A92.90

AISC E&R Library
6163

WELDED
INTERIOR BEAM-TO-COLUMN
CONNECTIONS

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, Inc.

TA492
.W4G7

6163

A92.90

000002

Welded Interior Beam-to-Column Connections

A report on investigations carried out by

J. D. GRAHAM A. N. SHERBOURNE R. N. KHABBAZ

under the direction of

C. D. JENSEN

American Institute of Steel Construction Research at Lehigh University

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 3100, Chicago IL 60601-2001

Copyright 1959, New York, New York

Reprinted 1990 by American Institute of Steel Construction. All rights reserved. No part of this publication may be reproduced without written permission.

Published by the American Institute of Steel Construction, Inc. at One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001.

Table of Contents

Symbols

A_b	Beam Cross Sectional Area
b	Beam Flange Width
b_s	Stiffener Width
d	Beam Depth
d_c	Column Depth
d_w	Mean Distance between Beam Flanges
d_y	Deflection of Beam Loading Point at V_y
E	Young's Modulus
f	Resultant Stress on Fillet Weld in force per unit length
f_m	Component of Resultant Stress on Fillet Weld due to Moment
f_v	Component of Resultant Stress on Fillet Weld due to Shear Force
H	Hinge Angle
I	Beam Moment of Inertia
k	Column 'k' Distance
L	Beam Span
m	Distance between Fillet Extremities of One Flange of Column
M_p	Theoretical Maximum or 'Plastic' Moment of the Beam
M_w	Beam Working Moment
M_y	Moment to cause First Yielding of the Beam
P_w	Column Working Load
Q_b	Force on Connection due to Beam
Q_c	Resistance supplied by Column Web
Q_s	Resistance supplied by Stiffeners
Q_t	Resistance supplied by part of Connection adjacent to Beam Tension Flange
S	Elastic Section Modulus
t	Beam Flange Thickness
t_c	Column Flange Thickness
t_s	Stiffener Thickness
V_u	Load (also the shear) on the Test Beam to produce M_p
V_w	Load (also the shear) on the Test Beam to produce M_w
V_y	Load (also the shear) on the Test Beam to produce M_y
w	Column Web Thickness
w_b	Beam Web Thickness
Z	Plastic Section Modulus
$\varepsilon_{st.}$	Strain at Strain Hardening
γ	Poisson's Ratio
σ_a	Average Plate Stress at Test Ultimate in Tension Tests
σ_{PL}	Stress at Proportional Limit
$\sigma_{ult.}$	Ultimate Stress
$\sigma_{yL.}$	Lower Yield Stress
$\sigma_{y\,st}$	Static Yield Stress
σ_{yU}	Upper Yield Stress
ϕ	Rotation per unit length
ϕ_p	Beam Rotation per unit length at M_p

Synopsis

Previous research on beam-to-column connections has not been carried to the point where definite conclusions, suitable for the designer, can be reached. In particular, information is lacking on the criteria for the need of column stiffening and on the criteria for designing it when it is needed. Information is also lacking concerning the moment-rotation capacity of a connection and concerning the effect on connections of beams framing into the column web as occurs in four-way connections.

A satisfactory connection is defined as one which is capable of

a. developing the theoretical maximum or "plastic" moment of the beam when working axial load is on the column and

b. permitting sufficient rotation at this moment to allow the second plastic moment to form at the mid-span of the beam.

This report is a summary of experimental and analytical investigations into the behavior of connections both with and without stiffeners. The first stage of this work comprised an investigation into two-way beam-to-column connections, first by detailed tests copying practical conditions and later by simpler tests simulating these conditions. The second stage comprised an investigation into four-way beam-to-column connections, again by detailed tests copying practical conditions. The design rules stemming from these investigations apply to those connections in which —

1. Beams and columns are members of the wide flange series listed in the AISC Manual.
2. Beams are connected to both column flanges and may or may not be connected to both sides of the column web such that approximately equal moments are applied on opposite sides of the column.
3. Connecting welds are so designed and executed that they are as strong as, or stronger than, the parts connected.

The design rules finally arrived at, for the connections of fully-loaded beams to column flanges, are:

I. Column stiffeners are *not* required

(A) adjacent to the beam compression flanges if

$$w \geqslant \frac{bt}{t + 5k}$$

(B) adjacent to the beam tension flanges if

$$t_c \geqslant 0.4\sqrt{bt}$$

II. Column stiffeners are required if the formulas in (A) and (B) are not satisfied, and their minimum thicknesses are given by

(C) in the case of horizontal plate stiffeners

$$t_s = \frac{1}{b}\left[bt - w\,(t + 5k)\right]$$

and, as a further limitation,

$$t_s \geqslant \frac{b_s}{16}$$

(D) in the case of horizontal plate stiffeners eccentric by 2″ or less,

$$t_s = \frac{1.7}{b}\left[bt - w\,(t + 5k)\right]$$

where, again,

$$t_s \geqslant \frac{b_s}{16}$$

(E) in the case of vertical plate stiffeners parallel to the column web and located at the toes of the column flanges

$$t_s = \frac{bt}{t + 5k} - w$$

and, as a further limitation,

$$t_s \geqslant \frac{d_c}{30}$$

The limitations of this investigation, the analysis leading to the above formulas and design examples are given in Part C.

Outline of Investigation

In this investigation, studies are made of two-way and four-way interior beam-to-column connections. Attempts are first made to copy the most severe conditions found in practice, while in later tests those items having a negligible effect on the connection performance are eliminated. Beam and column sizes used are typical of those in a building frame.

The primary purpose is the study of the connection under the following items:

a. *Stiffening requirements.* When are stiffeners needed? What are the factors involved in the behavior of the connection with and without stiffeners? These assume significance in the application of "plastic analysis" to the design of tier buildings. To assure the formation of plastic hinges in the beams, the connection and the column should be capable of sustaining a plastic moment in excess of, or at least equal to, the plastic moment value of the beams.

b. *Rotation capacity.* This is another important feature in the "plastic" analysis of structures since it expresses the ability of the connection to sustain a full plastic moment through the required hinge angle.

The beams were welded directly to the columns for three reasons:

1. The direct-welded connection has certain advantages and may eventually come into more general use.
2. The emphasis in this investigation being upon the study of the stresses and strains in the column at the intersection, the elimination of top plates and seat angles removed a few unnecessary variables.
3. The direct-welded connection, without seat angles, represents the severest loading on the column at the connection.

However, the formulas developed by this investigation may be used for determining the need for, and the design of stiffeners when the beam flanges are connected to the columns by butt-welded plates. In this case the width and thickness of the connection plate is used in design.

Part A—Test Program

1. Two-way Connection Tests

This program consisted of the design, preparation and testing of specimens as shown in Table 1 and Figures 1, 2, 3 and 4 for the purpose of determining the behavior and stress distributions in the connection and its component members. Attention was limited primarily to the study of what was considered to be the most important practical problem, viz., whether column stiffeners are required and if so how to design them, although other aspects of the problem merited consideration. Beam and column sizes were chosen to duplicate conditions existing in a tier building. Three basic column sizes were chosen. The first used was an 8 WF 31 column which was loaded to simulate conditions existing at the top of a building frame where axial loads are small compared to beam loads. The second group utilized 8 WF 67, 12 WF 40, and 12 WF 65 columns on the basis of beam and column loads being of the same order of magnitude. The third

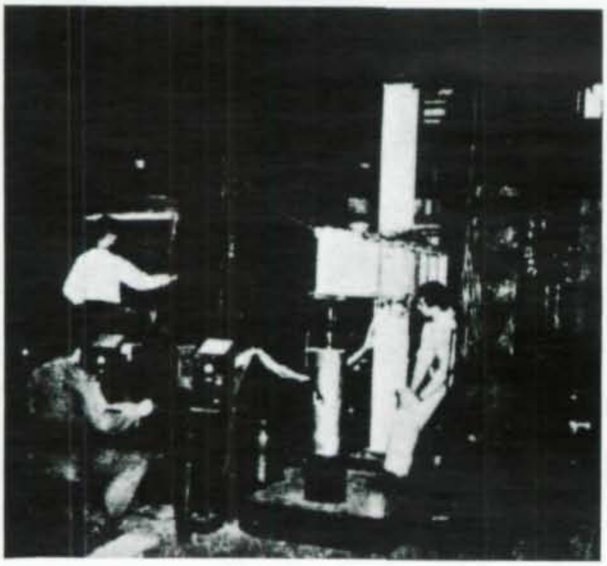


FIGURE 1

General View of Two-Way Test in Progress

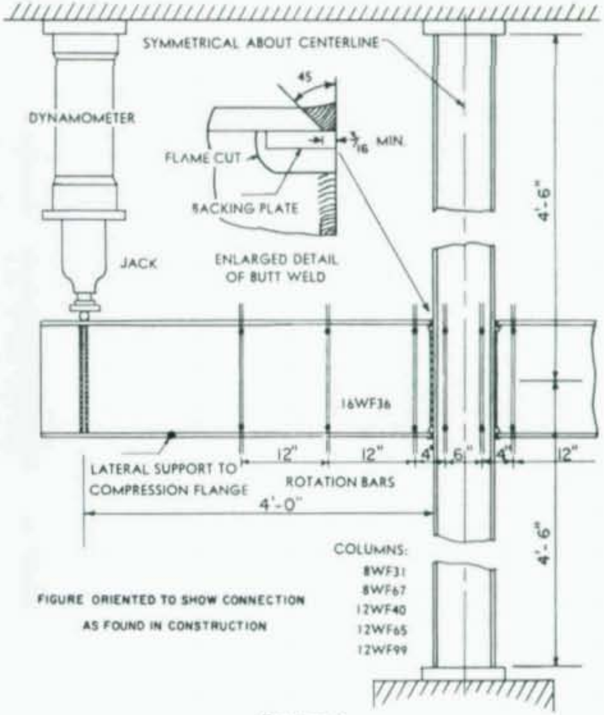


FIGURE 2

Test Arrangement—Two-Way Connections

TABLE 1

PROGRAM OF TWO-WAY DIRECT-WELDED BEAM-TO-COLUMN TESTS

Test No.	Column Shape	Column Web*	Column Flange*	Beam Shape	Beam Web*	Beam Flange*	Stiffener Type	Stiffener Dimension
A-1	8 WF 31	0.288	0.433	16 WF 36	0.299	0.428	None	None
A-2	8 WF 67	0.575	0.933	"	"	"	"	"
A-4	12 WF 65	0.390	0.606	"	"	"	"	"
A-5	12 WF 99	0.580	0.921	"	"	"	"	"
B-6	8 WF 31	0.288	0.433	"	"	"	‡	3.9″ x $\frac{7}{16}$″
B-8	12 WF 40	0.294	0.516	"	"	"	‡	3.9″ x $\frac{1}{4}$″
C-9	8 WF 31	0.288	0.433	"	"	"	‡‡	$\frac{5}{16}$″ x 22″
C-11	12 WF 40	0.294	0.516	"	"	"	‡‡	$\frac{5}{16}$″ x 22″
D-12	12 WF 40	0.294	0.516	"	"	"	Tee stiffener	ST6 WF 32.5 x 22″
H-1	8 WF 31	0.288	0.433	"	"	"	Doubler plate	$\frac{5}{16}$″ x 20″

* Indicates AISC Manual value.

‡ Horizontal plate stiffeners, at level of tension and compression flanges.

‡‡ Vertical plate stiffeners at toes of column flanges.

size was a 12 WF 99 column used under conditions representing the lower tiers of a frame where axial loads are high in comparison with beam loads. One size of beam was selected throughout this program to eliminate beam size as a variable, and because it is likely that floor loadings will be constant through successive stories of a building. The size selected (16 WF 36) has dimensions that ensure the development of the plastic bending strength, M_p, without local buckling of either the flange or the web.

The test program was divided into five groups of tests depending upon the type of stiffening employed. (See Figures 3 and 4). The specimens consisted of two 16 WF 36 beam stubs, 4′-6″ long, welded directly to the flanges of the WF column sections as shown in Figure 2.

The point of load application on the beams was at a distance of 4′-0″ from the face of the column flange. Axial load was applied to the specimen by an 800 kip screw type universal testing machine. The specimen was inverted in the machine to permit the beam loads to be applied by mechanical compression jacks which were mounted on dynamometers. The dynamometers, in turn, were set on bearing blocks seated on the table of the machine (See Figures 1 and 2).

All welding was done by qualified welders using $\frac{3}{16}$″ diameter E6020 electrodes, except that an E6012 electrode was used for the first weld pass. There was much instrumentation on the specimens, measurements being taken during the test of strain distribution, deflections, rotations and tendencies towards both local and lateral

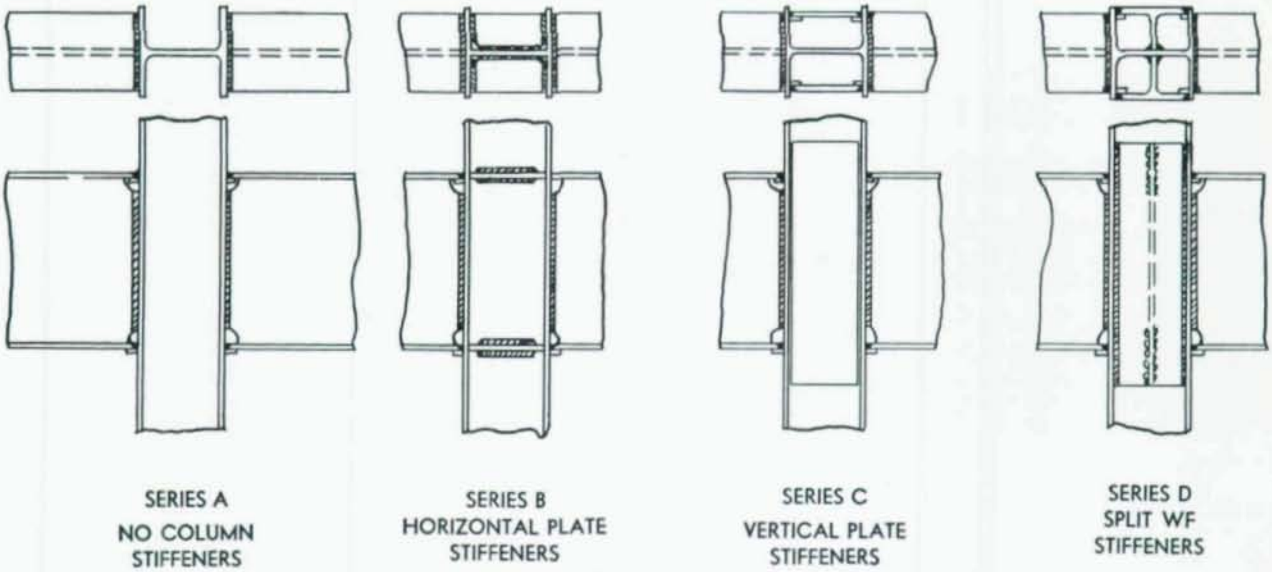


FIGURE 3

The Test Series of Two-Way Beam-to-Column Connections

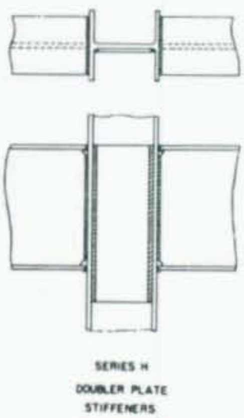


FIGURE 4

The Test Series (Continued)

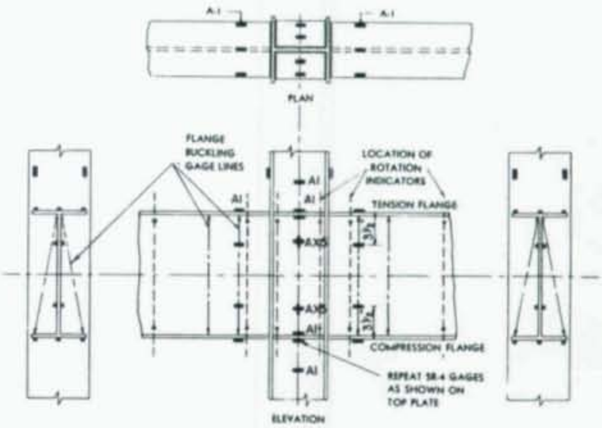


FIGURE 5

Instrumentation Plan—Series B

buckling of the beam. Figure 5 shows the instrumentation in Series B, there being few differences in the other series.

Before proceeding with a test, the column was checked for axial alignment by observing the strains in four electrical strain gages located at the same level in the column and mounted at the outer edges of each column flange. The maximum variation permitted in the gage reading was about 10% at full column working load.

The sequence of loading in the tests was arranged in five stages as follows:

1. The column load was increased in five equal increments to working load, P_w, with no load on the beams. (This axial load was the same for the full height of the column).
2. The beam load was increased in four equal increments to working load, V_w, while maintaining working load, P_w, at all times in the portion of the column "below"* the beams. At the conclusion of this stage the "upper" portion of the column sustained a load equal to $P_w - 2V_w$ where

   $P_w =$ the column working load (refer to Section 2.2 of Appendix) and

   $V_w =$ the applied beam working load.
3. With this working load, V_w, maintained on the beams, the column was then subjected to a first overload which increased the load in the "lower" portion to 1.65 times the working load and which increased the load in the "upper" portion correspondingly. This was done in three equal increments. The column load was subsequently reduced to working load in the "lower" portion. This left the specimen under the same loading that existed at the end of stage 2.
4. With working load, P_w, maintained in the "lower" section of the column the beams were loaded in increments until failure occurred.
5. As a last step in the testing, with the connections damaged and with the last beam load still in the jacks, the column was subjected to a second overload equal to twice the working axial load.

The test program was divided into five groups of tests (namely A, B, C, D and H) depending upon the type of stiffening employed (See Figures 3 and 4). Specimen dimensions are given in Table 1.

