HYSTERETIC BEHAVIOR OF REINFORCED CONCRETE BEAMS AND JOINTS

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SUMMARY

This paper presents a general analytical model which describes the hysteretic behavior of reinforced concrete joints and gir er inelastic regions under large deformation reversals. The interaction of reinforcing steel and surrounding concrete through bond and the deterioration of such interaction under cyclic load reversals is taken into account. In order to satisfy the equilibrium of horizontal forces and bending moments at cracked R/C sections a new layer model is presented which accounts for the deterioration of bond in the vicinity of the crack. The model is applied to an interior beam-column joint and analytical predictions are compared with experimental evidence on the hysteretic behavior of interior beam-column subassemblages. A series of parametric studies on the hysteretic response of interior joints is also presented.

INTRODUCTION

Reinforced concrete structures designed according to present building codes are expected to deform well into the inelastic range and dissipate the energy input by an extreme base motion through stable hysteretic behavior of structural components. Since inelastic deformations are typically concentrated at certain critical regions within the structure (Ref. 1), the accurate prediction of the mechanical behavior of a structure during earthquake excitations depends on the development of reliable analytical models which describe the hysteretic behavior of these regions. Following present earthquake resistant design philosophy the energy input by the base motion should be dissipated in the largest possible number of inelastic regions within the structure. Ductile moment resisting frames as well as coupled wall systems are designed so that yielding starts to develop at the girder ends. Columns of a ductile moment resisting frame should remain elastic during the earthquake response, except at the base of the building, to avoid the formation of a partial sidesway collapse mechanism. Attention is thus focused on understanding and predicting the hysteretic behavior of critical regions in girders as well as that of beam-column or girder-wall joints.

In modeling the hysteretic behavior of reinforced concrete members under large deformation reversals the interaction of reinforcing steel and surrounding concrete through bond and the deterioration of such interaction under cyclic load reversals appears to be an important factor. Cyclic bond deterioration has however received only limited attention in previous analytical studies (Refs. 2, 3 and 4) mostly due to lack of reliable experimental data on the bond behavior of deformed bars embedded in R/C members and subjected to cyclic load reversals.

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PROPOSED ANALYTICAL MODEL

In order to formulate an analytical model describing the hysteretic behavior of R/C members with due account of cyclic bond deterioration between reinforcing steel and concrete, the region of the member undergoing inelastic action is divided into a number of subregions at locations where cracks form (Fig.1). In members subjected to severe moment reversals with low shear stresses, cracks run almost vertically to the axis of the member through the depth of the cross-section. The positions where cracks are expected to form are not known a priori and can be established in the course of an analysis by determining the sections where the concrete tensile strength is first exceeded. In the present model for reasons of simplicity the cracks have been assumed to run vertically across the section and form at predetermined locations. This is, strictly speaking, true only at beam-column interfaces of interior and exterior joints and at the ends of coupling girders (Fig.2).

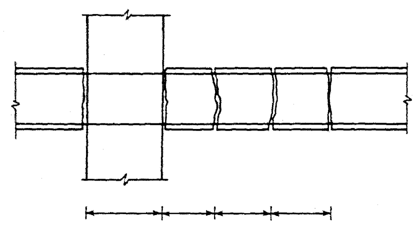

FIGURE 1 SUBDIVISION OF R/C MEMBER INTO SEVERAL SUBREGIONS

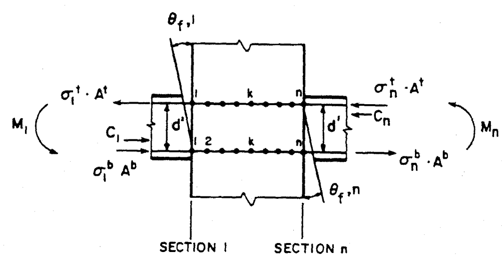

FIGURE 2 ANALYTICAL MODEL FOR INTERIOR BEAM-COLUMN JOINT

The hysteretic response of each subregion is determined by satisfying the equilibrium of horizontal forces and bending moments at both end sections and by establishing the stress transfer between steel and concrete within the region. The effect of shear stresses is neglected. The analytical model thus consists of two main parts:

