INELASTIC SEISMIC BEHAVIOR OF FRAME-WALL SYSTEMS

by

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SYNOPSIS

The frame-wall structure is first modelled as a system with a single degree of freedom in order to gain an overall view of the problem. Spectral curves for several earthquakes are presented. The influences of such parameters as the stiffness and inelastic energy capacity of the wall are illustrated by numerical results. Then an analysis of the structure with the frame considered as being made of elasto-plastic beam members and wall panels of plane stress finite elements is described. Numerical results indicate that the system could be modelled with sufficient accuracy using one degree of freedom per floor.

INTRODUCTION

Observations of the behavior of structures under recent strong motion earthquakes indicate that past and current practices have generally under-estimated both the earthquake forces on and the resistance of the structure. The paper is concerned with the second aspect of the problem, i.e., a more realistic estimate of the structural resistance. A large number of buildings are constructed of skeletal frames with various types of walls. It is generally recognized that walls, partitions and claddings add stiffness to the structure and aid in dissipating energy. But, with the exception of specifically designed shear walls, thus far the effects of walls and claddings are generally considered to be accounted for by lumping them in the damping assumed to exist in the system. The influence of their stiffness is usually neglected.

The numerical results obtained in the first part of this report indicate that by explicitly taking the wall stiffness into account the response of the frame could be reduced substantially for structures having relatively small natural periods of vibration, say, less than 0.5 sec. The results further indicate that, as a measure of inelastic response, the inelastic energy absorbed by the frame may be an appropriate alternative to the commonly used "ductility factor" related to displacement. Data are also presented showing that under certain combinations of parameters the response could be very sensitive to the limit resistance and the inelastic energy capacity of the wall.

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With modern theories of structural mechanics and computers, one could model the behavior of a complex structural system almost as accurately as desired provided that he is willing to pay the computational cost. However, even with modern computers the computational cost increases rapidly with the fineness of the modeling. The second part of the paper describes the analysis of the inelastic behavior of a frame-filler wall system using more elaborate models with special attention given to a balance between realism and computational needs. Among the results are indications that a model with one degree of freedom per floor would be sufficiently accurate for most engineering purposes.

BEHAVIOR EXHIBITED BY SIMPLE MODELS

The structure is assumed to be represented by a single mass, M, supported by two springs in parallel. The spring representing the frame is of the usual elasto-perfectly plastic type as illustrated in Fig. 1-a. To represent the wall stiffness (including its interaction effects with the frame), two types of springs are considered. Fig. 1-b shows the stiffness properties of wall Type I. It behaves linearly elastically up to a capacity, C_w. Further deformations in the same direction are governed by a negative stiffness. When the distortion reaches a value denoted by e_u, the resistance becomes zero and remains so thereafter. If the direction of the deformation reverses before it equals e_u, the wall would behave elastically with a deteriorated stiffness. Consequently, the maximum amount of inelastic energy that the wall can absorb, $\overline{U}_{w1}$, is equal to one half the product of C_w and $(e_u - e_o)$. Wall Type II has the same type of resistance-deformation relation as the frame (with C_w and e_o replacing C_f and e_f in Fig. 1-a, respectively). However, like the Type I wall, the resistance vanishes when the inelastic energy exceeds a prescribed value of $\overline{U}_{w1}$.

The system, therefore, may be characterized by the following parameters:

$\alpha = C_f/Mg$ = (frame elastic limit resistance)/(weight of structure);

$\gamma = C_w/C_f$ = ratio of elastic limit resistance;

$E_R = (1/2\, C_w e_o)/(1/2\, C_f e_f)$ = elastic energy capacity ratio;

T_f = undamped natural period based on frame elastic stiffness only;

and $U_{w1} = \overline{U}_{w1}/(1/2\, C_f e_f)$ = (wall inelastic energy capacity)/(frame elastic energy capacity).

