HYSTERESIS MODELS OF STEEL MEMBERS FOR
EARTHQUAKE RESPONSE OF BRACED FRAMES

by

Ashok K. Jain[I] and Robert D. Hanson[II]

SUMMARY

Two multilinear models are presented to describe the hysteresis behavior of steel bracing members and columns. The buckling model can be used for axially loaded bracing members and the end moment-buckling model can be used for columns and bracing members where end moments are as significant as the axial forces. This end moment-buckling model permits study of the effects of column buckling on structural response. These models have been used to study the response of a seven-story concentric K braced frame subjected to a severe horizontal earthquake ground motion.

INTRODUCTION

Bracing members are considered effective earthquake-resistant elements in steel frames. A number of investigations (1-5) have been reported which modelled the post-buckling hysteresis behavior of bracing members. None of the hysteresis models which are practical for use in nonlinear structural analysis programs have accurately included the increase in member length and reduction in compressive strength with number of cycles as observed experimentally. These factors are included in the buckling model described here.

The authors (1) have analyzed the seismic response of 18 concentric and eccentric braced steel frames and concluded that bracing members of small and intermediate slenderness ratio ($KL/r<100$) may develop significant end moments in addition to axial forces. Also, building columns may be subject to buckling conditions. A hysteresis model which considers the interaction between axial buckling and flexural inelasticity in steel members was needed. One model for this interaction is described.

These two relatively simple models were developed for use with the DRAIN-2D computer program developed by Kanaan and Powell (4) at the University of California, Berkeley.

BUCKLING MODEL

Singh (5) presented a multilinear hysteresis model for pin-ended bracing members. The authors compared their experimental hysteresis curves

[I] Lecturer in Earthquake Engineering, University of Roorkee, Roorkee, U.P., India.
[II] Chairman and Professor of Civil Engineering, University of Michigan, Ann Arbor, Michigan, U.S.A.

with those obtained from Singh's model and suggested that his model would be improved by including the effects of change in cyclic compression forces and residual elongation. Residual elongation is defined as the net increase in length of a member, at zero force, which results from inelastic compressive cyclic deformation. These two parameters will be discussed next using experimental results of steel tube and angle specimens whose effective slenderness ratio varied from 30 to 160.

Reduction in Compression Force

Maximum compressive forces in 1st, 2nd and 16th cycles are shown in Fig. 1 as a ratio of the maximum compression force obtained in the first cycle. It can be seen that there is a significant reduction in compressive strength between the first two cycles. In subsequent cycles, the reduction continues but at a slower rate. The maximum compressive strength, P_{ync}, in the second and subsequent cycles varies between 0.3 and 0.6 times of that in the first cycle as shown in Fig. 1 by lower and upper bounds. Similar observation was made for the results of angle specimens (1).

Residual Elongation

Residual elongation expressed in terms of residual strain is shown in Fig. 2 for tube specimens with compressive displacements of $3\Delta y$, $6\Delta y$, $9\Delta y$ and $12\Delta y$. It can be seen that residual strain is directly proportional to the maximum compressive displacement Δ and is inversely proportional to the effective slenderness ratio, KL/r of the member. An empirical equation of the following type can be used to represent the residual strain:

$$\varepsilon = 0.0175 \left[\frac{0.55\Delta}{(KL/r)} + 0.0002\Delta^2 \right] \qquad \text{(Eq. 1)}$$

From the results of tube and angle specimens it was noticed that residual strain obtained in the first few cycles is about 1.5 to 2.0 times of that obtained in subsequent cycles. Therefore, the residual strain given by Eq. 1 includes a factor of 1.75 to represent the average residual strain irrespective of the numbers of cycles.

Description of the Model

The maximum compressive force in the first cycle, P_{yn}, is determined from the American Institute for Steel Construction formulas (6). Once the member buckles at point B (Fig. 3) it follows segment BC having a negative slope. The point C corresponds to a compressive displacement equal to five times the tension yield displacement, Δy, of the member.

From point C, continued compression results in segment CD. If the direction of axial displacement is reversed at D, it results in compression force decreasing to zero followed by an increasing tensile force along segment DFE''. To locate the point F, a line A''F' is drawn from the new origin A'' (A'A'' = E'E'') at a slope of minus 1/3 times the initial elastic slope AE to intersect the line DE'' at F'. Intercept A''F is taken as $60/(KL/r)$ times the distance A''F'. E'E'' is the residual elongation of the member calculated from Eq. 1.