Series A

In this group no stiffening was provided and the tests ranged from the very light, thin-web 8 WF 31 column to the heavier 12 WF 99. Connection A-1 with the 8 WF 31 column failed by column web buckling at a load slightly above the beam working load, namely 1.12 V_w. Connection A-4, with a thicker web showed much straining, both tension and compression, in the column webs opposite the beam flanges and failure occurred by column web buckling at a beam load of 44 kips, which is 1.82 V_w. In both cases the decrease in moment carrying capacity was quite rapid but no local buckling of the beam flanges was experienced. The column flanges in Test A-4 deformed considerably on the second column overload.

Specimens A-2 and A-5 behaved extremely well without stiffening. Local buckling of the beam flanges occurred at 2.08 V_w and 2.26 V_w respectively. The loss of beam strength was quite gradual and the specimens sustained large rotations before the tests were concluded. Upon application of the second column overload additional deformation of the column flanges was noted, but no other effect on the column was observed that would indicate that column failure was imminent.

Series B

Horizontal stiffeners were placed across the column flanges at the level of the beam flanges in this series as shown in Figure 3. These stiffeners were welded to both column flanges and to the column web. In test B-6 the stiffeners were of a thickness equal to the beam flanges but in B-8 the stiffeners were thinner. This is a very strong type of connection as borne out by the test results, as both exhibited excellent load and rotation capacities. Both specimens suffered local buckling of the beam compression flanges at the onset of the strain hardening range and the increase in beam load above this level was slight. The decline of strength from the maximum value was gradual as jacking continued and no harmful effects were observed in the column stiffeners beyond the presence of a few strain lines. The principal deformations occurred in the beams.

Series C

The stiffening provided in this series of tests consisted of plates positioned vertically near the toes of the column flange as shown in Figure 3. The stiffeners were arbitrarily made the same thickness as the column web. Both connections C-9 and C-11 carried the required loads. In both tests there was evidence of some slight local buckling on the beam compression flanges at loads of 2.17 V_w. In both tests, the column web between the beam compression flanges buckled. For specimen C-11 the critical load at which this effect was first noticed was 1.97 V_w. In C-11 weld failure occurred just after this in the tension flange butt welds. In test C-9 the connection continued to carry load until at 2.16 V_w the south stiffener plate buckled. From this point the load fell off rapidly.

* "Below" or "lower" and "upper" refer to the portions of a column below and above the beam as used in actual construction, not as in the laboratory.

Series D

Only one test, D-12, was performed in this group, the connection being a modification of the C type using split beam tee stiffeners instead of plates as shown in Figure 3. The tee stiffener, while devised principally for use in a four-way beam-to-column connection, actually served to eliminate buckling of both the stiffeners and the column web. The connection was found to be extremely stiff, the primary cause of failure being the local buckling of the beam compression flanges which became large at loads in excess of 2.22 V_w. Although large deformations occurred in the beams, the connection appeared to remain elastic and little strain was observed in the flange of the tee stiffener. A marked difference was noted in the behavior of the two beams of the specimen and weld tears were observed in the beam tension flanges at loads greater than those required to cause beam buckling.

Series H

Only one test, H-1, was performed in this group. Since test A-1 was stronger in the tension region of the connection, this test investigated the effect of strengthening the column web by the addition of a $\frac{5}{16}$" doubler plate welded flush with the column web. Failure in H-1 occurred by the tension weld tearing at mid-length of the butt weld between the east beam and the column. The failure occurred at a beam load of 49.6 kips which is 2.05 V_w, just below the load corresponding to beam plastic moment. The rotation was adequate but the load fell off rapidly after the tearing of the weld.

A comparison of test beam deflections is presented in Figure 6. Views of four specimens at the completion of testing are shown in Figure 7.

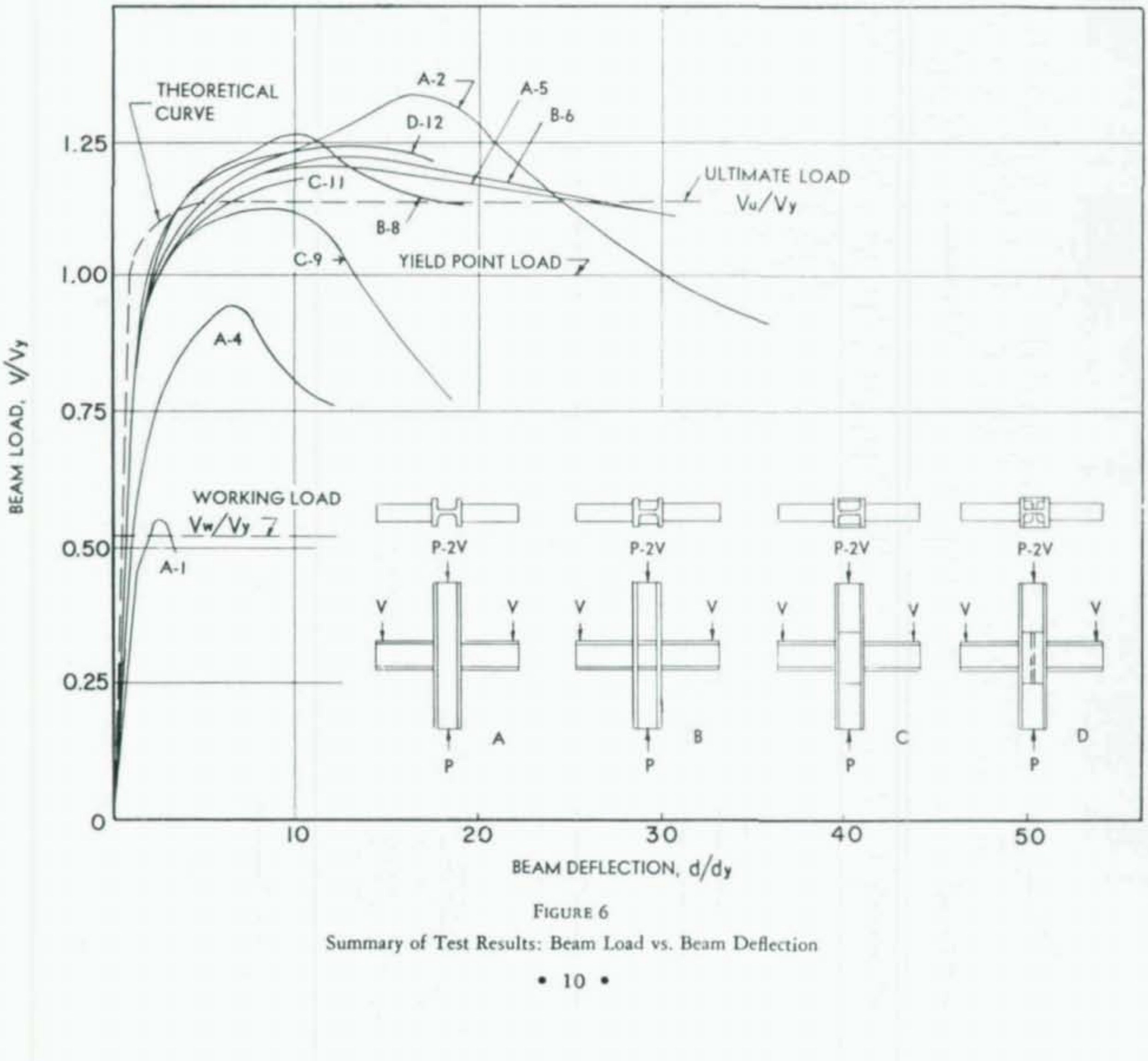


FIGURE 6

Summary of Test Results: Beam Load vs. Beam Deflection

The results in Figures 8 and 10 show that the columns of the A and H series, with no column flange stiffening, are not as stiff against rotation as are the 16 WF 36 beams which framed to the columns. In the B tests (See Figure 9) the stiffeners provide the equivalent of beam flanges to the columns, and the columns become as stiff against rotation as are the framing-in beams. The same applies to the C tests as shown in Figure 9. From an inspection of the strain readings taken on the C specimens it is noted that the column web carried a major part of the applied load, approximately 2½ to 3 times as much as the plate stiffeners at beam working load.

The A series of tests showed high stress concentrations at the center of the beam tension flanges as indicated in

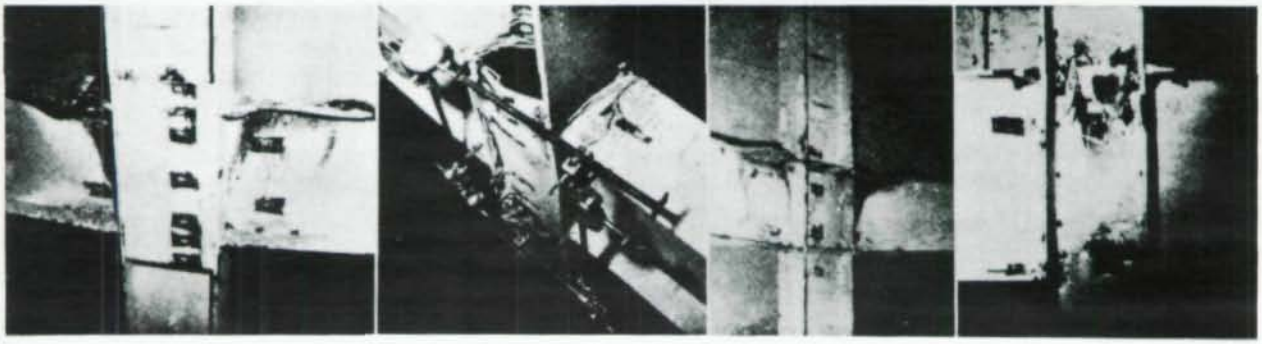


FIGURE 7

Tests A-1, B-8, C-9, and D-12 (left to right)

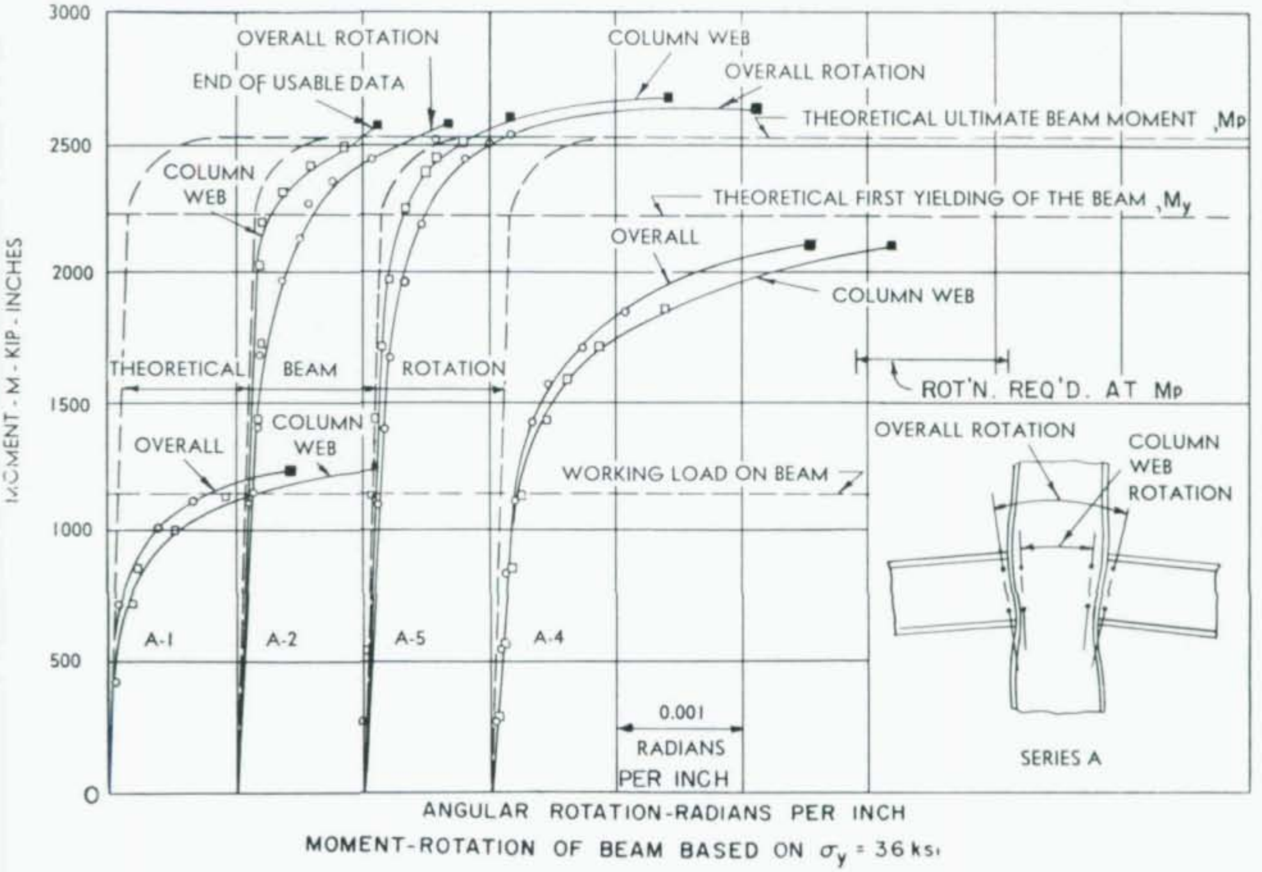


FIGURE 8

Comparison of Column Web and Overall Connection Rotation with that for the Beam: Series A

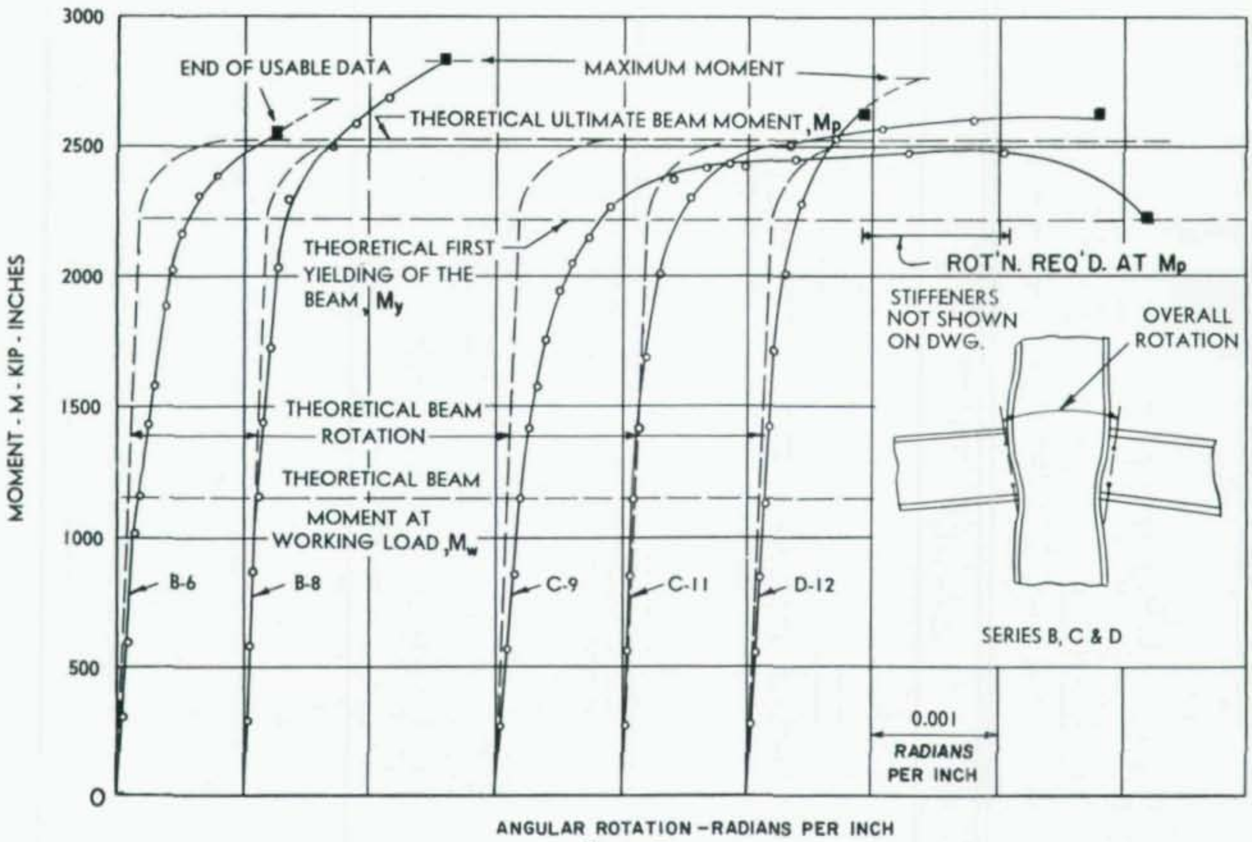


FIGURE 9

Comparison of Column Web and Overall Connection Rotation with that for the Beam: Series B, C, and D

