Discussion of Paper No. 97

"THE INFLUENCE OF THE ELASTIC PLASTIC DEFORMATION OF BEAM-TO-COLUMN CONNECTIONS ON THE STIFFNESS, DUCTILITY AND STRENGTH OF OPEN FRAMES"

by

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INTRODUCTION

In the above named paper, Professors Kato and Nakao have made an important contribution to accurately predicting the load-deformational response of moment-resisting frames by discussing the influence of the elastic plastic deformations of panel-zones on the restoring force characteristics of frames and by suggesting a simple model for estimating this influence. The main objective of this discussion is to add to this contribution by evaluating in more detail the overall behavior of connections and suggesting an alternative trilinear model for the force-deformational response of connections. The authors' and writers' models are compared and their differences discussed in detail.

DISCUSSION OF CONNECTION BEHAVIOR

The seismic behavior of steel beam-to-column connections as an integral part of structural subassemblages was investigated through a series of tests carried out in the Structural Laboratory of the University of California, Berkeley [1]. From the results of this experimental investigation and from a finite element study, several models of the force-deformational response of fully welded connections (joints) were derived [2]. Results obtained in experimental and analytical studies showed that the post-yield reserve strength of connections is strongly dependent on the bending resistance of the column flanges. The stiffness of the beams framing into the column can also have significant effects on the post-yield strength. For instance, if both beams remain elastic, it can be assumed that the panel zone is restrained by four rigid boundaries connected at the four corner points by springs representing the bending stiffness of the column flanges (see Fig. 1). In the other extreme, if both beams framing into the column form plastic hinges, the beam

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stiffness is practically negligible. This case can be modeled readily by taking the column and subjecting it to an external loading representing the plastic stress distribution of beam web and flanges. Such a model is developed in Reference 2 and can be considered a lower bound for the connection strength. When compared to experimental results, this model significantly underestimates the ultimate strength of connections.

From the above considerations it is clear that if one attempts to model the behavior of beam-to-column connections subjected to unsymmetrical beam moments, one has to recognize that the force-deformational response is a function of: (1) the shear resistance of the panel zone and (2) the frame action of the elements surrounding the panel zone. However, it is the writers' opinion that in order to develop a model that will be simple enough to permit its incorporation into practical computer programs, one cannot pay too much attention to the effect of different boundary conditions. What is essential is to include the two main components of the connection strength, namely, the shear resistance of the connection panel and the bending resistance of the column flanges.

The basic deformation parameter that can be used to describe analytically the connection behavior is an average distortion γ. Thus it would appear logical to describe the variation of this γ in terms of an average shearing stress, τ. However, since the connection strength does not solely depend on the shear resistance of the panel, it is believed that a better strength parameter is the actual cause of the distortion, namely, the difference in moments in the beams framing into the column, ΔM. To account for the beneficial effect of the column shear V_{col} above and below the connection, a factor

$$\rho = \frac{V_{col}\, d_b}{\Delta M} \qquad (1)$$

is included in the trilinear model that is proposed below.

With ΔM and γ as the basic parameters, the effect of connection deformations can be readily incorporated in standard computer analysis programs by adding an additional degree of freedom at the nodal points, i.e., the intersections of beams and columns. Such a program has been developed [3], and the response data for a three-story frame are in excellent agreement with experimental data obtained in a dynamic test of the same frame on the shaking table of the University of California, Berkeley [4].

TRILINEAR RESPONSE MODEL

In the elastic range the average shear stress in the panel can be computed by neglecting the influence of the frame surrounding the panel, i.e.,

$$\tau = \frac{\Delta M(1-\rho)}{(d_c - t_c^f)\, t\, d_b} \qquad (2)$$

where d_c = depth of the column

d_b = depth of the beam

t_c^f = thickness of the column flange

t = thickness of panel zone

Furthermore, it can be assumed that in this elastic range the stiffness of the beams framing into the column is sufficient to prevent flexural deformations in the connection. Therefore, distortion of the panel can be computed as $\gamma = \tau/G$ and the elastic stiffness can be expressed as

$$K_e = \frac{\Delta M}{\gamma} = \frac{G(d_c - t_c^f)t\,d_b}{1-\rho} \tag{3}$$

When ΔM is equal to ΔM_y as obtained from Eq. (2) for $\tau = \tau_y$ (general yielding of the panel), the stiffness of the panel is assumed to approach zero, and from then on the frame surrounding the panel carries the additional loads with a tangent stiffness equal to

$$K_t = \frac{\Delta M - \Delta M_y}{\Delta \gamma} = \frac{G}{1-\rho}\,\frac{62.4\,I_c^f}{5\,t_c^f} \tag{4}$$

where I_c^f is the moment of inertia of the column flange. The value given by Eq. (4) represents the bending stiffness of the column flanges at the four corners of the panel zone. The distortion can be computed as follows

$$\gamma = \frac{1}{G}\left[\tau_y + \frac{(\Delta M - \Delta M_y)(1-\rho)5\,t_c^f}{62.4\,I_c^f}\right] \tag{5}$$

The tangent stiffness K_t is assumed to remain constant until a ductility ratio of four is attained, i.e., $\gamma_{max} = 4\gamma_y$. At this value the column flanges closely attain their plastic moment capacity. If the connection is strained beyond this point, a strain hardening stiffness can be assumed to be given by

$$K_s = K_e \frac{E_s}{E} \tag{6}$$

where E_s is the strain hardening modulus of the material. This model (see Fig. 1) , leading to a trilinear response diagram, is simple and sufficiently accurate for analysis purposes. The results obtained for two specimens A and B are shown in Fig. 2.

