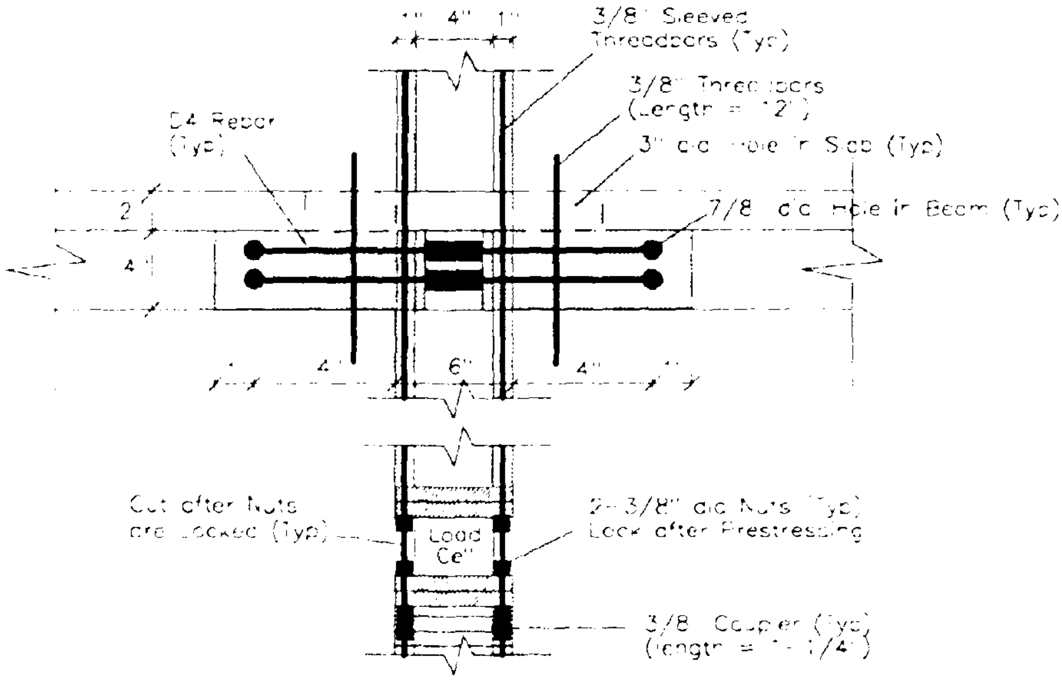


FIGURE 3-3 Side Elevation of Test Structure

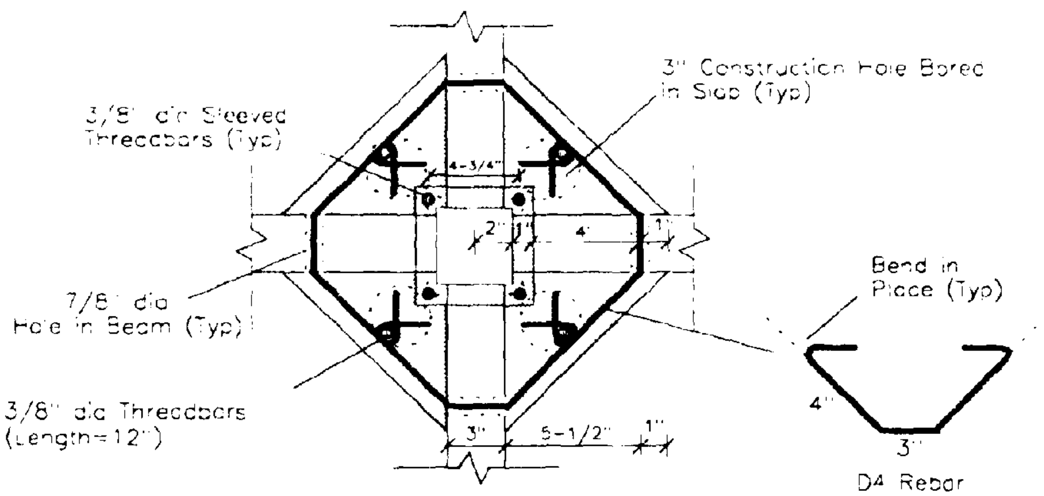


Elevation

FIGURE 3-4 Conventional Retrofit by Jacketing of Interior Columns



Section 1



Section 2

FIGURE 3-5 Details of Conventional Retrofit

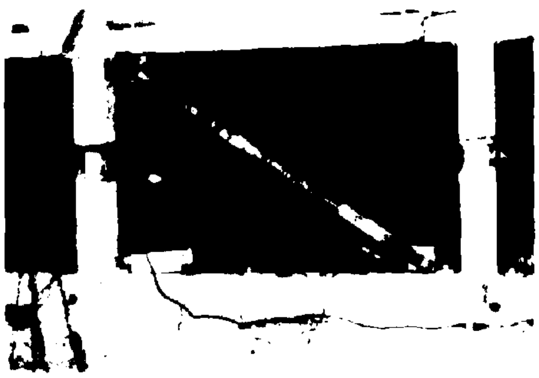
(Bracci et al., 1992b). Subsequently the damaged building was retrofitted conventionally (See Fig. 3.1) by strengthening the interior columns of the building using concrete jacketing, strengthening the beam-column joints with a reinforced concrete fillet, and post-tensioning the repaired columns to 20% of their ultimate axial strength as shown in Figs. 3.4, and 3.5, (Bracci et al., 1992c). The system was subsequently tested using the same motions as for the original building. The performance of the repaired structure was substantially improved producing only local damage in beams and slabs. However the complete beam-sidesway mechanism was not near full development, thus reducing the overall damage and collapse risk.

This damaged building served the objective for further experimental studies of retrofit using viscoelastic dampers of the direct shear type.

The building was retrofitted again by adding viscoelastic diagonal braces in the interior bay of each frame (see Fig. 3.6) and tested by Shen, Bracci, Soong and Reinhorn. For sake of completion the description of the test is repeated in here. The viscoelastic dampers made by 3M™ Company Minneapolis, MN, consisted of two pads of 3M™ manufactured viscoelastic material bonded between three steel plates and embedded in steel braces connected by steel brackets to the story slabs (see Fig. 3.7). The brackets were located above and below the horizontal beams strengthening somewhat the beam-column joint over a 2" distance at each end.

Two sets of 0.5" thick viscoelastic dampers of different sizes (type A with total shear area of 35 in.^{2} and type B with total area of 17.5 in.^{2}) were alternatively tested for the retrofit of the structure. The dampers were tested under cyclic loading prior to the shaking table tests. As shown in Fig. 2.2, the storage stiffness and the loss stiffness for each test are determined, from which the other relevant properties of the damper at a particular frequency can be calculated. The relevant properties required for the analysis predictions of the response of the structure with viscoelastic dampers were obtained from tests done by Shen, Soong, et al., and are listed in Table 3.1. The viscoelastic dampers display a behavior dependent on frequency, strain amplitude, and temperature. Although this

a) Elevation



b) Viscoelastic Brace Damper

c) Details of Viscoelastic Damper

FIGURE 3-6 R/C Frame with Viscoelastic Brace Dampers

TABLE 3-1 Properties of Dampers in Retrofitted Structure

Frequency f (Hz) (1)	Shear Storage Modulus G_s(ksi) (2)	Shear Loss Modulus G_l(ksi) (3)	Shear Storage Stiffness k_d (kip/in) (4)	Shear Loss Stiffness k_l (kip/in) (5)	Loss Factor η (6)	Damping Coefficient c (kips/in/sec) (7)
(a) Properties of Damper A						
1.0	0.182	0.248	12.74	17.36	1.36	2.76
1.5	0.244	0.305	17.08	21.35	1.25	2.27
2.0	0.294	0.366	20.58	25.62	1.24	2.04
2.5	0.335	0.396	23.45	27.72	1.18	1.76
3.0	0.345	0.431	24.15	30.17	1.25	1.60
(b) Properties of Damper B						
1.0	0.199	0.259	6.97	9.07	1.30	1.44
1.5	0.265	0.326	9.28	11.41	1.23	1.21
2.0	0.300	0.395	10.50	13.83	1.32	1.10
2.5	0.365	0.463	12.78	16.21	1.27	1.03
3.0	0.385	0.487	13.48	17.05	1.26	0.90

temperature dependency is the most significant, the variations in the damper properties can be neglected in a temperature controlled environment (such as room temperature in most office buildings and laboratories).

The frame structure was subjected to a shaking table testing schedule as shown in Table 3-2. Wide banded (0 - 50 Hz.) white noise excitations were used for identification of the dynamic characteristics of the structure before and after every earthquake shaking table motion. Since testing was conducted

TABLE 3-2 Testing Program for the Retrofitted Model with Viscoelastic Braces

Test (1)	Test Description (2)	VE Damper Type (3)	Test Label (4)	Purpose (5)
0.	Taft N21E, PGA 0.20g	None	TF20_WO	Comparison Response
00.	Taft N21E, PGA 0.30g		TF30_WO	Comparison Response
1.	White Noise, PGA 0.025g		WNB_VEA	Identification
2.	White Noise, PGA 0.025g		WNC_VEA	Identification
3.	**Taft N21E, PGA 0.05 g**	A	**TF05_VEA**	**Minor Earthquake**
4.	White Noise, PGA 0.025g		WND_VEA	Identification
5.	**Taft N21E, PGA 0.20 g**		**TF20_VEA**	**Moderate Earthquake**
6.	White Noise, PGA 0.025g		WNE_VEA	Identification
7.	White Noise, PGA 0.025g		WNA_VEB	Identification
8.	**Taft N21E, PGA 0.05 g**		**TF05_VEB**	**Minor Earthquake**
9.	White Noise, PGA 0.025g	B	WNB_VEB	Identification
10.	White Noise, PGA0.025g		WNC_VEB	Identification
11.	**Taft N21E, PGA 0.20 g**		**TF20_VEB**	**Moderate Earthquake**
12.	White Noise, PGA0.025g		WND_VEB	Identification

Note: _WO indicates no dampers and _VEx indicates viscoelastic dampers of type x.

over several days, consecutive white noise excitations were used to validate the current dynamic characteristics of the building. Focus on the analytical performance evaluation is drawn to tests #3, #5, #8 and #11, as they are indicative of the response of the building to the representative base motions for minor and moderate earthquakes with additional stiffness and damping.

3.2 Stiffness Identification

3.2.1 Experimental Identification of Stiffness

The stiffness matrix is computed from the experimentally determined frequencies, mode shapes and the mass at each story level (Bracci, 1992) as

$$\mathbf{K} = \mathbf{M\Phi\Omega\Phi}^T\mathbf{M} \tag{3.1}$$

TABLE 3-3 Dynamic Characteristics History of the Retrofitted Model From Low Level Vibrations (White Noise PGA 0.025g)

Test Name	Frequency	Modal Shapes	Stiffness Matrix	Story Stiffnesses	Equivalent Viscous Damping
	f (Hz.)	$\mathbf{\Phi}_{ij}$	$\mathbf{K}_{ij}$ (kip/in)	$\mathbf{k}_i$ (kip/in)	ξ_i (%)
(a) Before Earthquake Test Taft N21EPGA 0.20g					
White noise WHNR_B	$\begin{pmatrix} 2.78 \\ 9.38 \\ 16.75 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.86 & -0.51 \\ 0.79 & 0.48 & 1.00 \\ 0.42 & 1.00 & -0.89 \end{pmatrix}$	$\begin{pmatrix} 205.2 & -238.6 & 71.6 \\ -238.6 & 421.4 & -278.2 \\ 71.6 & -278.2 & 432.7 \end{pmatrix}$	$\begin{pmatrix} 238.6 \\ 278.2 \\ 154.5 \end{pmatrix}$	$\begin{pmatrix} 3.0 \\ 1.9 \\ 1.3 \end{pmatrix}$
White noise WHNR_C	$\begin{pmatrix} 2.64 \\ 9.18 \\ 16.70 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.86 & -0.49 \\ 0.79 & 0.45 & 1.00 \\ 0.44 & 1.00 & -0.83 \end{pmatrix}$	$\begin{pmatrix} 198.9 & -238.2 & 65.2 \\ -238.2 & 438.5 & -279.1 \\ 65.2 & -279.1 & 404.6 \end{pmatrix}$	$\begin{pmatrix} 238.2 \\ 279.1 \\ 125.5 \end{pmatrix}$	$\begin{pmatrix} 4.7 \\ 1.8 \\ 1.6 \end{pmatrix}$
(b) After Earthquake Test Taft N21E PGA 0.20g					
WHNR_D (East) Whitenoise	$\begin{pmatrix} 1.98 \\ 8.11 \\ 15.33 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.86 & -0.56 \\ 0.82 & 0.42 & 1.00 \\ 0.46 & 1.00 & -0.81 \end{pmatrix}$	$\begin{pmatrix} 182.7 & -218.2 & 71.9 \\ -218.2 & 356.9 & -229.3 \\ 71.9 & -229.3 & 318.3 \end{pmatrix}$	$\begin{pmatrix} 218.2 \\ 229.3 \\ 89.0 \end{pmatrix}$	$\begin{pmatrix} 6.6 \\ 2.6 \\ 1.4 \end{pmatrix}$
WHNR_D (West) Whitenoise	$\begin{pmatrix} 1.93 \\ 7.98 \\ 15.48 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.88 & -0.59 \\ 0.82 & 0.38 & 1.00 \\ 0.48 & 1.00 & -0.80 \end{pmatrix}$	$\begin{pmatrix} 196.0 & -226.9 & 80.6 \\ -226.9 & 356.5 & -233.8 \\ 80.6 & -233.8 & 311.9 \end{pmatrix}$	$\begin{pmatrix} 226.9 \\ 233.8 \\ 78.1 \end{pmatrix}$	$\begin{pmatrix} 8.1 \\ 2.8 \\ 0.8 \end{pmatrix}$
(c) After Earthquake Test Taft N21E PGA 0.30g					
WHNR_E (East) Whitenoise	$\begin{pmatrix} 1.88 \\ 7.5 \\ 14.84 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.83 & -0.56 \\ 0.82 & 0.36 & 1.00 \\ 0.45 & 1.00 & -0.76 \end{pmatrix}$	$\begin{pmatrix} 168.1 & -205.3 & 69.6 \\ -205.3 & 342.7 & -215.8 \\ 69.6 & -215.8 & 277.5 \end{pmatrix}$	$\begin{pmatrix} 205.3 \\ 215.8 \\ 61.7 \end{pmatrix}$	$\begin{pmatrix} 5.5 \\ 1.9 \\ 1.5 \end{pmatrix}$
WHNR_E (West) Whitenoise	$\begin{pmatrix} 1.73 \\ 7.50 \\ 14.84 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.84 & -0.55 \\ 0.83 & 0.36 & 1.00 \\ 0.49 & 1.00 & -0.76 \end{pmatrix}$	$\begin{pmatrix} 165.0 & -203.8 & 67.5 \\ -203.8 & 344.0 & -217.8 \\ 67.5 & -217.8 & 277.8 \end{pmatrix}$	$\begin{pmatrix} 203.8 \\ 217.8 \\ 60.0 \end{pmatrix}$	$\begin{pmatrix} 6.7 \\ 1.9 \\ 1.2 \end{pmatrix}$

where

$$\Omega = diag(\omega_1^2, \omega_2^2, \ldots\ldots \omega_n^2)$$

$$\Phi = \text{mass normalized mode shape matrix } (\phi^t \mathbf{M} \phi = \mathbf{I})$$

Table 3-3 shows the history of dynamic characteristics of the building prior to the retrofit with the viscoelastic braces (from Bracci et al., 1992c).

3.2.2 Analytical Identification of Stiffness

The analytical stiffness matrix was computed by a standard matrix condensation of the massless degrees of freedom of the structure. Expressing the overall stiffness matrix **K** without addition of dampers in (2.1) as

$$\mathbf{K} = \begin{bmatrix} \mathbf{K}_{\alpha\alpha} & \mathbf{K}_{\alpha\beta} \\ \mathbf{K}_{\beta\alpha} & \mathbf{K}_{\beta\beta} \end{bmatrix} \tag{3.2}$$

where subscripts α and β correspond to mass and massless degrees of freedom respectively. The reduced stiffness matrix is determined as

$$\mathbf{K}^* = \mathbf{K}_{\alpha\alpha} - \mathbf{K}_{\alpha\beta}\mathbf{K}_{\beta\beta}^{-1}\mathbf{K}_{\beta\alpha} \tag{3.3}$$

Reinforced concrete has a nonlinear hysteretic behavior in which the force depends on the past history of deformations and the current state of deformation. The stiffness variations are also memory dependent and are defined by the past as well as current state of deformation dictated by the hysteretic activity it undergoes. In order to predict the response of buildings which have previously experienced inelastic deformations, the hysteretic properties for all the components would need to be updated before proceeding any new analyses. As the model was subjected to a number of damaging base motions prior to retrofitting with concrete jacketing, the response predictions for subsequent tests became questionable. To overcome this hurdle, a simplifying

TABLE 3-4 Analytical Versus Experimental Damping and Stiffness

Analytical Stiffness Matrix (kip/in) (1)	Experimental Stiffness Matrix (kip/in) (2)
(a) White Noise, PGA 0.025 Before Earthquake Test #0	
$\mathbf{K} = \begin{pmatrix} 183.6 & -239.6 & 70.0 \\ -239.6 & 421.4 & -278.5 \\ 70.0 & -278.5 & 430.0 \end{pmatrix}$	$\mathbf{K} = \begin{pmatrix} 205.2 & -238.6 & 71.6 \\ -238.6 & 421.4 & -278.2 \\ 71.6 & -278.2 & 432.7 \end{pmatrix}$
(b) White Noise, PGA 0.025 during Test #1	
$\mathbf{K} = \begin{pmatrix} 152.5 & -184.8 & 39.6 \\ -184.8 & 315.0 & -166.8 \\ 39.6 & -166.8 & 222.0 \end{pmatrix}$	$\mathbf{K} = \begin{pmatrix} 168.1 & -205.3 & 69.6 \\ -205.3 & 342.7 & -215.8 \\ 69.6 & -215.8 & 277.5 \end{pmatrix}$

assumption was made by which the member structural properties were determined from engineering data by slightly modifying the gross moments of inertia such that the overall dynamic characteristics of the building were in agreement with those obtained experimentally from the first low level vibrations under earthquake excitation test. The identification of the stiffness matrix using this procedure insured that the influence of the viscoelastic braces can be suitably incorporated. The analytical stiffness matrix is compared in Table 3-4, with the one identified from experiments using the measured properties.

3.3 Identification of Natural Frequencies and Damping Ratios

The experimental damping ratios are estimated by the half-power method, from the story transfer functions. The analytical damping ratios are computed from Eqs. (2.13), (2.16) and (2.26). The identified properties using the two sets of dampers are listed in Table 3-5. Adding the inherent viscous damping properties of the structure without the additional braces, the total damping ratio

TABLE 3-5 Structure's Properties with Viscoelastic Dampers From Strong Vibrations

Identified Dynamic Characteristic (1)	Retrofitted Structure with Dampers A (2)	Retrofitted Structure with Dampers B (3)
(a) Experimental Properties		
Modal Matrix (Φ)	$\begin{pmatrix} 1.00 & -0.72 & 0.55 \\ 0.87 & 0.26 & -1.00 \\ 0.49 & -1.00 & 0.65 \end{pmatrix}$	$\begin{pmatrix} 1.00 & -0.72 & 0.56 \\ 0.88 & 0.24 & -1.00 \\ 0.50 & 1.00 & 0.64 \end{pmatrix}$
First Mode Frequency, f [Hz.]	2.62	2.13
Total Damping [Exp.] ξ(%)	22.0	18.0
(b) Analytical Properties from Equivalent Dynamic Analysis		
1st. mode Freq. [[Rad] / [Hz]]	15.38 / 2.45	13.07 / 2.08
Added Damping[1] ξ(%)	19.7	15.3
Total Damping ξ(%)	21.2	16.8
(c) Analytical Properties from Complex Eigenvalue Analysis		
1st. Mode Rotational Freq. [Rad]	2.96 ± i 14.96	1.98 ± i 12.94
1st. Mode Freq. [[Rad] / [Hz]]	14.96 / 2.38	12.94 / 2.06
Added Damping[2] ξ (%)	19.4	15.1
Total Damping ξ	20.9	16.6

obtained is close to that identified from the experiment. It is observed that the damping ratios computed analytically are slightly lower than that obtained from experiment. This could be because the energy dissipated by hysteretic dampers is not included in the analytical computations of equivalent damping. Also for the range of damping in consideration, the response is not very sensitive to the additional damping, either inherent viscous, or the inaccuracies in the determination of the appropriate supplemental damping.

