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J. D. Graham

A. N. Sherbourne

R. N. Khabbaz

C. D. Jensen

WELDED INTERIOR BEAM 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

This project has been sponsored by the American Institute of Steel Construction.

Fritz Engineering Laboratory

LEHIGH UNIVERSITY

Bethlehem, Pennsylvania.

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TABLE OF CONTENTS

WELDED INTERIOR BEAM COLUMN CONNECTIONS

TABLE OF CONTENTS (Cont'd)

TABLE OF CONTENTS (Cont'd)

SYMBOLS

w	Column Web Thickness
k	Column 'k' Distance
t_c	Column Flange Thickness
d_c	Column Depth
m	Distance between Fillet Extremeties of Column
b	Beam Flange Width
t	Beam Flange Thickness
t'	Beam Web Thickness
d	Beam Depth
A_b	Beam Cross Sectional Area
s	Stiffener Thickness
b'	Stiffener Width
M_p	Theoretical Maximum or 'Plastic' Moment of the Beam
M_y	Moment to Cause First Yielding of the Beam
M_w	Beam Working Moment
V_u	Load (also the shear) on the Test Beam to Produce M_p
V_y	Load (also the shear) on the Test Beam to Produce M_y
V_w	Load (also the shear) on the Test Beam to Produce M_w
P_w	Column Working Load
d_y	Deflection of Loading Point of Beam at V_y
S	Elastic Section Modulus
Z	Plastic Section Modulus

SYMBOLS (Cont'd)

E	Young's Modulus of Steel
I	Beam Moment of Inertia
ϕ	Rotation
ϕ_p	Beam Rotation at M_p
L	Beam Span
H	Hinge Angle
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.
σ_{cr}	Buckling Stress in Plate
ν	Poisson's Ratio
σ_a	Average Plate Stress at Test Ultimate in Tension Tests
σ_y	Yield Stress
σ_{pL}	Stress at Proportional Limit
σ_{uy}	Upper yield Stress
σ_{Ly}	Lower Yield Stress
$\sigma_{st.y}$	Static Yield Stress
σ_{ult}	Ultimate Stress
$\epsilon_{st.}$	Strain at Strain Hardening
d_w	Distance Between Flanges
F_w	Force on Weld

SYNOPSIS

Previous research on beam-column connections has not been carried to the point where definite conclusions, suitable for the designer, could 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 a beam-column connection 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 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 column connections, again by detailed tests copying practical conditions. The design rules stemming from these investigations apply to those connections in which -

(1) The beams and columns are members of the wide flange series listed in the A.I.S.C. manual.

(2) The beams are connected to both column flanges and may or may not be connected to both sides of the column web such that equal moments are applied on opposite sides of the column.

(3) The 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:

(A) Column stiffeners are not needed adjacent to the beam compression flanges if

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

(B) Column stiffeners are not needed adjacent to the beam tension flanges if

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

When stiffeners are required their minimum thicknesses are given by

(C) In the case of horizontal plate stiffeners

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

and, as a further limitation,

$$s \geq \frac{b'}{16}$$

(D) In the case of horizontal plate stiffeners eccentric by 2" or less,

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

where, again,

$$s \geq \frac{b'}{16}$$

(E) In the case of vertical plate stiffeners,

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

and, as a further limitation,

$$s \geq \frac{d_c}{30}$$

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. 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 be much used in practice.

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.

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. column stiffening requirements, although other aspects of the problem merited consideration. As previously mentioned, 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 8WF31 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 8WF67 and 12WF40 and 65 columns on the basis of beam and column loads being of the same order of magnitude. The third size was a 12WF99 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 (16WF36) has dimensions that ensure the development of M_p without local buckling of either the flange or the web.

TABLE 1.

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

Test No.	Column			Beam			Stiffening	Stiffener Dimension
	Shape	Web*	Flange*	Shape	Web*	Flange*		
A-1	8WF31	0.288	0.433	16WF36	0.299	0.428	None	None
A-2	8WF67	0.575	0.933	"	"	"	"	"
A-4	12WF65	0.390	0.606	"	"	"	"	"
A-5	12WF99	0.580	0.921	"	"	"	"	"
B-6	8WF31	0.288	0.433	"	"	"	Horiz. plate stiff-eners, at level of tension and compress-ion flanges	3.9"x7/16"
B-8	12WF40	0.294	0.516	"	"	"		3.9"x1/4"
C-9	8WF31	0.288	0.433	"	"	"	Vertical plate stiff-eners at edges of col. flanges	5/16"x22"
C-11	12WF40	0.294	0.516	"	"	"		5/16"x22"
D-12	12WF40	0.294	0.516	"	"	"	Split tee stiffener	ST6WF32.5 22" long
H-1	8WF31	0.288	0.433	"	"	"	Doubler plate	5/16"x20"

* Indicates A.I.S.C. Handbook Value

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 16WF36 beam stubs, 4'-6" long, welded directly to the flanges of the WF column sections as shown in Figures 1 and 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 Riehle 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).

During fabrication much care was taken with the welding. All welding was done by qualified welders using 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 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

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* "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.

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 8WF31 column to the heavier 12WF99. Connection A-1 with the 8WF31 column failed by column web buckling (See Figure 7) at a load slightly above the beam working load, namely $1.12V_w$. Connection A-4, with a thicker web showed much straining, both tension and compression, in the column webs opposite the beam flanges and failure occured by column web buckling at a beam load of 44 kips, which is $1.82V_w$. 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 occured at $2.08V_w$ and $2.26V_w$ 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 edges 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 approximately $2.17V_w$. 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 $1.97V_w$. In C-11 weld failure occurred just after this in the tension flange butt welds. The tear occurred at one end of the tension flange butt weld owing either to failure of the welder to weld out completely onto the run-out pad, to stress concentrations caused by the stiffener, or to a lateral moment. In test C-9 the connection continued to carry load until at approximately $2.16V_w$ the south stiffener plate buckled. From this point the load fell off rapidly.

Series D

Only one test, D-12, was performed in this group, the connection being a modification of the C type using split beam stiffeners instead of plates as shown in Figure 3. The split beam stiffener, while devised principally for use in a four-way beam-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 $2.22V_w$. Although large deformations occurred in the beams, the connection appeared to remain elastic and little strain was observed in the flange of the 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 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 beams and the column. The failure occurred at a beam load of 49.6 kips which is $2.05V_w$, 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.

The A-series of tests showed high stress concentrations at the center of the beam tension flanges, a condition which becomes more aggravated at values above working load. 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 $1.5V_W$ 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 $1.5V_W$.

The results in Figures 6, 8 and 10 show that the columns of the A and the H series, with no column flange stiffening, are not as stiff against rotation as are the 16WF36 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 $2\frac{1}{2}$ to 3 times as much as the plate stiffeners at beam working load.

2. Four-way Connection Tests.

This program consisted of three specimens with details as shown in Table 2 and Figures 14 and 15. Test AA is similar to Test A-4 of the "Two-way" series except for two additional 16WF36 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 16WF36 as before and were direct-welded, but the other pair of beams were 12WF27, the tension flanges of which were welded to horizontally placed column plate stiffeners, the compression flanges resting on a tee-type seat which also acted as a column stiffener (but 4" away from its ideal location as a stiffener).

TABLE 2

PROGRAM OF FOUR-WAY CONNECTION TESTS

Test	Column			Beam			Stiffener	
	Size	Web, w	Flange, t_c	Size	Web, t'	Flange, t	Type	Size
AA	12WF65	0.39	0.606	16WF36	0.299	0.428	None	None
BB	12WF40	0.294	0.516	16WF36 12WF27	0.299 0.240	0.428 0.400	Horiz. plates that served as top plate and as seat (plate)	½"thick
DD	12WF40	0.294	0.516	16WF36	0.299	0.428	Split tee stiffener	ST6WF32.5 22" long

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 the five million pound Baldwin Hamilton machine which provided ample space for placing these specimens and for the lateral supports, Figure 13 showing a test in progress. The test arrangement was similar to that for the Two-way tests, Figure 14 showing the test arrangement 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.

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 while the other connection, with only partial stiffening provided, showed considerable flexibility (See Figure 20). Local buckling of the beam flanges was observed at a load of 53 kips ($2.28V_w$) 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 indicating that the column suffered considerable yielding, but there was no other evidence of failure in the column.

Test DD.

The connection involving the beams welded directly to the column flanges proved stiffer than the other one (See Figure 20). The stiffness of the other connection, that is the one welded to the split tee stiffeners, 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. On the other hand, the column web is ably assisted in preventing rotation at the connection by the flanges of the split tee stiffeners. The two beams that were connected to the stiffeners had very good load and rotation capacities, but the east and west beams that were connected to the column flanges just reached the required ultimate load and showed a lesser rotation capacity caused by a butt weld failure starting at a load of 49 kips ($2.18V_w$). 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.

Test BB.

The connection involving the 16WF36 beams, welded directly to the column flanges, proved to be relatively stiff. The connection involving the 12WF27 beams framing to the seats and top plates was considerably more flexible than an equivalent 12WF27; however this flexibility did not prevent the connection from fully meeting the established criteria for a satisfactory connection.

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 was in compression against the column and consisted of a stub 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 7/16", simulating the flange of the 16WF36 beam used in all the two-way tests. The bars were tack welded to the flanges at the mid-length of the stub columns, which were approximately 3'-0" long. The specimen was then tested in the 300 kip Baldwin testing machine with the simulated column in a horizontal position (See Figure 21).

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

TABLE 3.

PROGRAM OF COMPRESSION CRITERION TESTS

Test No.	Column			Bar		Simulated Beam	Failure Load (kips)
	Shape	Web*	Flange*	Width	Thickness		
E-14	8WF48	0.405	0.683	7"	1/2"	16WF36	137
E-15	8WF58	0.510	0.808	"	"	"	202.5
E-16	10WF66	0.457	0.748	"	"	"	175.7
E-17	10WF72	0.510	0.808	"	"	"	190
E-1	12WF40	0.294	0.516	"	"	"	102.5
E-18	12WF65	0.390	0.606	"	"	"	143
E-19	12WF85	0.495	0.796	"	"	"	247.5
E-20	14WF61	0.378	0.643	"	"	"	137.5
E-21	14WF68	0.418	0.718	"	"	"	164
E-22	14WF84	0.451	0.778	"	"	"	221
E-23	14WF103	0.495	0.813	"	"	"	250

* Indicates A.I.S.C. Handbook 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 24 shows E-16 and E-18 at failure.

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 the 800k Riehle 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 stub column of length about 3'-0" as shown in Figure 22 and the specimen then lined up in the testing machine with the stub column horizontal.

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 24 shows F-4 and F-15 at failure.

TABLE 4.

PROGRAM OF TENSION CRITERION TESTS

Test No.	Shape	Column		Plate		Failure load (kips)	Method of Failure
		Web*	Flange *	Width	Thick-ness		
F-1	8WF31	0.288	0.433	7"	3/4"	100	Crack in column fillet
F-2	8WF31	0.288	0.433	7"	7/16"	95	Crack in center of weld.
F-3	12WF65	0.390	0.606	8½"	5/8"	149	"
F-4	14WF68	0.418	0.718	8½"	5/8"	167	"
F-5	14WF84	0.451	0.778	11½"	7/8"	212	"
F-9	12WF65+	0.390	0.606	8½"	5/8"	82	Crack in column fillet
F-10	14WF84++	0.451	0.778	11½"	7/8"	125	Crack in center of weld.
F-12	12WF65	0.390	0.606	8½"	1½"	189	"
F-13	14WF68	0.418	0.718	8½"	1½"	199	"
F-14	8WF67	0.575	0.933	7"	3/4"	256	Crack at outside of weld.
F-15	14WF176	0.820	1.313	11½"	7/8"	444	"

* Indicates A.I.S.C. Handbook Value

\+ Column Flange Machined to 5/16"

++ Column Flange Machined to 3/8"

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 stiffening is questionable. To determine the degree of such stiffening a series of tests were carried out on 12WF40 and 14WF61 column stubs approximately 4'-0" long. The columns were compressed between bars for cases of 0, 2", 4" and 6" eccentricity as shown in Figure 23 by means of the 300k Baldwin machine, the tests being similar to the compression criterion tests in Part 3.1. Included in the tests on the 12WF40 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.

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.

TABLE 5.

PROGRAM OF TESTS WITH ECCENTRIC STIFFENERS

Test	Stub Column	Stiffener	Eccentricity (in)	Failure Load (kip)
E-0	12WF40	10.75"x3.75"x1/4"	0	172
E-2	"	"	2	146
E-3	"	"	4	113
E-3a	"	10.75"x3.75"x1/4" +8"x3"x1/4" Tee	4	116
E-4	"	10.75"x3.75"x1/4"	6	104
E-1	"	none	*	102.5
E-9	14WF61	12.5"x4.25"x3/8"	0	282
E-6	"	"	2	232.5
E-7	"	"	4	167.6
E-8	"	"	6	142.8
E-24	"	none	*	137.5

* i.e., no stiffening used.

PART B

DISCUSSION OF TEST RESULTS

1. Connection Requirements

In a beam 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 maintain its moment capacity for a considerable rotation at the ultimate load. The rotations required at plastic hinges (namely, the "hinge angle") for a variety of practical structures have 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 properties of sections that will buckle just at the onset of strain hardening. The width to thickness ratio of the beam flange, b/t, must not exceed 17, and the ratio, d/t', (beam depth to web thickness) must not exceed 55. The beam section chosen (16WF36) was just within these values in both these respects, with the result that local buckling, as predicted by Haaijer, 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 16WF36 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 connection can be ascertained in part by comparing the moment and rotation capacity of the beams with that for the column with the consideration that the column must have equal or greater moment capacity than the beam but it need not necessarily be as stiff. When the column has the requisite strength the desired rotation capacity is supplied jointly by the column and the end portions of the beams. 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 observation of strain gage readings it can be calculated that the vertical plate stiffeners of Series C in the elastic range, each transmitted only about 3/16ths of the forces coming from the beam flanges and the web transmitted 5/8ths. However, since the prime purpose of this type of connection is to afford a convenient four-way connection, the plate 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 passed 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 18, was stronger than its two-way counterpart, Test A-4. This evidently shows that the stiffening action provided by the two beams framing 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 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 x working load* except that specimen BB showed straining in the web of the 12WF40 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 in so doing 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.

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* working load corresponds to an average axial stress of 14.5 k.s.i.

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 12WF65 stub column failed at a simulated beam flange load of 143 kips, whereas test A-4 in which the 12WF65 section was used in an actual connection failed at a computed load of 110 kips from the beam flange together with a computed beam web load of 40 kips.

Test BB showed much straining in the web of the 12WF40 column at a beam flange load of 110 k whereas the simulated test with no beam web force failed at a simulated beam flange force of 116 k (See Test E-3a, Table 5).

5.2 <u>Series F - Tests to Determine Tension Criterion</u>

The simulated tension side 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 k while test F-2, a simple tension test suffered the same failure at 95 k. 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 <u>Eccentric Stiffener Tests</u>.

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 12WF40 and 14WF61 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".

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.

(2) The strength of that region of the connection adjacent to the beam tension flange.

(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 flange.

(6) The rotation required of connections and their capacity to rotate.

Items (1), (2) and (3) will be discussed in Parts 1.1 and 1.2

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 will 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 20 which show moment rotation curves of tested connections. Inspection does show 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 then the necessary rotation will occur in the end of the beam.

1.1 Analysis of Compression Region of Connection

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 Figure 27) 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 $\sigma_y\, w(t+5k)$.

This implies that, as shown in Figure 25, 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 with no stiffeners

$$Q_c = \sigma_y\, w\,(t+5k) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

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

$$bt\sigma_y = \sigma_y\, w\,(t+5k) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

or

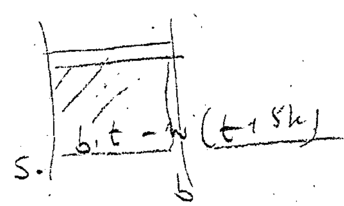
$$w = \frac{bt}{t+5k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

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

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

In the case of horizontal plate stiffeners A_{st} may be approximated as

$$\boxed{A_{st} = sb}$$



Hence,

$$s = \frac{bt - w\,(t+5k)}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

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

$$s \geq \frac{b'}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

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,

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

so that

$$s = \frac{bt}{t + 5k} - w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

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

$$s \geq \frac{d_c}{30} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

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, any results derived from this testing concerning their action cannot be very conclusive; however, since very light columns were used, then these results should if anything be conservative.

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

$$bt\sigma_y = \sigma_y\ w\ (t + 5k) + \sigma_y . 0.6\ sb$$

which reduces to

$$s = \frac{1.7}{b}\left[bt - w\ (t + 5k)\right] \qquad \ldots\ldots\ldots\ldots\ldots (9)$$

where again

$$s \geq \frac{b'}{16} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Two other methods of analysis of the compression region of the connection have been suggested in the Appendix but the above, the Modified A.I.S.C. approach, is advocated for use.

1.2 <u>Analysis of Tension Region of Connection</u>

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 of this "plate" for wide flange columns is $3.5\sigma_y t_c^2$. The central rigid part of length 'm' adjacent to the column web will be highly strained and hence will carry a force corresponding to its area at yield stress. Hence

$$Q_t = \sigma_y tm + 7\sigma_y t_c^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(10)$$

The force in the beam tension flange when plastic moment is on the beam is $bt\sigma_y$. To give 20% conservatism in this region of the connection to correspond approximately with the average conservatism in the compression region we have

$$bt\sigma_y = 0.8\left[\sigma_y tm + 7\sigma_y t_c^2\right] \quad \ldots\ldots\ldots\ldots(11)$$

This reduces to

$$t_c^2 = \frac{bt}{7}\left[1.25 - \frac{m}{b}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(12)$$

t_c being the required column flange thickness.

If beam and column sizes are taken from the A.I.S.C. handbook then the value of $\frac{m}{b}$ for all those connections in which formula (12) is approximately satisfied varies from 0.15 to 0.20. Making the conservative assumption $\frac{m}{b} = 0.15$ (12) reduces to

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

In cases where $t_c < 0.4\sqrt{bt}$ and stiffeners are required we have 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). While (6) and (8) apply only to compression members it is recommended that, as a practical measure, they also be used in the tension region of the connection.

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

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

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.,

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

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

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

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

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

Figure 28 shows a plot of the values of $\frac{t_c}{\sqrt{wk}}$ for all 8", 10", 12" and 14" deep 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.

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

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

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 7.

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.

TABLE 6.

COMPARISON OF MODIFIED A.I.S.C. COMPRESSION REGION CRITERION WITH CONNECTION TEST RESULTS.

Specimen	bt	k	Req'd w	Handbook w	Measured w	Req'd s	Actual s	Remarks
	in^2	in	in	in	in	in	in	
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	0.25*	0.437	Stiffened connections satisfactory
B-8	2.99	1.125		0.294	0.300	0.25*	0.250	
C-9	2.99	0.812		0.288	0.284	0.382	0.437	Connections O.K. but some stiffener buckling
C-11	2.99	1.125		0.294	0.300	0.34**	0.250	
D-12	2.99	1.125		0.294		0.39**	0.606	Connection O.K.
H-1	2.99	0.812	0.666		0.600			Column web O.K. up to $0.95M_p$ when failure occurred in tension region of connection.
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	Seat 4" above *** compression flange connection O.K.
DD	2.91	1.125		0.294	0.317	0.34**	0.6	Connection O.K.

Notes:

* Determined by slenderness limitation, Equation (6)

** Determined by slenderness limitation, Equation (8)

*** Stiffening also included a plate perpendicular to the seat - See Figure (15).

TABLE 7

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

Test	Column	Bar Thickness	Column Web Yield, σ_y	w	k	Computed Q_c	Test Q_c
		in	ksi	in	in	kip	kip
E-1	12WF40	½	40.2	0.294	1.125	99	102.5
E-14	8WF48	½	34.4	0.405	1.063	110.1	137
E-15	8WF58	½	36.2	0.510	1.188	162.6	202.5
E-16	10WF66	½	40.0	0.457	1.25	169.0	175.7
E-17	10WF72	½	35.0	0.510	1.313	173	190
E-18	12WF65	½	37.2	0.390	1.188	129	143
E-19	12WF85	½	37.8	0.495	1.375	190	247.5
E-20	14WF61	½	36.2	0.378	1.25	127	137.5
E-21	14WF68	½	38.3	0.418	1.313	155	164
E-22	14WF84	½	39.3	0.451	1.375	180	221
E-23	14WF104	½	38.5	0.495	1.438	201	250

The theoretical restraint provided by horizontal stiffeners in a connection is given by σ_y bs (refer to Formula (4)).