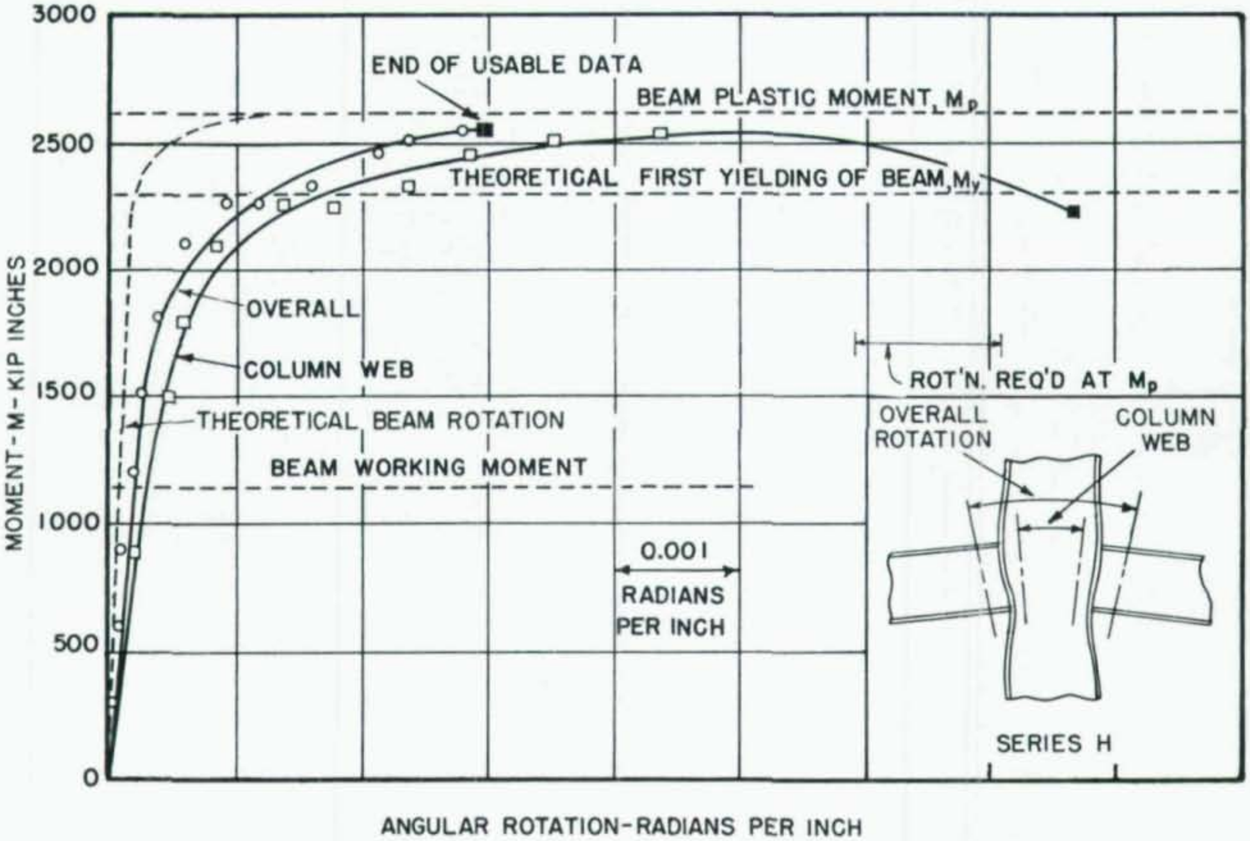


FIGURE 10

Comparison of Column Web and Overall Connection Rotation with that for the Beam: Series H

Figure 11. The stress distribution on the compression flanges in the B series was uniform on the whole, while in the tension areas the stresses were somewhat higher in the center. For the C series the distribution of stress was uniform in both flanges at V_w, while at 1.5 V_w high tensile stresses occurred at mid flange. Specimen D-12 also showed a generally uniform distribution throughout. Both C-11 and D-12, however, appeared to suffer from eccentric effects as indicated by the higher stresses on one side of the flange, and this probably caused the weld tearing. Specimen H-1 showed a stress concentration in the center of the beam tension flange, the concentration being very pronounced at 1.5 V_w. Measurements of horizontal strain in the column web were taken during the tests. Figure 12 shows a plot of these strains in specimens A-1 and A-2 at a beam load of 1.5 V_w.

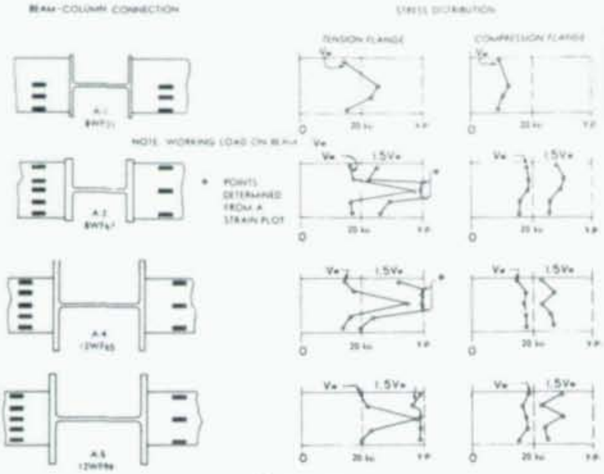


FIGURE 11

Stress Distribution in Beam Flanges: Series A

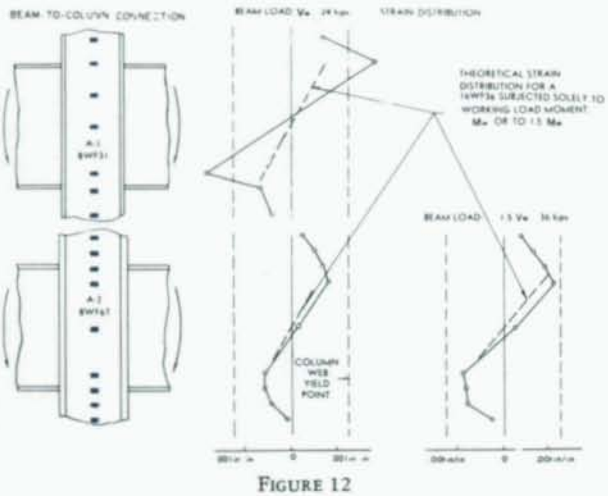


FIGURE 12

Distribution of Horizontal Strain in Column Web: Tests A-1 and A-2

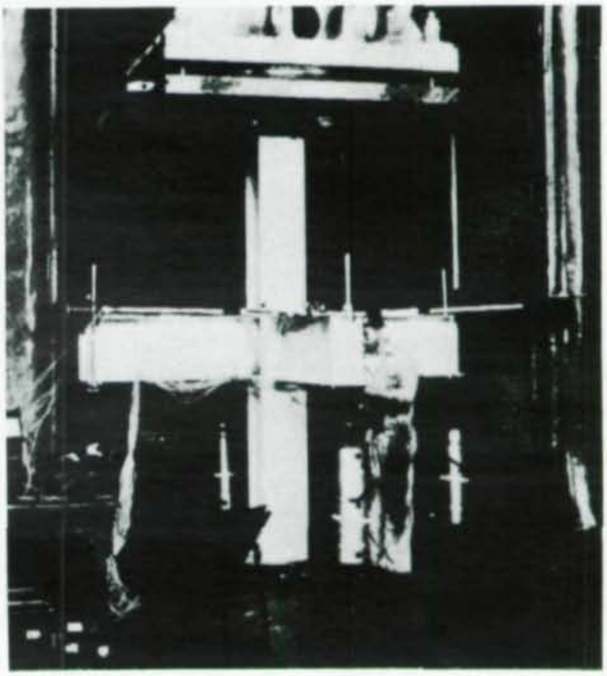


FIGURE 13

General View of Four-Way Test in Progress

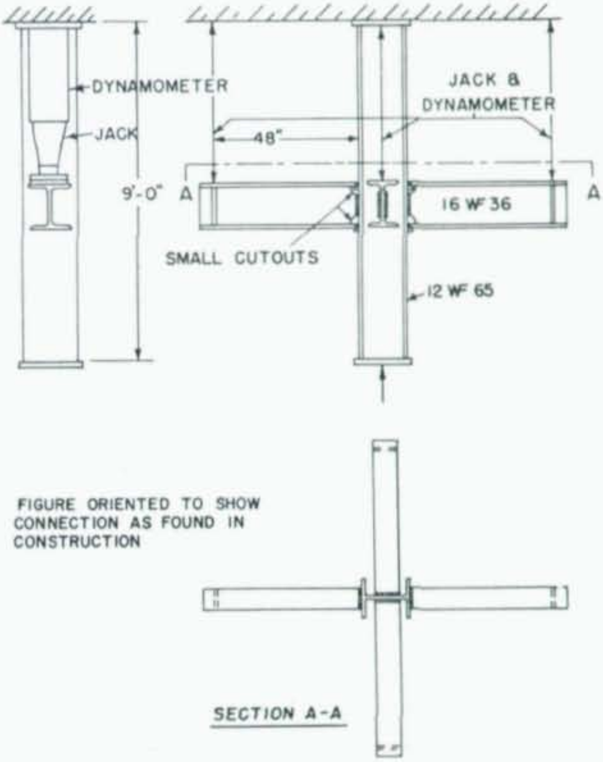


FIGURE 14

Test Arrangement—Four-Way Connections

2. Four-way Connection Tests

This program consisted of three specimens with details as shown in Table 2 and Figures 13, 14 and 15. Test AA is similar to Test A-4 of the "Two-way" series except for two additional 16 WF 36 beams framing into the column web and directly welded thereto. In the same manner Test DD is similar to Test D-12 of the Two-way series. Test BB was exploratory in nature and does not have its two-way counterpart. The beams framing to the column flanges were 16 WF 36 as before and were direct-welded. The other pair of beams were 12 WF 27, the tension flanges of which were welded to horizontally placed column plate stiffeners. Their compression flanges rested on tee-type seats which acted as column stiffeners. However, these seat plates were 4" away from the ideal stiffener locations.

The specimens were fabricated of the WF sections indicated in Table 2, the beams being each 4'-3" long and the columns 9'-0" long.

The testing was done in a five million pound universal testing machine which provided ample space for placing these specimens and for the lateral supports. The test arrangement was similar to that for the two-way tests. Figure 14 shows the test arrangement and is oriented to show the positioning of loads as found in a typical building connection. The measurements taken were much the same as in the two-way tests, Figure 16 showing the instrumentation plan in Test AA.

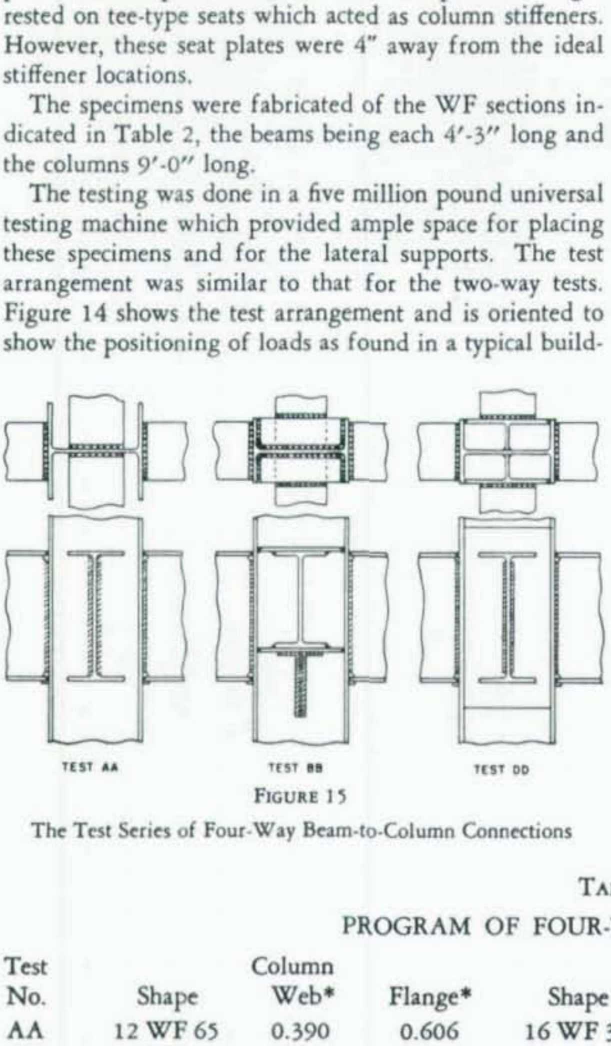


FIGURE 15

The Test Series of Four-Way Beam-to-Column Connections

Test AA

For the beam-to-column flange connection in Test AA that portion of the column web which was stiffened by the flanges of the other pair of beams showed little rotation compared with the part of the connection consisting of 3" of the beam, the column flange and about 1" of the unstiffened column web. As expected, the beams directly welded to the column web and subjected to equal opposing moments provided a stiff connection. With only partial stiffening provided, the connection of the beams to the column flange showed considerable flexibility (See Figure 17). Local buckling of the beam flanges was observed at a load of 53 kips (2.28 V_w) in the beams framing to the column flanges, and at a slightly higher load in the beams framing to the column web. The falling off of the beam loads was rather slow. When the beam loads had fallen off by 15% of V_u, twice working load was applied to the column, the whitewash indicated that the column suffered considerable yielding, but there was no other evidence of failure in the column. Figure 18 shows specimen AA at the end of the test.

Test BB

The connection involving the 16 WF 36 beams, welded directly to the column flanges, proved to be relatively stiff. The connection involving the 12 WF 27 beams framing to the seats and top plates was considerably more flexible than an equivalent 12 WF 27; however this flexibility did not prevent the connection from fully meeting the established criteria for a satisfactory connection.

Test DD

The connection involving the beams welded directly to the column flanges proved stiffer than the connection of the beams to the tee stiffeners (See Figure 17). The stiffness of the latter connection is mainly dependent on the thickness of the stem of the tee stiffener, the flanges of the column being too far away to offer much resistance.

TABLE 2

PROGRAM OF FOUR-WAY CONNECTION TESTS

Test	Column			Beam			Stiffener	
No.	Shape	Web*	Flange*	Shape	Web*	Flange*	Type	Dimension
AA	12 WF 65	0.390	0.606	16 WF 36	0.299	0.428	None	None
BB	12 WF 40	0.294	0.516	16 WF 36	0.299	0.428	‡	½" thick
				12 WF 27	0.240	0.400	‡	
DD	12 WF 40	0.294	0.516	16 WF 36	0.299	0.428	Tee stiffener	ST6 WF 32.5 x 22"

* Indicates AISC Manual value.

‡ Horizontal plates that served as top plate and as seat (plate).

On the other hand, the column web is ably assisted in preventing rotation at the connection by the flanges of the split beam tee stiffeners. The two beams that were connected to the stiffeners had very good load and rotation capacities. The east and west beams connected to the column flanges just reached the required ultimate load and showed a smaller rotation capacity caused by a butt weld failure starting at a load of 49 kips (2.18 V_w). The first crack occurred in the west beam at the interface between the column flanges and the end of the butt weld to the beam tension flange, and increased until weld failure penetrated to the fillet welds connecting the beam web to the column flange. The tension flange butt welds of the north and south beams, connected to the stiffeners, had very small cracks starting at a load of 55 kips, but they did not progress any further since, at this load, the beam compression flanges buckled. Figure 19 shows specimen DD at the end of the test.

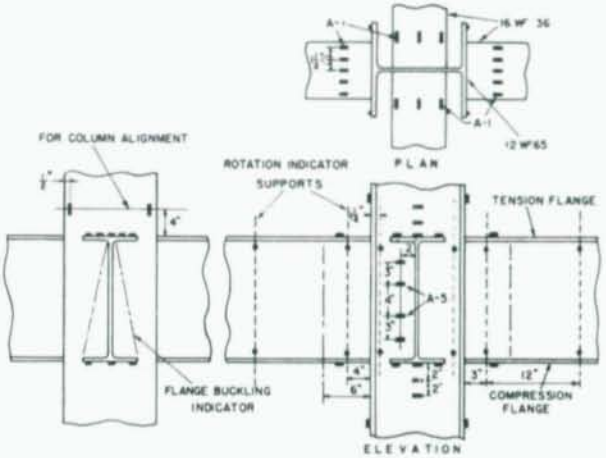


FIGURE 16

Instrumentation Plan—Test AA

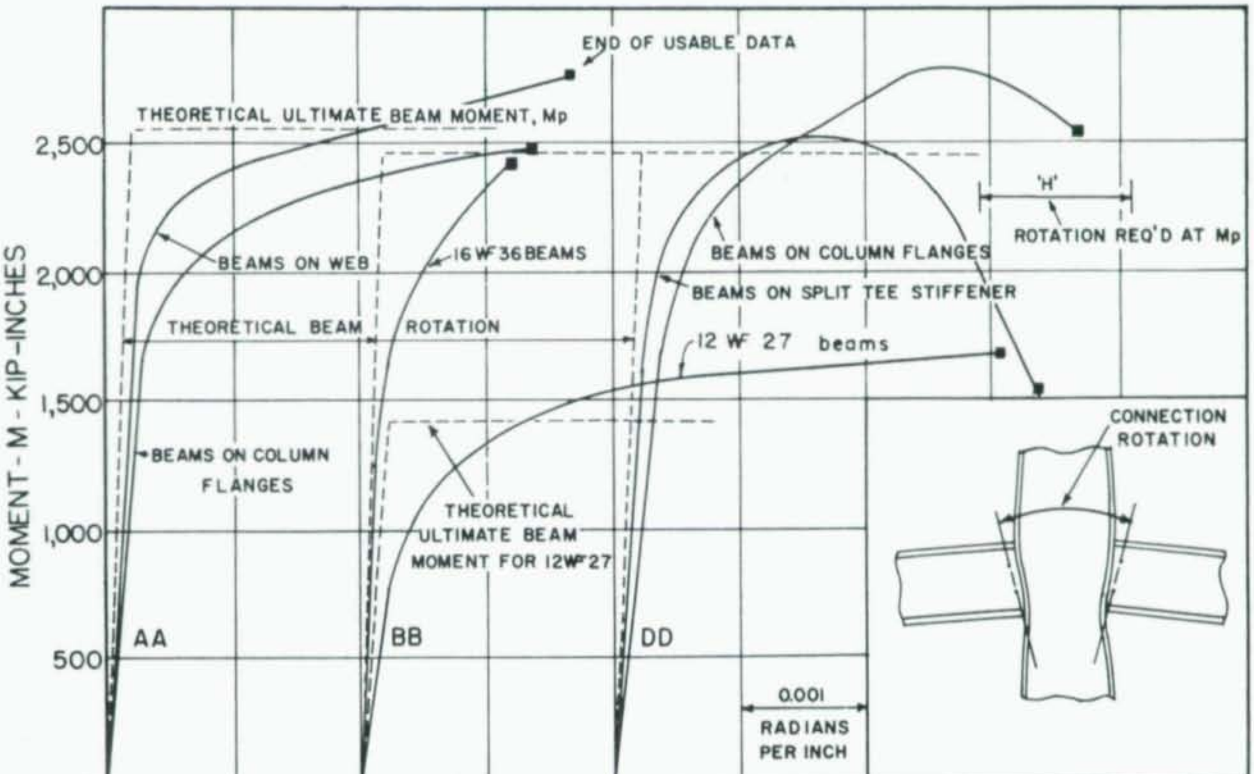


FIGURE 17

Comparison of Overall Connection Rotation with that for the Beam: AA, BB, DD

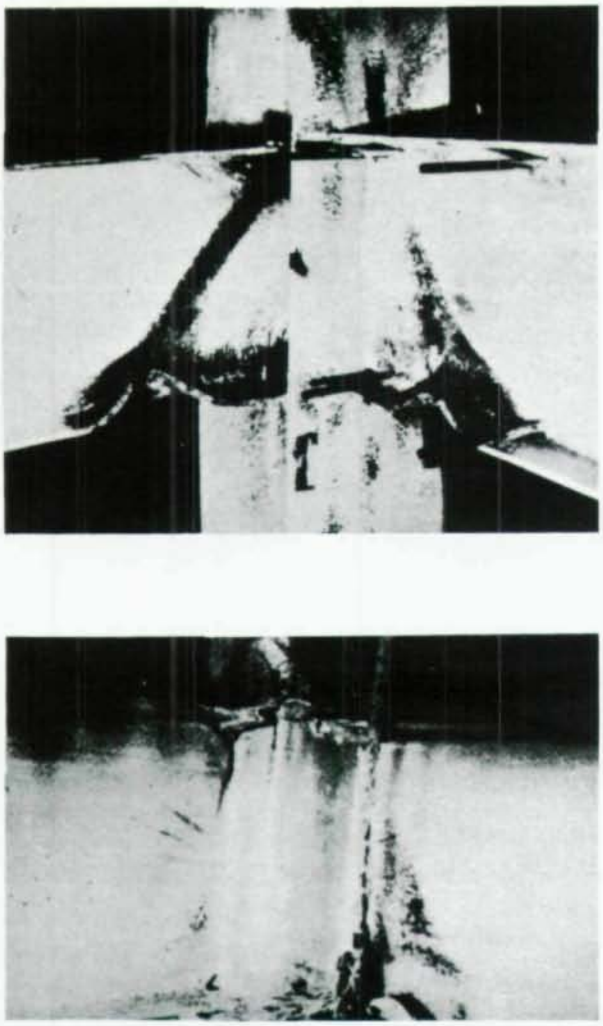


FIGURE 19

Test DD—Failure Details

3. Simulated Connection Tests

After examining the results of the two-way tests it was realized that practically the same stress and strain state in a connection could be produced by far simpler and quicker tests. These tests were of three types described as follows:

3.1 Tests to Determine Column Web Buckling Criterion

These tests simulated the lower part of the connection in which the beam flange was in compression against the column and consisted of a piece of column compressed at the flanges between two bars, the size of the bars being made the same as the section of the flange of the simulated beam.

