Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VI)

9-1-7

ANALYTICAL MODELING OF R/C STRUCTURAL WALLS

Alfonso VULCANO[1] , Vitelmo V. BERTERO[2] and Vincenzo COLOTTI[1]

[1] Dipartimento di Strutture, Università della Calabria,
Arcavacata di Rende (Cosenza), Italy
[2] Department of Civil Engineering, University of California,
Berkeley, U.S.A.

SUMMARY

A relatively simple and reasonably reliable wall model is proposed, which is suitable to be efficiently incorporated in a practical nonlinear seismic analysis of R/C frame-wall structural systems. A numerical investigation, conducted with reference to a series of R/C structural walls tested at the University of California at Berkeley, shows that the proposed model accurately predicts the measured flexural response. However, under high shear stresses, further improvements of the wall model are needed to accurately predict the hysteretic shear response as well as the flexural and shear displacement components.

INTRODUCTION

R/C frame-wall structural systems prove to be very effective during severe earthquake ground motions, particularly when adopted for tall buildings. In order to predict the inelastic response of such structural systems under seismic loads, the hysteretic behaviour of the structural members and their interaction should be accurately described by reliable analytical models. Nevertheless, relatively simple models should be used such that the analysis could be performed with a reasonable computational effort.

As emphasized in Ref. 1, the nonlinear analysis of complex structural systems can be efficiently carried out by using analytical models based on a macroscopic approach rather than detailed models. Although suitable analytical models have been proposed for realistic and practical prediction of the hysteretic behaviour of R/C beam members, many uncertainties about the formulation of a reliable model for a practical analysis of R/C structural walls persist.

The use of wall models based on the concept of equivalent beam or equivalent truss involves many limitations pointed out in the mentioned Ref. 1. Many important features of the hysteretic behaviour observed during experiments on a full-scale model of a seven-story R/C frame-wall structure have been incorporated in the Three-Vertical-Line-Element Model (TVLEM) proposed by Kabeyasawa et al. (Ref. 2) to simulate the inelastic response or R/C structural walls. Even though there is good correlation between observed and computed responses for the overall structure, further improvements in the TVLEM are believed possible. In Ref. 1 the TVLEM was modified by incorporation of an axial-stiffness hysteresis model (ASHM) consisting of two axial elements in series whose hysteretic behaviour was described by considerably simple laws. The results of an extensive numerical investigation, besides showing some limitations of the modified TVLEM, indicated the opportunity of obtaining a more refined description of the flexural behaviour of the wall from one or both the following approaches: a) use of more refined laws, based on the actual behaviour of the materials and their interaction, to describe the response of the two elements in series constituting the ASHM; b) modification of geometry of the wall model to gradually account for the progressive yielding of the steel.

In this paper a wall model is proposed by following both these approaches. In order to check effectiveness and reliability of the proposed wall model, a numerical investigation is carried out with reference to a series of R/C structural walls tested at the University of California at Berkeley (Ref. 3).

PROPOSED WALL MODEL

The model in Fig. 1 is proposed to simulate the response of the generic wall member. The flexural response is simulated by a multi-uniaxial-element-in-parallel model with infinitely rigid beams at the top and bottom floor levels: the two external elements represent the axial stiffnesses (K_1 and K_2) of the boundary columns, while the interior elements (at least two, with stiffnesses $K_3,..,K_n$) represent the axial and flexural stiffnesses of the central panel. A horizontal spring, with stiffness K_H and hysteretic behaviour described by the origin-oriented hysteresis model (OOHM) proposed in the mentioned Ref. 2, simulates the shear response of the wall member. The relative rotation $\Delta\phi$ is intended around the point placed on the central axis of the wall member at height ch. A suitable value of the parameter c can be selected on the basis of the expected curvature distribution along the inter-storey height h: for instance, $0 \leq c \leq 1$, if the curvature sign does not change along h.

