EARTHQUAKE ANALYSIS OF STEEL FRAMES
WITH NON-RIGID JOINTS

by

Jorge Vasquez[I], E.P. Popov[II], V.V. Bertero[II]

SYNOPSIS

A portion of a steel column bounded by the flanges of the beams that frame into a joint is known as the panel zone. This zone experiences shear deformations regardless of the beam connection employed. A frame model which includes the non-rigidity of the panel zone is discussed in this paper. A procedure for assembling the structure's stiffness matrix, considering the deformable joint, is presented. Some results of the analysis of a specific frame are also included.

INTRODUCTION

The flexibility or non-rigidity of the portion of the column bounded by the flanges of the beams that frame into a joint is becoming well recognized. The deformation of this portion, known in structural steel literature as the "panel zone" (PZ), is due to the existence in it of very large shear forces, and is not related to the connecting media itself, which is assumed rigid.

The effect of the PZ deformation on the behavior of a frame is of the same order of magnitude, at least in the elastic range, as the correction due to the use of the full span lengths instead of their clear span. Therefore, the depths of columns and girders are included in the analysis.

PANEL ZONE BEHAVIOR

Structural joints in frames are regions of high stress concentrations. A reasonably accurate analysis of the PZ's mechanical behavior can only be made from a continuum mechanics approach. A model of this behavior based on a "strength of materials" description of columns and girders has to be rather simplistic.

In formulating a simplified interpretation of PZ mechanics, the following assumptions were made:

a) The single component of a PZ's distortion is a shear deformation γ, by virtue of which the rotation of the columns, ϕ_c, and the rotation of the beams at the joint, ϕ_b, need no longer be identical. Accordingly, the PZ shear force, (which has

[I]Associate Professor of Civil Engineering, Universidad Catolica de Chile.
[II]Professor of Civil Engineering, University of California, Berkeley.

the dimensions of moment) can be found to be

$$M = .5(M_R+M_L)-.5(M_T+M_B)+.25h_b(Q_T-Q_B)-.25h_c(Q_L-Q_R) \quad (1)$$

The quantities of the right hand side of this expression are defined in the list of symbols.

b) The constitutive relationship for the PZ is the simplest of those proposed by Krawinkler, Bertero and Popov in Reference 1. Namely, an expression of the type

$$M = S\gamma \quad (2)$$

where S represents a stiffness of a bilinear nature, defined through an elastic stiffness, a plastic limit and a strain - hardening rate.

These assumptions are reasonable for PZ's in interior columns, where both girders framing into the joint are approximately of the same depth. Their extension to exterior columns and top story joints had to be accepted without experimental subtantiation.

STRUCTURAL ASSEMBLAGE

The degrees of freedom that have to be considered for each joint are the horizontal displacement u, the vertical displacement v and the rotations ϕ_b and ϕ_c already defined. From a virtual work analysis, one can conclude that the force components associated with ϕ_b and ϕ_c are

$$M_b=M_R+M_L+.5h_b(Q_T-Q_B) \qquad M_c=M_T+M_B+.5h_c(Q_L-Q_R) \quad (3)$$

respectively. Equation (1) can then be written

$$M = - .5 (M_c - M_b) \quad (4)$$

Assume now that the stiffness matrices of columns and girders are assembled into the structure's stiffness matrix. Once this process is completed, the partially assembled stiffness matrix can be recognized to represent a fictitious structure, identical to the given one, except that all PZ's are infinitely flexible.

