Behavior of Steel Frames Under Repeated Loading

PROPOSAL FOR SECOND SERIES OF TESTS ON STEEL FRAMES
SUBJECTED TO CYCLIC LATERAL DISPLACEMENTS

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1. INTRODUCTION AND PROBLEM AREAS TO BE STUDIED

An understanding of the elastic and inelastic behavior of steel frames under the action of constant gravity loads and cyclic lateral displacements is essential in the development of reliable methods for predicting the dynamic response of such structures during an earthquake. Recently completed tests of Frames A and B (first series of test frames) at Lehigh University under this type of loading showed nearly a 50 per cent increase in the maximum story shear over the maximum story shear predicted under monotonically increasing loads. In addition, the test results showed a substantial toughness of the frames against the cycles of horizontal displacements of gradually increased amplitudes. In particular, Frame A, the one-story, single-bay frame, sustained 60 cycles without faltering and Frame B, the three-story, single-bay frame of identical member sizes, geometry and loading as Frame A, was cycled 54 times. In both cases, at each selected amplitude of displacement, the frame was cycled five times and the resulting hysteresis loops compared closely.

Significantly, in these initial tests, inelastic behavior was permitted to occur only the beams of these frames which were designed according to the provisions of the SEAOC lateral force requirements and the current aseismic design practice. The columns were designed by the allowable-stress method using the AISC interaction formula. The effect of axial loads on the stiffness of columns is not considered.
in design and no "plastic hinges" form in the columns. Therefore, a more optimum design would include the participation of the maximum strength of the columns as well as the maximum moment capacity of the beams.

Recently, tests have been conducted at the University of California on cantilever beams which are framed into the weak-axis direction of the column stub. The behavior of this type of connection in a frame as well as the behavior of a column oriented for weak-axis bending is of great importance, considering the random directions from which a three-dimensional structure is disturbed by an earthquake.

Many of the cantilever beam specimens tested at the University of California have shown pronounced local buckling of the beam flanges. The buckle is alternately produced and then straightened by the next half-cycle of loading. This behavior is unlikely to occur in frames due to the initial bending imposed on the beam flanges by the gravity loading. Therefore, the local buckle should appear at a certain time during the test but it may not disappear completely on the next half-cycle. In addition, for larger and larger cycles the buckle would become more pronounced and may have significant influence on the overall behavior of the frame under additional cycles.
2. PROPOSED TEST PROGRAM

2.1 Description of Test Frames

Considering the current research work in earthquake engineering and the additional areas of interest as discussed in the Introduction, the following tests are proposed.

First, a single-story single-bay frame, Frame C, is proposed which will be identical to Frame A in every way except that the beams to be used will be a welded section having a b/t of 20 (The limiting value for A36 steel is 17 according to the AISC Specification). The dimensions of this frame are shown in Fig. 1 (a). This specimen will develop the local buckle on the lower flange at the column face. The effect of the local buckle at both ends of the beam on frame behavior can be found by direct comparison with the results from the test of Frame A.

Second, a single-story single-bay frame, Frame D, is proposed with an 8W48 column oriented for weak-axis bending. The beam size, as shown in Fig. 2 (a), is 12W27 and the same connection detail will be used as in the University of California's recent tests labeled "Type W1". The results of this test will permit an evaluation of the performance of framed columns with weak-axis bending.

Third, a two-story, single-bay frame, Frame E, is proposed with the 12W27 beams and using essentially an 8W24 column oriented for strong-axis bending. This frame was sized to permit the columns to reach their reduced plastic moment values as well as allowing plastic hinges in the beams. Cover plates were added to the lower half-story columns to prohibit the
formation of plastic hinges in those columns initially. These plates were also sized so that the reduced plastic moment values in the upper and lower half story columns were reached at nearly the same load. The member sizes of the test frame are shown in Fig. 3 (a).

2.2 Design Loads Assumed

The loads used for design and analysis of the proposed Frames C, D, E are the same, as those used on the previous Frames A and B. The working gravity loads as well as the working value of horizontal shear used to design Frame B are shown in Fig 4 along with the gravity loads to be applied to each test specimen.