The two-element-in-series model shown in Fig. 2 describes the response of the generic uniaxial element in Fig. 1. The two elements in series are representative of the axial stiffness of the column segments in which the bond remains active (element 1) and those segments for which the bond stresses are negligible (element 2). Each element consists of two parallel components to account for the mechanical behaviour of the concrete (C) and the steel (S); a suitable law for the dimensionless parameter λ defining the length of the two elements permits an accurate description of the measured tension-stiffening effect. It should be noted that the model in Fig. 2, though similar to the ASHM adopted in Ref. 1 with a one-component element 1, differs from this model in having also the element 1 constituted by two components C and S. Moreover, unlike the considerably simple assumptions for the ASHM in Ref. 1, refined constitutive laws are herein adopted to idealize the hysteretic behaviour of the materials and the tension-stiffening effect.

Concrete Model A different uniaxial stress-strain relationship is adopted for cracked and uncracked concrete, that is for component C of the elements 2 and 1 in Fig. 2, respectively. Even though many models are available, the Bolong's et al. model (Ref. 4) is here considered for the cracked concrete, because it also accounts for the contact stresses due to the progressive closure of cracks. However, a limitation of this model consists in the lack of knowledge of the effect of the longitudinal and transverse steel ratios on shape and parameters of the characteristic curves of the model. Therefore, the same shape of these curves is assumed for confined and unconfined concrete, but for this last one a skeleton compressive curve of the kind assumed in Ref. 3 is adopted. The stress-strain relationship for uncracked concrete and a set of rules allowing for a generalized load history, both previously proposed in Ref. 5, are adopted in these studies.

Steel Model The stress-strain relationship originally proposed by Giuffrè and Pinto and later implemented in Ref. 6 is adopted to describe the hysteretic response of the reinforcing steel. In order to avoid the storage of all parameters required for a generalized load history to retrace all previous reloading curves which were left incomplete, in this paper the set of simple rules suggested by Jennings (Ref. 7) is used.

Modeling of the Tension-Stiffening Effect Under monotonic tensile loading the tension-stiffening effect is taken into account by calculating the value of λ such that the tensile stiffness of the uniaxial model in Fig. 2 would be equal to the actual tensile stiffness of the uniaxial R/C member which is intended to be idealized. The actual tensile stiffness of this member is evaluated on the basis of the empirical law suggested by Rizkalla and Hwang (Ref. 8). Due to these assumptions, until the concrete remains uncracked or successively to the steel yielding, the model in Fig. 2 specializes in the element 1 (λ=0) or in the element 2 (λ=1), respectively. Under cyclic loadings it is here assumed that, during an unloading from a tensile stress state, the value of λ is maintained constant, equal to the value corresponding to the maximum tensile strain which has been previously attained; if this maximum strain is exceeded during a tensile reloading, the value of λ is updated as for the case of monotonic tensile loading. Further detail can be found in Ref. 5.

NUMERICAL RESULTS AND CONCLUDING REMARKS

On the basis of the models in Figs. 1 and 2, a numerical procedure analogous to that described in detail in Ref. 1 has been coded as a computer program for the nonlinear analysis of R/C structural walls and uniaxial members.

A first test has been conducted to check the reliability of the uniaxial element model in Fig. 2 with reference to a R/C prism under axial load reversals tested by Morita et al. (Ref. 9). A good correlation of numerical and experimental results is shown in Fig. 3, except for the branches corresponding to the progressive closure of cracks. This discrepancy is due to the fact that, as previously pointed out, the adopted concrete model does not account for the effects produced by different values of the actual longitudinal and transversal steel ratios: e.g., the effects on the stress-strain relationship during the closure of cracks.

Many numerical tests have been conducted in order to check effectiveness and reliability of the wall model in Fig. 1 with reference to four 1/3-scale R/C wall specimens previously tested by Vallenas et al. (Ref. 3). Test walls and loading patterns are schematically shown in Fig. 4.

The same test walls, which were subjected to high shear stresses, were considered in Ref. 1, where parametric studies shown the difficulty of describing accurately by the modified TVLEM, already mentioned, the measured flexural and shear displacement components, which proved to be sensitive to the choice of many parameters. A difficulty of the same kind has been met in these studies. This is shown in Fig. 5, where the measured displacement components at the

