COMPUTED NONLINEAR ANALYSIS OF R/C COUPLED SHEAR WALLS

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SUMMARY

A new 2-D planar R/C line element model is presented which considers axial force-moment interaction in calculating the element stiffness. Using a simplified Takeda type hysteresis model to describe the moment-curvature relationship at the critical section, a frame column stiffness is derived which also accounts for the finite length of any inelastic regions. The versatility of this model is demonstrated via its comparison with test results on a single column element. The new model is implemented in a computer program suitable for static and dynamic analysis of wall-frame and/or coupled shear wall structures. The reliability of the results is tested against measured responses of two coupled shear wall structures , which were subjected to simulated earthquake motions on the University of Illinois Earthquake Simulater.

INTRODUCTION

Computer analysis of a structure requires an analytical model that accurately represents the characteristics of the structural system. A case in point is the coupled shear wall where the existance of strong coupling between axial force and flexure needs a proper modeling. This paper presents procedures that provide for modeling of column elements which may have this coupling present. The material of this paper is taken from the Ph. D. thesis prepared by the senior author. A detailed review of existing analytical models for general R/C systems is given in Ref. 1.

COLUMN ELEMENT MODEL

For this model, the element chord zone, i.e. clear span, is considered to consist of two types of regions, an elastic zone plus two variable length inelastic zones at the ends of the member, as shown in Fig. 1. Inelastic actions are confined to these element ends where the curvature distribution is determined with the aid of idealized moment-curvature hysteresis rules. In order to represent the joint core zones at the member ends, rigid end links can also be specified. The cross sectional stiffness properties of the elastic zone, which are not constant, are calculated based on the change of the axial force. For the inelastic zones the effective section stiffness properties are determined from an appropriate moment-curvature hysteresis idealization. The effective section stiffness of each inelastic zone is assumed to be constant throughout the length of that zone. The inelastic length is considered to depend on the loading history and the axial force.

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The inelastic zone lengths, which may be different at the two ends of the member, do not remain constant throughout the response. However, change in the inelastic length is considered only when the end moment is in the strain-hardening phase. The inelastic length is assumed to be constant and equal to the maximum value of inelastic length when the end moment moves back out of strain-hardening.

The flexural flexibility matrix of a member chord zone can be readily formulated once the inelastic zone stiffness and inelastic length at each end have been established. Due to the significant contribution to end rotation resulting from bond slippage in the joint a nonlinear rotational spring, as an additional flexibility for an element, is provided at each member's ends. It should be mentioned here that for constant axial force, the proposed model is almost similar to the model which was initially developed by Soleimani (Ref. 2). The main modifications in the proposed model include the effect of changing axial force on the element stiffness as well as on the yield moment.

Effect of Axial Force on Moment-Curvature Curve

During the response of a structure to static or dynamic loading, there can be continual adjustments in the level of axial force in the vertical members. Thus there should be smooth shifts between the moment-curvature curves corresponding to these various axial forces. These shifts reflect either a hardening or a softening of the member due to an increase or decrease in the axial force. The current section stiffness , EI_i, of the moment-curvature curve in which the effect of axial force on the $M-\phi$ curve is taken into account, is developed by introducing appropriate shifts or movements between the series of $M-\phi$ curves for the different constant axial forces.

The bending moment is assumed to be a function of both curvature and axial force, while the axial force is assumed to be a linear function of only the average axial strain.

$$m = M(\phi,n) \tag{1}$$

The incremental form of moment can be expressed by differentiating this function:

$$\Delta m = \left(\frac{\partial M}{\partial \phi} + \frac{\partial M}{\partial n}\frac{\Delta n}{\Delta \phi}\right) \Delta\phi = EI_i * \Delta\phi \tag{2}$$

This current section stiffness , EI_i , contains two terms. The first term is the slope of the $M-\phi$ curve under a constant axial force. While the second term represents the effect of a change in the axial force on the slope of $M-\phi$ curve, Fig. 2. The yield moment of a section, corresponding to the current axial force, is determined from the axial force-moment interaction diagram for each loading increment. This yield moment is used in calculating both the section stiffness and the inelastic length at each end of the element. It interesting to note that the same procedure used to consider fluctuations of axial force in $M-\phi$ curve can be applied for the end moment-end rotation relationship. Thus, the stiffness of the nonlinear rotational spring at each end of the element as well as the elastic element stiffness for one-component model can be modified in the same way to consider the effect of changing axial force.