Comparison with tests show -

(a) Test E-1 in which an unstiffened 12WF40 was compressed failed at 103 k whereas test E-0 in which the same column was stiffened with two $\frac{1}{4}$" horizontal stiffeners failed at 172 k. The difference of 69 k compares favourably with the calculated difference of 63 k .

(b) A similar examination of tests E-9 and E-20 on a 14WF61 show an experimentally determined difference of 144 k compared to the calculated difference of 115 k .

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

$$w = \frac{bt + 3.5kt'}{t + 7k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

is consistent when applied to the connection tests and to the simplified tests. In the simplified tests of course $t' = 0$. However formulas (3) and (24) give very close results when applied to practical connections, it being preferable to use (3) since it is 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 0.43" 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).

TABLE 8

COMPARISON OF TENSION REGION ANALYSIS WITH TENSION TESTS

Test No.	Column Stub	Yield Stress*		Ultimate Capacity, Q_t		Av. Plate Stress at Test Ult., σ_a	Flange Thickness, t_c		$\frac{\sigma_a}{\sigma_y}$
		Column Flange	Plate σ_y	Computed from (10)**	Test		Computed from (13)**	Actual	
F-1	8WF31	37.0	38.9	81	100	19	0.94	0.43	0.49
F-2	8WF31	37.0	38.9	68	95	31	0.72	0.43	0.80
F-3	12WF65	36.0	31.6	123	149	28	0.86	0.61	0.89
F-4	14WF68	34.2	31.6	155	167	32	0.89	0.72	1.01
F-5	14WF84	34.2	31.9	191	212	21	1.27	0.78	0.66
F-9	12WF65	36.0	31.6	55	82	15	0.86	0.31	0.47
F-10	14WF84	34.2	31.9	80	125	12	1.27	0.38	0.38
F-12	12WF65	36.0	31.8	167	189	15	1.35	0.61	0.47
F-13	14WF68	34.2	31.8	200	199	16	1.37	0.72	0.50
F-14	8WF67	33.5	38.9	242	256	45	0.99	0.93	1.16
F-15	14WF176	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.

3. <u>Limitations of This Investigation</u>

The investigation has considered two and four-way connections in which every beam of the connection has been loaded equally and gradually to failure. Detrimental effects could be caused by:

(a) <u>Repetitive Loading.</u> A sufficient number of cycles of loading and unloading could cause premature failure but this is unlikely since much of the load in a building is dead load and any variation would be of small magnitude.

(b) <u>Unequal Loading of Opposing Beams.</u> In this case shear stresses would be induced in the column web. However when the beam loadings are approximately the same as would be the case when beams of the same size framed into the column flanges it would be expected that the above design formulas would be valid. They would probably not be valid in the extreme case of a beam framed into only one column flange. This problem is one for further research.

(c) <u>Wind Loading.</u>This would tend to cause moments in the same direction and hence high shear stresses in the column web. As with (b) this is another problem for further research.

4. <u>Advocated Design Methods</u>.

There follow examples of connection design using the proposed formulas.

4.1 <u>Connection in Which no Stiffening is Required</u>.

Consider a two-way connection in which 16WF50 beams frame onto the flanges of a 12WF99 column. From formula (3)

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

$= 0.546''$

Actual $w = 0.580''$

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

From formula (13)

$$\text{required} \quad t_c = 0.4 \sqrt{bt}$$
$$= 0.842''$$
$$\text{Actual} \quad t_c = 0.921''$$

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 12WF99 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 16WF58 beams frame onto the flanges of a 10WF89 column.

From formula (3)

$$\text{required} \quad w = 0.792''$$
$$\text{But Actual} \quad w = 0.615''$$

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)

$$\text{required} \quad s = \frac{bt - w\,(t + 5k)}{b}$$
$$= 0.144''$$

But from equation (6)

$$s \geq \frac{b'}{16}$$

$$\geq 0.25''$$

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

From formula (13)

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

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

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 18WF105 beams frame onto the flanges of a 12WF65 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

$$s = 0.487''$$

which satisfies equation (6).

Hence use $\frac{1}{2}$" horizontal plate stiffeners in both tension and compression regions of the connection.

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

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

$$= 0.636''$$

From Equation (8)

$$s \geq \frac{d_c}{30}$$

$$= 0.405''$$

Hence use 11/16" vertical plate stiffeners flush with the edges of the column flanges.

4.4 <u>Eccentric Stiffening.</u>

Consider the same connection as in Section 4.3 with, in addition, two 16WF36 beams framing into opposite sides of the web of the 12WF65 column. If the tension flanges of the beams are at the same level then the seating plates of the 16WF36 beams can be used as stiffeners of approximately 2" eccentricity for the 18WF105 beams.

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

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

$$= 0.828''$$

This satisfies equation (6)

Hence use 7/8" seating plates for the 16WF36 beams.

APPENDIX

1. THEORETICAL ANALYSIS

1.1 Limiting Slenderness of Stiffeners.

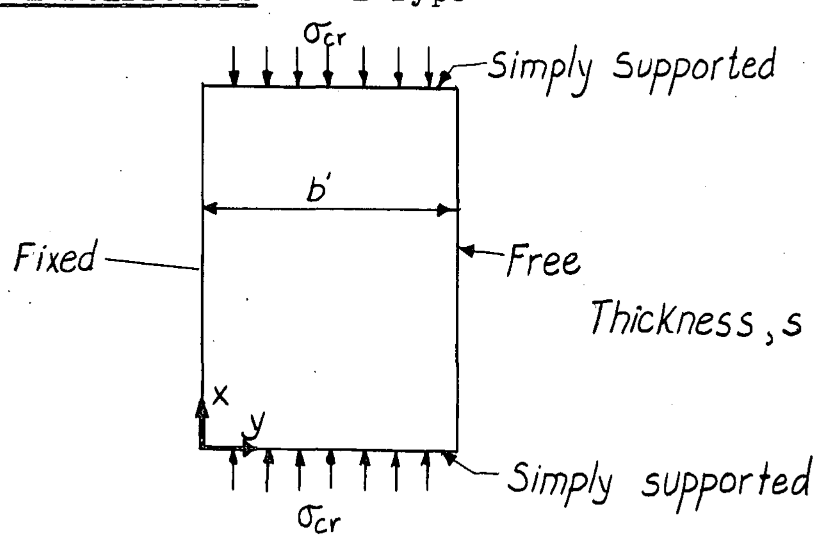
The slenderness limits for the stiffeners are difficult to establish because -

(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 Stiffeners - B Type



As shown in the Figure, consider the stiffener as 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

$$D_x = 8{,}000 \text{ ksi}$$
$$D_{xy} = 16{,}000 \text{ ksi}$$
$$D_y = 31{,}000 \text{ ksi}$$
$$G_t = 2{,}400 \text{ ksi} \qquad \text{(Ref. 6)}$$

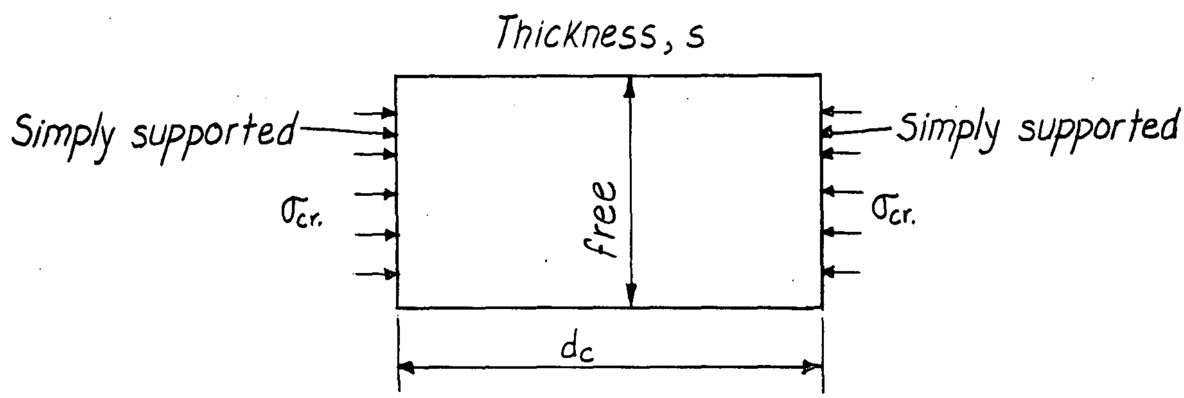
$$\sigma_{cr} = \left(\frac{s}{b'}\right)^2 \cdot 7820$$

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

$$\frac{b'}{s} = 15.5$$

To round figures $\frac{b'}{s} = 16$ ....(6)

<u>Vertical Stiffener</u> - C Type



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

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

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

$$\frac{d_c}{s} = 30 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(8)$$

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

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

or

$$H = \frac{M_p L}{6\ EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots(19)$$

Taking a practical case of a 16WF36 beam of span 24' 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 of 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 considered 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 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 27 represents to $\pm 5\%$ the elastic stress distribution along the column 'k' line for all wide flange shapes used in practice. The scale of Figure 27 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_o and σ_o 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.

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 27 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_o , as used by Parkes causes the ratio σ_y/σ_o to have values very close to 0.1 for all the specimens tested except the column section 12WF65 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 27 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 Modified A.I.S.C. 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 26. 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 26.

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

and $\quad Q_w = \sigma_y \, w \left[\frac{d}{2} + 3k\right]$

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

$$Q_b = Q_w$$

$$\text{or } \sigma_y \, w \left(\frac{d}{2} + 3k\right) = \frac{A_b}{2} \sigma_y \quad \ldots\ldots\ldots\ldots\ldots (20)$$

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

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

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

therefore $s = \frac{1}{2b}\left[A_b - (d + 6k)\right] w$ ....(22)

s is again subject to the limitation that $s \geq \frac{b'}{16}$ 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 edge 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

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

Hence $\frac{A_b}{2} \sigma_y = \sigma_y \, w \left(\frac{d}{2} + 3k\right) + \sigma_y \, s \, (t + 6k)$

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

The stiffener thickness is again restricted by the inequality, $s \geq \frac{d_c}{30}$.

1.52 Modified Plastic Analysis Approach

The preceding analysis assumes that at failure a length of $\left(\frac{d}{2} + 3k\right)$ of web is at yield stress (See Figure 26). 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 26 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.

$$bt\,\sigma_y + t' \cdot \frac{7k}{2}\,\sigma_y = w\,\sigma_y \quad (t+7k)$$

or

$$w = \frac{bt+3.5\ kt'}{t+7k} \quad \ldots\ldots\ldots\ldots(24)$$

By going through an analogous procedure as that in Section C we have the results

(b) Horizontal Plate Stiffeners.

$$s = \frac{1}{6}\left[bt+3.5\ kt' - w\ (t+7k)\right] \quad \ldots\ldots\ldots\ldots(25)$$

where s is again subject to the limitation that $s \geq \frac{b'}{16}$

(c) Vertical Plate Stiffeners.

$$s = \frac{bt+3.5\ kt'}{t+7k} - w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(26)$$

where $s \geq \frac{d_c}{30}$

Table 9 compares the results of these two methods with the Modified A.I.S.C. Approach for the connections tested.

TABLE 9

COMPARISON OF THE THREE COMPRESSION SIDE CRITERIA

Speci-men	Web Thickness, w				Stiffener Thickness, s				Remarks
	Mod. AISC	Plas-tic	Mod. Plas-tic	Act-ual	Mod. AISC	Plas-tic	Mod. Plas-tic	Act-ual	
A-1	0.666	0.504	0.624	0.284					Col.web buckled
A-2	0.428	0.440	0.450	0.587					Col.web O.K.
A-4	0.470	0.453	0.480	0.417					Col.web weak
A-5	0.378	0.420	0.412	0.580					Col.web O.K.
B-6					0.25*	0.326	0.297	0.437	Stiffened connections satisfactory
B-8					0.25*	0.261	0.25*	0.250	
C-9					0.382	0.429	0.340	0.437	Connections O.K. but some stiffener buckling
C-11					0.34**	0.39**	0.34**	0.250	
D-12					0.34**	0.34**	0.34**	0.606	Connection O.K.
H-1	0.666	0.504	0.624	0.600					Col. web O.K. up to 0.95M_p when failure occurred in tension region of connection
AA	0.474	0.445	0.479	0.395					Connection O.K.
BB					0.25*	0.25*	0.25*	0.5	Seat 4" above*** compression flange. Connection O.K.
DD					0.34**	0.34**	0.34**	0.6	Connection O.K.

Notes:

* Determined by slenderness limitations, Equation (6).

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

*** Stiffening also included a plate perpendicular to the seat - see Figure 15.

1.6 Analysis of Tension Region of Connection.

Figure $\bar{A}$ 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 are assumed to be $12t_c$ 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

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

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

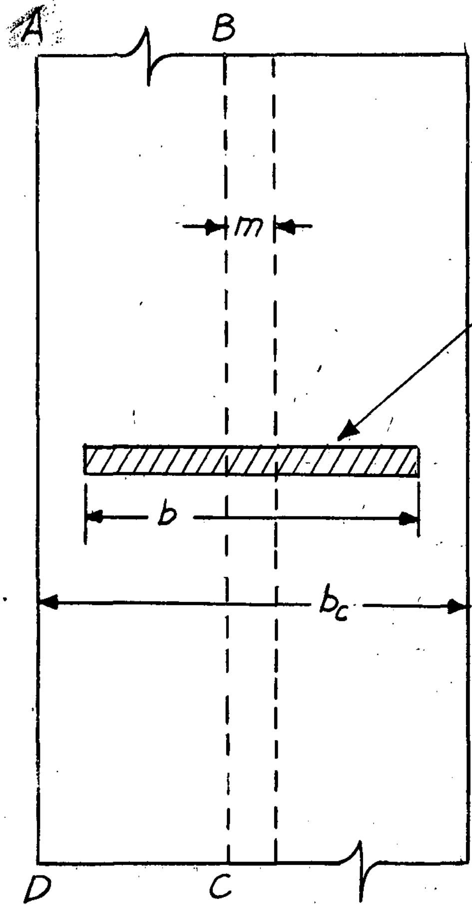
and $$\eta = \beta/4 \left[\sqrt{\beta^2 + 8\lambda} - \beta \right]$$

$\beta = p/q$ (refer to figure A)

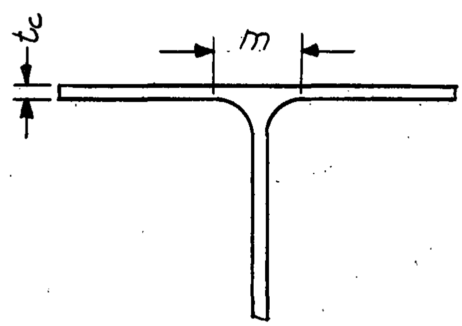
$\lambda = 1/q$ (refer to figure A)

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.

As a conservative approximation, take $c_1 = 3.5$.

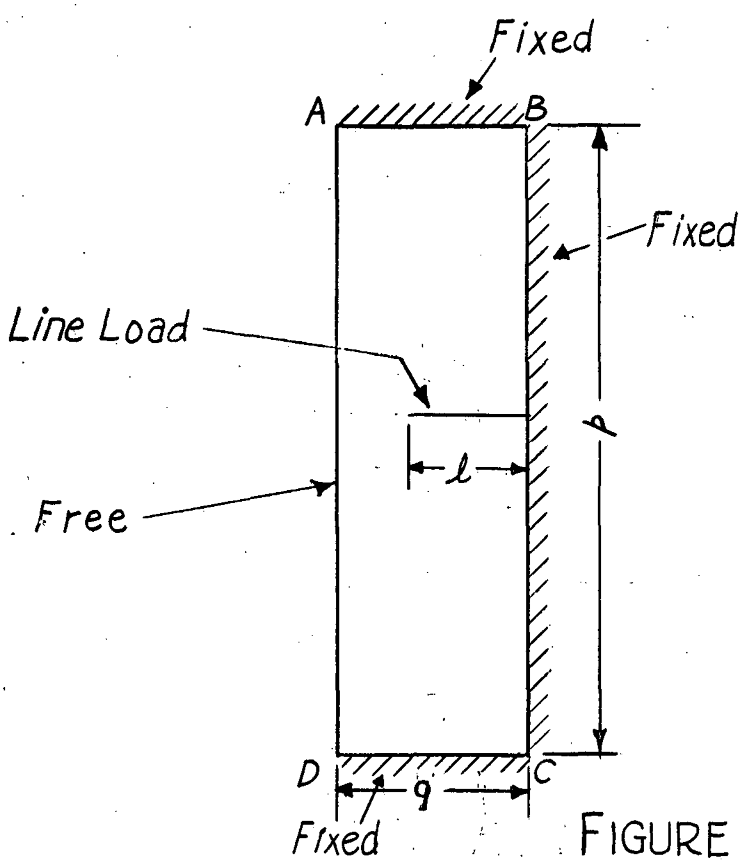


COLUMN FLANGE



'm' DISTANCE

$m = w + 2(k - t_c)$



$$q = \frac{b_c}{2} - \frac{m}{2}$$

$$\ell = \frac{b}{2} - \frac{m}{2}$$

$$p = 12\,t_c$$

CONSIDERED PLATE & ITS LOADING

FIGURE $\bar{A}$

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. APPENDIX TO TWO-WAY TESTS.

2.1 Summary of Coupon Tests.

Section	Mark	E ksi	σ_{pL} ksi	σ_{uy} ksi	σ_{yL} ksi	σ_{ult} ksi	ϵ_{st} in/in
7/16"plate	59E/8/3 t	30,000	-	35.6	34.8	59.2	1.5×10^{-2}
	59E/5/3 t	29,500	-	35.8	34.2	59.6	-
	59E/2/3 t	30,200	-	35.6	34.6	60.0	-
1/2" plate	68E/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	68E/6/1 c	29,800	24.1	32.8	-	-	-
	68E/6/2 c	30,600	26.7	33.6	-	-	-
12WF40	38G/1 tf		35.2	36.9	37.3	62.0	1.66×10^{-2}
	38G/2 tf		34.3	36.3	36.5	61.7	1.7
	38G/3 tw		42.8	44.0	42.8	65.4	2.02
	38G/4 tf		36.6	38.3	37.6	61.9	1.9
8WF31	54E31/1tf		34.7	39.4	37.8	63.4	1.72
	54E31/2tf		36.3	-	38.1	63.0	1.94
	54E31/3tw		35.4	39.7	38.3	63.0	1.98
16WF36	53E939/1tf		33.5	40.8	40.0	61.7	2.16
	53E939/2tf		38.2	-	39.5	61.8	2.22
	53E939/3tw		41.4	43.5	42.7	64.5	2.17
	53E939/4tf		-	39.6	39.2	61.2	1.94
8WF67	54E67/1 tf		-	32.4	32.2	61.4	1.18
	54E67/2 tf		28.5	35.2	34.6	61.9	1.25
	54E67/3 tw		-	38.8	37.7	60.6	1.94
	54E67/4 tf		-	34.1	33.2	61.3	1.44

Summary of Coupon Tests (Cont'd.)

Section	Mark		E ksi	σ_{pL} ksi	σ_{uy} ksi	σ_{yL} ksi	σ_{ult} ksi	ϵ_{st} in/in
12WF99	55E/2	tf		31.3	34.6	34.5	62.5	1.31
	55E/4	tf		34.3	36.7	35.8	63.7	1.41
12WF65	42E/1	tf		-	37.2	36.4	62.0	1.61
	42E/2	tf		-	36.4	36.1	62.1	1.55
	42E/3	tw		-	40.6	38.8	61.5	1.43
	42E/4	tf		-	37.1	36.1	62.2	1.48

Notes:

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 $\frac{L}{r} = 72$

Then from A.I.S.C. Manual

$$\frac{P}{A} = 17{,}000 - 0.405 \left(\frac{L}{r}\right)^2$$

Column Working Stress = 14.5 ksi

Structural Section Details

Column Size	Area "*	Area as measured	P_w kips	1.65 P_w	$2xP_w$	Test No.
8WF31	9.12	9.01	132	218	264	A1,B6,C9
8WF67	19.70	19.94	286	472	572	A2
12WF40	11.77	11.31	171	283	344	B8,C11,D12
12WF65	19.11	18.66	278	459	550	A4
12WF99	29.09	28.45	422	696	800k**	A5

* A.I.S.C. Handbook Value.