1from Eq. (2.13)
2from Eq. (2.26)

TABLE 3-6 Analytical Versus Experimental Damping and Stiffness

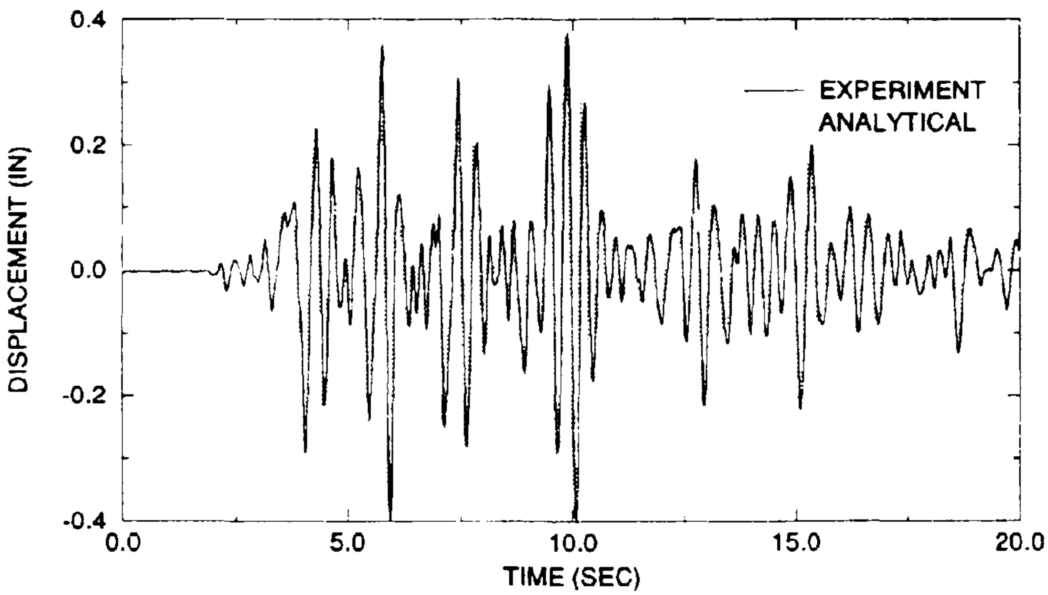
Structures Properties (1)		Retrofitted Structure with Dampers A (2)	Retrofitted Structure with Dampers B (3)
Story Damping, c (kip/in/sec)	Experimental	2.10	1.55
	Analytical (total)	2.07	1.60
Story Stiffness, k (kip/in)	Experimental	49.0	27.5
	Analytical (dampers only)	34.0	15.0
	Analytical (total)	50.0	28.0

The damping properties and the stiffness of each floor, shown in Table 3-6, were calculated from data in Table 3-1 and Eq. (2.7) and compared with those measured in the identification tests. The properties corresponding to frequencies of 2.5 Hz and 2.0 Hz, closet to the actual 2.6Hz and 2.2Hz for braces with dampers A and B respectively were selected for analytical evaluation.

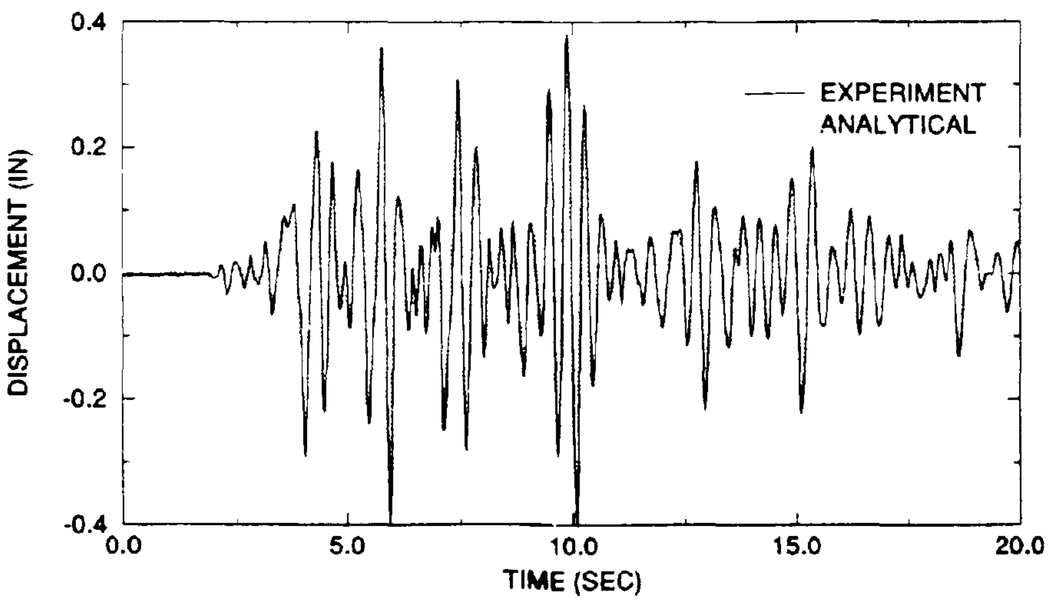
The stiffness properties calculated without considering the influence of the mounting brackets of braces differ largely from those considering the influence of the brackets influence; (see contribution of stiffness from damper alone computed from Table 3-1 to the total stiffness in Table 3-6). The "total" values are used in further analysis for comparison of performances.

3.4 Memory Dependancy in R/C Members

The effect of "memory" in the inelastic properties and the sensitivity of structural response to this memorize effect is shown in Fig. 3.1. The analysis for Test episode #5, subsequent to various other tests (see Table 3-2), was done in two ways: (i) independently without precise knowledge of the modified hysteretic properties of reinforced concrete members [see Fig. 3-7(a)] and (ii) in a sequential consecutive fashion (ie. analyzing all prior episodes of testing and the current one consecutively), such that the hysteretic properties are automatically updated [see Fig. 3-7 (b)]. It is evident that the "memorization" of hysteretic properties is important and the sequential analysis



a. Hysteretic Properties not Updated - Single Analysis



b. Hysteretic Properties Updated - Following Sequential Analysis

FIGURE 3-7 Top Story Displacement During Taft Earthquake, PGA=0.2g (Test #5)

duplicates the experimental results suitably.

3.5 Comparison of Displacement and Acceleration Time Histories

A comparison of story displacement, and acceleration, time histories for tests #3 #5 #8 and #11 are shown in figs. Figures 3.8 through 3.15. The analytical response is in good agreement with the experimental response. Due to the high level of damping the inelastic response is reduced substantially and with it many of the possible errors usually involved in nonlinear dynamic analysis.

3.6 Damper Forces

For two dampers placed at an angle θ with the horizontal, the component of damping in the lateral direction is

$$c_{lateral} = 2c\ \cos^2\theta \tag{3.4}$$

and the component of additional stiffness in the lateral direction is

$$k_{lateral} = 2k_s\ \cos^2\theta \tag{3.5}$$

The lateral force in the damper was computed as a combination of elastic and damping components. The force in each damper (assuming two dampers per floor) is therefore:

$$f_i = [k_{lateral}(u_i - u_{i-1}) + c_{lateral}(\dot{u}_i - \dot{u}_{i-1})]/2\cos\theta \quad \text{for } i \neq 1 \tag{3.6a}$$

$$f_1 = [k_{lateral}u_1 + c_{lateral}\dot{u}_1]/2\cos\theta \qquad \text{for } i = 1 \tag{3.6b}$$

where i = the story level.

A comparison of the forces obtained in the dampers from Eq. (3.6a) and Eq. (3.6b) to those obtained from the experiment are shown in Figs. 3-16 through 3-19. The differences are minimal.