To examine this procedure, a small-scale cantilever column element (Ref. 3) is considered. In this test, a cantilever column was subjected to the cyclic lateral load, V. The axial force in the column, N, was changed in direct proportional to the lateral load, ($\Delta N / \Delta V = 4$). Experimental tip load-deflection curves appear in Fig. 3. Also shown in this figure are the computed results obtained by using the extended one-component model, the multiple spring model, and the new model. It is seen that the agreement between the analysis and experimental curves is quite good.

COMPUTED RESULTS

Models used for verification analysis are depicted in Figs. 4, and 5. The line elements representing beams and walls are connected by rigid links. Beams are idealized as an elastic line element with inelastic rotational springs located at members ends, one-component model (Ref. 4). The proposed model is used for the walls. A simplified Takeda type hysteresis model (Ref. 5) with bilinear primary M-ϕ curve is adapted in the wall model to represent the M-ϕ relations of critical section for walls. For coupling beams, the Takeda hysteresis rules are modified to include pinch action and strength decay which were observed to have a significant effect on the behavior of a coupled shear wall, (Refs. 6, 7).

The inelastic structural response is evaluated by numerically integrating the equation of motion using the Newmark Beta method based on a linear acceleration (Beta=1/6). The damping matrix is assumed to be proportional to the stiffness matrix with a damping factor of 2 % for the first mode shape. The damping matrix is updated every step based on a variable first frequency but constant first mode shape assumption from the current structural stiffness matrix.

The first structure is the 6-Story coupled shear wall which was tested by Lybas (Ref. 8). In the analytical study, the structure is reduced to a three story system to prevent possible numerical error causes by the lack of mass at the odd stories. Because the overall structural stiffness is dominated by the walls, the 6-Story structure is reduced to the 3-Story by lumping of the beams at every other level without changing the wall section stiffness. Fig. 6 shows a comparison of the 6-Story and the "Reduced" model with test results under cyclic static loading. The effect of the changing axial force in wall stiffness is considered only in the reduced model Case-1. Moment distribution patterns in the walls when the base shear equals +1.32 Kips, and when it is -1.31 Kips are shown in Fig. 7. The concentration of flexural moment on the compression wall at the base is clearly observed in this figure. These results indicate that maximum forces in the walls can be affected significantly by axial force-flexural interaction. The analysis which ignored the effect of axial force in flexural strength and stiffness underestimates maximum shear and moment at the base by as much as 50 %. However, the average of the base moments of the two walls in Case-1 at any step is roughly equal to the base moment of the wall in Case-2.

Comparison of computed responses (using "Reduced" model) and test result time histories show good agreement as indicated in Fig. 8. The distribution of the base shear between two walls is clearly observed in the base shear of left wall time history. During the first and second response peaks at 0.70 Sec. and 1.2 Sec., the left wall which is subjected to a tensile force does not resist as much shear at the base as the right wall. The effect of the individual walls can be seen in Fig. 9. It should be noted that yielding of the tension wall at the base does not mean that the structural system loses it resistance to further load since the compression wall is still capable of carrying additional loads with increased section stiffness due to the large value of comprssion force.

The second structure is the 10-Story coupled shear wall which was tested by Aristizabal-Ochoa (Ref. 9) and also studied analytically (Refs. 6, 7) using two different column elements. The test results and the computed results using the proposed model are compared in Fig. 10, where the basic responses appear in good agreement.

CONCLUSIONS

1---The accuracy of the model is demonstrated by the analyses of a one column element and by two coupled shear wall systems. The comparison between experimental and analytical results shows very good agreement, leading to the conclusion that the model is very effective in predicting the nonlinear behavior of R/C column frame members.

2---Fluctuations of axial forces in the coupled shear wall plays a major role in maximum forces in the individual walls. The analysis which ignored the effect of axial force in flexural strength and stiffness underestimates maximum shear and moment at the base of the walls.