In addition, there is a linear viscous damping force. All numerical results presented in this section were obtained by using a damping corresponding to ten percent of the critical damping based on the frame elastic stiffness alone. The Type II wall was used unless otherwise noted. Attention was focused on the response of the frame as, for inelastic behavior, it represents the last line of defense against total structural collapse. Hence, the responses are presented in terms of the following two quantities:

D/e_f = (maximum displacement relative to ground)/ (frame elastic limit displacement);

and U_{pf}/U_{ef} = (inelastic energy absorbed by frame)/(elastic energy capacity of frame).

The first quantity is also known as the "ductility factor."

Fig. 2 shows a spectral curve for the California Institute of Technology Simulated Earthquake Type C-1(1) (denoted as CIT-C1). This motion is supposed to model the shaking in the epicentral area of a Magnitude 5 or 6 earthquake. As indicated by the parameters listed, the frame has an elastic limit resistance of 5% of the weight of the structure. The wall has an elastic limit resistance equal to one half of that of the frame. The inelastic energy capacity of the wall is ten times the frame elastic energy capacity. Note that the scale for the dimensionless energy is ten times smaller than that for the dimensionless displacement or the ductility factor. The responses with the wall stiffness ignored are also shown.

It is seen that generally the wall stiffness reduces the magnitude of the responses. The reduction is substantial for smaller values of the period T_f and less pronounced for larger periods. Within a given moderate range of values of the period, the reduction of the displacement can vary from a few percent to over 100 percent, depending on the value of the period. But the reduction of the inelastic energy within the range is less dependent on the period. This tendency of less fluctuation, in a local sense, of the inelastic energy was also noted in other data. For convenience of data interpretation, the inelastic energy could be a more appropriate measure of the response than the ductility factor. A more important consideration is that the former may be more directly related to the safety of the frame than the ductility factor.

The spectral curves presented in Fig. 3 are for the CIT-B1 motion which simulates a Magnitude 7 earthquake. The frame has an elastic limit resistance equal to 20% of the weight. The responses are generally larger than those in the previous figure. The influence of the wall stiffness in reducing the displacement and energy absorbed is less than that shown in the previous figure. There are even regions where having the wall stiffness results in greater displacements. But the inelastic energy absorbed by the frame is always less when the wall stiffness is taken into account. For T_f greater than 0.25 sec. the inelastic energy decreases monotonically with increasing values of T_f while the displacement "oscillates" somewhat.

Fig. 4 shows a set of spectral curves for the El Centro 1940 earthquake, N-S component. The general characteristics of these curves are similar to those shown in the preceding figure.

Fig. 5 shows, for the system considered, how a small change in the wall elastic limit resistance could affect the frame response. Note the steepness of the response curves in the neighborhood of $\gamma = 2.5$. The curves presented correspond to $T_f = 0.5$. Such sensitivity generally increases with decreasing values of T_f, and vice versa.

For two given values of γ and a fixed value of E_R, Fig. 6 shows the effects of the wall inelastic energy capacity on the frame response. For $\gamma = 2.75$ the frame behaves elastically with $U_{wl} \geq 10$. That is, 10 is the minimum value of U_{wl} for which no inelastic work would be done on the frame. As γ reduces to 2.5, this minimum value is increased to 18. Again, the displacement oscillates somewhat while the inelastic energy behaves monotonically.

In Fig. 7 curves of the maximum displacement versus a scaling factor applied to the CIT-B1 earthquake are plotted for the system indicated. The factor may be interpreted to represent an intensity level of the ground motion. It is seen that without the wall stiffness, the maximum "safe" level would be approximately 0.4. With the stiffness of the Type I and II walls, it is approximately 0.6 and 0.75, respectively.

INELASTIC ANALYSIS OF FRAME-FILLER WALL SYSTEMS

Considerable work has been done on the dynamic analysis of elasto-plastic frames (see, for example, ref. 2). Most works on walls, however, are related to statics (3, 4, 5). One of the major problems is the treatment of cracks in a presumably relatively brittle wall.