** Testing Machine capacity = 800k

Analysis of Beams and Beam-column Flange Welds:

All dimension of sections as measured on specimens

Beams: 16WF36 $\sigma_w = 20\text{ksi}$

Bending:

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

use 24kips

$$V_y = \frac{\sigma_y S}{L}, \qquad \sigma_y \text{ (avg. for 16WF36)} = 39.6 \text{ ksi}$$

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

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

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

Elastic Analysis of Welds at Working Beam Load:

Design butt welds to carry applied moment, and fillet weld to carry applied shear.

$$M_w = 24 \times 48 \text{ k"}, \qquad d_w = 15.91 - 0.43 = 15.48"$$

$$F_w = \frac{M_w}{d_w} = \frac{24 \times 48}{15.48} = 74.5 \text{ kips}$$

$$\sigma = \frac{F_w}{\text{Weld Area}} = \frac{74.5}{7.09} \times 0.431 = 24.4 \text{ ksi} > 20\text{ksi (overstressed)}$$

$$V_w = 24 \text{ k}$$

Length of welds = 26",

Try 1/4" fillet weld

$$q = \frac{V_w}{L} = \frac{24}{26} = 0.922 \text{ k/"}$$

$$q_{allow} \text{ (1/4" fillet)} = 9.6D = 9.6 \times \frac{1}{4} = 2.4 \text{ kips/in} \quad \text{(OK)}$$

Shear:

$$\text{(in plastic range)} = 18 \text{ wd}^{(3)}$$

$$= 18 \cdot (0.29)\, 15.91$$

$$= 83 \text{ k} \quad 52.5 \text{ k}$$

$$V_u \text{ (predicted)} = 52.5 \text{ k}$$

$$\text{(in elastic range)} = 13 \text{ wd}$$

$$= 13 \cdot (0.29)\, 15.91$$

$$= 60 \text{ k} > 24 \text{ k} \quad \text{(OK)}$$

Influence of shear on V_u may be neglected if

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

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

$$\frac{63.76}{48} < \frac{(14.94)(0.29)}{1.732}$$

$$1.33 < 2.5 \qquad \text{(OK)}$$

Lateral Buckling:

(in the elastic range) $\frac{Ld}{bt} = \frac{96 \times 15.91}{7.09 \times 0.431} = 500 < 600$ (OK)

whence $\sigma_{allow} = 20$ ksi

Local Buckling: elastic range - See Clause 18(b) of A.I.S.C. spec.

Actual $\frac{b}{t} = \frac{7.09}{0.431} = 16.45 < 32$ (OK)

To reach strain hardening $\frac{b}{t} \leq 17$ (6)

Therefore beams critical for local flange buckling in plastic range.

To reach strain hardening $\frac{d}{w} \leq 55$ (6) for web

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

Therefore beams critical for local buckling in plastic range.

Deflections: $\delta_{elastic} = \frac{VL^3}{3EI}$ - assuming complete restraint.

$$\delta_y = \frac{V_y L^3}{3EI}$$

$V_y = 46.5$ kips
$L = 48"$
$E = 30 \times 10^3$ ksi
$I = 448.96\ in^4$

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

$= 0.127"$

$\delta_{ult.} = \frac{52.5}{46.5} \times 0.127 = 0.144"$ assuming idealized

$\sigma - \epsilon$ and $M - \phi$ relationship.

In nondimensional form:

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

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

<u>Beam Rotations:</u>

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:

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

Therefore $\theta_{yield} = \frac{V_y L^2}{2EI}$ and $V_y = \frac{2EI}{L^2} \cdot \theta_y$

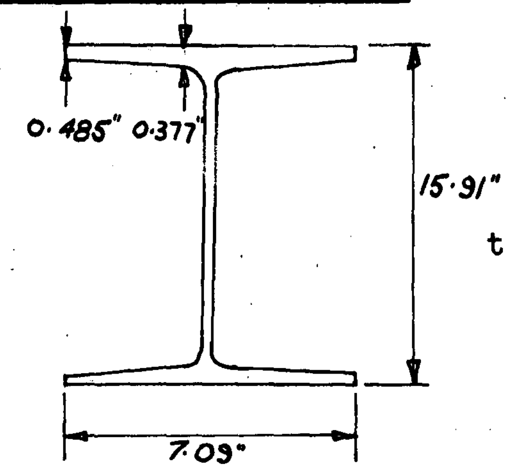
But $V_y = \frac{3EI}{L^2} \delta_y$

Therefore $\theta_y = \frac{3EI}{L^2} \delta_y \cdot \frac{L^2}{2EI} \quad \frac{3}{2} \delta \frac{y}{L}$

$$= 0.031 \, \delta_y \text{ radians}$$

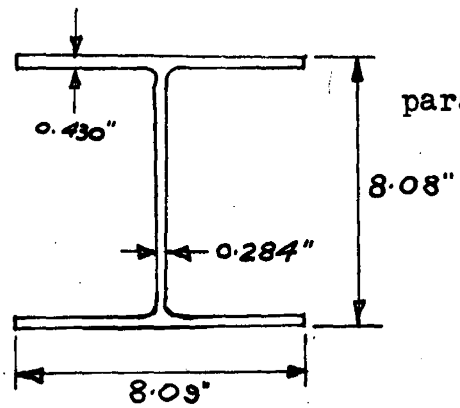
2.3 Material Dimensions and Properties - average values

16WF36:
beam



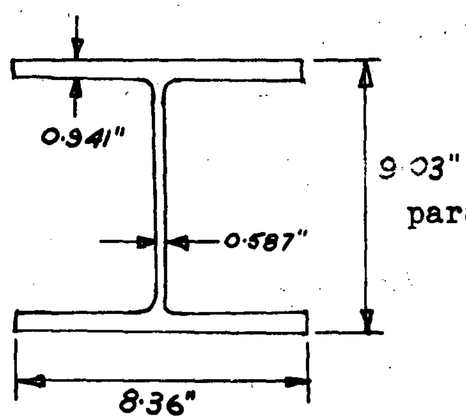
tapered flanges

8WF31: (A-1)
Column (B-6)
(C-9)



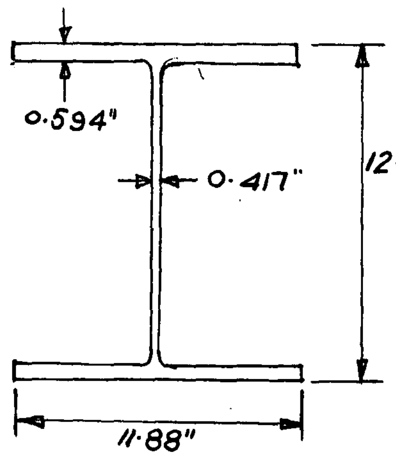
parallel flanges

8WF67: (A-2)
Column



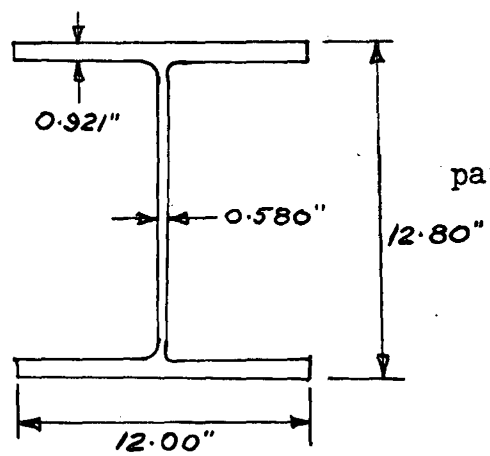
parallel flanges

12WF65: (A-4)
Column



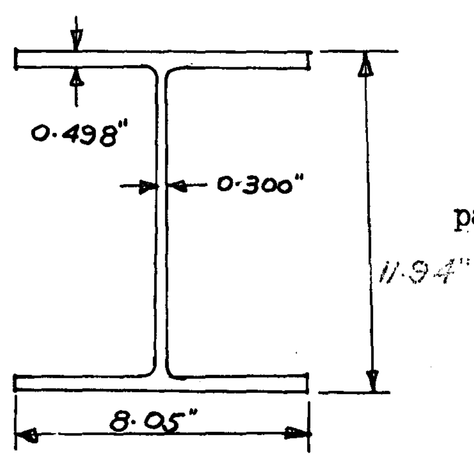
parallel flanges

12WF99: (A-5)
Column



parallel flanges

12WF40: (B-8)
Column (C-11)



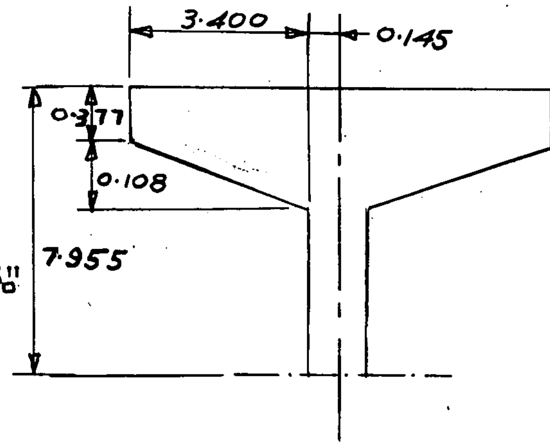
parallel flanges

Plate thickness: average values - from coupon and measurements on specimens

B-6	0.439"	(7/16)	b = 3.9"
B-8	0.264"	(1/4)	b = 3.9"
C-9, C-11	0.305"	(5/16)	d = 22"

2.4 Section Properties.

16WF36 Beams:



$$\frac{A}{4} = (3.4)(.377) = 1.283$$
$$\frac{.108(3.4)}{2} = 0.184$$
$$(.145)(7.96) = \underline{1.155}$$
$$2.622$$

$$A = 10.488 \square''$$

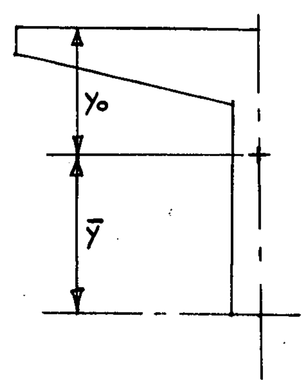
$$\frac{I_{xx}}{4} = (1.283)(7.766)^2 = 77.40$$
$$(0.184)(7.542)^2 = 10.46$$
$$\frac{(0.145)(7.955)^3}{12} = 6.11$$
$$(1.155)(3.978)^2 = \underline{18.27}$$
$$112.24$$

$$I_{xx} = 448.96 \text{ in}^4$$

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

$$\frac{Z}{4} = (1.283)(7.766) = 9.96$$
$$(0.184)(7.542) = 1.39$$
$$(1.155)(3.978) = \underline{4.59}$$
$$15.94$$

$$Z = 63.76 \text{ in.}^3$$



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

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

8WF31 Column: $A = (2)(0.430)(8.09) = 6.96$
$(7.22)(0.284) = \underline{2.05}$
$9.01 \square''$

8WF67 Column: $A = (2)(0.941)(8.36) = 15.75$
$(7.148)(0.587) = \underline{4.19}$
$19.94 \square''$

12WF65 Column: $A = (2)(.594)(11.88) = 14.10$
$(10.942)(0.417) = \underline{4.56}$
$18.66 \square''$

12WF99 Column: $A = (2)(0.921)(12) = 22.10$
$(10.958)(0.58) = \underline{6.35}$
$28.45 \square''$

12WF40 Column:

$$A = (2)(0.498)(8.05) = 8.03$$
$$(10.944)(.300) = 3.28$$
$$11.31 \ \square''$$

3. Appendix To Four-way Tests.

3.1 Summary of Coupon Tests.

Section	Mark	E Ksi	σ_{uy} Ksi	σ_{yl} Ksi	$\sigma_{st\ y}$ Ksi	$\sigma_{ult.}$ Ksi	ϵ_{st} in./in.
1/4"Plate	233/P	30,900	40.65	39.87		62.00	0.01725
		25,100	41.02	39.74		61.54	0.01775
12WF65	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
12WF40	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
16WF36	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
12WF27	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

Notes:

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 Section Details:

Test	Column Size	Area in^2*	Area as measured	P_W	$1.65P_W$	$2P_W$
AA	12WF65	19.11	19.00	276	455	552
BB	12WF40	11.77	11.70	170	280	340
DD	12WF40	11.77	11.49	167	276	334

* A.I.S.C. Handbook Value

Analysis of Beams and Beam-Column Flange Welds:

All dimensions of sections as measured on specimens

$$\sigma_w = 20 \text{ ksi}$$

Bending:

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

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

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

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

12WF27 Beams:

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

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

$T = C \quad \dfrac{684}{11.95} = 57.2$ kips

Spec. section (26h) A.I.S.C.:

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

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

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

From Table 25 in the A.I.S.C. text of Structural Shop Drafting Vol. 2, the choice is:

4" wide seat; 1/4" Fillet Welds; L = 7"

Plate Thickness 1/2"

Top Plate Weld Design:

Required plate thickness $= \dfrac{57.2}{20 \times 9.75} = 0.3"$

At ultimate load the unit stress will be $\dfrac{125}{0.3 \times 9.75} = 42.7$ kips

Use 1/2" Plate

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

Using butt welds on the plate

$$\frac{125}{17.25} = 7.25 \text{ k/in}$$

$\dfrac{7250}{1x1/2} = 14500$ psi. This is OK.

Weld connecting Top Plate to Beam Flanges:

The fillet welds are limited to 3/8" size.

Working stress for 3/8" fillets is 3600 pounds/in.

Using the factor of safety of 3, we use for design

$$3 \times 3600$$

$$= 10,800 \quad lb/in$$

Required length of weld

$$= \frac{125}{10}$$

$$= 12.5 \text{ in}$$

Length of weld available = 6" overhead fillets.6 - 1/2" fillet on top of flange.

Check on Tee Seat:

From Grover's "Manual of Design for Arc Welded Steel Structures", page 123

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

where $D = \frac{3"}{16}$ $\quad L = 8"$ $\quad e = 3.2"$

$$R = \frac{23.04 \times \frac{3}{16} \times 64}{\sqrt{64 + 16\ (3.2)^2}} = 18.3 \text{ kips}$$

Predicted ultimate $R = 3 \times 18.3 = 54.9$ K

This is nicely in excess of 41.4, the predicted ultimate load.

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 is 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

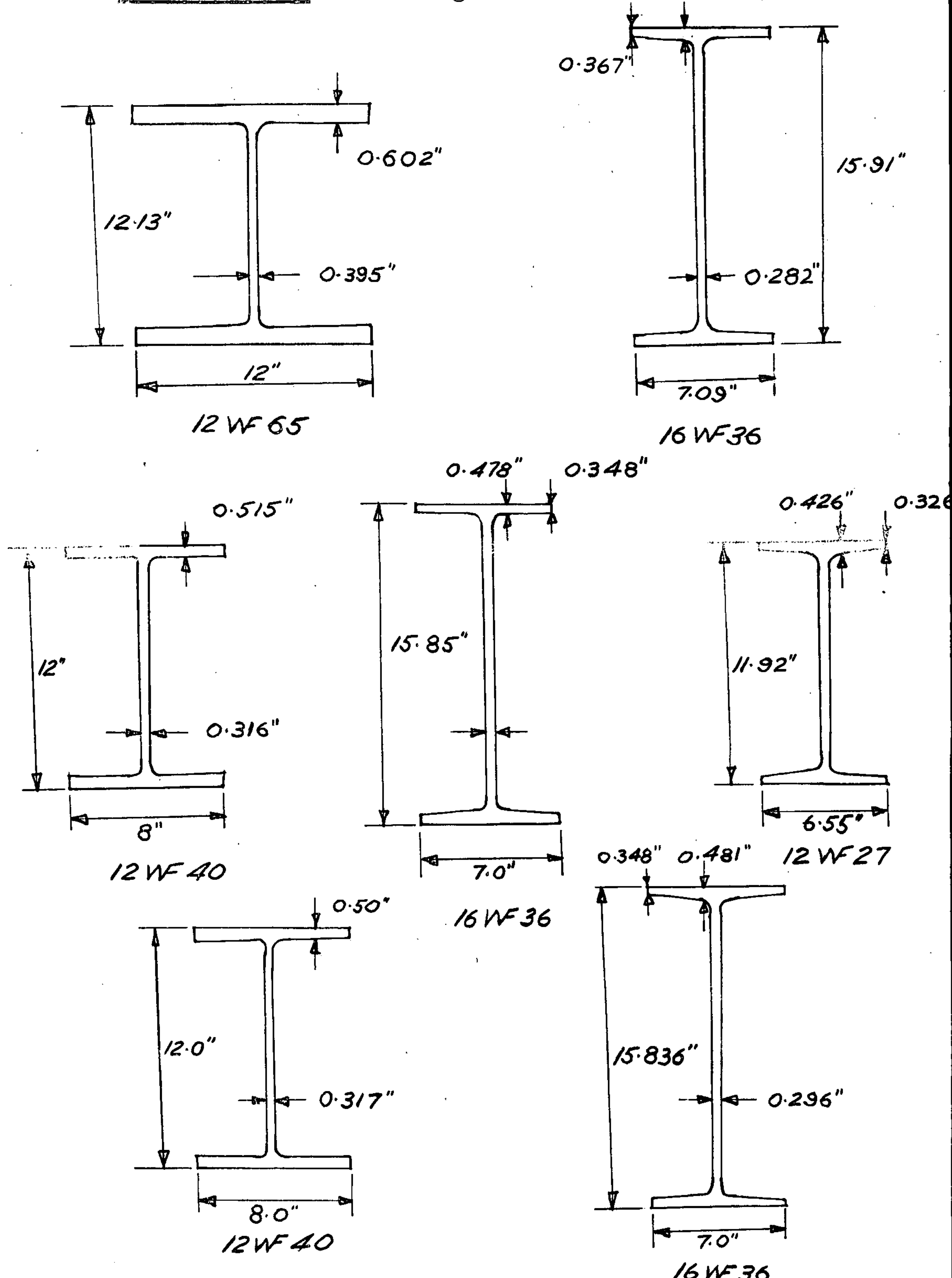
Test	Beam Size	Area	Section Modulus	Plastic Modulus
AA	16WF36	10.28	55.59	62.73
BB	16WF36 12WF27	10.29 7.83	54.20 32.60	61.52 36.56
DD	16.WF36	10.24	54.06	61.37

WF SECTIONS - average values

TEST AA

TEST BB

TEST DD



WF SECTIONS, ACTUAL DIMENSIONS

FOUR-WAY CONNECTIONS

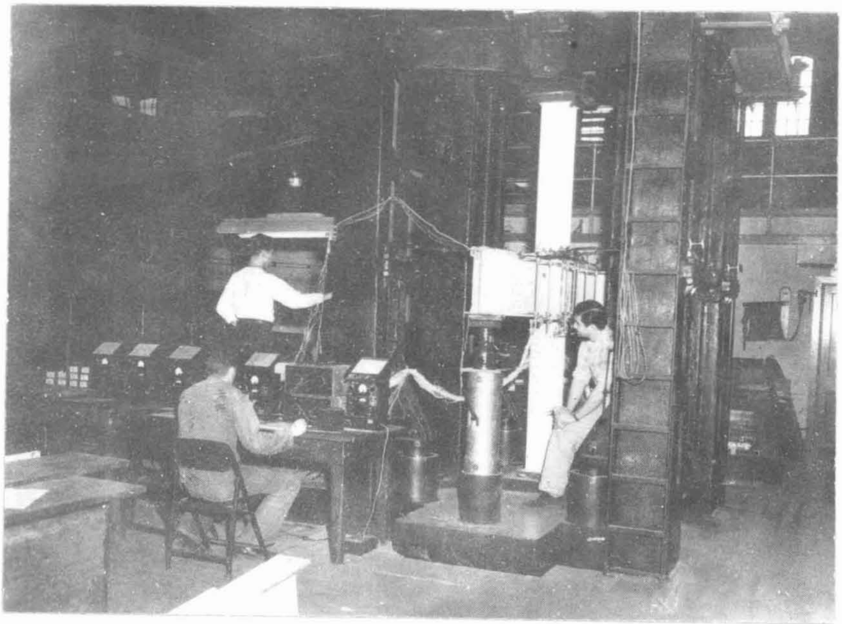


FIGURE 1 - GENERAL VIEW OF TWO-WAY TEST IN PROGRESS.

