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# PARTICIPATION OF BEAM-TO-COLUMN CONNECTION DEFORMATION IN HYSTERETIC BEHAVIOR OF STEEL FRAMES

Minoru YAMANARI[1], Hiroshi KANATANI[2], Mototsugu TABUCHI[2] and Teruyasu KAMBA[2]

[1]Department of Architecture, Kumamoto University
Kumamoto, JAPAN
[2]Department of Architecture, Kobe University
Kobe, JAPAN

## SUMMARY

This paper describes the effect of deformation of beam-to-column connections on hysteretic behavior of steel rigid frames with tubular columns. In analyses of rigid frames beam-to-column connections are usually assumed to be infinitely rigid. However, connections do not behave rigidly in steel frames with tubular columns under the influence of severe earthquakes. In this paper, contributions of beam-to-column connections to flexibility of frames in a post-elastic region are evaluated. The paper concludes with a discussion on the modeling of flexibility of beam-to-column connections.

## INTRODUCTION

In recent years, many authors have presented useful results regarding the effect of deformation of connections on behavior of steel frames under cyclic loading[1,2]. These studies have dealt with moment-resisting frames with wide-flange columns, paying attention only to shear deformation in the joint panels. However, in moment-resisting frames with tubular columns, not only shear deformation in the joint panels, but also local deformation of beam-to-column connections plays an important role in determining the hysteretic behavior of frames. The local deformation of a beam-to-column connection as shown in Fig.1 means out-of-plane deformation of column walls where the beam flanges are welded.

The general practice in analyzing steel structures is to assume that connections behave either as perfectly pinned or completely fixed elements. This approach may result in unconservative predictions of drifts of frame[3]. It is possible to incorporate effects of connection flexibility in an analysis by taking into account moment-rotation characteristics of connections, that are obtained from experimental results[4]. In this paper extensional slope-deflection equations are proposed for analyzing semi-rigid frames. Using these equations, numerical studies were performed to obtain relationships between total energy absorbed in steel frames and story drifts for differently proportioned frames.

## DEFORMATIONS IN STEEL SEMI-RIGID FRAME

A beam-to-column subassemblage in steel semi-rigid frames under lateral force deforms as shown in Fig. 2(a) and (b). Four components of deformation in the subassemblage shown in Fig. 2(c), (d), (e) and (f) is used to explain behavior of frames in this paper.

Both bending and shearing deformations of beams and columns are commonly used in conventional analysis of frames. Shearing and local deformations of connections are selected to describe the connection deformation. The shearing deformation of connection is the transformation of shape of the joint panel, while the local deformation of connection is out-of-plane deformation in the column walls caused by forces from the beam flanges. The local deformation of connections should be dealt with independently of the shearing deformation of connections, because the shearing deformation in the connection causes rotations at the ends of both beams and columns.

Referring to Fig. 3, shearing deformation of the joint panels is expressed by Eq. (1).

$$\gamma_p = \kappa_p \frac{Q_p}{GA_p} \tag{1}$$

where, $\kappa_p$ is the shape factor of the joint panel. $A_p$ is the cross-sectional area of the joint panel.

On the other hand, local deformation is given by Eq. (2).

$$\alpha = \frac{M}{k_l} \tag{2}$$

where, $k_l$ is the stiffness of the local deformation. This is obtained from test results[4].

## EXTENSIONAL SLOPE-DEFLECTION EQUATIONS

To incorporate the effect of connection flexibility in the slope-deflection equations, it is common practice to model connections as springs with shear-distortion and moment-rotation relationships described as Eqs. (1) and (2). In Fig. 4, the extensional slope-deflection equations for beam and column are expressed by Eqs. (3) and (4) respectively.

$$\left.\begin{aligned} M_{ij} &= 2EK_b\,(a\theta_{ij} + b\theta_{ji} + cR_{ij}' + d\gamma_i + e\gamma_j + f\alpha_{ij} + g\alpha_{ji}) + C_{ij} \\ M_{ji} &= 2EK_b\,(b\theta_{ji} + a\theta_{ji} + cR_{ij}' + d\gamma_i + e\gamma_j + f\alpha_{ij} + g\alpha_{ji}) + C_{ji} \end{aligned}\right\} \tag{3}$$

where, $a = \dfrac{2l(2+\gamma')+3B_i}{2l(1+2\gamma')}$, $b = \dfrac{2l(1-\gamma')+3B_j}{2l(1+2\gamma')}$, $c = -\dfrac{3(2l+B_i+B_j)}{2l(1+2\gamma')}$,