By comparison with experimental results of static tests, it was deemed reasonable to assume that (i) a wall element would "crack" if the maximum principal stress exceeds a certain prescribed value, and (ii) after cracking, the wall element would continue to behave elastically but with a greatly reduced Young's modulus. Figure 8 shows the computed and experimental static load-displacement relations of a single story frame-filler wall system. In the analysis the wall panel was divided into an 8 x 4 mesh with 64 triangular elements. Results are plotted for two values of the reduction factor. The experimental data were taken from reference 4. The results indicate that a reduction factor of 0.01 appears reasonable. The location of the initial crack and the general pattern of crack propagation agreed well with the results reported in reference 5 (6). Similar comparisons have been made with the experimental work reported by Sachanski (3). Again using a reduction factor of 0.01 produced good agreement between the measured and calculated displacements.

Comparisons of the displacement of the top level of a rectangular panel subjected to a horizontal force at each top corner of the panel were also made by using a single rectangular finite element and using up to 32 elements. The results for using one and using 32 elements differed by less than 15%. Thus a single finite element was used to represent a wall panel.

With the mass of the system lumped at the frame joints, the structure thus has a number of degrees of freedom (DOF) equal to three times the number of free joints. A further reduction in the number of DOF is realizable by eliminating the rotational and vertical degrees of freedom of the joints.

With the above-mentioned assumptions, the top story displacement of a three story frame-wall system subject to the CIT-B1 earthquake was calculated and is presented in Fig. 9. As indicated therein, the frame has three types of members. For types 1, 2 and 3, respectively, the cross-sectional areas are (265), (200) and (176) in^2, and the moments of inertia are (5, 156), (3, 730) and (2, 195) in^4, and the yield moments are (2, 500), (1, 800) and (1, 350) in-kips. The wall panels are 6 in. thick, the limit tensile stress is 150 psi, and the Poisson's ratio is assumed to be zero. The modulus of elasticity is 2, 000, 000 psi. The structure also carries a mass load of 2.4 kips per ft. on each girder. A 10% critical damping (based on the fundamental mode) was assumed.

From the figure it is seen that after approximately six seconds of shaking the structure without the wall stiffness was considered to have failed as the first floor joints suffered a plastic rotation of (a somewhat arbitrarily set value of) 0.2 radians. The structure with the wall stiffness included behaved stably.

In Fig. 10 the history curve of the top story displacement of the structure modelled with three DOF per joint is compared with that corresponding to one DOF per floor. The two curves are sufficiently close. It seems that the simpler model would suffice for most engineering purposes. The ratio of computation time for the two models is approximately four to one.

ACKNOWLEDGMENT

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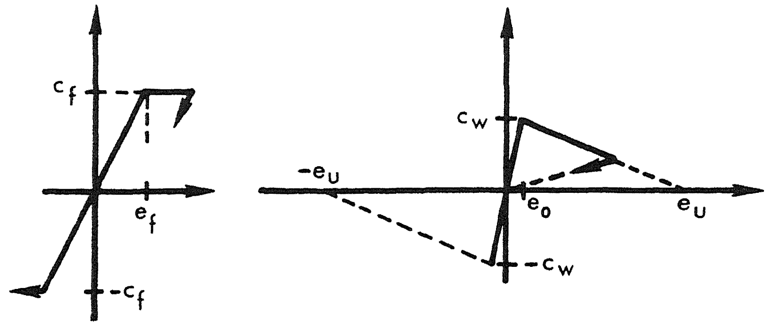