The size of the bars was kept constant at 7″ x ½″, simulating the flange of the 16 WF 36 beam used in all the two-way tests. The bars were tack welded to the flanges at the mid-length of the columns, which were approximately 3′-0″ long. The specimen was then tested in a 300 kip universal testing machine with the simulated column in a horizontal position (See Figure 20).

Eleven tests were carried out, the details of which are given in Table 3.

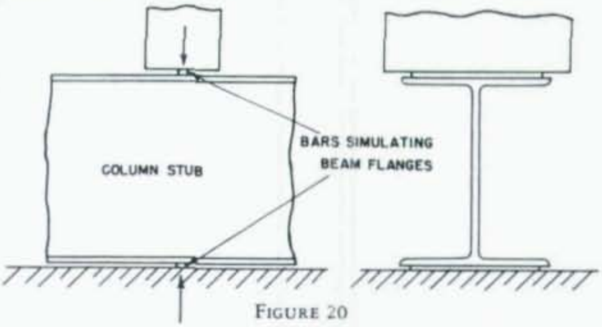


FIGURE 20

The E Series. Tests to Determine Compression Region Criterion

TABLE 3

PROGRAM OF COMPRESSION CRITERION TESTS

Test No.	Column Shape	Web*	Flange*	Bar Width	Bar Thickness	Simulated Beam	Failure Load (kips)
E-14	8 WF 48	0.405	0.683	7″	½″	16 WF 36	137
E-15	8 WF 58	0.510	0.808	"	"	"	202.5
E-16	10 WF 66	0.457	0.748	"	"	"	175.7
E-17	10 WF 72	0.510	0.808	"	"	"	190
E-1	12 WF 40	0.294	0.516	"	"	"	102.5
E-18	12 WF 65	0.390	0.606	"	"	"	143
E-19	12 WF 85	0.495	0.796	"	"	"	247.5
E-20	14 WF 61	0.378	0.643	"	"	"	137.5
E-21	14 WF 68	0.418	0.718	"	"	"	164
E-22	14 WF 84	0.451	0.778	"	"	"	221
E-23	14 WF 103	0.495	0.813	"	"	"	250

* Indicates AISC Manual value.

In all these tests yielding began first in the column fillet immediately beneath the bar. Yielding was seen to progress into the web by means of lines radiating from this point and other semicircular lines orthogonal to these.

The yielding continued some distance into the web until the column web failed by buckling. At a load within 20% of the failure load, a slight bending of the column flanges was noticed. Table 3 presents the maximum loads obtained in the tests. Figure 21 shows E-1 at failure.

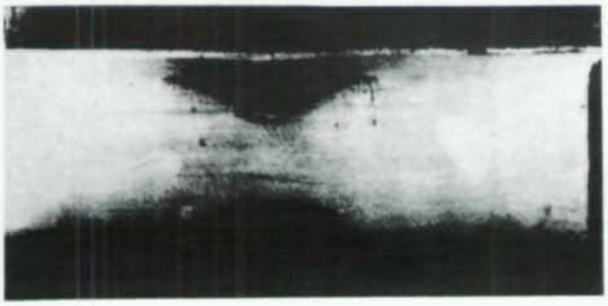


FIGURE 21

Test E-1

3.2 Tests to Determine Connection Tension Criterion

These tests simulated the upper part of the connection in which the beam flange is in tension, and consisted of two equal plates welded to the flanges of the column, the size of the plates being made the same as the section of the flange of the simulated beam. Tension was applied to these plates by means of an 800 kip universal testing machine. The dimensions of both the plate and the column flange were varied to study their respective influences. The effect of changing the column flange thickness was further studied by repeating certain of the tests with the column flanges machined to about half the original thickness. The plates simulating the beam flanges were also changed in size, keeping the column section constant. Table 4 summarizes these tests. The plates were butt welded to the centers of a column of length about 3'-0", as shown in Figure 22, and the specimen then lined up in the testing machine with the column horizontal.

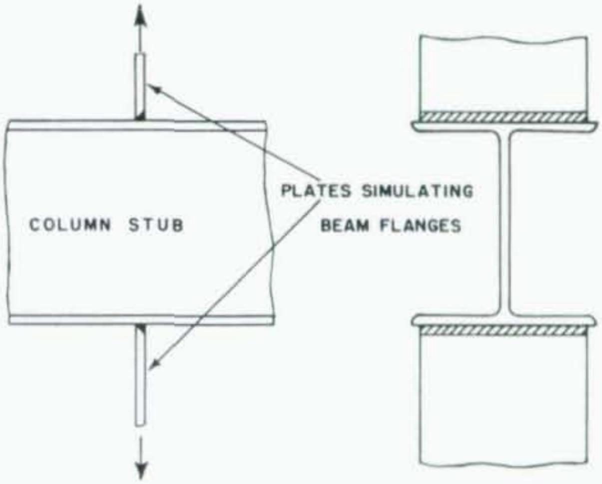


FIGURE 22

The F Series. Tests to Determine Tension Region Criterion

TABLE 4

PROGRAM OF TENSION CRITERION TESTS

Test No.	Column Shape	Web*	Flange*	Plate Width	Plate Thickness	Failure load (kips)	Method of failure
F-1	8 WF 31	0.288	0.433	7"	$\frac{3}{4}''$	100	Crack in column fillet
F-2	8 WF 31	0.288	0.433	7"	$\frac{7}{16}''$	95	Crack in center of weld
F-3	12 WF 65	0.390	0.606	$8\frac{1}{2}''$	$\frac{5}{8}''$	149	"
F-4	14 WF 68	0.418	0.718	$8\frac{1}{2}''$	$\frac{5}{8}''$	167	"
F-5	14 WF 84	0.451	0.778	$11\frac{1}{2}''$	$\frac{7}{8}''$	212	"
F-9	12 WF 65‡	0.390	0.606‡	$8\frac{1}{2}''$	$\frac{5}{8}''$	82	Crack in column fillet
F-10	14 WF 84‡‡	0.451	0.778‡‡	$11\frac{1}{2}''$	$\frac{7}{8}''$	125	Crack in center of weld
F-12	12 WF 65	0.390	0.606	$8\frac{1}{2}''$	$1\frac{1}{2}''$	189	"
F-13	14 WF 68	0.418	0.718	$8\frac{1}{2}''$	$1\frac{1}{2}''$	199	"
F-14	8 WF 67	0.575	0.933	7"	$\frac{3}{4}''$	256	Crack at outside of weld
F-15	14 WF 176	0.820	1.313	$11\frac{1}{2}''$	$\frac{7}{8}''$	444	"

* Indicates AISC Manual value.

‡ Column flange machined to $\frac{5}{16}''$ for test F-9.

‡‡ Column flange machined to $\frac{3}{8}''$ for test F-10.

The first yield lines were noted in the column fillet immediately beneath the plate at a load of about 40% of the ultimate load. The yielding proceeded

a. into the column web

b. underneath the column flange parallel to the plate and

c. on the column flange starting from the center of the weld in lines parallel to the column web.

By the time failure occurred, yielding had progressed 2" into the web in tests F-1, F-2, F-3, F-4, F-5, F-9 and F-10 and had progressed across the web in tests F-12, F-13, F-14 and F-15. All specimens except F-1, F-9, F-14 and F-15 failed by the occurrence of a crack in the center of the butt weld, the fracture taking place after noticeable flange bending. F-1 and F-9 cracked in the column fillet while F-14 and F-15 suffered a tearing out which started from the outside of the column flange and proceeded to its center. The tear pulled out part of the column flange material. Table 4 presents the maximum loads obtained in the tests. Figure 23 shows F-5 and F-15 at failure.

FIGURE 23
Tests F-5 and F-15

3.3 Eccentric Stiffener Tests

In four-way connections the columns may be stiffened, opposite the compression flanges of the flange-connected beams, by the support provided by the compression flanges or the seating plates of the beams which frame into the column web. In a connection such as specimen BB (Figure 15), where the flange-connected and web-connected beams are of different depths, their compression flanges are not opposite, and the degree of such

TABLE 5

PROGRAM OF COMPRESSION TESTS WITH ECCENTRIC STIFFENERS

Test	Column	Stiffener	Eccentricity (in)	Failure Load (kip)
E-0	12 WF 40	3¾" x ¼" x 10¾"	0	172
E-2	"	"	2	146
E-3	"	"	4	113
E-3a	"	3¾" x ¼" x 10¾" + 3" x ¼" x 8" Tee	4	116
E-4	"	3¾" x ¼" x 10¾"	6	104
E-1	"	none	*	102.5
E-9	14 WF 61	4¼" x ⅜" x 12½"	0	282
E-6	"	"	2	232.5
E-7	"	"	4	167.6
E-8	"	"	6	142.8
E-24	"	none	*	137.5

* No stiffening used.

stiffening is questionable. To determine the degree of such stiffening a series of tests were carried out on pieces of 12 WF 40 and 14 WF 61 columns approximately 4'-0" long. The columns were compressed between bars for cases of 0, 2", 4" and 6" eccentricity as shown in Figure 24 by means of a 300k universal testing machine, the tests being similar to the compression criterion tests in Part 3.1. Included in the tests on the 12 WF 40 was one (E-3a) in which the compression region of test BB was simulated—that is, a tee seat was added to a stiffener of 4" eccentricity.

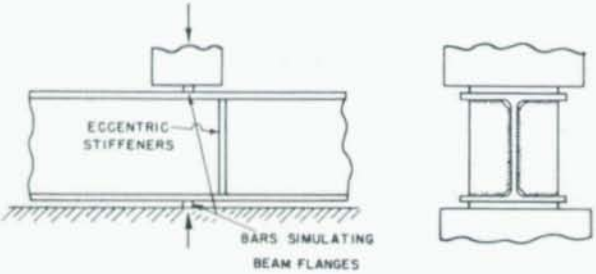


FIGURE 24

Tests with Eccentric Stiffeners

The results of the eccentric stiffener tests are given in Table 5. As can be seen from both series the stiffeners of eccentricity 2" provided about 65% of the stiffening action of the concentric stiffener whereas the stiffeners of eccentricity 4" and greater provided less than 20% of the concentric stiffening action. Figure 25 shows E-8 at failure.

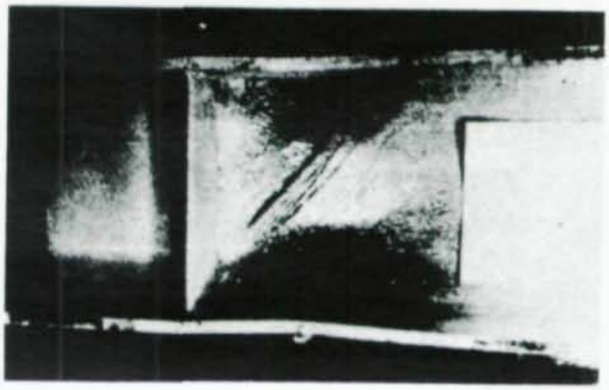


FIGURE 25

Test E-8

Part B—Discussion of Test Results

1. Connection Requirements

In a beam-to-column welded connection there are several regions which are subject to local overstress and therefore it appears pertinent, before discussing the behavior of the tested connections, to define a satisfactory connection. It is defined as one which is capable of developing the theoretical maximum moment of resistance of the beams (the "plastic moment") when working axial load is on the column. A desirable additional quality of a satisfactory connection is that it maintains its moment capacity for a considerable rotation at the ultimate load. The rotation required at plastic hinges (namely, the "hinge angle") for a variety of practical structures has been determined in Reference 4 and its particular application to this investigation is treated in Section 1.2 of the Appendix.

2. Two-way Connection Tests

A significant feature of these tests was the ability of the connections to develop the strength of the beams. In all cases except two, (A-1 and A-4) where column web crippling was responsible for failure, the beams were not only able to reach their predicted ultimate load, but were able to sustain this load over considerable rotation.

Local buckling is a factor which might influence the value of the plastic moment of a beam section and of its rotation capacity. Haaijer[6] has determined the proportions of sections that will not buckle before the onset of strain hardening. The width to thickness ratio of the beam flange, b/t, should not exceed 17, and the ratio, d/w_b, (beam depth to web thickness) should not exceed 70 for pure bending. The beam section chosen (16 WF 36) was just within these values, with the result that local buckling coincided with the beginning of strain hardening and was not detrimental to the strength of the connection.

In comparing the theoretical and experimental moment-rotation curves (Figures 8, 9, and 10) in the elastic range, the connections are not as stiff as the 16 WF 36 beams. This flexibility is of course due to strains in the column. These were greatest in Specimen A-1, with A-4, B-6, B-8, C-9 and C-11 also showing noticeable deviation from the theoretical curve.

The structural adequacy of a particular type of welded beam-to-column connection can be ascertained in part by comparing the moment and rotation capacity of the beam with the local resistance and the local stiffness of the column. The column must have strength to resist the beam moment, but it need not necessarily be as stiff as the beam. The desirable strength and rotation capacity is supplied partly by the column and partly by the end portion of the beam. Specimen A-1 with its unstiffened, thin-web column section is a notable example where column web buckling was the principal cause for the high rotations at low moments. In border-line cases, as for example A-4, the buckling of the column web did not become excessive and the deformations are due to a combination of high inelastic strains in the column web in areas of both tension and compression and to some web buckling. Thus this investigation clearly demonstrates the importance of the column web opposite the compression flanges of the beams.

From strain gage readings it was calculated that the vertical plate stiffeners of Series C in the elastic range, each transmitted only about $\frac{3}{16}$ths of the forces coming from the beam flanges and the web transmitted $\frac{5}{8}$ths. Placing these stiffener plates closer to the column web might have improved the distribution. However, since the prime purpose of this type of connection is to afford a convenient four-way connection, the plate usually needs to be positioned flush with the edge of the column flange.

Although there were high stress concentrations at the centers of the butt welds in the Series A and H tests, it was noted that no weld failures occurred until after excessive rotation had taken place.

3. Four-way Connection Tests

All three specimens satisfied the criteria by both possessing the strength to develop the theoretical beam plastic moment and by showing sufficient rotation capacity at peak loads.

Test AA, as shown in Figure 26, was stronger than its two-way counterpart, Test A-4. This evidently shows that the stiffening action provided by the two beams framing

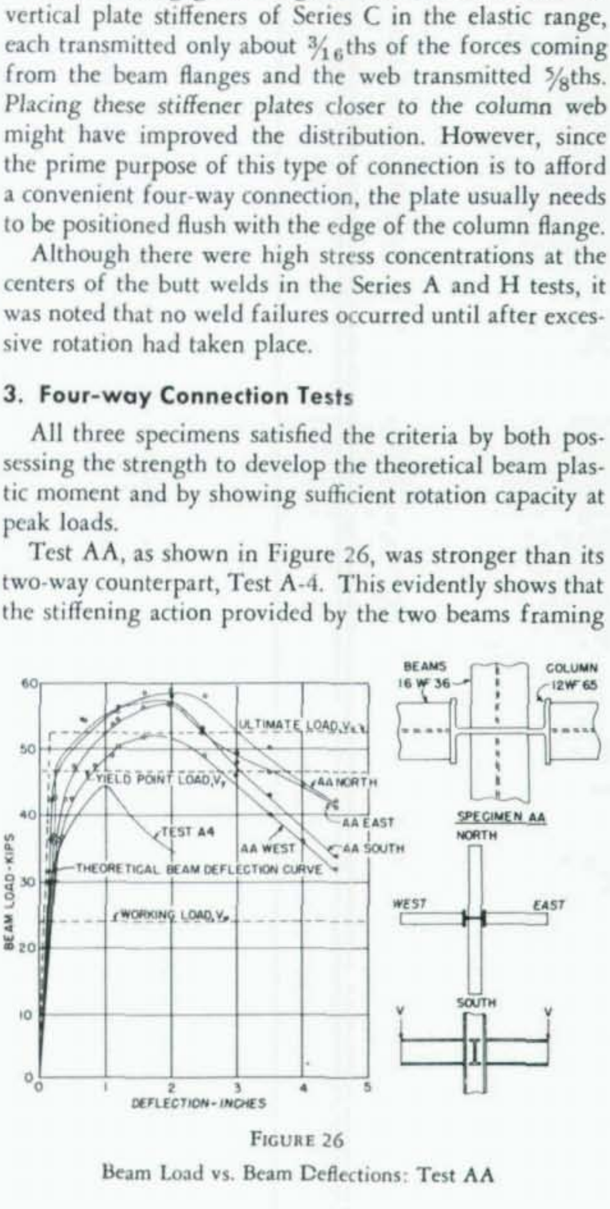


FIGURE 26

Beam Load vs. Beam Deflections: Test AA

onto the column web strengthens the connection more than it is weakened by consequences of the triaxial stresses. In both tests DD and D-12 the split beam tee stiffeners effectively prevented any buckling of the connection. Test BB cannot be compared with a two-way test since it had no two-way counterpart.