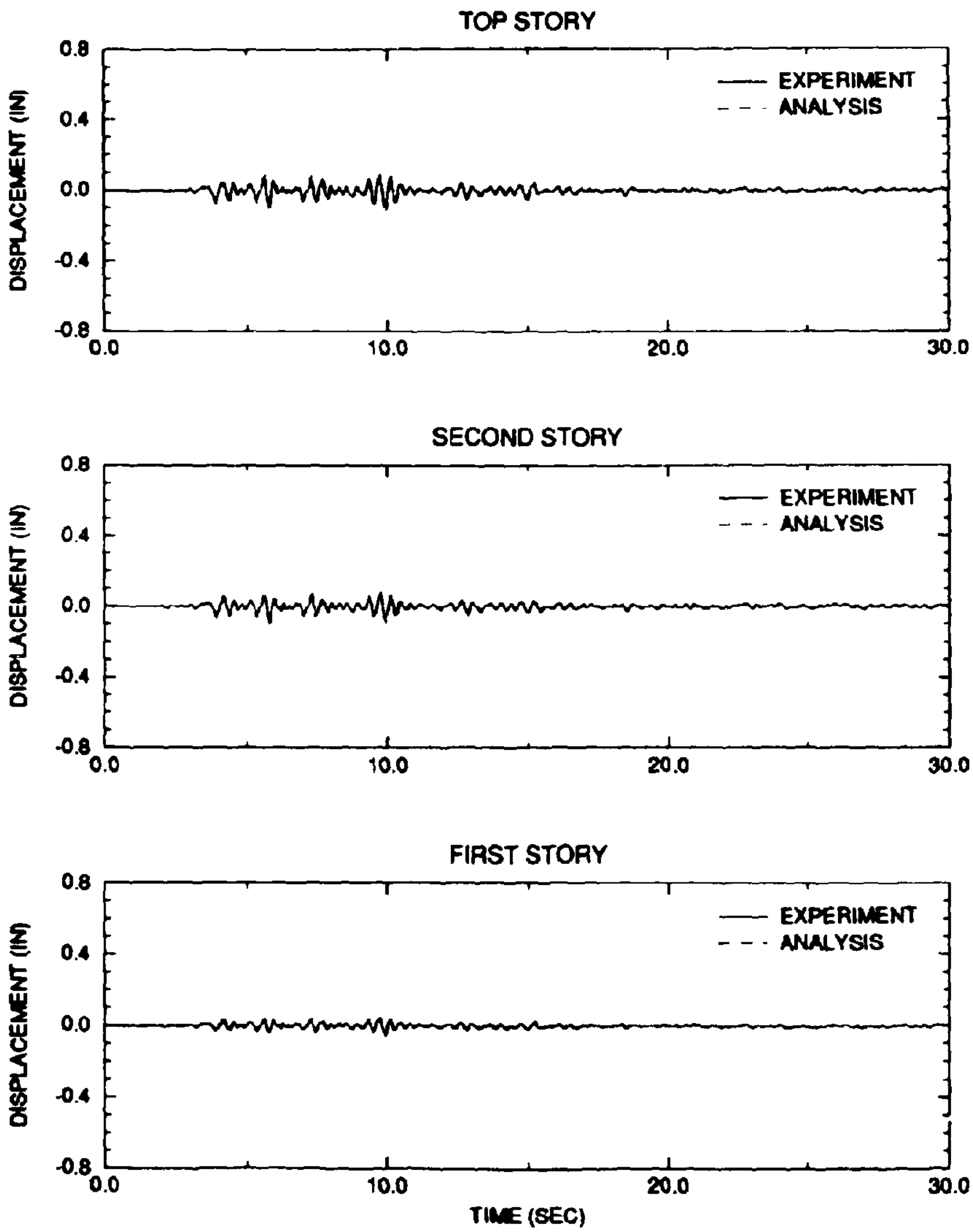


FIGURE 3-8 Displacement Time History with Dampers A for Taft, PGA 0.05g

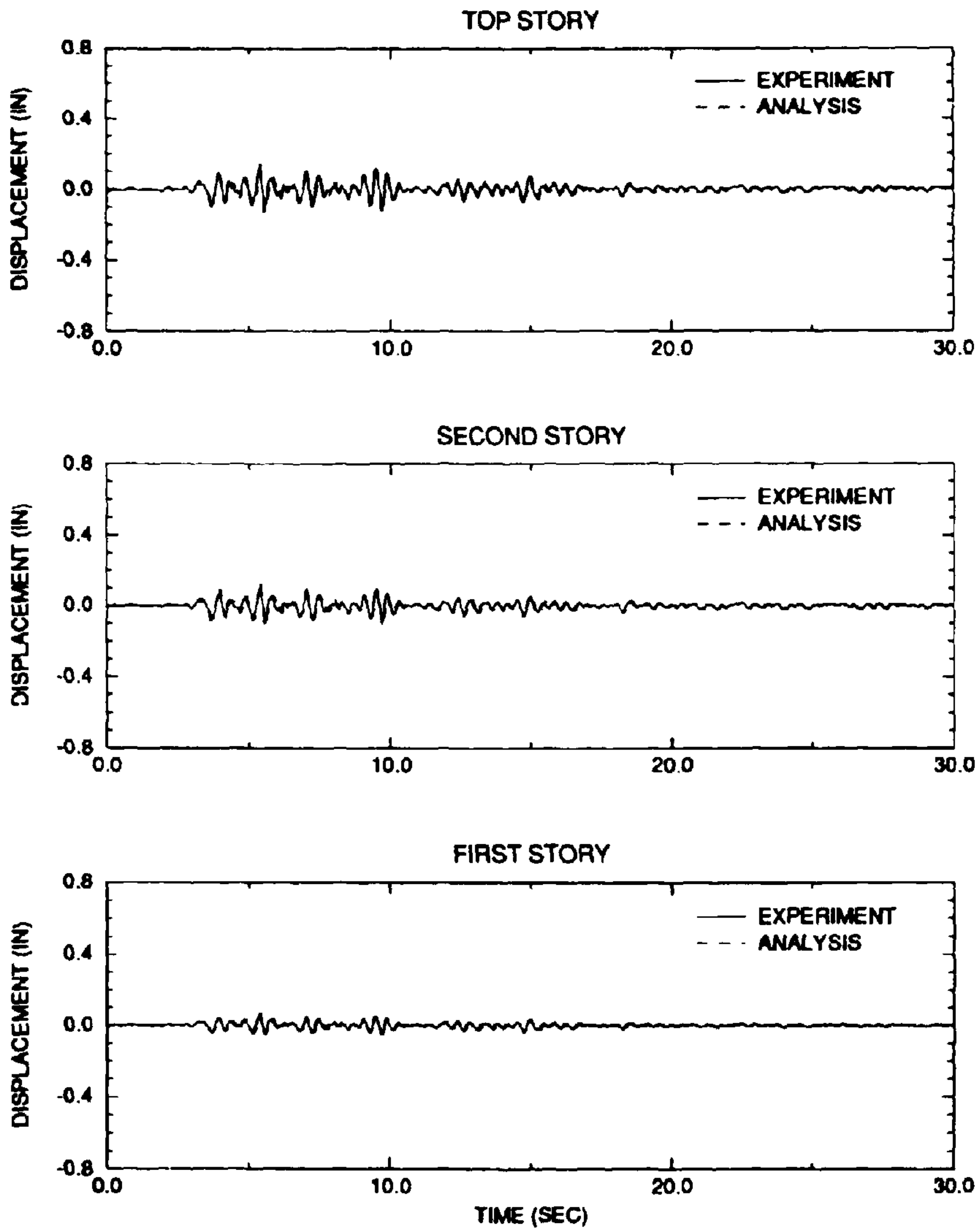


FIGURE 3-9 Displacement Time History with Dampers B for Taft, PGA 0.05g

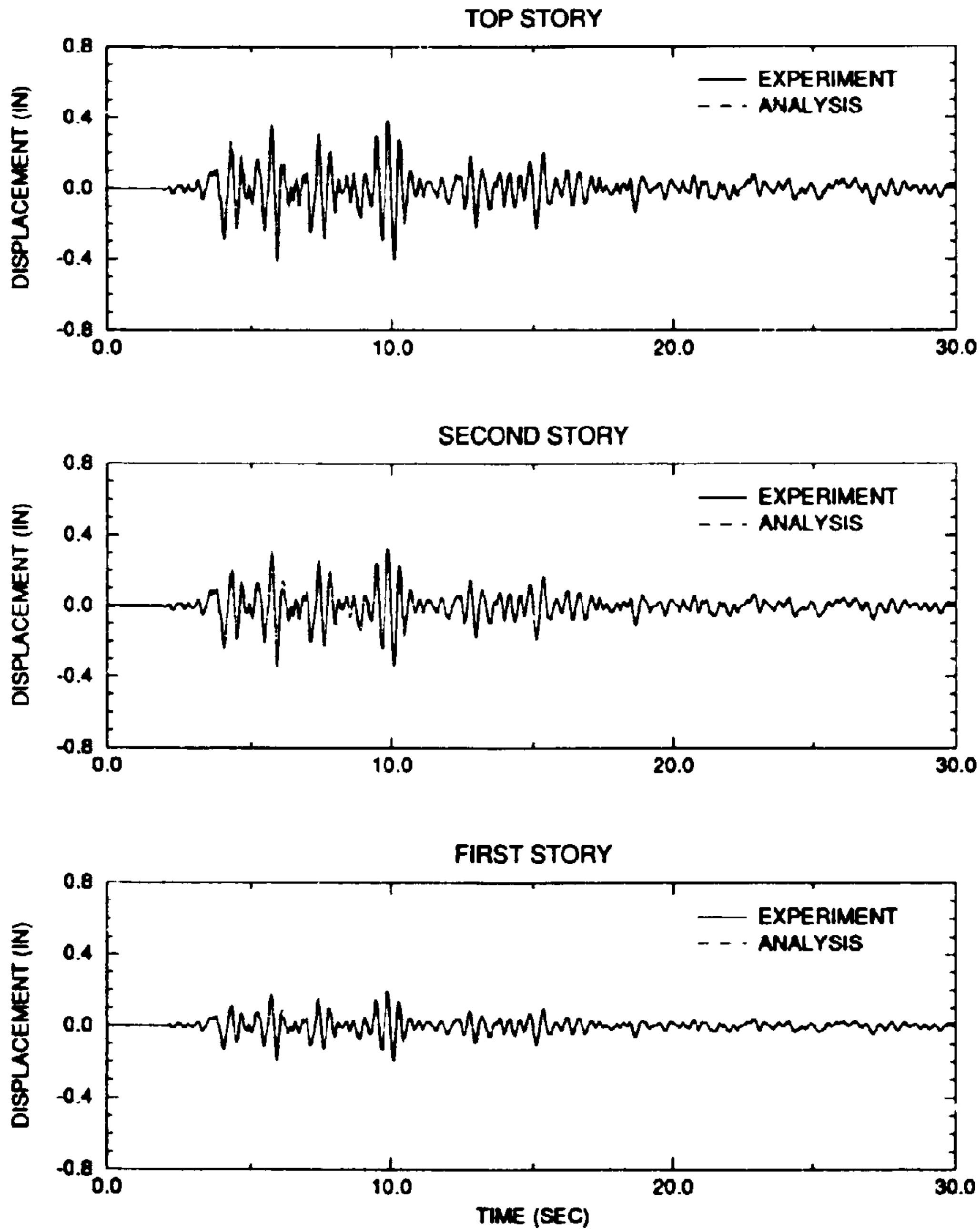


FIGURE 3-10 Displacement Time History with Dampers A for Taft, PGA 0.2g

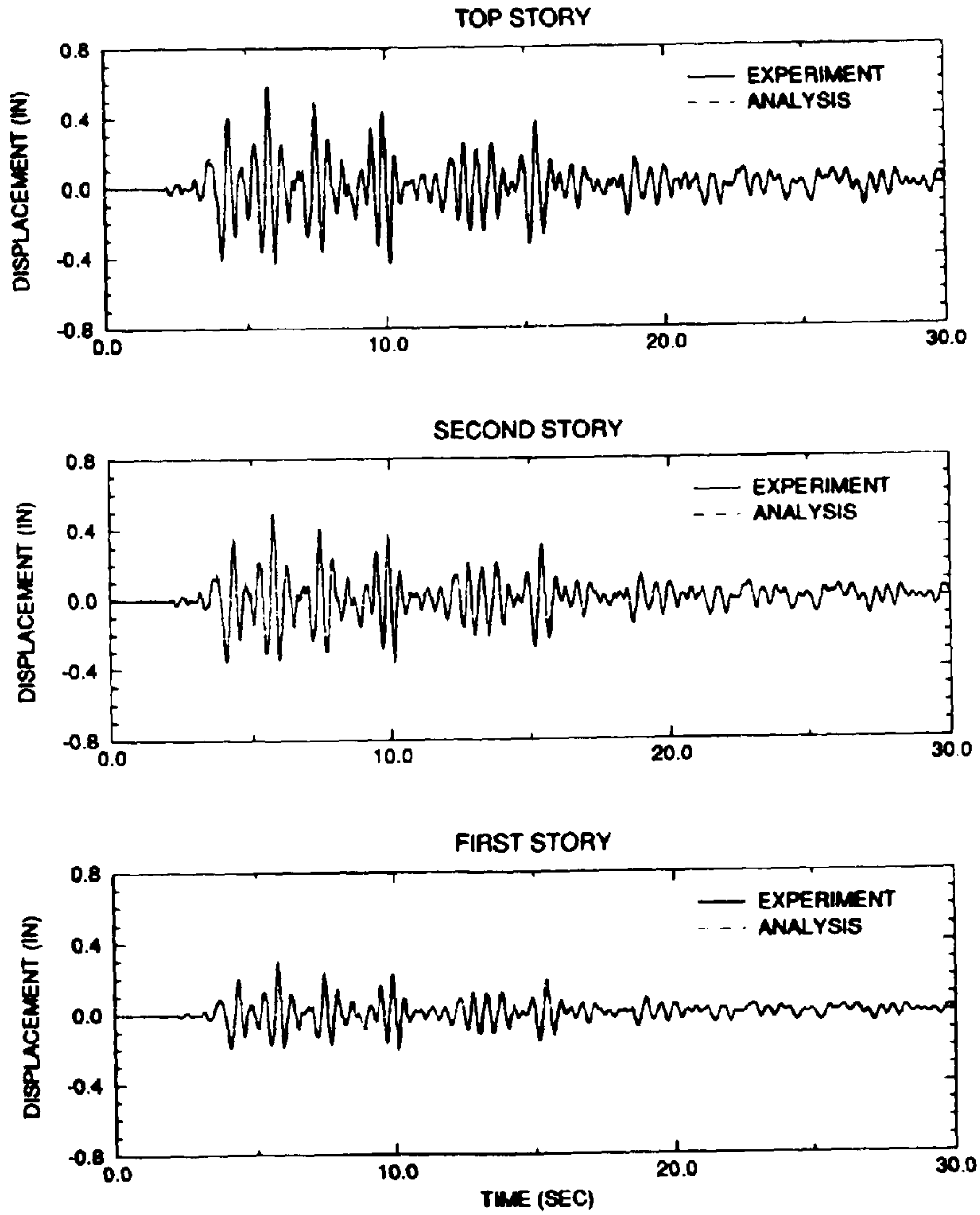


FIGURE 3-11 Displacement Time History with Dampers B for Taft, PGA 0.2g

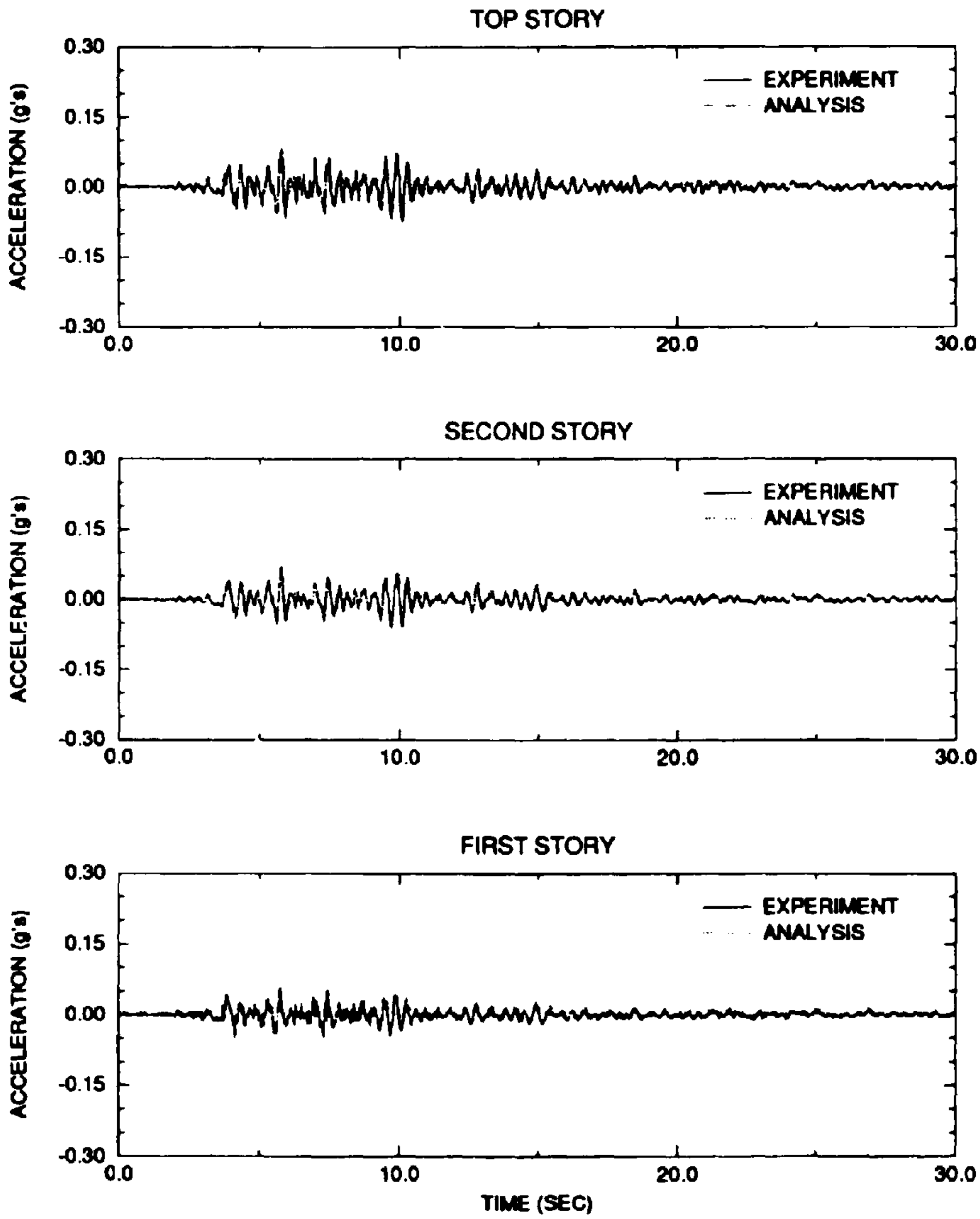


FIGURE 3-12 Acceleration Time History with Dampers A for Taft, PGA 0.05g

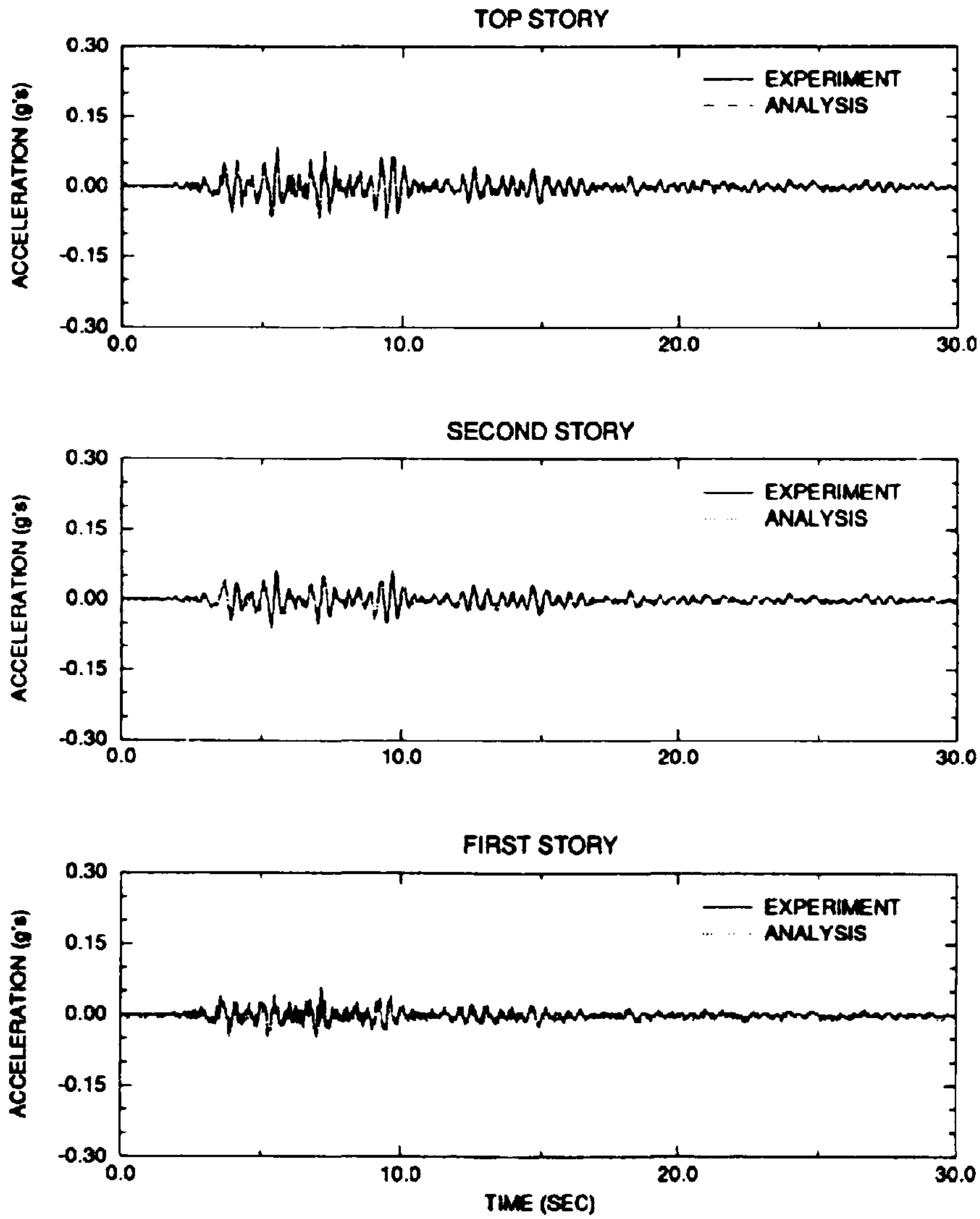


FIGURE 3-13 Acceleration Time History with Dampers B for Taft, PGA 0.05g

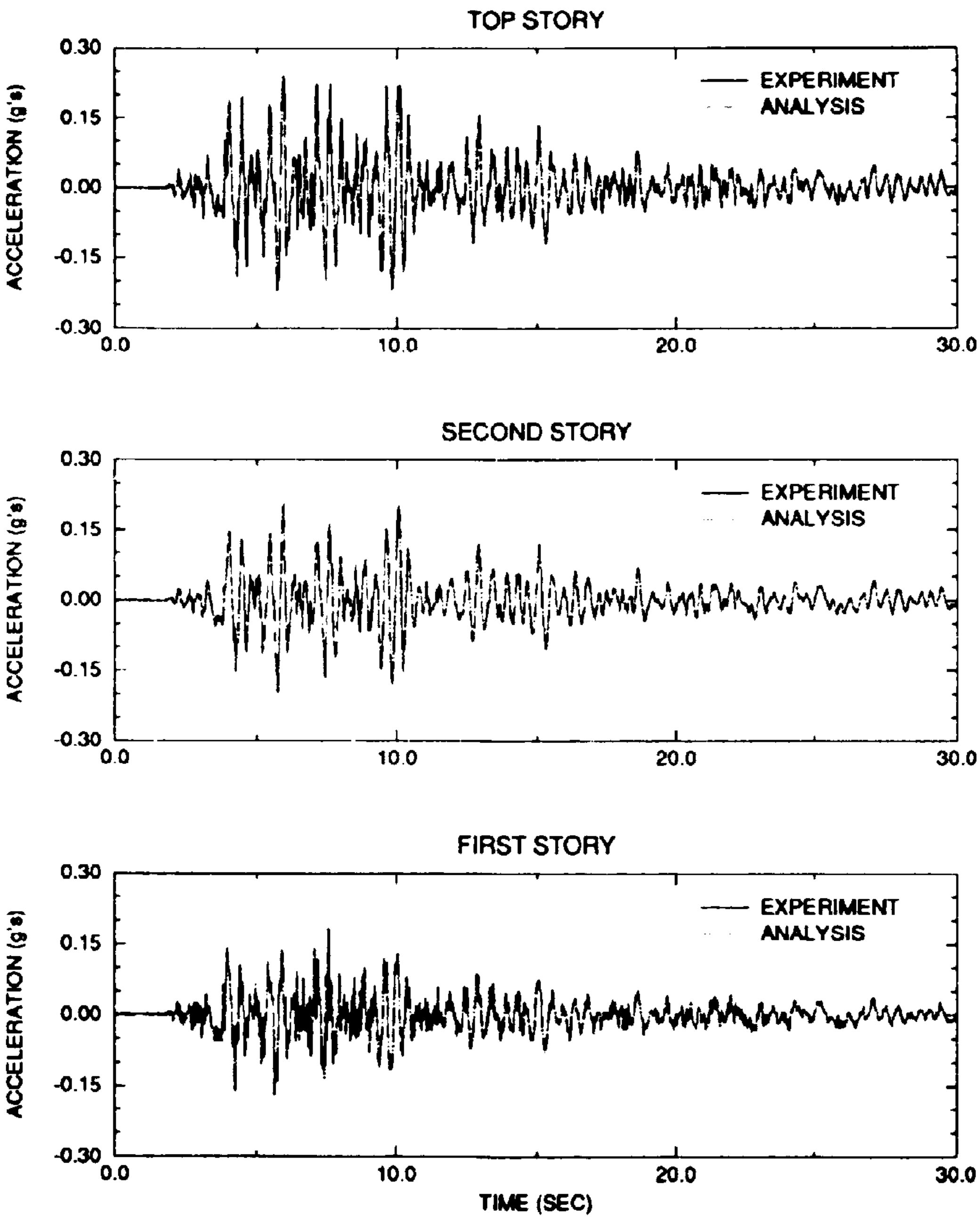


FIGURE 3-14 Acceleration Time History with Dampers A for Taft, PGA 0.2g

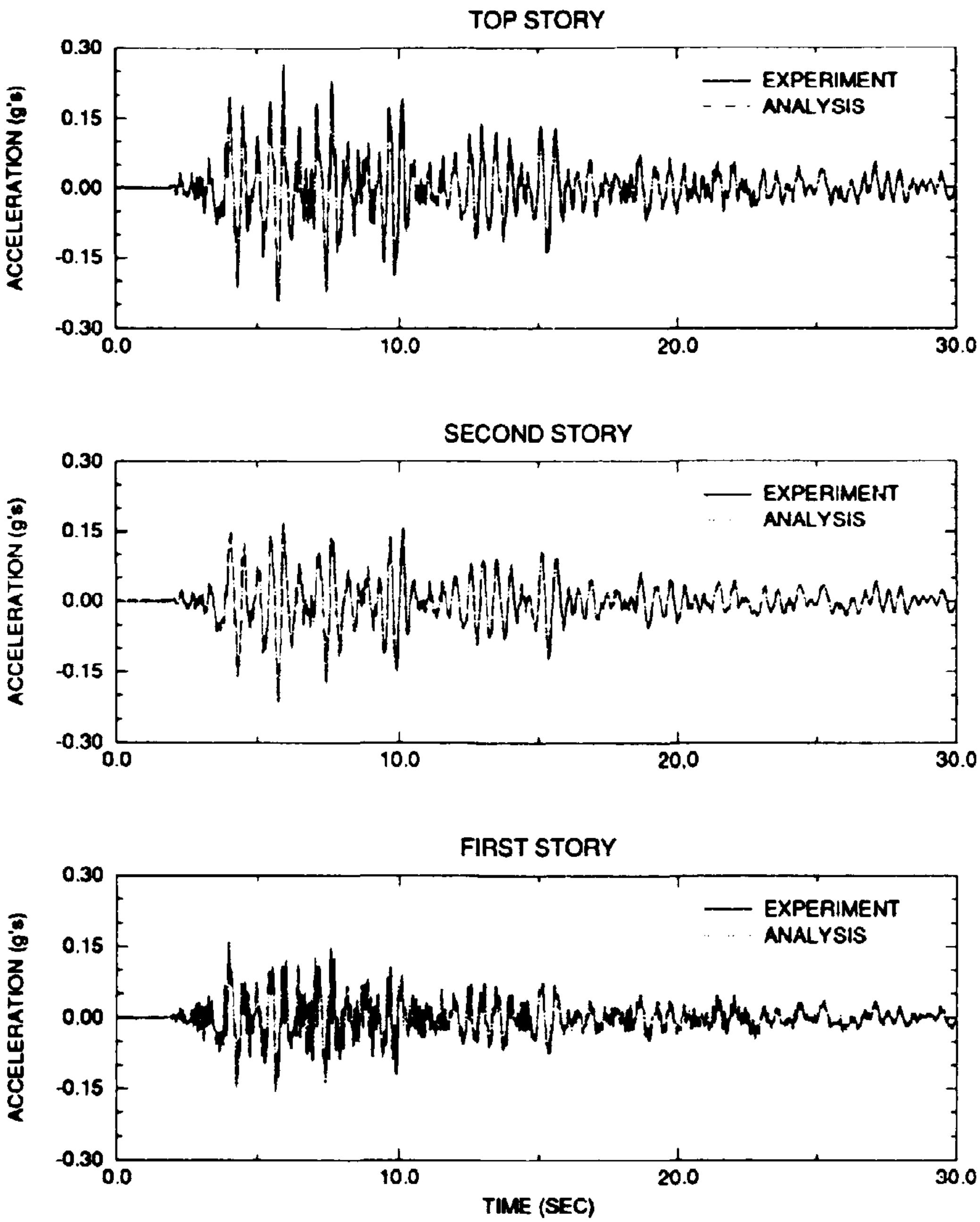


FIGURE 3-15 Acceleration Time History with Dampers B for Taft, PGA 0.2g

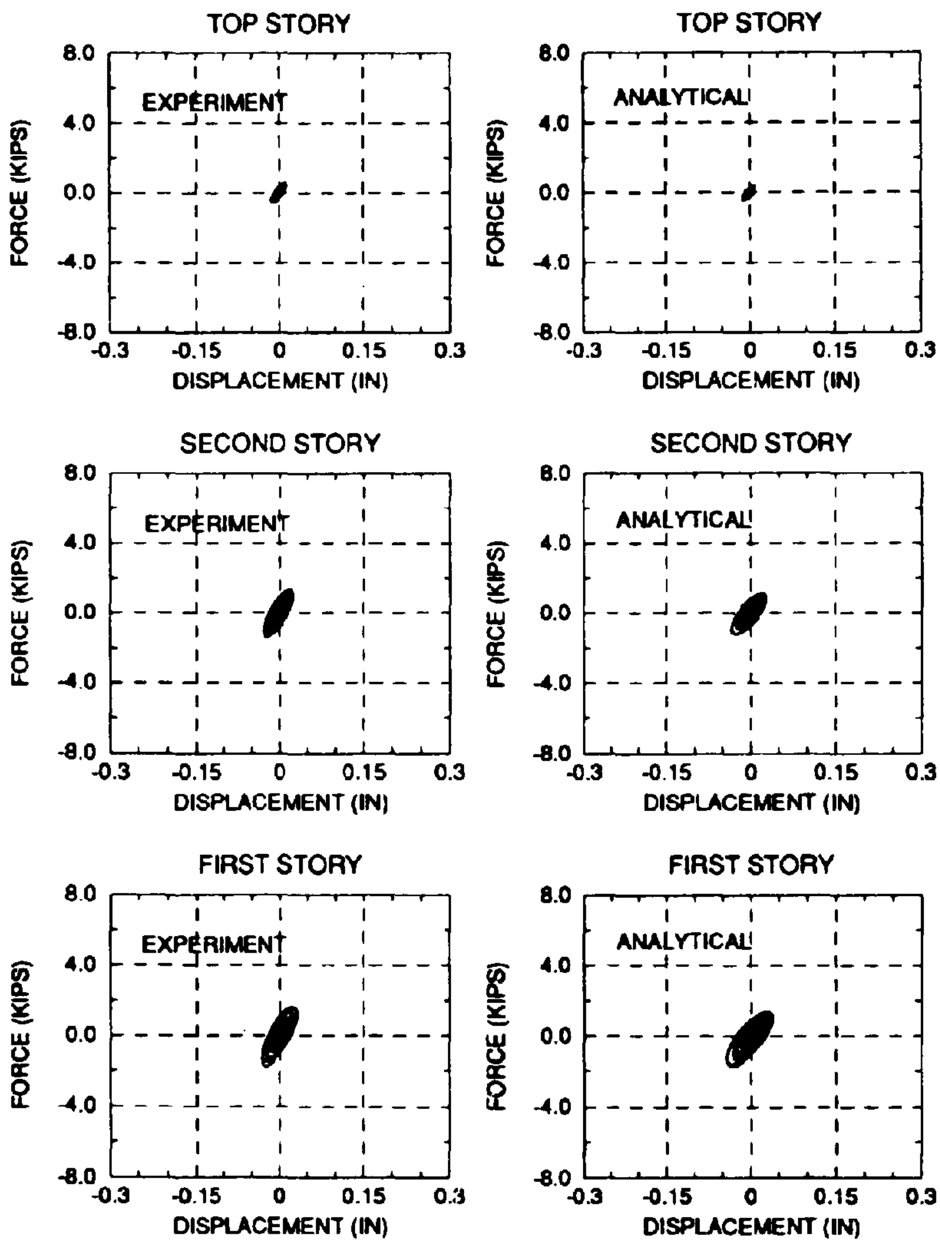


FIGURE 3-16 Force Displacement of Dampers A for Taft, PGA 0.05g

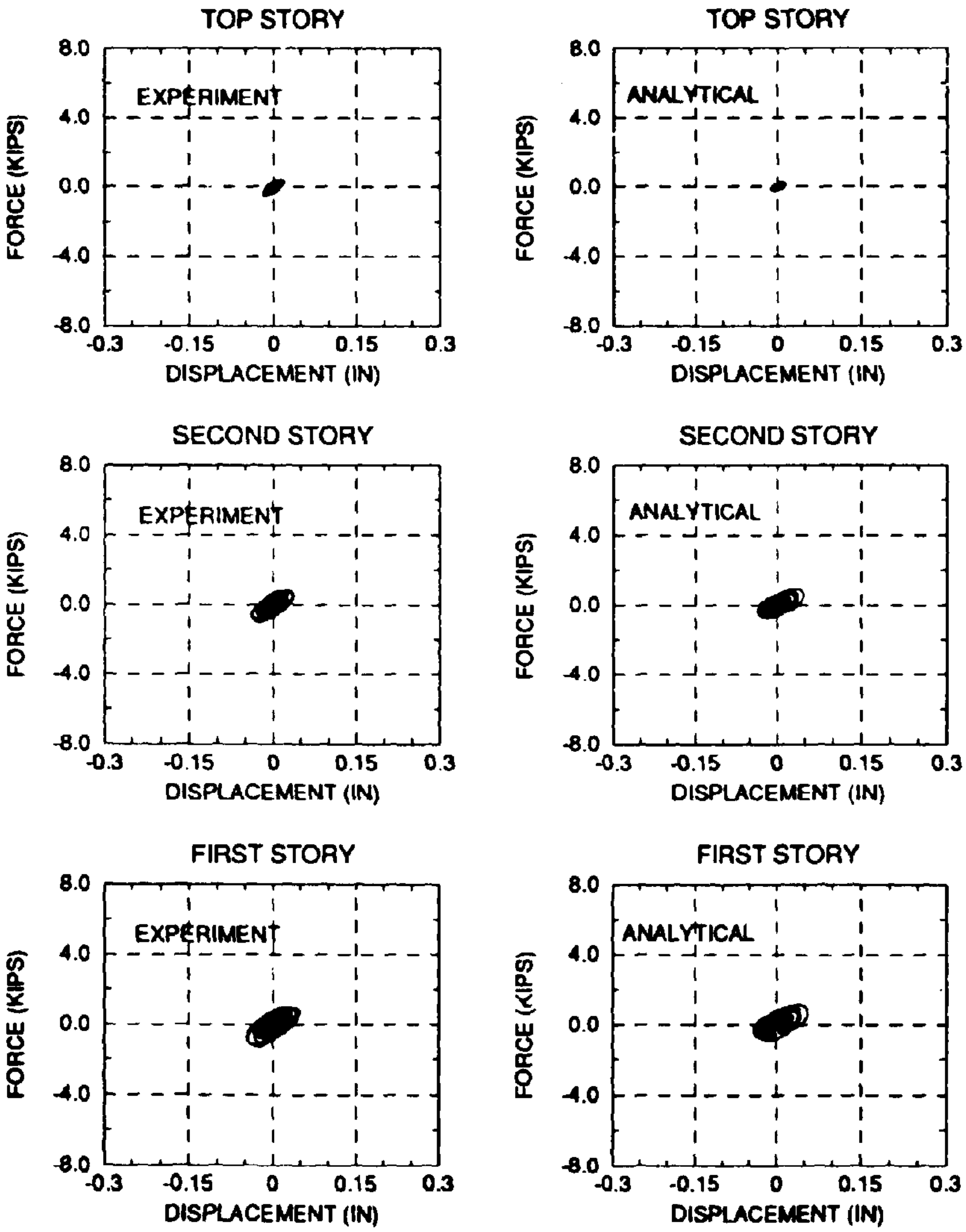


