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WELDED INTERIOR BEAM-COLUMN CONNECTIONS

DIRECT-WELDED TWO-WAY BEAM-COLUMN CONNECTIONS

by

A.N. Sherbourne and C.D. Jensen

(x) Add a list of symbols used

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WELDED INTERIOR BEAM-COLUMN CONNECTIONS

1. INTRODUCTION

Previous researches on restraining beam-column connections have not been carried to the point where definite conclusions, suitable for the designer, could be reached. In particular, information is lacking on the effect of restraining connections on column capacity, and also as to whether or not column stiffening is required and, when needed, how to design it. Information is also lacking concerning the designer's ability to estimate the moment-rotation capacity of a connection, the degree of restraint, and the reserve strength of a designed assembly. This report ~~, confined to the~~ presents information obtained in a study of two-way direct-welded connections. ~~is designed to obtain information on these items.~~

Heretofore, tests of beam-column connections have usually disregarded the axial column loads. In the present program, of which this report is a part, the column is subjected to an axial compression comparable to that existing in practice; further at one stage of the tests, the column is subject to 1.65 times the working axial load, while simultaneously carrying the two-beam working load reactions and moments. At the conclusion of the load applications to the beams, and before the jack loads are removed, the column load is increased to 2.0 times the working load and the specimen is examined for signs of physical distress.

1.1 Previous Investigations

The present program is an extension of work previously carried out at Lehigh University on the subject of beam-column connections. Johnston & Hechtman[1] conducted a project on the study of "semi-rigid" connections, and evolved a method of analysis which effected a reduction of the required section modulus of a beam, (the amount of reduction) depending upon the ratio of its stiffness to the sum of the stiffnesses of all members meeting at a joint, and the degree of restraint developed at the connection. It was found that savings of fifteen to twenty per cent by weight resulted as a direct application of this method. This work was followed by a program of tests carried out by Brandes and Mains which laid the foundation for the present researches being conducted at Lehigh University. A total of seventeen tests were conducted on top plate and seat connections, eleven being of the "semi-rigid" type, i.e., possessing, according to their definition, from twenty to ninety per cent restraint, and six of the "flexible" type, i.e., designed for less than twenty per cent restraint. The criteria used in designing their connections were the following:

1. "The lightest column section into which the beams could frame was selected in order to put the connection at its greatest disadvantage."
2. "The seat was designed to resist the shear by a standard procedure."
3. The top plate was designed to carry a direct force arising from the applied moments on the beams. The magnitude of this force was obtained by dividing the moment ~~of~~ by the depth of the beam section; the butt weld joining the plate to the column flange was designed to resist this same force, as was the fillet weld connecting the plate to the top flange of the beam.

4. The lower flange of the beam was fillet welded to the seat to carry the same force described above.
5. The beam was designed on the basis of fifty per cent restraint at its ends, which, for a uniformly distributed load, gives a midspan moment of WL/12.

Pilot tests were conducted on top plates to obtain a basis for predicting their behavior under actual test conditions, and the specimens were tested by loading a pair of beam stubs which framed into a short column stub. (See Figure 32). No axial loading of the column was considered, and at no time were differential moments set up in the connection. Of course this did not preclude the possibility of one of the beams yielding sooner than the other, due to local variations in shape or material. However, this condition was compensated for by allowing the column stub to rotate freely in space, thus keeping the magnitude of the applied loads equal at all times.

Consequent upon the findings of Brandes and Mains, a pilot test was performed by Pray and Jensen(3) to determine a suitable profile of top plate in the standard plate and seat connections. They attempted to predict the behavior of the top plate on the basis of a simple tension test. Two types of plate were tested: - flared at one end only, and flared at both ends - and a beam-column specimen was fabricated and tested to determine the adequacy of the plate selected on the basis of the tension test. Horizontal stiffeners were introduced into the column to eliminate flange distortion and web crippling. In designing the specimen, Pray and Jensen used the criteria developed by Brandes and Mains in that the beam was designed for fifty per cent end restraint and the connection for seventy-five per cent restraint. However, they went a step further than Brandes and Mains, in that they recognized that the actual connection restraint would be greater than the designed value, and designed the butt weld between the plate and column flange to carry this increased load, flaring out the top plate as necessary to prevent overstressing the butt welds under working load.

1.2 The Present Investigation.

The present report is confined to studying the action of two-way, interior beam-to-column building connections. Attempts are made to simulate actual conditions that exist in a building frame, and column sections were chosen to duplicate these conditions. Axial stresses have also been introduced in the column to obtain a more realistic condition and to determine the effect on the capacity of the connection.

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 should be capable of sustaining a plastic moment in excess of, or at least equal to the plastic moment value of the beams. In elastic analysis, the problem is one of investigat ~~eliminat~~ing such ~~detrimental~~ effects as column flange distortion and column web buckling where light column sections are employed. The types of stiffening used (Fig. 3) were chosen with a view to simplicity, economy, and possible use in four-way welded connections.

Dealt with? Beat down on it.

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 a large rotation or ~~the required~~ "hinge angle."

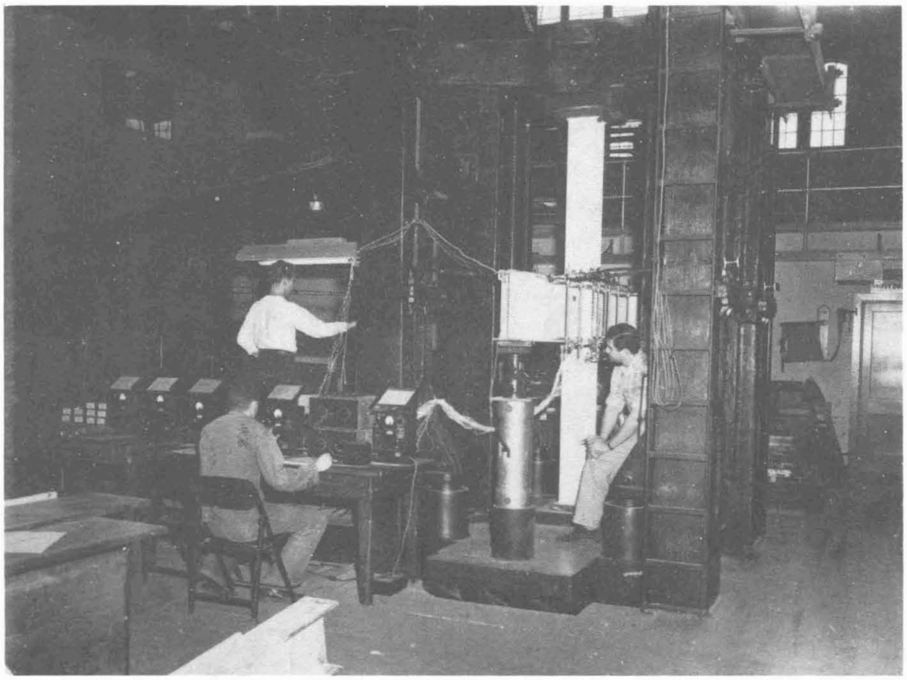


Fig. 1 General View of Test in Progress

c. <u>Reserve strength of the designed assembly</u>. This is a measure of the strength of the connection above the working load. The study of reserve strength applies to both beams and columns; and both members were at times subjected to loads in excess of their working value in order to assess their behavior under these conditions. ~~of overstrain.~~

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 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.

2. TEST PROGRAM

This program consisted of the design, preparation and testing of specimens as shown in Table I and Figures 2 & 3 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 size 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 & 65 columns on the basis of beam and column

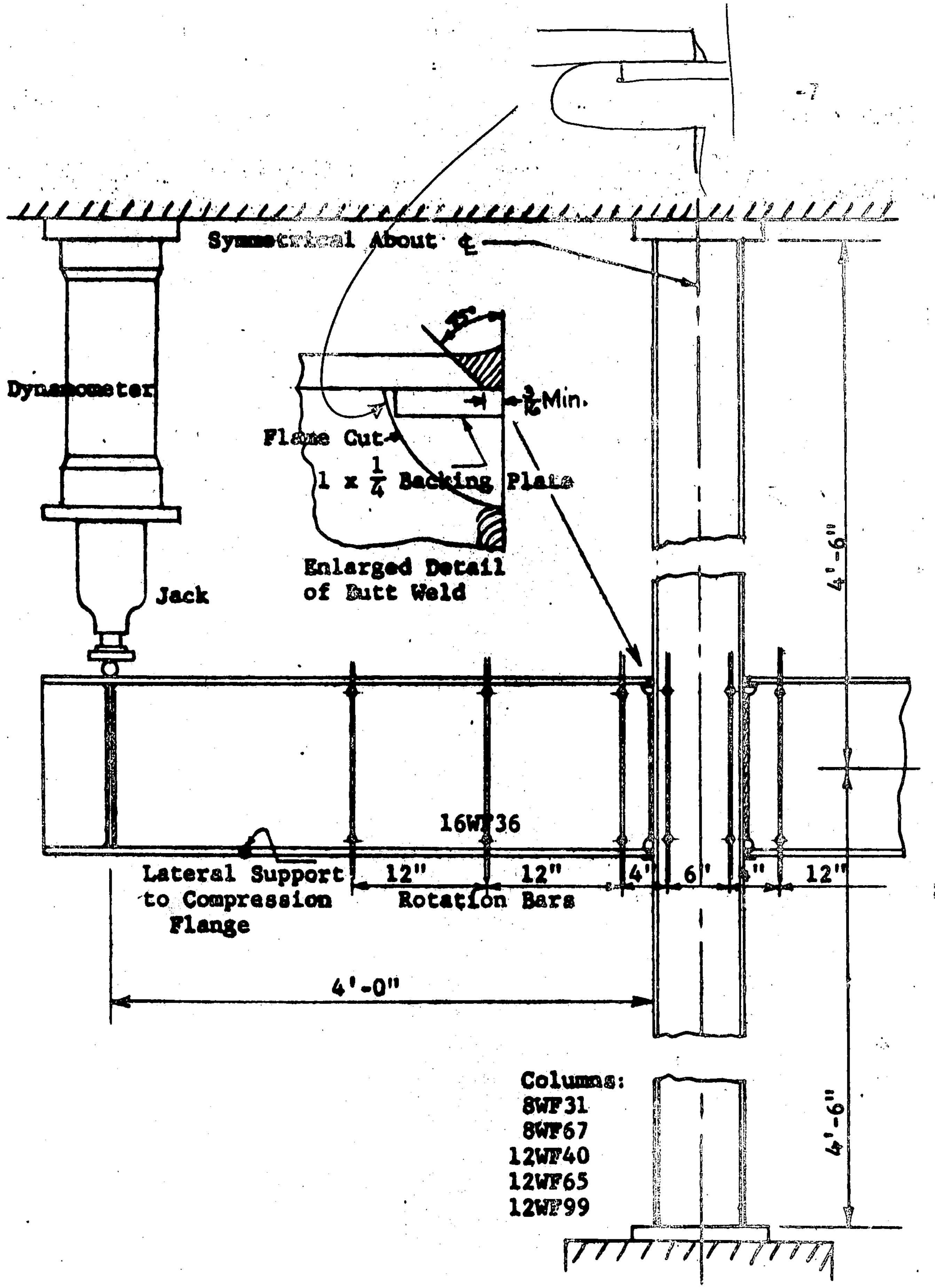
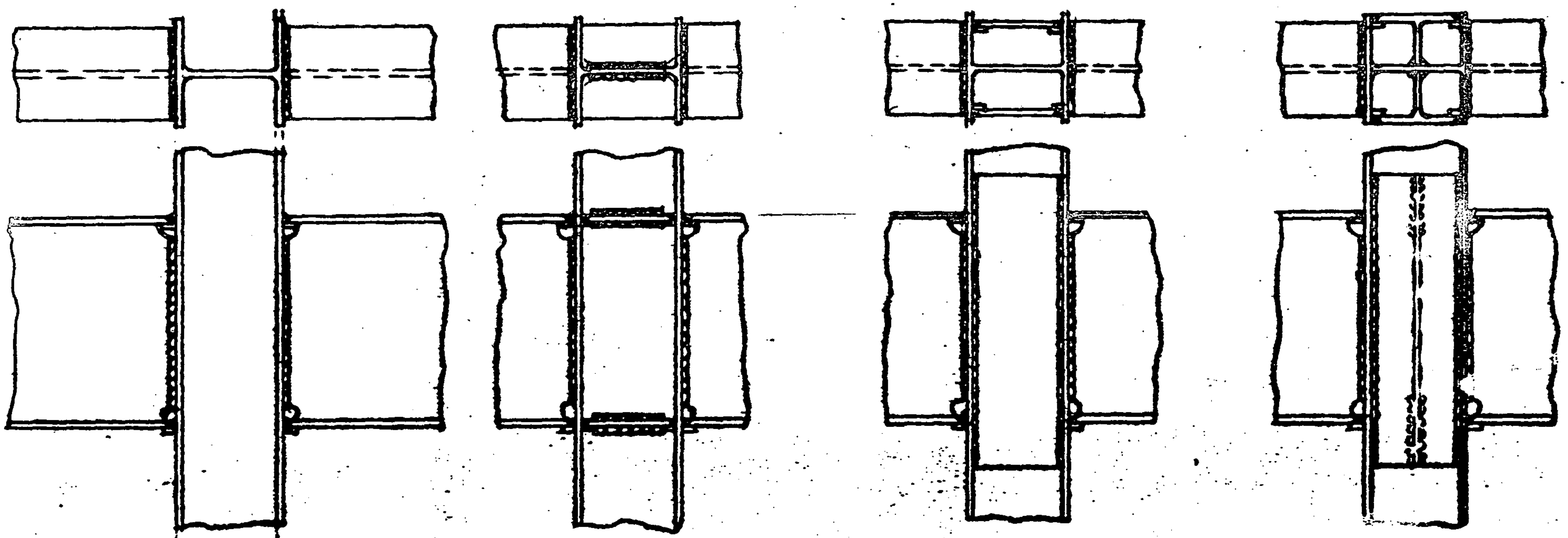


Fig. 2 - TEST ARRANGEMENT - TWO-WAY CONNECTIONS



Series A — No-Column Stiffeners

Series B — Horizontal Plate Stiffeners

Series C — Vertical Plate Stiffeners

Series D — Split WF Stiffeners

Fig. 3 THE TEST SERIES OF TWO-WAY BEAM COLUMN CONNECTIONS 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, ~~as a further argument~~, because it is likely that floor loadings will be constant through successive stories of a building ~~out a building.~~

The test program was divided into four groups of tests depending upon the type of stiffening employed. (See Figure 3). Each group, ~~was~~ designated series A, B, C or D, ~~and~~ will be discussed individually.

<u>Series A</u>

In this group no stiffening was provided and the tests ranged from the very light thin-web 8WF31 column to the heavy thick-web 12WF99 section. The problem was essentially one of crippling of the column web under an applied beam moment, and four tests were performed in this series to determine a critical web thickness beyond which no stiffening need be provided.

<u>Series B</u>

Horizontal stiffeners were introduced across the column flanges at the level of the beam flanges. These stiffeners were initially of a thickness equal to the beam flanges although in a later test in this group, the thickness of stiffeners was reduced after consideration of the initial ~~earlier~~ test results and a theoretical anlaysis. At first sight this would appear to be the "ideal" in stiffening, and test results confirmed the strength of this assembly. However, this type of stiffening becomes complicated when beams of different sizes frame into the column flanges, and horizontal stiffening is to be provided at different levels on the column.

The colum load was increased when

In one of these

Furthermore, when a four-way connection is involved, the amount of welding and fabricating on this type of connection becomes considerable.

Series C

The stiffening provided in this series of tests consisted of plates positioned vertically near the edges of the column flange. The stiffeners were arbitrarily made the same thickness as the column web, thus effectively three column webs were provided to resist horizontal forces from the beams. This type of stiffening was considered with a view to the four-way connection.

Series D

Only one test was performed in this group; the connection was a modification of the C type using split beam stiffeners instead of plates. 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. It might be considered as an expensive detail for ordinary two-way connections. Its merits will be investigated more thoroughly, however, in the forthcoming program of four-way tests.

3. TEST ARRANGEMENT

The specimen consisted of two 16WF36 beams stubs welded directly to the flanges of WF column sections as shown in Figure 1 & 2. Stiffening was provided as indicated in Figure 3, and the column and stiffener sizes were as indicated in Table I.

All testing was done on an 800 kip Riehle screw -type universal testing machine which applied axial load to the columns. ~~A working axial stress of 14.5 ksi was maintained in the column through-~~ out the test except at two stages, when states of overstress were induced in the column concurrently with working stresses in the beams under working load in one case and ultimate load in the other. The column working

TABLE I 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 stiffeners, at level of tension and compression flanges	3.9" x 7/16"
B-8	12WF40	0.294	0.516	"	"	"		3.9" x 1/4"
C-9	8WF31	0.288	0.433	"	"	"	Vertical plate stiffeners at edges of col. flanges	5/16" x 22"
C-11	12WF40	0.294	0.516	"	"	"		5/16" x 22"
D-12	12WF40	0.294	0.516	"	"	"	Split tee stiffener	ST6WF325 22" long

* Indicates AISC Handbook Value

stress was derived from a consideration of the AISC formula for axially loaded columns with $l/r < 120$, viz. $\sigma = 17000 - 0.485\,(L/r)^2$. An L/r of 72 was selected as a nominal value, and the axial stress of 14.5 ksi computed ~~on this~~ indicated the required ~~basis was reduced to a~~ working load on each column.

The beam stubs were each 4' 6" long and directly welded to the column flanges as shown in Figure 2 through butt welds at the beam flanges and fillet welds at the beam web. The point of load application on the beams was at a distance of 4' 0" from the face of the column flange corresponding to the point of inflection for a fully fixed-ended beam over a span of approximately 20' 0" (L/d = 15).

Fig. 4 Specimen B-8 As Welded, Showing Rotation Posts

darken it out

Fig. 5 Specimen D-12 As Welded, Showing Split Beam Stiffener

~~Beam flanges were butt welded to column flanges~~

The test specimen was inverted in the testing machine to permit the use of mechanical compression jacks which were mounted on dynamometers (see Figures 1 ~~& 2~~). The dynamometers, in turn, were set on bearing blocks seated on the table of the testing machine 2. In this way, with column load, P, applied through the testing machine and beam load, V, applied through each jack, the one portion of the column sustained load P while the other portion sustained a load of P-2V. ~~Upon increasing V, failure due to high beam loading was finally obtained.~~

?

3.1 Fabrication

~~The columns were fillet welded to base and cap plates before insertion in the testing machine.~~ Columns and Beams were flame cut ~~to 4'-6" lengths~~ ~~and the flanges given a 45° bevel~~ beam webs end coped ~~to receive the butt welds.~~ All welds ~~were in accordance with AWS specifications and all welding was done by qualified personnel.~~ Figures 4 and 5 are close-up views of two of the connections after welding. The sequence of fabrication was: (A) next pg. ~~at end~~

~~1. Beams and columns were flame cut to length.~~

2. Base and cap plates were fillet welded to the columns; 3/8" plate stiffeners were welded to the beams at points of load application; ~~beam flanges bevelled at 45° to receive butt welds;~~ cut-outs were made at junction of beam web and flange to allow for insertion of back up ~~plate~~ bar for the top flange butt welds and for welding past the web in the bottom flange weld.

3. Beams were fitted ~~square~~ to column flanges; 1" x 1/4" back-up ~~plates~~ bar were tack welded to beam flanges with 3/16" min. root gap; ~~plates carried out~~ back-up bars extended beyond edge of flange.

4. Beam flanges were butt welded the to column flanges each ~~Butt weld formed at beam flange-column flange interface;~~ with the column in an upright position. The 1/4" fillet welds ~~at~~ connecting the beam web to the column flanges ~~interface~~ were made in the horizontal position. ~~All Welding was made in downhand position using~~ 3/16" dia. E6020 electrodes.