4. Effect of Axial Load

In both the two and the four-way tests the column axial load had little effect on the strength and rotation capacity of the connection. The columns showed no particular signs of distress when subjected to an axial load of 1.65 times working load* except that specimen BB showed straining in the web of the 12 WF 40 column. Since the strain lines were not found throughout the cross-section it may be presumed that residual stresses may have been at least partly responsible for the appearance of these strain lines. Further, at the end of each test, with the final beam loads still applied, twice column working load was applied with no evidence of marked distress in the column.

5. Correlation of Tests

5.1 Tests to Determine Compression Criterion

These Series E tests give much information about the actual resistance of the web of a column to local forces applied at the flanges and they are intended to simulate the compression region of a connection. However they neglect:

1. the effect of the column axial load
2. the effect of the tension region of the connection on the compression region
3. the effect of the compression from the beam web.

The discussion in Section 4 indicates that column axial load has negligible effect whereas the stress concentrations caused on the tension and compression regions are so far apart that any interaction would be small. If the tension region of the connection does not fail then we can assume that its effect on the compression region is negligible. The compression from the beam web does have some effect and this probably caused the difference in results in the following two sets of tests. Test E-18 on a 12 WF 65 column failed at a simulated beam flange load of 143 kips, whereas test A-4 in which the 12 WF 65 section was used in an actual connection failed at a computed beam flange load of 110 kips together with a computed beam web load of 40 kips.

Test BB showed much straining in the web of the 12 WF 40 column at a beam flange load of 110 kips whereas the simulated test with no beam web force failed at a simulated beam flange force of 116 kips (See Test E-3a, Table 5).

5.2 Tests to Determine Tension Criterion

The simulated tension region tests ignore:

1. the effect of the column axial load
2. the effect of the compression region of the connection on the tension region.

For similar reasons to those in Section 5.1 both of these effects should be negligible. This is borne out by the results of tests F-2 and H-1. Test H-1, in which an actual connection was subject to axial load, suffered a weld failure at a beam flange tension load of approximately 100 kips while test F-2, a simple tension test suffered the same failure at 95 kips. All of the tension failures occurred because of excessive straining in a region close to the column fillet and the center of the weld, as a result of the outward yielding of the column flanges. The shear stresses resulting from the narrowing of the tension plates due to the Poisson effect may have influenced the mode of failure in tests F-14 and F-15. These two specimens were under much higher unit tension than the other F specimens.

5.3 Eccentric Stiffener Tests

Both series of tests showed a rapid decline in the effectiveness of the stiffener for eccentricities greater than 2″. In the tests on both the 12 WF 40 and 14 WF 61 column stubs the stiffeners with 2″ eccentricity proved 65% as effective as the concentric stiffeners while those with 4″ eccentricity were only 20% as effective as the concentric stiffeners. Stiffening with still greater eccentricity had virtually no effect. For design purposes it would probably be advisable to neglect the resistance of stiffeners having eccentricities greater than 2″.

* Working load corresponds to an average axial stress of 14.5 ksi.

Part C—Analysis and Design of Connections

1. Analysis of Connections

As stated in Part B a satisfactory connection is defined as one which is capable of developing the theoretical maximum moment of resistance of the beams when working axial load is on the column. It is also desirable for the connection to have sufficient rotation capacity as explained in Part B.

The analysis then should determine those items which are necessary at the joint to ensure development of the plastic moment at the connection and, if possible, adequate rotation capacity. Potential items for investigation are:

1. The strength of that region of the connection adjacent to the beam compression flange when no column stiffening is required.
2. The strength of that region of the connection adjacent to the beam tension flange when no column stiffening is required.
3. The increase in the strength of the connection due to the presence of stiffeners.
4. The possibility of column failure due to a combination of axial and local stresses.
5. The effect of the pair of beams framing into the column web on the connection of the other pair of beams onto the column flanges.
6. The rotation required of connections and their capacity to rotate.

Items 1, 2 and 3 will be discussed in Sections 1.1 and 1.2 of this Part and also in the Appendix. Items 4 and 5 have been discussed in Part B, their effects having been deduced from the observation of tests. It has been explained that the effects of column axial load can be neglected and that the stiffening action of the second pair of beams strengthens the connection more than the triaxial stresses set up in the column web weaken it. A conservative procedure would then be to analyze the connection as if the second pair of beams were not present. *Item 6 has been investigated both analytically and* experimentally. The rotation required of connections can be found from Reference 4. This of course varies with the beam loading, size and span but in Section 1.2 of the Appendix there is calculated a sample value of the required rotation which will be greater than that required by most connections. For purposes of comparison this value has been plotted on Figures 8, 9, 10 and 17 which show moment rotation curves of tested connections. Inspection of these figures shows that all tested connections do have sufficient rotation capacity. Moreover, if the connection is made stronger, so that it is much stiffer than the beam at M_p, the necessary rotation will occur in the end of the beam.

1.1 Analysis of Compression Region of Connection

This analysis, the idealized approach, idealizes the beam as placing on the compression region of the connection a concentrated force at the level of the beam flange. *The force of the connection from the beam web* is neglected.

The critical item in this region in an unstiffened connection is the buckling of the column web. From experimental evidence as discussed later (for illustration see Appendix 1.3 and 1.4) a conservative estimate of the strength of the compression region of a connection could be obtained by assuming that the resistance supplied by the column web in resisting the beam flange force is $\sigma_y\, w\, (t + 5k)$.

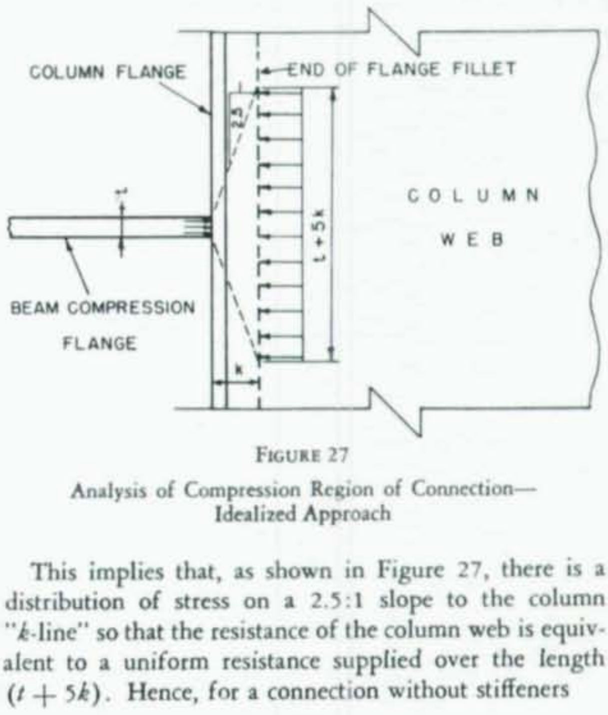


FIGURE 27

Analysis of Compression Region of Connection—
Idealized Approach

This implies that, as shown in Figure 27, there is a distribution of stress on a 2.5:1 slope to the column "k-line" so that the resistance of the column web is equivalent to a uniform resistance supplied over the length $(t + 5k)$. Hence, for a connection without stiffeners

$$Q_c = \sigma_y\, w\, (t + 5k) \tag{1}$$

Now the force supplied by the beam flange when the beam is under plastic moment is $bt\,\sigma_y$ so the minimum column web thickness required is given by

$$bt\,\sigma_y = \sigma_y\, w\, (t + 5k) \tag{2}$$

or

$$w = \frac{bt}{t + 5k} \tag{3}$$

In cases where $w > \dfrac{bt}{t+5k}$ stiffeners are not required in the compression region of the connection.

In cases where $w < \dfrac{bt}{t+5k}$ and stiffeners are required formula (2) is modified to include the resistance of these stiffeners.

$$bt\,\sigma_y = \sigma_y\,w\,(t+5k) + \sigma_y\,A_{st} \tag{4}$$

In the case of horizontal plate stiffeners having a total width equal to the width of the beam flange A_{st} may be approximated as

$$A_{st} = t_s\,b$$

Hence, the required stiffener thickness

$$t_s = \frac{bt - w\,(t+5k)}{b} \tag{5}$$

As a further limitation (See Section 1.1 of the Appendix),

$$t_s \geqslant \frac{b_s}{16} \tag{6}$$

Tests C-9, C-11 and D-12 indicate that the vertical plate stiffeners carry about half the stress that the column web does. Making this assumption, formula (4) becomes in the case of vertical plate stiffeners,

$$bt\,\sigma_y = \sigma_y\,w\,(t+5k) + \frac{\sigma_y}{2}\,2\,t_s\,(t+5k)$$

so that

$$t_s = \frac{bt}{t+5k} - w \tag{7}$$

As a further limitation (See Section 1.1 of the Appendix),

$$t_s \geqslant \frac{d_c}{30} \tag{8}$$

In those cases in which the beam flange width is much less than the column flange width these C type stiffeners would not be as effective as assumed and it would be inadvisable to rely on their stiffening action when the column web is greatly deficient according to formula (3).

Eccentric Stiffening

Since the testing done on eccentric stiffeners was very limited, observations concerning their action cannot be conclusive. Very light columns were used; hence the results should if anything be conservative.

The tests have indicated that the horizontal plate stiffeners of eccentricities greater than 2″ had very little stiffening action. A conservative design procedure then would be to neglect the action of such stiffeners and to consider stiffeners of eccentricities of 2″ or less as 60% effective as compared to concentric stiffeners. In this latter case, equation (4) becomes

$$bt\,\sigma_y = \sigma_y\,w\,(t+5k) + 0.6\,\sigma_y\,t_s b$$

which reduces to

$$t_s = \frac{1.7}{b}\,[bt - w\,(t+5k)] \tag{9}$$

where again

$$t_s \geqslant \frac{b_s}{16} \tag{6}$$

Two other methods of analysis of the compression region of the connection have been suggested in the Appendix but the above analysis for both concentric and eccentric stiffeners is advocated for use.

1.2 Analysis of Tension Region of Connection

The mechanism of failure in this region is as follows: a column flange acts as two plates, each of which is fixed along three edges and free along the other together with a central rigid portion, the whole being loaded by the beam tension flange. The load remains more or less uniformly distributed until the "plates" reach their ultimate carrying capacity. At this stage, the "plates" deflect at their outer edges causing excessive straining in the central portion of the butt weld, in the column flange adjacent to the weld and in the column fillet. Failure then occurs by cracking in one of these regions. The "plates" are under bending action so their ultimate capacity depends on the square of their thickness. Analysis in the Appendix (Section 1.6) illustrates that a conservative estimate of the capacity for each "plate" for wide flange columns is $3.5\,\sigma_y\,t_c^2$. The central rigid part of width 'm' adjacent to the column web will be highly strained and hence will carry a force corresponding to its area at yield stress. Hence

$$Q_t = \sigma_y\,tm + 7\sigma_y\,t_c^2 \tag{10}$$

The force in the beam tension flange when plastic moment is applied to the beam is $bt\,\sigma_y$. To give 20% conservatism in this region of the connection corresponding approximately with the average conservatism in the compression region one obtains

$$bt\,\sigma_y = 0.8\,[\sigma_y\,tm + 7\,\sigma_y\,t_c^2] \tag{11}$$

This reduces to

$$t_c^2 = \frac{bt}{7}\left[\,1.25 - \frac{m}{b}\,\right] \tag{12}$$

t_c being the required column flange thickness.

If beam and column sizes are taken from the AISC

Manual then the value of m/b for all those connections in which formula (12) is approximately satisfied varies from 0.15 to 0.20. Making the conservative assumption $m/b = 0.15$, (12) reduces to

$$t_c = 0.4\sqrt{bt} \qquad (13)$$

In cases where $t_c > 0.4\sqrt{bt}$ stiffeners are not required in the tension region of the connection.

In cases where $t_c < 0.4\sqrt{bt}$ and stiffeners are required one has equilibrium configurations exactly the same as those in the compression region of the connection. Hence stiffening requirements will be given by equations (5), (6), (7) and (8).

1.3 Relative Strengths of Tension and Compression Regions of the Connection

Equation (3) states that a connection will be on the verge of needing stiffeners in the compression region if

$$w = \frac{bt}{t + 5k}$$

or

$$bt = w\,(t + 5k) \qquad (14)$$

From equations (13) and (14) this connection will or will not need stiffeners in the tension region according to whether

$$t_c \lesseqgtr 0.4\sqrt{w\,(t + 5k)}$$

i.e.,

$$\frac{t_c}{\sqrt{wk}} \lesseqgtr 0.4\sqrt{5 + t/k} \qquad (15)$$

Since for all practical connections in which (12) is approximately satisfied

$$0.2 < t/k < 0.8$$

then by taking $t/k = 0.2$ it can be seen that this connection will need stiffeners in the tension region if

$$\frac{t_c}{\sqrt{wk}} < 0.91 \qquad (16)$$

and by taking $t/k = 0.8$ it can be seen that this connection will not need stiffeners in the tension region if

$$\frac{t_c}{\sqrt{wk}} > 0.96 \qquad (17)$$

Figure 28 shows a plot of the values of $t_c/\sqrt{wk}$ for all 8″, 10″, 12″ and 14″ nominal depth columns of the wide flange series. It can be seen from this figure that in most cases the critical region of the connection depends only on the column parameters. For values of $t_c/\sqrt{wk}$ between 0.91 and 0.96 the need for column stiffening will depend on the beam.

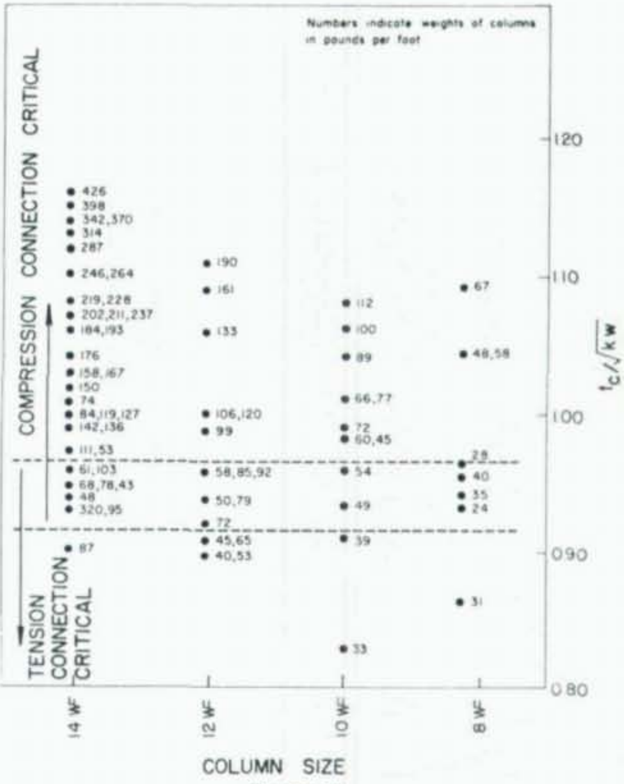


FIGURE 28

Critical Parts of Connections between Standard WF Beams and Columns

2. Comparison of Test Results with Analysis

2.1 Compression Region of Connection

As explained in Part B, the connection tests gave somewhat different results from the analogous compression tests because the former involved the additional compression supplied by the beam web. As can be seen from Table 7, the assumption of a length of $(t + 7k)$ of column web at yield stress resisting the force applied through the simulated beam flange in the compression tests (Series E) is conservative. Also, as seen from Table 6, the use of the compression design criterion

$$w = \frac{bt}{t + 5k} \qquad (3)$$

advocated in the last section leads to conservative results when compared with connection tests. The results from Table 6 are summarized as follows:

1. For test A-1, formula (3) requires that the column web be 0.666″ thick. The actual thickness was 0.284″, and the column web failed at a load slightly in excess of working load as shown in Figure 6.
2. For test A-2, the formula requires a web thickness of 0.428″ and as would be expected the thickness of 0.587″ proved satisfactory.

3. Connection A-4 requires a web thickness of 0.470″. With an actual thickness of 0.417″, the connection attained over 80% of the required moment.
4. The formula shows A-5 to be entirely adequate without stiffeners and it so proved to be.
5. The formula shows H-1 to be slightly inadequate but it did take the maximum moment reached in the test, this moment being 95% of the plastic moment. There was some straining in the column web, but failure did not appear to be imminent in the compression region.
6. The formula shows AA to be inadequate, but, probably because the stiffening action of the second pair of beams was not considered in the analysis, the connection proved satisfactory.
7. For B-6, B-8 and BB, the formulas show thin stiffeners to be required. In the tests there was no evidence of overstress in the stiffeners actually supplied, except for a few strain lines in the B-8 stiffeners.
8. The formulas showed the C, D and DD connections to be adequate and so they proved to be. By the time the beams had failed, however, there was some buckling in the column stiffeners.