It is convenient to think of this fictitious stiffness matrix partitioned so as to group the degrees of freedom into partial vectors $\underline{u}$, $\underline{v}$ $\underline{\phi}_b$, $\underline{\phi}_c$. In its partitioned form, the stiffness equation for the fictitious frame will read

$$\begin{vmatrix} \underline{K}_{uu} & \underline{K}_{uv} & \underline{K}_{ub} & \underline{K}_{uc} \\ & \underline{K}_{vv} & \underline{K}_{vb} & \underline{K}_{vc} \\ & & \underline{K}_{bb} & \underline{K}_{bc} \\ \text{Sym} & & & \underline{K}_{cc} \end{vmatrix} \begin{vmatrix} \underline{u} \\ \underline{v} \\ \underline{\phi}_b \\ \underline{\phi}_c \end{vmatrix} = \begin{vmatrix} \underline{F}_u \\ \underline{F}_v \\ \underline{M}_b \\ \underline{M}_c \end{vmatrix} \qquad (5)$$

The PZ stiffness equations can be written, assembling the expression for all joints in one matrix equation, as follows

$$- .5\ (\underline{M}_c - \underline{M}_b) = \underline{S}\ (\underline{\phi}_c - \underline{\phi}_b) \qquad (6)$$

and the same can be done with the joint moment equillibrium equations

$$\underline{M}_c + \underline{M}_b = 0 \qquad (7)$$

Using equations (6) and (7) to solve for $\underline{M}_c$ and $\underline{M}_b$ in terms of $\underline{\phi}_c$ and $\underline{\phi}_b$, substituting into (5) and regrouping the equations, one obtains

$$\begin{vmatrix} \underline{K}_{uu} & \underline{K}_{uv} & \underline{K}_{ub} & \underline{K}_{uc} \\ & \underline{K}_{vv} & \underline{K}_{vb} & \underline{K}_{vc} \\ & & \underline{K}_{bb}+S & \underline{K}_{bc}-S \\ & & & \underline{K}_{cc}+S \end{vmatrix} \begin{vmatrix} \underline{u} \\ \underline{v} \\ \underline{\phi}_b \\ \underline{\phi}_c \end{vmatrix} \begin{vmatrix} \underline{F}_u \\ \underline{F}_v \\ 0 \\ 0 \end{vmatrix} \qquad (8)$$

Equation (8) shows how the PZ stiffness has to be assembled into the structure's stiffness matrix.

APPLICATIONS

A computer program for the elasto-plastic analysis of frames that includes the model described above has been developed (2). In addition, this program considers geometric second order effects in columns, gravity loads in beams, with the possibility of in-span plastic hinges, column shortening and interaction of axial and flexural deformations in column plastic hinges.

A run was made for a ten story, four bay frame, considering an excitation of 1.5 times the accelerations of the first ten seconds of the 1940 NS El Centro earthquake record. Ductility factors of the order of ten were found for most PZ's. This was due to the use of unreinforced PZ's. In future work, for ductility factors of this order, a trilinear model will have to be used.

Comparisons were also made with the case where the PZ's are considered rigid. Table I shows the ratio of maximum story displacements of the rigid assumption with respect to the deformable one. It can be seen how this frame, under the rigid joint assumption, exhibits a soft first story; the PZ deformability leads to a more gradual increase in deformation towards the top of the frame.

TABLE I

STORY	RATIO	STORY	RATIO
1	1.79	6	.753
2	1.29	7	.752
3	1.01	8	.710
4	.841	9	.691
5	.810	10	.674

GLOSSARY OF TERMS NOT DEFINED IN TEXT

M_L, M_R End moment of left or right beam at the face of column.

M_T, M_B End moment of top or bottom columns at the face of girder.

Q_L Q_R Q_T, Q_B End shear force of left or right beam, of top or bottom column.

h_c, h_b Depth of column or beam.

F_u, F_v Joint force in horizontal or vertical direction.

REFERENCES

1. Krawinkler, H., Bertero, V.V. and Popov, E.P., "Inelastic Behavior of Steel Beam-to-Column Subassemblages", Earthquake Engineering Research Center, University of California, Berkeley, 1972.

2. Vasquez, Jorge, "Seismic Response of Unbraced Steel Frames with Panel Zone Deformations", Ph.D. Thesis, University of California, Berkeley, 1972.