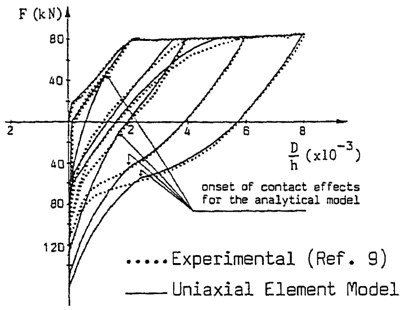
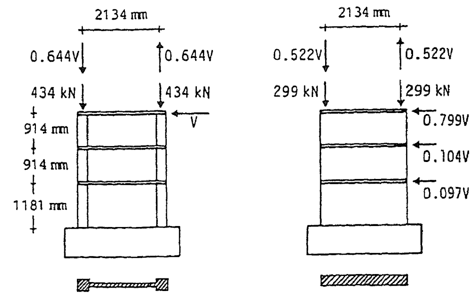


Fig. 3 Analytical and Experimental Curves for a R/C Uniaxial Member



(a) Specimens 3,4 (b) Specimens 5,6

Fig. 4 Test Walls and Loading Patterns

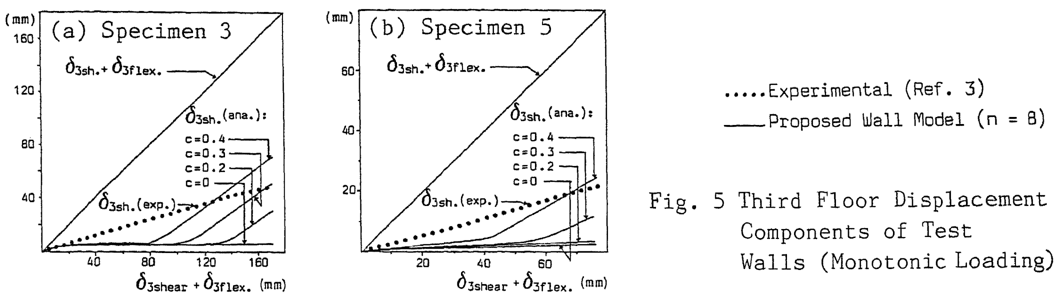
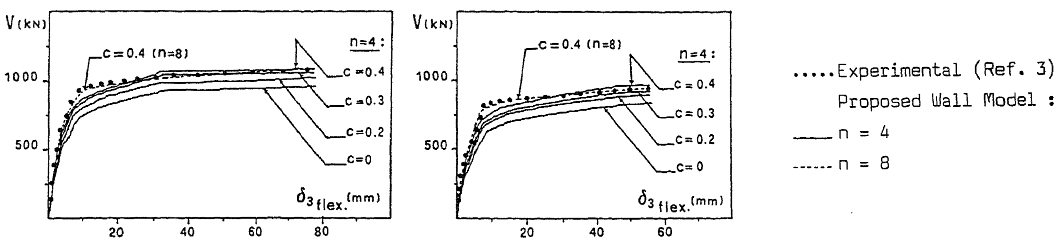
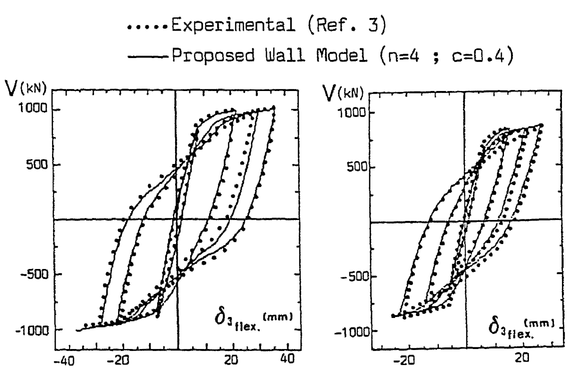


Fig. 5 Third Floor Displacement Components of Test Walls (Monotonic Loading)



(a) Specimen 3 (b) Specimen 5

Fig. 6 Flexural Response of Test Walls Under Monotonic Loading



(a) Specimen 4 (b) Specimen 6

Fig. 7 Flexural Response of Test Walls Under Cyclic Loading

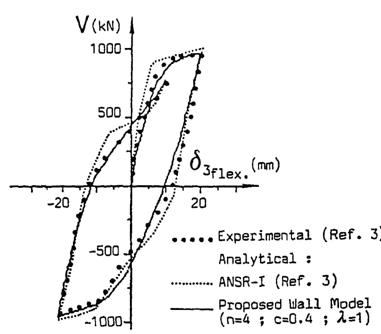


Fig. 8 Experimental and Analytical Results for Specimen 4