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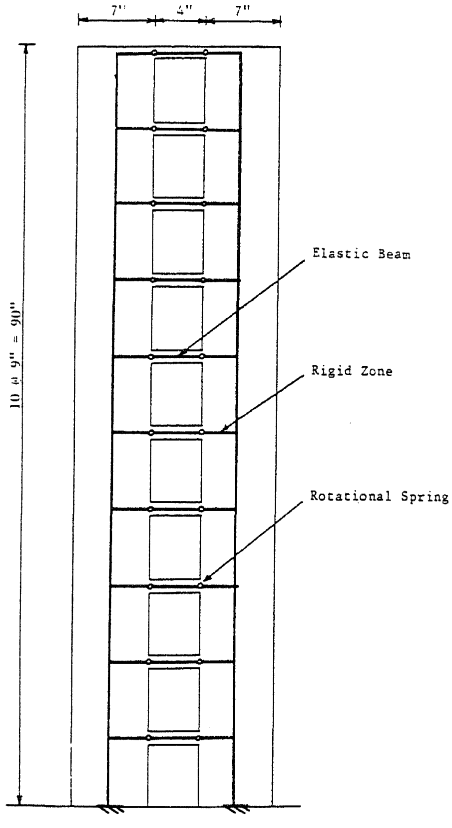