~~This~~ This welding was done by qualified welders using

13A

5. Posts for rotation dials and lateral supports were welded in position.

3.2 Strain Measurements

Extensive use was made of electrical strain resistance gages (SR-4) in determining strain distribution throughout the connection. (For type and positioning of the gages see Figures 6 to 9). Attention was focused on the following points of interest:

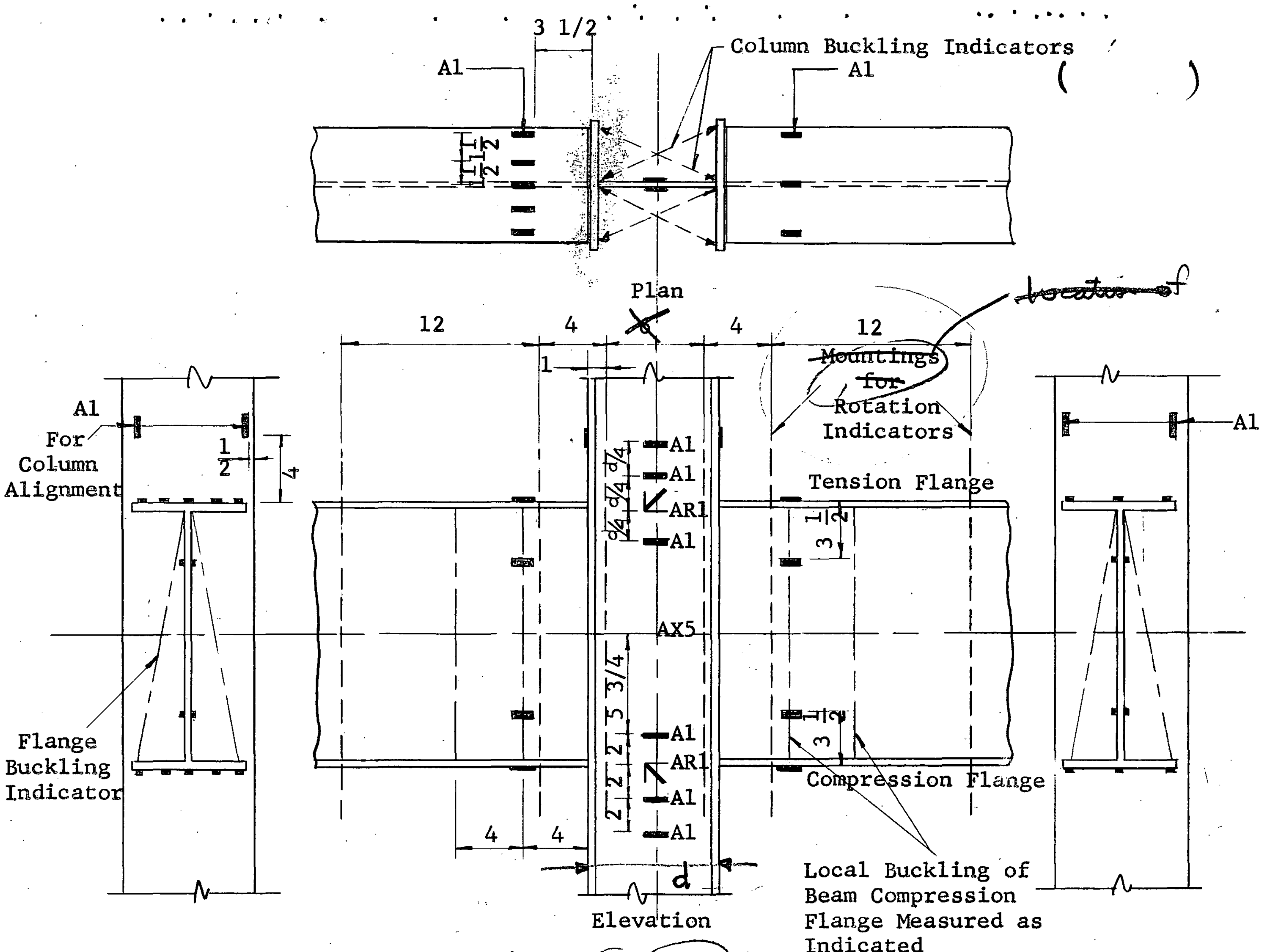
a. Strain distribution in beams: A-1 type gages were used to measure strains in beam flanges and webs. The gages were positioned as near to the connection as possible, yet far enough away as to avoid any local effect arising out of residual stresses in the butt welds.

b. Strain distribution in column web: In ~~early~~ initial tests type AR-1 strain rosettes were used in evaluating principal strains in the column web. These were later replaced by type AX-5 strain gages which measured strains in a vertical and horizontal direction only.

c. Strain distribution in stiffeners: Type A-1 gages were employed in the B, C and D series of tests. In the B series the strain distribution was mapped across the tension and compression stiffeners; in the C and D series the distribution of horizontal strains was mapped on or near the center line of the vertical stiffeners.

3.3 Deflections

A theodolite ~~and~~ sighting scales ~~were~~ was used to determine the vertical deflection of the beams and also the axial shortening of the column. Scales were mounted vertically on the column flange and also on the beam stiffeners above the point of load application and were read to the



SERIES A

FIG. 6 - Instrumentation Plan — A

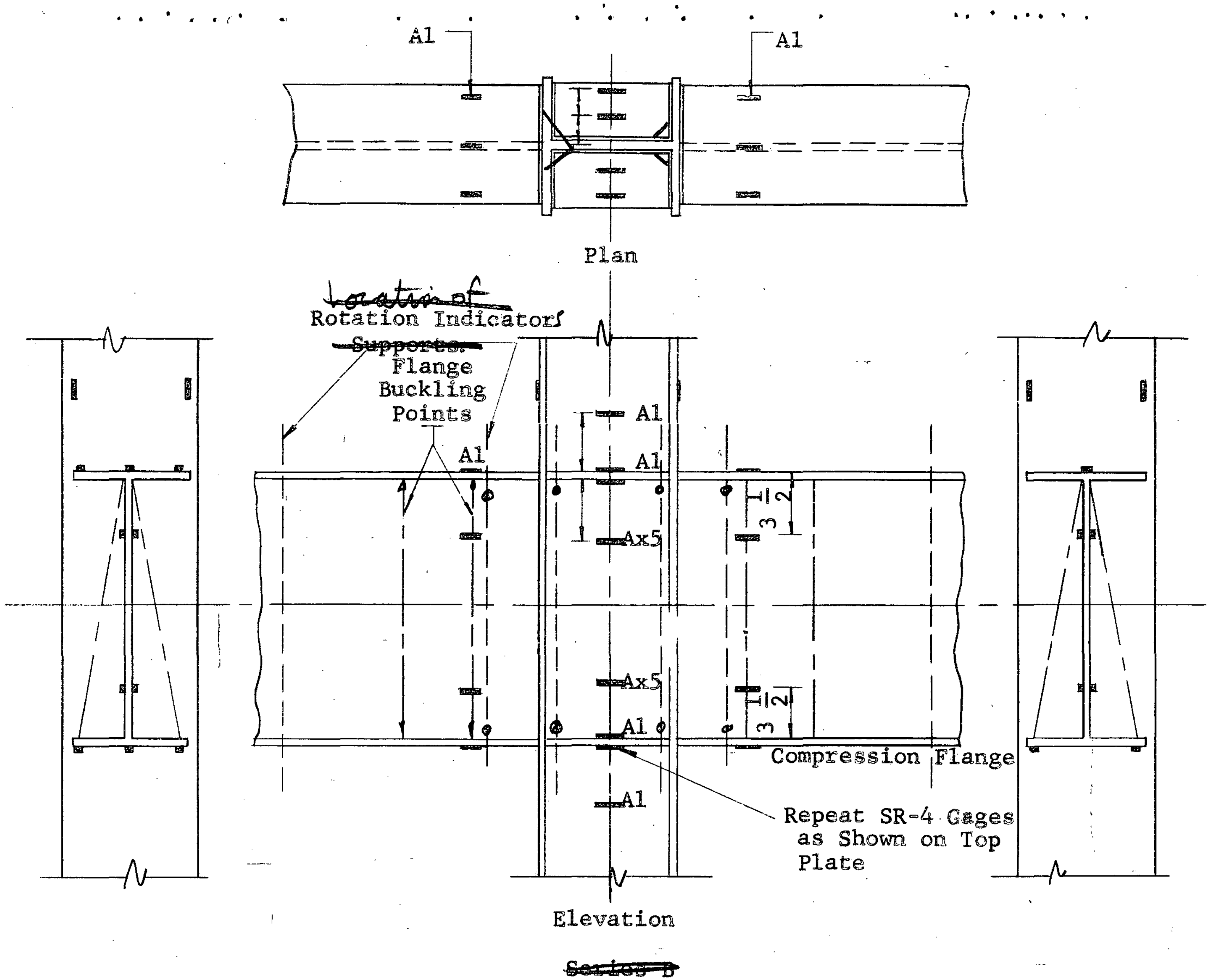


FIG. 7 - Instrumentation Plan - Series B

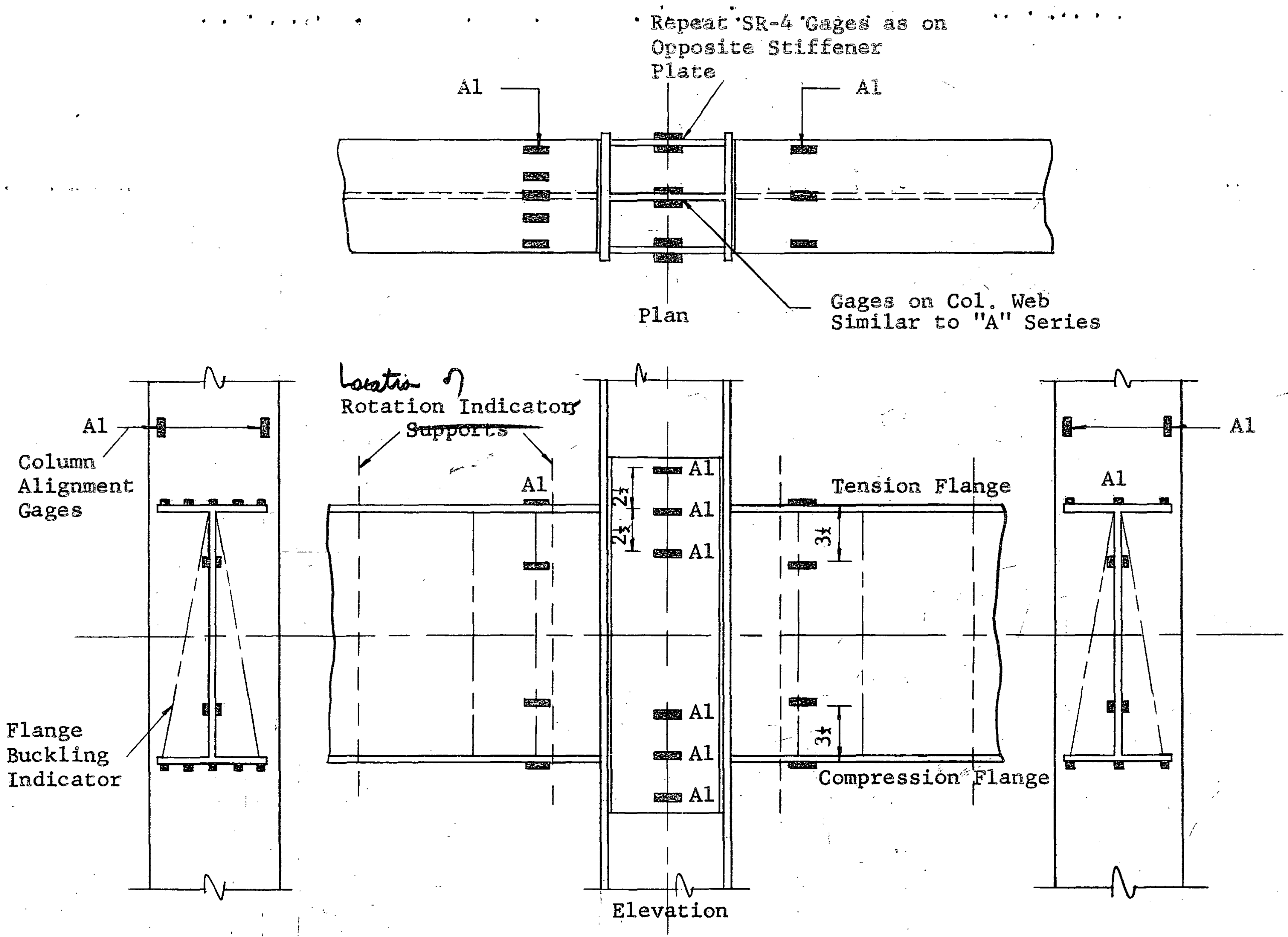


Fig. 8 - INSTRUMENTATION PLAN - SERIES C

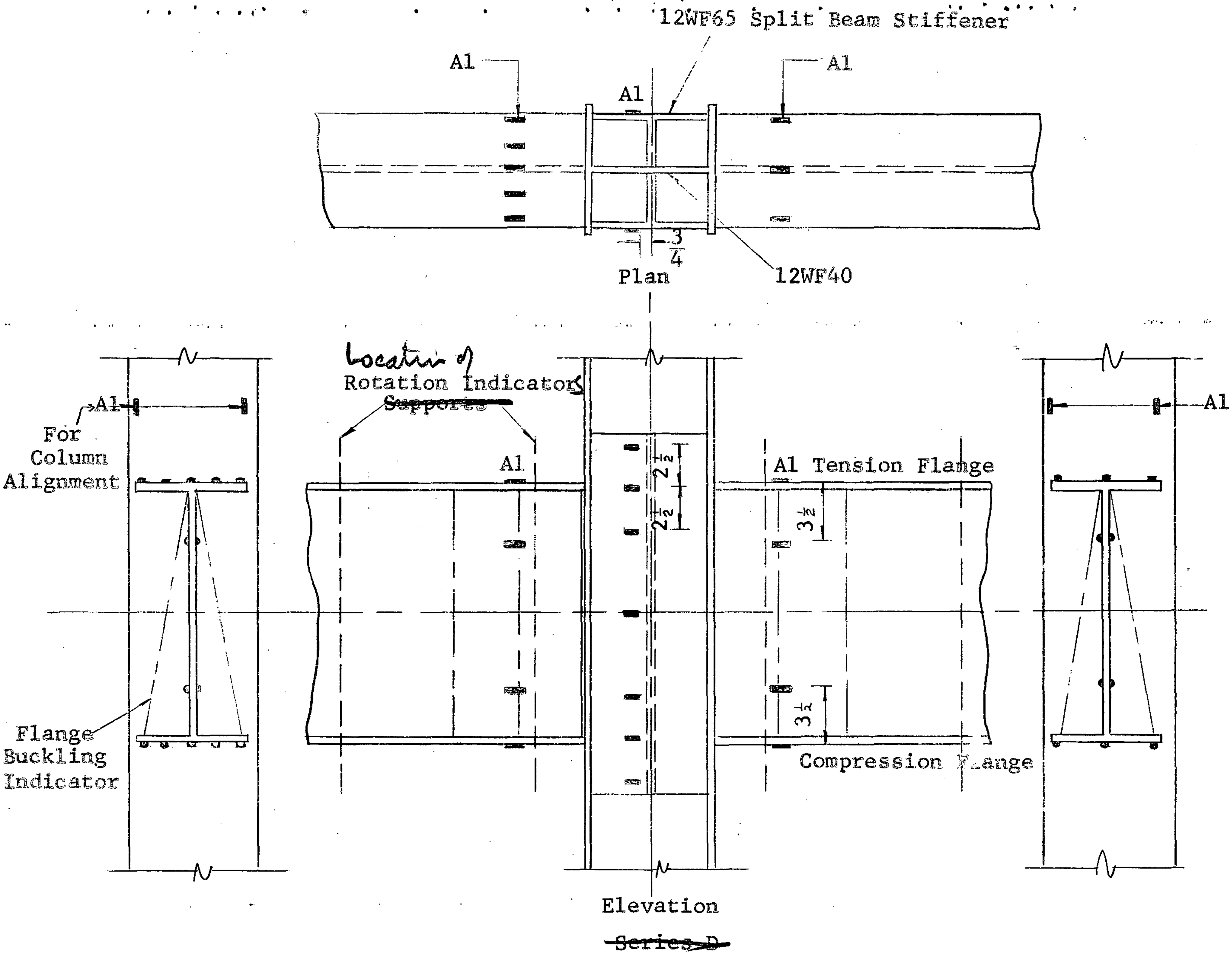


FIG. 9 - Instrumentation Plan - Series D

nearest 0.01". Deflections were observed through the telescope of the theodolite with reference to a horizontal plane maintained by sighting on a bench mark some distance from the specimen.

Lateral deflections of the column, resulting from an increase in the axial loads, were observed by noting the deviation of a horizontal scale from a vertical plane through the theodolite and the bench mark.

3.4 Local Buckling

Local buckling of the beam compression flanges also and distortions of the column flanges for Series A were measured ~~using a dial gage~~ with, in the locations ~~noted~~ shown in Figures 6 to 9. In preparation for measuring these ~~values~~, center punch marks were made on the flange near the outer edge and on the fillet between web and flange. A portable dial gage indicated ~~any~~ change in the distance between these two points, which was a measure of the local buckling.

~~In~~ for the B C and D series ~~of tests, buckling~~ The measurements ~~were confined to the compression flanges of the beams~~ of column flange buckling was ~~eliminated~~ omitted since the use of ~~vertical~~ stiffeners ~~eliminated~~ prevented measurable column flange distortion.

In all instances where local buckling of the beam compression flange was observed, readings were taken at a distance of 4 and 8 inches from the face of the column flange. Delete

3.5 Lateral Buckling

Lateral support was provided by anchoring the compression flange of the beams to the frame of the testing machine at points 3' from the column as shown in Figure 1 and 2. No other support was provided other than that inherent in the column with its "fixed" ends. Dynamometers were inserted in the supports to measure the intensity of the lateral forces produced; the supports were also fitted with turnbuckles to permit adjustment. Although the Ld/bt value of the ~~particular beams chosen~~

(Rotations measured in
12-inch gage lengths in beam
were used only for control
of testing procedure and
does not appear in this
report.)

16WF36 ~~was below~~ was below 600, it was believed that lateral buckling might become a problem in the plastic range. However ~~As it turned out~~ the maximum lateral force measured was of the order of 2 to 3 kips. ~~(Note that~~ Based on $\sigma_y = 33$ ksi., this lateral force is 2 to 3% of the force in the top flange, and ~~this~~ ~~which agrees~~ is consistent with previous ~~tests)~~ observations.

3.6 Rotation Measurements

The ~~overall~~ rotations of the connection ~~was~~ ~~and the rotation of the column web was~~ were measured using dial gages, mounted horizontally, as shown in Figs. 1 and 15 ~~(specimen C2)~~. In the A and B series ~~of tests~~, the portion of the rotation within ~~rotation of both~~ the column web and the portion of rotations in each connection were measured. In the C and D series, only ~~the~~ overall rotations ~~of the connection was~~ were measured.

portion of this

Ref.

Average unit rotations were expressed as angle changes in radians per inch. The angle change was determined by adding the changes in upper and lower dial readings ~~due to tensile and compressive effects~~ and dividing by the vertical distance between the dial gages. To obtain the rotation per inch, the angle change was divided by the horizontal distance between dial supports.

Rotation data was taken through the elastic and part way through the plastic range~~s~~. Readings were discontinued when local buckling of either the beam flange or web at or near the rotation post caused obviously erroneous results.

3.7 Test Procedure

Before proceeding with the 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 readings was about 10% at full column working load.

The sequence of loading in the test was arranged in ~~four~~ five stages as follows:

(1) The column load was increased in five equal increments to working load, Pw, with no load on the beams. (This load was the same in the "upper" and "lower" portions of the column).* See ? on p 13

(2) The beam load was increased in four equal increments to working load, Vw, while maintaining working load, P_W at all times in the portion of the column "below" the beams. At the conclusion of this stage ~~In this condition~~ the "upper" portion of the column sustained a load equal to P_W-2V_W where Insert from page 11

P_W = the column working load and

V_W = the applied beam working load

(3) 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 ~~with working load maintained in the beams.~~ The column load was subsequently reduced to working load in the "lower" portion. This left the specimen under the same load ~~and the connection was under a condition of loading similar to~~ that ~~which~~ existed at the end of stage 2.

(4) The beams were loaded in increments until "failure" occurred ~~of the connection was reached.~~ 22A

(5)

In applying beam loads in the elastic range, the jacks were pumped evenly in increments of 1 ~~kips~~ kip at a time. Readings of strain, deflection, buckling, and rotation were taken in 6 kip increments of beam load. Beam loads were brought up to 5 kips, the column load was

* [footnote added to define "upper" and "lower"

adjusted to working load by varying the load applied by the testing machine, and the final kip was added on the jacks. No difficulty was experienced in the elastic range in realizing predetermined load increments on beams or column. In the inelastic range however, where creep was quite evident, a criterion of $\pm$ 0.10 kip drift in five minutes was used as the maximum allowable variation in beam load before measurements for a particular load increment were taken. Also column loads were adjusted to a tolerance of $\pm$ 1 kip where increments of axial load were applied.

At all stages of testing, equal loads were maintained on both beams to keep the connection under equal moments and shears. However, in a few cases where it was evident that one beam was stronger than the other, a criterion of equal deflection was used to determine the load deflection relationship of each beam. This was particularly necessary at loads in excess of the predicted ultimate load of the beams.