FIGURE 3-17 Force Displacement of Dampers B for Taft, PGA 0.05g

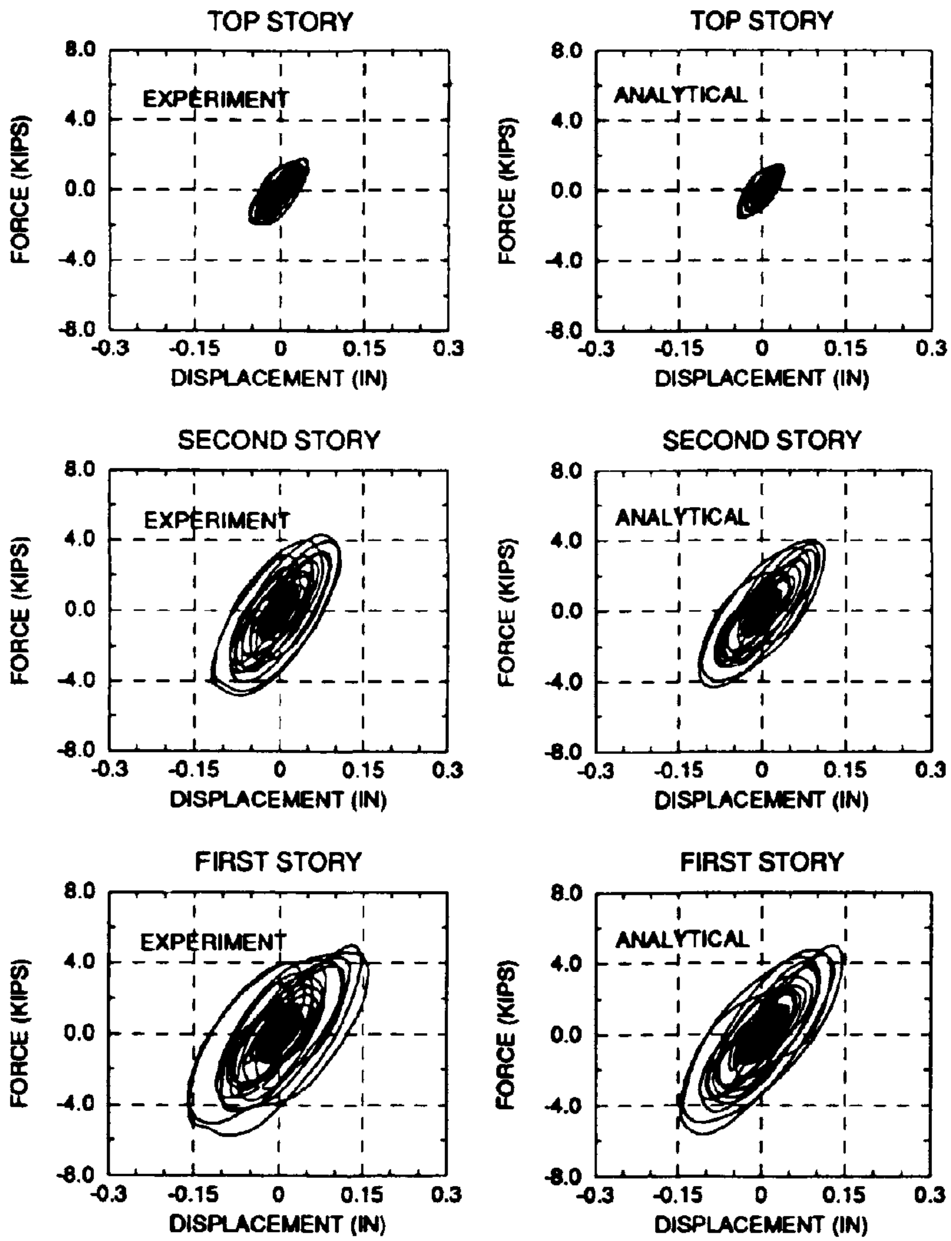


FIGURE 3-18 Force Displacement of Dampers A for Taft, PGA 0.2g

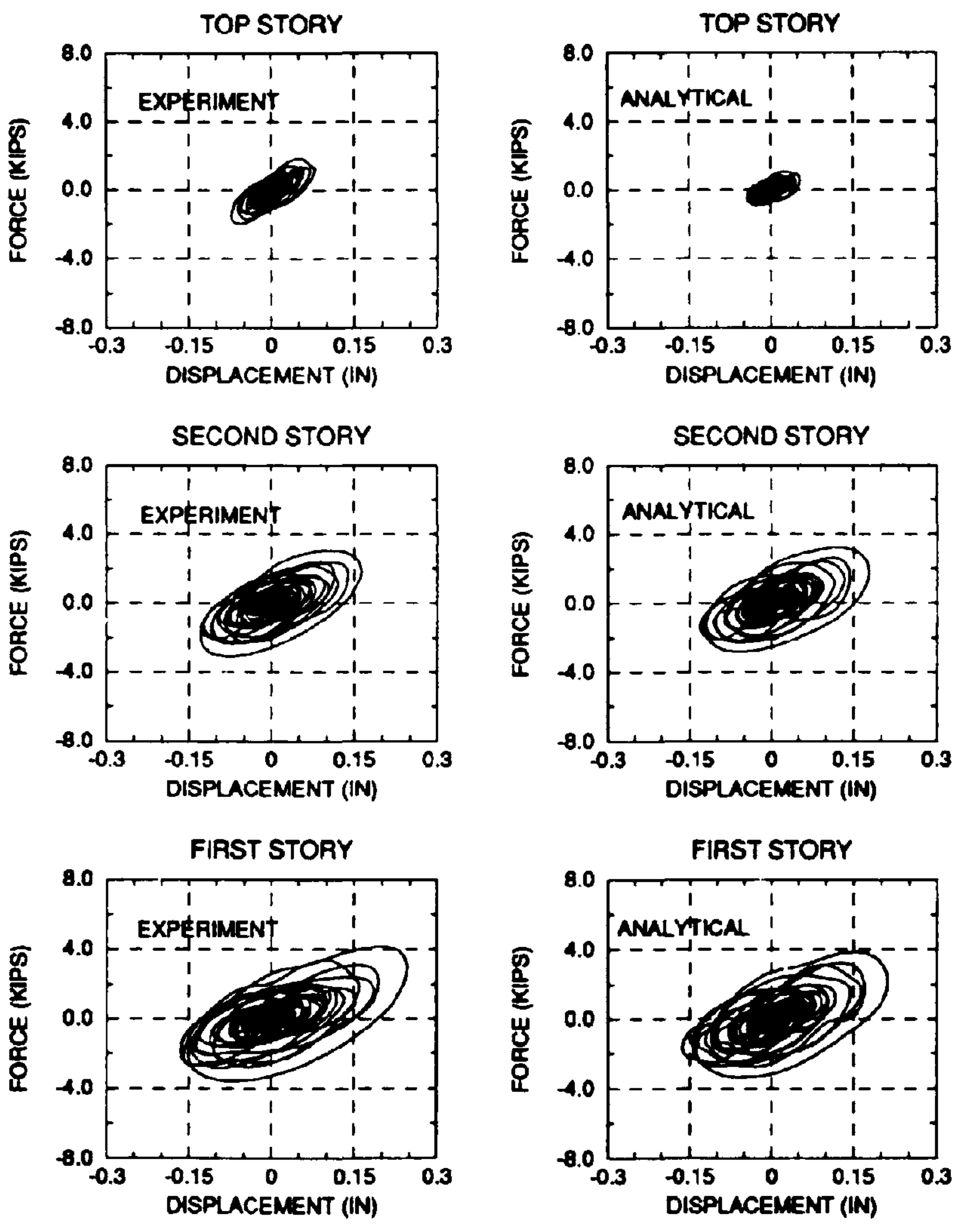


FIGURE 3-19 Force Displacement of Dampers B for Taft, PGA 0.2g

3.7 Base Shears and Damper Stiffnesses

The base shear developed in the columns is compared with this obtained from the experiment in Figs. 3-20 through 3-23. Only a limited inelastic response occurs in columns while energy is mostly dissipated by the viscoelastic braces. The braces also display substantial stiffness as shown by the sloped hysterias in Figs. 3-16 through 3-19. The stiffness calculated without considering the influence of mounting brackets is largely different from this considering the brackets influences. (see contribution of stiffness from damper alone computed from Table 3-1 to the total stiffness in Table 3-4). The "total" values are used in further analysis for comparison of performances.

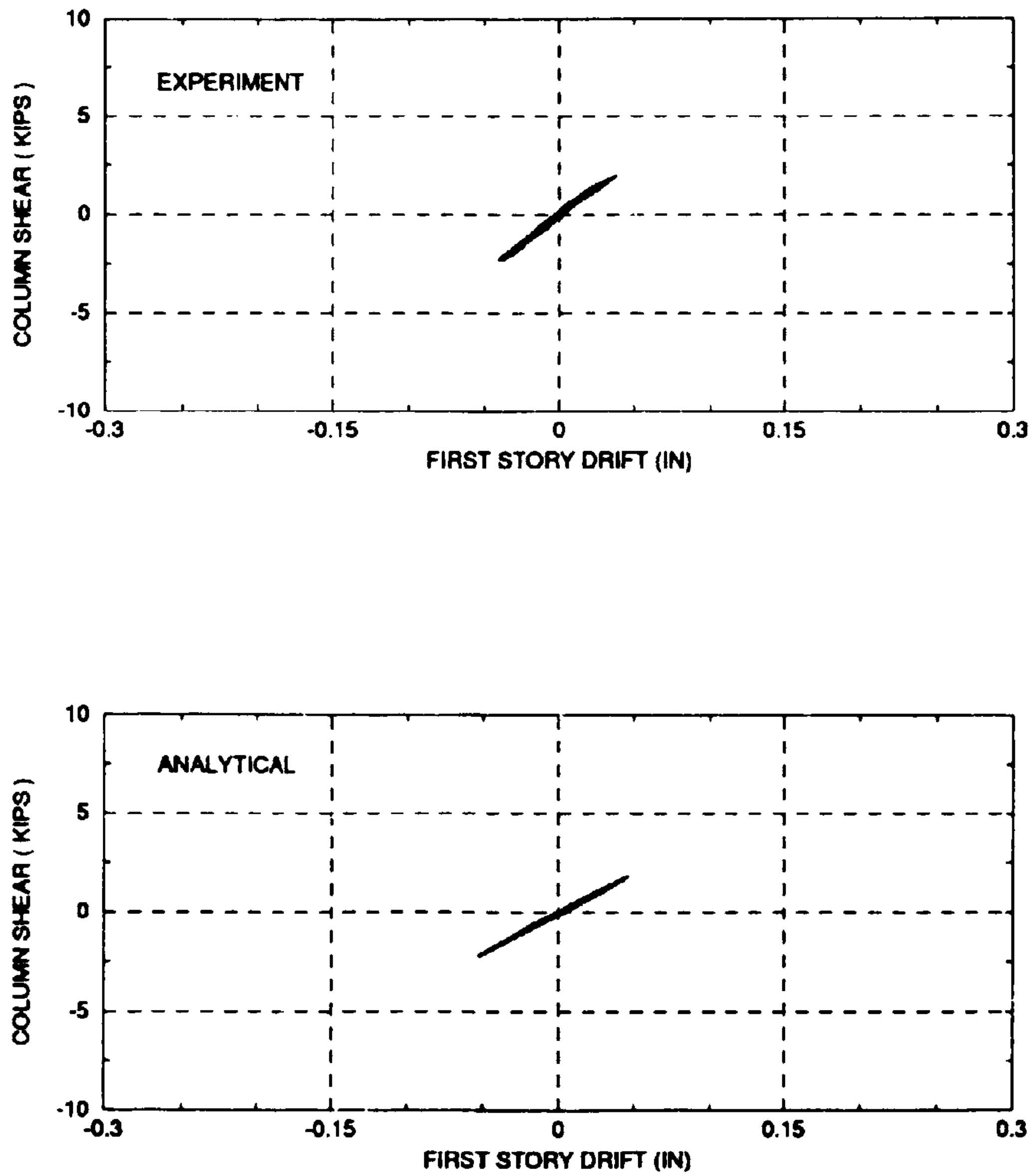
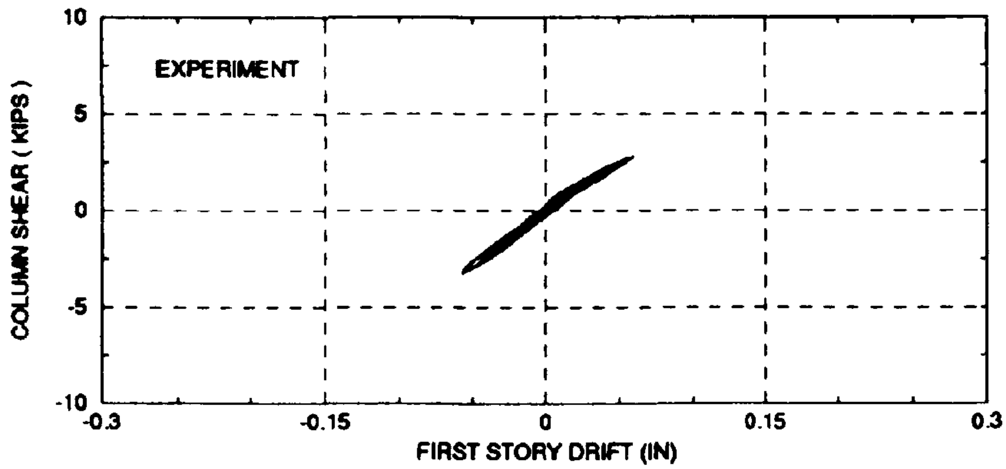


