I-69 THE ANALYSIS OF FRAMES WITH WELDED SEMI-RIGID JOINTS

Kyushu University SMem. JSCE, *Amar Mahmood MIAN
Kyushu University SMem. JSCE, Junji Kamata
Kyushu University Mem. JSCE, Hiroshi Hikosaka

INTRODUCTION

An attempt is made to provide some insight into semi-rigid welded steel beam-to-column connections in which stiffeners are avoided for economy and simplicity. On the basis of finite element method simplified formulas are proposed to calculate the rigidity and moment capacity of semi-rigid connections. In order to demonstrate the applicability of these formulas some practical combinations of H sections are chosen from the JIS(G3192) for numerical calculations. It is shown that for certain combinations of beam and column sections the omission of stiffeners in the joint panel does not reduce the moment capacity of the connection.

ANALYSIS MODEL OF WELDED BEAM-TO- COLUMN CONNECTION

The following assumptions are made in the analysis. (1). The connections are formed between beams and columns of rolled H- sections which can carry the full plastic moment. (2). Beams and columns are subjected to bending about their major axis. (3). The beam flanges are welded directly to the column flange mainly by groove welding, while the beam web and column flange is fillet welded.

Consider a typical unstiffened beam-to-column connection as shown in Fig.1, in which M_L and M_R are end moments of the beams connected to the left and right side of the column flanges. In order to investigate the static behavior of the beam-to-column connection, the applied moments are replaced by a pair of statically equivalent forces in the beam flanges, F_L and F_R.Such kind of connection can be regarded as a planar structure, which is symmetrical about the centroidal axis of the column and beam members. Applied force is decomposed into the symmetrical and antisymmetrical components, and the response is calculated separatly. Finally, the full response is obtained by applying the principal of superposition.

ELASTO-PLASTIC BEHAVIOR OF SEMI-RIGID JOINTS

Consider an unstiffened beam-to-column connection (Fig.2) subjected to equal end moments M_b with relative rotation ϕ_b.When analyzing welded semi-rigid beam-to column connection, the relation between M_b and ϕ_b can be assumed to be linear until they reach the elastic limits M_{yb} and ϕ_{yb} respectively. $M_b = C_b\,\phi_b$, in which C_b is the relative rotational rigidity.

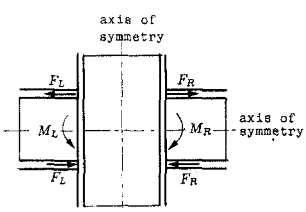

Fig. 1

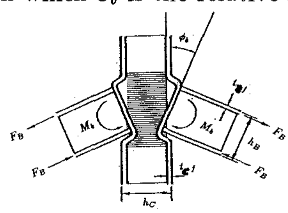

Fig. 2

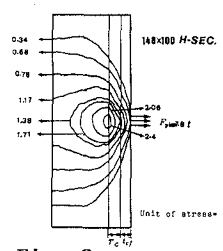

Fig. 3

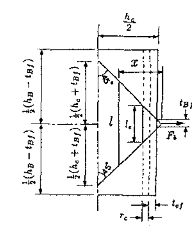

Fig. 4

Fig.3 shows the principal stress contours in the connection panel corresponding to the first yielding, which occurs at the fillet toe of the column web. It is suggested that the elastic effective length for the concentrated force F_b be diminished to zero in a straight line taper, as shown in Fig.4. From the model shown in Fig. 4, the following approximate formulas for ϕ_{yb}, and C_b are derived;

$$\phi_{yb} = \frac{F_{yb}}{E\,t_{cw}\,(h_B - t_{Bf})} \ln\left(\frac{t_{Bf}+h_c}{t_{Bf}+2t_{cf}}\right), \qquad C_b = \frac{E\,t_{cw}(h_B - t_{Bf})^2}{\ln\left(\frac{t_{Bf}+h_c}{t_{Bf}+2t_{cf}}\right)}$$

The difference of end moments of the beams connected to the column induces shear force in the connection panel, in this case tension and compression zones exist along with shear zones, therefore the relative rotation of joint, ϕ, is the sum of ϕ_s due to shear deformation and ϕ_b. The relation between M_s and ϕ_s is linear until they reach the elastic limit M_{ys} and ϕ_{ys}, respectively. $M_s = C_s\,\phi_s$.