Control curves of beam load vs. end deflection and column load vs. lateral deflection were maintained throughout the test, to show the state of deflection of the beams and to indicate any excessive buckling of the column.

As a last step in the testing with the connections damaged as shown in Figures 14 to 16 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.

22A

Put on page 1

4. TEST RESULTS

Of the data obtained from the nine tests conducted in this research program, curves are plotted illustrating the following points of interest with regard to the behavio~~u~~r of the connections.

~~1. Summary of test results. Beam load vs. beam deflections.~~ end

1 ~~2~~. Beam load vs. beam ~~end~~ deflection~~s~~.

2 ~~3~~. Moment-rotation relationships for the welded connections and the column webs.

3 ~~4~~. Stress distribution in beam flanges and resulting distribution in horizontal stiffener plates.

4 ~~5~~. Distribution of the horizontal strains in the column web and in the vertical stiffener plates.

~~5 6. Summary chart. Moment vs. total joint rotation.~~

The summary of the beam deflections, Figure 10, is expressed in terms of a non-dimensional curve of applied beam load, V/V_y, versus end vertical deflection, d/d_y, where V_y and d_y are respectively the theoretical beam load and beam deflection at first yielding of the beam. In Figure 10, average values of load and end deflection for each pair of beams are plotted, while in Figures 11 to 13 the actual experimental values of load and deflection for each beam are plotted. These latter figures illustrate the variation in behavior between the two beams of a pair and also indicate the type of failure experienced in each test. The actual load-deflection curves were plotted as the data was obtained, and served as a control.

The connections were said to have failed and formal testing was concluded when the beam strength decreased to approximately 85% of the ultimate load. The single exception occurred in test C-11 when a weld failure was experienced at one end of the tension flange butt welds owing either to failure of the weld~~o~~er to weld out completely onto the run-out pad or to stress concentrations caused by the stiffener.

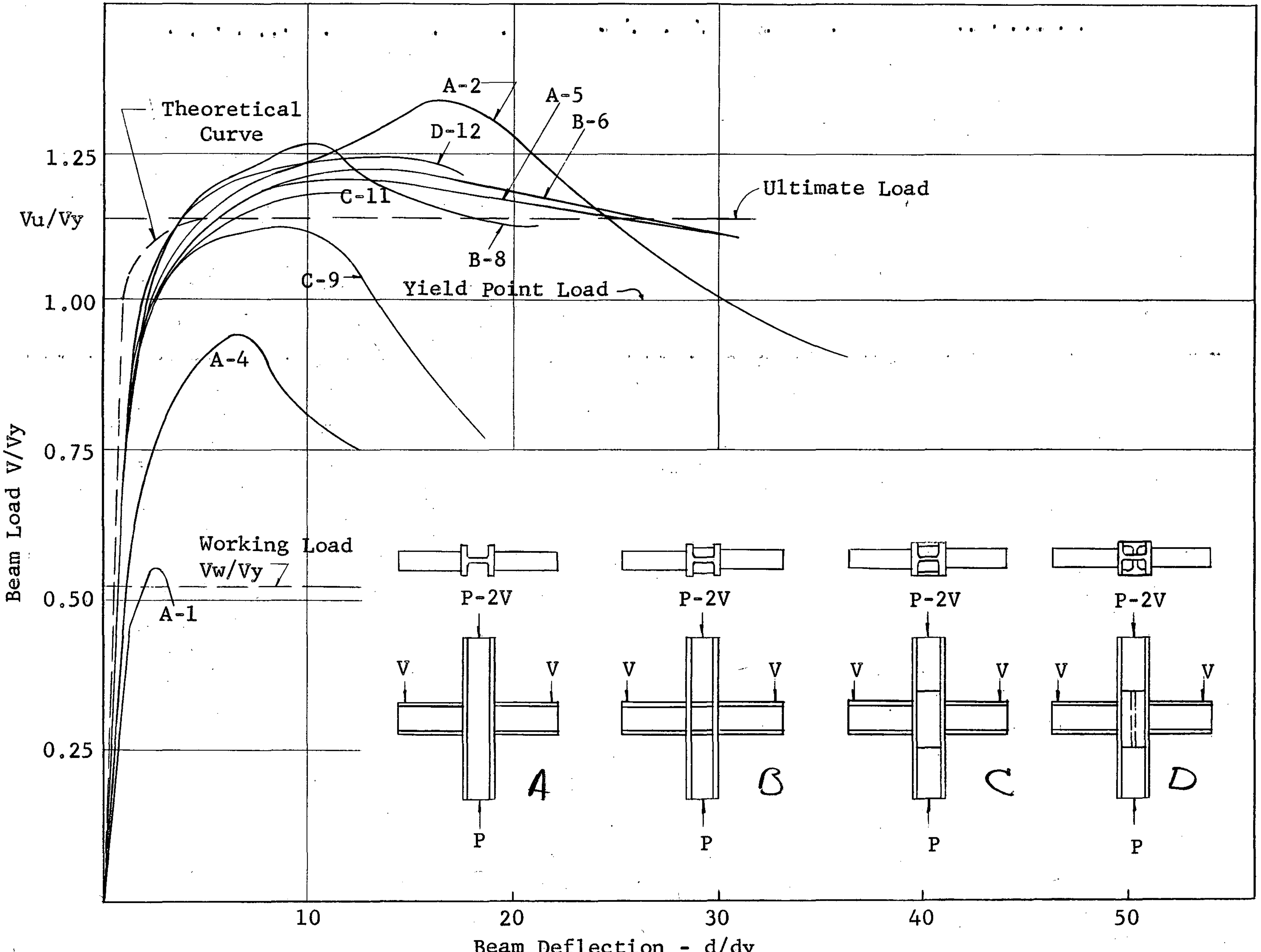
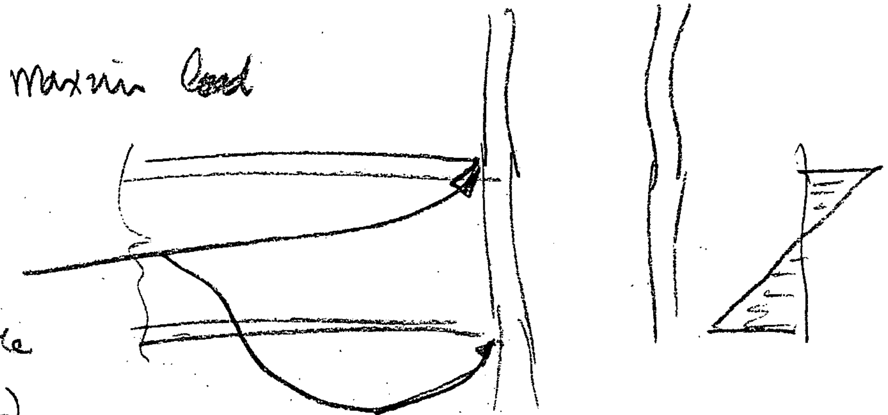


FIG. 10 - Summary of Test Results: Beam Load Vs. Beam Deflection

Design requirement of the beam at maximum load



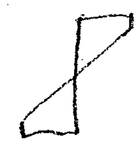
$\sigma_{max} = 33$ ksi

For comparison with a test to check out the rule:

$$V_y \times 48'' = \sigma_y S \cancel{= (39.2)}$$

$$V_y = \frac{(\cancel{39.6}\,39.2)(56.3)}{(48.0)} = \cancel{46.5}\ 46.0 \text{ K}$$

What load is on the beam stub when the flange is loaded to the point assumed in the theory



$V > 46.0 +$

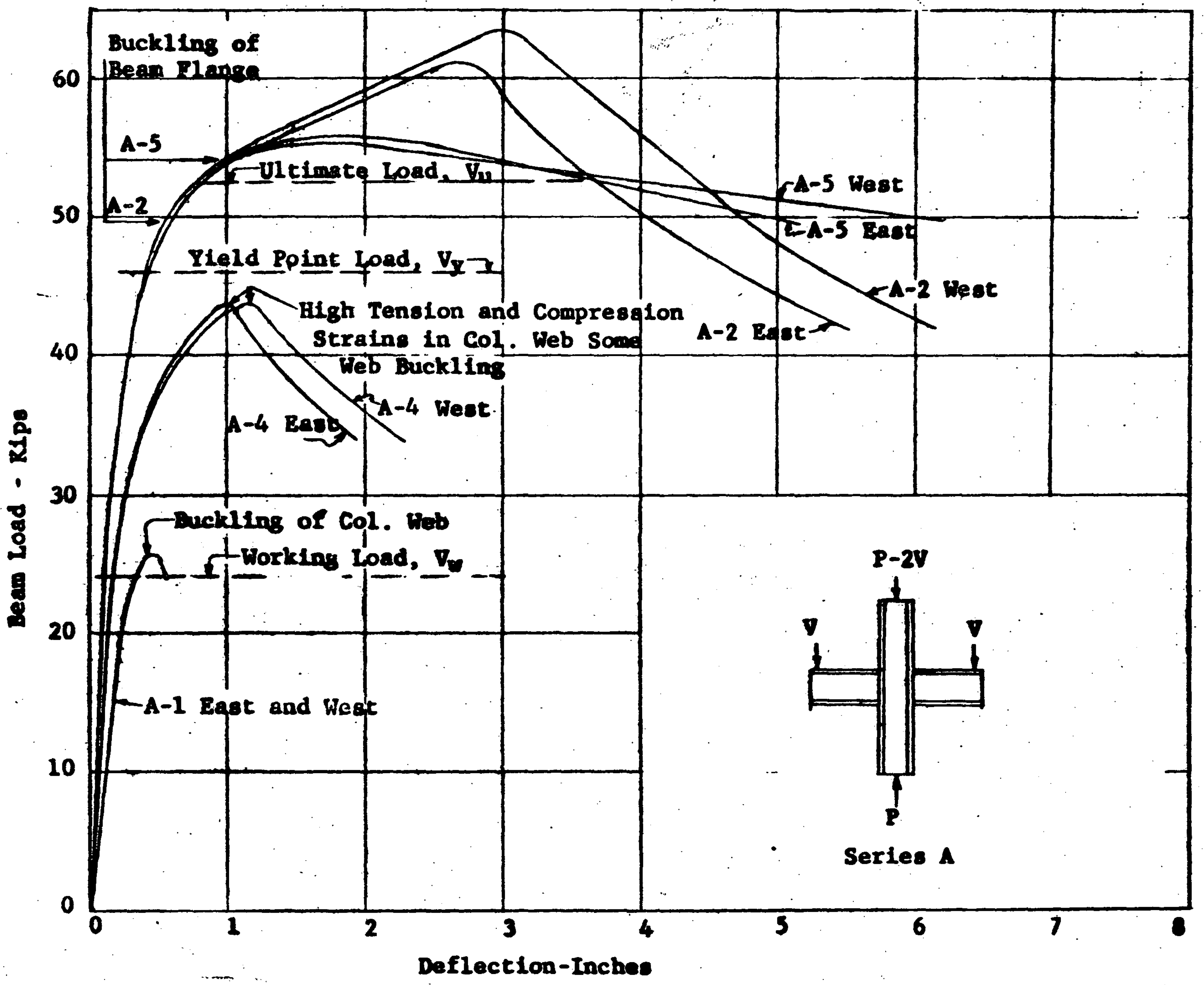


Fig. 11 - BEAM LOAD VS ~~END~~ Beam DEFLECTIONS: SERIES A

$46 \times \frac{33}{39}$

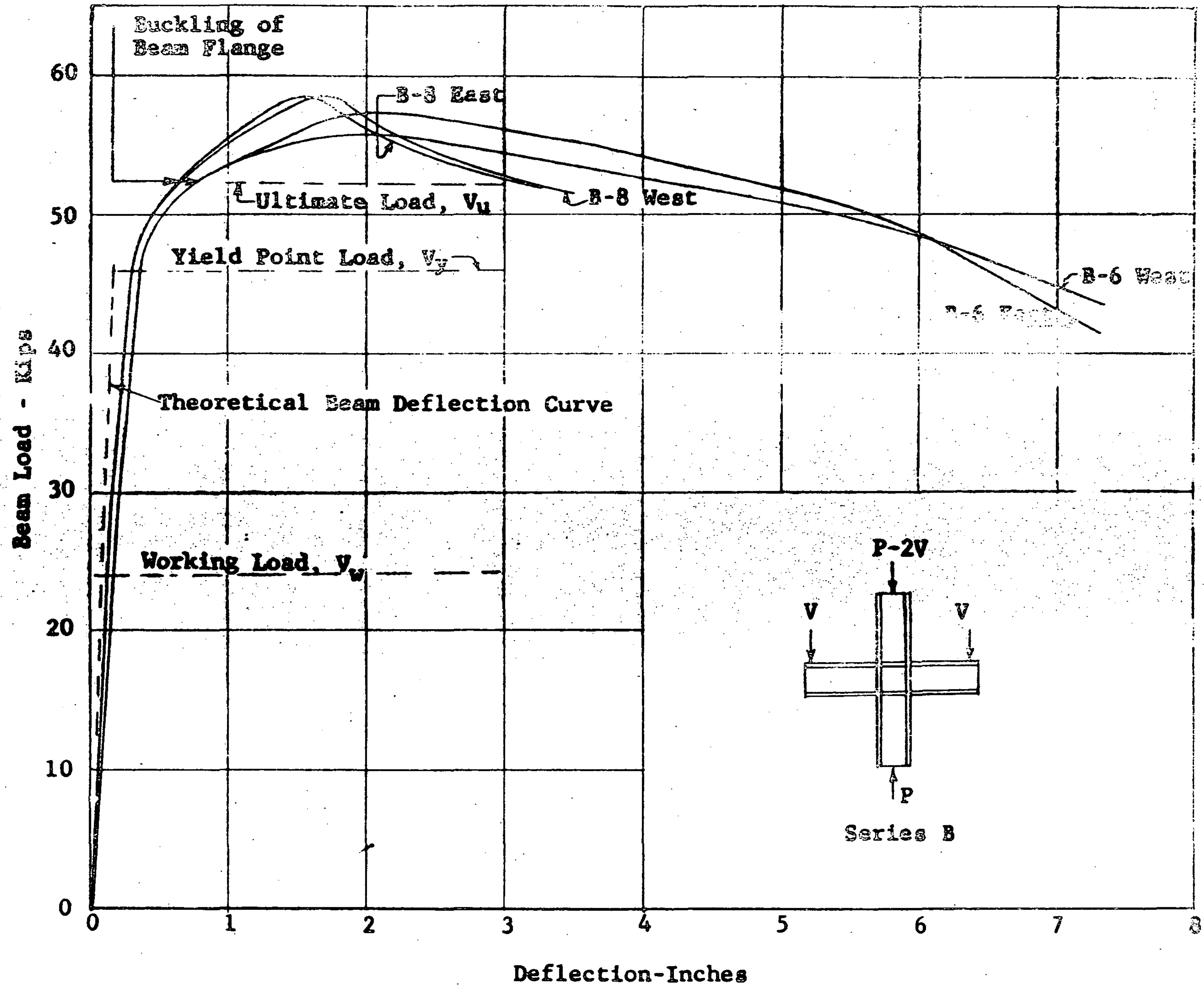


Fig. 12 - BEAM LOAD VS END DELFECTIONS

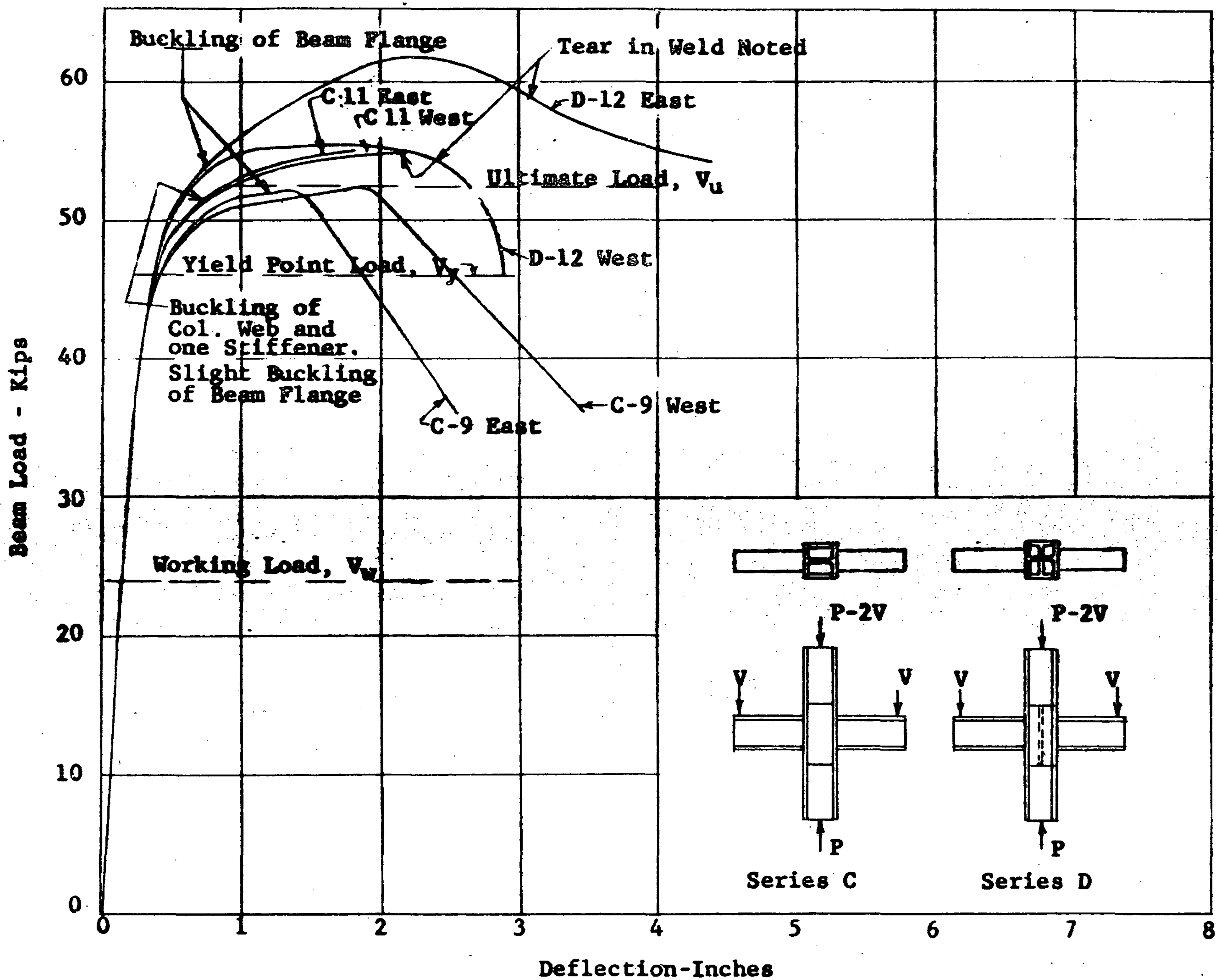
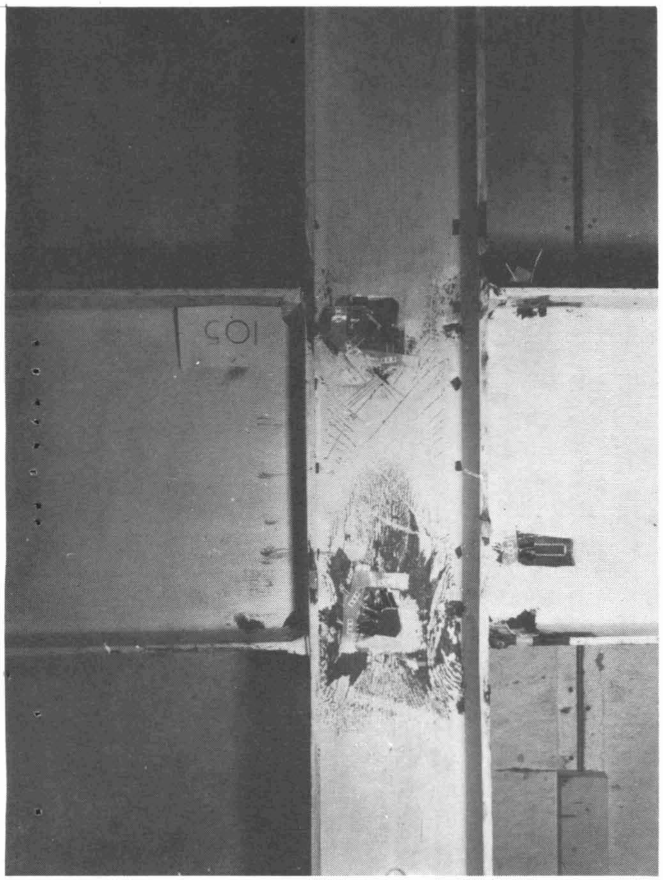


Fig. 13 - BEAM LOAD VS END DEFLECTIONS

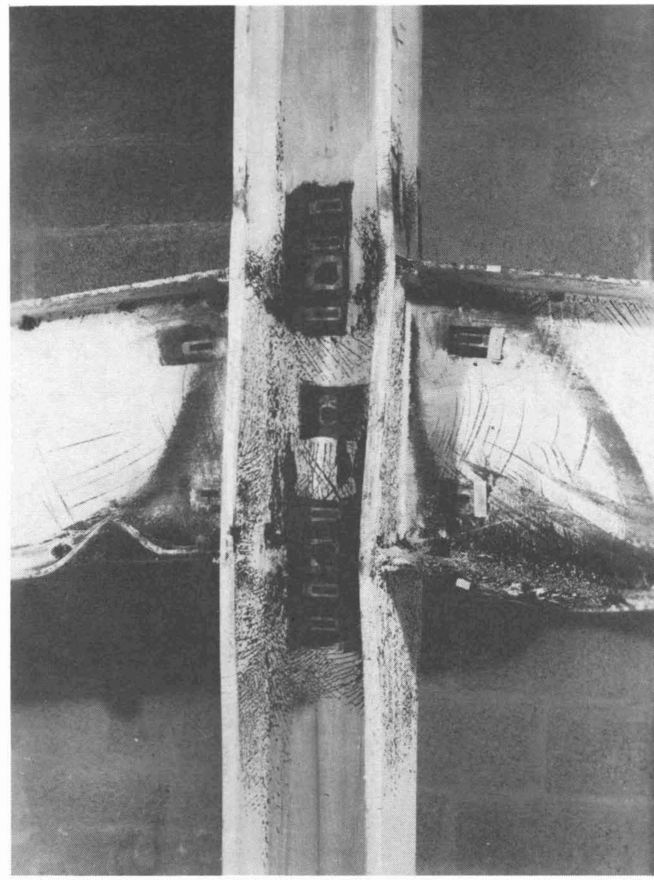
Connection A-1, with its thin-webbed column, failed by column-web buckling (see Figure 14) at a load slightly above the working load on the beams, namely, 1.12 V_w. Connection A-4, with a slightly thicker column (0.390") failed by excessive strains, both tension and compression, in the column webs opposite the beam flanges and by column web buckling at a beam load of 44 kips, which is 1.82 V_w. In both instances the decrease in beam capacity was quite rapid. Test A-4 was carried out to a fairly high deflection until the capacity reduction criterion was fully realized; test A-1, on the other hand, was curtailed rather prematurely and as a result its behavior on unloading can only be surmised. In neither case was local buckling of the beam flanges experienced, although in test A-4 the column flanges deformed considerably on the second column overload.