FIGURE 3-20 Base Shear in Columns with Dampers A for Taft, PGA 0.05g



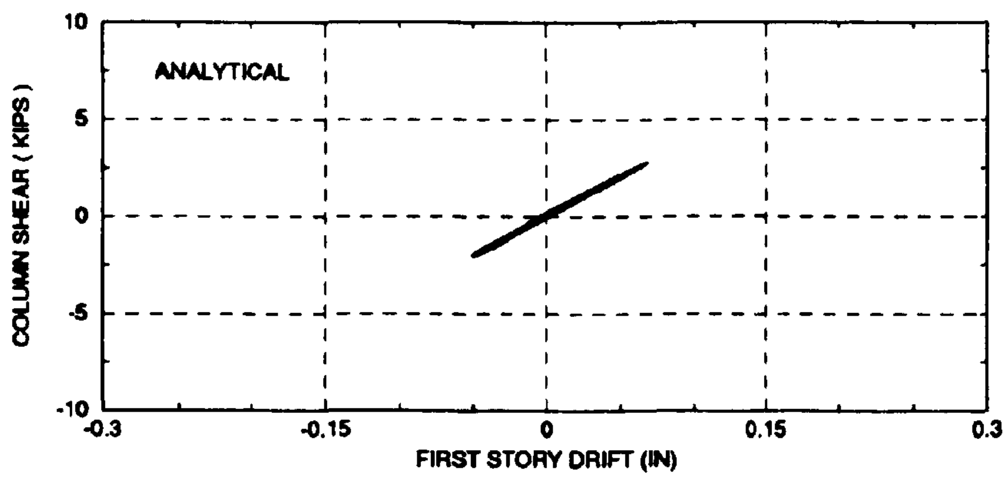


FIGURE 3-21 Base Shear in Columns with Dampers B for Taft, PGA 0.05g

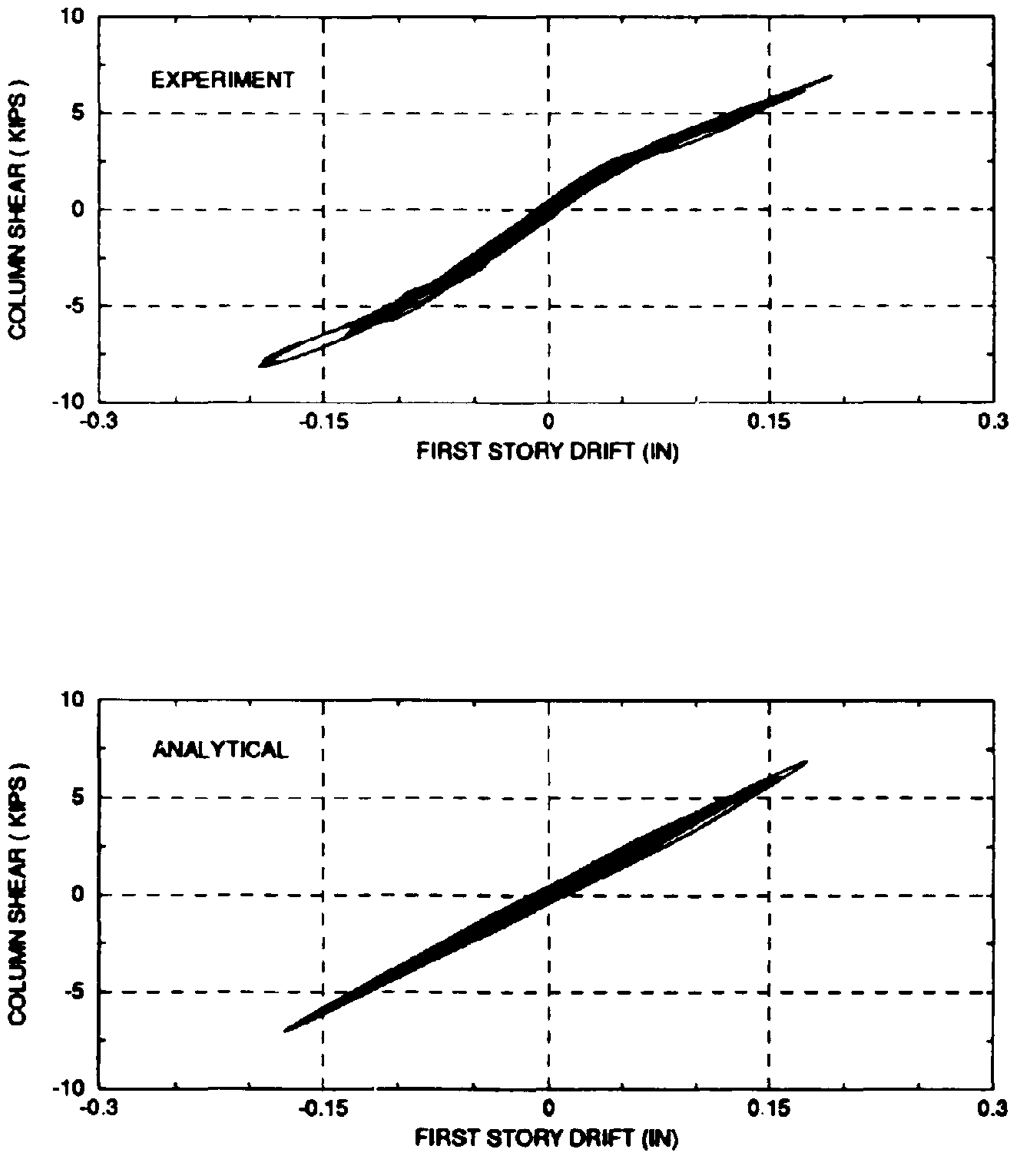


FIGURE 3-22 Base Shear in Columns with Dampers A for Taft, PGA 0.2g

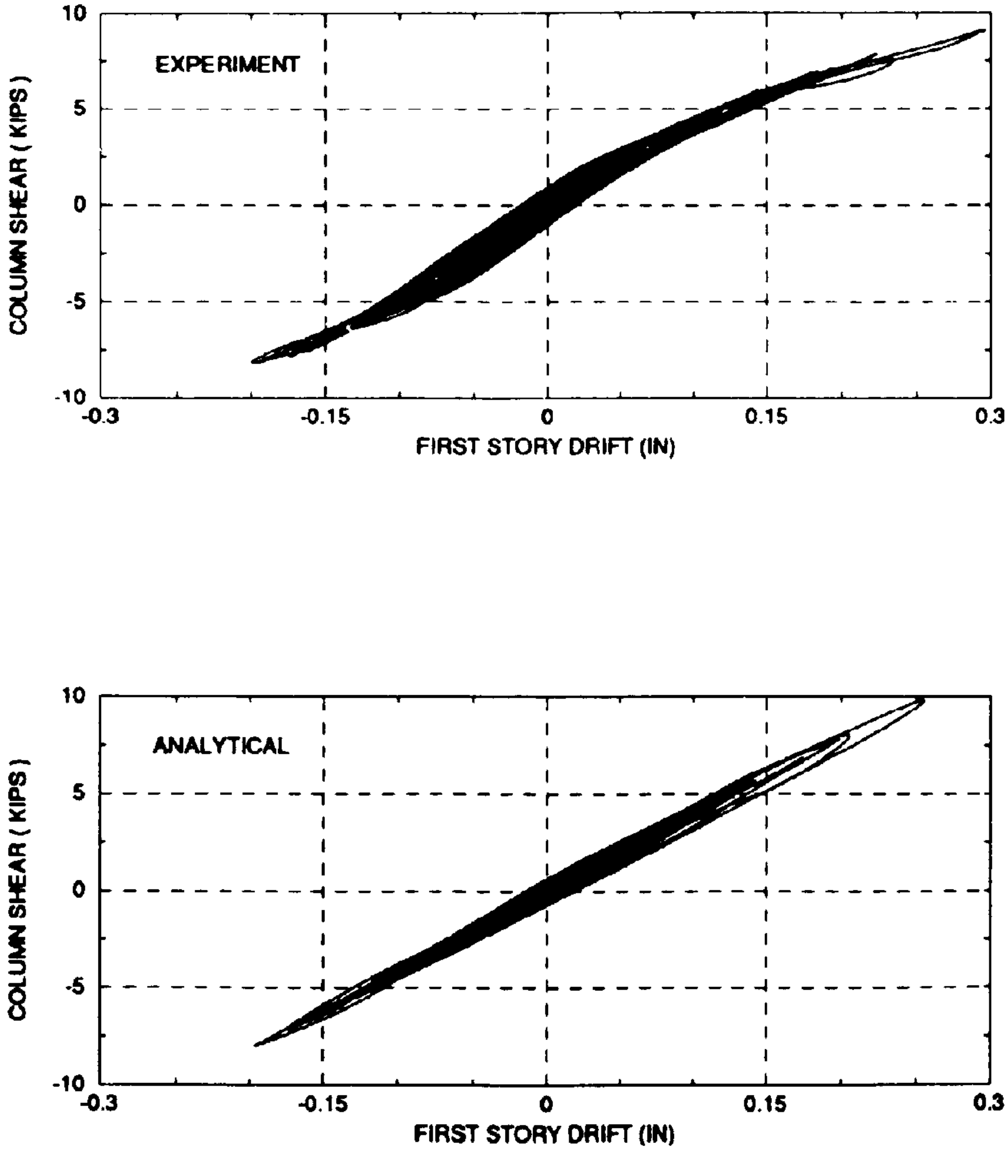


FIGURE 3-23 Base Shear in Columns with Dampers B for Taft, PGA 0.2g

SECTION 4

EFFECTS OF VISCOELASTIC BRACES ON STRUCTURAL RESPONSE

The interpretation of the experimental data requires a good analytical model that is capable of providing internal information of forces, local deformations and changes in structural characteristics. The analytical model specified and verified in the previous sections is used in conjunction with the experimental results to identify the influence of the dampers on the modification of stiffness, redistribution of internal forces and redistribution of energy dissipation between elements. The influence of viscoelastic dampers is summarized as follows:

4.1 Natural Frequencies/Period

The structure with viscoelastic braces subjected to low level (white noise) displays simultaneous increase in frequencies and equivalent viscous damping in all modes as shown in Figs. 4-1 and 4-2 and numerically in Table 4-1. The apparent damping increased 4 times in the structure with dampers A and 3 times in the structure with dampers B. Both types of dampers contribute to an increase in structural stiffness and therefore a reduction of the natural period that might contribute to an increase of the overall base shear.

TABLE 4-1 First Mode Dynamic Characteristics During Low Level Vibration Tests

(1)	Natural Frequency (Hz) (2)	Period (sec) (3)	Equivalent Viscous Damping % (4)
No dampers	1.88	0.53	5.5
With damper A	2.93	0.34	22.0
With damper B	2.54	0.39	18.0

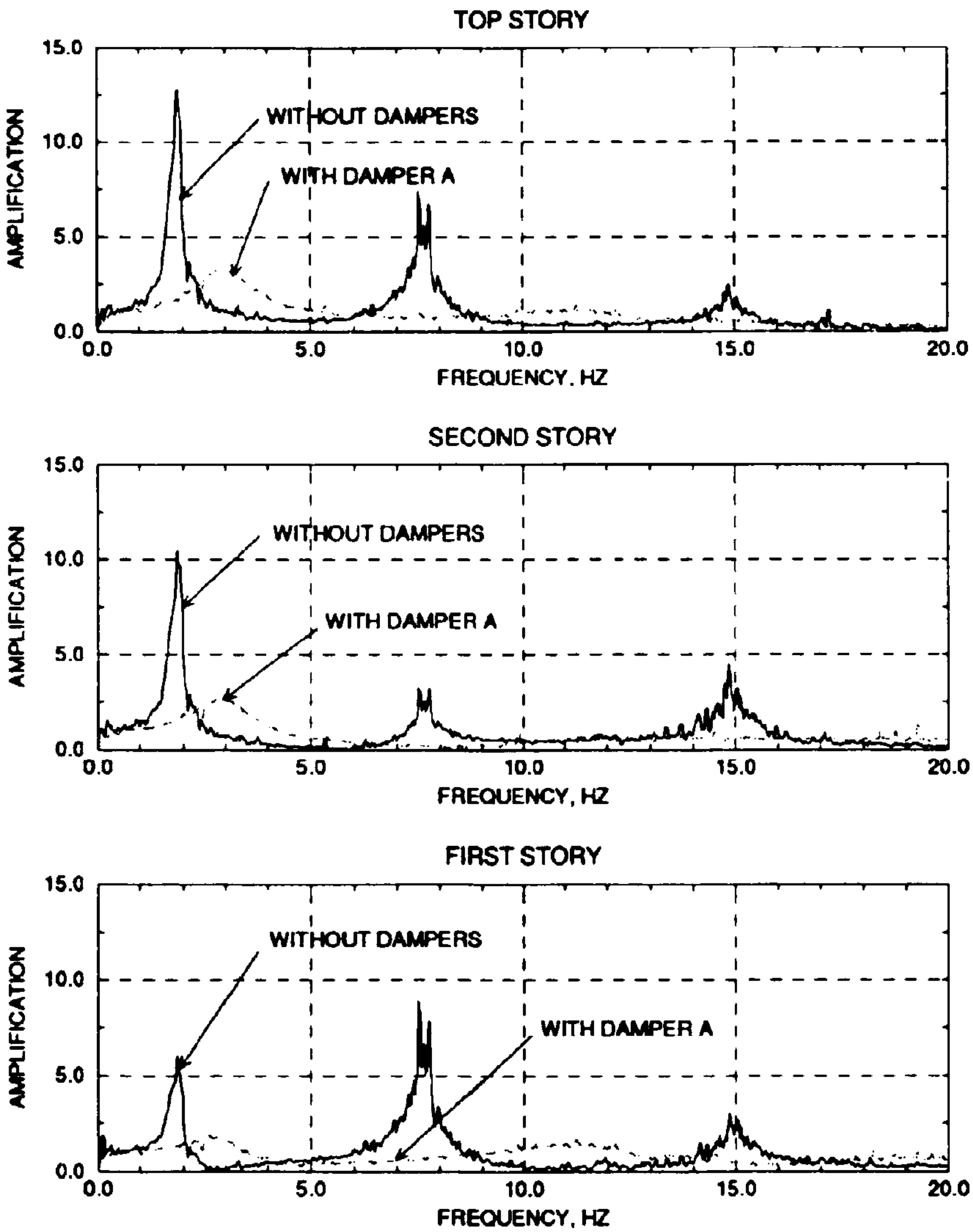


FIGURE 4-1 Acceleration Transfer Function with Dampers A (for White Noise PGA 0.025g)

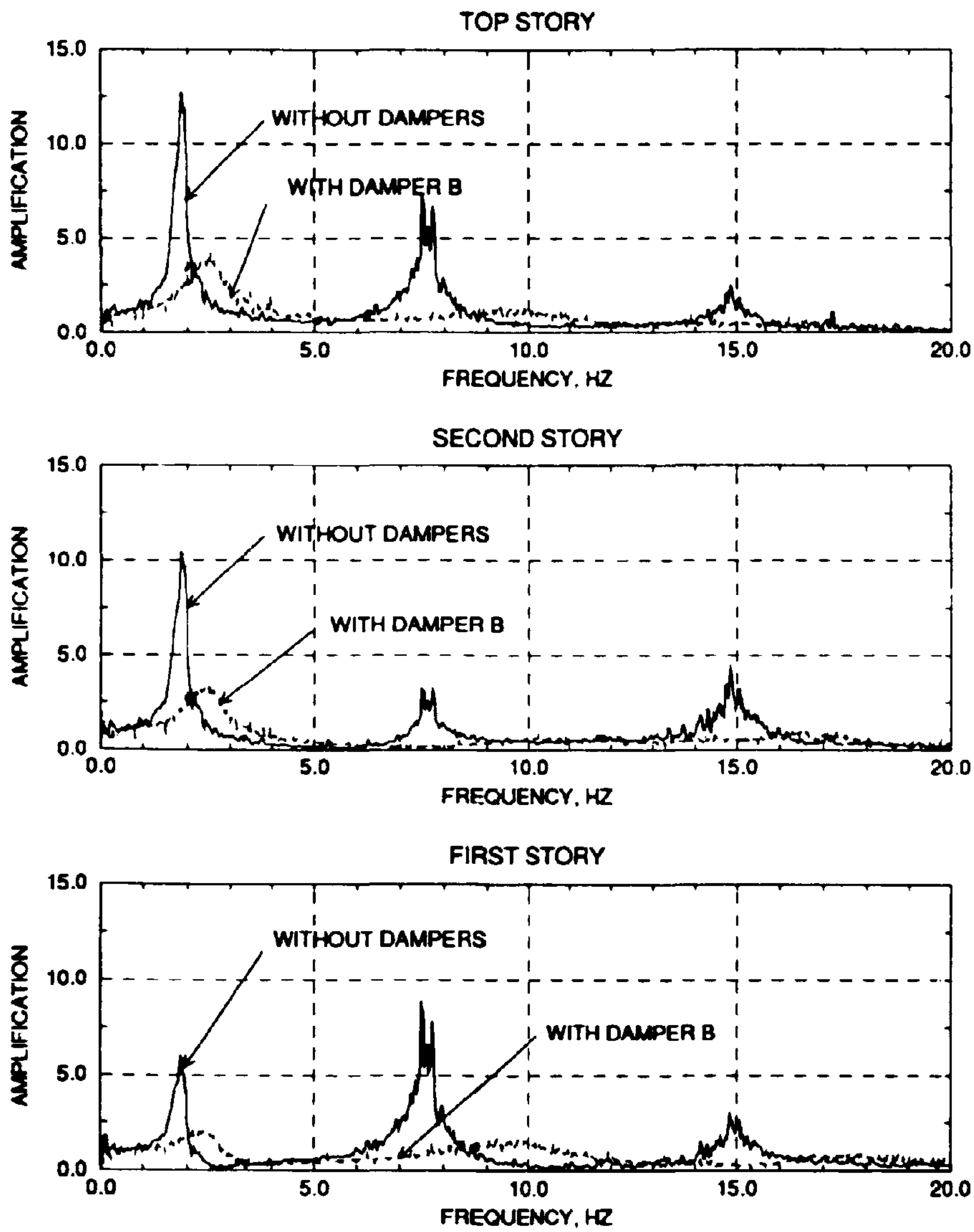


FIGURE 4-2 Acceleration Transfer Function with Dampers B (for White Noise PGA 0.025g)

The frequencies identified from the white noise tests show a higher natural frequency for both dampers types A and B than that determined during earthquake (Tests #5 and #11, Table 3-2) from the transfer functions for the top story acceleration. The reason for these differences is in the nonlinearity of the cracked reinforced concrete sections. At very low vibrations, pre-existing cracks do not open and the sections behave almost like ideal "gross sections". At larger vibrations, such as those created during earthquakes, the cracks open thus reducing the stiffness and their "natural frequency". Small variations are observed also for the equivalent damping.

4.2 Story Forces and Drifts

The inter-story drifts and story shears in the columns are substantially reduced at all floors as indicated in Table 4-2. While the deformations are reduced approximately 3 times, the shear forces are reduced only twice. These forces are much smaller than the ultimate strength of the columns, moreover smaller than their yielding strength (see also Bracci et. al., 1992a and 1992c). A set of force-deformations at the first floor for Taft earthquake motion (PGA 0.20g) (see Figs. 4-3 and 4-4) indicates that the column forces and deformations are substantially reduced, while most of the energy dissipation (area of hysteretic loops) is transferred from the columns to the viscoelastic dampers. Although some inelastic deformations are experienced by the columns, their response is substantially improved in the presence of the viscoelastic braces.

4.3 Columns Axial Forces

The addition of braces changes the load transfer pattern in the structure. Additional axial forces will be generated in the columns by the added brace stiffness which are in phase with the other forces from the structural stiffening system.