$d = -\dfrac{3B_i}{2l(1+2\gamma')}$, $e = -\dfrac{3B_j}{2l(1+2\gamma')}$, $f = d$, $g = e$ and $\gamma' = \dfrac{6\kappa_b EK_b}{GA_b l}$

$$\left.\begin{aligned} M_{ik} &= 2EK_c\,(a\theta_{ik} + b\theta_{ki} + cR_{ik}' + d\gamma_i + e\gamma_k) + C_{ik} \\ M_{ki} &= 2EK_c\,(b\theta_{ik} + a\theta_{ki} + cR_{ik}' + d\gamma_i + e\gamma_k) + C_{ki} \end{aligned}\right\} \tag{4}$$

where, $a = \dfrac{2l(2+\gamma')+3D_i}{2l(1+2\gamma')}$, $b = \dfrac{2l(1-\gamma')+3D_k}{2l(1+2\gamma')}$, $c = -\dfrac{3(2l+D_i+D_k)}{2l(1+2\gamma')}$,

$d = -\dfrac{3D_i}{2l(1+2\gamma')}$, $e = -\dfrac{3D_k}{2l(1+2\gamma')}$ and $\gamma' = \dfrac{6\kappa_c EK_c}{GA_c h}$

In the above equations, E is Young's modulus, $K_b$ and $K_c$ are the stiffness of beams and columns, and $A_b$ and $A_c$ are the cross section of beams and columns respectively.

Eqs.(3) and (4) assume the following:
(1) The axial deformation of the beam is neglected.
(2) The local deformation is independent from the shear deformation.

## CONTRIBUTION OF CONNECTION DEFORMATION

For the purpose of finding the contribution of beam-to-column connections to the energy absorbed by frames in the post-elastic range, three steel semi-rigid frames with tubular columns were analyzed,Fig.5. Table 1 shows the dimensions of members of the frames.

The contribution of the connection deformation to the total energy absorbed in frames was obtained from the results of analyses as shown in Fig.6. $C_{pl}$, $C_{ps}$ and $C_{ms}$ denote the contribution of the local deformation of connections, and the shearing deformation of joint panels and the shearing deformation of members respectively.

## FRAME TESTS

There were three types of 2-story,2-bay steel frames. The frames were called Frame A,B and C. Each frame had the circumferential reinforcing ribs at the beam-to-column connections as shown in Fig.7. All frames were subjected to cyclic lateral loads at the height of the beams.The loading system is outlined in Fig.8.

A preliminary test was conducted using a cyclic load in the elastic region for every frame. Then a load was applied cyclically as shown in Fig.9. Testing was terminated when one of the members or joints failed.

Monotonic curves[5] obtained from cyclic load-deflection curves are shown in Fig.10. From these tests, it was found that the geometric parameters of connections affected the elastic-plastic behavior of frames.

In the case of Frame B where the stiffness was the smallest for the types of frames used, the story drifts were calculated using the above equations. Fig.11 shows the contribution of the components in the frame drifts to the total frame drifts.

## CONCLUSIONS

The following conclusions may be drawn from this study.
1.It is possible to incorporate the effect of connection flexibility in the analysis by including the moment-rotation characteristics of the connection, obtained from experimental results.
2.The contribution of the connection deformation to the total energy absorbed by the frames varies with the volume of the joint panels.
3.Closer agreement between test data and analyzed results was obtained when shearing deformations and local deformations of beam-to-column connections were included in the analysis.