Fig. 5 - ANALYTICAL MODEL OF 10-STORY COUPLED SHEAR WALL

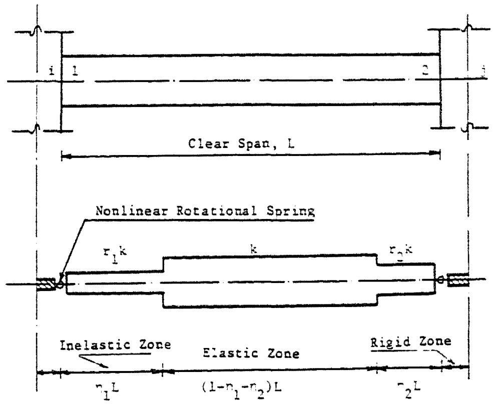

Fig. 1 - MODEL CHARACTERISTICS

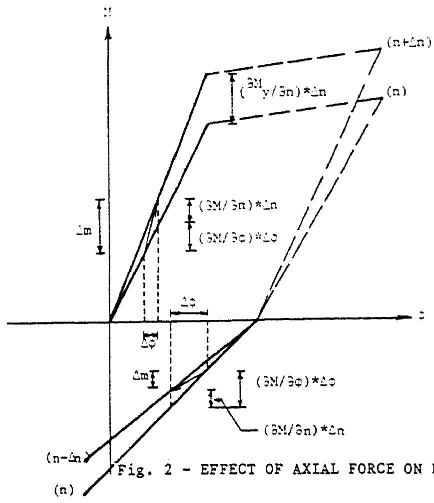

Fig. 2 - EFFECT OF AXIAL FORCE ON M-ϕ CURVE

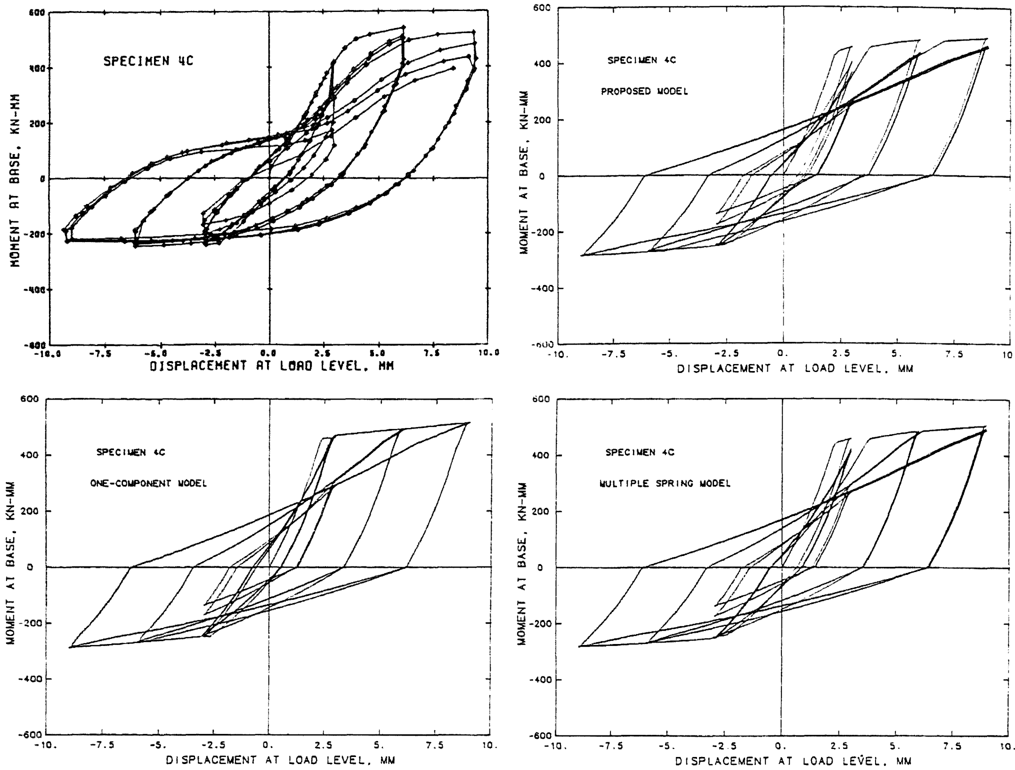

Fig. 3 - HYSTERESIS RELATIONSHIPS OF A COLUMN ELEMENT

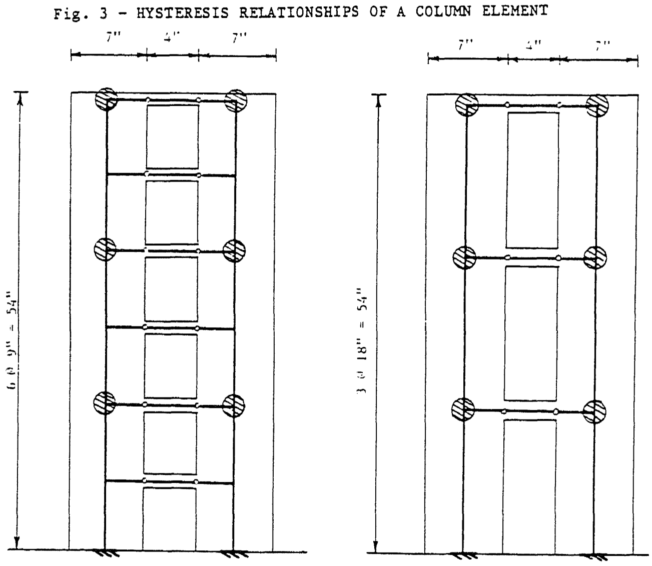

Fig. 4 - ANALYTICAL MODEL OF 6-STORY COUPLED SHEAR WALL

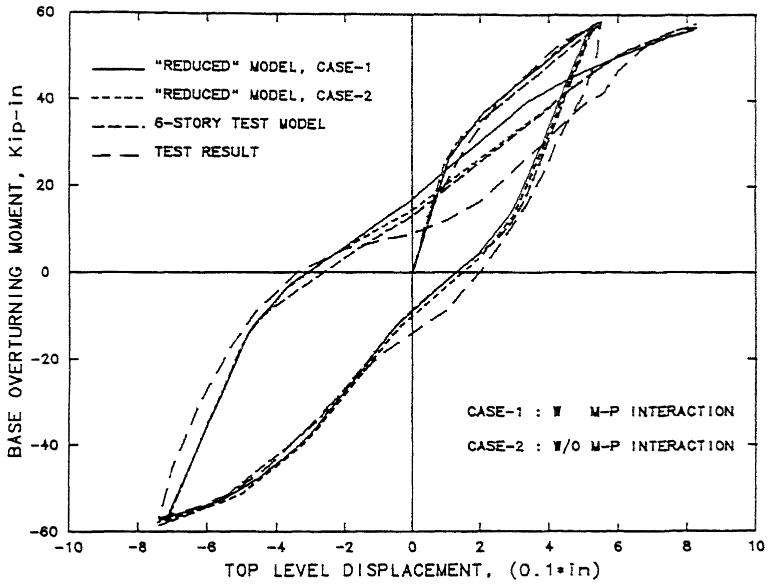