The axial force variation in the columns in the presence of dampers is shown in Figs. 4-5 and 4-6. The trajectory of variation of the axial forces and moments are plotted in comparison to the failure envelopes on a P-M interaction curve. The reduction in the moment demand (horizontal fluctuation)

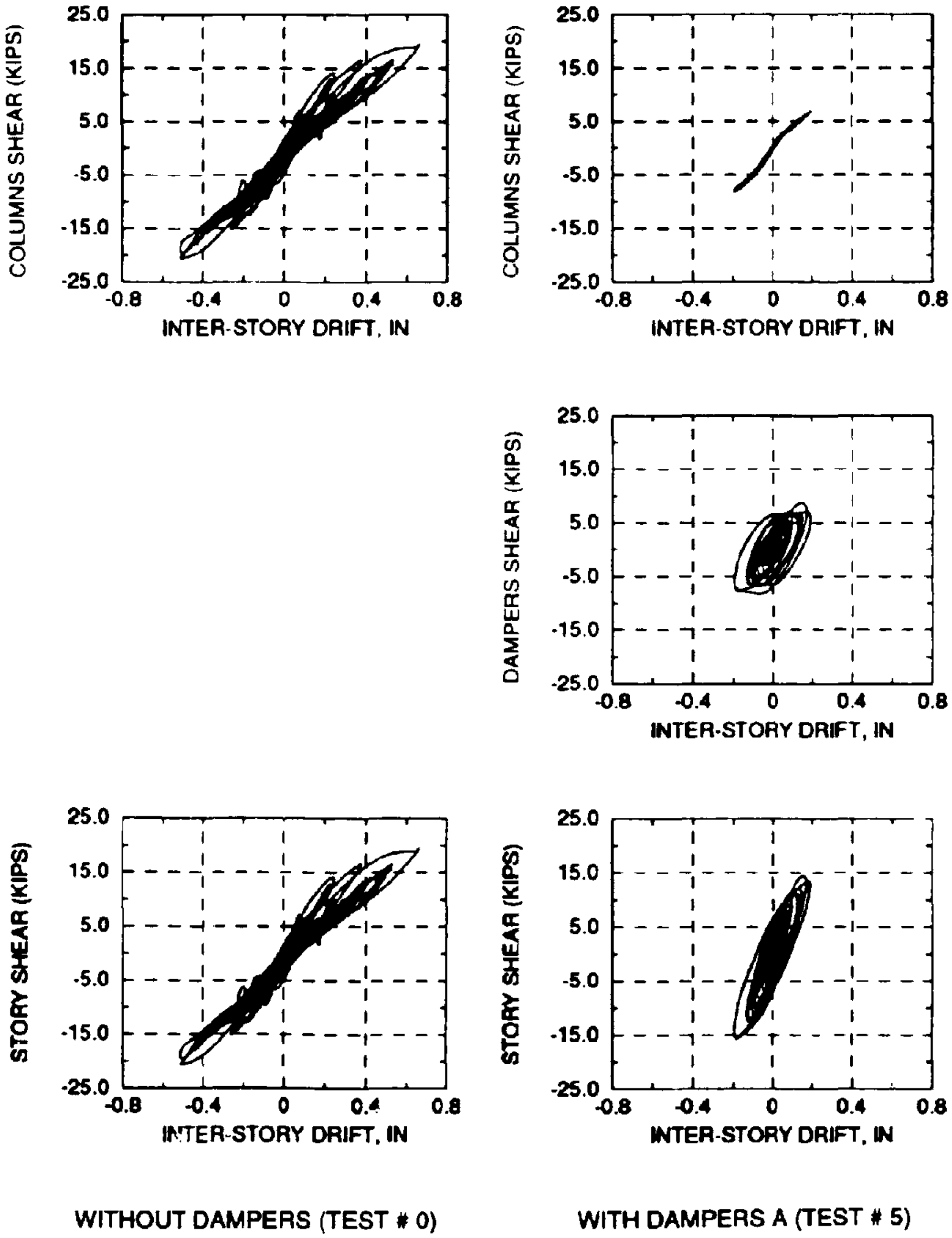


FIGURE 4-3 Forces-Deformations at First Floor with Dampers A for (PGA 0.2g)

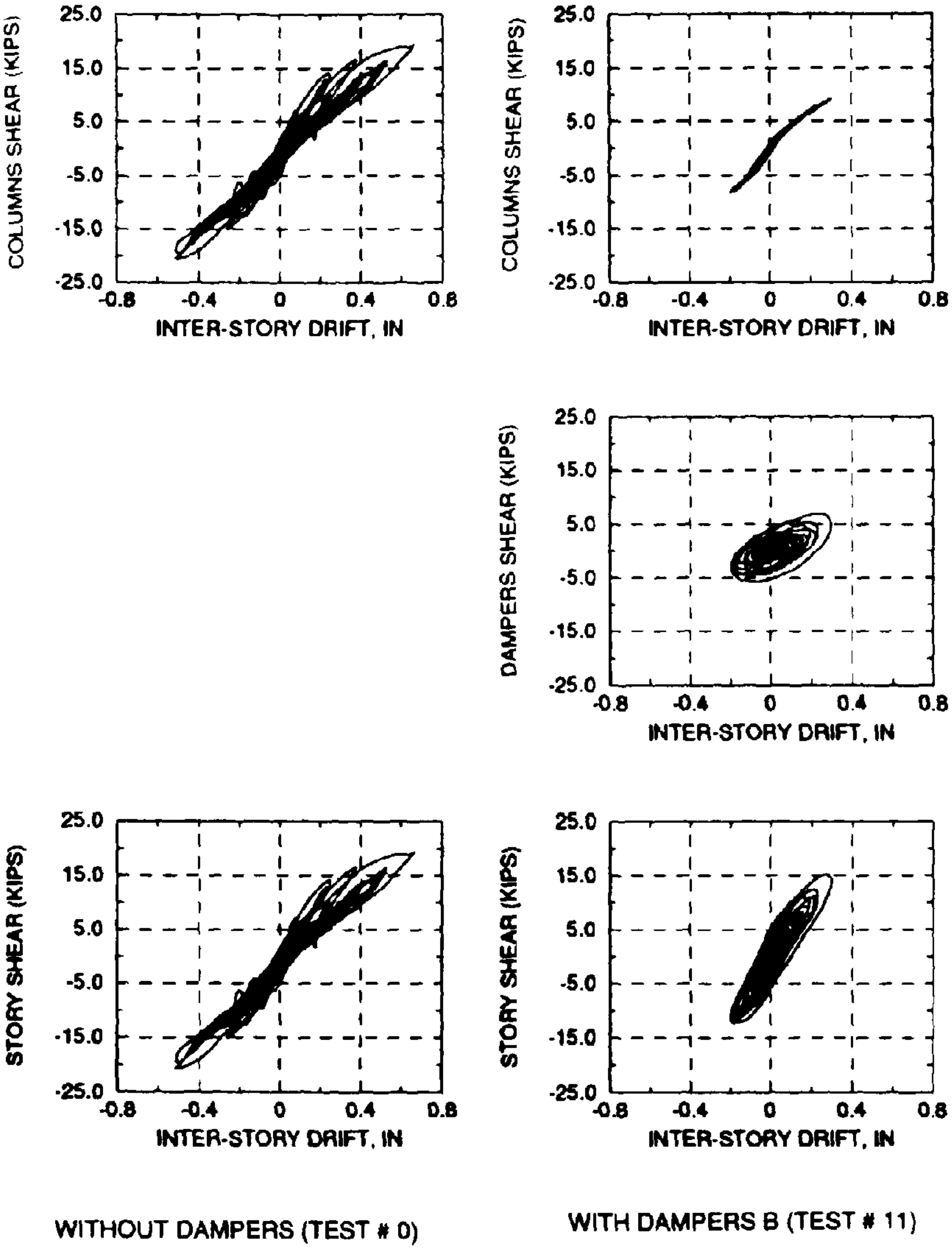


FIGURE 4-4 Forces-Deformations at First Floor with Dampers B for (PGA 0.2g)

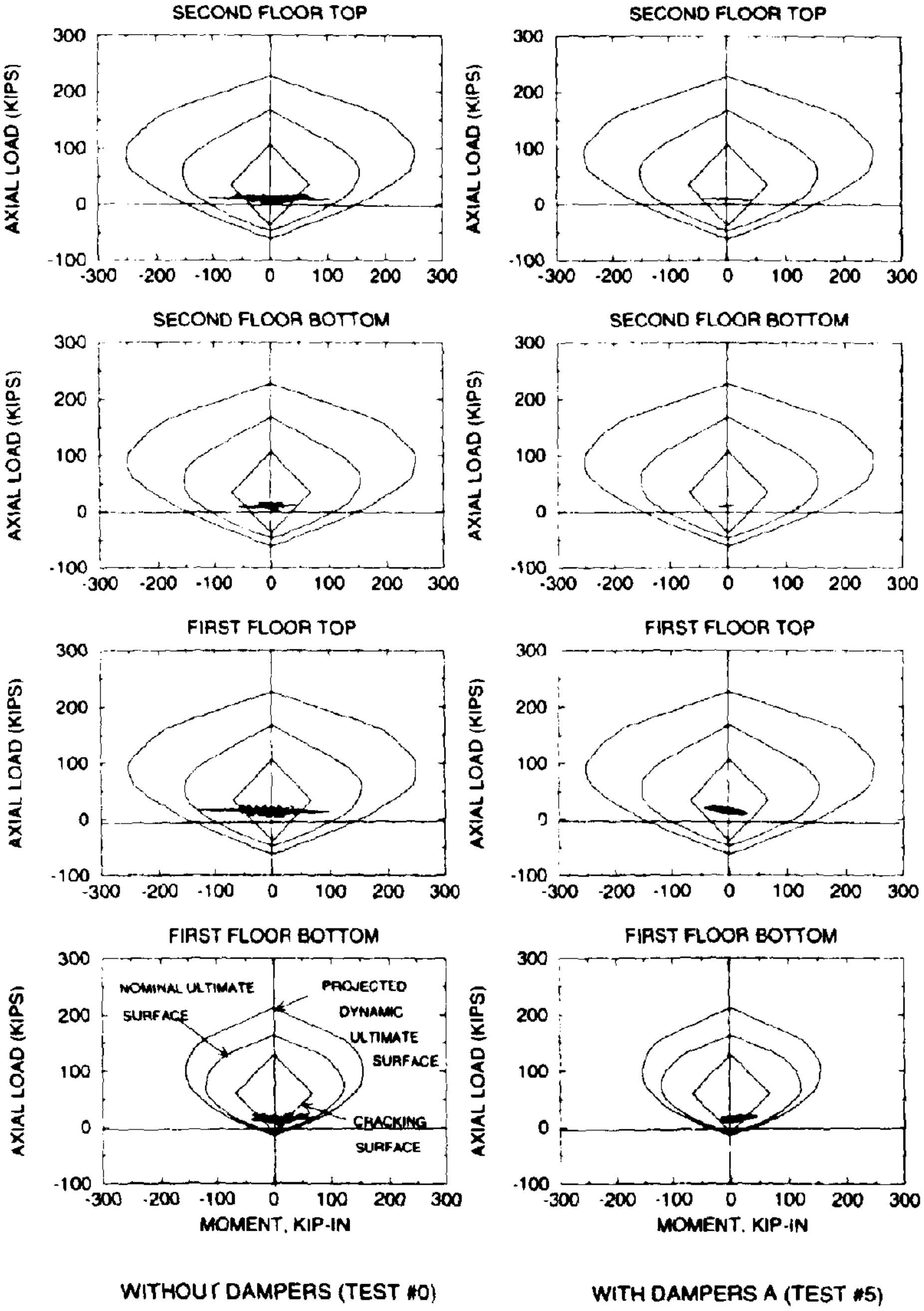


FIGURE 4-5 Capacity Diagrams versus Force Demands in Interior Columns with Dampers A

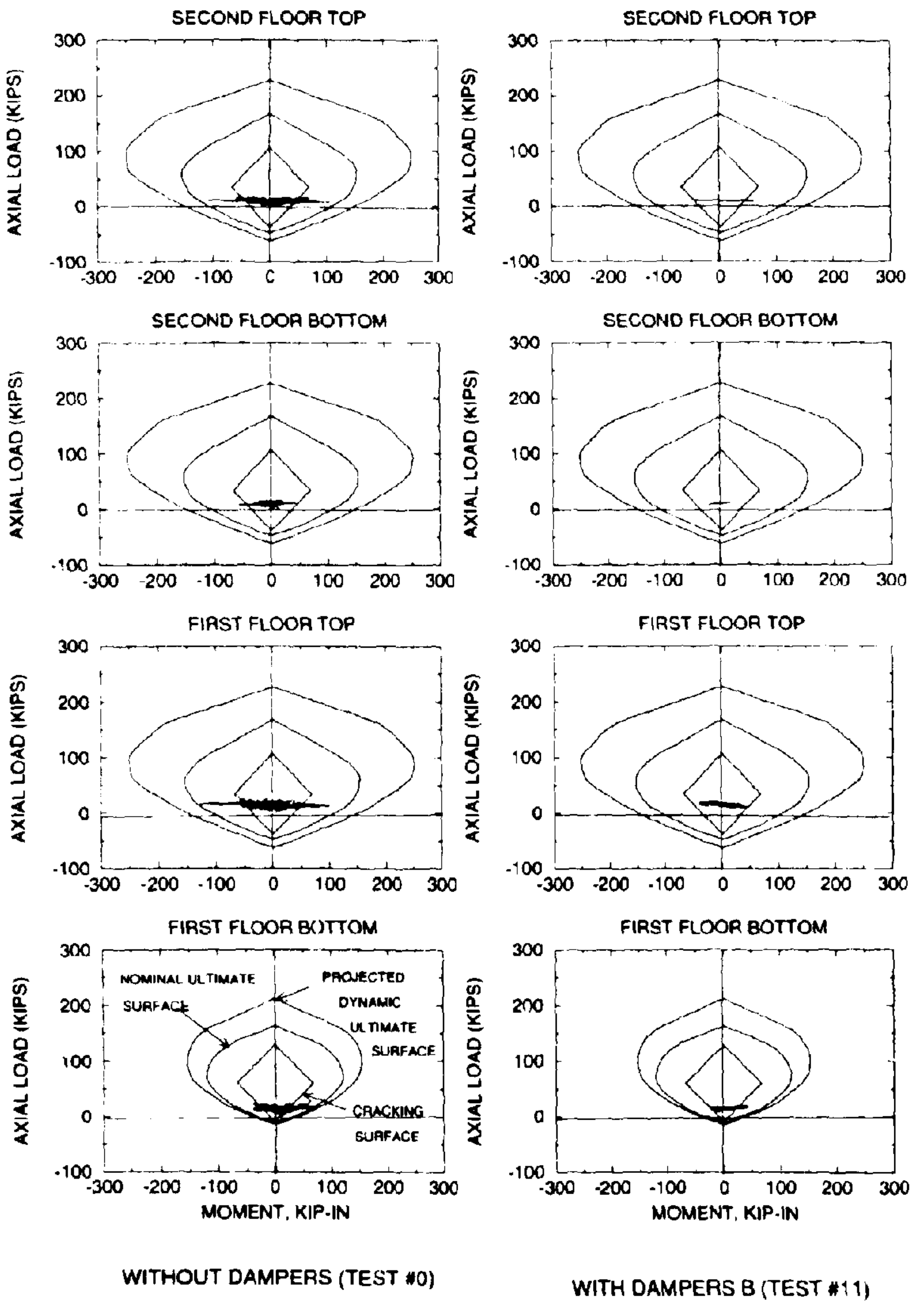


FIGURE 4-6 Capacity Diagrams versus Force Demands in Interior Columns with Dampers B

TABLE 4-2 Maximum Measured Story Response

	Inter-Story Drifts, (in.)			Column Story Shears, (kips)		
	First	Second	Third	First	Second	Third
(1)	(2)	(3)	(4)	(5)	(6)	(7)
(a) For White Noise Excitation						
No Dampers	0.047	0.034	0.018	2.38	1.53	1.33
Damper A	0.016	0.013	0.008	0.97	0.65	0.46
Damper B	0.019	0.017	0.008	1.25	0.84	0.52
(b) For Taft 0.2g Excitation						
No Dampers	0.656	0.388	0.167	20.63	16.20	10.71
Damper A	0.194	0.147	0.066	7.68	5.71	4.19
Damper B	0.297	0.196	0.097	9.47	8.25	4.67

is quite significant with the addition of dampers. However some inclination of the trajectory is noticed, more significantly in the first story columns. This indicate some variation of the axial load. Although insignificant in this test case, increase in axial forces might lead to exceedence of capacity envelope. Care should be taken in the design of columns with dampers such that the axial load/ moment demand do not intersect with the failure envelope. This could be of serious concern especally in the design of taller structures, where the axial load gets accumulated at the base.

4.4 Energy Input

The effect of the viscoelastic dampers is more evident in the distribution of the energy input throughout the structural system. Assuming that the energy balance (Uang and Bertero, 1990) at each time step in any structure is:

$$E_k + E_p + E_H + E_\xi = E_I \tag{4.1}$$

where E_k is the kinetic energy, E_p is the elastic/potential energy, E_H is the hysteretic energy dissipated by the structural system, E_ξ is the viscous damped energy, and E_I is the total energy input. The hysteretic energy (E_H) is usually associated with the permanent damage in the structural system. A reduction of this energy can result in a reduction of damage.

The addition of dampers adds another term to the energy balance:

$$E_t + E_H + E_\xi + E_{VE} = E_I \tag{4.2}$$

where E_{VE} is the energy dissipated by the added viscoelastic dampers and E_E is the elastic-kinetic energy ($E_k + E_p$).

The viscoelastic dampers alter the overall energy input balance as shown in Fig. 4-7. For the earthquake used in the experiment (Taft 1952), shown in Figs. 4-7a,b the total input energy is increased primarily due to stiffness increase. However the added viscoelastic dampers dissipate the majority of this energy, leaving only a small amount of hysteretic energy to be dissipated by the structure. In the structure without dampers, the majority of input energy is dissipated in form of hysteretic energy by the structural components, that are actually damaged. Similar pictures are obtained analytically for other earthquakes (see Fig. 4-9), although the overall energy input may vary depending on the match between the structural frequencies and the earthquake frequency content.

4.5 Damage Mechanism

The amount of damage to the individual members, story levels, and overall structure from seismic excitations can be described analytically in terms of damage indicators defined as damage indicies. These damage indicies are used to evaluate the extent of damage on a scale representing minor, moderate, or severe damage. Damage index models have been developed to incorporate effects of ductility demand and low cycle fatigue or strength deterioration by Park et al. (1985), Chung et al.