(A) FRAME RESISTANCE (B) TYPE 1 WALL RESISTANCE

FIG. 1. RESISTANCE-DEFORMATION DIAGRAMS

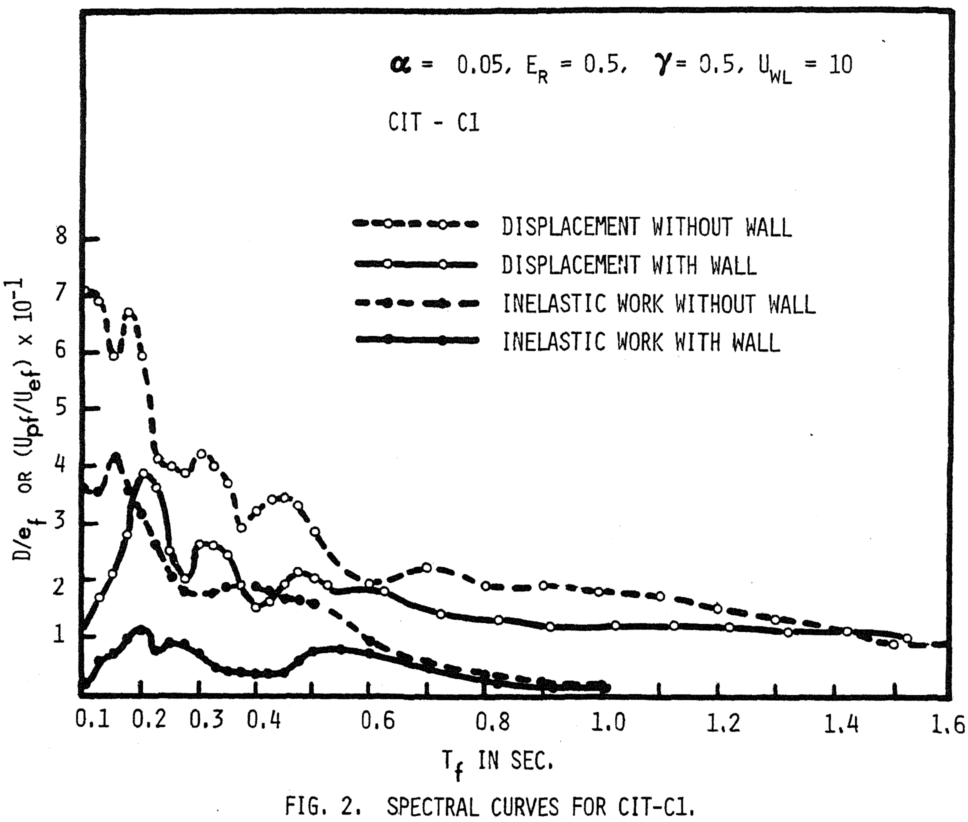

FIG. 2. SPECTRAL CURVES FOR CIT-C1.

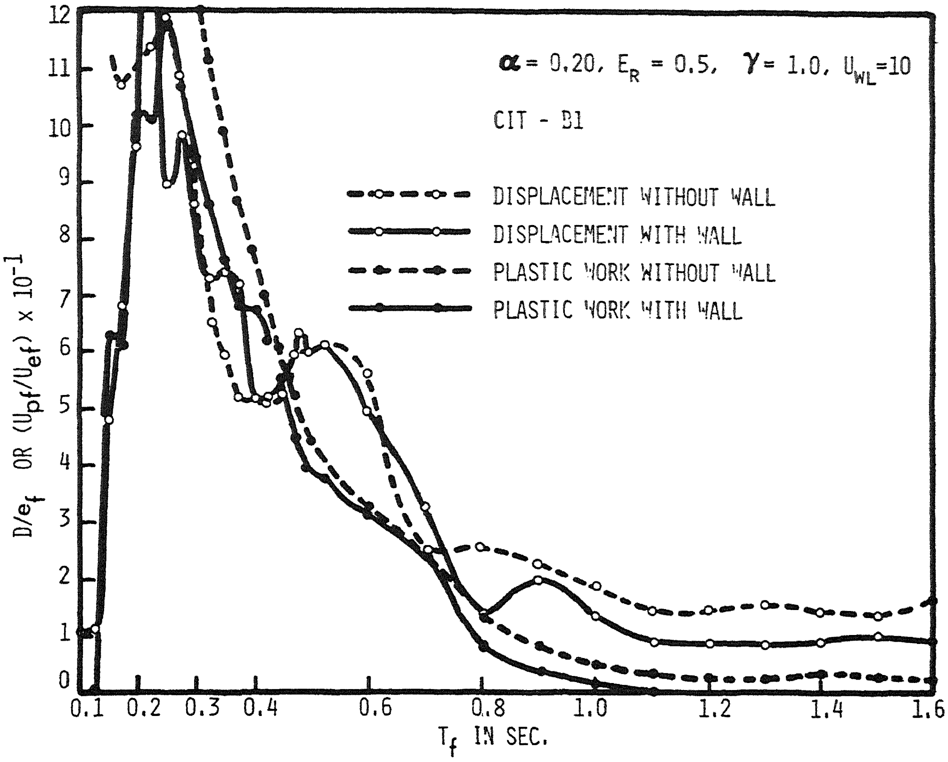

FIG. 3. SPECTRAL CURVES FOR CIT-B1.