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**Table 1 Dimensions of beams and columns**

| Type of Beam | $H \times B \times t_w \times t_f$ (mm) | A ($cm^2$) | Yield Strength ($tonf/cm^2$) | $Kappa_b$ |
|---|---|---|---|---|
| B1 | 400 200 8 13 | 81.4 | 2.4 | 2.81 |
| B2 | 496 199 9 14 | 101.3 | 2.4 | 2.41 |
| **Type of Column** | **$D \times D \times t$ (mm) D/t** | **A ($cm^2$)** | **Yield Strength ($tonf/cm^2$)** | **$Kappa_c$** |
| C1 | 250 250 12 20.8 | 114.2 | 2.4 | 2.03 |
| C2 | 350 350 12 29.2 | 162.2 | 2.4 | 2.03 |

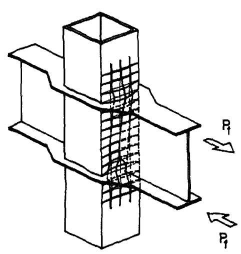

**Fig.1 Local deformation**

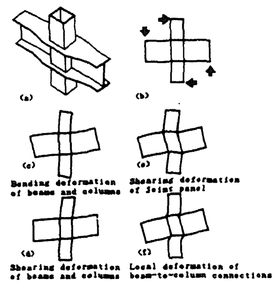

**Fig.2 Deformations in a subassemlage**

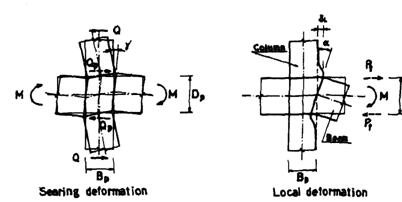


