COMPUTED BEHAVIOR OF COUPLED SHEAR WALLS

By

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SYNOPSIS

A procedure using a modified frame method is presented for the nonlinear analysis of coupled shear wall structures subjected to strong earthquake motions. The walls and coupling beams are replaced by flexural elements. Axial flexural and shear rigidities of the wall members are considered in the analysis. The coupling beams are taken as individual beams connected to the walls through a rigid link and rotational springs. The hysteresis rules used are an adaptation of those presented by Takeda et al (1970). A linear acceleration method is used for the solution of the equations of motion.

INTRODUCTION

The use of shear walls is a very popular scheme used in the design of multi-story buildings to provide resistance to horizontal movement from wind loads or earthquake motion. This paper outlines a procedure and presents some results for the computation of the nonlinear response of shear wall structures subjected to strong motion earthquakes. The procedure is intended for use as a research tool for the investigation of the influence that variations in various parameters have on the response of shear wall systems.

Although there are several variations and configurations of wall systems in use, the procedure is discussed only in reference to reinforced concrete coupled shear walls, two walls with connecting beams. There is a considerable body of existing literature dealing with coupled shear walls. Only a few of these are referenced here. The early study by Beck [1], the report by Jennings [2], and the ACI meeting [3] provide ample introduction to the pertinent literature.

To predict the actual behavior of a structure due to strong earthquake motions, the dynamic structural properties in the highly inelastic range must be taken into consideration. Inelastic properties such as cracking and crushing of the concrete, and yielding and bond slip of reinforcing steel complicate the problem. Therefore, idealizations and simplifications of the mechanical models for the constituent members were considered necessary in the analytical procedure.

For simplification inelastic beam model techniques which have been used extensively for the response of frame structures [4,5,6,7] are modified for use on the wall frame system.

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MECHANICAL MODEL

The lateral resistance of coupled shear walls results primarily from three sources of structural actions; flexural rigidities of the walls and of the connecting beams and the moment effect of the couple growing out of the axial rigidity of the two walls. The mechanical model chosen to represent the coupled shear wall is shown in Fig. 1. The walls and the coupling beams are replaced by massless line elements having flexural, axial, and shear rigidities. To follow the current fad and use two dimensional plane stress elements for the walls was judged less desirable.

The beams are taken as individual beams connected to the walls through a rigid link and a rotational spring. The rotational spring takes care of any beam end rotation which is produced by steel bar elongation and concrete compression in the joint core as well as the inelastic flexural and shear action over the beam length. Such inelastic flexural action is expected to be localized near the beam ends. The beam itself is considered to be a flexural member with uniform elastic rigidity along its length.

The shear wall is also considered to act initially as a beam with a linear variation of strain over the cross section. The wall members are exposed to a more general moment distribution than are the connecting beams. Therefore the inelastic flexural behavior in the wall can be expected to expand along the length of the member rather than be localized.

In order to allow the inelastic action to cover a partial length of a wall member, those members are divided into several subelements. The degree of subdivision decreases with story height since the major inelastic action is expected at the base. The internal subelements or degrees of freedom are condensed out of the stiffness matrix before the system equations are written so that only horizontal story movements appear in the final equations.

FORCE DEFORMATION RELATIONS OF FRAME ELEMENTS

The inelastic force deformation relations of the wall subelements and the rotational springs of the coupling beams are used as primary curves in a Takeda based hysteresis loop development for the respective elements.

The properties of the wall subelements are adjusted based on the deformation levels occurring at the midpoint of each subelement. Each subelement has flexural, axial, and shear stiffnesses. The shear rigidity is assumed to remain constant throughout. The flexural rigidity obeys the hysteresis loop shown in Fig. 2. The moment curvature relation for the subelement is adjusted to reflect the level of axial force present in the wall. The axial rigidity is determined by the cracking depth and any inelastic conditions of the steel and concrete.

The moment is assumed to be a function of curvature, φ, and axial force n, while the axial force is a function of curvature φ and axial strain ε.

$$\begin{aligned} m &= M(\varphi, n) \\ n &= N(\varphi, \varepsilon) \end{aligned} \tag{1}$$

Using these functions the differential form of the force deformation relations can be written after some manipulation as

$$\begin{Bmatrix} dm \\ dn \end{Bmatrix} = \begin{bmatrix} \frac{\partial M}{\partial \varphi} \frac{1}{1-\frac{\partial M}{\partial n}\frac{dn}{dm}} & 0 \\ 0 & \frac{\partial N}{\partial \epsilon} \frac{1}{1-\left(\frac{\partial N}{\partial \varphi}\frac{\partial M}{\partial \varphi}\right)\left(\frac{dm}{dn}-\frac{\partial M}{\partial n}\right)} \end{bmatrix} \begin{Bmatrix} d\varphi \\ d\varepsilon \end{Bmatrix} \quad (2)$$

These relations are used in incremental form for the wall subelement stiffnesses.