Specimens A-2 and A-5 behaved extremely well without stiffening. As before, the tests were carried to excessive deflections with the incidence of local buckling at 2.08 V_w and 2.26 V_w respectively. However, this buckling occurred at the beginning of the strain hardening range and the specimens were therefore able to carry increased load with little trouble. 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 the column failure was imminent.

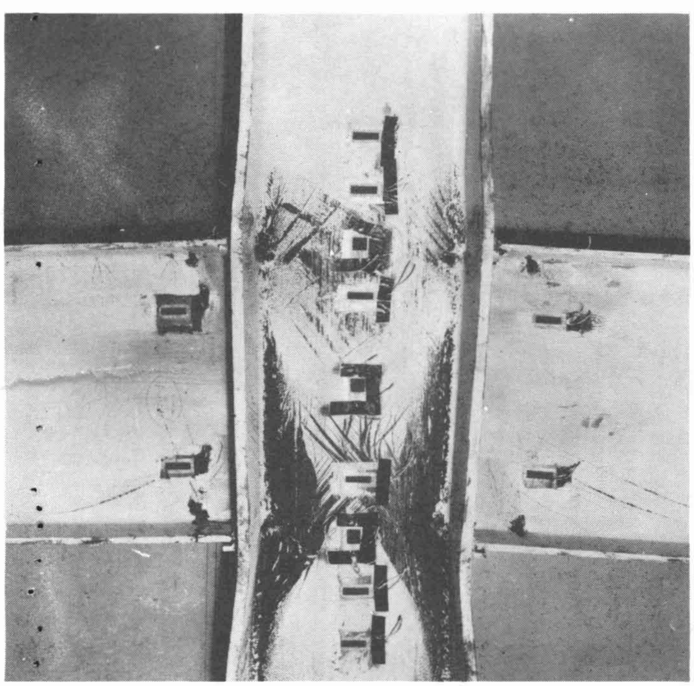
Specimens B-6 and B-8, employing horizontal plate stiffeners, exhibited excellent load and rotation capacities. Both specimens suffered local buckling of the beam compression flanges at the onset of the strain hardening range and the increase in beam load above this level was slight. The decline of strength from the maximum value was gradual as jacking continued and no harmful effects were observed in the column stiffeners beyond the presence



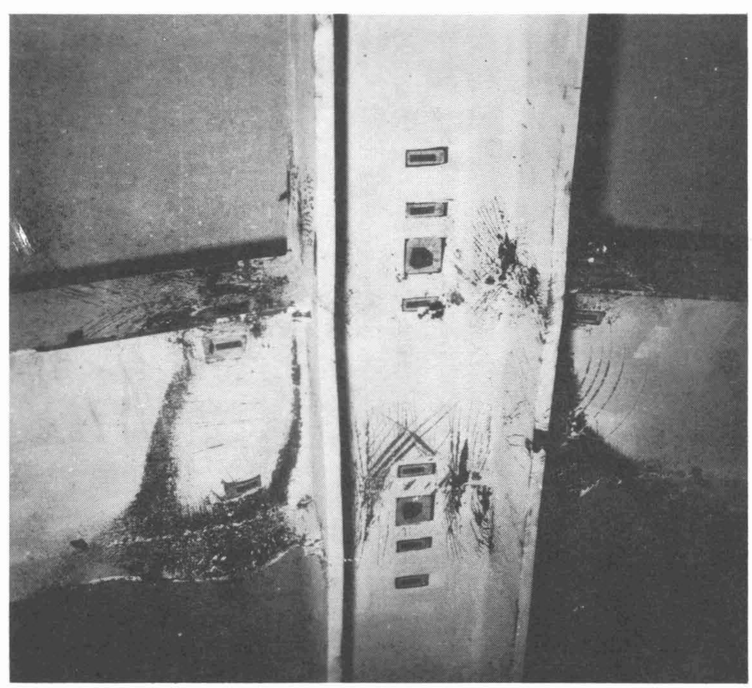
A-1



A-2

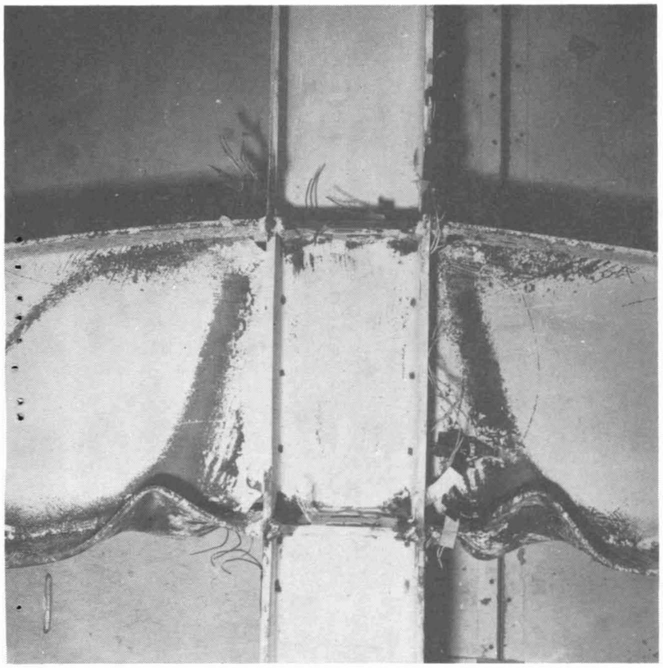


A-4

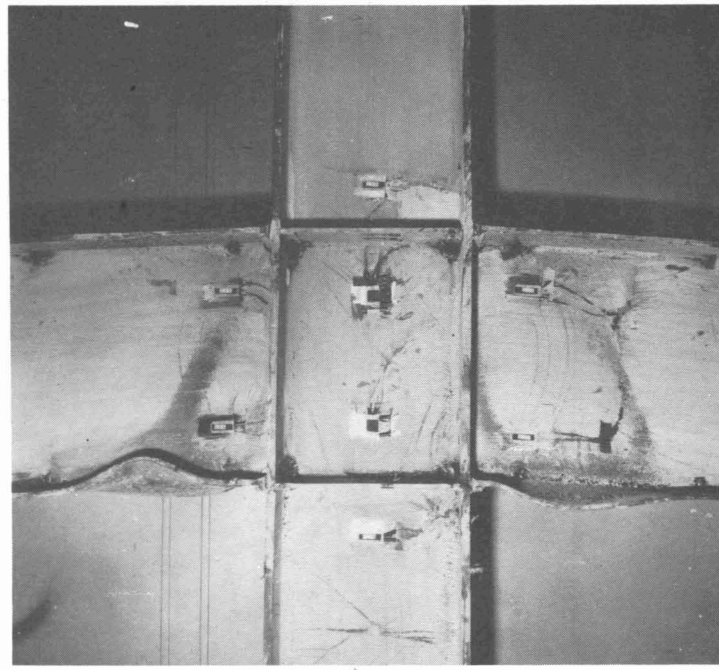


A-5

Fig 14 Photographs of Tests A-1, -2, -4, & -5

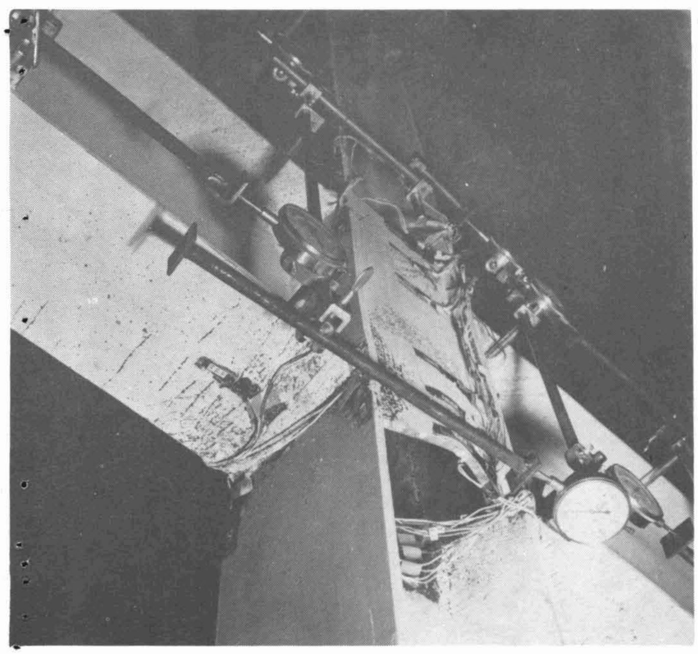


B-6

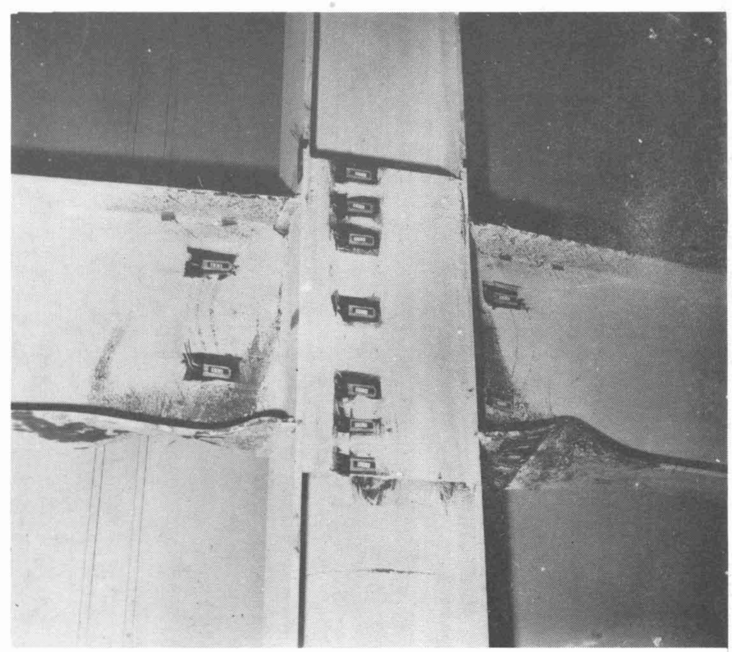


B-8

.30
.25

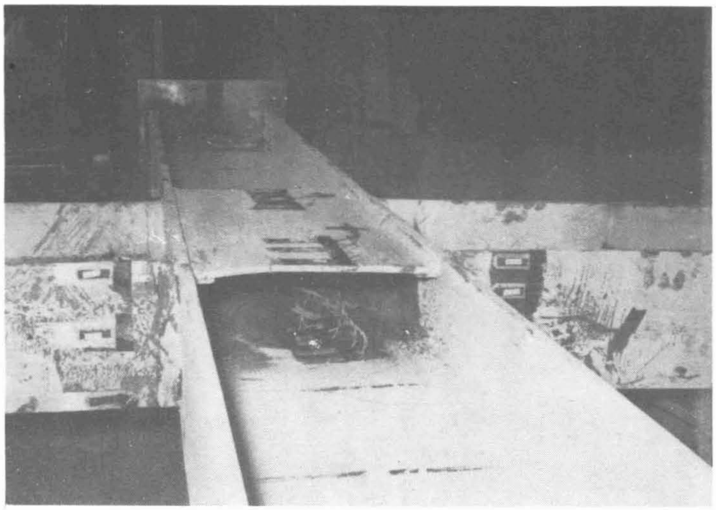


C-9

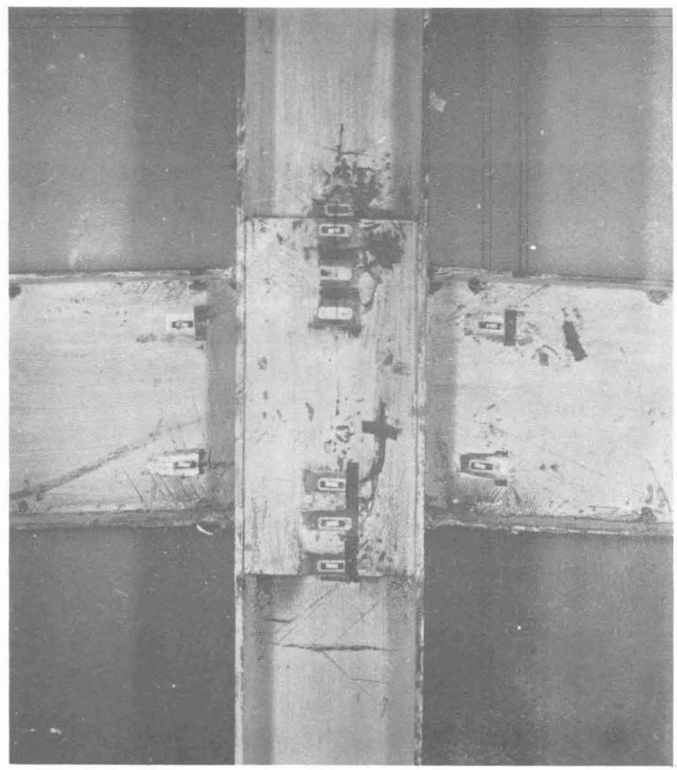


D-12

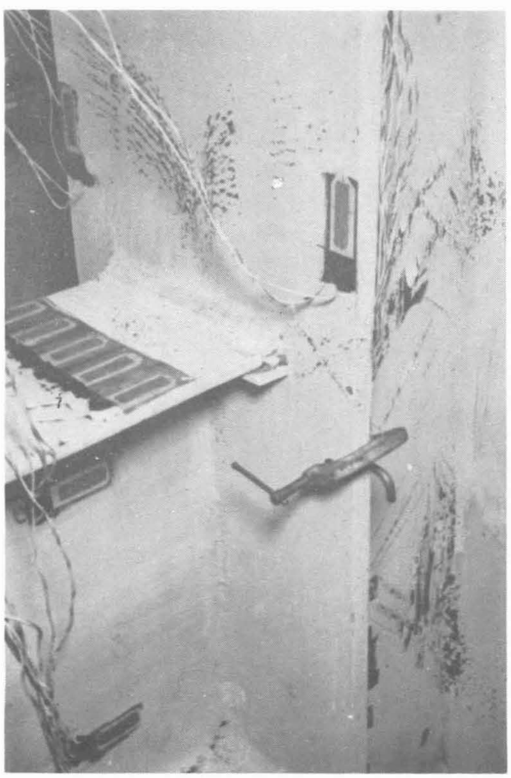
Fig. 15 Photographs of Tests B-6, -8, C-9, & D-12



C-11 Top View



C-11 Side View



A-4 At Maximum Load of 44 Kips
on Each Beam

Fig. 16 Photographs of Tests C-11 and A-4 of a few strain lines. The connections appeared to be quite elastic, the principal deformations occurring in the beams.

The two tests in the C-series (vertical stiffener plates), shown in Figure 13, carried the required loads and showed fairly good rotation capacity. This latter item is shown more satisfactorily in Figure 31, Summary Chart: Moment Vs. Total Joint Rotation.

In both tests of the C-series there was evidence of some slight local buckling on the beam compression flanges at loads of approximately 2.17 V_w (see Figure 16). In both tests too, the web of column between the beam compression flanges buckled. The critical load at which this effect was first noticed was 1.97 V_w for specimen C-9 and 2.18 V_w for specimen C-11. In test C-9 the connection continued to carry load until at approximately 2.16 V_w the south stiffener plate buckled (Figure 15). From this point the load fell off fairly rapidly until the test was concluded. **Test C-11 was stopped when the weld fracture occurred. It is ~~expected~~ probable that the beam strength decrease would be similar to D-12, west, Figure 13.**

Connection D-12 was found to be extremely stiff, the column flanges being stiffened with an ST6WF 32.5 having a flange thickness 5/8" as shown in Figure 5. A marked difference was noted in the behavior of the two beams of the specimen and, as in the C-series, weld tears were observed in the beam tension flanges at loads greater than those required to cause buckling in the beams. The primary cause of failure (see Figure 15) was the local buckling of the beam compression flanges which became large at loads in excess of 2.22 V_w. Although large deformations occurred in the beams, the connection, the column and its stiffeners, appeared to remain elastic and little strain was observed in the flange of the stiffener.