The theoretical restraint provided by horizontal stiffeners in a connection is given by $\sigma_y bt_s$ (refer to formula (4)).

Comparison with tests show:

a. Test E-1 in which an unstiffened 12 WF 40 was compressed failed at 102.5 kips whereas test E-0 in which the same column was stiffened with two 1/4″ horizontal stiffeners failed at 172 kips. The difference of 69.5 kips compares favorably with the calculated difference of 63 kips.

b. A similar examination of tests E-9 and E-20 on a 14 WF 61 show an experimentally determined difference of 144.5 kips compared to the calculated difference of 115 kips.

TABLE 6

COMPARISON OF COMPRESSION REGION CRITERION WITH CONNECTION TEST RESULTS

Specimen	bt in.²	k in.	Req'd w‡ in.	Manual w in.	Measured w in.	Req'd t_s in.	Actual t_s in.	Remarks
A-1	2.99	0.812	0.666	0.288	0.284			Column web buckled
A-2	2.99	1.312	0.428	0.575	0.587			Column web O.K.
A-4	2.99	1.188	0.470	0.390	0.417			Column web weak
A-5	2.99	1.500	0.378	0.580	0.580			Column web O.K.
B-6	2.99	0.812		0.288	0.284	0.258‡‡	0.437	Stiffened connections
B-8	2.99	1.125		0.294	0.300	0.25*	0.250	satisfactory
C-9	2.99	0.812		0.288	0.284	0.382†	0.437	Connections O.K. but
C-11	2.99	1.125		0.294	0.300	0.40**	0.250	some stiff. buckling
D-12	2.99	1.125		0.294		0.39**	0.606	Connection O.K.
H-1	2.99	0.812	0.666	(0.288)	0.600‡‡‡			(1)
AA	3.02	1.188	0.474	0.390	0.395			Connection O.K.
BB	2.89	1.125		0.294	0.316	0.25*	0.5	Connection O.K.***
DD	2.91	1.125		0.294	0.317	0.40**	0.6	Connection O.K.

(1) Column web O.K. up to $0.95M_p$ when failure occurred in tension region of connection.

* Determined by slenderness limitation, Equation (6).

** Determined by slenderness limitation, Equation (8).

*** Seat 4″ above compression flange. Stiffening also included a vertical plate beneath the horizontal stiffener.

‡ Determined by equation $w = \dfrac{bt}{t + 5k}$

‡‡ Determined by equation $t_s = \dfrac{bt - w(t + 5k)}{b}$

‡‡‡ 5/16″ doubler plate added to web.

† Determined by equation $t_s = \dfrac{bt}{t + 5k} - w$

There is some inconsistency in the above compression region analysis since a length of column web of $(t + 7k)$ is assumed to be effective in the simulated tests whereas an effective length of only $(t + 5k)$ is assumed in the connection tests. Formula (24) given in the Appendix is possibly a more rational approach to the analysis of the compression side. This formula

$$w = \frac{bt + 3.5kw_b}{t + 7k} \tag{24}$$

is consistent when applied to the connection tests and to the simplified tests. In the simplified tests of course $w_b = 0$. However formulas (3) and (24) give nearly the same results when applied to practical connections and are simpler.

2.2 Tension Region of Connection

The only connection specimen in which the primary cause of failure was in the tension region was test H-1 where failure occurred at approximately 95% of the beam plastic moment. The actual column flange thickness in this case was 0.433″ while that required by formula (13) is 0.69″. Hence in this case formula (13) appears conservative.

Table 8 compares the tension tests with the analysis by means of two methods—first through the ultimate capacity equation (10) and then through the final design equation (13).

The comparison with equation (10) shows conservatism in all cases except test F-15. However in this case the plate was strained into the strain hardening range and failure was probably caused by shearing stresses at the ends of the weld due to drawing down of the plate. A further indication of this is that the weld failure began at one end of the weld. This type of failure would not occur in an actual connection since the beam flange is not stressed above yield stress.

The second comparison, between actual column flange thickness and that required by equation (13), is mainly of statistical interest. The last column shows the ratio of tension plate stress at column failure to tension plate yield stress and illustrates that in all but three tests (F-4, F-14 and F-15) the tension plate was much stronger than would have been sufficient to cause column failure at or prior to tension plate yield. Considerable conservatism in equation (13) is illustrated in the cases of F-4 and F-14. This is probably due to the 20% conservatism introduced in equation (11).

3. Limitations of This Investigation

The investigation considered two- and four-way interior beam-to-column connections in which every beam of the connection was loaded equally and gradually to failure. Some modification of the reported behavior might have been observed if the following variations had been included:

a. *Repetitive Loading.* A sufficient number of cycles of loading and unloading could cause premature failure but this is unlikely since much of the load in a build-

TABLE 7

COMPARISON OF FORMULA, $Q_c = \sigma_y w (t + 7k)$ WITH COMPRESSION TESTS

Test	Column	Bar Thickness in.	Yield, σ_y ksi	Column Web w* in.	k* in.	Computed Q_c kip	Test Q_a kip
E-1	12 WF 40	½	40.2	0.294	1.125	99	102.5
E-14	8 WF 48	½	34.4	0.405	1.063	110.1	137
E-15	8 WF 58	½	36.2	0.510	1.188	162.6	202.5
E-16	10 WF 66	½	40.0	0.457	1.25	169.0	175.7
E-17	10 WF 72	½	35.0	0.510	1.313	173	190
E-18	12 WF 65	½	37.2	0.390	1.188	129	143
E-19	12 WF 85	½	37.8	0.495	1.375	190	247.5
E-20	14 WF 61	½	36.2	0.378	1.25	127	137.5
E-21	14 WF 68	½	38.3	0.418	1.313	155	164
E-22	14 WF 84	½	39.3	0.451	1.375	180	221
E-23	14 WF 103	½	38.5	0.495	1.438	201	250

* AISC Manual values.

ing is dead load and any variation of total stress would be of small magnitude.

b. *Unequal Loading of Opposing Beams.* In this case shear stresses would be induced in the column web. However, when the beam loadings are approximately the same as is usually the case at interior columns the above design formulas would be valid. They might require modification in the extreme case of a beam framed into only one column flange.

c. *Wind Loading.* This would tend to cause moments in the same direction and hence high shear stresses in the column web.

4. Advocated Design Methods

There follow examples of connection design using the proposed formulas.

4.1 Connection in Which No Stiffening is Required

Consider a two-way connection in which 16 WF 50 beams frame onto the flanges of a 12 WF 99 column. From formula (3) required

$$w = \frac{bt}{t + 5k}$$

$$w = 0.546''$$

$$\text{actual } w = 0.580''$$

Hence no stiffening is needed in the compression region of the connection.

From formula (13)

$$\text{required } t_c = 0.4\sqrt{bt}$$

$$= 0.842''$$

$$\text{actual } t_c = 0.921''$$

Hence no stiffening is needed in the tension region of the connection. The computation for tension stiffening could have been omitted by inspection of Figure 28 which shows that the compression region of the connection for a 12 WF 99 is the critical one regardless of beam dimensions.

4.2 Connection in Which Stiffening is Required in Compression Region Only.

Consider a two-way connection in which 16 WF 58 beams frame onto the flanges of a 10 WF 89 column.

From formula (3)

$$\text{required } w = 0.670''$$

$$\text{But actual } w = 0.615''$$

Hence stiffening is required in the compression region of the connection. The required size of horizontal plate stiffeners is given by equations (5) and (6).

From equation (5)

$$\text{required } t_s = \frac{bt - w\ (t + 5k)}{b}$$

$$= 0.053''$$

TABLE 8

COMPARISON OF TENSION REGION ANALYSIS WITH TENSION TESTS

Test No.	Column Stub	Yield Stress* Column Flange	Plate σ_y	Ultimate Capacity, Q_t Computed from (10)**	Test	Av. Plate Stress at Test Ult., σ_a	Flange Thickness Computed from (13)**	Actual	σ_a/σ_y
F-1	8 WF 31	37.0	38.9	81	100	19	0.94	0.43	0.49
F-2	8 WF 31	37.0	38.9	68	95	31	0.72	0.43	0.80
F-3	12 WF 65	36.0	31.6	123	149	28	0.86	0.61	0.89
F-4	14 WF 68	34.2	31.6	155	167	32	0.89	0.72	1.01
F-5	14 WF 84	34.2	31.9	191	212	21	1.27	0.78	0.66
F-9	12 WF 65	36.0	31.6	55	82	15	0.86	0.31	0.47
F-10	14 WF 84	34.2	31.9	80	125	12	1.27	0.38	0.38
F-12	12 WF 65	36.0	31.8	167	189	15	1.35	0.61	0.47
F-13	14 WF 68	34.2	31.8	200	199	16	1.37	0.72	0.50
F-14	8 WF 67	33.5	38.9	242	256	45	0.99	0.93	1.16
F-15	14 WF 176	36.0	31.9	456	444	44	1.24	1.31	1.38

Dimensions of the specimen are given in Table 4.

* Measured from coupon tests.

** Adjusted for variation in yield stresses from 33 ksi.

But from equation (6)

$$t_s \geqslant \frac{b_s}{16}$$

$$\geqslant 0.25''$$

Hence in compression region of connection use $\frac{1}{4}''$ horizontal plate stiffeners, welded along three edges.

If vertical plate stiffeners are required equation (7) gives

$$\text{required } t_s = \frac{bt}{t+5k} - w$$

$$= 0.056''$$

But from equation (8)

$$t_s \geqslant \frac{d_c}{30}$$

$$\geqslant 0.362''$$

Hence use $\frac{3}{8}''$ vertical plate stiffeners.

From formula (13)

$$\text{required } t_c = 0.934''$$

$$\text{But actual } t_c = 0.998''$$

Hence no stiffening is required in the tension region of the connection.

4.3 Connection in Which Stiffening is Needed in Both Tension and Compression Regions

Consider a connection in which 18 WF 105 beams frame onto the flanges of a 12 WF 65 column. Equations (3) and (13) indicate that stiffeners are required in both the tension and compression regions of the connection.

If horizontal plate stiffeners are to be used, equation (5) gives

$$t_s = 0.685''$$

which satisfies equation (6).

Hence use $1\frac{1}{16}''$ horizontal plate stiffeners in both tension and compression regions of the connection.

If vertical plate stiffeners are to be used equation (7) gives

$$\text{required } t_s = \frac{bt}{t+5k} - w$$

$$= 1.179''$$

From Equation (8)

$$t_s \geqslant \frac{d_c}{30}$$

$$= 0.405'' < 1.179''$$

Hence use $1\frac{3}{16}''$ vertical plate stiffeners flush with the toes of the column flanges.

4.4 Eccentric Stiffening

Consider the same connection as in Section 4.3 with, in addition, two 16 WF 36 beams framing into opposite sides of the web of the 12 WF 65 column. If the tension flanges of the beams are at the same level then the seating plates of the 16 WF 36 beams can be used as stiffeners of approximately 2" eccentricity for the 18 WF 105 beams.

The required thickness, t_s, is given by equation (9);

$$t_s = \frac{1.7}{b}[bt - w\,(t+5k)]$$

$$= 1.164''$$

This satisfies equation (6).

Hence use $1\frac{3}{16}''$ seating plates for the 16 WF 36 beams.

Conclusions

Results of this investigation show that stiffening may be omitted in many beam-to-column connections. Summarized in the Synopsis are recommendations for design defining the cases for which stiffening may be omitted, and also suggesting the proportions of stiffeners for cases when they are needed.

Appendix

1. Theoretical Analysis

1.1 Limiting Slenderness of Stiffeners

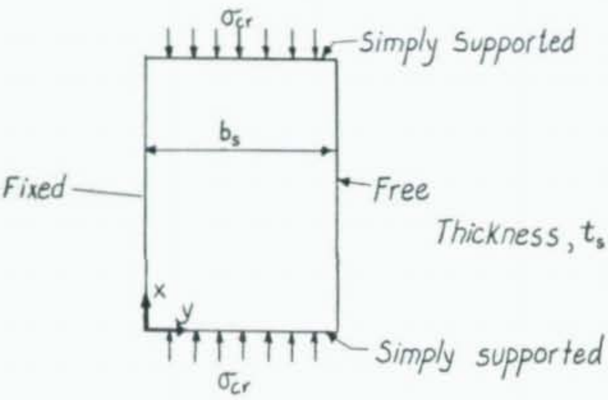
The slenderness limits for the stiffeners are difficult to establish because —

a. The restraint provided by welds at the ends of stiffeners is not known.

b. The stress distributions in the stiffeners are not known.

The assumptions made in the following analysis probably lead to conservative limits. The calculations for the limiting slenderness of stiffeners are taken from formulas and figures in Reference 6.

Horizontal Stiffener — B Type



As indicated in the Figure, consider the stiffener fixed along the edge welded to the column web and conservatively assume it simply supported along the edges welded to the column flanges.

Using formula (3.15) of Reference 6 and the constants

$$D_x = 8{,}000 \text{ ksi}$$

$$D_{xy} = 16{,}000 \text{ ksi}$$

$$D_y = 31{,}000 \text{ ksi}$$

$$G_t = 2{,}400 \text{ ksi} \quad \text{(Ref. 6)}$$

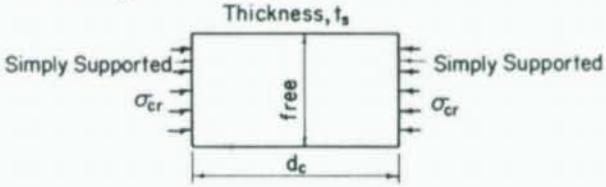
$$\sigma_{cr} = 7570 \left(\frac{t_s}{b_s}\right)^2$$

For $\sigma_{cr} = \sigma_y = 33$ ksi

$$\frac{b_s}{t_s} = 15.2$$

To round figures $\dfrac{b_s}{t_s} = 16$ (6)

Vertical Stiffener — C Type



As indicated in the Figure, consider the stiffener simply supported along the edges welded to the column flanges.

$$\sigma_{cr} = \frac{\pi^2 E}{12\,(1-\gamma^2)} \left(\frac{t_s}{d_c}\right)^2$$

For $\sigma_{cr} = \sigma_y = 33$ ksi

$$\frac{d_c}{t_s} = 30 \qquad (8)$$

1.2 Rotation of Connections

Examination of Figure 13 of Reference 4 shows that the "hinge angle" or rotation at plastic moment required at the ends of a fixed ended beam uniformly loaded along its length, so that it will be able to form a mechanism, is given by

$$H = \frac{1}{6}\ \phi_p L \qquad (18)$$

$$\text{or } H = \frac{M_p L}{6 EI} \qquad (19)$$

Taking a practical case of a 16 WF 36 beam of 24′ span the required rotation is calculated to be

$$H = 7.2 \times 10^{-3} \text{ radians}$$

Here a particular case is taken but the above value ot the rotation will be greater than that required of most connections. Considering a 12″ gage length spanning across the column the average rotation required across this length is 1.2×10^{-3} radians per inch. This value is plotted on all figures showing connection rotation characteristics.

1.3 Elastic Distribution of Stress on Column 'k' Line

E. W. Parkes[5] developed a theory giving the stress distribution just inside the flange of a column (in this case the column 'k' line) for either a tension or compression loading on the flanges while the stresses are still in the elastic range. For purposes of our case we will make the idealizations that —

1. The load applied to the column flange can be con-

sidered as a line load perpendicular to the column web.

2. The moment of inertia of the beam flange about the axis through its own centroid parallel to the flange can be considered as infinite.
3. The distance between the column 'k' line and the centroid of the column flange can be considered as negligible compared to the depth of the column.
4. As far as stress analysis is concerned the web of the column can be considered as infinitely wide so that the stress distribution at mid width is uniform.

Parkes analyzes the case mentioned above and also the realistic case where the above idealizations do not apply. For the case of all wide flange columns as used in practice however the deviation in the elastic stress distribution between the idealized and the realistic cases is less than 5%. Being based on the idealized case then the non dimensionalized curve as drawn in Figure 29 represents to ±5% the elastic stress distribution along the column 'k' line for all wide flange shapes used in practice. The scale of Figure 29 has been made so that the area beneath this curve represents the ultimate load as obtained from tests. For purposes of plotting this figure Parkes used the non dimensionalizing parameters x_0 and σ_0 which were functions of the column dimensions. The curve, of course, is not the stress distribution at failure since yielding will have taken place. However, by the use of the appropriate vertical scale factor this curve will represent the stress distribution until the first yielding occurs.

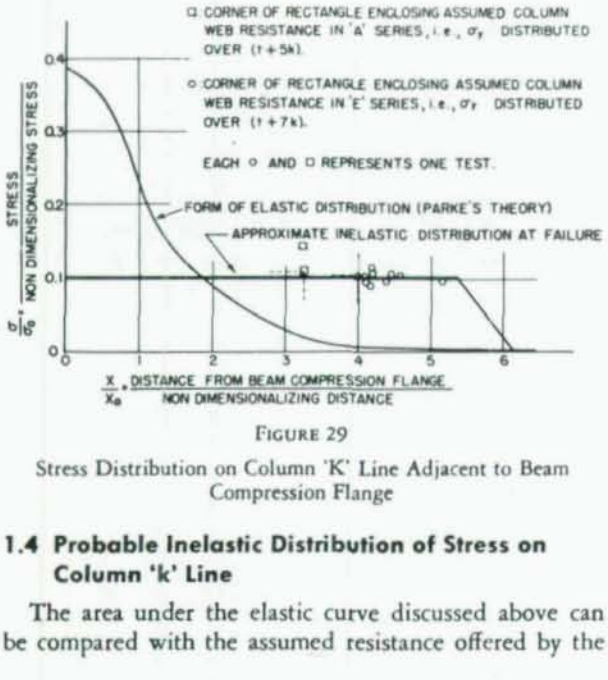


FIGURE 29

Stress Distribution on Column 'K' Line Adjacent to Beam Compression Flange

1.4 Probable Inelastic Distribution of Stress on Column 'k' Line

The area under the elastic curve discussed above can be compared with the assumed resistance offered by the column web in the development of the compression criterion in Section C. This resistance is represented by the corners of the rectangle in Figure 29 which show yield point stress distributed over a distance $(t + 5k)$ for the 'A' Series Tests and over a distance $(t + 7k)$ for the 'E' Series Tests.