INFLUENCE OF COLUMN AXIAL LOAD P ON CONNECTION

For the range of P/P_y ratios investigated (up to 0.50), the test results and the finite element analysis of the mathematical model indicate that a reduction of the yield stress in shear according to von Mises yield criterion, i.e.

$$\bar{\tau}_y = \frac{\sigma_y}{\sqrt{3}} \sqrt{1 - \left(\frac{P}{P_y}\right)^2} \tag{7}$$

is sufficiently accurate to account for the effect of axial column loads on the yield strength of the connection.

EFFECT OF DOUBLER PLATES IN PANEL ZONE

Since doubler plates affect only the shearing strength of the panel zone, their effect can easily be included in the trilinear model by adding the shearing strength and stiffness of the plate to the elastic strength and stiffness of the column web, while the post-elastic stiffness K_t remains unchanged. In this manner, the "yield strength" of the connection may be somewhat overestimated; but the computed ultimate strength still seems to be conservative.

COMPARISON WITH MODEL BY KATO AND NAKAO

In the elastic range the model suggested by Kato and Nakao and the writers' model exhibit similar stiffness. However τ_y as defined by Kato and Nakao does not include any effect of the axial load on the column. Since both models overestimate the yield strength of the panel, the writers find it advisable to minimize this overestimation by reducing the yield stress in shear under the presence of an axial load as shown in Eq. (7). Kato's and Nakao's assumption of a post-elastic stiffness of 5% of the elastic one is advantageous because of its simplicity and gives good agreement with experimental results for columns with heavy flanges. However, the writers believe that for columns with light flanges, the above assumption may lead to a significant overestimation of the post-elastic stiffness and strength, since this stiffness is maintained up to a ductility of approximately 20. As previously pointed out, the writers feel that the post-elastic stiffness of the connection depends primarily on the bending stiffness of the column flanges and can be approximated by Eq. (4). The advantage of this model can be seen from Fig. 2 where the normalized force-deformational responses of two specimens with heavy and light column flanges are presented.

A study of the authors' and writers' test results also show that connections are structural elements of almost unlimited ductility and that at very high ductilities a strain hardening stiffness as indicated by Eq. (5) seems to be a realistic assumption.

REFERENCES

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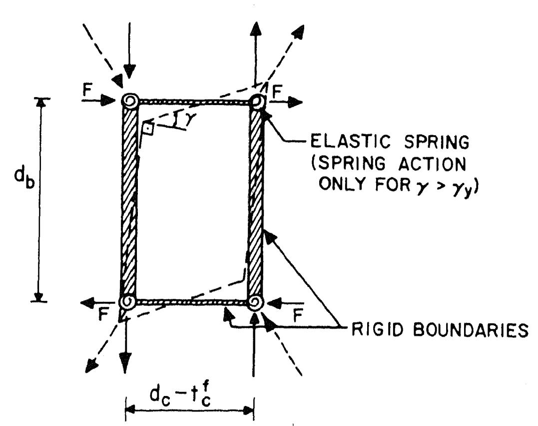

FIG. I CONNECTION MODEL

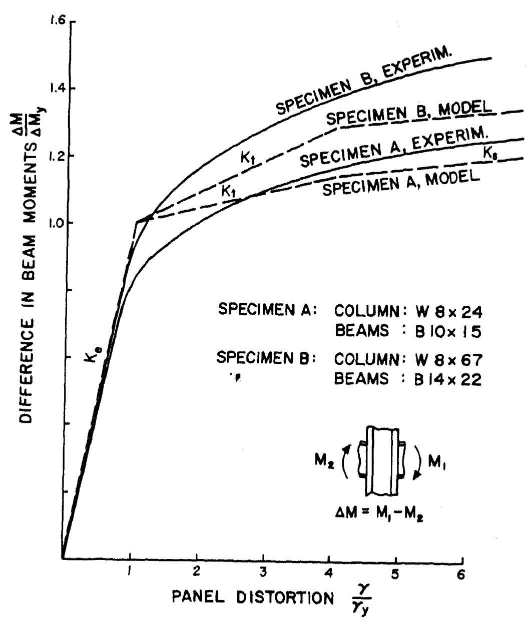

FIG. 2 EXPERIMENTAL VS. PREDICTED STRENGTH