The relationship between the rotation and the applied moment is shown in Figures 17 and 18. Rotations were plotted in terms of the angle change

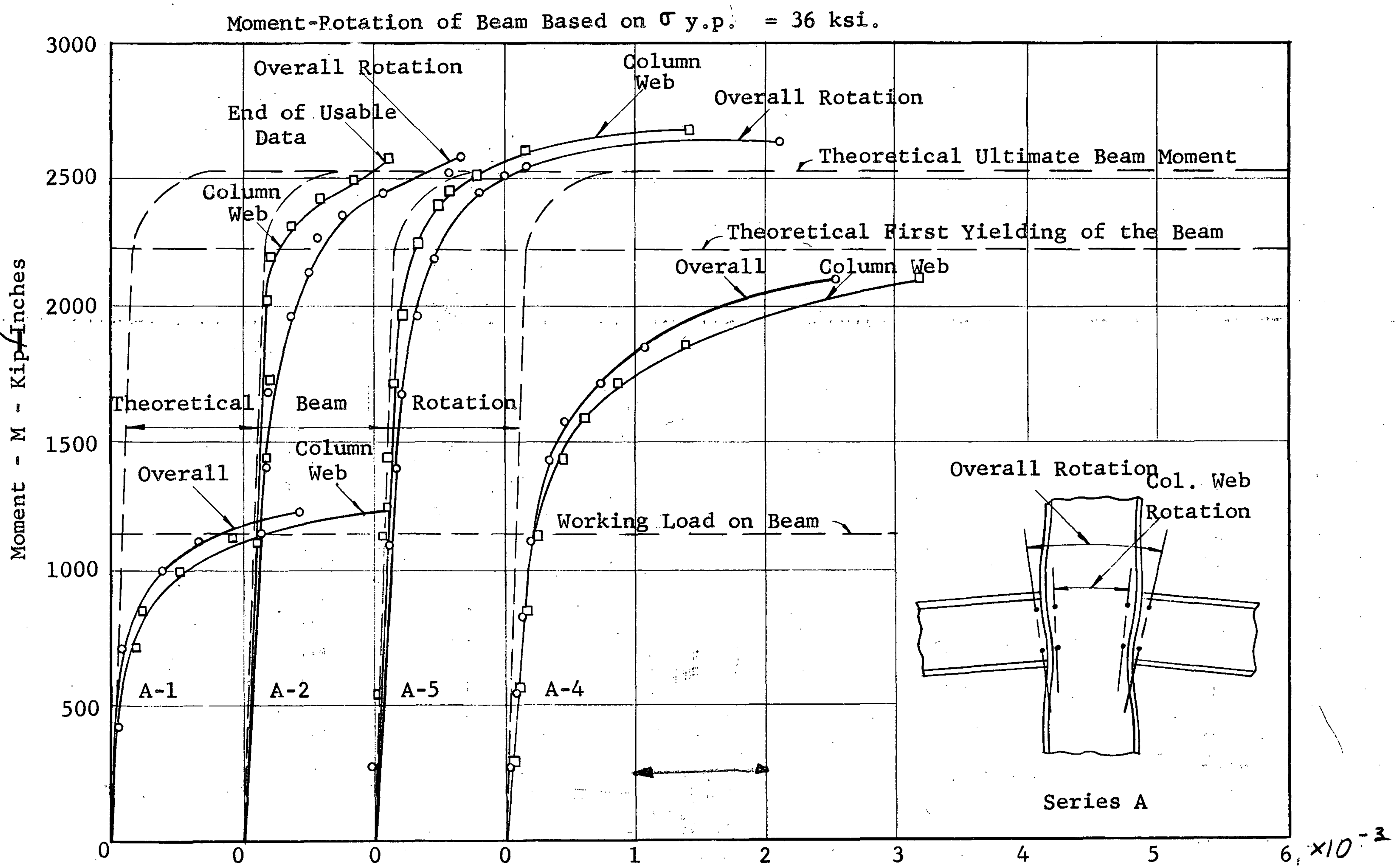


FIG. 17 - Comparison of Column Web and Connection Rotation Characteristics With That for the Beam: Series A

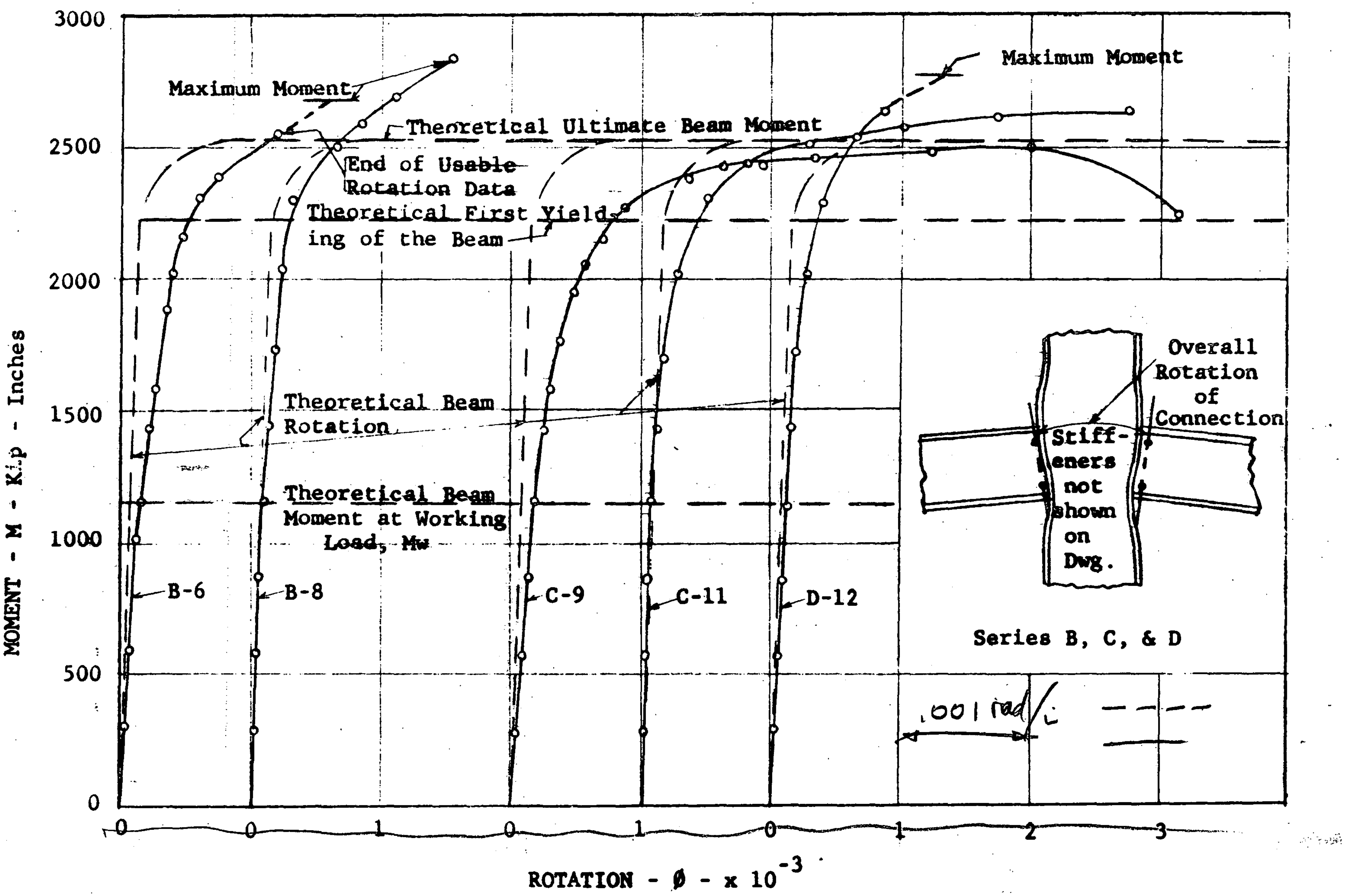


Fig. 18 - COMPARISON OF COLUMN AND CONNECTION ROTATION CHARACTERISTICS WITH THAT FOR THE BEAM: SERIES B, C, & D

per inch of horizontal distance between the gage supports. In determining the unit column web rotation for Series A, under the action of the applied beam moments, only the dial gages spanning across the column web were used. In determining the overall unit rotation of the connection, however, the sum of the three rotations was used and properly averaged; in this way the deformation of the column flanges and its effect on the behavior of the connection were included. The overall unit rotation thus determined was an average value which included one inch of beam, the welded joint, and the column flanges and web. In calculating the applied moment a lever arm of four feet was used, the distance from the point of load application to the face of the column flange. Where stiffening was provided as in the B, C and D series of tests (Figure 18) the only item of interest was the overall behavior of the connection. Rotation readings were taken throughout the test for every increased increment of beam loading, but were discontinued when local buckling of the beam flanges became excessive and tended to rotate the dial gage supports, which were mounted at the junction of beam web and flange. In each test, the experimental data is compared with the theoretical moment-rotation characteristics for a 16WF36 beam section, the latter obtained on the basis of an idealized stress-strain and M-ϕ relationship.

The distribution of the stress in the beam flange was plotted for each of the nine specimens (Figures 19 to 21). The distribution was confined to a section at which longitudinal strain gages were positioned, and curves were plotted for both the tension and compression flanges. Where horizontal stiffeners were used as in the B series, a plot of the distribution in the stiffener itself was also included, and superimposed on the stress distribution in the beam flanges. In this way the intensities of the stress entering the connection could be compared with those taken by the stiffener plates and by inference the stress carried by the column web could be determined.

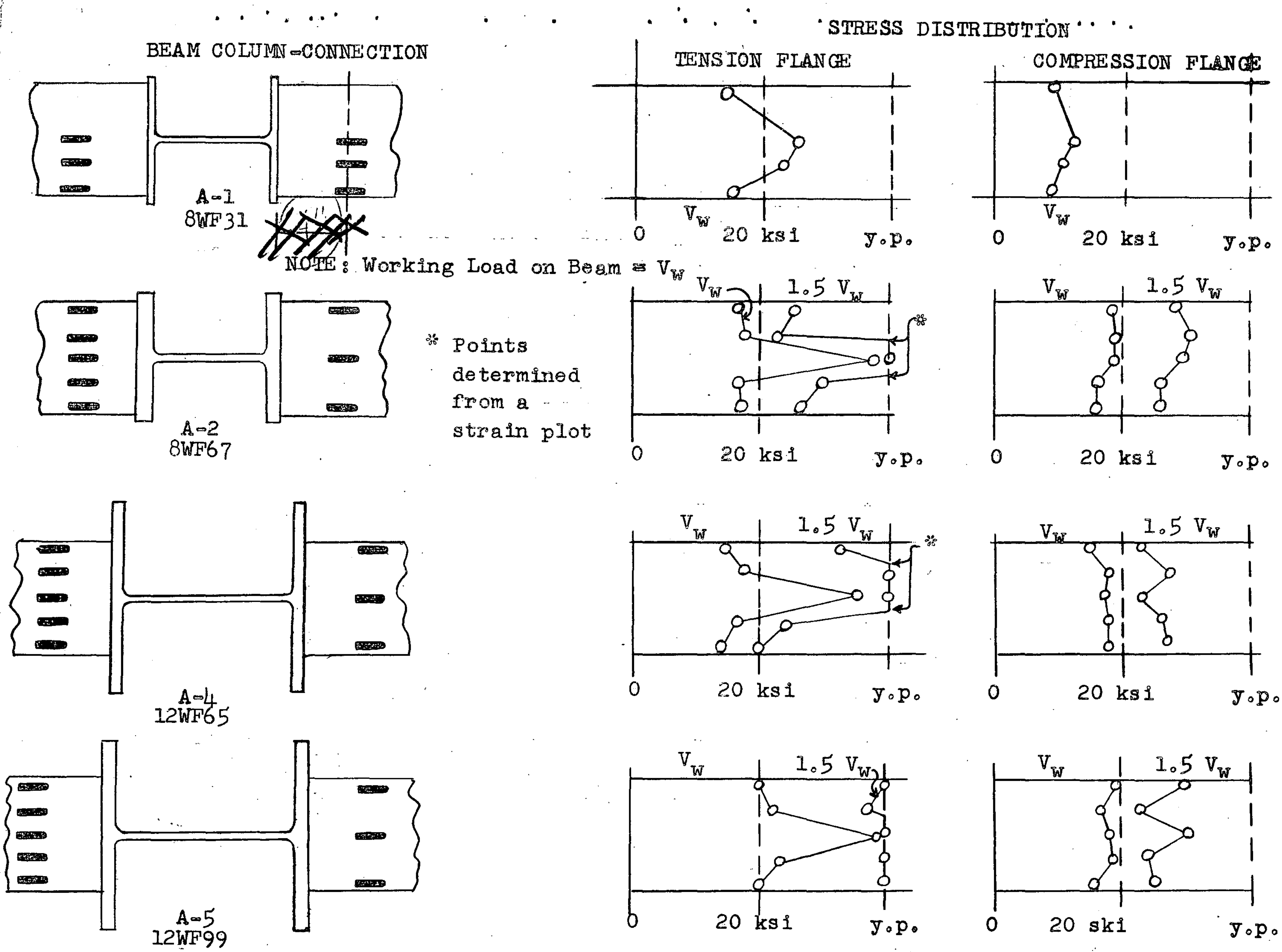
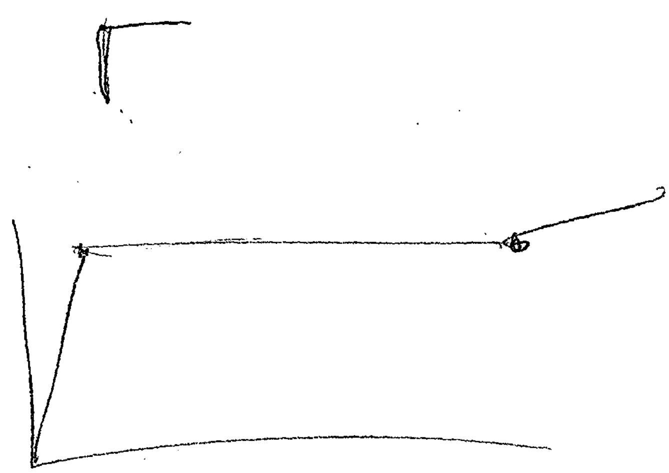


Fig. 19 - STRESS DISTRIBUTION IN BEAM FLANGES: SERIES A



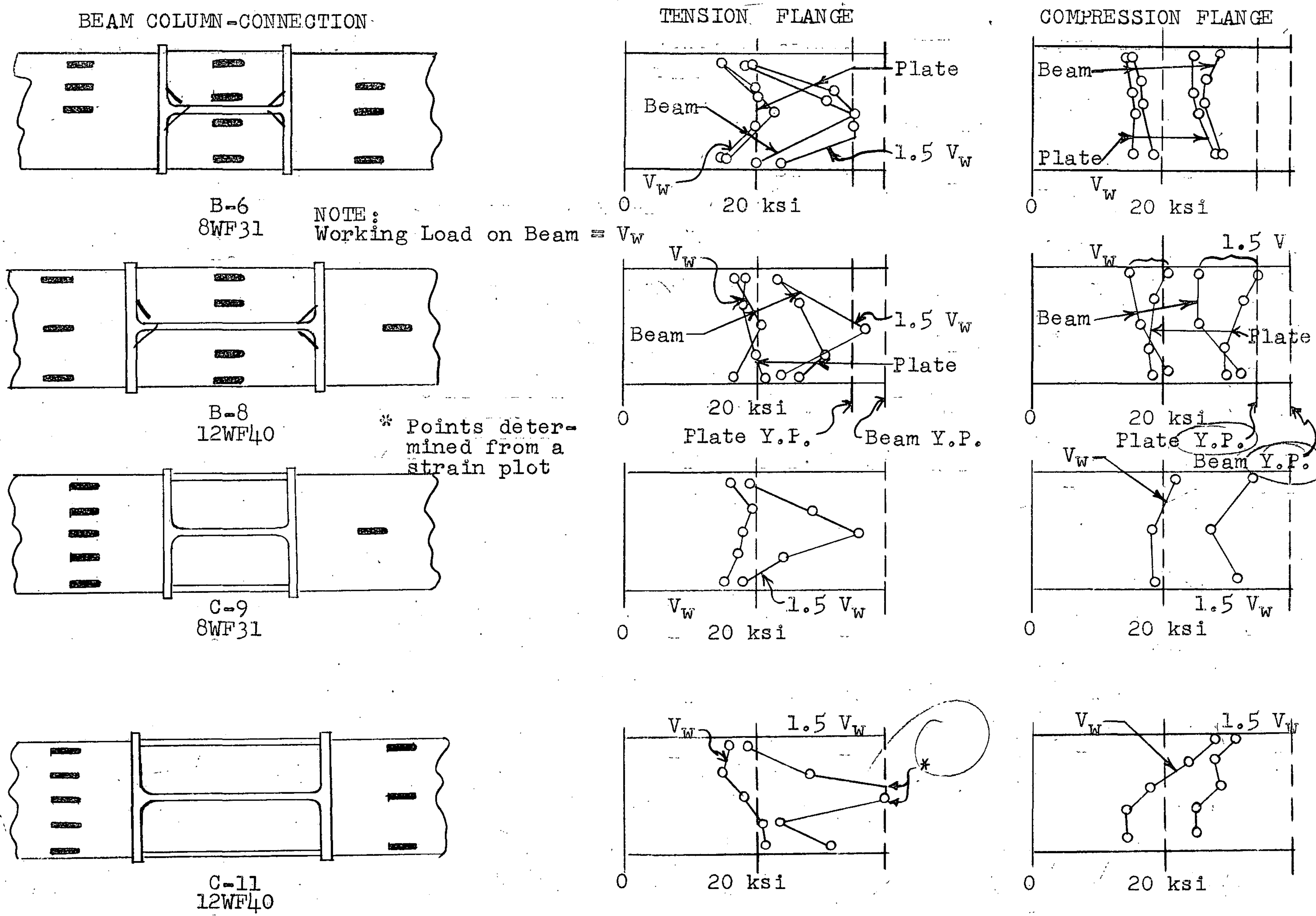
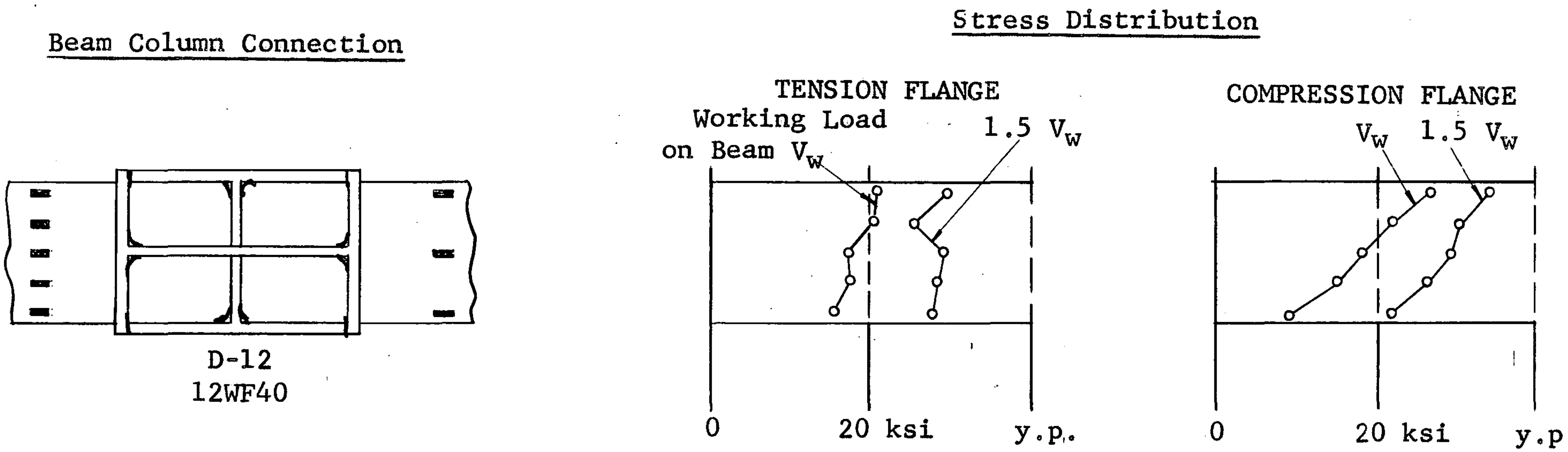


Fig. 20 - STRESS DISTRIBUTION IN BEAM FLANGES AND COLUMN STIFFENERS FOR SERIES B & C



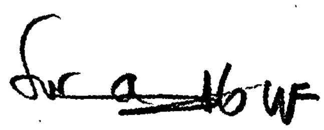
~~* Note: Points determined from a strain plot~~

FIG. 21 - Stress Distribution in Beam Flanges: Series D

The strain gages measuring the strains induced in the beam flanges were positioned at a section about 4 inches away from the face of the columns flange in order to minimize the local residual effects of the butt welds, yet be sufficiently close to the column flanges to obtain a distribution of the strains induced from beam moments, and deformation of the column flange.

The above curves (Figures 19 to 21) were plotted showing the stress distribution with working load, V_W, and $1.5\ V_W$ on the beams. The stresses, if in the elastic range, were obtained simply by multiplying the unit strain by the modulus of elasticity. In the case of inelastic strains the strain plot was first drawn; this was then converted to a stress plot by an appropriate change of scale and by cutting off the strains where they crossed the yield stress of the material. The condition of working load on the beams corresponded to a stress of slightly less than 20 Ksi (the actual stress was $20 \times 44/48 = 18.35$ Ksi) at that section on the beam where strain gages were positioned. The second condition of loading, with $1.5\ V_W$ on the beams, was quite arbitrarily selected, more with a view to showing any build up in stress concentrations, etc. which might develop at loads in excess of working load. Perhaps a more meaningful value might have been $1.65\ V_W$ (derived from the factor of safety against first yield assuming a yield point stress of 33 Ksi). This, however, would have involved interpolation of the test data in a region beyond the limit of proportionality and would have been open to question. In plotting the distribution of stress an average value of the measured strain in both beams was used and the curves are laid out so as to give a ready comparison between all specimens in a particular series.

The A-series of tests (Figure 19) show high stress concentrations at the center of the beam tension flanges, a condition which becomes more aggravated at values above working load. In specimen A-1 no distribution



of stress could be plotted for the condition 1.5 V_W since the specimen failed much below this value. For the B-series, Figure 20, it was possible to show the stress distribution both across the beam flanges and across the stiffeners. The results were not too conclusive. In the compression areas the stress distribution 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 tension and compression flanges; while at 1.5 V_W high tensile stresses occurred at mid-flange with uniform distribution in the compression flange. Specimen C-11 appeared to suffer from some eccentric effects as indicated by the higher stresses on one side of the flange. This may have been due to the effect of initial imperfections e.g. initial flange distortion or non-parallel flanges, or some mis-alignment of the beams in fabricating the specimen. Specimen D-12, Figure 21, showed uniform distribution of tensile stresses in the beam flanges, and uniform, but eccentric as described for C-11, stress distribution for the compression flanges.