2.3 Design Procedure

Proposed Frame C is essentially a duplicate of Tested Frame A except for the geometric shape of the beam cross section. The b/t ratio was set at about 20. Then, using the same depth as the 10W29 beam used in Frame A, the component plates of the welded section were sized to give the same elastic section modulus as well as the same plastic moment capacity. The resulting cross section is shown in Figure 1 (b). The beam-to-column connections will be identical to those in Frame A.

In the design of Frames D and E several factors were considered.

1. The predicted value of the maximum shear that can be applied to either frame should be nearly the same as that predicted for Frames A and B. In addition, since the strength of the proposed frames will be about the same as the frames already tested, the same test set-up and equipment can be reused.
2. The column sizes selected should permit major participation by the columns in the inelastic behavior of the frames.

3. Possibility of using a typical weak-axis beam-to-column connection detail which has been tested under cycled loading at the University of California.

In the design of Frame D, the one-story one-bay frame with columns oriented for weak-axis bending, a 12W27 beam was selected along with 8W48 columns. The 8W48 column when it is oriented for weak axis bending has about 80 per cent of the strong-axis strength of the 8W40 column used previously. But, since its stiffness is less than the 8W40, a more stiff beam the 12W27 was selected to be the companion member. In addition, the 12W27 beam has 10 per cent greater strength and 30 per cent greater stiffness than the 10W29 used previously. Therefore, the 8W48 column and 12W27 beam were selected on the basis of the overall strength and stiffness requirements of the frame. The weak-axis beam-to-column connection detail to be used is the same as that used in recent tests at the University of California called "Type W1". The details for this connection for Frame D are shown in Figure 2 (b).

For Frame E, the two-story frame, 12W27 beams were used in conjunction with 8W24 columns oriented for strong-axis bending. This frame was selected as two-story to permit hinges at both ends of a full length column as well as to permit the test set-up to accommodate displacements to be applied to the frame relative to those applied to Frame B. Cover plates were added to a portion of the lower half-story columns so that a hinge would not form in that column first and also to isolate on the load deflection curve the occurrence of the first two hinges in the full length column. The cover plates and connections details are shown in Fig. 3 (b).
The test will therefore provide information on the cyclic behavior of a double curvature column having plastic hinges forming at both ends. Additional considerations for making the frame only two stories high are concerned with instrumentation requirements as described in Section 3.

2.4 Analytical Behavior of Test Frames

The three proposed frames were analyzed for constant gravity loads and a monotonically increasing horizontal load applied to the top of the frame.

Frame C

Since this frame is made up of 8W40 columns and a welded beam having the same strength and stiffness of a 10W29 beam, the analysis yields the same results as for Frame A. The resulting load-deflection curve in Fig. 5 (a) is computed by a second order elastic-plastic analysis. The frame reaches its maximum resistance to horizontal load at the mechanism load for the frame of 14.8 kips.

Frame D

Using Frames A and B as the basis, the design of this frame resulted in using an 8W48 column oriented for weak-axis bending in conjunction with a 12W27 beam. The second order elastic-plastic analysis yielded the load-deflection curve shown in Fig 6 (a). Here again the mechanism load for the frame and maximum horizontal load resistance coincide at 17.1 k. In this frame a similar hinge pattern was found as in the previous frames, that is, the plastic hinges are located only in the beam. However, in the lower column the ratio of maximum axial load to the yield load for the section is somewhat larger than in the previous frames. Therefore, when axial load and bending are added at the top of the lower column at
at the maximum horizontal load, the top of this column has surpassed the bending moment when first yielding occurs. This moment will increase slowly under additional deflection beyond the deflection at the maximum horizontal load. Then, considering the beneficial effects of strain-hardening in the beam hinges, a third hinge may form at this column location after the maximum horizontal load is reached.