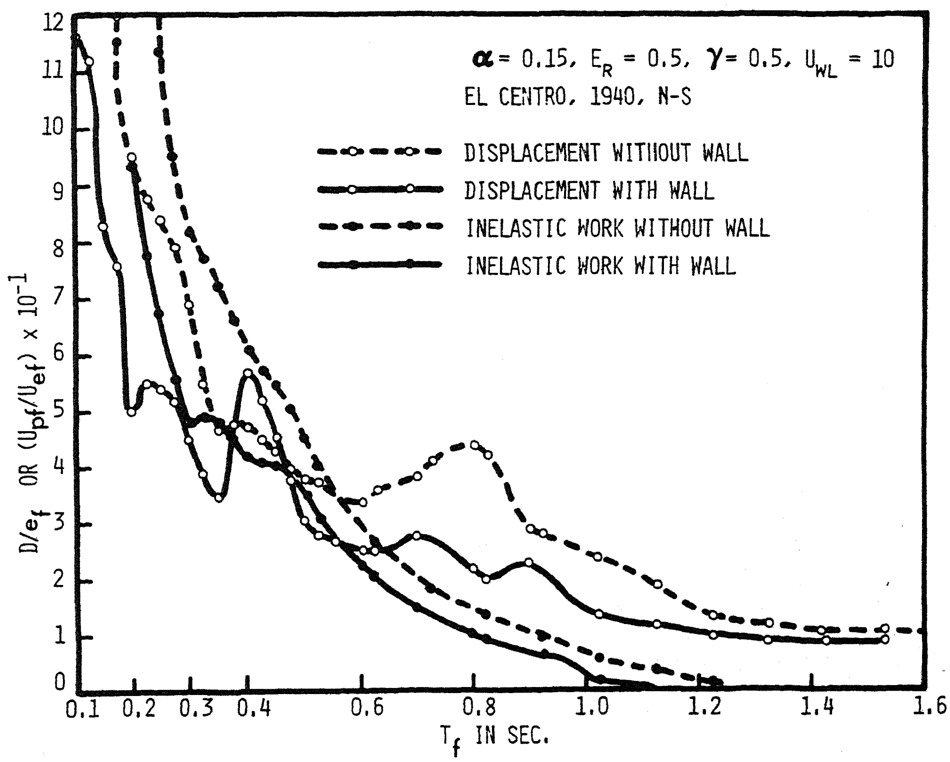

FIG. 4. SPECTRAL CURVES FOR EL CENTRO EARTHQUAKE, 1940.