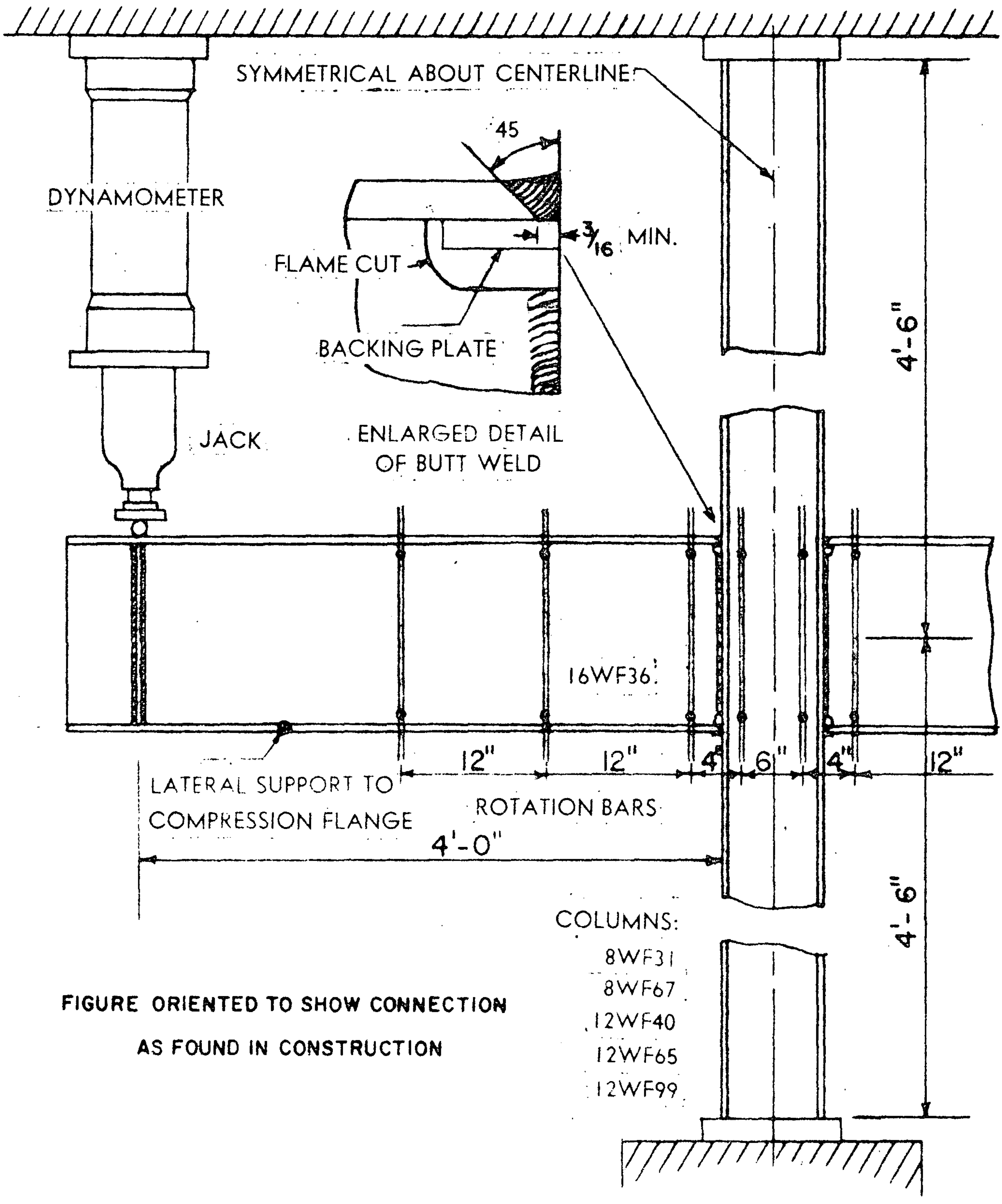
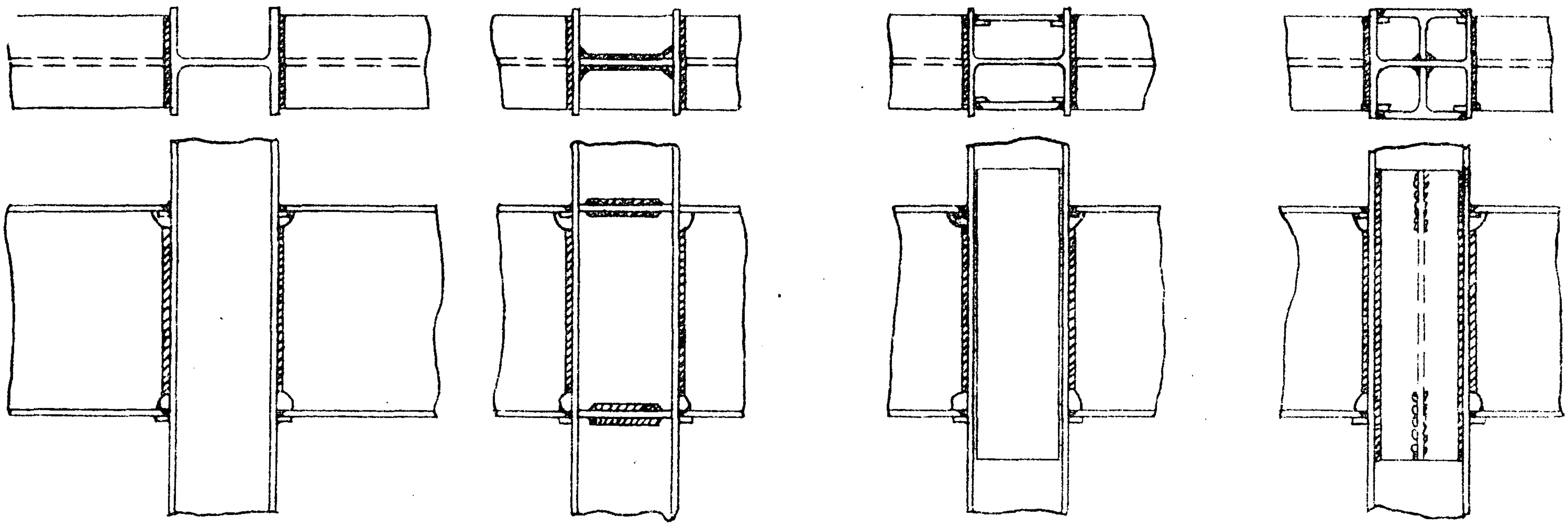


FIGURE 2. TEST ARRANGEMENT — TWO-WAY CONNECTIONS



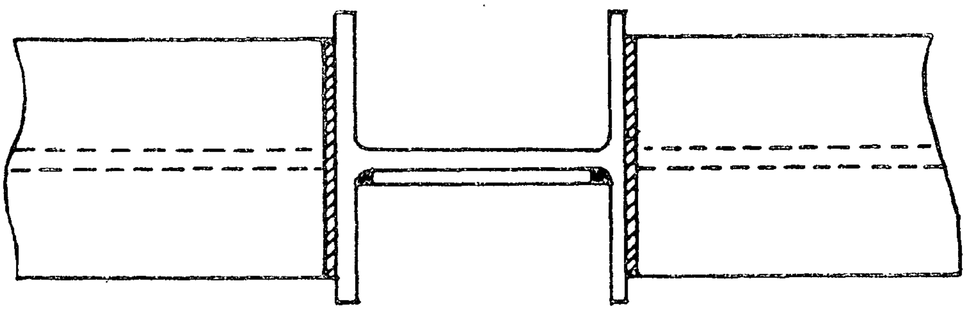
SERIES A
NO COLUMN
STIFFENERS

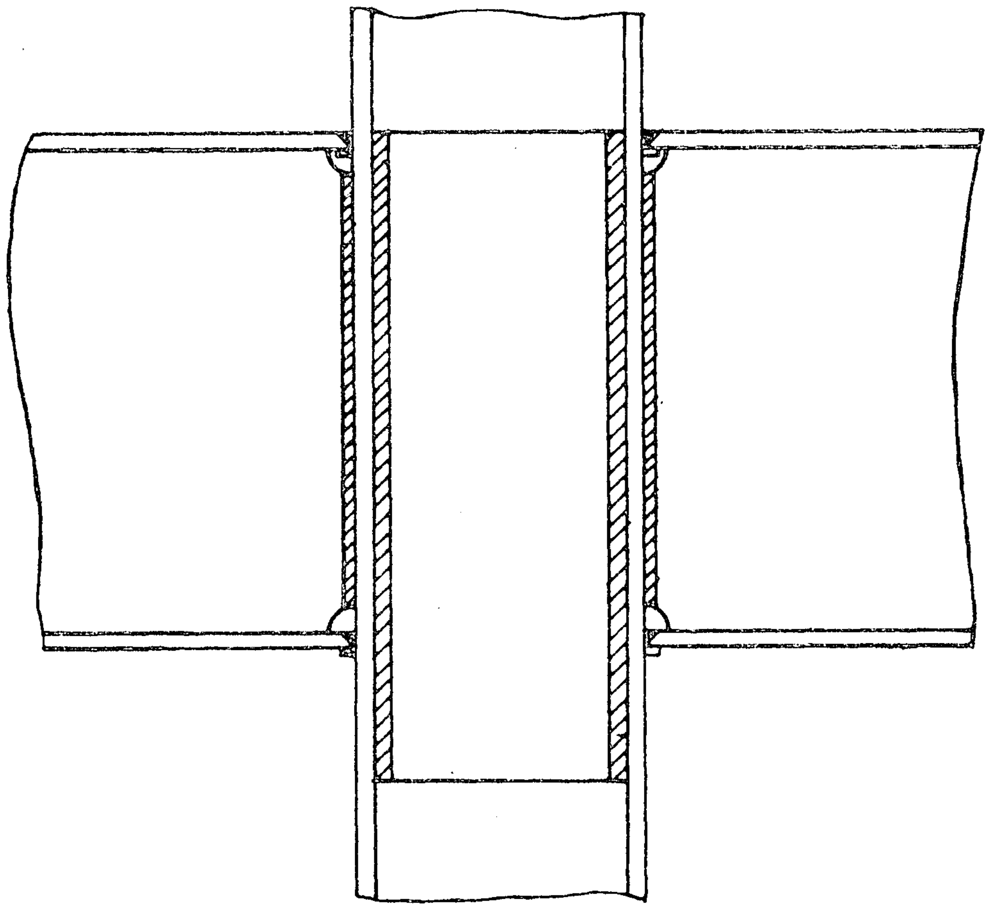
SERIES B
HORIZONTAL PLATE
STIFFENERS

SERIES C
VERTICAL PLATE
STIFFENERS

SERIES D
SPLIT WF
STIFFENERS

FIGURE 3 – THE A,B,C,D SERIES OF TWO-WAY BEAM COLUMN CONNECTIONS





SERIES H DOUBLER PLATE STIFFENERS

FIGURE 4 — THE TEST SERIES OF TWO-WAY BEAM COLUMN CONNECTIONS (CONTINUED)

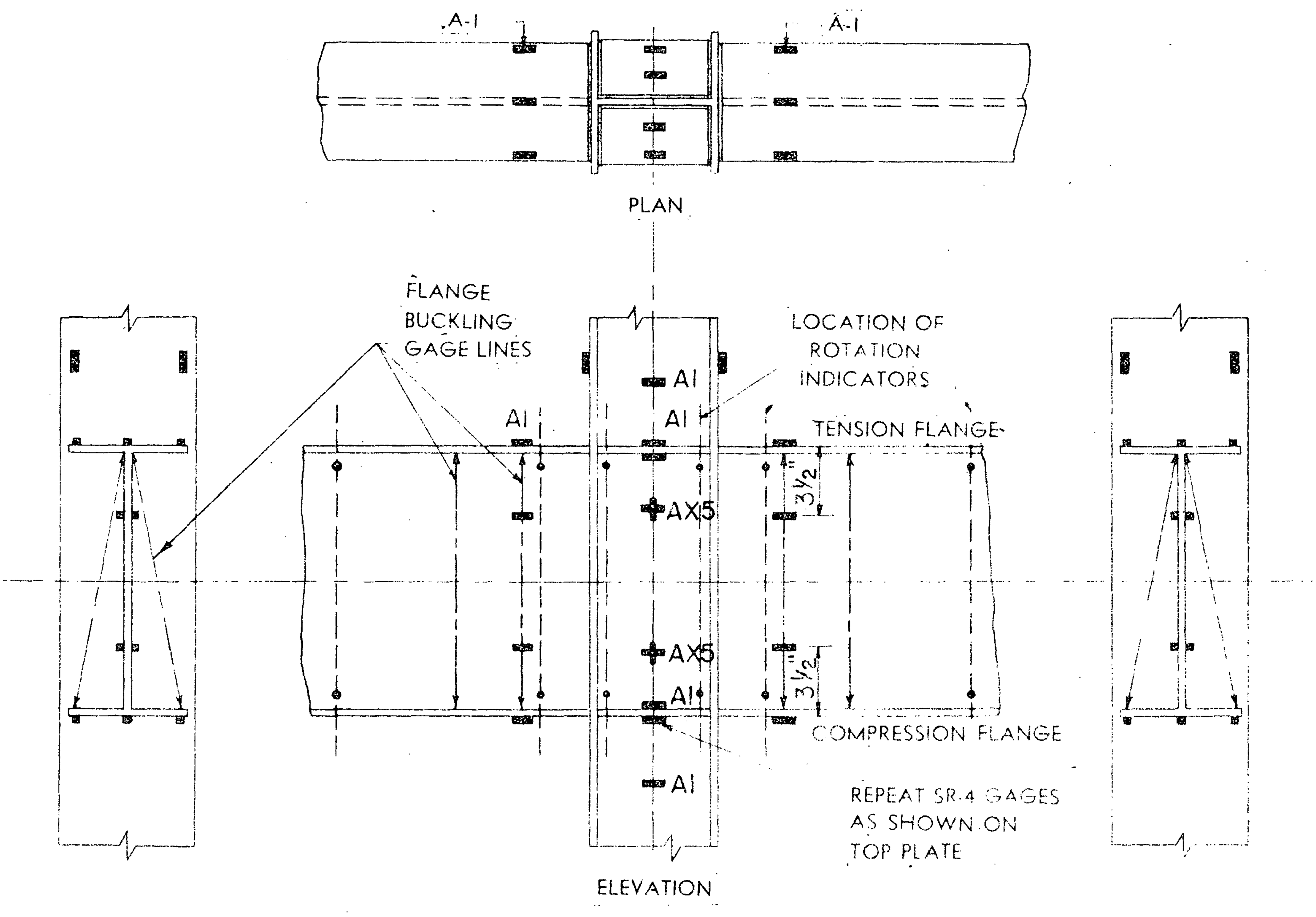


FIGURE 5 – INSTRUMENTATION PLAN—SERIES B

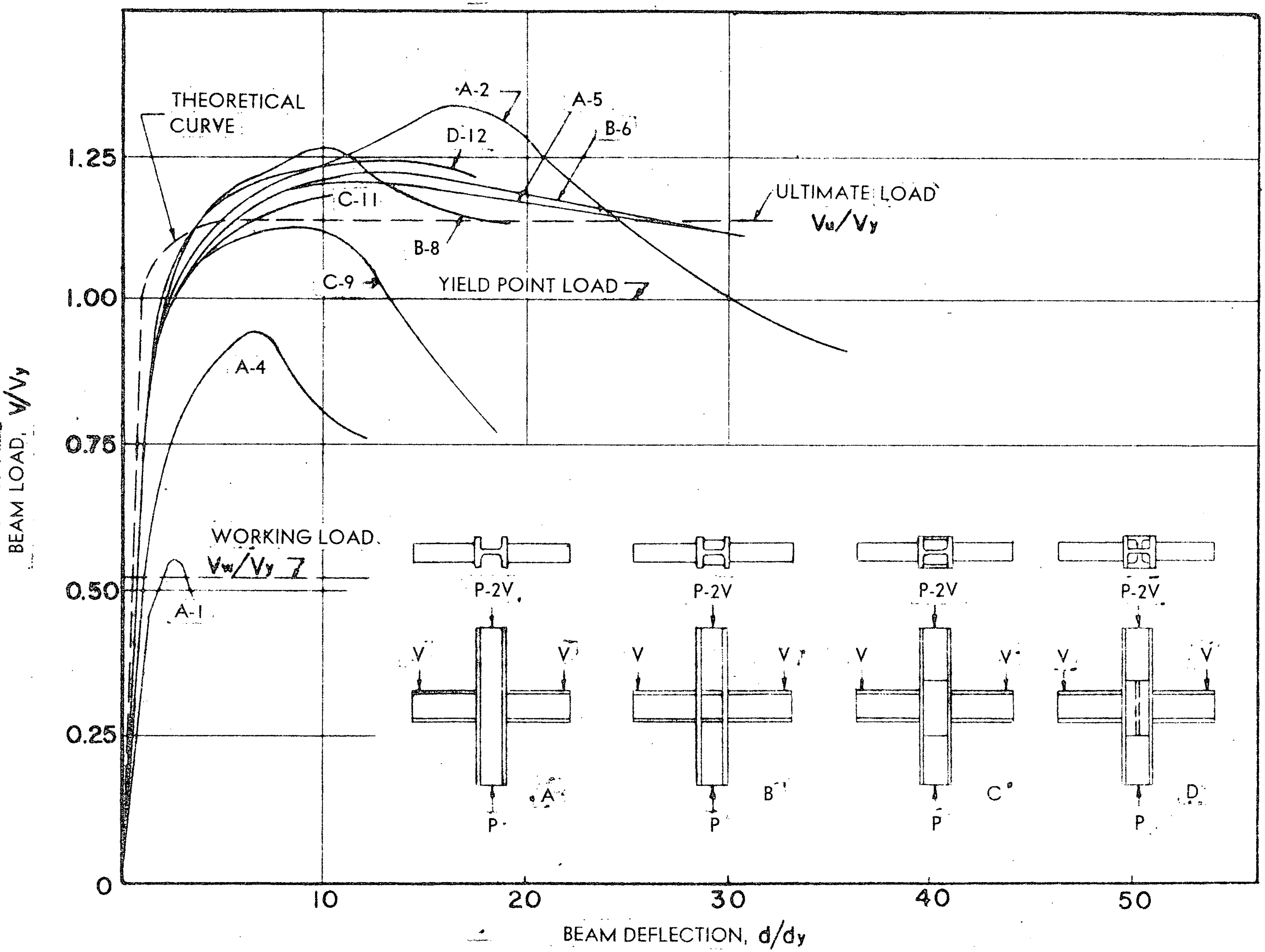
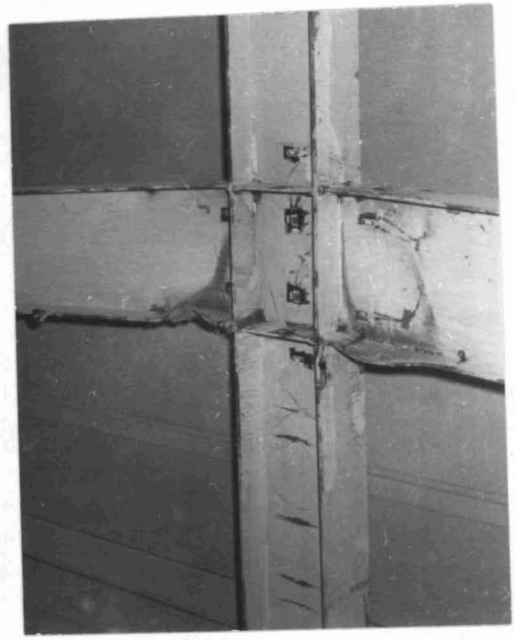
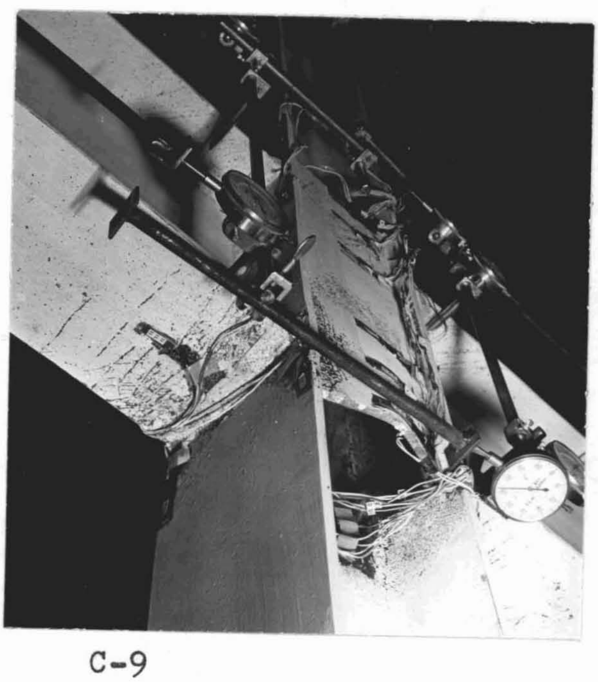


FIGURE 6 SUMMARY OF TEST RESULTS: BEAM LOAD VS. BEAM DEFLECTION

A-1



B-8



C-9

D-12

FIGURE 7 - PHOTOGRAPHS OF TESTS A-1, B-8, C-9 AND D-12.