(1) a model describing the transfer of stresses between reinforcing steel and surrounding concrete within the subregion. This model is based on a solution of the differential equations of bond using a mixed finite element method. It results in a nonlinear transfer matrix which relates steel stress and relative slip increments at section 1 to those at section n of the subregion (Figs.1 and 2). In order to describe the reinforcing steel stress-strain relation under arbitrary strain reversals the model proposed by Menegotto and Pinto in Ref. 5 has been modified to include isotropic strain hardening as a function of plastic strain history. In modeling the bond stress-slip behavior between reinforcing steel and surrounding concrete the model proposed by Eligehausen et. al in Ref. 6 has been modified to include a gradually increasing slope during reloading as shown in Fig. 3 (curve (d)).

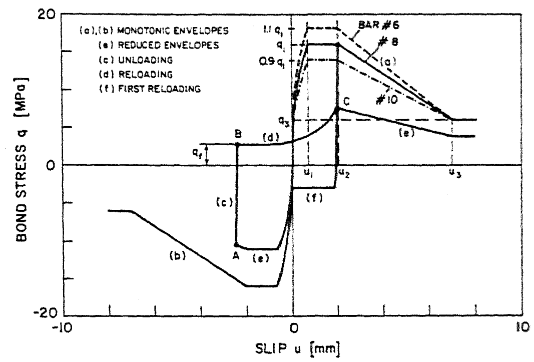

FIGURE 3 PROPOSED BOND STRESS-SLIP RELATION

(2) a section layer model for cracked reinforced concrete sections which determines the relative contribution of reinforcing steel and concrete to the equilibrium of horizontal forces and bending moments at a cracked section. Since the relative slip of reinforcing bars with respect to the surrounding concrete can be computed with the aid of the model under (1) it is possible to compute and trace the time history of crack width at the level of the top and bottom reinforcing layer (Fig.4(a)). Using a simple extrapolation formula the crack width at the top and bottom of the cracked R/C section can be estimated (Fig.4(b)). Once the crack width at the top and bottom of a cracked R/C section can be estimated at each load step, a crack closure criterion determines whether the crack is open or closed. If the crack is open, the concrete contribution to the equilibrium of horizontal forces and bending moments at the cracked section is equal to zero. If the crack is closed, a rule derived from experimental evidence relates the strain increments in the reinforcing bars to concrete strain increments in the section layer containing the bars. Once the distribution of concrete strain increments over the depth of the cross-section is established the concrete contribution to the equilibrium of section forces can be readily determined. More details on this new layer model for cracked R/C sections are presented in Ref. 7.

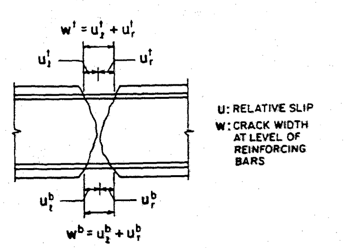

FIGURE 4a DEFINITION OF RELATIVE SLIP AND CRACK WIDTH AT A CRACKED REINFORCED CONCRETE SECTION

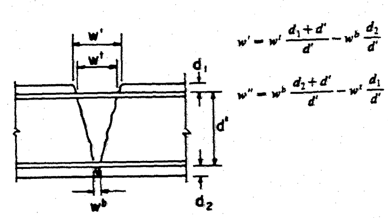

FIGURE 4b COMPUTATION OF CRACK WIDTH AT THE TOP AND BOTTOM OF A CRACKED R/C SECTION

RESULTS ON INTERIOR BEAM-COLUMN JOINTS

The validity of the analytical model was tested by comparing its predictions with experimental results from interior beam-column subassemblages subjected to cyclic loading simulating the effects of severe seismic excitations; two specimens from Ref. 8 were used as sample cases. The two specimens were subjected to entirely different load histories: specimen BC4 was subjected to a single large displacement reversal simulating a very severe seismic pulse; specimen BC3 was subjected to a large number of deformation reversals of gradually increasing magnitude. Only the results pertaining to specimen BC3 will be presented here. A more complete discussion along with results from specimen BC4 is presented in Refs. 7 and 9.