To show the composite effect on the column webs of the beam moments and shears and of the column axial loads it was felt desirable to show a strain plot rather than a stress plot. These are presented in Figures 22 to 26. In the case of the C tests it was also possible to include the strain distribution in the stiffeners; while for the D-12 test it was considered impracticable to measure the column web strains, thus this test shows the distribution only in the stiffeners. Included in these figures are dashed lines showing the theoretical strain distribution for a 16 WF 36 beam ~~had the column cross-~~ subjected to the same bending moments ~~section in reference to the bending forces been replaced by a 16WF36.~~ From this a comparison is afforded between the relative rotations of the column, stiffened or unstiffened, and a theoretical beam section. Also shown in Figures 22 and 23 are full lines showing the strain distribution,

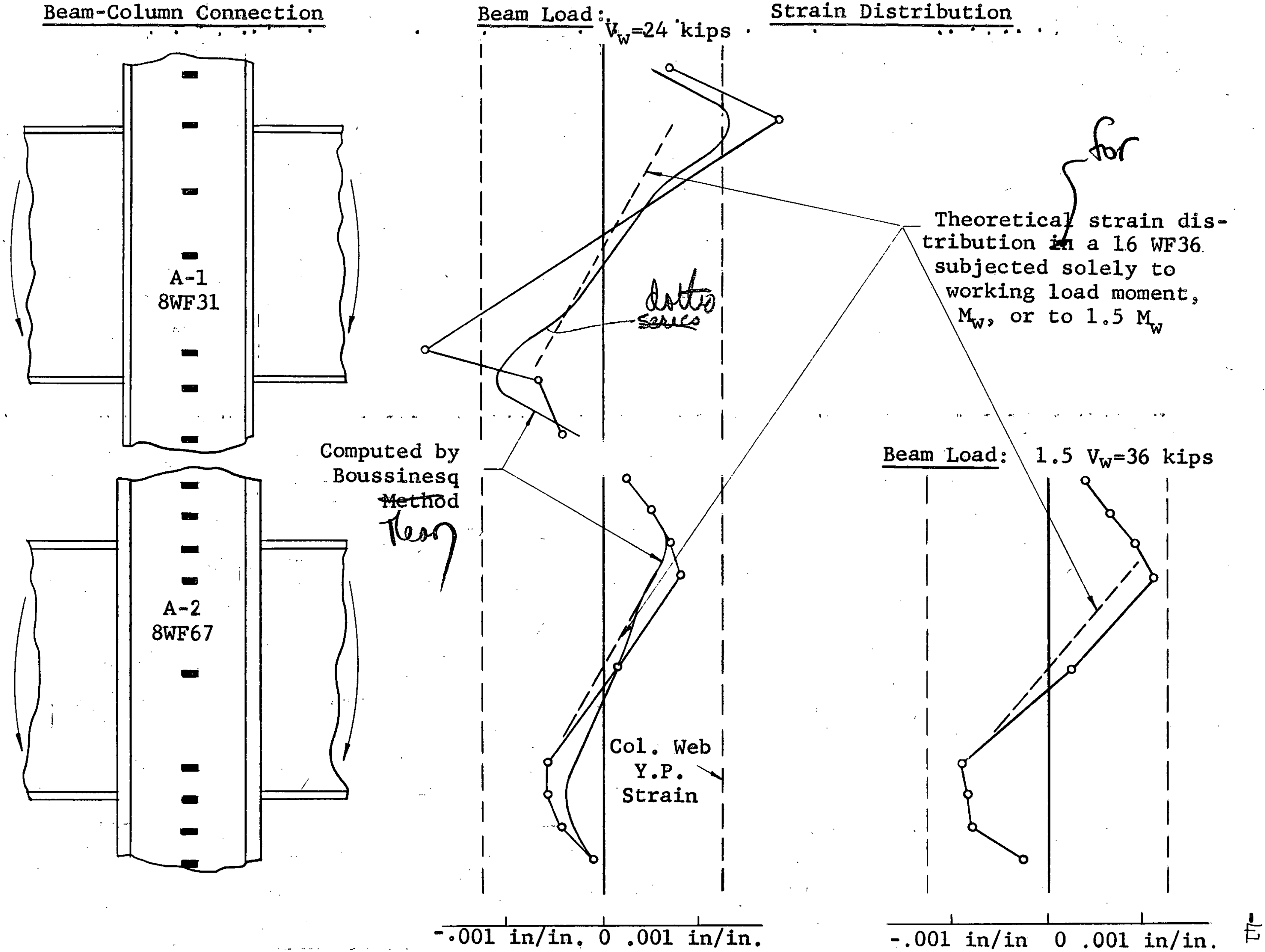


FIG. 22 - Distribution of Horizontal Strain in Column Web: Tests A-1 and A-2

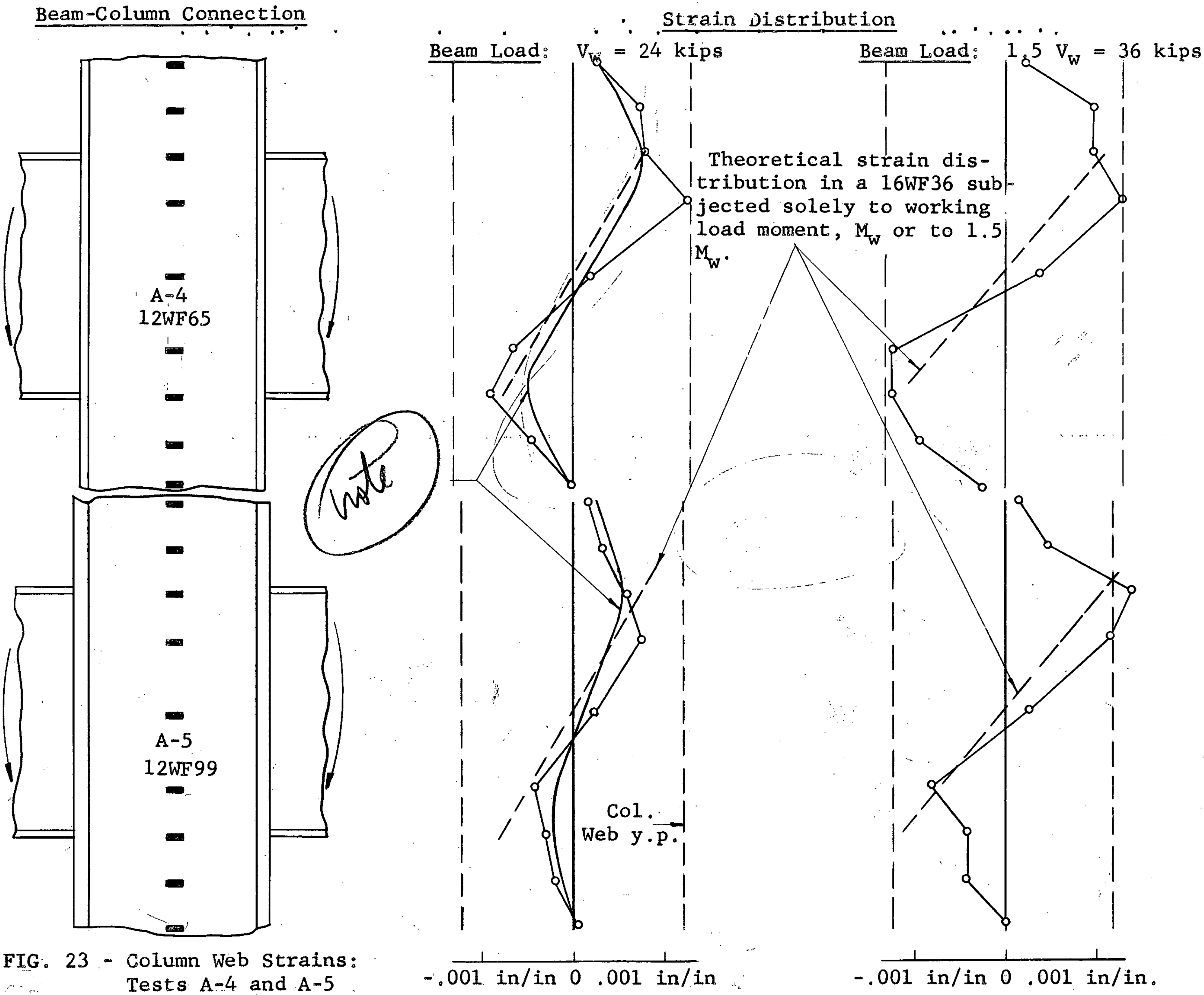


FIG. 23 - Column Web Strains: Tests A-4 and A-5

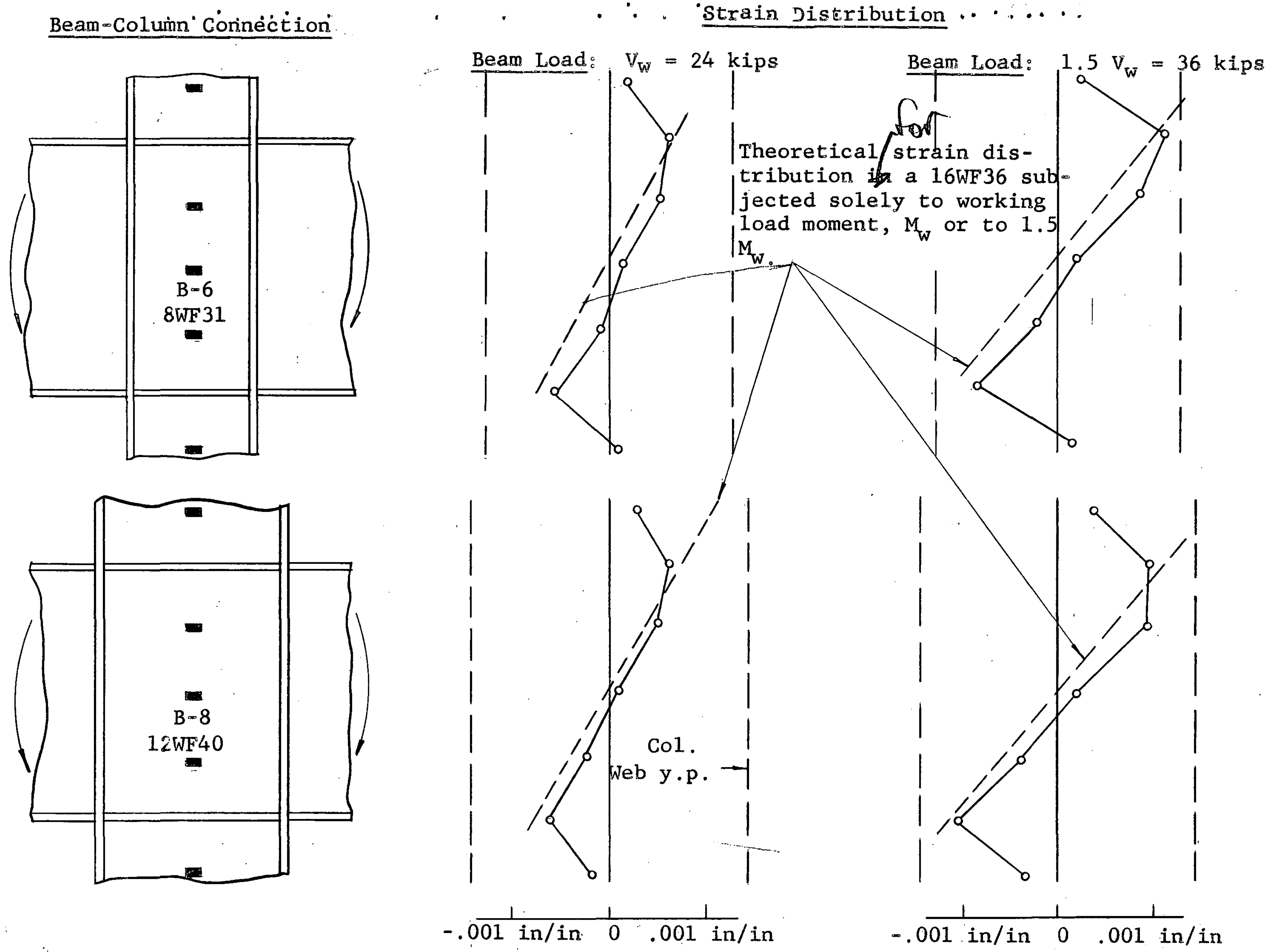


FIG. 24 - Distribution of Horizontal Strain in Column Web: Tests B-6 & B-8

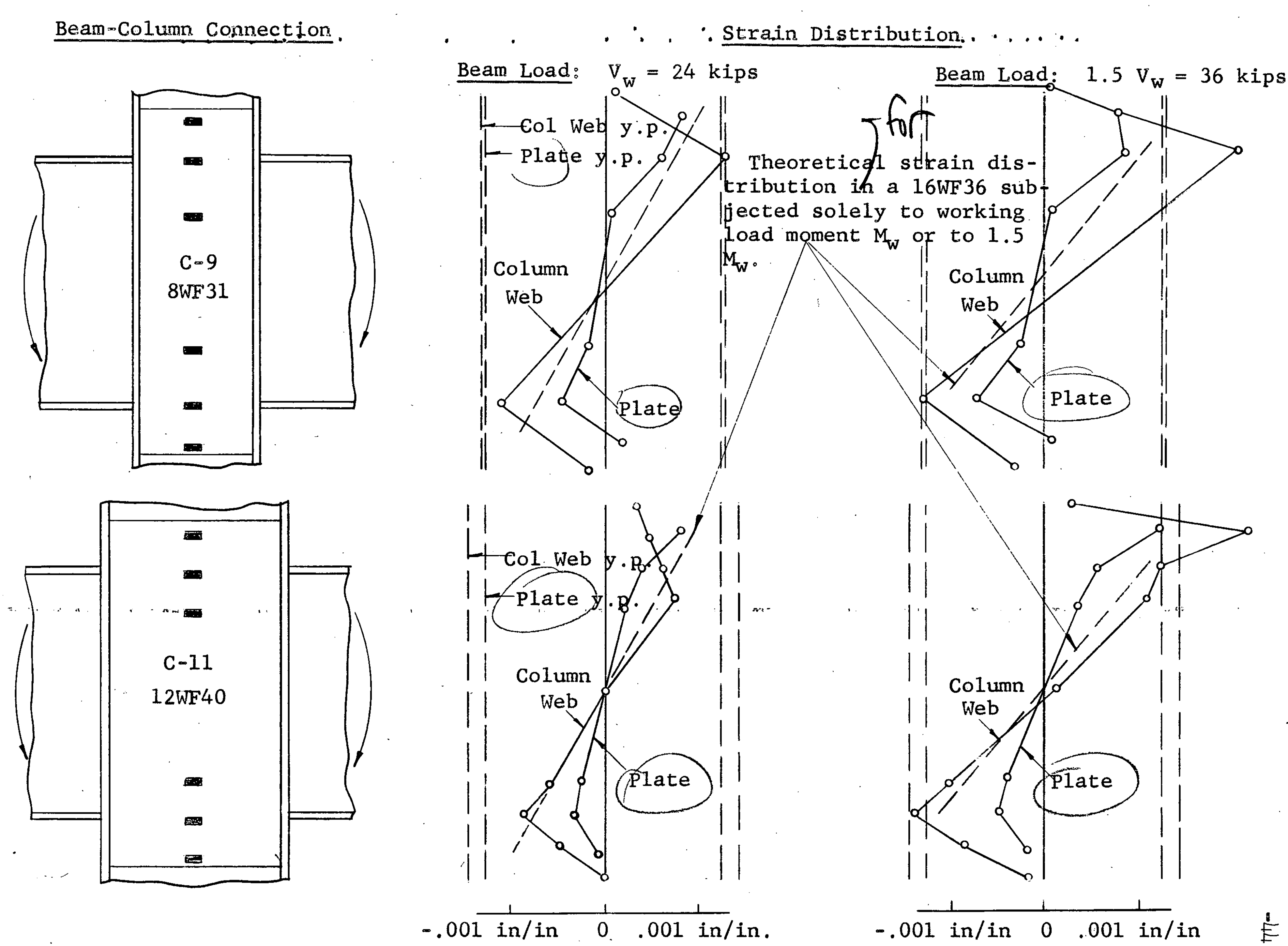


FIG. 25 - Distribution of Strain in Column Web and in Stiffeners: Tests C-9 and C-11

Beam-Column Connection

Strain Distribution

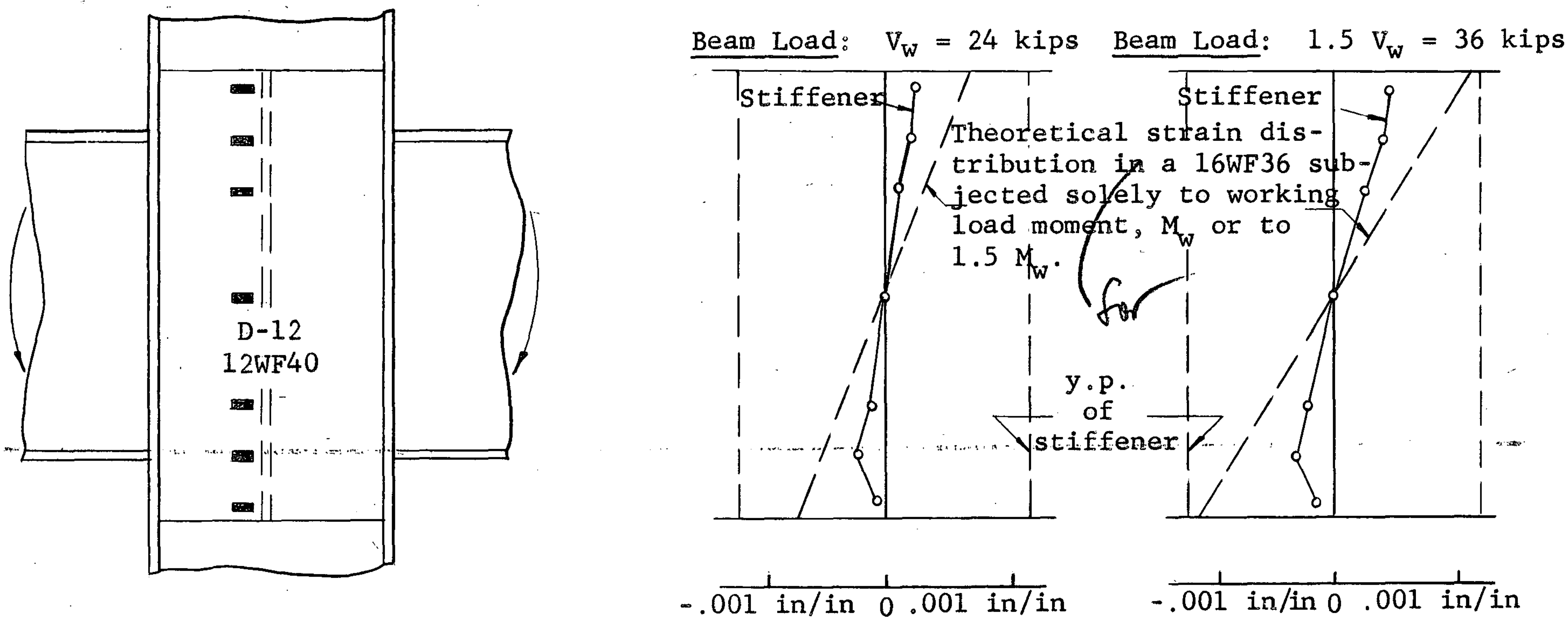


FIG. 26 - Distribution of Stress in Column Web ~~and In~~ Stiffeners: Test D-12

computed by the elastic theory as developed by Boussinesq. This is discussed under THEORETICAL ANALYSIS.

The results in Figures 22 and 23 show that the columns of the A specimens, with no stiffeners, are not as stiff against rotation as are the 16WF36 beams which framed to the columns. The increased tensile strains and the decreased compressive strains are due to the Poisson effect of the axial compression in the column. In the B-tests, Figure 24, the stiffeners provide the equivalent of beam flanges to the columns, and the columns became as stiff against rotation as the framing-in beams. The same applies to the C-tests as shown in Figure 25. Form an inspection of these C-test strain plots, it can be noted that the column web carried a major part of the applied load, approximately 2-1/2 to 3 times as much as the plate stiffeners at beam working load. Note in Figure 26 that the gages in measuring the strains on Specimen D-12 were offset in order to clear the fillet at the junction of the flange and the stem of the tee.