As illustrated in the figure it does so happen that the non dimensionalizing stress, σ_0, as used by Parkes causes the ratio σ_y/σ_0 to have values very close to 0.1 for all the specimens tested except the column section 12 WF 65 as used in test A-4. Hence the actual inelastic stress distribution at failure for all the test cases except A-4 is represented closely by the plot on Figure 29 which includes the horizontal line at yield stress representing the inelastic resistance and the oblique line representing the elastic resistance. Since the area under this curve is greater than the area under the curves representing the assumed resistance of the column webs then the assumption of a distribution of yield stress over a distance of $(t + 5k)$ or $(t + 7k)$ as the case may be is conservative.

It is also interesting to note the stress distribution at various stages of loading. In the elastic stages of the tests, the distribution of stress is similar to that shown by the elastic curve. After a little yielding has occurred, a plateau will develop at yield stress. This plateau will become wider as the load increases until at failure the distribution is as shown.

1.5 Alternative Design Formulas for Compression Region of Connections

The idealized method of design has been described in Section C. Two other approaches are however worthy of note:

1.51 Plastic Analysis Approach

This approach assumes a stress distribution in the beam, loaded to its capacity M_p, as shown by Section a-a in Figure 30-a. The corresponding stress distribution in the column web at the end of the flange-to-web fillet is shown by Section b-b. This procedure results in the following analysis:

a. *Unstiffened Columns.* (*Series A*). Assume the beam is developing its plastic moment, M_p. For the compression flange the pressure against the column will be approximately as shown in Figure 30-a.

$$\text{Then } Q_b = \frac{A_b}{2}\sigma_y$$

$$\text{and } Q_c = \sigma_y w \left[\frac{d}{2} + 3k\right]$$

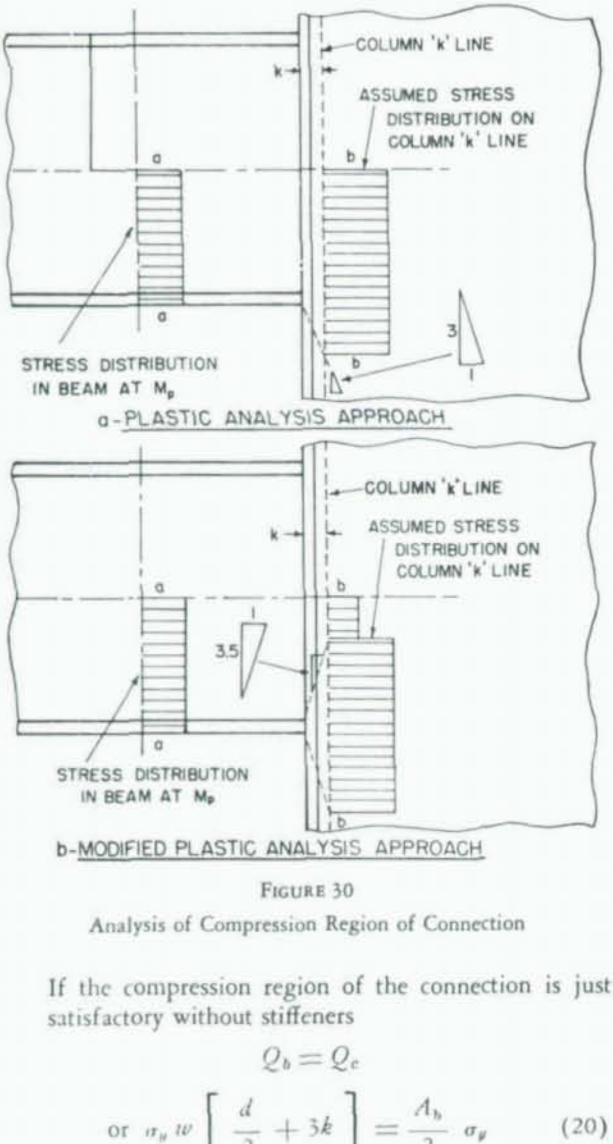


FIGURE 30

Analysis of Compression Region of Connection

If the compression region of the connection is just satisfactory without stiffeners

$$Q_b = Q_c$$

$$\text{or } \sigma_y w \left[\frac{d}{2} + 3k \right] = \frac{A_b}{2} \sigma_y \qquad (20)$$

$$\text{therefore } w = \frac{A_b}{d + 6k} \qquad (21)$$

b. *Columns with Horizontal Plate Stiffeners (Series B).* The presence of the stiffeners modifies equation (20) to

$$\frac{A_b}{2} \sigma_y = \sigma_y w \left[\frac{d}{2} + 3k \right] + \sigma_y t_s b$$

$$\text{therefore } t_s = \frac{1}{2b} [A_b - (d + 6k) w] \qquad (22)$$

t_s is again subject to the limitation that $t_s \geqslant b_s/16$ as shown in Part 1 of the Appendix.

c. *Columns with Vertical Stiffeners (Series C and D).* The presence of the stiffeners modifies equation (21) to

$$Q_b = Q_c + Q_s$$

Since the stiffener plate is at the toe of the flange it will not be as effective in resisting the beam compression as is the column web. Strain readings on web and stiffener indicate that the stresses in the stiffeners are approximately one-half those in the web.
Assuming the latter

$$Q_s = \frac{2\sigma_y}{2} t_s (t + 6k)$$

Hence

$$\frac{A_b}{2} \sigma_y = \sigma_y w \left[\frac{d}{2} + 3k \right] + \sigma_y t_s (t + 6k)$$

$$\text{therefore } t_s = \frac{1}{2} \left[\frac{A_b - w (d + 6k)}{t + 6k} \right] \qquad (23)$$

The stiffener thickness is again restricted by the inequality, $t_s \geqslant d_c/30$.

1.52 Modified Plastic Analysis Approach

The preceding analysis assumes that at failure a length of $(d/2 + 3k)$ of web is at yield stress. However in most connections the beam web is thinner than the column web so that near the horizontal centerline of the connection where the effect of the beam flange force is negligible the column web merely resists the beam web force and so is not at yield stress.

If we assume as we have done in the Series E tests and as shown in Figure 30-b that the length of column web effective in resisting the beam flange force is $(t + 7k)$ and that the beam web force outside this region is resisted by the column web immediately adjacent to it then equilibrium over the length of $(t + 7k)$ gives

a. *Unstiffened Connection.*

$$bt \sigma_y + w_b \frac{7k}{2} \sigma_y = w (t + 7k) \sigma_y$$

$$\text{or } w = \frac{bt + 3.5 k w_b}{t + 7k} \qquad (24)$$

By following the same procedure as that in Section C we have the results

b. *Horizontal Plate Stiffeners.*

$$t_s = \frac{1}{b_s} [bt + 3.5 k w_b - w (t + 7k)] \qquad (25)$$

where t_s is again subject to the limitation that $t_s \geqslant b_s/16$.

c. *Vertical Plate Stiffeners.*

$$t_s = \frac{bt + 3.5\,k\,w_b}{t + 7k} - w \qquad (26)$$

where $t_s \geqslant d_c/30$.

Table 9 compares the results of these two approaches with the approach in Part C for the connections tested.

1.6 Analysis of Tension Region of Connection

Figure *A* illustrates the action of the column flange in the tension region of the connection. The column flange can be considered as acting as two plates both of type ABCD. The beam flange is assumed to place a line load on each of these plates. The effective length of the plates is assumed to be $12\,t_c$ and the plates are assumed to be fixed at the ends of this length. The plate is also assumed to be fixed adjacent to the column web. Analysis of this plate by means of yield line theory[7] leads to the result that the ultimate capacity of this plate is

$$P_u = c_1\,\sigma_y\,t_c^2$$

$$\text{where } c_1 = \frac{4/\beta + \beta/\eta}{2 - \eta/\lambda}$$

$$\text{and } \eta = \beta\,[\sqrt{\beta^2 + 8\lambda} - \beta]/4$$

$$\beta = p/q \quad \text{(refer to figure A)}$$

$$\lambda = h/q \quad \text{(refer to figure A)}$$

For the wide flange columns and beams used in practical connections, it has been found that c_1 varies within the range 3.5 to 5.

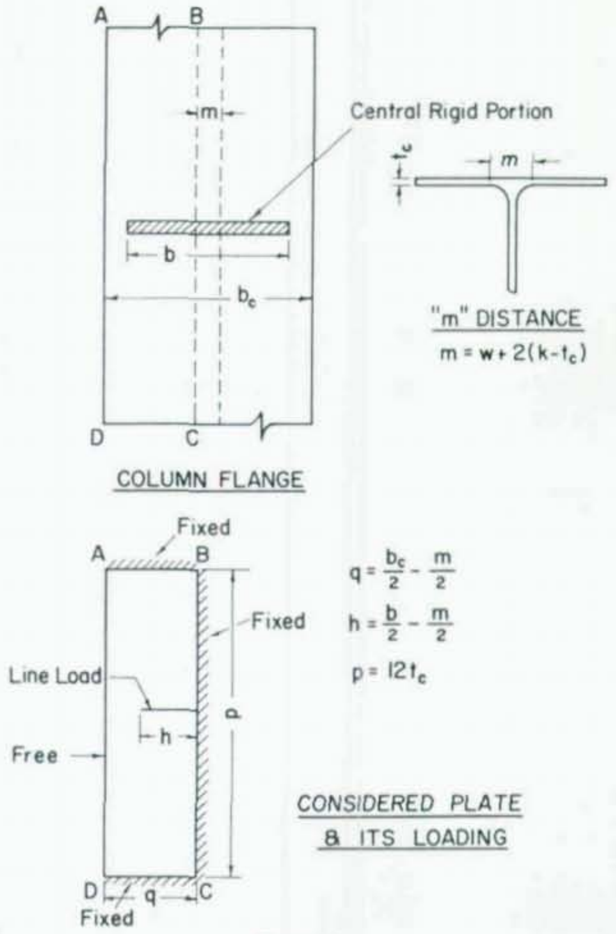


FIGURE A

TABLE 9

COMPARISON OF THE THREE COMPRESSION SIDE CRITERIA

	Web Thickness, w				Stiffener Thickness, t_s				
Specimen	Idealized	Plastic	Mod. Plas.	Actual	Idealized	Plastic	Mod. Plas.	Actual	Remarks
A-1	0.666	0.504	0.624	0.284					Column web buckled
A-2	0.428	0.440	0.450	0.587					Column web O.K.
A-4	0.470	0.453	0.480	0.417					Column web weak
A-5	0.378	0.420	0.412	0.580					Column web O.K.
B-6					0.25*	0.326	0.297	0.437	Stiffened connections
B-8					0.25*	0.261	0.25*	0.250	satisfactory
C-9					0.382	0.429	0.340	0.437	Connections O.K. but
C-11					0.34**	0.39**	0.34**	0.250	some stiffener buckling
D-12					0.34**	0.34**	0.34**	0.606	Connection O.K.
H-1	0.666	0.504	0.624	0.600					(1)
AA	0.474	0.445	0.479	0.395					Connection O.K.
BB					0.25*	0.25*	0.25*	0.5	Connection O.K.***
DD					0.34**	0.34**	0.34**	0.6	Connection O.K.

(1) Column web O.K. up to $0.95M_p$ when failure occurred in tension region of connection.

* Determined by slenderness limitations, Equation (6).

** Determined by slenderness limitations, Equation (8).

*** Seat 4" above compression flange. Stiffening also included a plate perpendicular to the seat—see Figure 15.

000018

As a conservative approximation, take $c_1 = 3.5$

Then $P_u = 3.5\,\sigma_y\,t_c^2$

Hence capacity of two plates is given by $2P_u = 7\,\sigma_y\,t_c^2$

Force carried by central rigid portion $= \sigma_y\,t\,m$

Hence $Q_t = \sigma_y\,t\,m + 7\,\sigma_y\,t_c^2$

2. Two-way Tests

2.1 Summary of Coupon Tests

Shape	Mark	E ksi	σ_{PL} ksi	σ_{yU} ksi	σ_{yL} ksi	$\sigma_{ult.}$ ksi	$\varepsilon_{st.}$ in./in.
7/16″ plate	59 E/8/3*t*	30,000	—	35.6	34.8	59.2	1.5×10^{-2}
	59 E/5/3*t*	29,500	—	35.8	34.2	59.6	—
	59 E/2/3*t*	30,200	—	35.6	34.6	60.0	—
1/2″ plate	68 E/6/3*t*	30,000	—	33.1	32.1	56.0	—
5/16″ plate	48/9/3*t*	29,900	—	38.2	37.2	62.5	—
	48/3/3*t*	31,700	—	38.2	37.8	61.3	—
1/2″ plate	68 E/6/1*c*	29,800	24.1	32.8	—	—	—
	68 E/6/2*c*	30,600	26.7	33.6	—	—	—
12 WF 40	38 G/1*tf*		35.2	36.9	37.3	62.0	1.66×10^{-2}
	38 G/2*tf*		34.3	36.3	36.5	61.7	1.7
	38 G/3*tw*		42.8	44.0	42.8	65.4	2.02
	38 G/4*tf*		36.6	38.3	37.6	61.9	1.9
8 WF 31	54 E 31/1*tf*		34.7	39.4	37.8	63.4	1.72
	54 E 31/2*tf*		36.3	—	38.1	63.0	1.94
	54 E 31/3*tw*		35.4	39.7	38.3	63.0	1.98
16 WF 36	53 E 939/1*tf*		33.5	40.8	40.0	61.7	2.16
	53 E 939/2*tf*		38.2	—	39.5	61.8	2.22
	53 E 939/3*tw*		41.4	43.5	42.7	64.5	2.17
	53 E 939/4*tf*		—	39.6	39.2	61.2	1.94
8 WF 67	54 E 67/1*tf*		—	32.4	32.2	61.4	1.18
	54 E 67/2*tf*		28.5	35.2	34.6	61.9	1.25
	54 E 67/3*tw*		—	38.8	37.7	60.6	1.94
	54 E 67/4*tf*		—	34.1	33.2	61.3	1.44
12 WF 99	55 E/2*tf*		31.3	34.6	34.5	62.5	1.31
	55 E/4*tf*		34.3	36.7	35.8	63.7	1.41
12 WF 65	42 E/1*tf*		—	37.2	36.4	62.0	1.61
	42 E/2*tf*		—	36.4	36.1	62.1	1.55
	42 E/3*tw*		—	40.6	38.8	61.5	1.43
	42 E/4*tf*		—	37.1	36.1	62.2	1.48

For WF members E is in range $25{,}000 < E < 30{,}000$ ksi.
c = compression coupon.
tf = tension flange coupon.
tw = tension web coupon.

2.2 Calculations for Design of Specimen

Columns. Assume $L/r = 72$.

Then from AISC Manual

$P/A = 17{,}000 - 0.485\,(L/r)^2$

Column Working Stress $= 14.5$ ksi

Structural Shape Details

Column	Area*	Area as measured	P_w kips	$1.65\ P_w$	$2\ P_w$	Test No.
8 WF 31	9.12	9.01	132	218	264	A1, B6, C9
8 WF 67	19.70	19.94	286	472	572	A2
12 WF 40	11.77	11.31	171	283	344	B8, C11, D12
12 WF 65	19.11	18.66	278	459	550	A4
12 WF 99	29.09	28.45	422	696	800k**	A5

* AISC Manual value.

** Testing machine capacity = 800 kips.

Analysis of Beams and Beam-to-column Flange Welds:

All dimension of sections as measured on specimens

Beams: 16 WF 36 $\sigma_w = 20$ ksi

Bending:

$$M_w = \sigma_w S = V_w L$$

$$V_w = \frac{\sigma_w S}{L} = \frac{20 \times 56.4}{48} = 23.5 \text{ kips}$$

$$V_y = \frac{\sigma_y S}{L} \qquad \sigma_y \text{ (avg. for 16 WF 36)} = 39.6 \text{ ksi}$$

$$V_y = \frac{39.6 \times 56.4}{48} = 46.5 \text{ kips}$$

$$V_u = \frac{\sigma_y Z}{L}$$

$$= \frac{39.6 \times 63.76}{48} = 52.5 \text{ kips}$$

Elastic Analysis of Welds at Beam Working Load:

Use butt welds on flanges and fillet welds on web. Web fillet welds carry both shear and bending forces of web.

$\sigma_w = 20$ ksi $\qquad d_w = 15.91 - 0.86 = 15.05''$

Maximum bending stress in web

$$= \frac{15.05}{15.91} \times 20 = 18.9 \text{ ksi}$$

Bending moment of web $= \sigma S_w$

$$= \frac{18.9 \times 0.29 \times (15.05)^2}{6} = 207 \text{ in-kips}$$

Length of each weld $= 13''$

$$f_m = \frac{Mc}{I} = \frac{6M}{2L^2} = \frac{3 \times 207}{(13)^2} = 3.67 \text{ kips/inch}$$

$$f_v = \frac{V_w}{L} = \frac{23.5}{2 \times 13} = 0.922 \text{ kips/inch}$$

$$f = \sqrt{f_m^2 + f_v^2} = 3.79 \text{ kips/inch}$$

Weld required when $f_{all} = 0.6$ kips per $\frac{1}{16}''$ leg of fillet is $\frac{7}{16}''$.

Total throat area of $\frac{1}{4}''$ fillet welds actually used is greater than total area of web, so the $\frac{1}{4}''$ weld should be able to carry any forces that the beam web can carry.

Butt weld of flanges can carry any forces the beam flanges can carry.