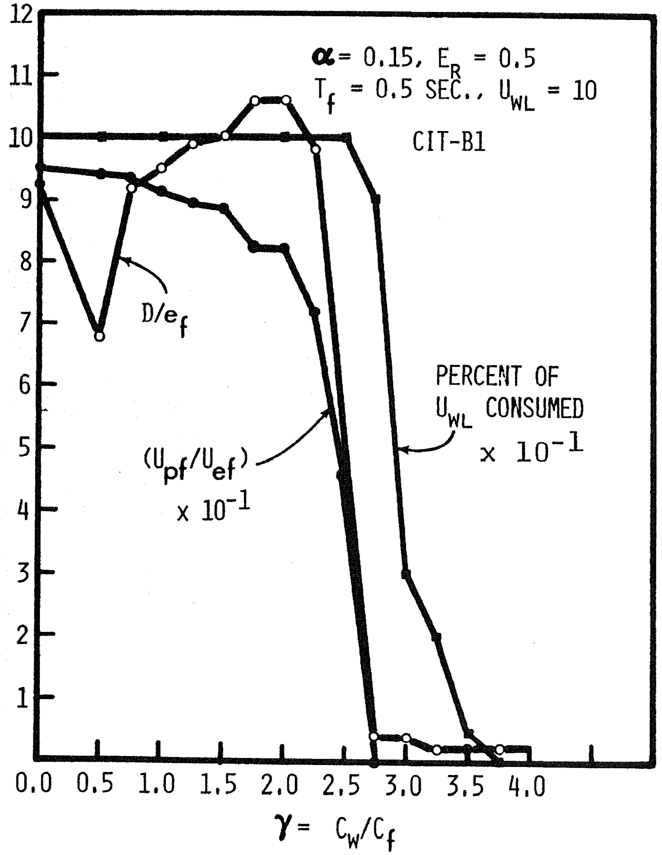

FIG. 5. INFLUENCE OF WALL STIFFNESS.

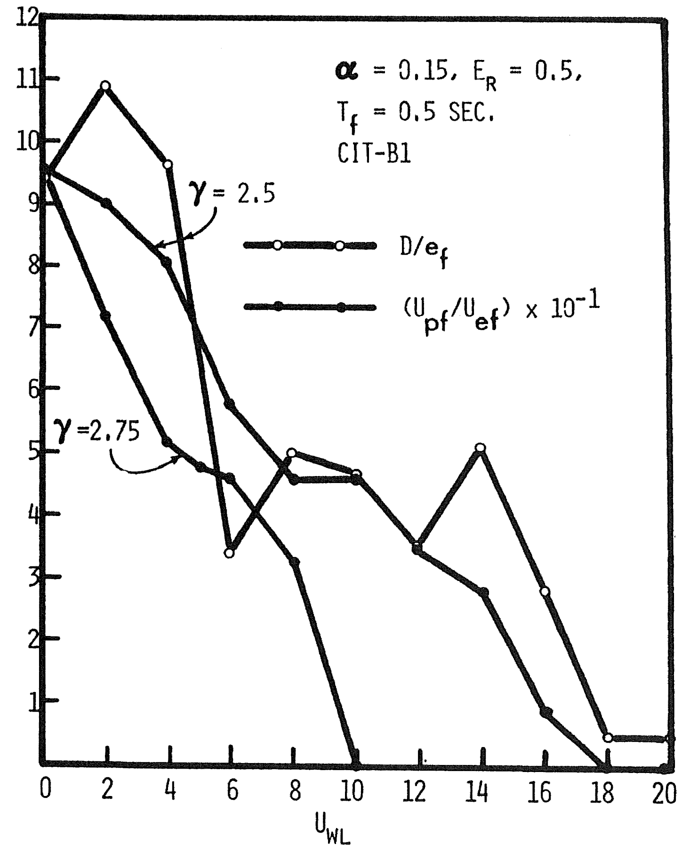

FIG. 6. INFLUENCE OF WALL INELASTIC ENERGY CAPACITY.

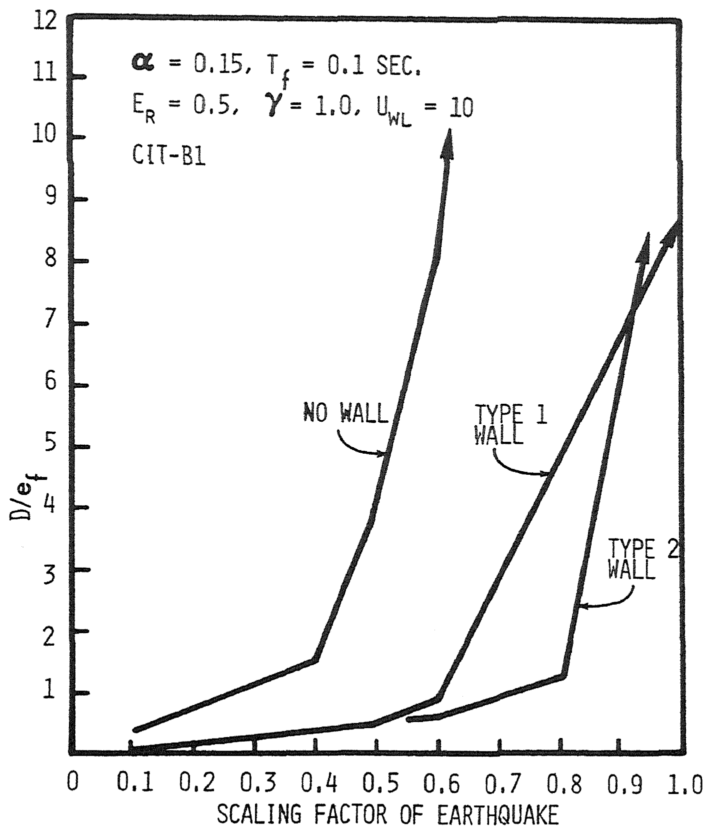

FIG. 7. INFLUENCE OF "INTENSITY LEVEL."