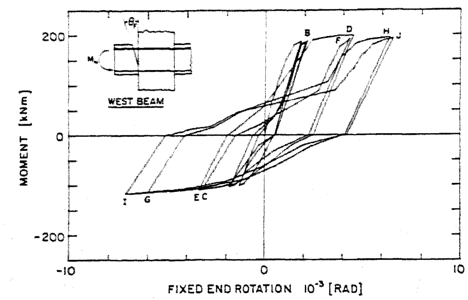

FIGURE 5a END MOMENT VERSUS FIXED-END ROTATION. SPECIMEN BC3

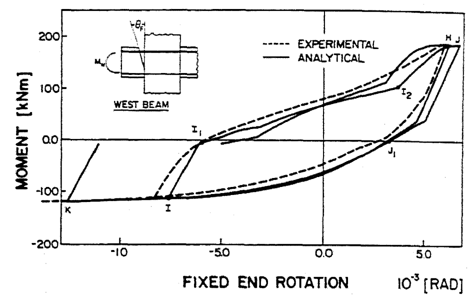

FIGURE 5b END MOMENT VERSUS FIXED-END ROTATION. SPECIMEN BC3

Fig. 5a depicts the analytically predicted moment-fixed end rotation relation at the west beam-column interface of specimen BC3. Selected loops are compared with experimental evidence in Fig. 5b indicating very satisfactory agreement. It should be noted that fixed-end rotations are computed by considering the relative slip of top and bottom reinforcing bars at the interface and dividing by the distance between the top and bottom reinforcing layer. In calculating the relative slip of reinforcing bars at the beam-column interface only the influence of bond deterioration along the column anchorage length was accounted for in order to facilitate comparison with experimental results. The effect of bond deterioration in the girder inelastic regions adjacent to the column is thus not included in the fixed-end rotations depicted in Figs. 5a and 5b, since it is accounted for when computing the rotation of the girder inelastic region.

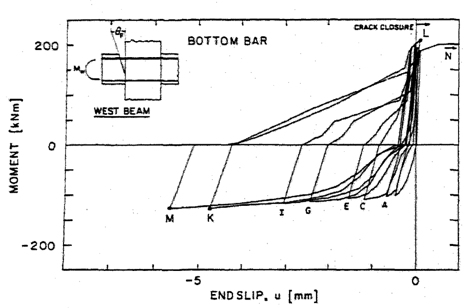

FIGURE 6a END MOMENT VERSUS RELATIVE BAR SLIPPAGE AT BEAM-COLUMN INTERFACE. SPECIMEN BC3

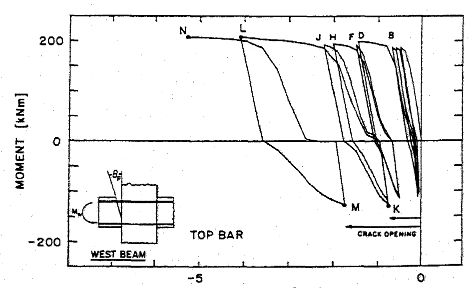

FIGURE 6b END MOMENT VERSUS RELATIVE BAR SLIPPAGE AT BEAM-COLUMN INTERFACE. SPECIMEN BC3

Figs. 6a and 6b depict the relative slip of the bottom and top reinforcing bars, respectively, at the west beam-column interface. Capital roman letters correspond to points of load reversal in Fig. 5a. It is observed that at load point K both the crack at the bottom and that at the top of the cross-section are open indicating that the crack is open through the depth of the beam-column interface. This is due to unequal amounts of top and bottom reinforcement which prevents the crack at the top from closing when the bottom reinforcing bars are subjected to tensile stresses.