Finally the ultimate moments of the semi-rigid connection have been derived for both symmetrical and antisymmetrical loadings. From the tests it has been reported that the interaction of the separately calculated ultimate moments for symmetrical and antisymmetrical loadings is negligible.

Fig.5 gives the calculated ultimate moment, M_{pb}, of the connection for various combination of beams and columns. The moment capacity is shown to increase monotonously with increased size of both beams and columns. Within the shaded area in the figure, the moment capacity of the connection exceeds the full plastic moment of the connected beam, M_{pB}, which means that the plastic hinge will be formed in the beam next to the connection. Fig.6 shows the moment capacity of the connection as a percentage of the full plastic moment of the connected beam. This percentage increases monotonously with increased size of columns, but decreases with increased size of beams. Fig.7 gives the ultimate moment, M_{ps}, of semi-rigid connection under antisymmetrical loading.

EXAMPLE OF FRAME ANALYSIS

In Fig.8 a single story two bay frame is shown. The elasto-plastic response for three different cases, fully rigid frame, semi-rigid frame (only with horizontal stiffeners), and semi-rigid frame (without stiffeners) was calculated and shown in Fig.9.

REFERENCES

[1] Witteveen J.,Stark W.B., 'Welded and bolted Beam-to-Column connections' by Journal of structural Div; Vol.108, No. St2. (1982), 433-455.

[2] Krawinkler H., and Popove E.P., 'Seisimic Behavior of Moment Connections and Joints' Journal of Structural Div; ASCE Vol.108 No. ST2, (1982),373-391.

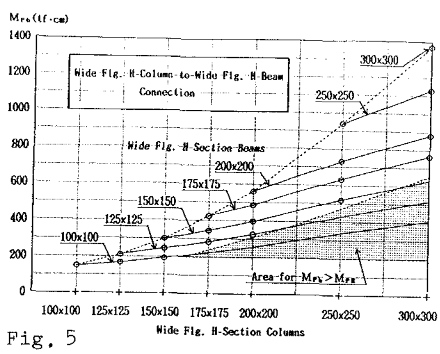

Fig. 5

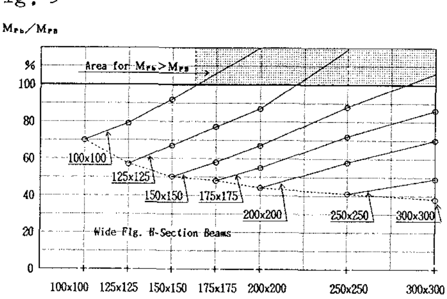

Fig. 6

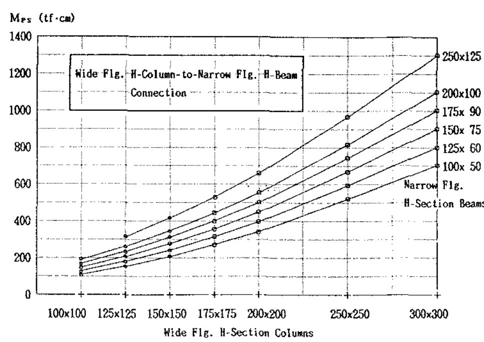

Fig. 7

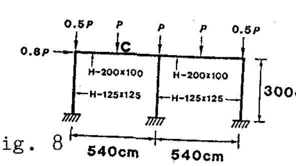

Fig. 8

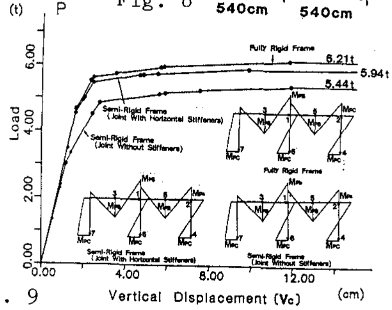

Fig. 9