**Frame E**

Using the 12W27 beams and 8W28 columns, this two-story, single bay frame was analyzed by second and first-order theory. In this analysis, the reduced strength of the 8W28 columns which were oriented for strong-axis bending was taken into account. But, the sway moment (P - Δ effect) was only included through hinge number three. The hinge pattern and order of hinge formation is shown in the load-deflection curve in Figure 7 (a).

The maximum horizontal load is 18.7 kips after the formation of six hinges. Considering that some second-order effects have been neglected in computing $H_{\text{max}} = 18.7 \text{ kips}$, the actual second-order maximum horizontal resistance should be about 16 kips.
3.1 Loading Program

The same general test setup will be used as for Frames A and B. Initially, the working gravity loads will be applied to the beams and column tops of the frames. A program of cycled lateral displacements similar to the technique used for Frames A and B will be applied to the top of the frames. This program will include five replications of the cycle at each amplitude. In each case, the sizes of the hysteresis loops are selected to bracket the plastic hinge occurrences on the respective load-deflection curves. The horizontal displacement programs are shown in Fig. 5 (b) for Frame C, in Fig. 6 (b) for Frame D and in Fig. 7 (b) for Frame E.

As in the previous testing program, the actual mechanical properties of the A36 material will be determined by coupon tests and the residual stress distribution will be found.

3.2 Test Equipment

The vertical loads will be applied to the frame utilizing the gravity load simulators in the manner used successfully on the previous tests. These loads are automatically maintained even under the displacements of the frame.

The horizontal displacements will be applied by a motor-driver mechanical screw jack. This feature is necessary due to the extent of time and manpower required to perform the tests of Frames A and B.

The test specimens will be assured of planar motion by the lateral braces used previously. The positioning of the braces will be the same as for Frames A and B.
3.3 Instrumentation

In the tests of Frames A and B extensive measurements were taken. Many of these measurements were automatically recorded but many others were recorded by hand utilizing mechanical or electrical devices. The proposed tests will be monitored automatically as much as possible to cut down on time and manpower requirements. This will be accomplished by reading the loads in the dynamometers, the strains throughout the members, the rotations of various points on the members and the horizontal deflections of the columns automatically. The vertical deflections of the beams may also be measured electrically. Therefore, by utilizing the motor-driven jack monitored by a load cell and a displacement transducer, the television monitor to scan the frame, and the automatic recording of all the data (save for the necessary periodic mechanical checking of various references), the tests should move along more smoothly with less manpower requirements. This should permit a larger number of cycles to be applied to the frame in one-third the time of the previous tests. More on-line data reduction is also anticipated for the proposed tests.
Three tests on welded steel (A36) frames are proposed. Frame C, a single-story, single-bay frame, having a beam with b/t = 20. Frame D, a single-story, single-bay frame, having its columns oriented for weak-axis bending. Frame E, a two-story, single-bay frame, having strong-axis orientation of columns which have plastic hinges at the ends.

The frames will be subjected to constant gravity loads and a program of cycled horizontal displacements of the top of the frames.

It is believed that the proposed tests would yield useful information on the behavior of repeatedly loaded frames, with particular emphasis on flange local buckling, weak-axis column bending and plastic hinge formation in columns.
5. FIGURES
FRAME C

8WF40 Columns; 10WF29 Nominal Beam
(Welded with b/t = 20)

(a)

Beam Cross Section
(b/t = 20)

Flange Rs 7½" x 3/8"

Web Rs 9½" x 5/16"

FIG. 1 FRAME C AND BEAM CROSS SECTION
FRAME D

8 WF 48 Column; 12 WF 27 Beam
(Oriented for Weak-Axis Bending)

(a)

(b)

FIG. 2 FRAME D AND CONNECTIONS DETAILS
8W-28 Columns; 12W-28 Beams
(Plus Cover Plates in Lower Story)

(a)

West Column

Diagonal Stiffeners
(All Plates ½" x 3½")

(b)

East Column

Doubler Plates
(¼" Plates Both Sides)