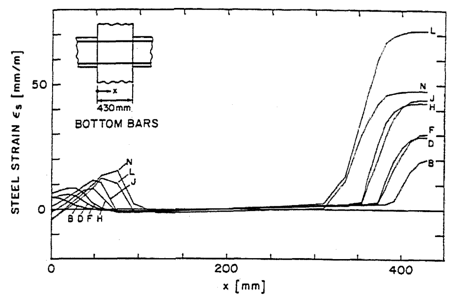

FIGURE 7a DISTRIBUTION OF STEEL STRAIN ALONG THE BOTTOM REINFORCING BARS WITH INCREASING MAGNITUDE OF END SLIP

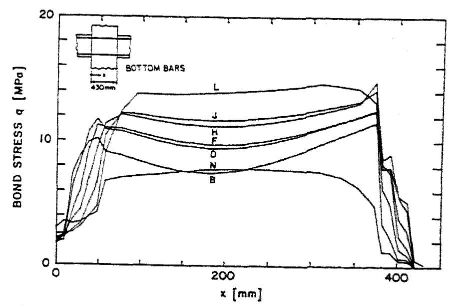

FIGURE 7b DISTRIBUTION OF BOND STRESS ALONG THE BOTTOM REINFORCING BARS WITH INCREASING MAGNITUDE OF END SLIP

Figs. 7a and 7b depict the distribution of steel strains and bond stresses along the column anchorage length of the bottom reinforcing bars with increasing magnitude of end slip. It is observed that up until the moment when bond is completely destroyed within the column (load point N) a portion of the reinforcing bars remains elastic.

ANALYTICAL PARAMETRIC STUDIES

Using the developed analytical model studies regarding the influence of various parameters such as yield strength of reinforcing bars, ratio of top to bottom reinforcement, reinforcing bar diameter and bond strength can be conducted. Such studies are presented in some detail in Ref. 7. A small selection of the findings is presented below.

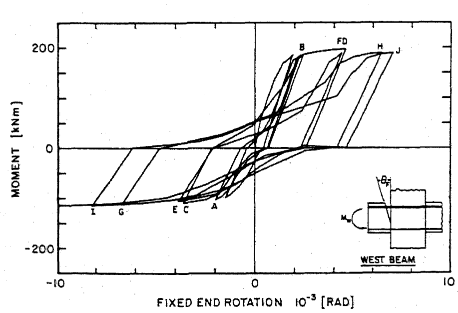

FIGURE 8a END MOMENT VERSUS FIXED-END ROTATION. BOND STRENGTH REDUCED BY 15% ALONG TOP AND BOTTOM REINFORCING BARS

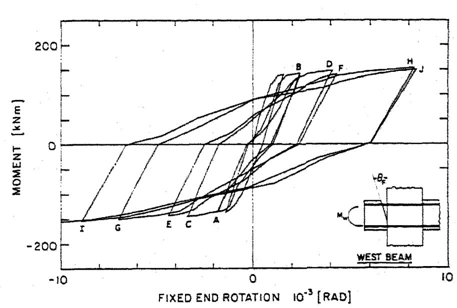

FIGURE 8b END MOMENT VERSUS FIXED-END ROTATION. EQUAL RATIO OF TOP TO BOTTOM REINFORCEMENT

In Fig. 8a the analytical moment-fixed end rotation relation resulting when the bond strength along the top and bottom reinforcing layer is reduced by 15% is presented. All other parameters are kept the same as those in specimen BC3. When comparing Fig. 8a to Fig. 5a it is concluded that a small reduction in bond strength considerably influences the hysteretic behavior of interior joints resulting in more pronounced pinching of hysteresis loops and less energy dissipation capacity. This fact has important consequences in practice. Poor construction quality and workmanship can lead to a substantial decrease in the energy dissipation capacity of R/C joints.