In ~~the earlier~~ tests, A1, B6, and C9, type AR-1 strain rosettes were used extensively to determine the principal stresses in the web. Upon reducing the data it was found that the principal stresses did not differ much in direction from the horizontal and vertical. In later tests, type AX-5 strain gages were employed in place of rosettes; These gages measured strain in the horizontal and vertical directions only. In this way an evaluation of the stresses could, if desired, be made in the horizontal direction with due allowance for Poisson's effect arising from stresses in a direction at right angles.

5. THEORETICAL ANALYSIS

In a beam-column welded connection there are several regions which are subject to local overstress and therefore, it appears pertinent before undertaking the analyses 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 one that maintains its moment capacity for a considerable rotation past the ultimate load. The rotations required at plastic hinges (namely, the "hinge angles") for a variety of practical structures have been determined Ref ____ (acd) ~~At the present time a quantitative value of the necessary rotation capacity cannot be specified, but it is hoped that a Ph. D. dissertation in progress at Lehigh University will supply this information.~~

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 ~~analysis~~ investigation are (1) the stresses in the welds at the beam-column joint as affected by column flange deformation (2) ~~(2) the deformations in the column web opposite the points where the beam flanges are welded to the columns~~ (3) the deformations of the column flanges at these same points where the beam flanges are welded to the column (3) the stresses and deformations in stiffeners, if used, and (4) the possibility of ~~of buckling in three areas: the column web,~~ in ~~the stiffeners, and the~~ column failure ~~flanges (the latter due to the~~ ~~as a result of~~ due to a combination of axial and ~~bending~~ local stresses.

From a study of previous researches (2) & (10) and from observation of the test results in the present tests, the critical item appears to be the concentration of web stresses in the column immediately opposite the ~~contact points of the beam flanges. This applies to both the tension and compression flanges, but more especially to the~~ compression flanges of the beam because of the ~~consequent~~ buckling which may take place after the web becomes

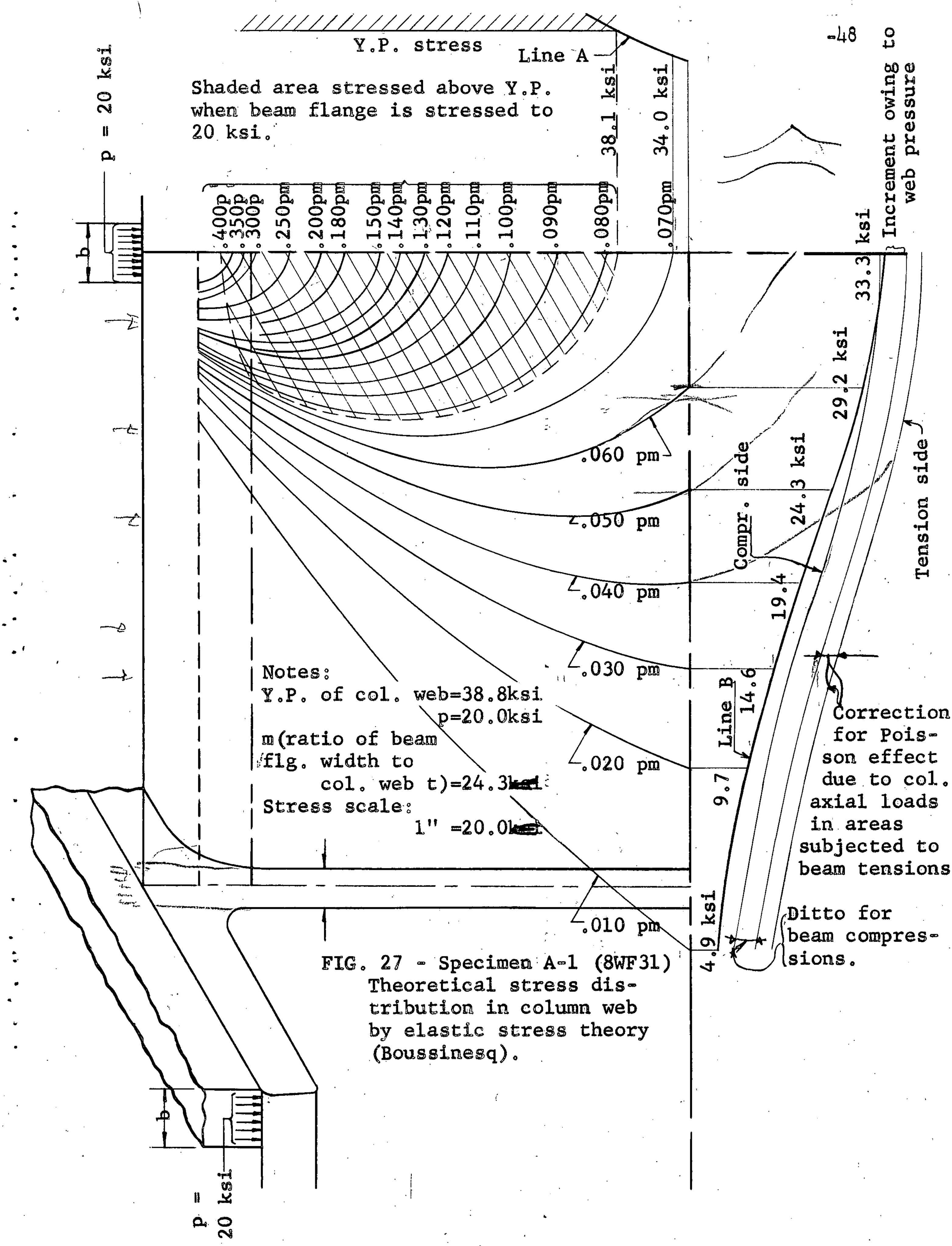
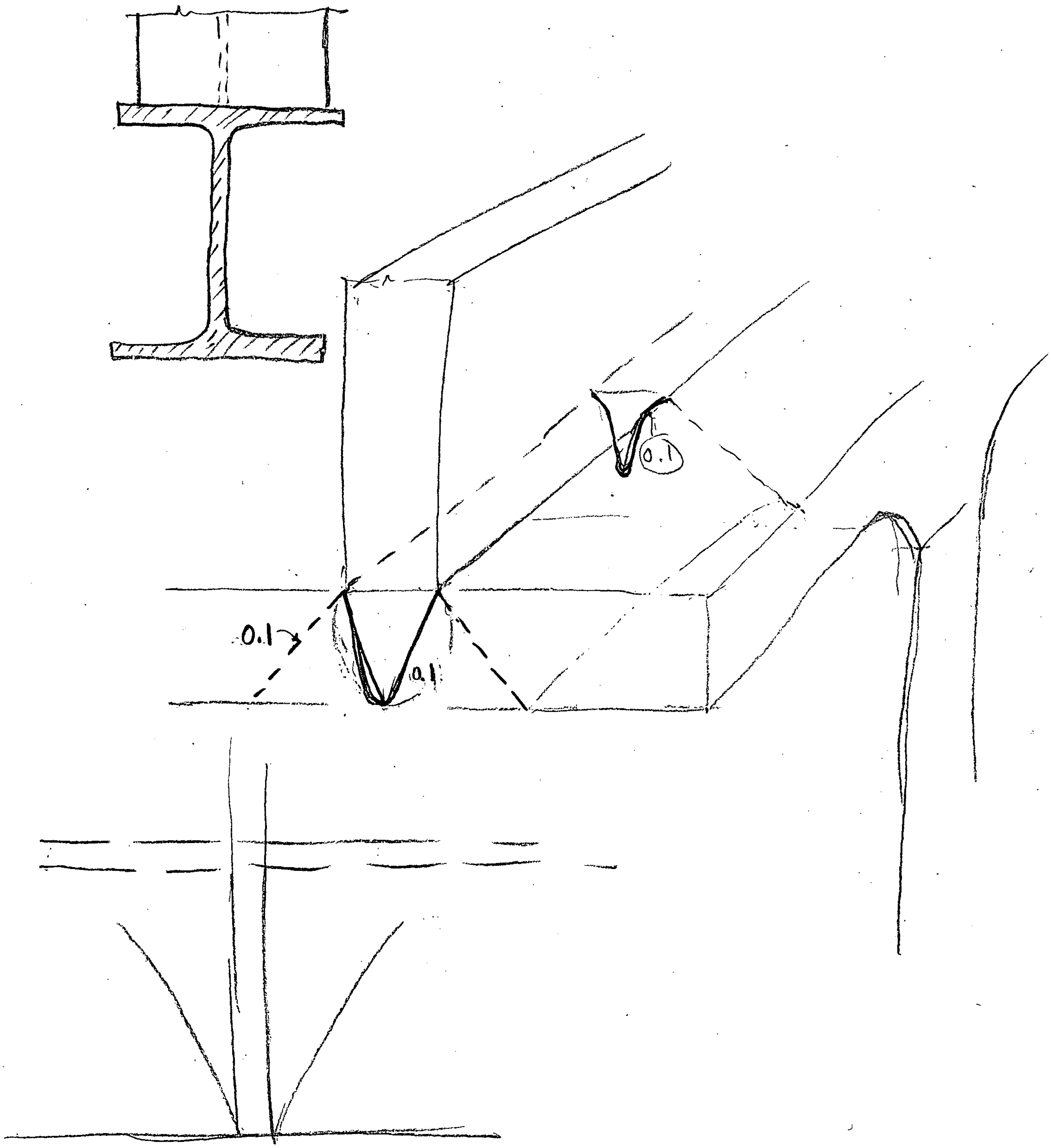


FIG. 27 - Specimen A-1 (8WF31) Theoretical stress distribution in column web by elastic stress theory (Boussinesq).


0.1
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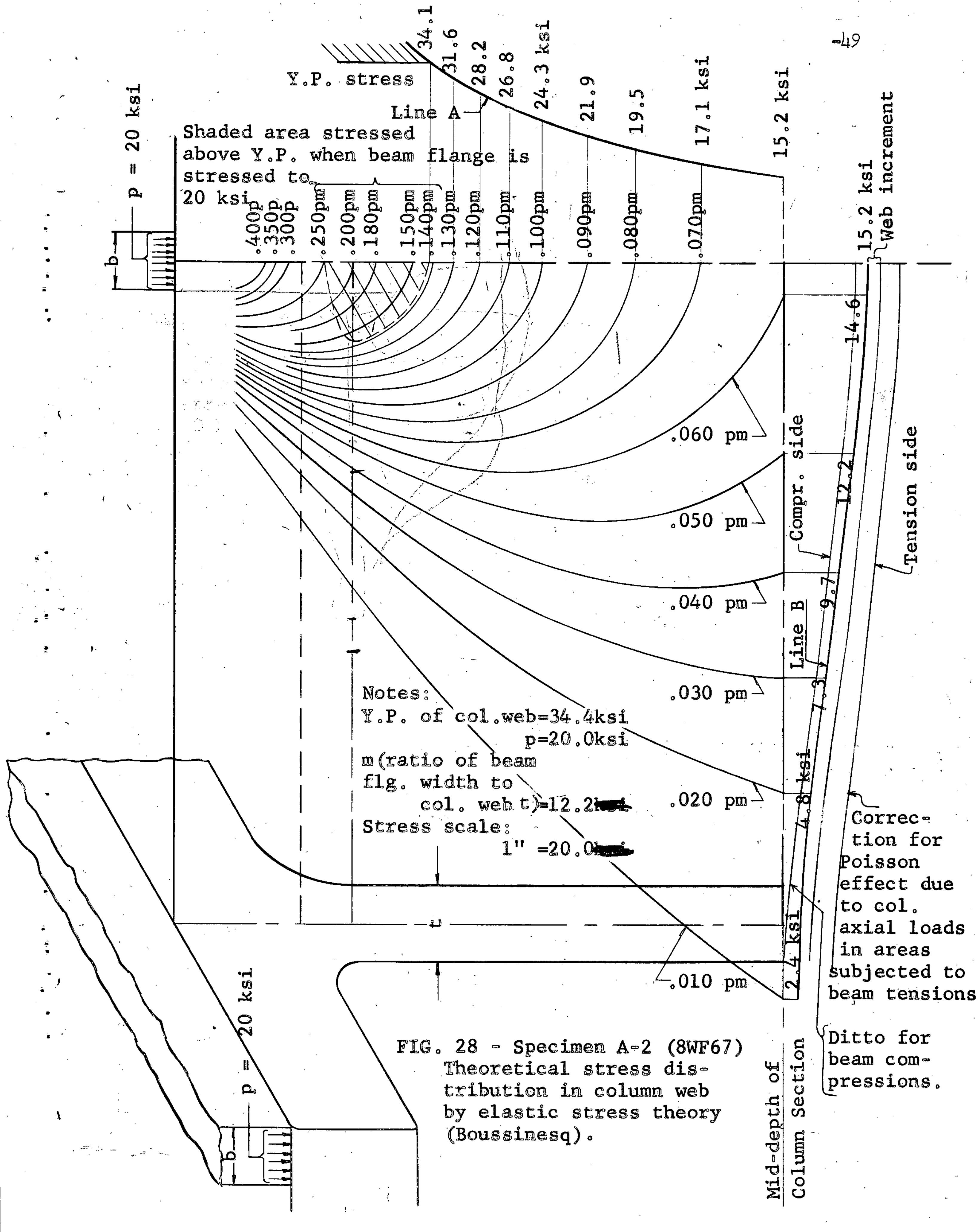
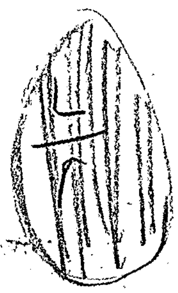


FIG. 28 - Specimen A-2 (8WF67) Theoretical stress distribution in column web by elastic stress theory (Boussinesq).

$(A_f \times 33) = 99^k$

$V_y \times a = M_y$

$V_y = \frac{M_y}{a} = \frac{\sigma_y S}{48''} = (34)($



This subject, Items 2 and 3 above, is discussed in the following sections, S.1. Items 3 and 4 above are discussed in section S.2

Column web and Stiffener Stresses and Deformation

plastic.

S.1

To ~~show~~ study these ~~high~~ concentrations of stress in the column web an elastic analysis was made, which ~~cannot claim to be~~ is not exact because of certain assumptions, but which does show clearly the nature of the problem encountered. In 1885 Boussinesq presented a solution for the stress distribution in a semi-infinite mass owing to a point loading at the surface. This theory has been extended to cover the case of a strip loading on a semi-infinite mass and is ~~(solution available in most of the texts in soil mechanics). This theory is also~~ presented by Timoshenko (12) ~~(beginning on p. 82)~~ which covers the case of a plate with concentrated or uniform loading on one edge. The principal assumption in this report is that the theory of ~~the~~ stress distribution for a plate with an edge loading applies ~~equally~~ also to a wide flange section. ~~however, with this assumption in mind, the Boussinesq method, when applied to the specimens under consideration, does show the problem with considerable clarity, and does check reasonably well with the test measurements.~~ Fix where?

Figures 27 and 28 show the Boussinesq ~~method~~ theory applied to Specimens A-1 and A-2. Similar studies have been made of Specimens A-4 and A-5. ~~Since~~ The most important element causing stresses in the column web is the strip loading from the beam flange. ~~these were prepared first and~~ The resulting lines of equal horizontal stress for this loading, labeled 0.010 pm, 0.020 pm, etc., are given in the figures. For the two given cases, and with a beam flange stress of 20 ksi., there are areas (shown shaded) which show stresses above the yield point of the steel. The distribution of stress by this elastic method obviously does not apply in the immediate zones of inelastic strains, but for purposes of illustrating the general trend it is assumed that ~~Timoshenko shows, by means of a free body sketch containing the given load and a semi-circular cut section which envelops the shaded inelastic area, that~~ the theory ~~remains good~~ is valid in the elastic region. In the case of Figure 27 (Test A-1) a large area of inelastic straining takes place and one can infer that, to compensate, the stresses will actually spread out at a considerably wider angle in this immediate area, while in the case of the small area of inelastic strains encountered in Figure 28 (Test A-2) the spreading of the forces from the strip loading into the column web will be but slightly wider than indicated in the figure. Considering the early failure of Test A-1 and the ability of the column in Test A-2 to serve without benefit of stiffeners, it is apparent that at least a general check is afforded on this Boussinesq approach. Line A in each figure shows the theoretical stress distribution in the column web directly in line with the applied strip (flange) loads, inward from the flange to mid-depth of the column. Line B in each figure shows the distribution of stress along the axis (mid-depth) of the column due only to the strip loading from the beam flange.

There are two additional effects causing horizontal strains in the column web. The first of these is the added effect of the compression, or tension, from the beam web. The second is the Poisson effect from the axial load on the column. These two increments are shown in the figures. To give some sort of check on this Boussinesq method as applied to a wide flange section, the resultant computed strains have been plotted on Figures 22 and 23. Based on which unit?

To arrive at a workable design procedure on the assumption that the column web stress as described above is the critical item, two lines of approach have been made. For convenience of identification these approaches will be labeled the Modified A.I.S.C. approach and the Plastic Analysis approach.

The Modified A.I.S.C. approach acknowledges that a small area of inelastic strains in the column web immediately opposite the beam flange is not dangerous, that it merely causes a redistribution or spreading out of the stresses as they are transmitted into the column web. This is verified by Test A-2 with its small area of inelastic strains as shown in Figure 28 and in which the column web proved entirely adequate without stiffeners. The present A.I.S.C. specification, Sec: 26h, requires that the computed column web stress, σ_c, must not exceed 24 Ksi., where σ_c is computed from the formula

$$\sigma_c = \frac{R}{t_c\ (N+2k)} \qquad (1)$$

where R = concentrated load from the beam flange

t_c = column web thickness

N & k are shown in Figure 29

From an inspection of Figure 29 it is shown that the stresses from the beam flange are assumed to spread out into the column web at an angle of 45° until the end of the flange-to-web fillet is reached. This, on inspection of Figures 27 and 28, is in close agreement with the actual elastic distribution of stress into the column web as determined by the Boussinesq method. However, when inelastic stress distribution is considered it appears equally obvious (and borne out by the tests) that a wider distribution of stress can be safely considered. The proposal here is to change the 45° angle (a 1:1 slope) to a 2:1 slope and to otherwise leave the A.I.S.C. formula unchanged. The modified A.I.S.C. formula would then read:

$$\sigma_c = \frac{\cancel{R}\ A\frac{M}{S}}{\cancel{t_c}\ t_b\ (N+4K)} \qquad \text{(applicable to Series A tests)} \qquad (2)$$