FIG. 3 FRAME E AND CONNECTION DETAILS
Prototype Frame

Working Loads used in the Design of Test Frames

Gravity Dead Load 80 psf = 1440 lbs/ft

Full Design Live Load 80 psf = 1440 lbs/ft

\[
P_{DL} = \frac{W_{DL} (L)}{2} = \frac{1440 (15)}{2} = 10.8 \, k
\]

\[
P_{LL} = 10.8 \, k; \quad P_{LL} (design) = 60\% \quad P_{LL} = 6.48 \, k
\]

\[
P_{TOTAL} (design) = 17.28 \, k
\]

Earthquake \( H = H_1 + H_2 + H_3 + H_4 + H_5 = 5.19 \, k \)

FIG. 4  "FRAME DIMENSIONS AND LOADINGS"
FIG. 5 FRAME C: LOAD DEFLECTION CURVE AND DISPLACEMENT PROGRAM
Fig. 6 Frame D: Load-Deflection Curve and Displacement Program
FIG. 7  FRAME E: LOAD-DEFLECTION CURVE AND DISPLACEMENT PROGRAM
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7. **APPENDIX**
The Appendix to this report shows the evaluation of the AISC code interaction equation for critical points in the members of Frames D and E under combined axial load and bending moment. For each Frame two loading cases are evaluated; gravity loads alone and gravity loads plus the working value of horizontal shear.

Even though the evaluation shows the columns to be overstressed for both frames, the previous elastic-plastic analysis of each frame shows no inelastic action in the frames under these working load conditions. In fact, the respective load-deflection curves show the first hinge forms in Frame D at 13.5 kips and in Frame E at 9.1 kips horizontal load whereas the working shear is 5.2 kips.
FRAME D \( P = P_w = 17.28 \text{kips}, \ H = 0 \)

Moments and Axial Loads are:

\[
\begin{align*}
\text{Beam:} & \quad 12WF27 \\
S_x &= 34.1 \\
\text{Column:} & \quad 8W48 \ (\text{oriented for Weak-axis Bending}) \\
S_y &= 15.0 \\
A &= 14.11 \\
x &= 2.08 \\
\frac{f_a}{f_y} &= \frac{212.0}{15.0} = 14.15 \text{ ksi} \\
\frac{f_a}{f_y} &= \frac{8.57}{17.64} = 0.485 \\
0.485 + \frac{0.85(14.15)}{1 - 8.57} = 1.163 \\
\frac{8.57}{22} + \frac{14.15}{22} = 1.035 \\
\frac{P}{f_b} &= 12.5 \text{ ksi} < 24 \text{ ksi} \\
\end{align*}
\]
Frame D: $P = P_0 = 17.28 \text{k}, \quad H = H_0 = 5.20 \text{k}$

Moments and Axial Loads are:

\[ f_a = \frac{125.01}{14.11} = 8.85 \]
\[ f_b = \frac{395.8}{15.0} = 26.40 \]
\[ \frac{f_a}{F_a} = \frac{8.85}{23.50} = 0.377 \]

\[ F_a = 17.64 \times \frac{4}{3} = 23.50 \text{ksi} \]
\[ F_e = 44.64 \times \frac{4}{3} = 59.60 \text{ksi} \]
\[ F_b = 22 \times \frac{4}{3} = 29.26 \text{ksi} \]

\[
\frac{0.377 + \frac{0.85 (26.40)}{1 - \frac{8.85}{59.60} (29.26)}}{23.50 + 29.26} = \frac{0.377 + 0.900}{1.277} = \frac{0.377 + 0.901}{1.278}
\]

Beam:

\[ f_b = 23.1 \text{ksi} < 32 \text{ksi} \]
FRAME E \( P = P_0 = 17.28 \, k \), \( h = 0 \)

Level 6 Moments and Axial loads are

Beam 12WF27
\[ S_X = 34.1 \]
\[ M_X = 459.0 \, k \cdot i \]