Fig. 6 - BASE MOMENT-TOP DISPLACEMENT RELATIONSHIPS

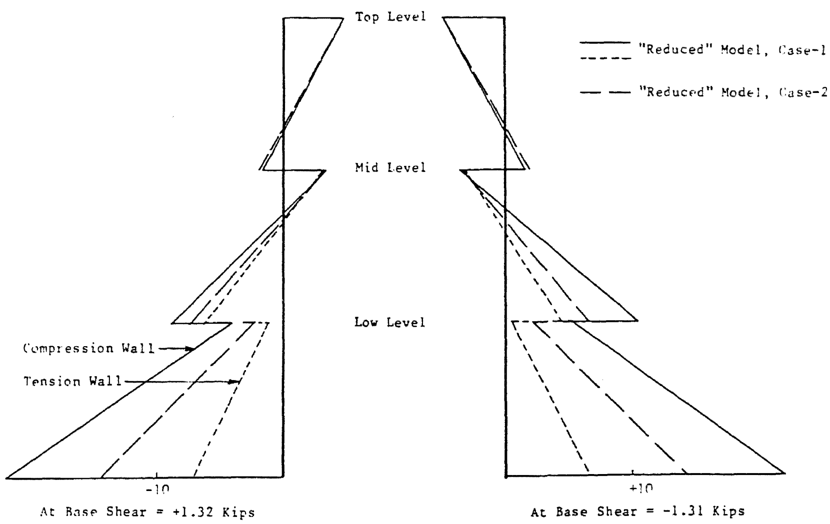

Fig. 7 - M-P INTERACTION EFFECTS, BENDING MOMENT, Kip-in

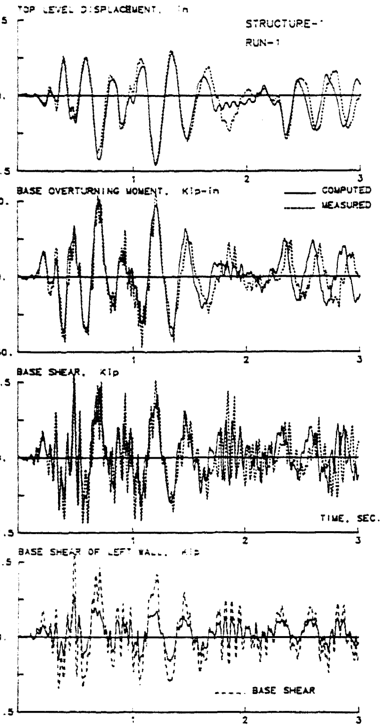

Fig. 8 - RESPONSE WAVEFORMS OF 6-STORY COUPLED SHEAR WALL

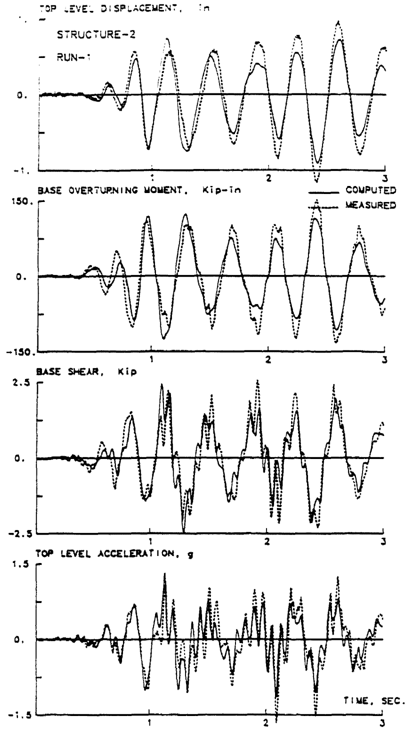

Fig. 10 - RESPONSE WAVEFORMS OF 10-STORY COUPLED SHEAR WALL

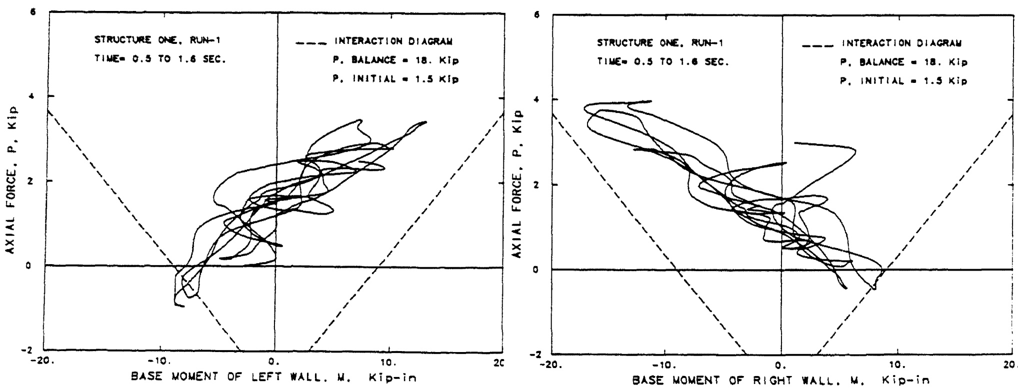

Fig. 9 - BASE MOMENT-AXIAL FORCE RELATIONS, 6-STORY COUPLED SHEAR WALL