Shear:

$$V_u \text{ (in plastic range)} = 18\ wd^{(3)}$$
$$= 18 \times 0.29 \times 15.91$$
$$83 \text{ kips} > 52.5 \text{ kips}$$
$$V_u \text{ (predicted)} = 52.5 \text{ kips}$$
$$V_w \text{ (in elastic range)} = 13\ wd$$
$$= 13 \times 0.29 \times 15.91$$
$$= 60 \text{ kips} > 23.5 \text{ kips (O.K.)}$$

Influence of shear on V_u may be neglected if

$$V_u < \frac{\sigma_y}{\sqrt{3}} A_w$$

$$\text{i.e.,} \quad \frac{Z}{L} < \frac{A_w}{\sqrt{3}}$$

$$\frac{63.76}{48} < \frac{14.94 \times 0.29}{1.732}$$

$$1.33 < 2.5 \qquad \text{(O.K.)}$$

Lateral Buckling: elastic range

$$\frac{Ld}{bt} = \frac{96 \times 15.91}{7.09 \times 0.431} = 500 < 600 \qquad \text{(O.K.)}$$

whence $\qquad \sigma_{allow} = 20$ ksi

Local Buckling: elastic range—See Section 18(b) of AISC Specification

$$\text{Actual } \frac{b}{t} = \frac{7.09}{0.431} = 16.45 < 32 \qquad \text{(O.K.)}$$

To reach strain hardening $b/t \leqslant 17$[6]

Therefore beams are not critical for local flange buckling in plastic range.

To reach strain hardening $d/w \leqslant 55^{(6)}$

$$\frac{d}{w} = \frac{15.91}{0.29} = 54.8$$

Therefore beams are not critical for local web buckling in plastic range.

Deflections: $\delta_{\text{elastic}} = \dfrac{VL^3}{3\,EI}$ — assuming complete restraint.

$$\delta_y = \frac{V_y L^3}{3\,EI} \qquad \begin{aligned} V_y &= 46.5 \text{ kips} \\ L &= 48'' \\ E &= 30 \times 10^3 \text{ ksi} \\ I &= 448.96 \text{ in}^4 \end{aligned}$$

$$= \frac{46.5 \times (48)^3}{3 \times 30 \times 10^3 \times 448.96}$$

$$= 0.127''$$

$$\delta_{\text{ult.}} = \frac{52.5}{46.5} \times 0.127 = 0.144''$$ assuming idealized $\sigma - \varepsilon$ and $M - \phi$ relationship.

In nondimensional form:

At yield $\qquad \dfrac{V}{V_y} = 1 = \dfrac{\delta}{\delta_y}$

At ultimate $\qquad \dfrac{V_u}{V_y} = \dfrac{52.5}{46.5} = 1.13 = \dfrac{\delta_u}{\delta_y}$

Beam Rotations:

The rotation of the beam can be expressed as a change in slope of the point of load application with respect to the connection assuming the latter to develop complete restraint.

Applying the moment area theorem:

$$\theta_{\text{end}} = \frac{1}{2}\,L\,\frac{VL}{EI} = \frac{VL^2}{2\,EI}$$

Therefore $\qquad \theta_{\text{yield}} = \dfrac{V_y L^2}{2\,EI}$ and $V_y = \dfrac{2\,EI}{L^2}\,\theta_y$

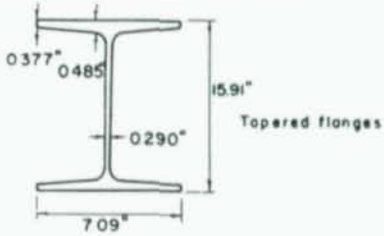
But $\qquad V_y = \dfrac{3\,EI}{L^2}\,\delta_y$

Therefore $\qquad \theta_y = \dfrac{3\,EI}{L^2}\,\delta_y \cdot \dfrac{L^2}{2\,EI} = \dfrac{3\,\delta_y}{2L}$

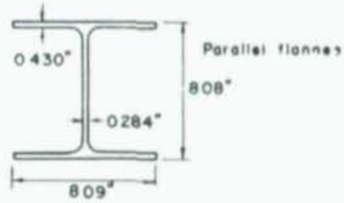
$$= 0.31\,\delta_y \text{ radians}$$

2.3 MATERIAL DIMENSIONS AND PROPERTIES — average values

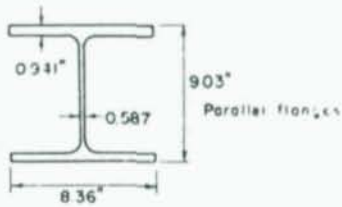
16 WF 36
beam



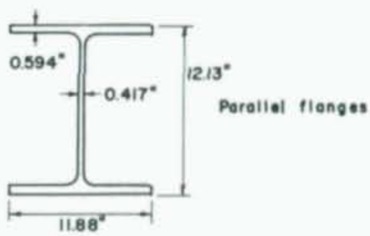
8 WF 31 (A-1)
column (B-6)
(C-9)



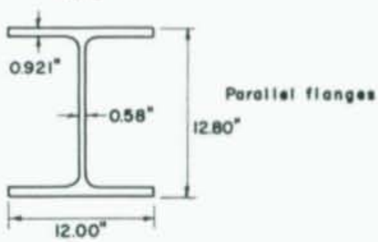
8 WF 67 (A-2)
column



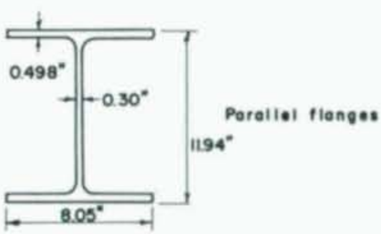
12 WF 65 (A-4)
Column



12 WF 99 (A-5)
Column

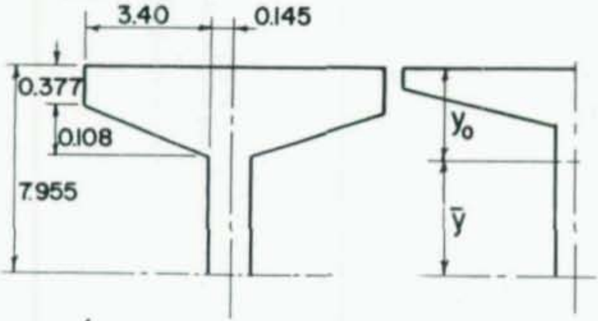


12 WF 40 (B-8)
Column (C-11)



2.4 Shape Properties

16 WF 36 Beams:



$$\frac{A}{4} = (3.4)\,(0.377) = 1.283$$
$$(0.108)\,(3.4)/2 = 0.184$$
$$(0.145)\,(7.96) = \underline{1.155}$$
$$2.622$$
$$A = 10.488\,\square''$$

$$\frac{I_{xx}}{4} = (1.283)\,(7.766)^2 = 77.40$$
$$(0.184)\,(7.542)^2 = 10.46$$
$$(0.145)\,(7.955)^3/12 = 6.11$$
$$(1.155)\,(3.978)^2 = \underline{18.27}$$
$$112.24$$
$$I_{xx} = 448.96 \text{ in.}^4$$

$$S = \frac{I_{xx}}{c} = \frac{448.96}{7.955} = 56.4 \text{ in.}^3$$

$$\frac{Z}{4} = (1.283)\,(7.766) = 9.96$$
$$(0.184)\,(7.542) = 1.39$$
$$(1.155)\,(3.978) = \underline{4.59}$$
$$15.94$$
$$Z = 63.76 \text{ in.}^3$$

$$\bar{y} = \frac{15.94}{2.622} = 6.08''$$

$$y_o = 7.96'' - 6.08'' = 1.88''$$

8 WF 31 Column: $A = (2)\,(0.430)\,(8.09) = 6.96$
$(7.22)\,(0.284) = \underline{2.05}$
$9.01\,\square''$

8 WF 67 Column: $A = (2)\,(0.941)\,(8.36) = 15.75$
$(7.148)\,(0.587) = \underline{4.19}$
$19.94\,\square''$

12 WF 65 Column: $A = (2)\,(0.594)\,(11.88) = 14.10$
$(10.942)\,(0.417) = \underline{4.56}$
$18.66\,\square''$

12 WF 99 Column: $A = (2)\,(0.594)\,(11.88) = 22.10$
$(10.958)\,(0.58) = \underline{6.35}$
$28.45\,\square''$

12 WF 40 Column: $A = (2)\,(0.498)\,(8.05) = 8.03$
$(10.944)\,(0.300) = \underline{3.28}$
$11.31\,\square''$

3. Four-way Tests

3.1 Summary of Coupon Tests

Shape	Mark	E ksi	σ_{yU} ksi	σ_{yL} ksi	$\sigma_{y\,st}$ ksi	$\sigma_{ult.}$ ksi	ε_{st} in./in.
¼" Plate	233/P	30,900	40.65	39.87	—	62.00	0.01725
		25,100	41.02	39.74	—	61.54	0.01775
12 WF 65	233/W1	30,400	42.57	41.81	—	67.74	0.015
	233/W1	29,700	38.37	38.54	—	67.74	0.00675
	233/F1	30,100	40.63	40.07	—	65.57	0.01875
	233/F1	30,600	44.28	40.46	—	64.86	0.200
12 WF 40	233/W2	31,200	47.17	44.16	39.85	68.93	0.021
	233/W2	30,700	50.00	48.86	43.60	70.87	—
	233/F2	31,300	43.47	41.77	37.86	68.00	0.0175
	233/F2	29,400	42.90	41.51	37.67	68.45	0.01875
16 WF 36	233/W3	29,500	50.58	48.95	—	63.63	—
	233/W3	30,600	47.00	45.66	—	61.64	—
	233/F3	30,400	41.86	40.25	—	61.18	0.0185
	233/F3	30,200	40.58	38.98	—	59.99	0.0215
12 WF 27	233/W4	31,200	43.70	43.70	38.81	61.62	—
	233/W4	31,100	45.14	41.89	37.83	61.02	—
	233/F4	31,100	40.36	38.65	34.74	61.24	0.0175
	233/F4	29,800	39.36	38.17	33.79	60.03	0.02075

W — Web.
F — Flange.

3.2 Calculations for Design of Specimens

Columns:

As in Section 2.2 of Appendix —
Column Working Stress = 14.5 ksi

Structural Shape Details:

Test	Column	Area*	Area as measured	P_w	$1.65\,P_w$	$2\,P_w$
AA	12 WF 65	19.11	19.00	276	455	552
BB	12 WF 40	11.77	11.70	170	280	340
DD	12 WF 40	11.77	11.49	167	276	334

* AISC Manual value.

Analysis of Beams and Beam-to-column Flange Welds:

All dimensions of sections as measured on specimens

$\sigma_w = 20$ ksi

Bending:

$$M_w = \sigma_w S = V_w L \qquad V_w = \frac{\sigma_w S}{L}$$

$$V_y = \frac{\sigma_y S}{L}$$

$$V_u = \frac{\sigma_y Z}{L} \qquad Z = \text{plastic modulus}$$

The calculations are similar to those in Section 2 of this Appendix. Lateral buckling, local buckling, shear, deflections and beam rotations were investigated and calculations are similar to those found in Section 2.

Analysis of Welds for Specimen BB
12 WF 27 Beams:

Use working load and allowable working stresses for the design of welds, seat, stiffener, etc. . . . .

$V_w = 19$ kips $\qquad M_w = 19 \times 36 = 684$ in-kips

$$T = C = \frac{684}{11.95} = 57.2 \text{ kips}$$

AISC Specification Section (26h):

$$\frac{R}{t\,(n+k)} = 24 \text{ ksi}$$

$$19 = 24 \times 0.24 \times (n + 0.813)$$

$$n = \frac{19 - 4.68}{5.75} = 2.5 \text{ inches required bearing length}$$

From Table 25 in the AISC Structural Shop Drafting Textbook, Vol. 2, the choice is:

4″ wide seat; ¼″ fillet welds; $L = 7''$;
Plate thickness ½″

Top Plate Weld Design:

$$\text{Required plate thickness} = \frac{57.2}{20 \times 9.75} = 0.3''$$

Use ½″ Plate

The length of weld available is
$9.75 + 2 \times 3.75 = 17.25''$

Using butt welds on the plate, the full strength of the plate can be developed.

Weld Connecting Top Plate to Beam Flanges:

Fillet welds of ¼″ size can be applied to toe of 12 WF 27 flange, and ½″ fillet welds can be applied on edge of top plate.

Working stress for ½″ fillet is 4.8 kips/in.

Working stress for ¼″ fillet is 2.4 kips/in.

Length of weld available = 6″ overhead fillet.
6.5″ fillet on top of flange.

$$\text{Safe load} = 6.5 \times 4.8 + 12 \times 2.4 = 60 \text{ kips} > 57.2 \text{ kips}$$

(Actually used 12.5 inches of ⅜″ fillet.)

Check on Tee Seat:

From Lawson's chart on Page 123 of the Airco "Manual of Design for Arc Welded Steel Structures."(8)

$$R = \frac{23.04\ DL^2}{\sqrt{L^2 + 16\,e^2}}$$

where $D = \frac{3}{16}''$ $\quad L = 8''$ $\quad e = 2.75''$

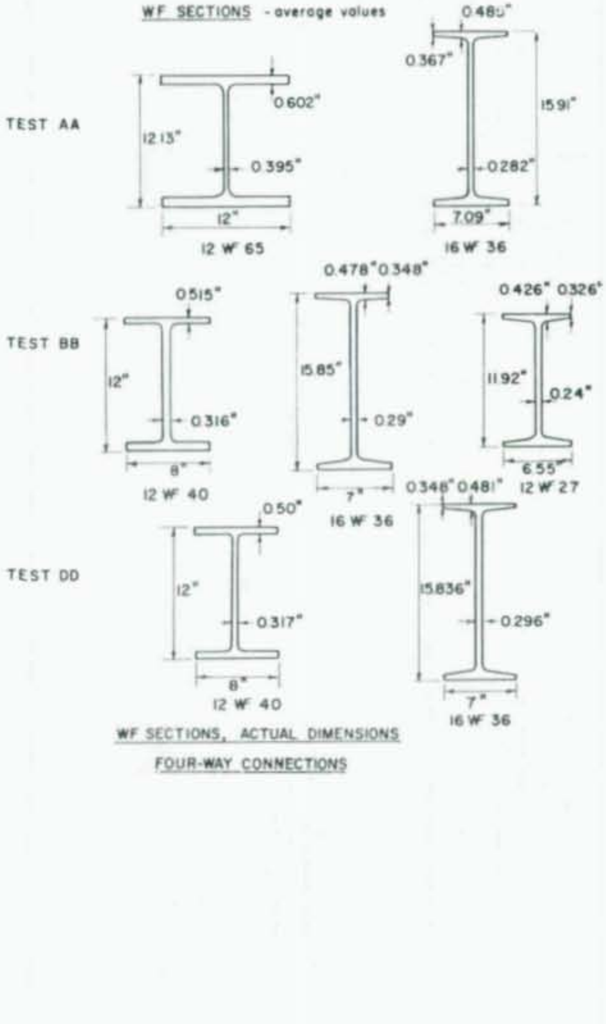
$$R = \frac{23.04 \times \frac{3}{16} \times 64}{\sqrt{64 + 16\,(2.75)^2}} = 20.3 \text{ kips} > 19 \qquad \text{(O.K.)}$$

3.3 Material Dimensions and Properties

In the figure below the average values of all the dimensions of the WF sections used in the tests are shown. The calculations of the section properties are similar to that presented in Section 2 of the Appendix. In the Table below the different section properties are shown:

SECTION PROPERTIES

Test	Beam	Area	Section Modulus	Plastic Modulus
AA	16 WF 36	10.28	55.59	62.73
BB	16 WF 36	10.29	54.20	61.52
	12 WF 27	7.83	32.60	36.56
DD	16 WF 36	10.24	54.06	61.37



WF SECTIONS, ACTUAL DIMENSIONS

FOUR-WAY CONNECTIONS

Bibliography

1. A. N. Sherbourne, C. D. Jensen—*"Direct Welded Beam Column Connections"*—Fritz Laboratory Report No. 233.12, Lehigh University, 1957.
2. R. N. Khabbaz, C. D. Jensen—*"Four-way Welded Interior Beam Column Connections"*—Fritz Laboratory Report No. 233.13, Lehigh University, 1957.
3. L. S. Beedle, B. Thürlimann and R. L. Ketter—*"Plastic Design in Structural Steel"*—Lehigh University, Bethlehem, Pa., and American Institute of Steel Construction, New York, 1955.
4. G. C. Driscoll, Jr.—*"Rotation Capacity Requirements for Beams and Frames of Structural Steel"* (Dissertation), Lehigh University, 1958.
5. E. W. Parkes—*"The Stress Distribution Near a Loading Point in a Uniform Flanged Beam"—Phil. Trans. of the Royal Society*, Vol. 244A, 1952.
6. G. Haaijer—*"Plate Buckling in the Strain Hardening Range"*—Proc. ASCE, 83 (EM-2) p. 1581, April 1958.
7. R. H. Wood—*"Yield Line Theory"*—Research Paper No. 22, Building Research Station, 1955.
8. L. Grover—*"Manual of Design for Arc Welded Steel Structures"*, Air Reduction Co., New York, 1946.

Acknowledgments

The entire program was carried out at the Fritz Engineering Laboratory of Lehigh University, of which Professor W. J. Eney is Director, with funds supplied by the American Institute of Steel Construction. The authors are indebted to members of the Research Committee on Welded Interior Beam-to-Column Connections who gave invaluable advice, technical and otherwise, in the organization and execution of this project. This committee included Messrs. F. H. Dill, Chairman, L. S. Beedle, Edward R. Estes, Jr., T. R. Higgins, C. L. Kreidler and H. W. Lawson with Mr. Jonathan Jones acting as an Advisor.

The authors also wish to acknowledge the assistance given them by members of the research staff of the Fritz Laboratory, particularly to Dr. L. S. Beedle, Chairman of the Structural Metals Division, to Dr. Bruno Thürlimann and Dr. G. C. Driscoll, Members of the Structural Metals Division, to Mr. I. J. Taylor, Mr. O. Darlington and Mr. R. Clark who were responsible for the instrumentation, to Mr. K. Harpel and his assistants who helped organize the actual tests, and to all those members responsible for the preparation of this manuscript.

The helpful advice and guidance given by members of the Steel Structures Committee of the American Welding Society is greatly appreciated.

000022

Pub. No. T105 (2M-5/90)