Once the member attains tension yield strength, P_{yp}, it may continue to stretch at constant force or may unload along E'A''. Maximum compressive strength, P_{ync}, in the second and subsequent cycles, varies between 0.3 and 0.6 times of that in the first cycle depending upon the slenderness ratio of the member.

END MOMENT-BUCKLING MODEL

This model consists of two components, the buckling component just described (Fig. 3) and a moment-axial force (M-P) interaction component (Fig. 4).

The axial force is determined by the buckling component which is then used to compute the modified plastic moment capacity of the member at the two ends using the M-P interaction component. Flexural stiffness is assumed to be independent of the axial force in this model. If the moment lies on or outside the yield interaction surface, a plastic hinge is introduced at that end. Combinations outside the yield surface are permitted only temporarily; they are returned to the yield surface by applying corrective loads in the following time steps. Once the axial force in the post-buckling region equals P_{ync} the program redefines the four branches of the M-P interaction curve in the compression region as shown by dashed lines in Fig. 4. Maximum compressive strength in all subsequent cycles remains at P_{ync}.

STRUCTURE RESPONSE

To demonstrate the results obtained using these two models, a seven-story concentric K braced interior frame from a 3-bay building is selected as shown in Fig. 5a. The dead and live loads are 65 psf (3.1 kN/m^2) and 100 psf (4.8 kN/m^2), respectively. The A36 steel frame was designed using an allowable stress design procedure in accordance with the 1976 Uniform Building Code (7). Horizontal force factor K equal to 1 was used to compute the base shear equal to 260 kips (1160 kN) from a contributory building weight of 3250 kips (14,500 kN). The members of this braced frame were designed for forces increased by 1.25 above the basic base shear value and the bracing members were designed without permitting 33% increase in the allowable stresses. The N-S component of the May 1940 El Centro earthquake with ground accelerations increased by a factor of 1.5 is used to study the structure's inelastic response. The column behavior is simulated by the end moment-buckling model, beam behavior by an elastoplastic hysteresis model and the bracing member behavior by the buckling model. Additional detail for this structure and structures with other member sizes and configurations are given in Ref. 3.

RESPONSE PARAMETERS

The parameters selected to report the earthquake response of the structure are: the maximum horizontal displacement of floors relative to ground, maximum story displacements, vertical displacement time history of the upper end of the buckled column, and maximum ductilities for the frame members. The ductility for the girders and columns is defined as the ratio of the maximum end rotation to the yield rotation where the normalizing yield rotation is obtained by assuming that a member has equal end rotations and no joint translations. As will be seen this ductility definition is not realistic for columns which buckle. The ductility for bracing members is defined as the ratio of maximum axial deformation to its tension yield elongation.

DISCUSSION OF RESULTS

In the response of DK-1 it was noted that the first story columns were about 3% below their specified buckling strength. For DK-1M the first story columns had their specified buckling strength decreased by 3.5%. The subsequent analysis showed that the first story left column buckled. The floor displacements and member ductilities are compared in Fig. 5b to Fig. 5g. Three obvious observations must be stated:

a. Once a column buckles, it cannot become straight in the subsequent cycles.

b. Although the reported member ductilities are smaller for the buckled column case, the consequences for the real building are not smaller.

c. The column ductility of 3.1 reported in the first story does not consider the buckling deformation. If the axial deformation definition had been used the column ductility would have exceeded 20.

The DK-1M structure was also subjected to 1.75 times the 1940 El Centro N-S component accelerogram. In this case no columns buckled, but more bracing members buckled.

CONCLUSIONS

1. Two models for the hysteresis behavior of axially loaded steel bracing members and columns have been described. These models are based on both theoretical and experimental considerations.

2. These models are useful in studying the effects of buckling of columns and bracing members on the inelastic seismic response of braced steel frames.