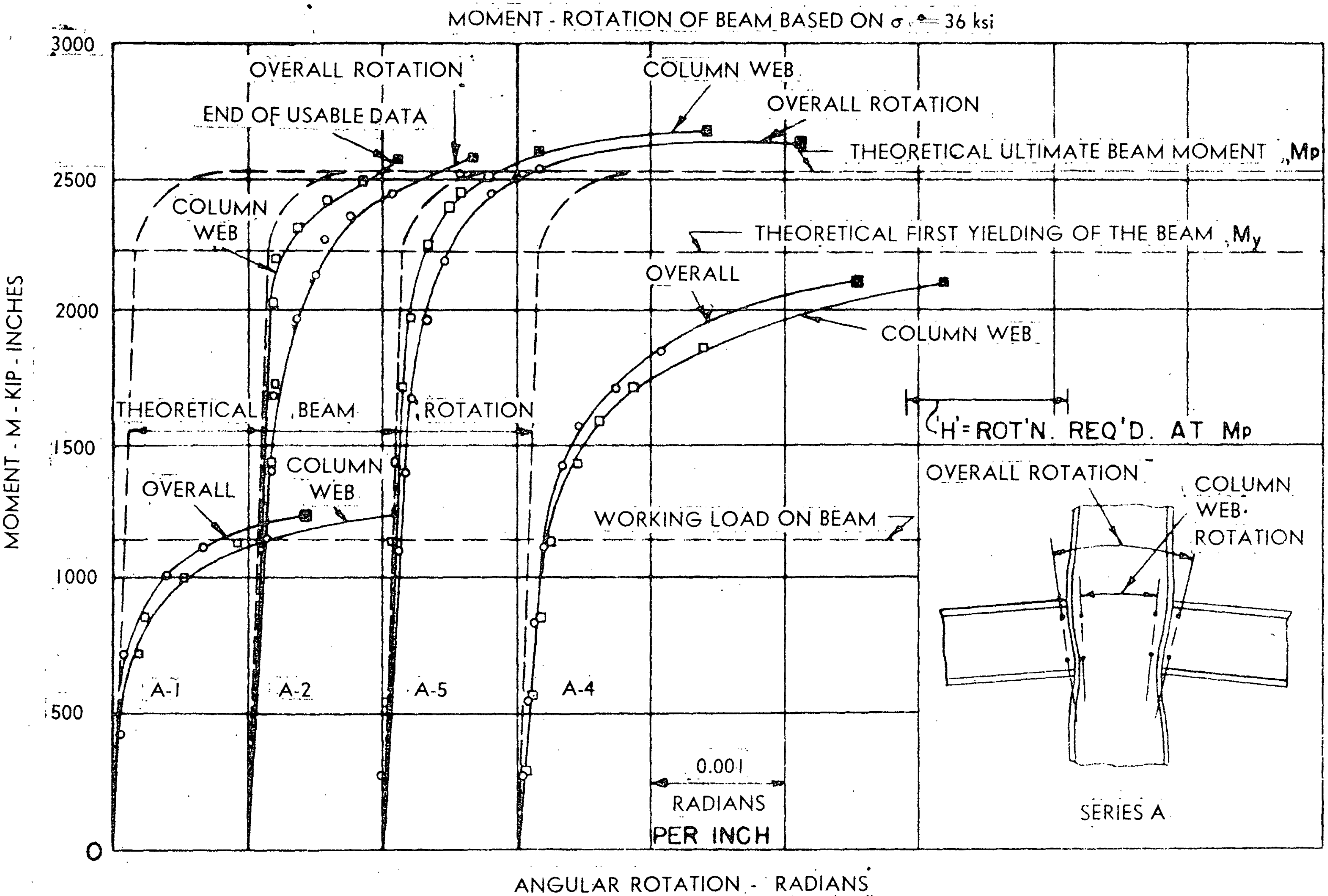


FIGURE 8 — COMPARISON OF COLUMN WEB AND CONNECTION ROTATION CHARACTERISTICS WITH THAT FOR THE BEAM: SERIES A

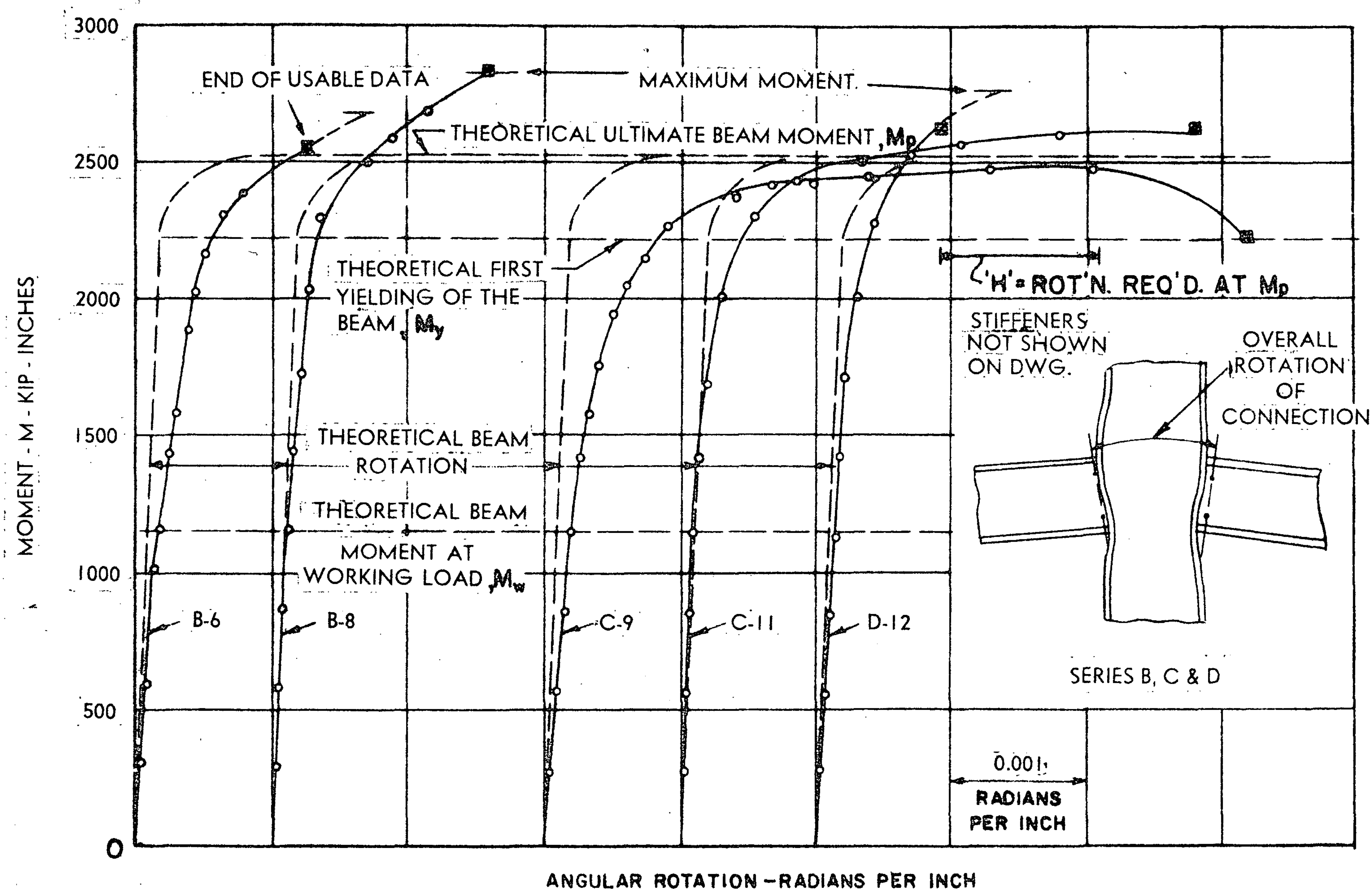


FIGURE 9 - COMPARISON OF COLUMN WEB AND CONNECTION ROTATION CHARACTERISTICS WITH THAT FOR THE BEAM: SERIES B,C,& D

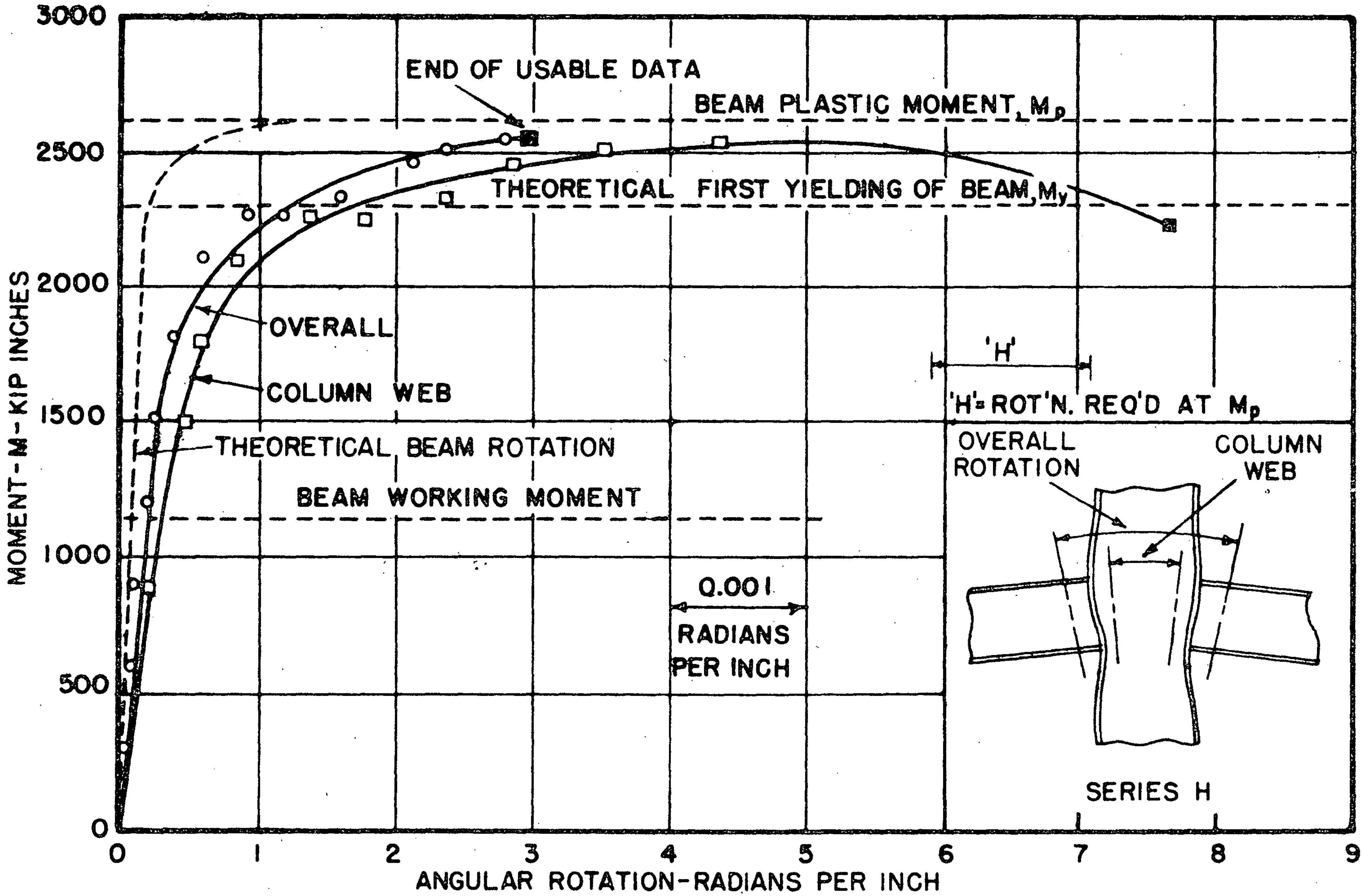


FIGURE 10 — COMPARISON OF COLUMN WEB AND CONNECTION ROTATION CHARACTERISTICS WITH THAT FOR THE BEAM: SERIES H

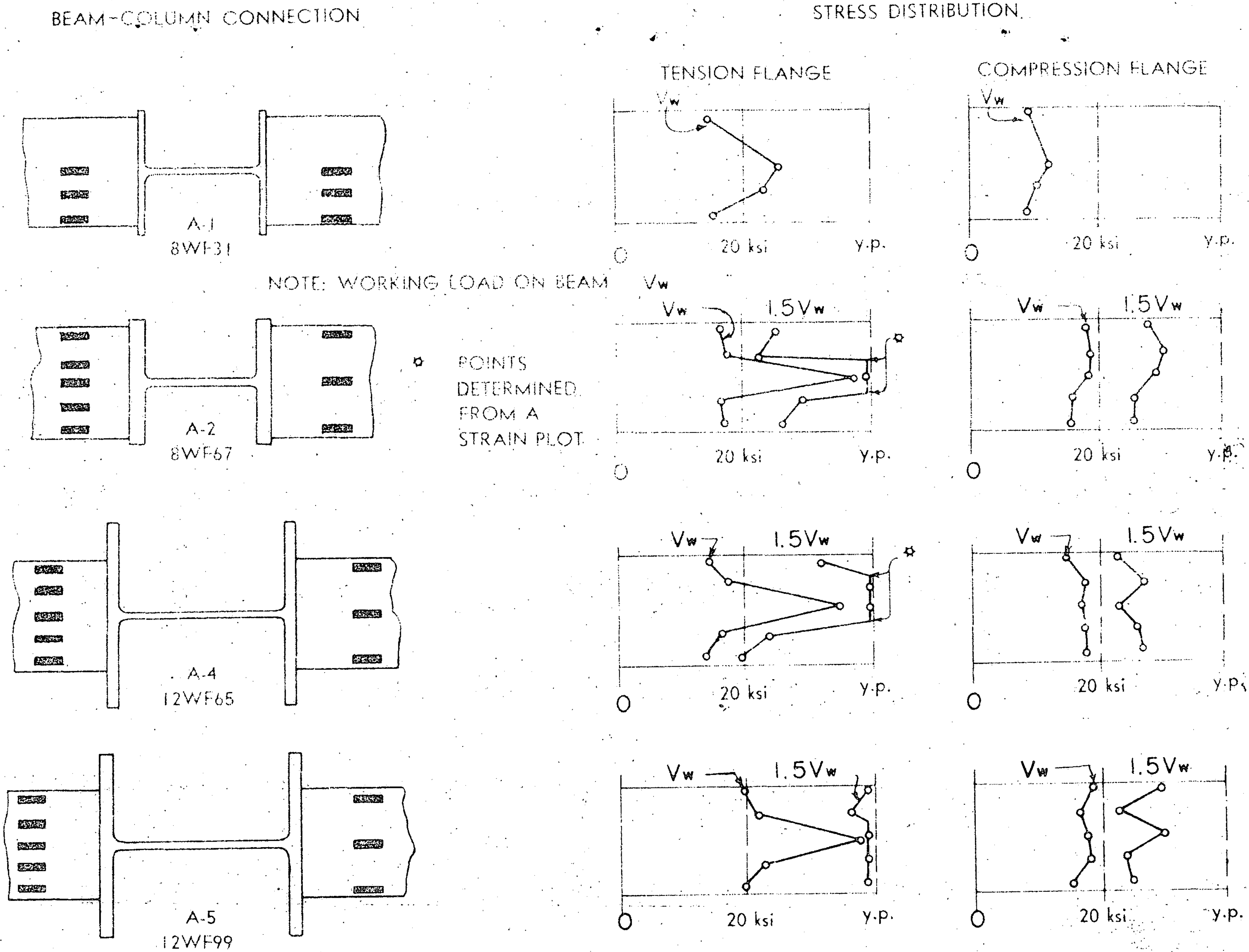


FIGURE 11 — STRESS DISTRIBUTION IN BEAM FLANGES: SERIES A

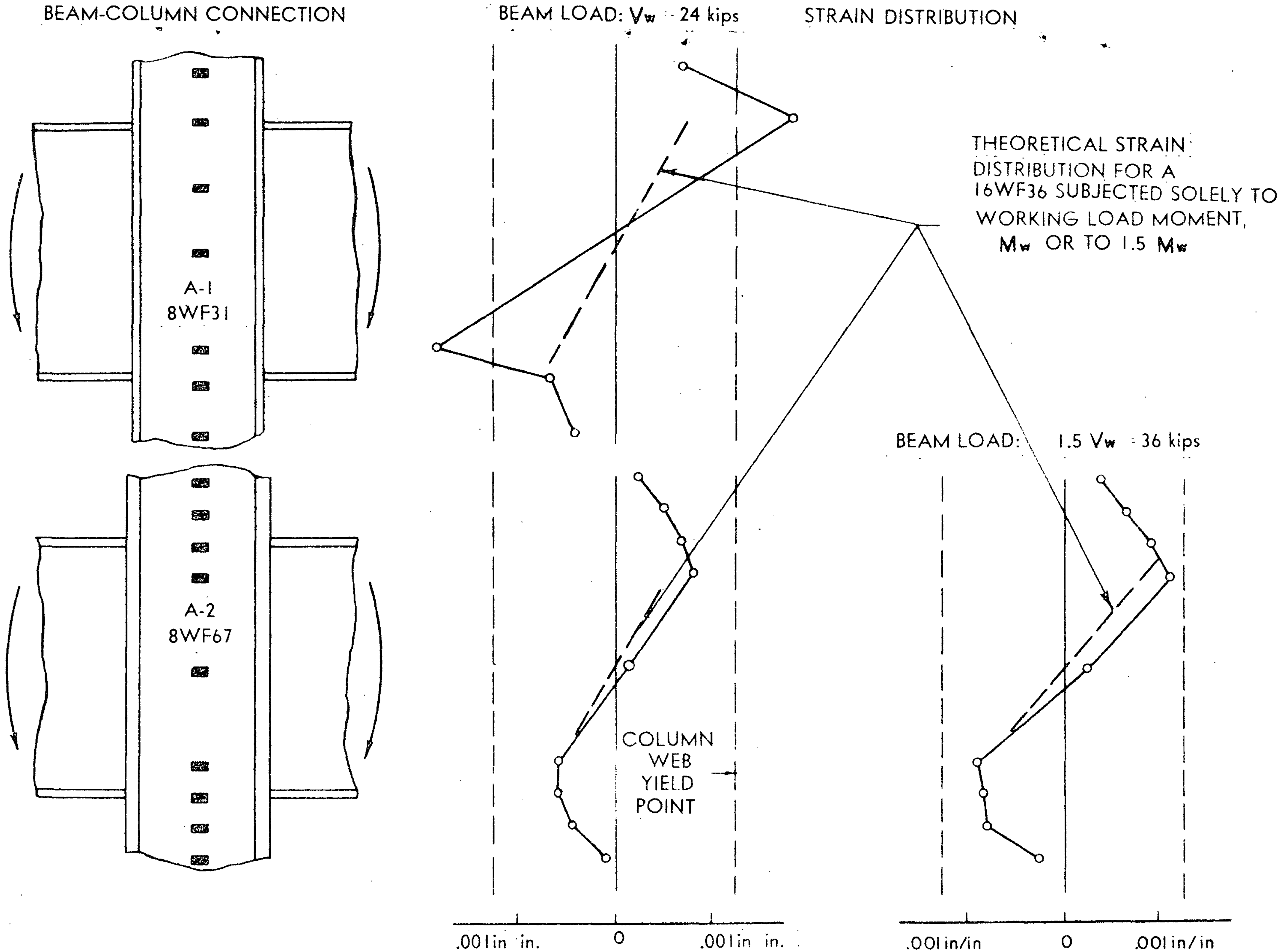


FIGURE 12 - DISTRIBUTION OF HORIZONTAL STRAIN IN COLUMN WEB: TESTS A-1 & A-2

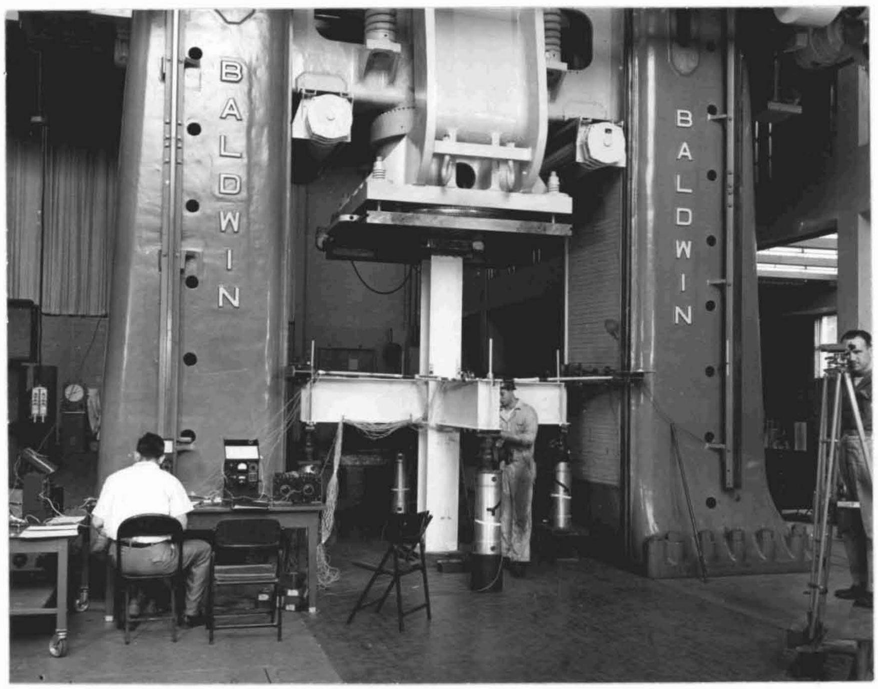


FIGURE 13 - GENERAL VIEW OF FOUR-WAY TEST IN PROGRESS.