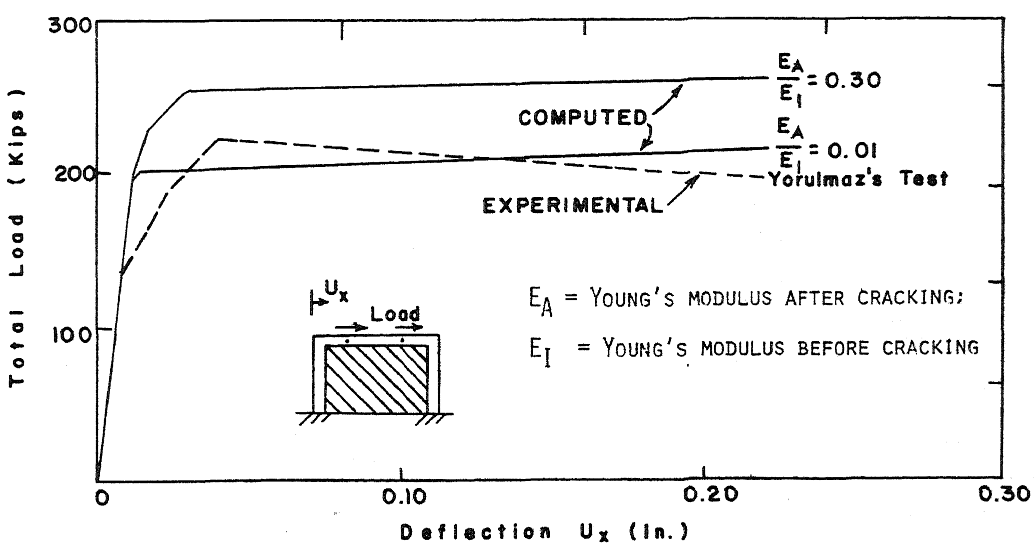

FIG. 8. COMPARISON OF COMPUTED AND MEASURED STATICAL DISPLACEMENTS.