The rotational springs of the coupling beams contain the deformations that occur within the joint core and any bond slip as well as all localized inelastic flexural action experienced by the beams. The flexibility matrix of the coupling beam system can be expressed in the following form [7]:

$$\begin{bmatrix} f_{11} & f_{12} \\ f_{21} & f_{22} \end{bmatrix} = \begin{bmatrix} L/6EI & -L/6EI \\ -L/6EI & L/6EI \end{bmatrix} + \begin{bmatrix} L/2SD(M_A) & 0 \\ 0 & L/2SD(M_B) \end{bmatrix} + \begin{bmatrix} f(M_A) & 0 \\ 0 & f(M_B) \end{bmatrix} \quad (3)$$

where the special symbols SD(M) represent the sum of the flexural and shear deflections of a cantilever beam of unit length and f(M) is the flexibility resulting from bond slip, etc., in the joint core. The reason the first matrix is not in the normally recognized form is that part of the elastic flexibility coefficients of the diagonal elements have been assigned to the second matrix (that containing the inelastic action) for computational ease.

ANALYTICAL PROCEDURE

These force deformation properties of the elements are used to develop the applicable equations of motion

$$[M]\{\ddot{X}\} + [c]\{\dot{X}\} + [K]\{X\} = -[M]\{\ddot{u}\} \quad (4)$$

where M is the lumped mass concentrated at each floor, X, $\dot{X}$, and $\ddot{X}$, are the relative story displacements, velocities, and accelerations, and $\ddot{u}$ is the base acceleration. Damping is made up of parts α proportional to mass and β proportional to stiffness. The stiffness matrix changes during the response to reflect the current structural state therefore the β matrix is likewise changed to keep within reasonable damping values.

The inelastic structural response and failure processes are evaluated by numerically integrating the equations of motion using the Newmark β method based on a linear accelration (β=1/6). Within any one time interval the properties of the structure are assumed constant. The residual forces that develop due to structure changes that actually occur within a time interval are applied to the subsequent time step.

HYSTERSIS LOOPS

The rules used in this analysis are an adaptation of those presented by Takeda et al [8]. For the wall subelements the moment curvature relations of the primary curve are approximated by a trilinear curve that varies with axial force. The resultant loops are shown in Fig. 2. In Eq. 2 the pseudo flexural rigidity $\partial M/\partial\varphi$ follows Takeda. The real flexural rigidity $dM/d\varphi$ is obtained by multiplying $\partial M/\partial\varphi$ by $(1/1-\partial M/\partial n * dn/dm)$ to account for axial force. The hysteresis loop for the rotational springs of the coupling beams start with a trilinearized version of Takeda but modify it for a pinching action from the compression reinforcement yielding before the cracks close and a beam strength decay due to changes in shear resisting mechanisms (Fig. 3).

ANALYTICAL RESULTS

The procedure has been applied to the ten-story coupled wall models tested on the Illinois earthquake simulator [9]. The dimensions of the model wall are shown in Fig. 1. The weight of .5 kip is placed at each floor level.

The base motion used is El Centro N.S., 1940, with the maximum acceleration of .41 g. The time span of the base motion is compressed by a factor of 2.5 from the original records. The maximum response values of the structure are shown in Table 1 along with measured results. The sequence of failure mechanism development of the structure is presented in Fig. 4. Within the first second all of the significant inelastic change of stiffness has occurred. First cracking of connecting beam appeared starting in the intermediate level followed by cracking in the lower part of walls. After these, yielding of some connecting beams began and the cracking of upper part of wall followed. For this level of motion there was no flexural yielding of wall.

Some response wave forms of the structure obtained by this analysis are shown in Fig. 5.

The top deflection and base overturning moment curve from a static analysis with the first mode load distribution is presented in Fig. 6 along with a comparison with the measured values. In this figure also the sequences of cracking and yielding of each element are shown. The top deflection and base overturning moment curves for a cyclic static loading are shown in Fig. 7 where the effects of nonlinear axial force and decay of beam strength can be seen.

SUMMARY

Inelastic action of the connecting beam plays a major role in controling the structure's response. The pinching effect of the connecting beam produced a larger displacement response than was the case without pinching. The decay of the connecting beam strength alongated the period of the response wave forms. Effect of nonlinear axial behavior on the whole structure increased the acceleration response.