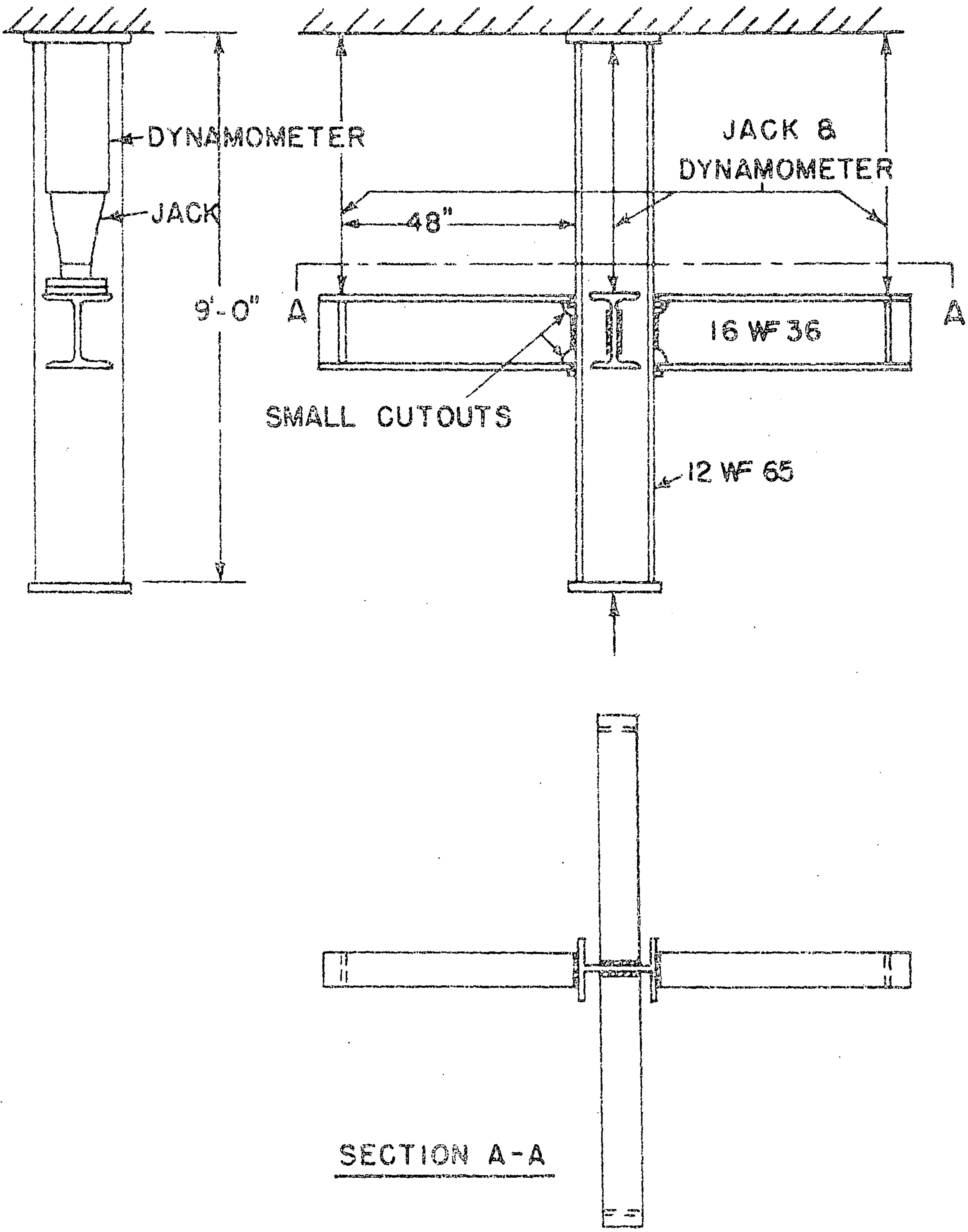
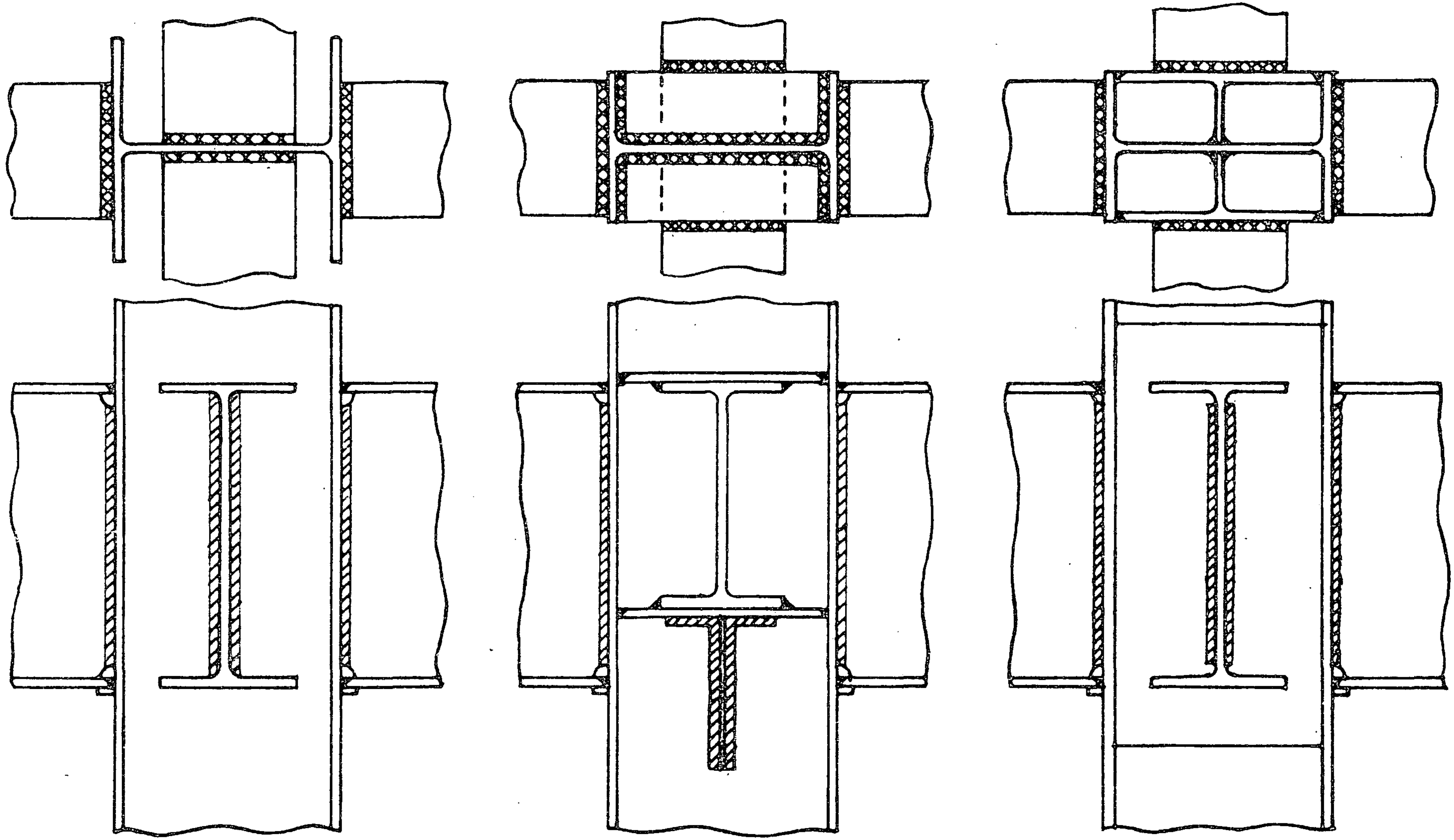


FIG. 14 - TEST ARRANGEMENT: FOUR WAY CONNECTIONS



TEST AA

TEST BB

TEST DD

FIGURE 15 - THE FOUR-WAY SERIES OF BEAM-COLUMN CONNECTIONS

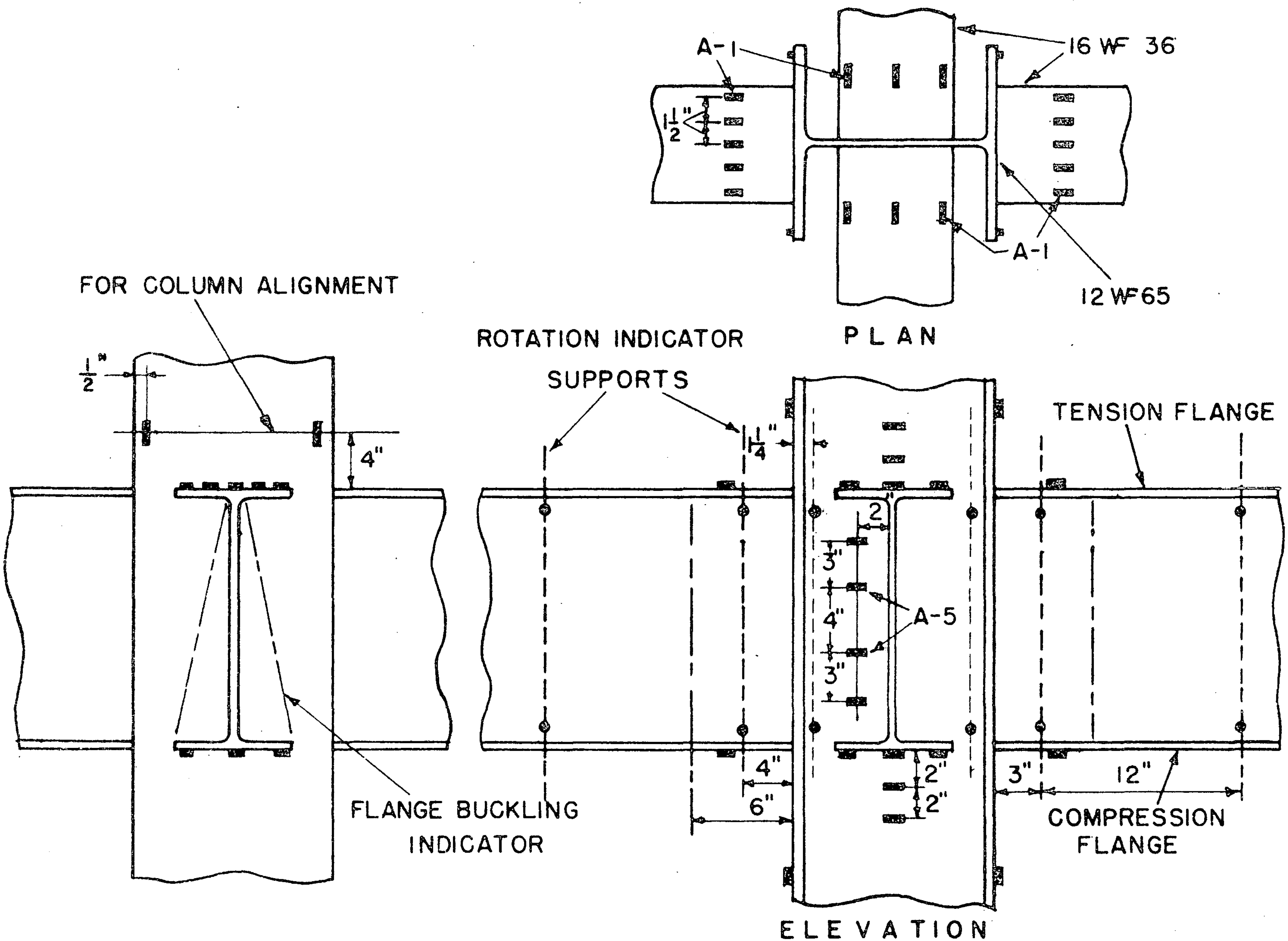


FIGURE 16 - INSTRUMENTATION PLAN: TEST AA

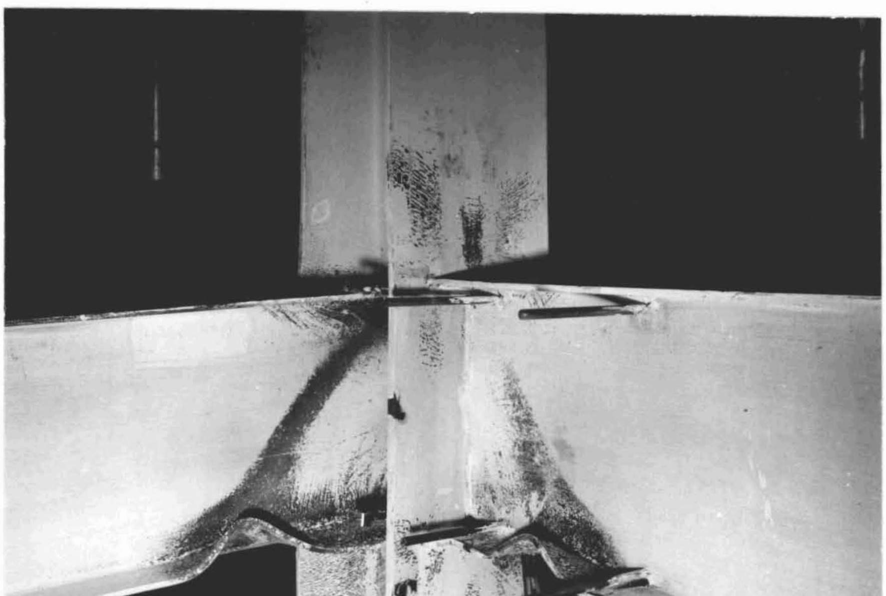


FIGURE 17 - SPECIMEN AA AT END OF TEST.

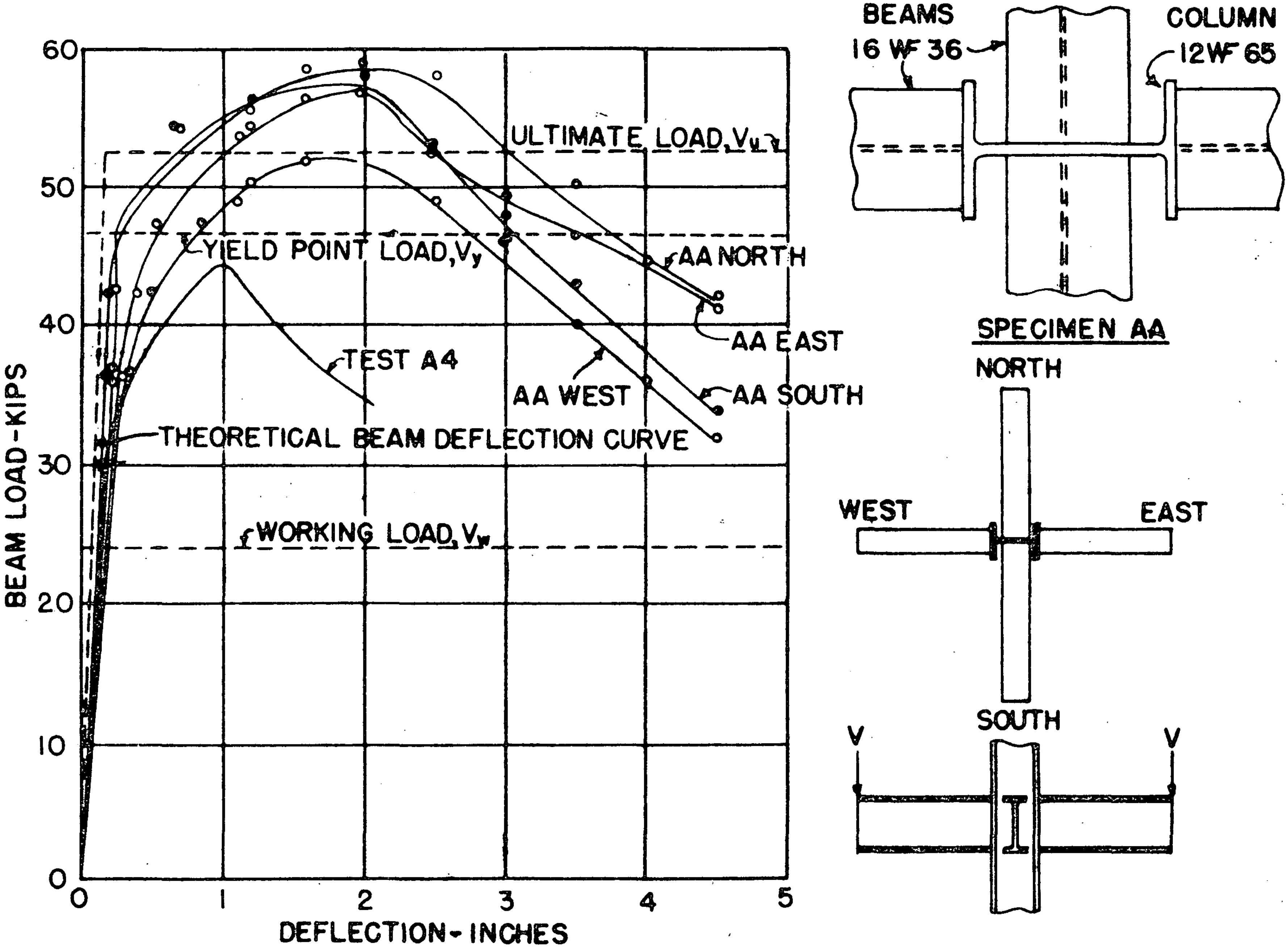


FIGURE 18 — BEAM LOAD VS. BEAM DEFLECTIONS: TEST AA

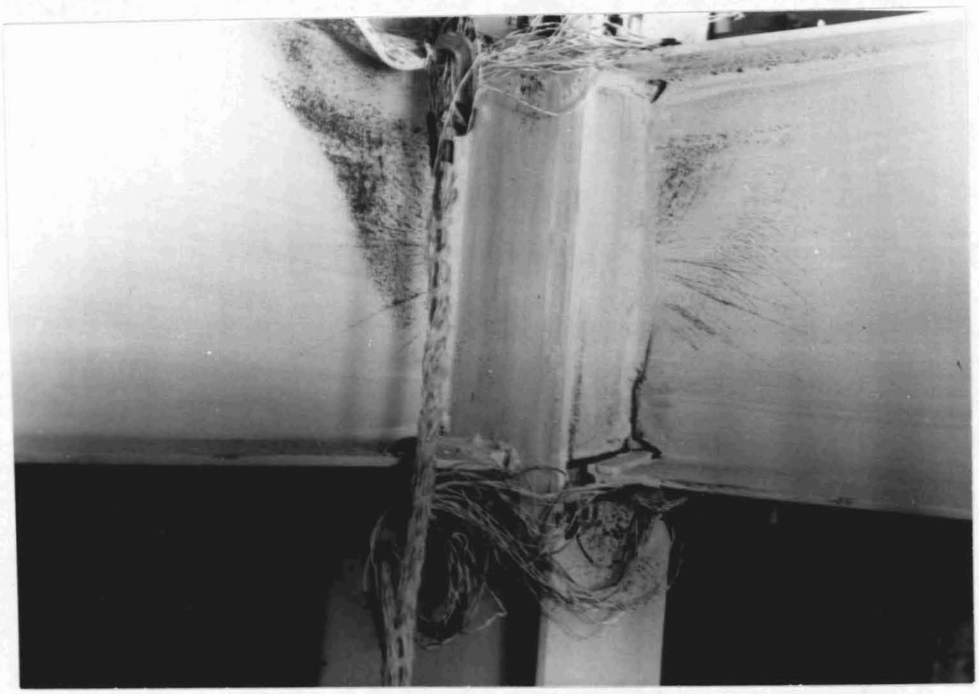


FIGURE 19 - FAILURE DETAILS - TEST DD.