ACKNOWLEDGMENTS

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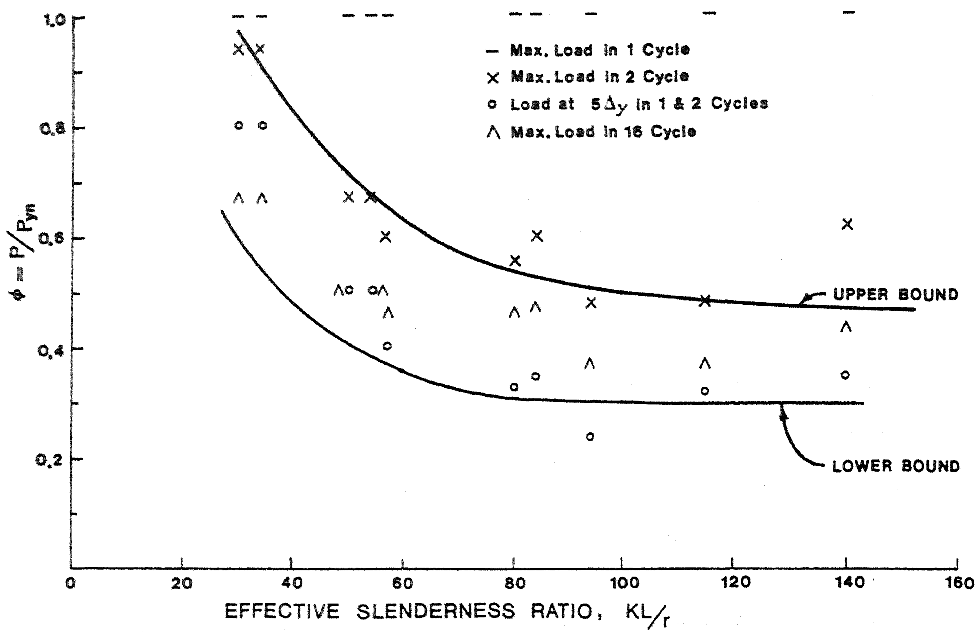