**Fig.3 Deformations of a joint panel and a connection**

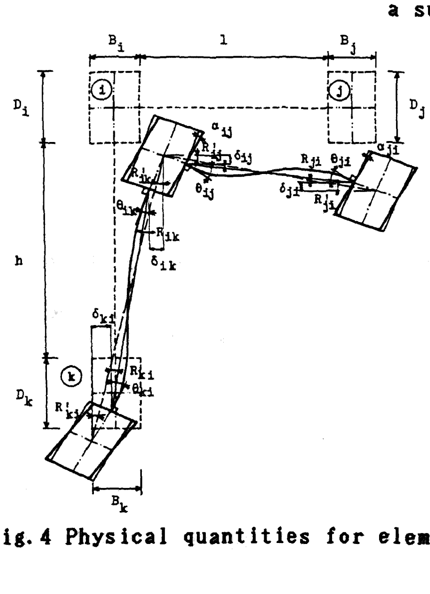

**Fig.4 Physical quantities for elements**

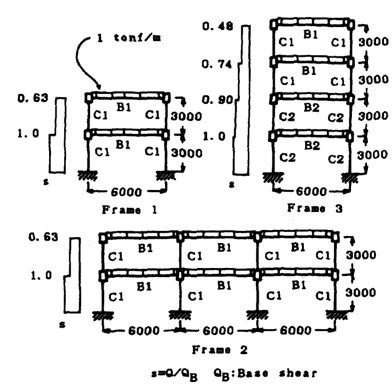


**Fig.5 Examples**

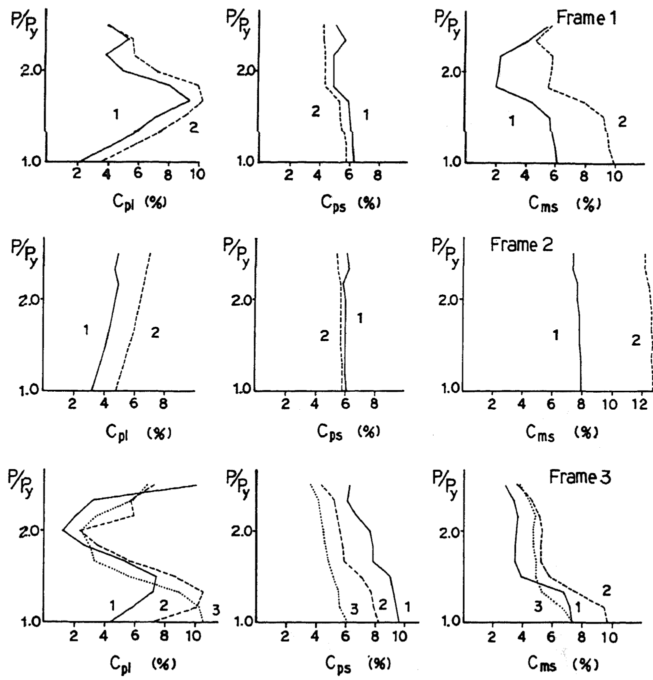


**Fig. 6 Contributions of connection deformations to energy absorbed in frames**

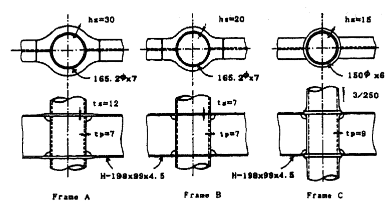


**Fig. 7 Details of specimens**

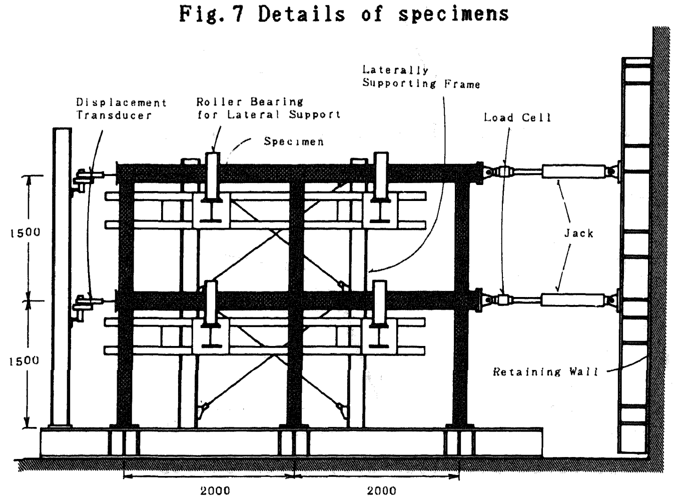


**Fig. 8 Test set-up**

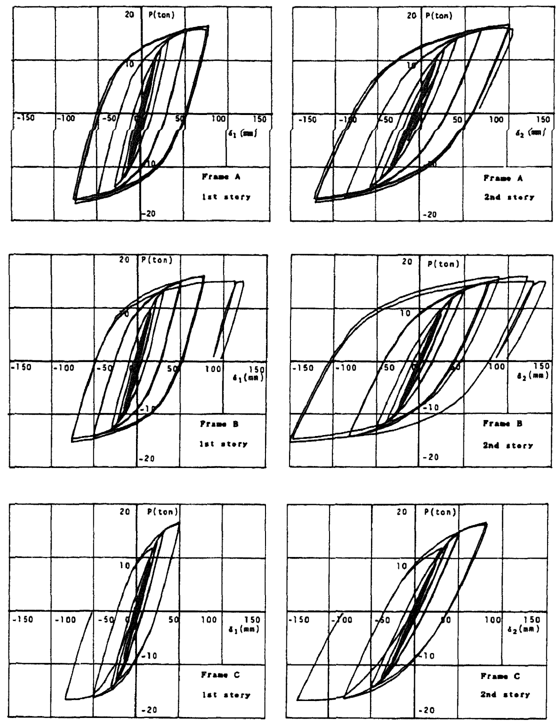


Fig. 9 Load-deflection curves

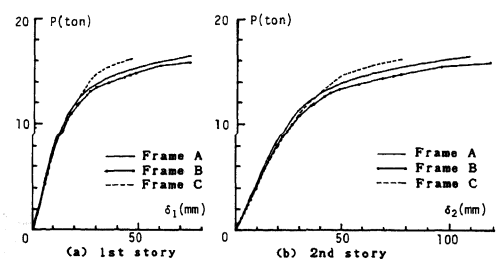


Fig. 10 Monotonic load-deflection curves

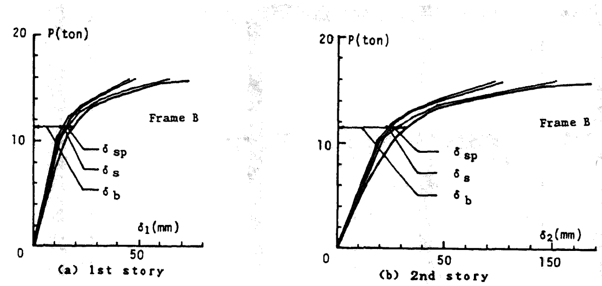


Fig. 11 Calculated load-deflection curves