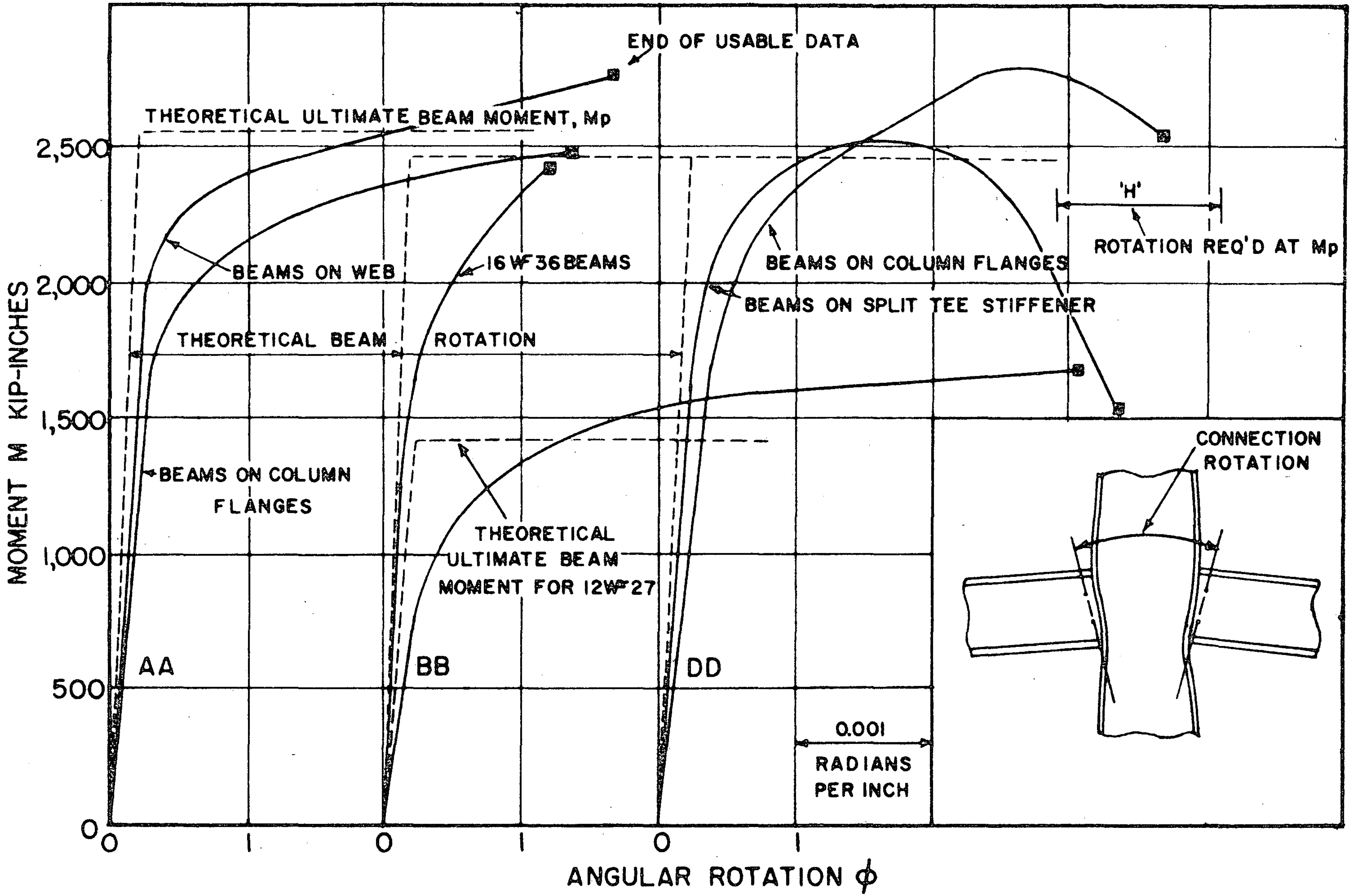


FIGURE 20 — COMPARISON OF OVERALL CONNECTION ROTATION CHARACTERISTICS WITH THOSE FOR THE BEAMS: AA, BB, DD.

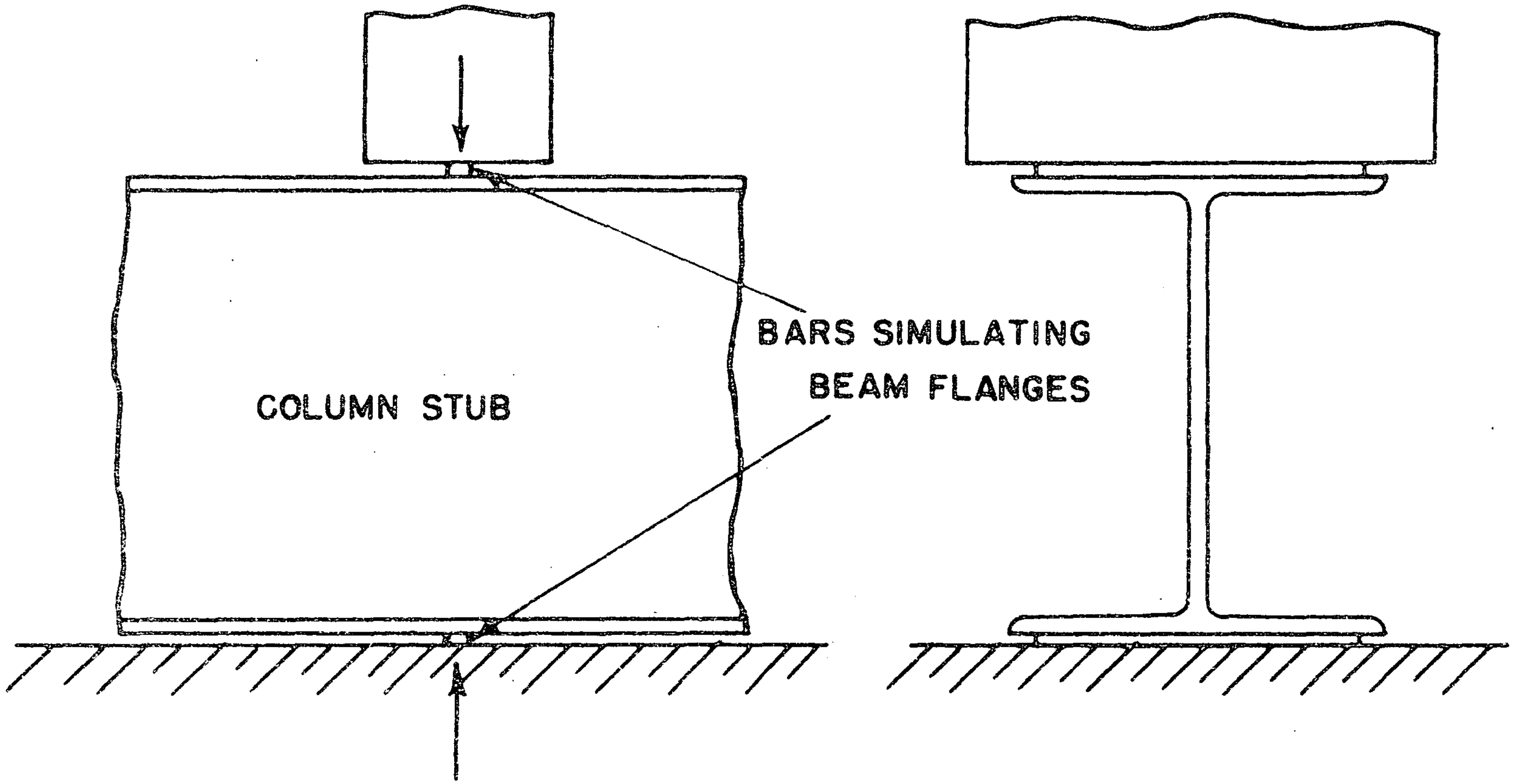


FIG.21 THE E SERIES

TESTS TO DETERMINE COMPRESSION SIDE CRITERION

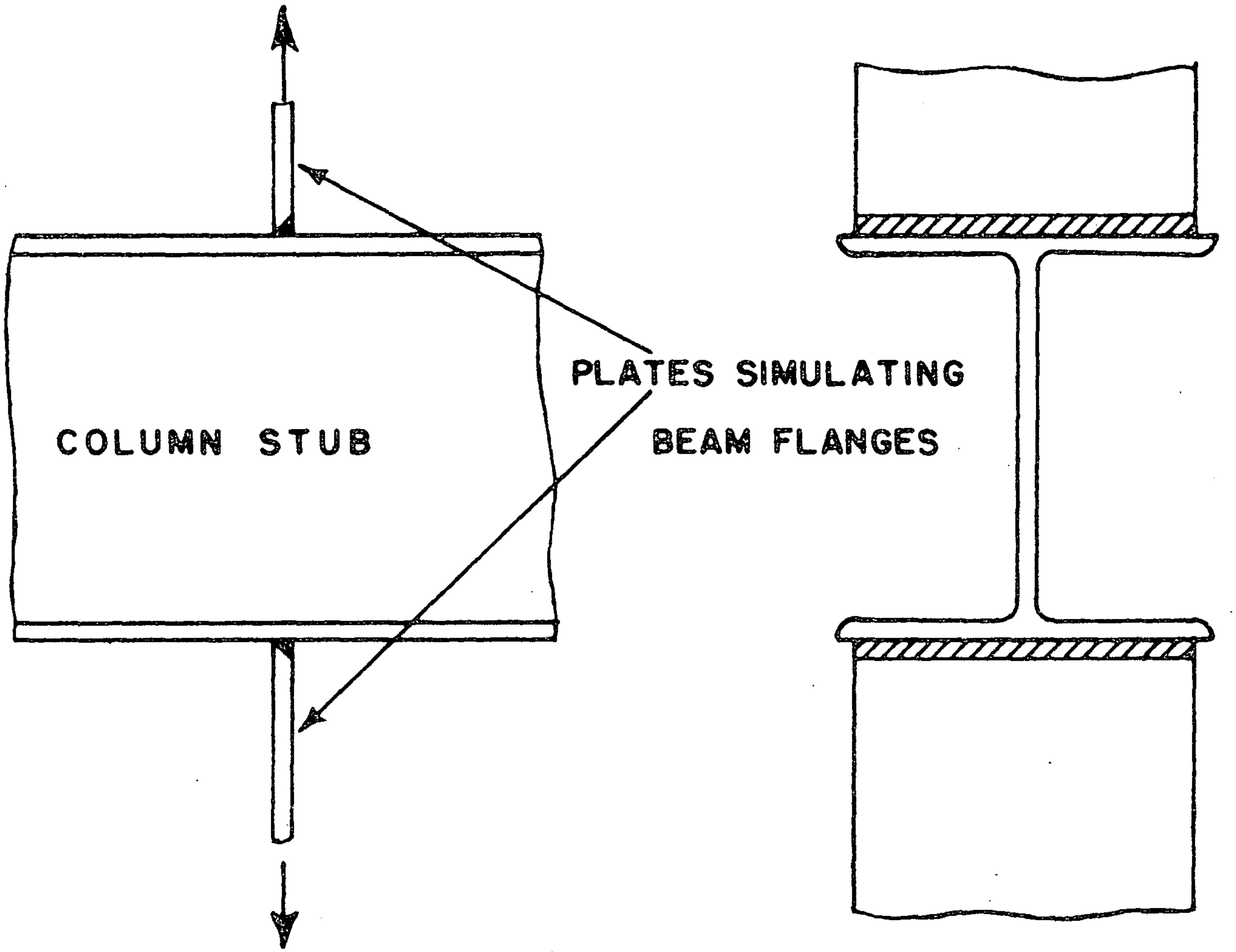


FIG. 22 THE F SERIES

TESTS TO DETERMINE TENSION SIDE CRITERION

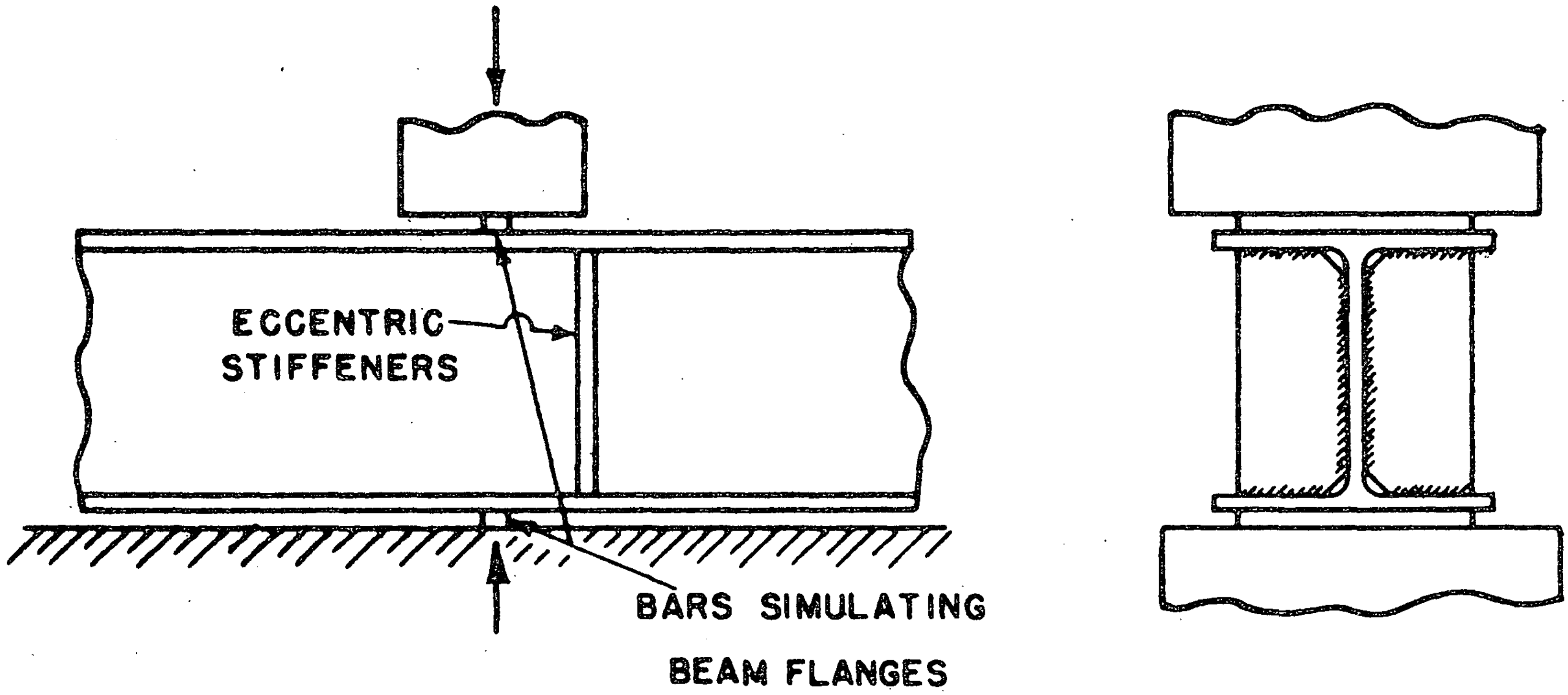
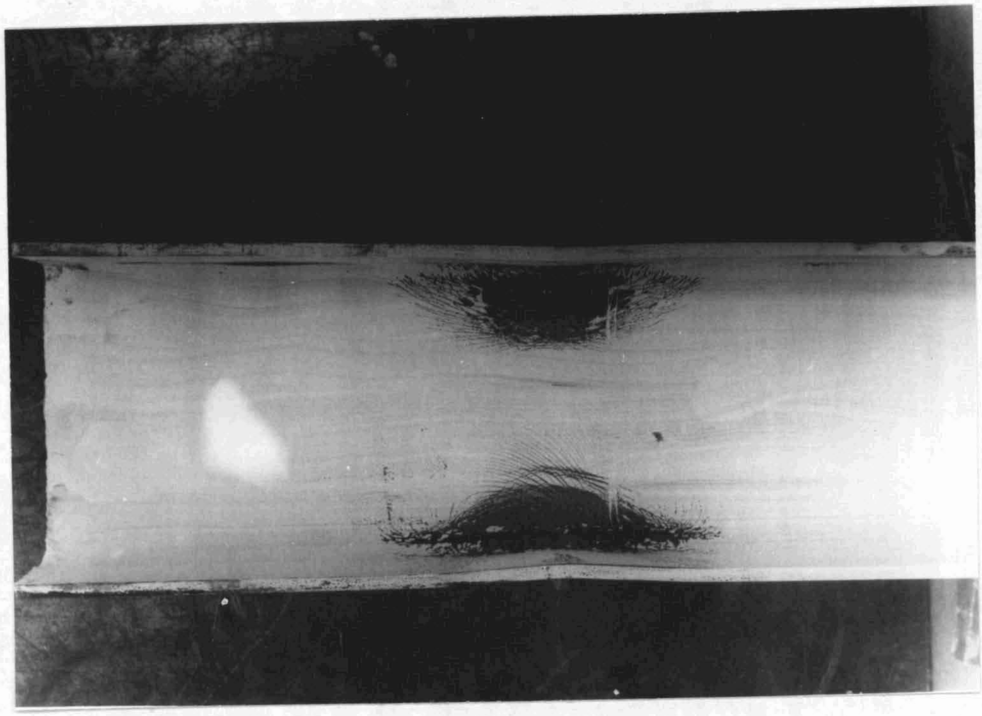
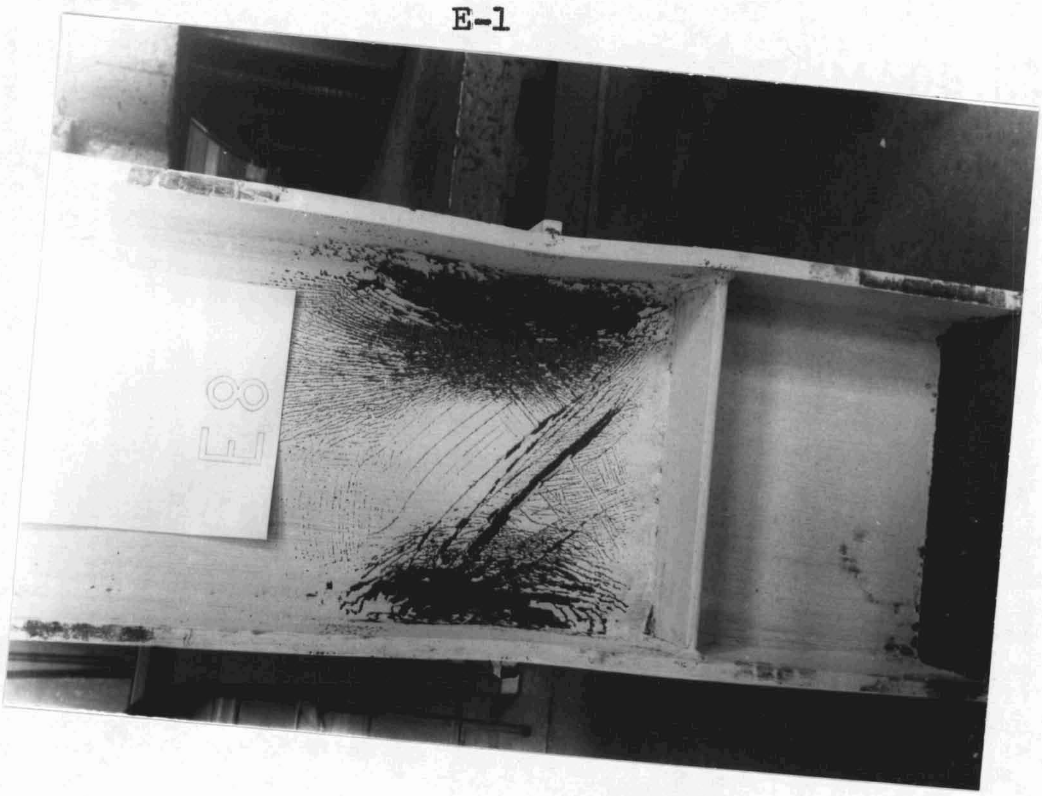