Figure 1. Reduction in Compressive Force

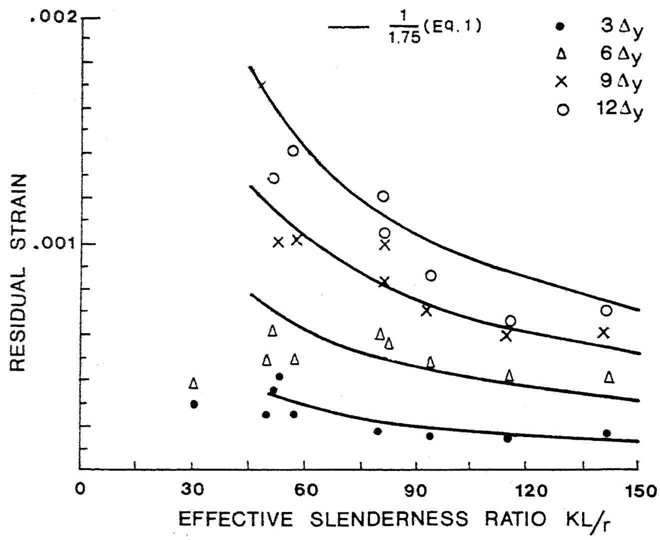

Figure 2. Average Residual Strain in Cycles 5 through 16

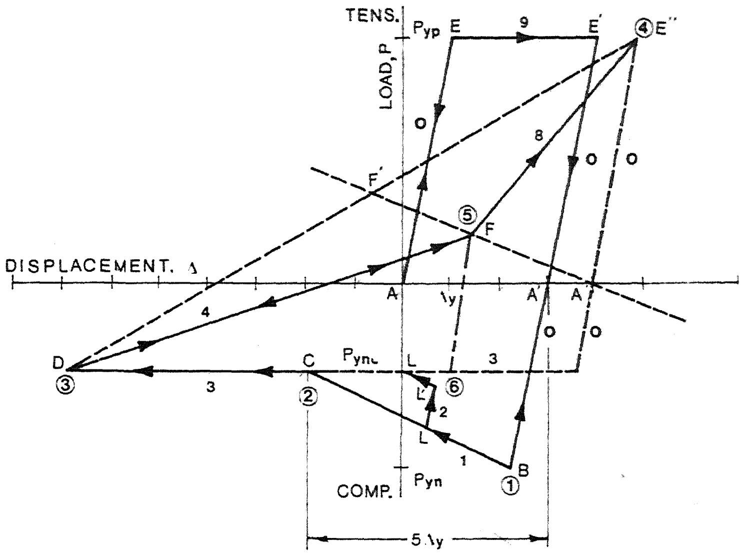

Figure 3. The Buckling Component

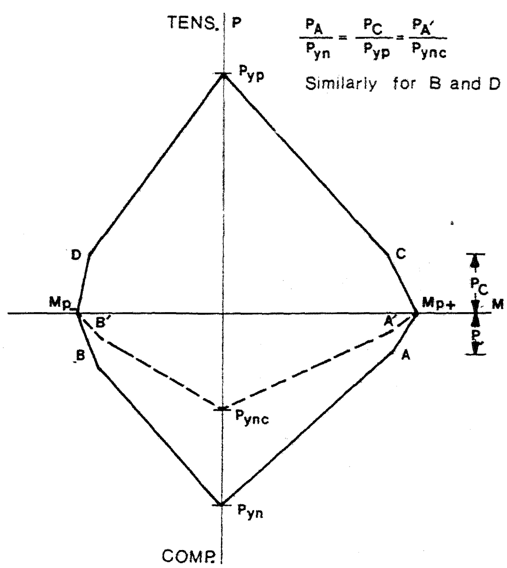

Figure 4. The Moment-Axial Force Interaction Component

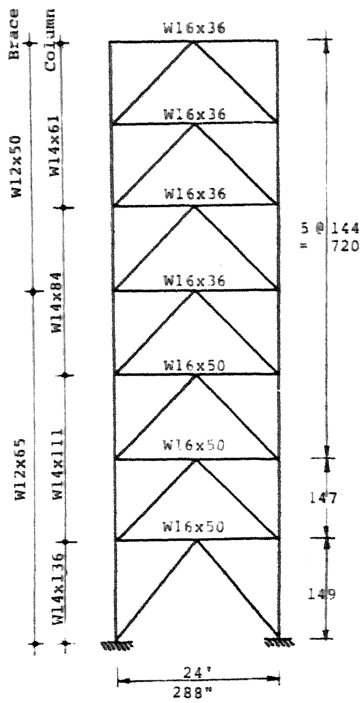

Fig. 5(a)

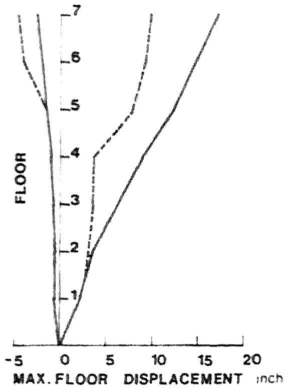

Fig. 5(b)

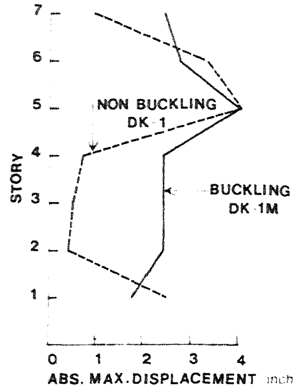

Fig. 5(c)

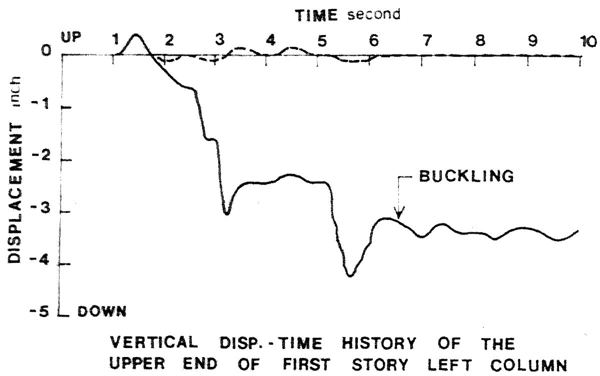

Fig. 5(d)

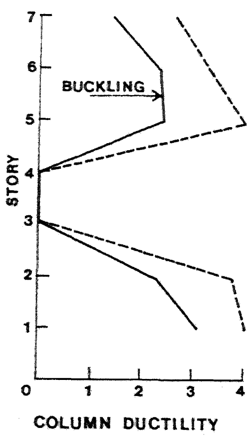

Fig. 5(e)

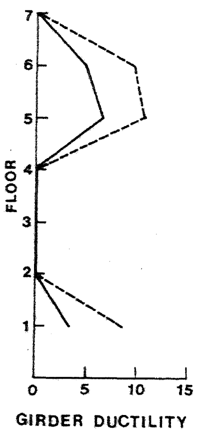

Fig. 5(f)

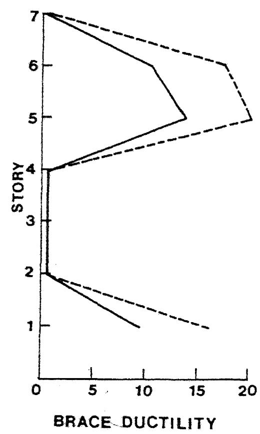

Fig. 5(g)

Figure 5. Example Building Frame Inelastic Response
With and Without Column Buckling