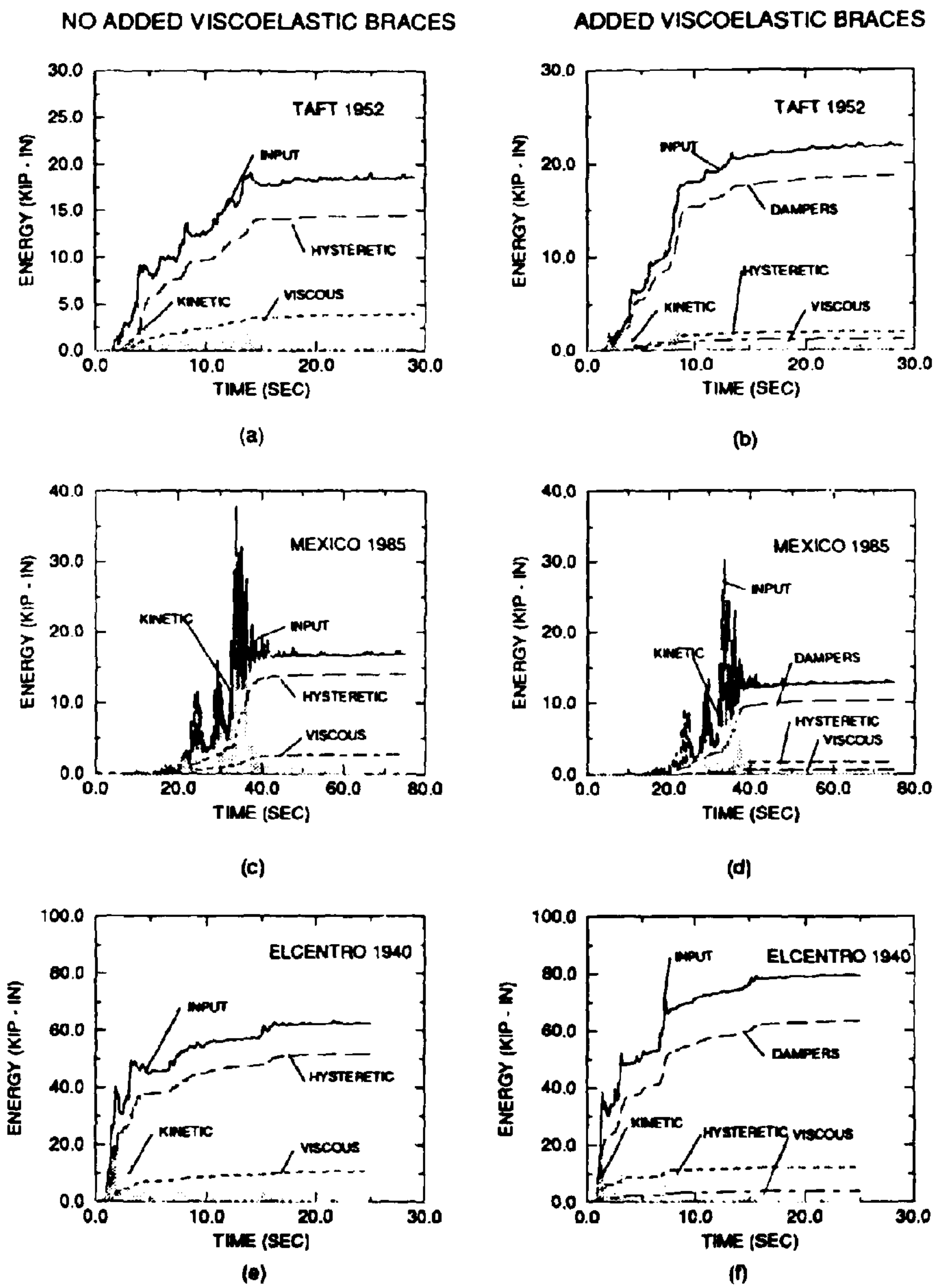


FIGURE 4-7 Energy Input in the Test Structure

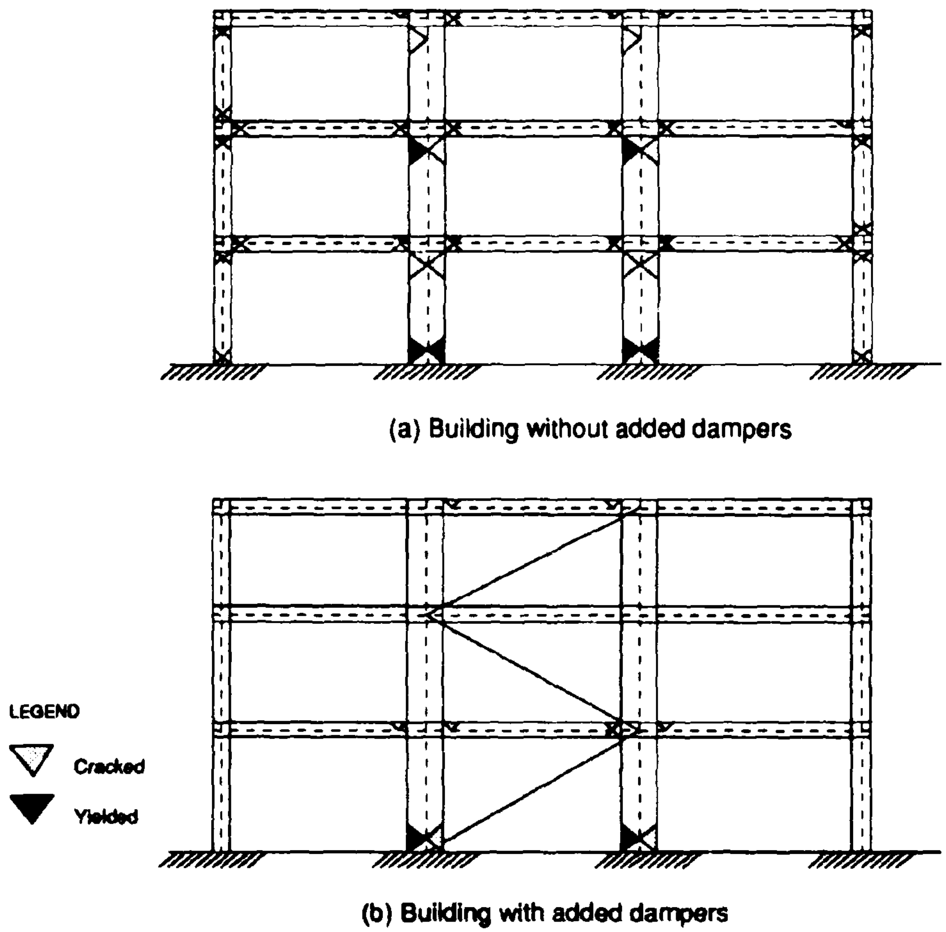
(1987), Powell et al. (1988), and Bracci et al. (1989). It has been shown, that a combination of deformation and strength deterioration damages provide an accurate assessment of the member damage and of the remaining reserve capacity. Such a damage model is used here to verify the structure performance and dampers. This model is a modified version of the Park and Ang's model [Kunnath et al. 1990] expressed in terms of moments and curvatures of structural members. The expression for this damage index is given by:

$$DI = \frac{\phi_{max}}{\phi_{ult}} + \frac{\beta \int dE}{\phi_{ult} M_y} \tag{4.3}$$

where ϕ_{max} = maximum observed curvature, ϕ_{ult} = ultimate curvature, β = strength deterioration factor, $\int dE$ = absorbed hysteretic energy, M_y = yield moment. A procedure for determining the ultimate curvature in both columns and beams was proposed by Bracci et al. (1989), with the damage index formulated to vary between 0 and 1. The extent of damage to the structure is determined from the following damage index table.

$DI = 1.0$	Collapse
$0.66 \le DI < 1$	Severe - "Irrepairable" Damage
$0.33 \le DI < 0.66$	Moderate - "Repairable" Damage
$0.0 < DI < 0.33$	Minor - "Serviceable" Damage

The structure with viscoelastic dampers experiences a reduced number of plastic hinges and cracks when subjected to the same earthquake motions (see Fig. 4-8). In fact, only minor cracks and some unavoidable base column hinging can be noticed. The damage configuration (hinging) does not indicate development of either the column-sidesway or beam-sidesway collapse mechanisms. The actual story damage evaluated using the above model is shown in Fig 4-9. It indicates the efficiency of the added braces to limit the damage to less than half of that developed in the original unretrofitted structure.



(a) Building without added dampers

(b) Building with added dampers

FIGURE 4-8 Mechanism Formed in the Building

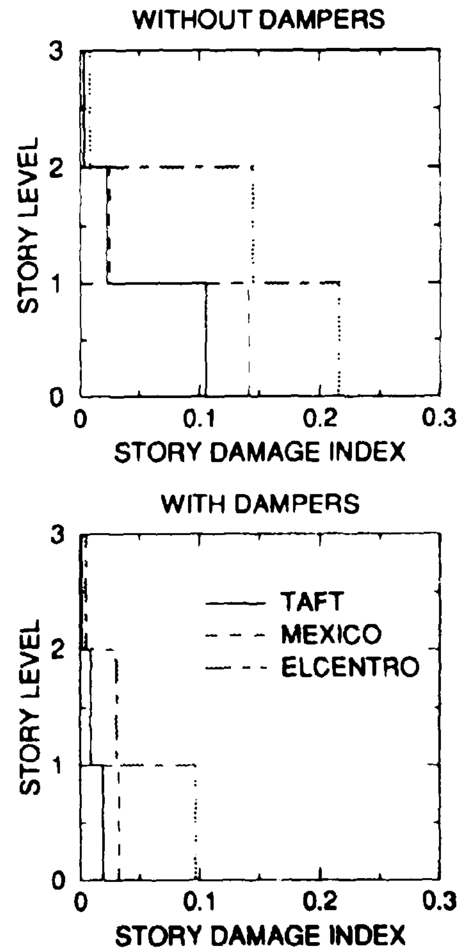


FIGURE 4-9 Story Damage Evaluation

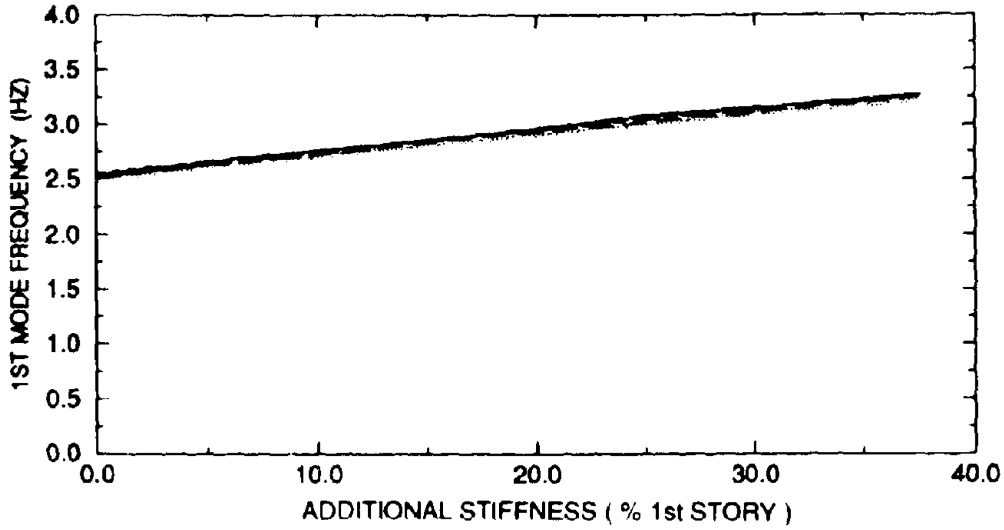
SECTION 5

INFLUENCE OF VISCOELASTIC PROPERTIES ON SEISMIC RESPONSE OF R/C STRUCTURES

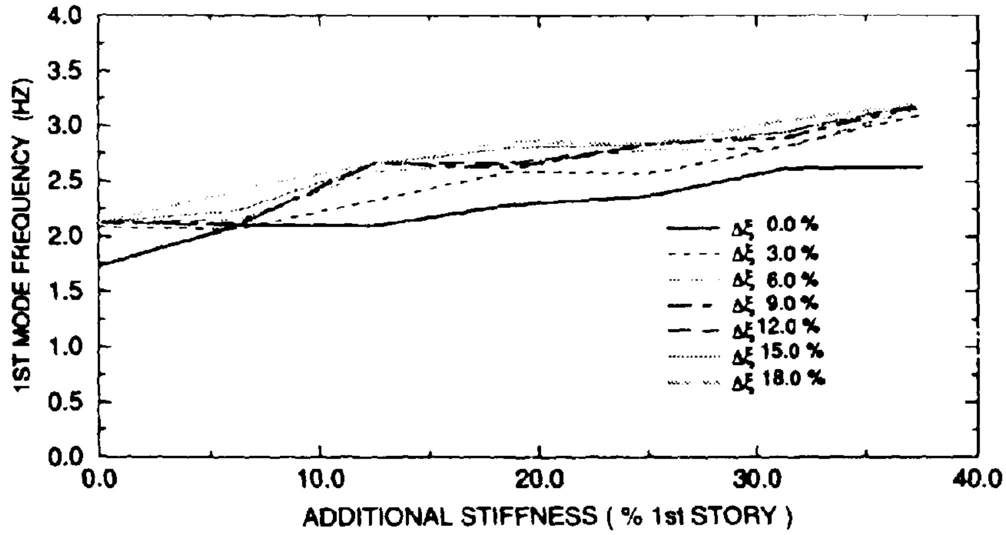
The 1:3 scale model structure described in the previous section is further used as the subject in an analytical evaluation for studying the effects of increasing either the viscous properties, or elastic stiffness properties, or both of the above, for seismic retrofit of reinforced concrete structures using viscoelastic materials.

It is well known that increased viscous properties in an elastic structure (ie. increase in the equivalent critical damping ratio) contributes to a reduction in the dynamic response amplification as shown in Fig. 5-1a. It is also known that a structure responding inelastically experiences a softening effect or a reduction in its fundamental frequency (see Fig. 5-1b). The effect of increasing the viscous properties is more drastic in an inelastic system, since it limits the decrease of the fundamental frequency to a stable level not far below its elastic value (see Fig. 5-2).

Viscoelastic dampers have also a substantial contribution to the initial stiffness of the structure. The added stiffness supplied by viscoelastic braces increases the first mode natural frequency in an elastic superstructure as shown in Fig. 5-3a, while the viscous properties have a small effect. In reinforced concrete structures experiencing inelastic deformations, the additional stiffness increase the natural frequency, only if substantial damping is added to the structure. Otherwise the tendency of stiffness softening during inelastic response will almost compensate for the increased stiffness due to addition of dampers. It should be noted that while the stiffening effect may lead to better control of lateral deformations, the same stiffening may lead to larger forces produced during various ground motions. In such cases, the positive effect of added damping might be diminished by the stiffening effect.



a. Elastic Superstructure

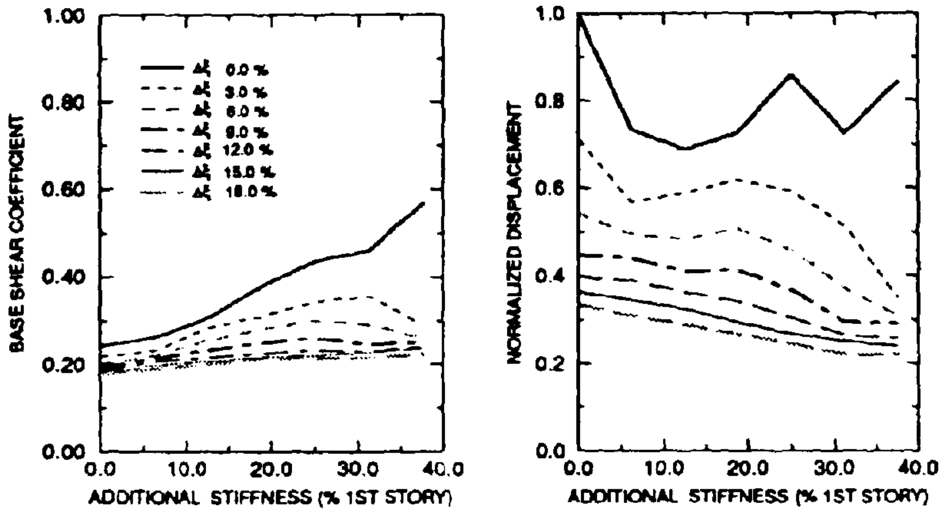


b. Inelastic Superstructure

FIGURE 5-3 Variation of First Mode Frequencies

The effect of viscoelastic properties is best summarized in Fig. 5-4 which shows the influence of increasing viscous properties and stiffness on the base shear and story displacement response of the structure. For the test type excitation, ie Taft 1952, the base shear increases almost 3 times due to 40% additional stiffness in the braces, if no viscous damping is added. However with the addition of more than 12% damping, the base shear is reduced independently of the stiffness increase (see Fig. 5-4a). It is worthwhile noting that the displacements are reduced somewhat by the stiffness increase, but the major reduction comes from the viscous properties of the braces, that increase the damping. The variation of response characteristics was obtained for the 1952 Taft ground motion. This particular ground motion produced substantial changes in the inelastic response, more than any other motion used in the study and therefore is being thought as representative.

It is also worthwhile noting that the change in the initial stiffness alters the overall apparent critical damping ratio (obtained from the free vibration "tail" of an earthquake analysis). In an inelastic response, the hysteretic behavior generally adds to the apparent damping. However in certain cases, the overall critical damping is slightly decreased (see Fig. 5-5). This is due to the more erratic response and possibly due to inaccuracies in determining the equivalent damping during the inelastic response.



a. Base Shear Response **b. Displacement Response**

FIGURE 5-4 Influence of Earthquake on Structural Response with Viscoelastic Braces

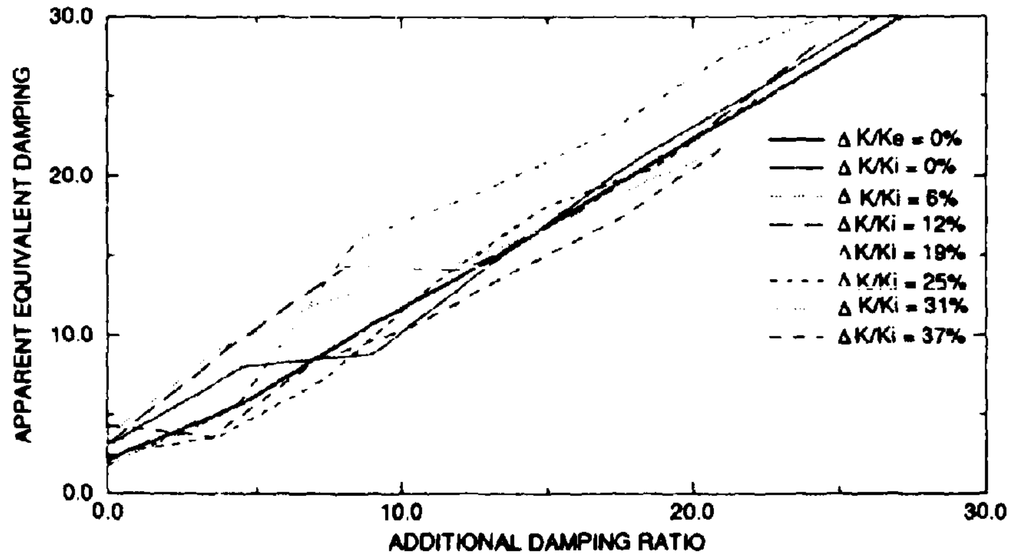


FIGURE 5-5 Apparent Equivalent Damping with Viscoelastic Braces with Added Stiffness (Percent of First Story)

SECTION 6

CONCLUDING REMARKS

The response of reinforced concrete structures, in general, and those that already suffered previous damage can benefit from the strengthening using viscous or viscoelastic dampers. The addition of substantial damping in many cases offsets the negative effect that might be caused from the stiffening of the system.

The analytical studies of reinforced concrete structures under various earthquake motions indicates that an increase of damping to an overall ratio of 15% or larger will produce effects that will outweigh the stiffness increase associated with viscoelastic dampers. These studies show an excellent benefit of increasing only the viscous damping, which can be obtained using other types of dampers such as liquid silicon dampers (Constantinou et al., 1992, Reinhorn et al., 1993).

The scaled model experiments and this analytical study indicate that retrofit using viscoelastic dampers can reduce the overall response, but more importantly, can reduce the risk of developing a damaging mechanism near collapse. In particular, the hysteretic energy dissipation is transferred from the load bearing elements, such as the columns or beams, to non-load bearing devices that can dissipate energy without damage.

This paper presents a simplified analytical model of viscoelastic braces that can be used in conjunction with a step-by-step dynamic analysis used for reinforced concrete structures. The model was verified by shaking table tests that emphasize the adequacy of the simplified modeling.

Finally, the analytical platform for evaluation of damage in R/C structures with viscoelastic dampers presented herein can also analyze more complicated damping devices that can be represented by alternative viscoelastic or hysteretic models. Due to its step-by-step solution characteristics, variable damping characteristics can also be considered. Dampers with such characteristics were proposed for further improvement and control of seismic response in structures (Reinhorn et al., 1993).

SECTION 7

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APPENDIX A

A 1-1 Reinforcement Details

The following provides details of the reinforcing steel used in the model based on scale factor of 3 for geometric length similitude. Detailed information is presented by Bracci et al., (1992a), but is repeated here for sake of completion of this report.

The slab steel in the prototype structure was designed by the direct design method of the ACI 318/83. The design required #3 rebars at 6 in. spacing in different sections of the slab. To avoid excess labor in the construction of the 3-story model, a 2 in. square mesh composed of gauge 12 galvanized wires is chosen for acceptable similitudes of strength and geometric spacing length. Since the slab strength is not the main emphasis for this study, the slight disparities of slab steel placement due to the mesh are considered satisfactory for the experiment. Figure A-1 shows the layout details for the top and bottom reinforcing steel mesh in the slab. The longitudinal (direction of motion) and transverse (perpendicular to the direction of motion) beam reinforcement details for the model are shown in Fig. A-2. Figure A-3 shows the reinforcement details for the columns in the model based on the prototype design.

A 1.2 Model Materials

The following outlines the materials used in the construction of the model. It is to be noted that the materials used in the model are identical to materials in assumed prototype structure (Bracci et al., 1992 a). Therefore the scale factors were appropriately developed based on the principles of modeling the same acceleration and material.

A 1.2.1 Concrete properties

The concrete mix analysis and design was based on trial mixes from various recipes and a design mix was established for a 28 day target strength of 3500 psi, slump of 4 in., and maximum aggregate size of 1/2 in (#1 crushed stone). Table A-1 shows the mix formula for a one cubic yard batch of concrete.

The mix formulation is based on a saturated, surface dry concrete sand. The water : cement (: sand : stone) ratio is 0.5 : 1.0 (: 3.0 : 3.6). The full gradation analysis of the aggregates in the concrete mix is shown in Fig. A-4.