Column 8WF24
\[ S_X = 20.8 \]
\[ M_X = 1.61 \, k \cdot i \]

\[ f_a = \frac{103.69}{7.06} = 14.70 \, ksi \]
\[ f_b = \frac{232.3}{20.8} = 11.20 \, ksi \]

\[ \frac{f_a}{f_a} = \frac{14.70}{15.96} = 0.920 \]

\[ 0.920 + 0.85 \left( 11.20 \right) \left( 1 - \frac{14.70}{121.06} \right) \frac{22}{22} = 0.920 + 0.493 = 1.413 \]

\[ \frac{14.70}{22} + \frac{11.20}{22} = 1.180 \]

Beam:
\[ f_b = 13.45 \, ksi < 24 \, ksi \]
Level 7: Moments and Axial loads are

Beam: 12WF27
\[ S_x = 34.1 \]
\[ f_a = \frac{120.99}{8.94} = 13.5 \text{ ksi} \]
\[ f_b = \frac{293.4}{26.4} = 11.1 \text{ ksi} \]
\[ \frac{f_a}{F_a} = \frac{13.5}{16.23} = 0.830 \]

Column (below beam):
\[ S_x = 26.4 \]
\[ f_a = 13.5 \text{ ksi} \]
\[ f_b = 11.1 \text{ ksi} \]
\[ \frac{f_a}{F_a} = \frac{13.5}{16.23} = 0.830 \]

\[ 0.830 + \frac{0.85(11.1)}{1 - \frac{13.5}{121.73}} = 0.830 + 0.483 = 1.313 \]

\[ \frac{13.5}{22} + \frac{11.1}{22} = 1.120 \]

Beam:
\[ f_b = 14.1 \text{ ksi} < 24 \text{ ksi} \]
FRAME E  \( P = P_D = 17.28 \text{k} \), \( H = H_D = 5.20 \text{k} \)

Level 6 Moments and Axial Loads are:
\[
\begin{align*}
86.40 \text{k} & \\
398.9 \text{k} \uparrow & \\
807.0 & \\
408.1 \text{kip} \downarrow & \\
107.56 \text{k} &
\end{align*}
\]

\[
\begin{align*}
f_a &= \frac{107.56}{7.06} = 15.25 \text{ksi} \\
f_b &= \frac{408.1}{20.8} = 18.80 \text{ksi} \\
\frac{f_a}{F_a} &= \frac{15.25}{21.30} = 0.716 \\
\frac{f_b}{F_b} &= \frac{18.80}{29.26} = 0.634 \\
F_a &= 12.10 \times \frac{4}{3} = 16.13 \text{ksi} \\
F_b &= 22 \times \frac{4}{3} = 29.33 \text{ksi}
\end{align*}
\]

\[
\begin{align*}
0.716 + \frac{0.85(18.80)}{(1 - \frac{15.25}{16.13}) 29.26} &= 0.716 + 0.596 = 1.312 \\
\frac{15.25}{21.30} + \frac{18.80}{29.26} &= 0.716 + 0.634 = 1.350
\end{align*}
\]

\[\text{Beam:} \quad f_b = 23.6 \text{ksi} < 32 \text{ksi}\]
Frame E \( P = P_w = 17.28 \text{k}, H = H_w = 5.20 \text{k} \)

Level 7 Moments and Axial loads are:

\[
\begin{align*}
  f_a &= \frac{128.75}{8.94} = 14.40 \text{ksc} \\
  f_b &= \frac{468.9}{26.4} = 17.80 \text{ksc} \\
  \frac{f_a}{F_a} &= \frac{14.40}{21.65} = 0.664 \\
  F_a &= 16.23 \times \frac{4}{3} = 21.65 \text{ksc} \\
  F_b &= 12.73 \times \frac{4}{3} = 16.20 \text{ksc} \\
  F_c &= 22 \times \frac{4}{3} = 29.26 \text{ksc}
\end{align*}
\]

\[
0.664 + 0.85(17.80) = 0.664 + 0.567 = 1.231
\]

\[
\frac{14.4}{21.65} + \frac{17.80}{29.26} = 0.664 + 0.608 = 1.272
\]

Beam:

\[
f_b = 24.3 \text{ksc} < 32 \text{ksc}
\]