ACKNOWLEDGEMENT

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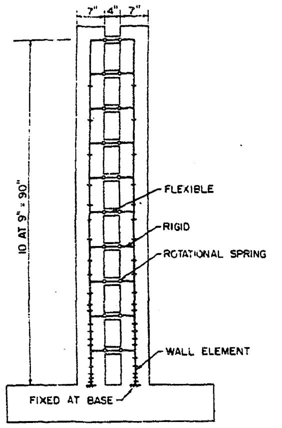
TABLE 1

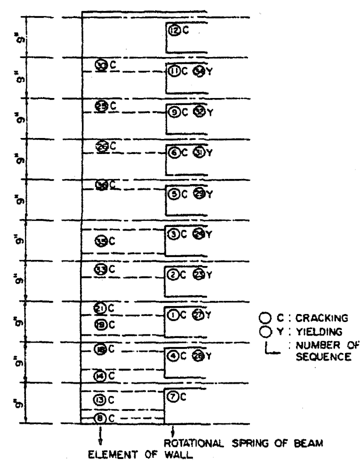
Maximum Response Values of Ten-story Coupled Shear Wall

	Story	Measured	Calculated 2	Calculated 2, Nonlinear Axial Force Effect
Acceleration(G)	10	1.66	1.43	1.70
Shear (kip)	1	2.54	3.06	3.02
Overturning Moment (kip-in)	1	151.5	153.0	152.5
Displacement (in)	10	1.16	1.10	1.00



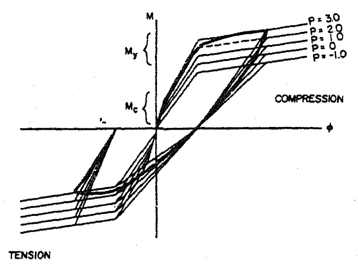
ANALYTICAL MODEL OF COUPLED SHEAR WALL

FIG. 1



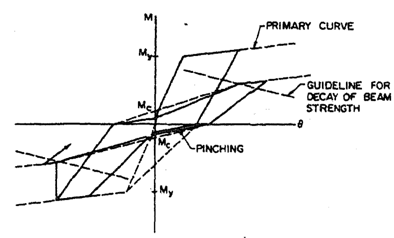
SEQUENCE OF CRACKING AND YIELDING IN DYNAMIC RESPONSES

FIG. 4



HYSTERESIS FOR ELEMENT OF WALL

FIG. 2



HYSTERESIS FOR ROTATIONAL SPRING OF BEAM

FIG. 3

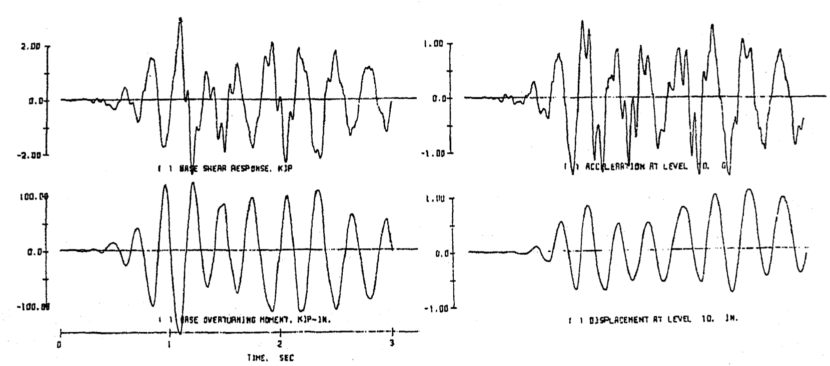


FIG. 5 RESPONSE WAVE FORMS

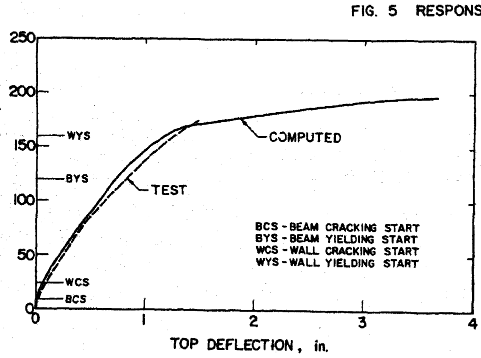


FIG. 6

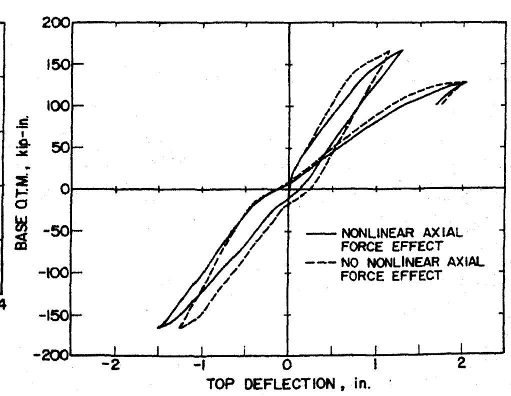


FIG. 7