FIGURE 23

TESTS WITH ECCENTRIC STIFFENERS



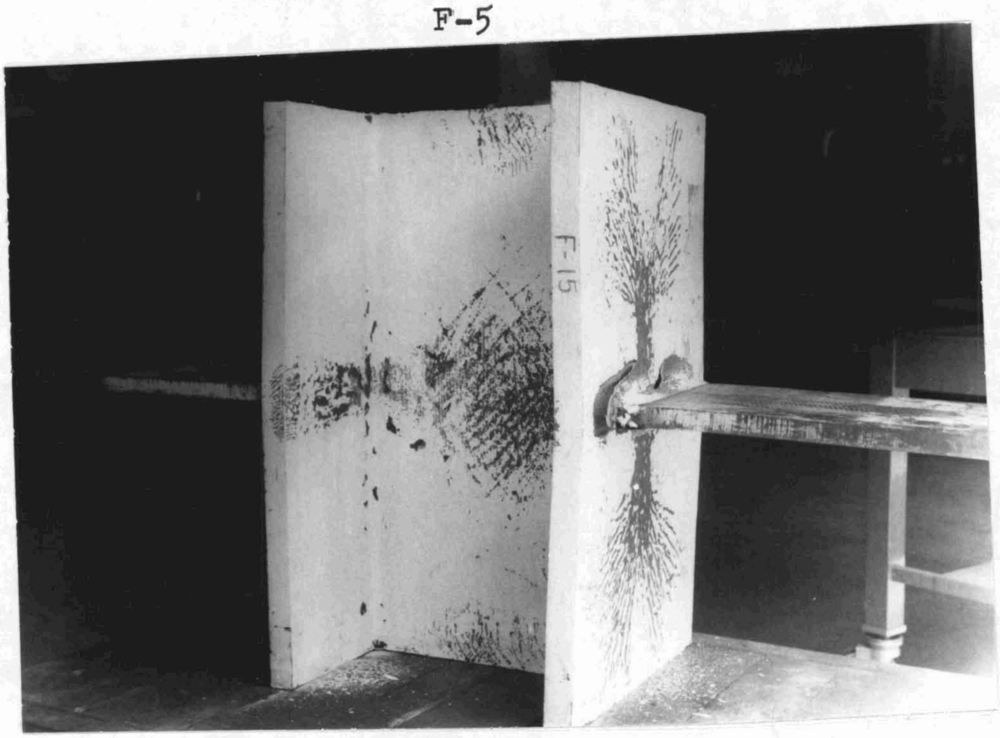
E-1



E-8

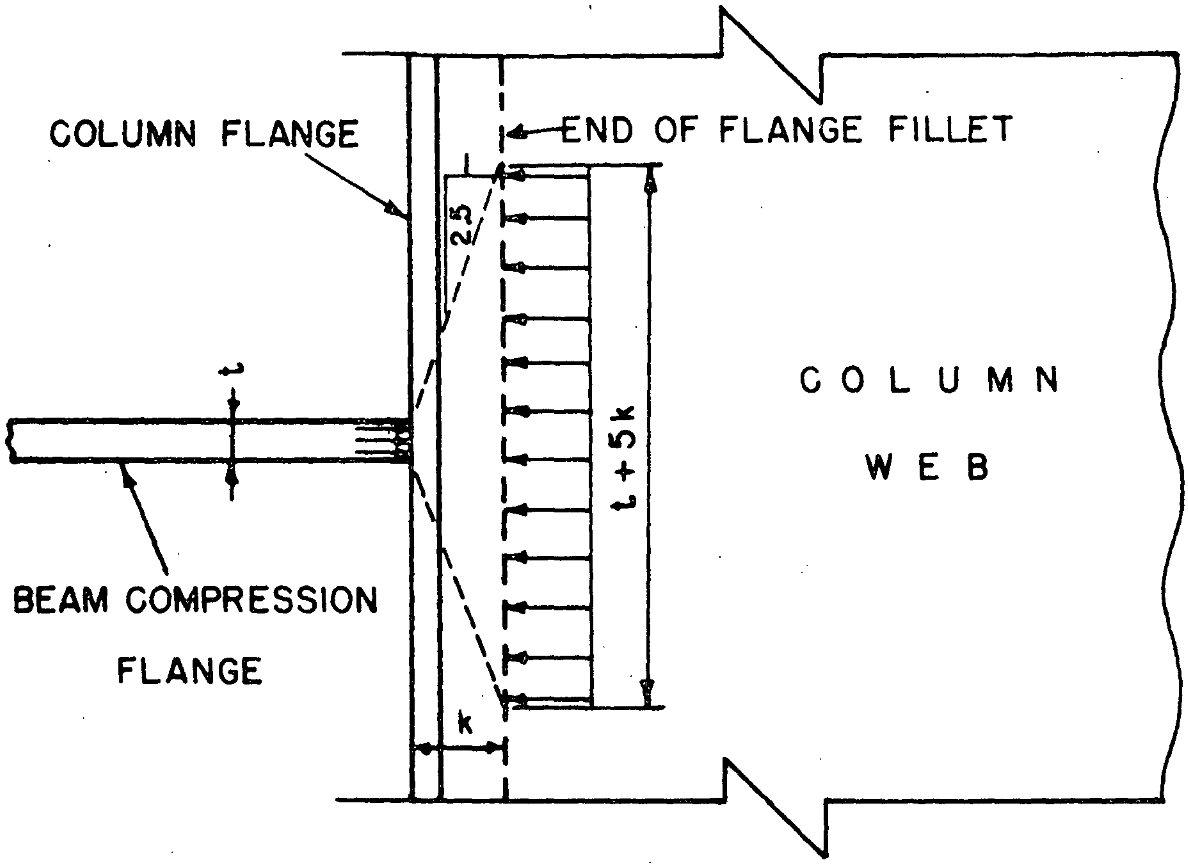
FIGURE 24 - PHOTOGRAPHS OF TESTS, E-1 AND E-8

F-5



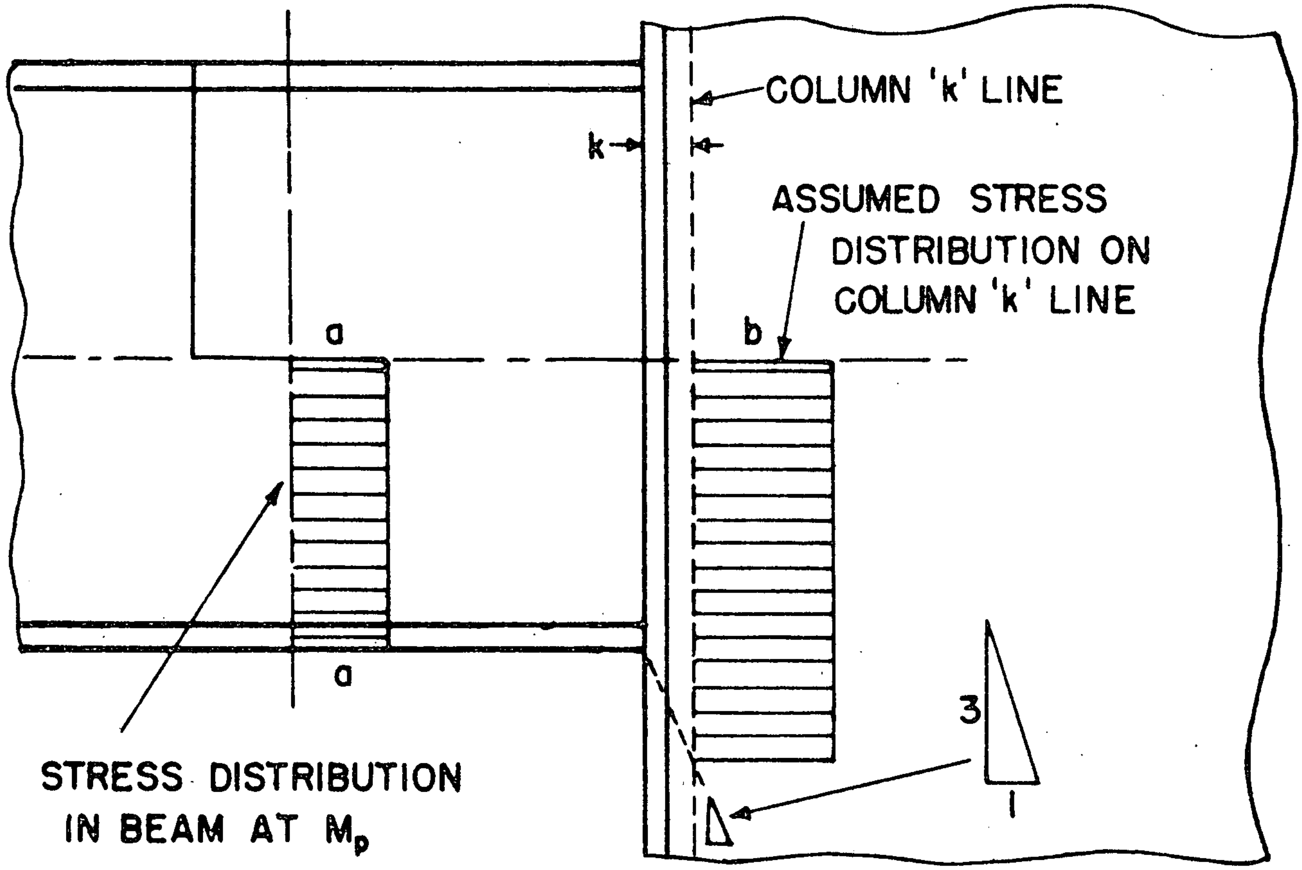
F-15

FIGURE 24 (Cont'd.) - PHOTOGRAPHS OF TESTS F-5 AND F-15

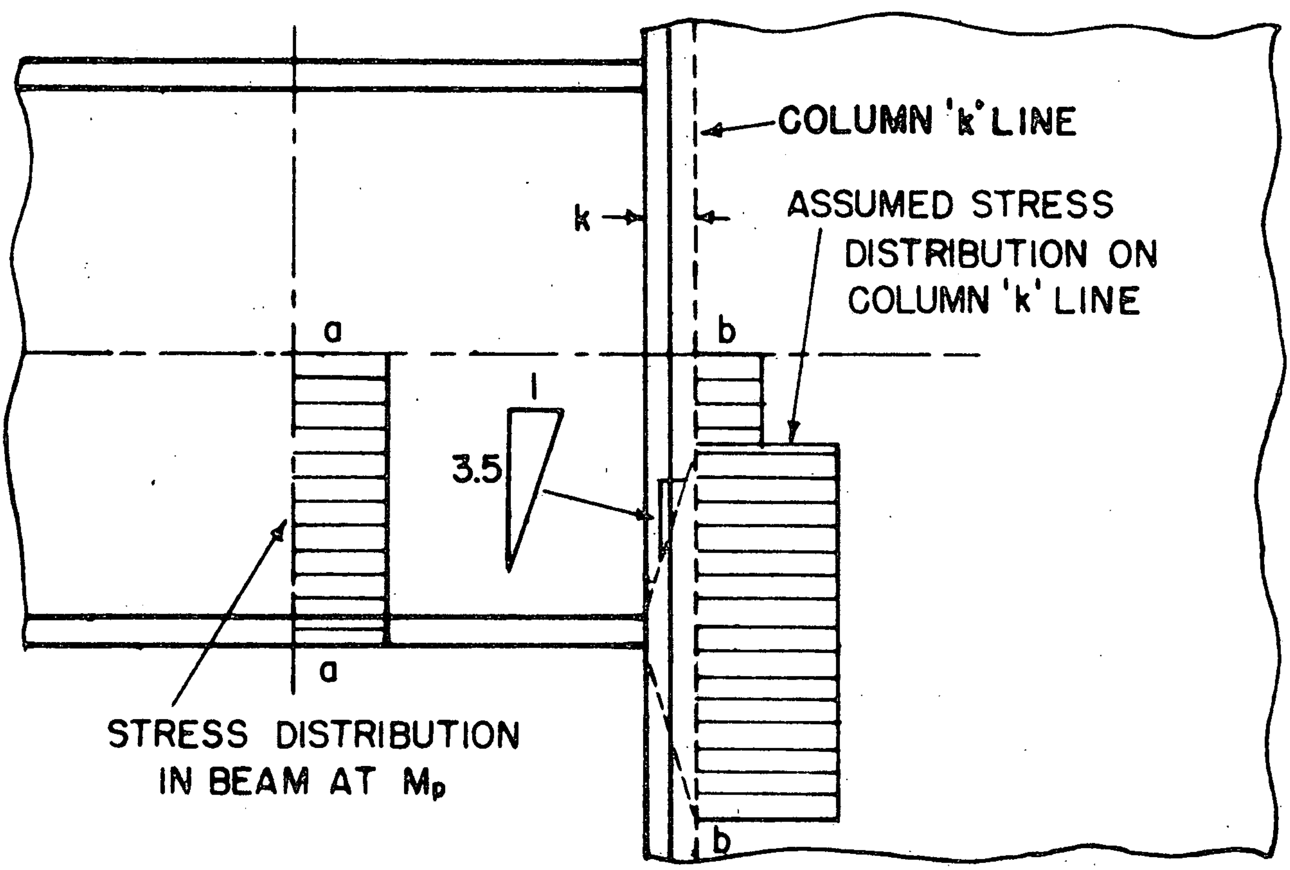


MODIFIED AISC APPROACH

FIGURE 25 — ANALYSIS OF COMPRESSION REGION OF CONNECTION



PLASTIC ANALYSIS APPROACH



MODIFIED PLASTIC ANALYSIS APPROACH

FIGURE 26 — ANALYSIS OF COMPRESSION REGION OF CONNECTION

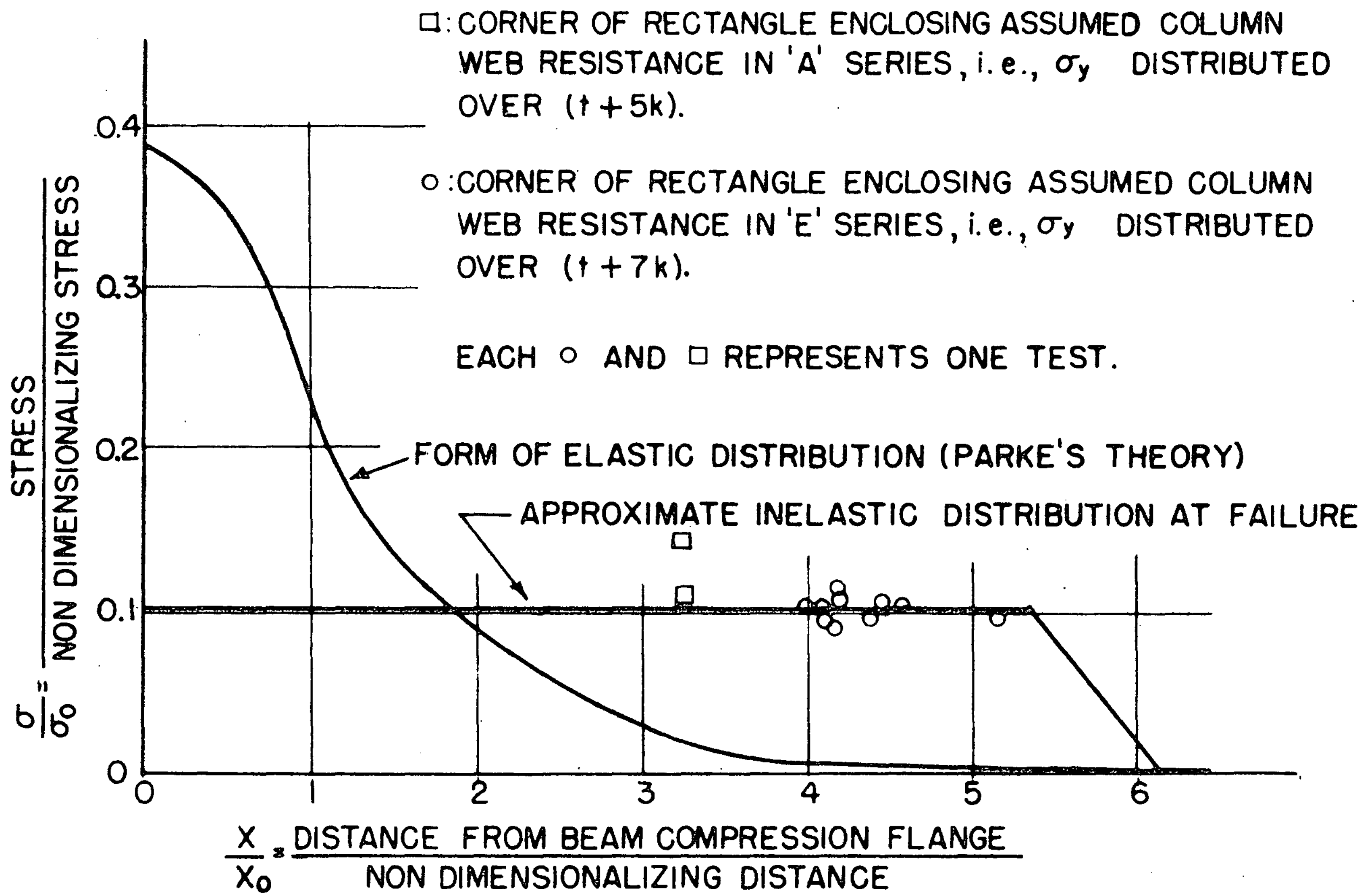


FIGURE 27 —STRESS DISTRIBUTION ON COLUMN 'K' LINE ADJACENT TO BEAM COMPRESSION FLANGE

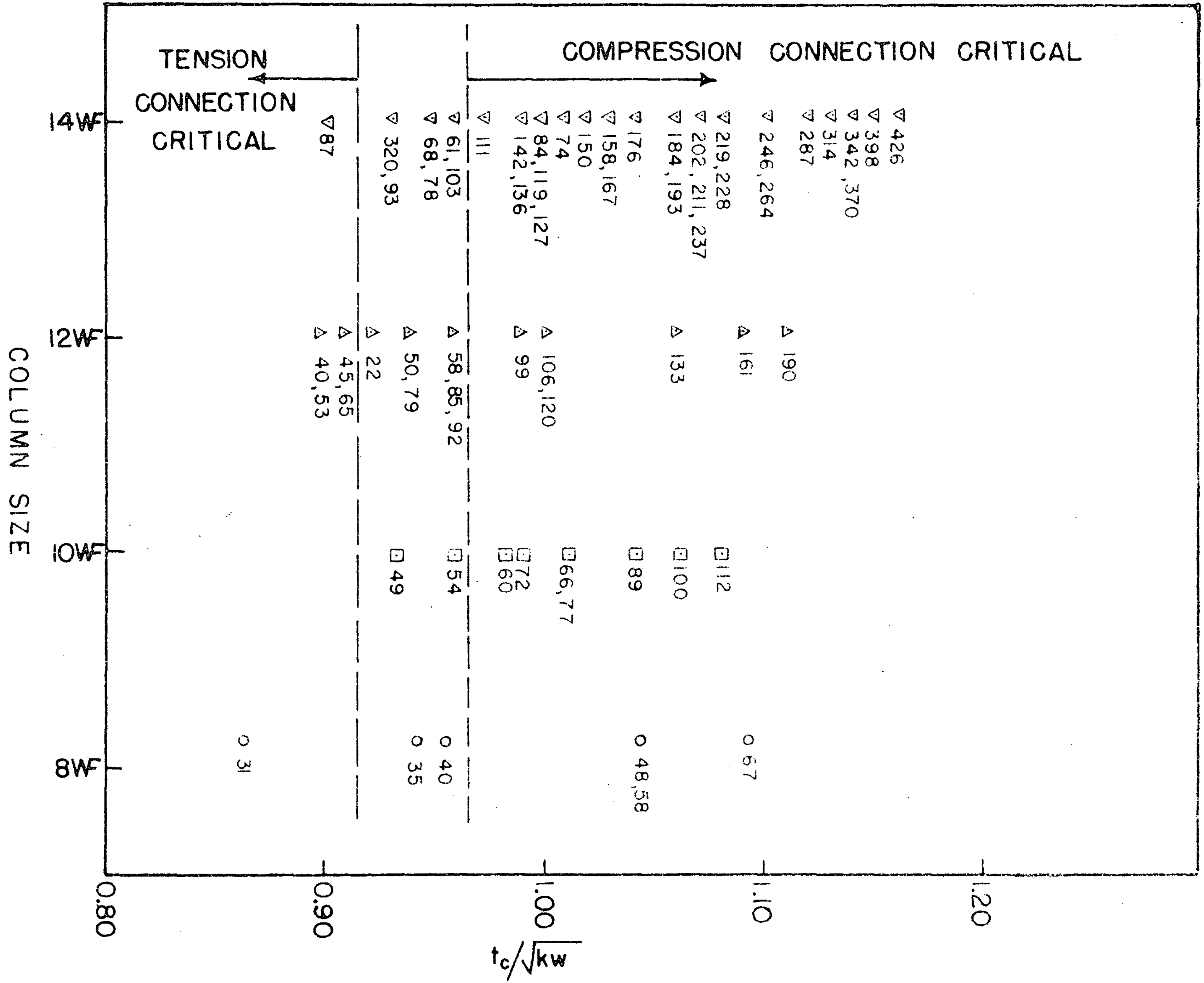


FIGURE 28 - CRITICAL PARTS OF CONNECTIONS BETWEEN STANDARD WF BEAMS AND COLUMNS

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4. George C. Driscoll - "Rotation Capacity of a Three Span Continuous Beam" - Fritz Laboratory Report No 268.2.

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, B.Thurlimann - "Plate Buckling in the Strain Hardening Range" - Progress Report No 20.

7. R.H.Wood - "Yield Line Theory" - Research Paper No.22, Building Research Station, 1955.

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