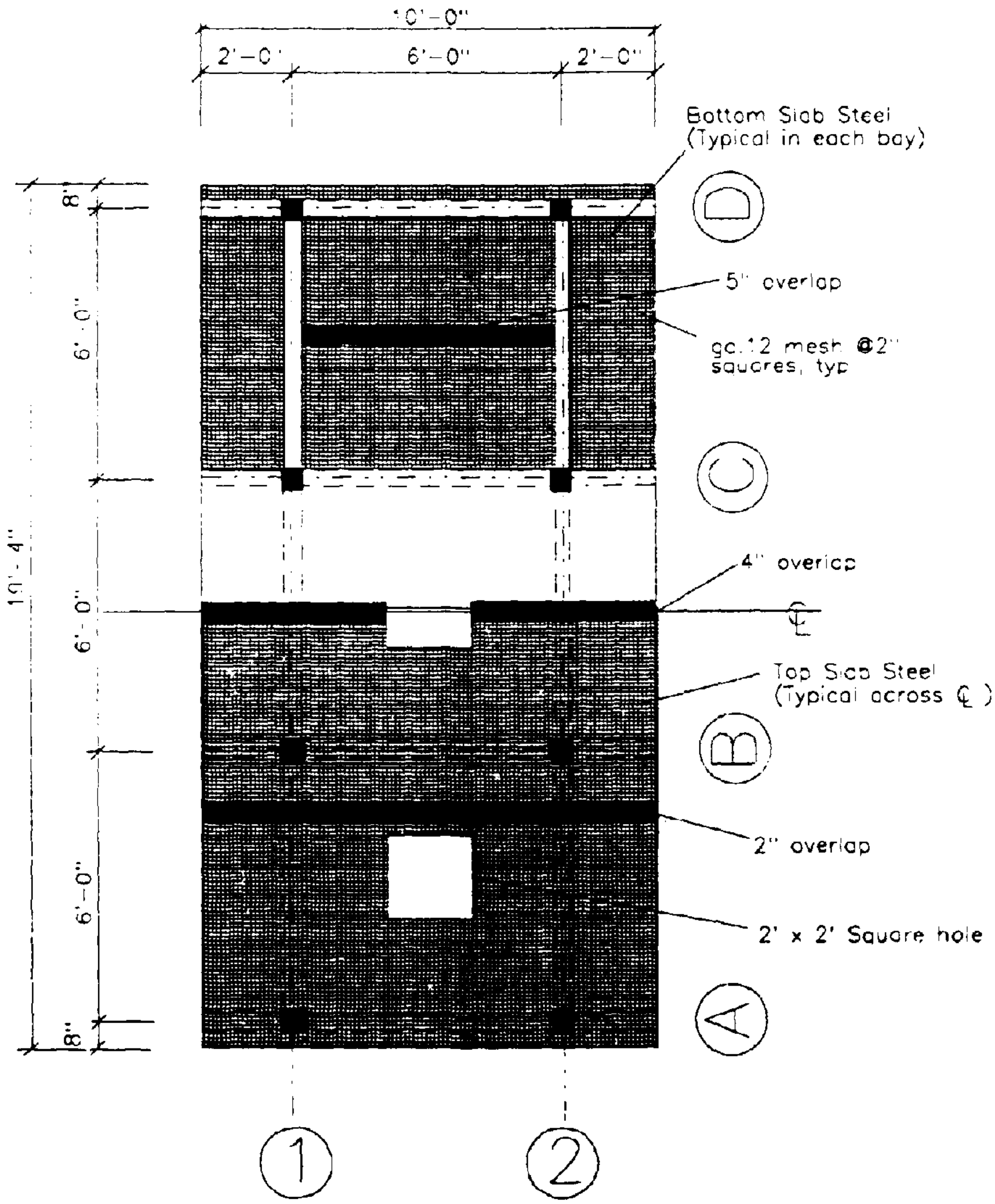
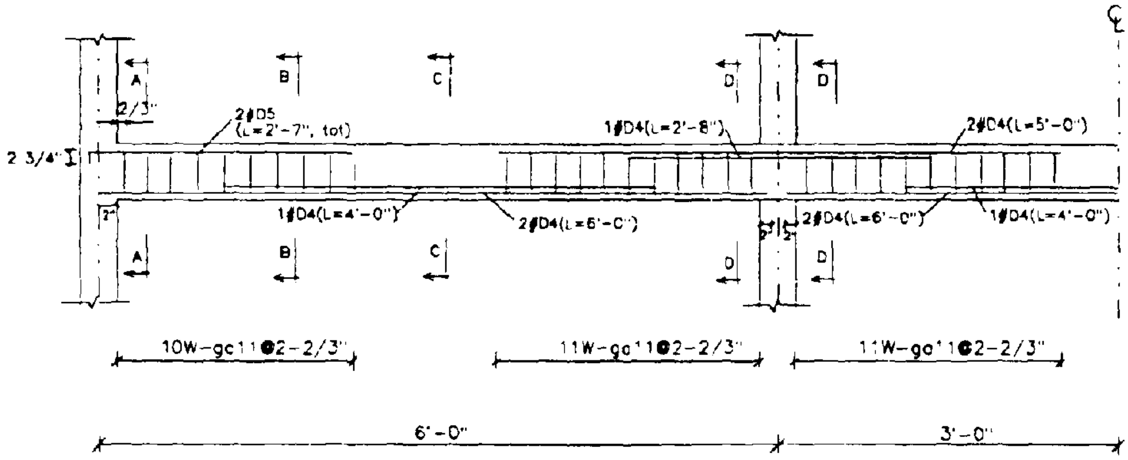
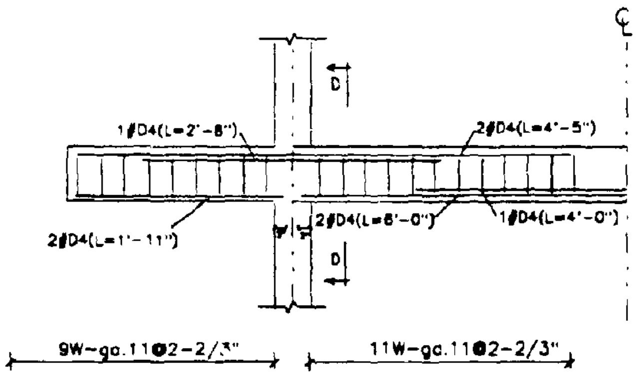


FIGURE A-1 Layout of Slab Steel Reinforcement

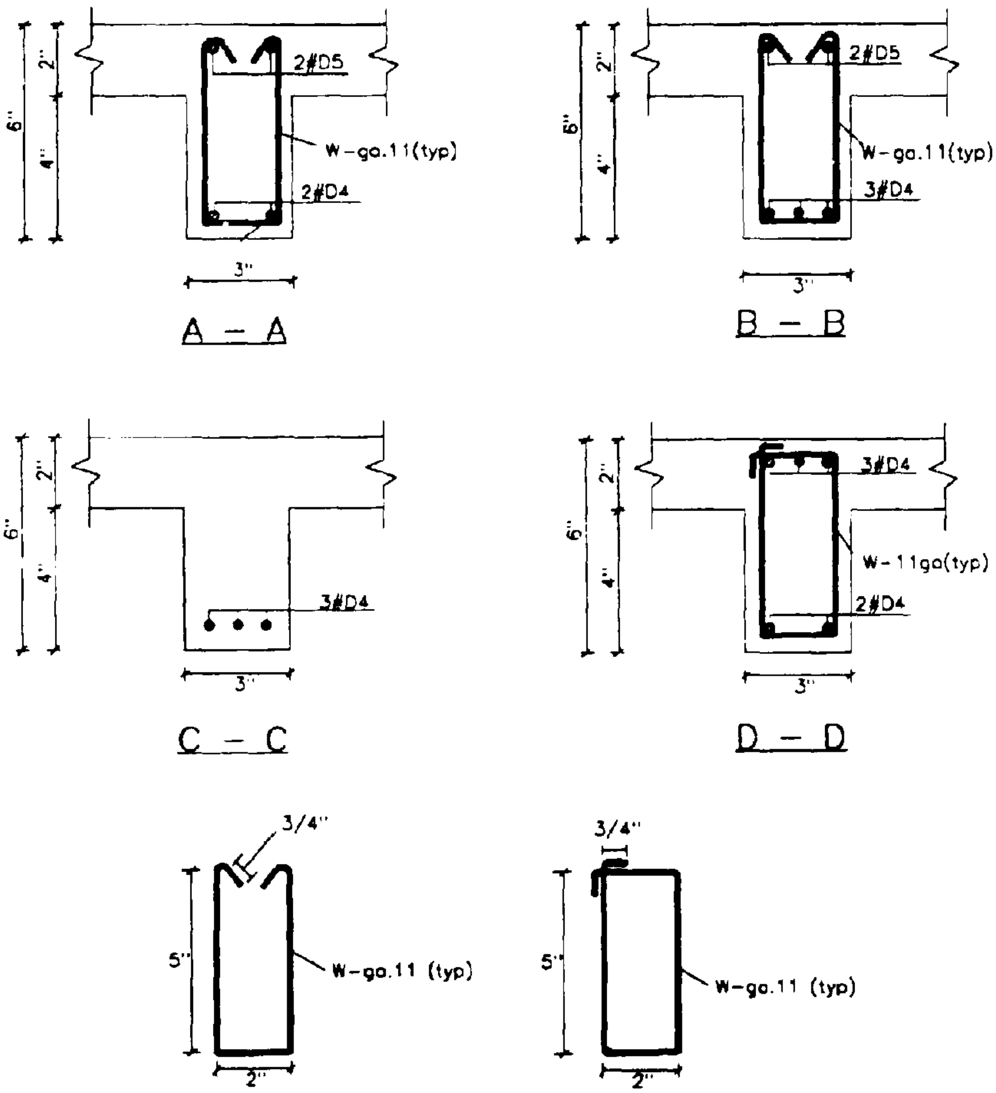


Longitudinal Beams (North-South)



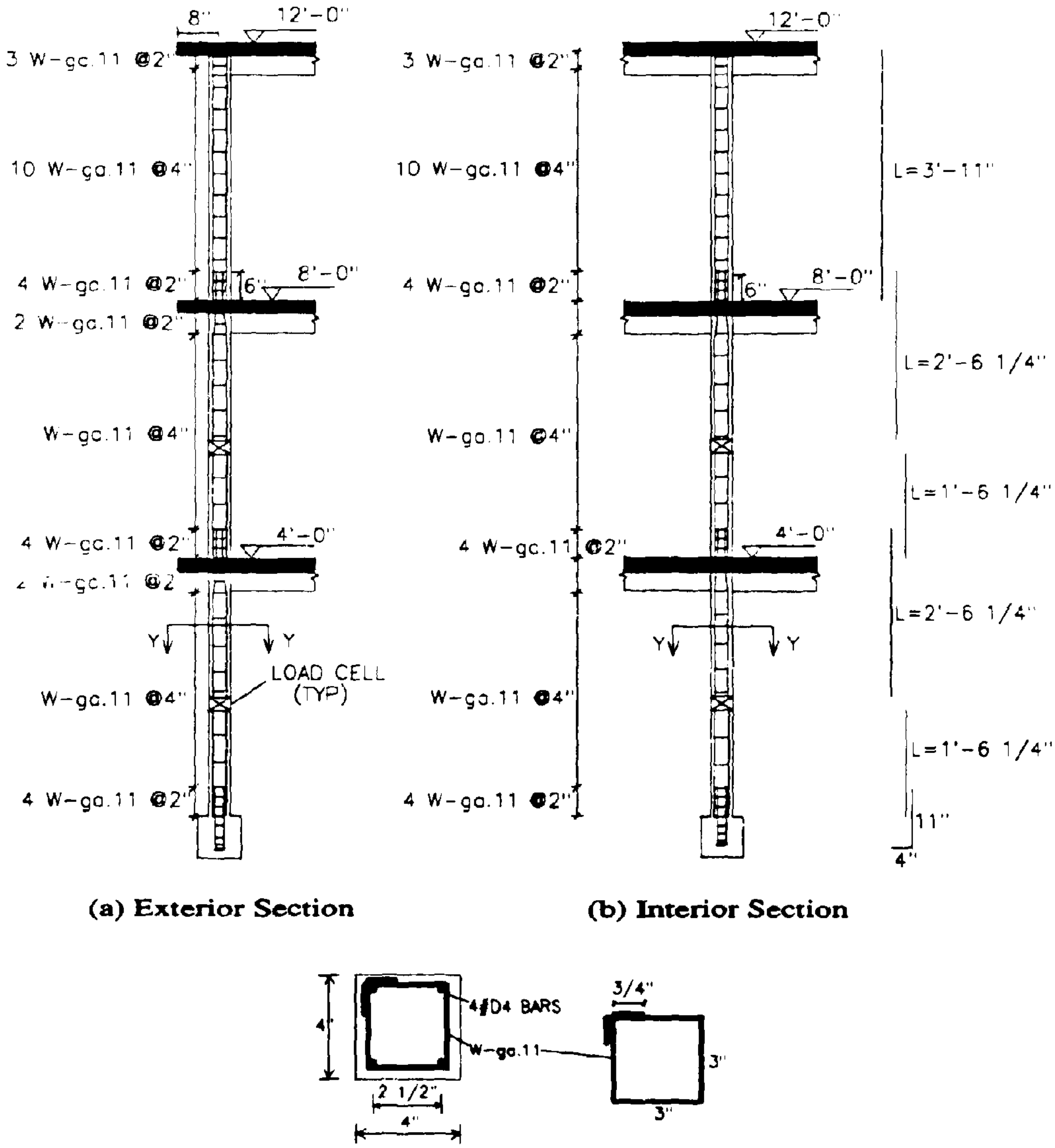
Transverse Beams (East-West)

FIGURE A-2a Details of the Beam Steel Reinforcement



Beam Sections

FIGURE A-2b Details of the Beam Steel Reinforcement (Continued)



(a) Exterior Section

(b) Interior Section

(c) Section Y-Y

FIGURE A-3 Details of the Column Steel Reinforcement

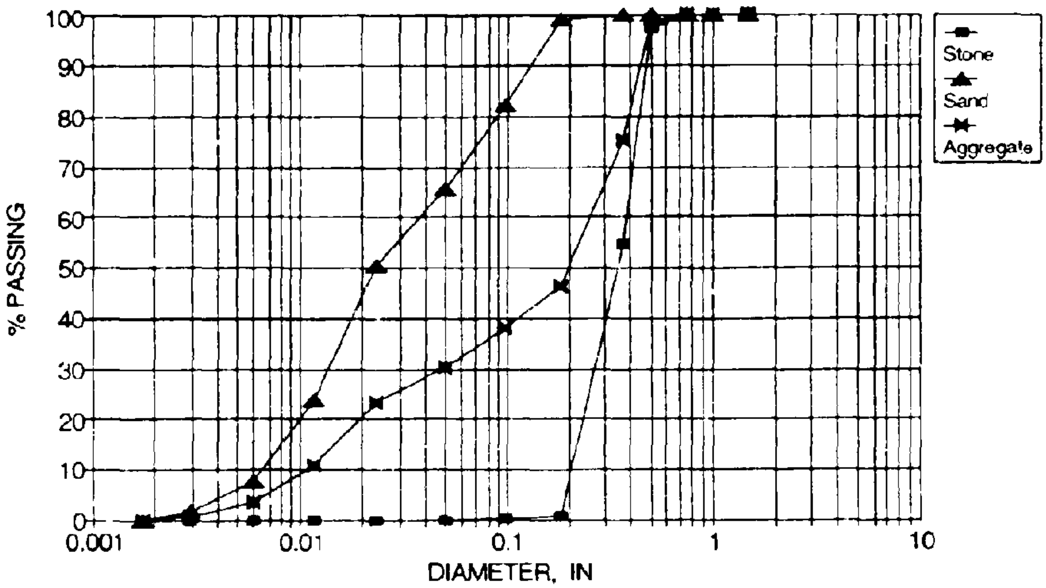


FIGURE A-4 Gradation Analysis of the Concrete Mix

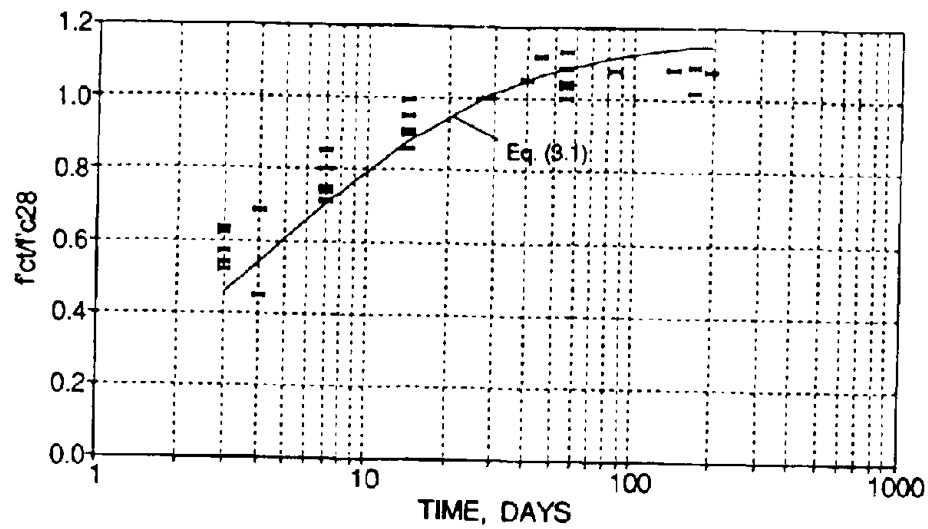


FIGURE A-5 Average Concrete Specimen Strength Versus Time

Table A-1 Mix Design Formula for the Model Concrete

Ingredient	Weight
Type I Cement	490 lb
Concrete Sand	1487 lb
#1 Crushed Stone	1785 lb
Water	242 lb
Superplasticizer	39.2 oz
Micro-Air	2.9 oz

A substantial variation can be observed in the mix strengths for the different components, even though all mixes had the same target strength (see Table A-2). The final strengths were very sensitive to moisture variations in the materials and the widely varying ambient temperatures at the time of construction. The variation of strength versus time is shown in Fig. 3-5, which indicates asymptotic stabilization of concrete strength.

Table A-2 Concrete Properties of the Model Structure

Pour Number and Location	f'_c (ksi)	E_c (ksi)	ε_{co} (strains)	ε_{spall} (strains)
1. Lower 1st Story Columns	3.38	2920	0.0020	0.011
2. Upper 2nd Story Columns	4.34	3900	0.0020	0.017
3. 1st Story Columns	4.96	3900	0.0021	0.009
4. Lower 2nd Story Column	4.36	3900	0.0026	0.014
5. Upper 2nd Story Column	3.82	3360	0.0022	0.020
6. 2nd Story Slab	2.92	2930	0.0015	0.020
7. 3rd Story Columns	3.37	3800	0.0019	0.020
8. 3rd Story Slab	4.03	3370	0.0021	0.012

The reinforcing steel uses a mix of #11 & #12 gage wires and D4, D5 annealed deformed bars. The summary of their properties is given in Table A-3

Table A-3 Reinforcing Steel Properties of the Model Structure

Bar	d_b (in)	A_b (in^2)	f_y (ksi)	E_s (ksi)	f_{max} (ksi)	ε_u
#12 ga.	0.109	0.0093	58	29900	64	0.13
	0.120	0.0113	56	29800	70	-
	0.225	0.0400	68	31050	73	0.15
	0.252	0.0500	38	31050	54	-

The D4 rebar was also annealed at different temperatures between 900^0 F and 1140^0 F to produce a yield strength between 49 and 73 ksi for yield force similitude with a #6 rebar. At a temperature of 1140^0 F, the average yield strength consistently reached was 68 ksi. Based on yield force similitude, the D4 rebar represented a #6 rebar with a yield strength of 55.6 ksi. Since a grade 40 steel has yield strengths between 40 and 60 ksi, the D4 rebar satisfied similitude with a #6 rebar. Both the original and annealed stress-strain relationships for the D4 and D5 rebars are shown in Fig. A-6.

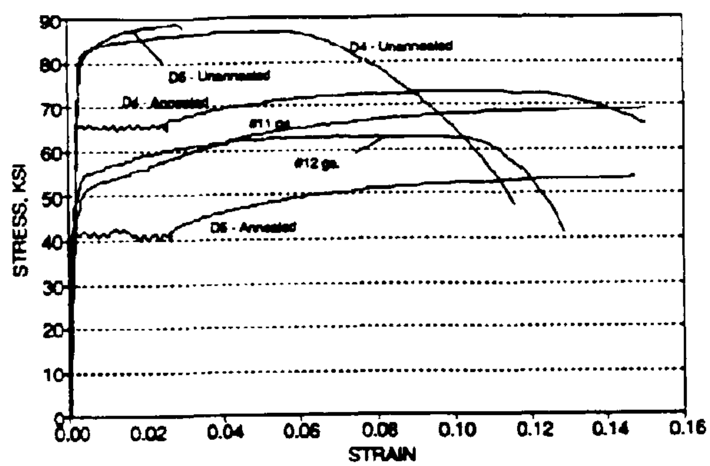


FIGURE A-6 Measured Representative Stress-Strain Relationships of the Reinforcing Steel

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