Fig. 8b depicts the analytical moment-fixed end rotation relation when the total area of bottom reinforcing bars is set equal to the area of reinforcement at the top of the girder. Considerable improvement in hysteretic behavior results in this case as a consequence of the reduction in the pinching of hysteretic loops leading to an increase in energy dissipation capacity of the joint.

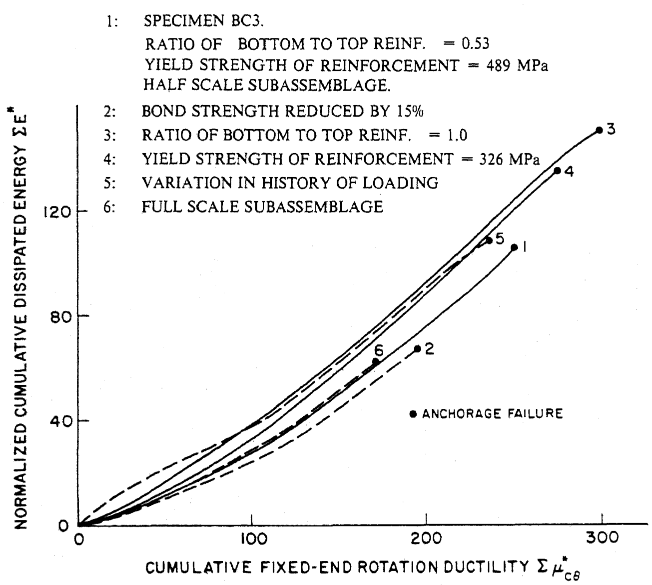

FIGURE 9 NORMALIZED TOTAL ENERGY DISSIPATION VERSUS CUMULATIVE FIXED-END ROTATION DUCTILITY FOR INTERIOR BEAM-COLUMN JOINT UNDER VARIATION OF PARAMETERS

In order to compare the behavior of interior beam-column joints under variation of different parameters, the normalized cumulative dissipated energy of the joint is plotted versus the cumulative fixed-end rotation ductility for a number of parametric studies in Fig. 9. The following parameters have been studied in addition to the bond strength and the ratio of top to bottom reinforcement: yield strength of reinforcing bars, history of loading, and influence of scale of the subassemblage. The following conclusions can be drawn from Fig. 9:

(a) Increasing the ratio of bottom to top reinforcement or decreasing the yield strength of reinforcing bars (which results in an increase in the number of bars if the yield moment remains the same) leads to considerable improvement in the total energy dissipation capacity of R/C joints.

(b) The history of loading affects the hysteretic behavior of R/C beam-column joints.

(c) The total energy dissipation capacity of a full-scale joint is considerably smaller than that of a half-scale beam-column joint.

RECOMMENDATIONS FOR FUTURE RESEARCH

The proposed model can be used in conducting additional investigations which go beyond the studies presented in this paper. In this sense it is recommended:

(i) to extend the model for predicting the hysteretic behavior of critical regions of reinforced concrete columns, shear-walls and coupling beams in coupled shear-walls. The hysteretic behavior of critical regions in girders should also be investigated. These studies can help in developing analytical models which improve upon existing ones which are based on point plastic hinges or on distribution of curvatures along the empirically estimated length of the plastic hinges,

(ii) to derive simple analytical models which account for the effects of bond deterioration in the seismic response of moment resisting frames,

(iii) to use the proposed model in connection with cyclic shear models at a cracked reinforced concrete section, since it allows computation and tracing of crack width in time. As it is well established, the crack width is the main parameter which affects the contribution of aggregate interlock and of the dowel action of reinforcing bars to the shear resistance at crack interfaces,

(iv) to include the effect of joint shear and diagonal cracking in the hysteretic behavior of interior and exterior joints. In many cases the transfer of shear stresses in the joint can significantly affect the overall behavior of the member.

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