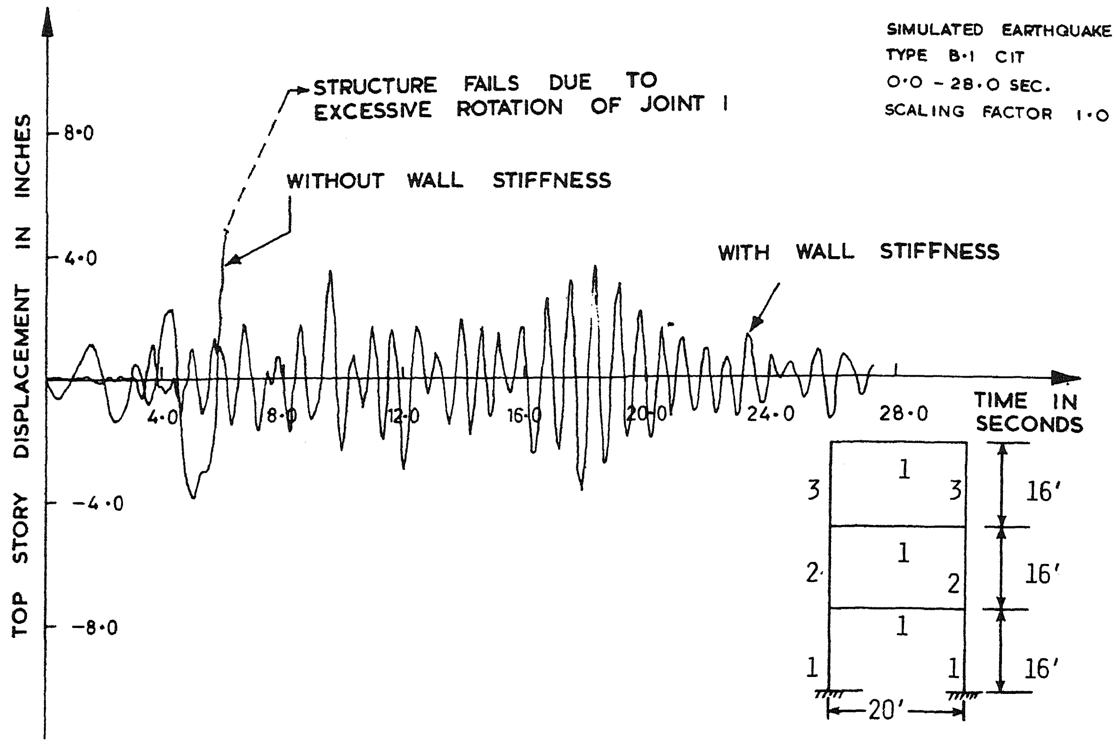

FIG. 9. HISTORY CURVES FOR A THREE STORY SYSTEM.

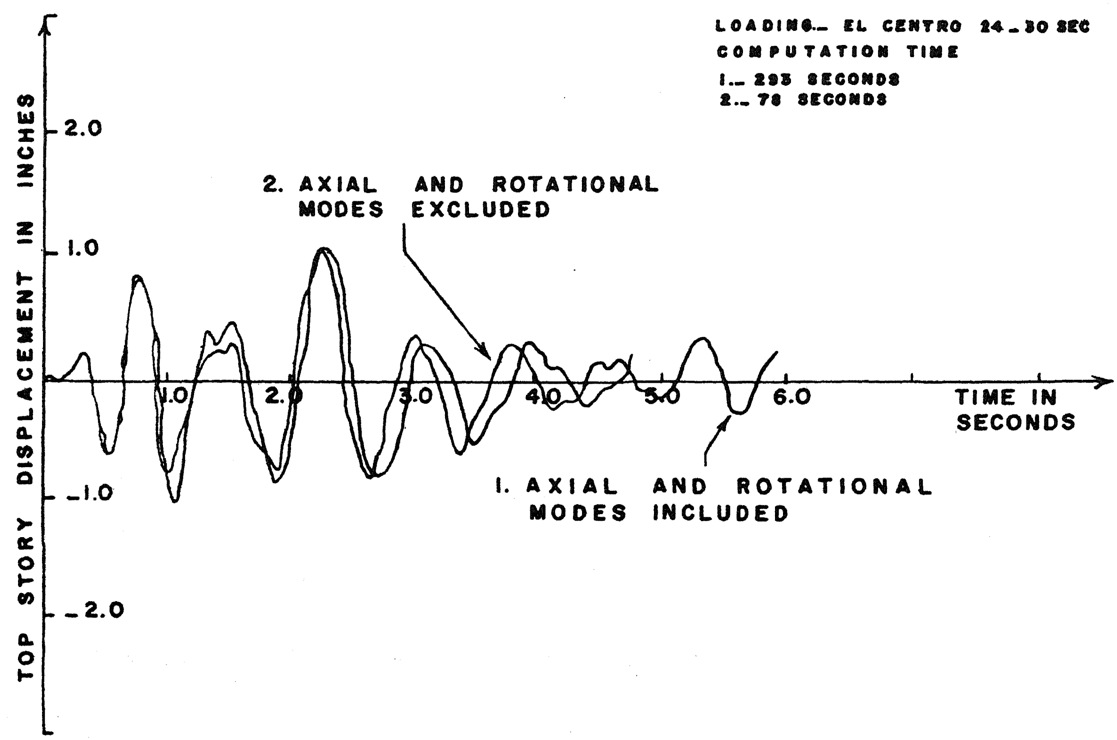

FIG. 10. EFFECTS OF AXIAL AND ROTATIONAL MODES.