For the case where horizontal plate stiffeners are added the formula is modified to:

$$\sigma_c = \frac{R}{t\ (N+4k) + \text{Stiffener area}} \qquad \text{(for series B tests)} \qquad (3)$$

and for the case of a pair of vertical plate stiffeners formula (2) is

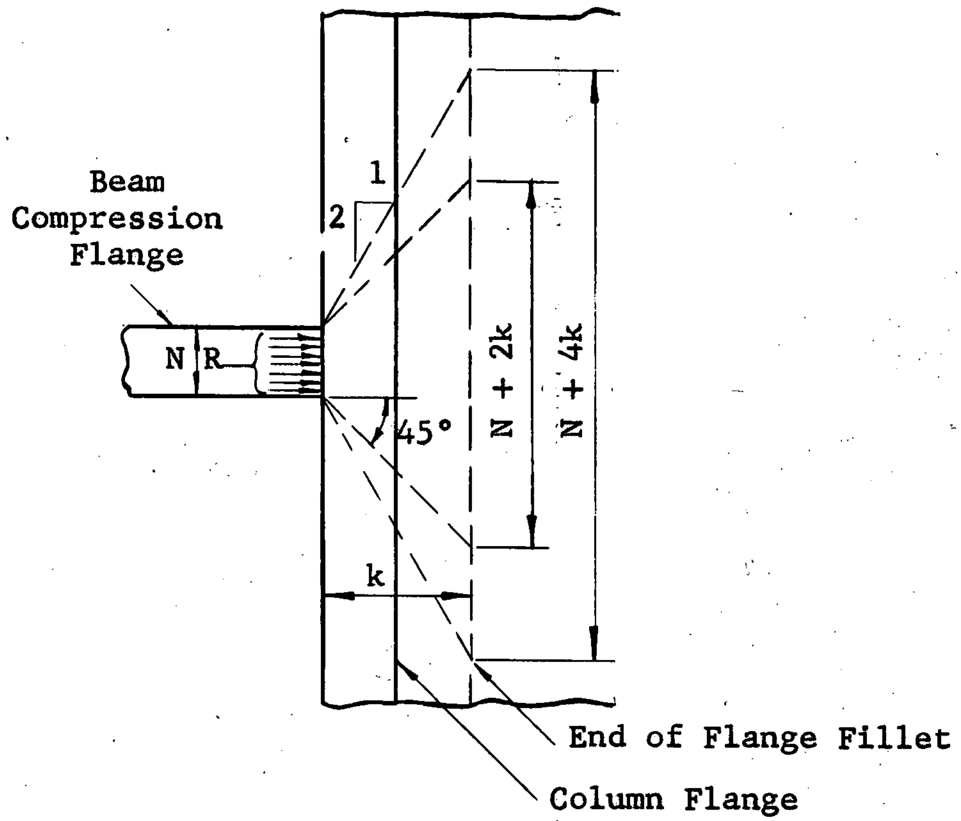


FIG. 29 - Analysis of Column Web Stresses by Modified AISC Approach

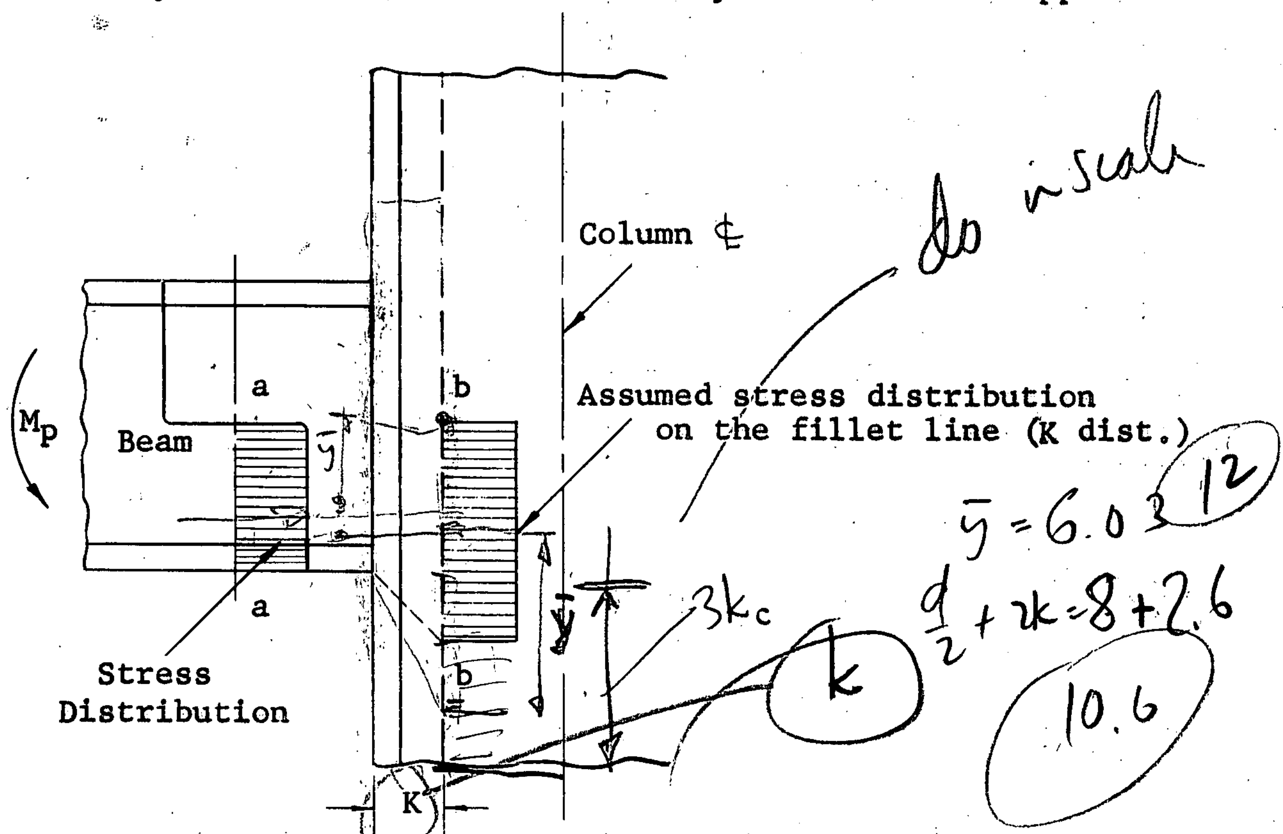
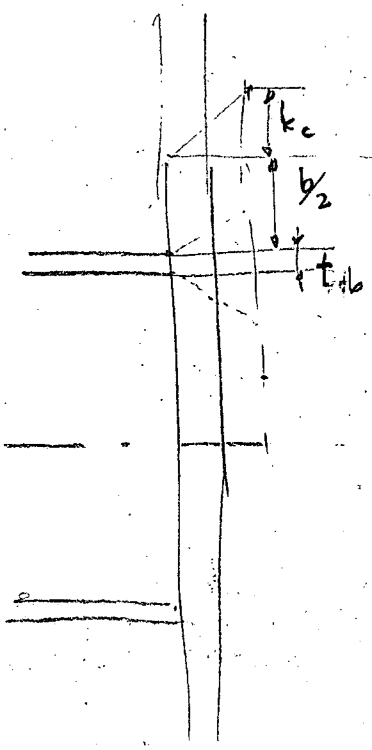
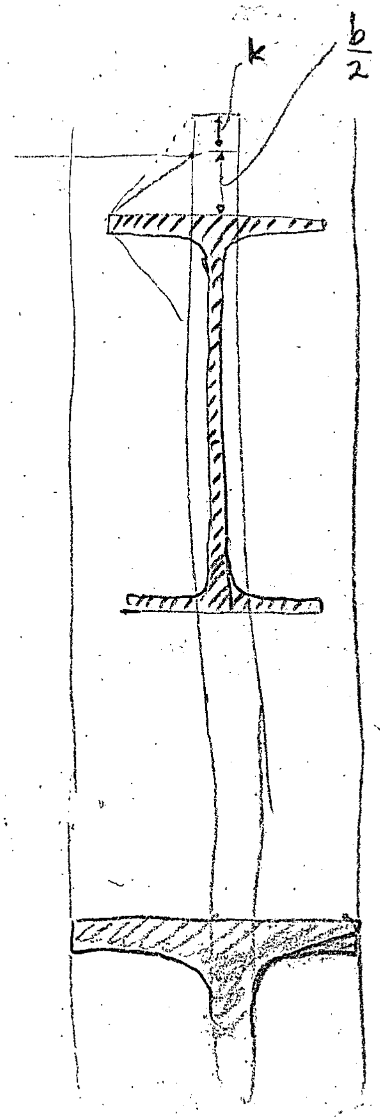


FIG. 30 - Analysis of Column Web Stresses by Plastic Analysis Approach





Assume column flange distributes at 45°
Assume web of beam is resisted by
web of column (only possible if
column web > beam web)

Force delivered by beam flange:

$= (A_F)(24\ \text{ksi})$

Force to resist the applied load

$=$ (Area)(Critical stress allowed on a beam web) (?) should this stress be related to buckling?

Area $= [t_b + b + 2k_c]\, w_c$

Stress $= 24\ \text{ksi}$

Equating,

$A_F \times \cancel{24} = [t_b + b + 2k_c]\, w_c\, (\cancel{24})$

Thus, $w_c \geq \dfrac{A_F}{t_b + b + 2k_c}$

It doesn't work!

modified by changing t_c to $(t_c + 2t_s)$ where t_s is the thickness of a stiffener to read:

$$\sigma_c = \frac{R}{(t_c + 2t_s)(N + 4k)} \quad \text{(for Series C \& D tests)} \tag{4}$$

If desired the above formulas can be put in form to solve for the required thickness as follows:

$$t_c = \frac{R/24}{N + 4k} \quad \text{(for Series A tests)} \tag{5}$$

where R is expressed in kips.

For series B tests, the stiffener area in (3) may be approximated as t_s x beam flange width, b, thus:

$$t_s = \frac{[R/24] - [t_c\ (N + 4k)]}{b} \tag{6}$$

As a further limitation, t_s should be at least 1/16th of the stiffener width.

Similarly for the Series C and D tests:

$$t_s = \frac{1}{2}\left[\frac{R/24}{N + 4k} - t_c\right] \tag{7}$$

~~54a~~ Limit to 1/32

The Plastic Analysis approach assumes a stress distribution in the beam, loaded to its capacity, M_p, as shown by Section a - a in Figure 30. 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, suggested by Lynn S. Beedle, results in the following analysis.

A. Unstiffened Columns

1. Assume that the ~~beam is developing its plastic moments,~~ maximum moment applied by the beam is equal to it plastic value M_p.

For the compression flange the pressure against the column will be approximately as shown in Figure 30.

$$\text{Let } C_b = \text{total thrust from beam} = \frac{A_b}{2}\sigma_y \tag{8}$$

where: A_b = area of beam section

σ_y = yield stress

$$t_s b + w_c \sigma_y \left[\frac{d_b}{2} + k_c \right]$$

C_w = total compression in column web, opposite C_b.

This equals $C_w = \sigma_y \, t_c \, (\frac{d_b}{2} + 3k)$ (9)

Should there be a 2k

where: W_c = thickness of col. web

d_b = depth of beam

Now $C_b = C_w$

Equating (1) & (2), $t_c = \frac{A_b}{d_b + 2k}$ 6 (10)

where t_c is the required thickness. If the actual thickness exceeds this, no stiffeners are required.

B. Columns with Horizontal Plate Stiffeners (Series B).

Same as (A) except that the compression in the column is carried jointly by the web and the pair of stiffeners.

Let b = total width of the pair of stiffeners (approx. equal to the width of the beam flange).

t_s = thickness of the stiffeners

$C_b = C_w + C_s$ (11)

where C_s = that portion of the compression carried by the stiffeners

$C_s = A_s \sigma_y = t_s b$ (12)

substituting in (11) and reducing:

Req'd. $t_s = \frac{1}{b}\left[\frac{A_b}{2} - (\frac{d_b}{2} + 3k)\, t_c\right]$

$= \frac{1}{2b}\left[A_b - (d_b + 6k)\, t_c\right]$ (13)

C. Columns with Vertical Stiffeners (Series C & D)

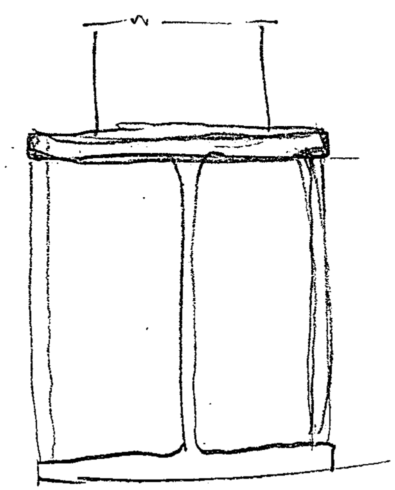
Same procedure as in (B)

Let t_b = thickness of the beam flange

$C_b = C_w + C_s$

$$\frac{A_b}{2}\sigma_y = \sigma_y \, t_c \, (\frac{d_b}{2} + 3k) + \sigma_y 2t_s \, (t_b + 3k) \quad (14)$$

(The 2 k in the last term is an approximation for convenience).



Whence $$t_s = \frac{\frac{A_b}{2} - t_c\left(\frac{d_b}{2} + 3k\right)}{2(t_b + 2\ k)}$$

or Req'd. $$t_s = \frac{1}{4}\left[\frac{A_b - t_c\ (d_b + 6k)}{t_b + 3k}\right] \qquad (15)$$

These two proposed methods of analyzing a connection for stiffener requirements have been applied to the present series of two-way tests with results as shown in Table II . The present unmodified A.I.S.C. Specimen 26h is so manifestly conservative that its application to the tests has been omitted. A study of these results may be summarized as follows:

A. Modified A.I.S.C. Approach

1. For Test A-1 formula (2) requires that the column web be 0.694" thick. The actual thickness was 0.288 and the column failed at a load slightly in excess of working load as shown in Figure 11.

2. For Test A-2 the formula requires a t_c of 0.449", while the column had an actual t_c of 0.575". This column proved entirely adequate without stiffeners.

3. Similarly for A-4, the required t_c is 0.492" while the actual t_c was less. This column failed to reach the required ultimate load.

4. The formula shows A-5 to be entirely adequate without stiffeners and it so proved to be.

5. For B-6 and B-8, formula (3) shows 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. At working load the measured stresses in these B-8 stiffeners were just below 20 ksi. It is not

$$w_c > \frac{A_F}{t_b + b + 2k_c} = \frac{(6.99)(.428)}{(.428)+(6.99)+2k_c}$$

$$w_c > \frac{2.98}{7.42 + 2k_c}$$

doesn't work!

3

6.99
.43
7.42

$$t_c = \frac{R/24}{N+4k} = \frac{(6.99)(.428)(.833)}{(.284)+4k} = \frac{2.49}{.284+4k}$$

6

1.62
7.42
9.04

=

2.674
7.42
10.09

$$\frac{A_b}{d_b + 2k_c} = \frac{10.59}{15.85 + 2k_c}$$

12WF99:

$4k_c$

10.59
18.85

	$4k_c$	$d_b + 4k_c$
A1	3.25	19.10
A2	5.28	21.13
A4	4.75	20.60
A5	6.0	21.85

$$\frac{d}{w} \not< 40$$

$$\frac{d/w}{2} = 20$$

$$\frac{KL}{r} = 60$$

2.376
15.85
18.23

TABLE II. APPLICATION OF THE PROPOSED METHODS

OF ANALYSIS TO THE TWO-WAY TESTS

Add size of member

Looks like this would have give unsatisfactory connection.

A. Modified A.I.S.C. Approach

Handbook

Specimen	N+4k	measured W_c	b	Req'd W_c	Req'd t_s	~~Actual~~ W_c	~~Actual~~ t_s	ACTUAL TEST Results	Req'd t_c ③
A-1*	3.68	.284		.694		.288		Col. web buckled	.33
A-2	5.68	.587		.449		.575		" " OK	.27
A-4*	5.18	.417		.492		.390		" " weak	
A-5	6.43	.580		.397		.580		" " OK	
B-6*	3.68	.284	6.99		.214		.437		
B-8*	4.93	.300	"		.153		.25	Stiffened Connection Satisfactory	
C-9*	3.68	.284			.205**		.312		
C-11*	4.93	.300			.109**		"		
D-11*	4.93	.300			.109**		.606		

B. Plastic Analysis Approach

Specimen	Beam A_b	Beam d_b	Beam b	Beam t_b	Column k	Column W_c	Req'd W_c	Req'd t_s	~~Actual~~ t_c	~~Actual~~ t_s
A-1*	10.59	15.85	6.99	~~[illegible]~~ 0.428	.812	.288	.606		.288	
A-2	"	"	"	"	1.312	.575	.573		.575	
A-4*	"	"	"	"	1.188	.390	.580		.390	
A-5	"	"	"	"	1.500	.580	~~.610~~ .560		.580	
B-6	"	"	"	"	0.812	.288		.398		.437
B-8	"	"	"	"	1.125	.294		.377		.25
C-9	"	"	"		0.812	.288		.635		.312
C-11	"	"	"		1.125	.294		.492		.312
D-12	"	"	"	"	1.125	.294		.492		.606

** Not passable without stiffeners

~~** A.I.S.C. specs. for compression members require at least 1/4" here.~~

* These values should be raised to about 4" — etc.

A_b = sectional area of beam

d_b = depth of beam

b = flange width

t_b = thickness of beam flg.

W = " "

wb = .299

C-11 .10

Treating the transverse stiffener plate as a column, a conservative width-thickness ratio of ~~30~~ 45 could be derived

$k = 0.75$

known how much thinner these stiffeners could have been made and still serve their purpose.

6. For the C and D tests formula (4) gives obviously too small values and it could be predicted that stiffeners of this size would fail by buckling. A.I.S.C. specifications for compression members (Section 18 (c)) limits the thickness to 1/40th of the clear distance between flanges (in these cases about 1/4"). Does it cover strength?

B. Plastic Analysis Approach

1. Same comment as in (A-1) except that the required t_c is 0.606,. This analysis likewise shows the web thickness entirely inadequate.

2. 3. 4. Same comments as above for Tests A-2, A-4 and A-5 except for the slightly changed required t_s.

5. For Test B-6, same comments as above, but for B-8 the required t_s is somewhat greater than that actually supplied. Since the test results indicate a balanced design it indicates that the Plastic Analysis formula may be slightly conservative in this case.

6. For test D-12 the supplied stiffener plates were greater than required by formula and since the stiffeners were obviously not stressed to capacity no check on the analysis is afforded; however, for Tests C-9 and C-11 the formula requires a thicker pair of plates than actually supplied. Since the stiffener plates did eventually buckle after a satisfactory ultimate load was reached this discrepancy is not regarded as serious and again indicates this analysis to be slightly conservative.

5.2 <u>Other Regions of High Stresses</u> ¶ ~~In unstiffened~~ For columns without stiffeners there will be unequal distribution of stress in the butt welds connecting beam flanges to

If a column web is

~~After the research covered by this report was completed, it was noted that a column web which, according to formula, was in need of stiffening, could be~~ reinforced by ~~means of~~ a doubler plate ~~placed on one side of the column web. Such a plate manifestly would prevent web crippling;~~ the question then arises as to whether the non-uniform stress distribution in the butt weld connecting beam tension flange to the ~~disproportionately thin~~ column flange ~~would~~ might cause ~~premature~~ failure before full development of the plastic moment of the beam. This special case of the light-section column with ~~"beefed-up"~~ reinforced web requires ~~additional study and~~ investigation. ~~Investigation to date seems to indicate that hot-rolled columns have proportions such that, if a column web is not in need of stiffeners, its flanges will be thick enough to provide a suitable distribution of stresses in the butt welds connecting beam flanges to column flanges.~~

~~If required, B-type stiffeners could be welded to the column web only, omitting the welding of the stiffeners to the column flanges. The advantage of this procedure is that it would~~

the columns. The stresses ~~being most~~ will be more severe at mid-length of the welds, ~~not~~ and less at the ends. ~~A stress concentration at the end of a weld has been proven time after time to be a potential source for early failure in a test under an increasing load application, but not when the concentration is at mid-length.~~ No weld failures ~~of this type~~ attributable to high stress concentration were encountered in the present project; moreover, a study of past researches reveals that the few times weld failures were encountered at mid-length of a butt weld the fault was attributed ~~of~~ to insufficient root gap and consequent poor penetration into the bottom of the joint. ~~In those cases the connections carried adequate test loads, exceeding a factor of safety of 2 except in one case where the factor was slightly under 2.~~ exhibited safety factors of from 1.__ to 2.__ ~~Considering test experience~~ it is advocated that the ~~unfavorable~~ non-uniform stress distribution which obtains in these butt welds in ~~unstiffened~~ columns at working load not be made a criterion in the determination of whether or not stiffeners are required. Insert 59a

No analyses are presented attempting to combine the ~~deformations~~ stresses caused by the deformations, which take place at the connection at the latter stages of a test to failure, with the stresses due to the column axial loads. The fact, that each column (except A-5 where the machine capacity was reached) ~~was subjected to and~~ safely withstood an axial stress of 29 Ksi. after the connection was severely deformed by test to failure of the beams, appears to be satisfactory evidence that an analysis is unnecessary.

6. DISCUSSION OF TEST RESULTS

A significant feature of the tests was the ability of the connections to develop the strength of the beams. In all cases except two -- 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. This is necessary in the design of structures using an ultimate load method, for not only would plastic hinges

Watch the fact
that B forces rotation
into the beams.
C does not

form at the ends of the beams, but adequate rotation capacity would permit the formation of hinges in other parts of the structure until a mechanism would be reached. X

Local buckling is a factor which might influence the value of the plastic moment of a beam section. Haaijer[7] 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 70 the depth of beam to web thickness ratio, d/w, must not exceed 55. The beam section chosen was just within these limitations ~~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. ?

As far as rotation capacity is concerned (Figures 17 & 18), there is little to choose between the ~~unstiffened~~ connection employing the light thin-webbed column reinforced with horizontal stiffeners (B-6 & B-8) or the C and D series connections that had vertical stiffeners. ? LSB re-write

In comparing the theoretical and experimental moment-rotation curves (Figures 17 & 18) 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, and C-9 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 in Figure 14 it is noted that the deformations are due to a combination of high inelastic strains in the column web in areas of both high 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 and in some cases opposite the tension flanges.~~

Another point of interest is that the sections of maximum tension and compression in the column-web are located not at the level of the beam flanges but rather nearer the center of the connection. This is particularly true in the unstiffened connections. Yet where horizontal stiffeners are provided the sections ~~of maximum tensile~~ and compressive effect occur at ~~the level of the beam~~ flanges. It is shown in Figure 25 that the vertical plate stiffeners of Series C in the elastic range, each transmitted only about 3/16ths of the forces coming in 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, for the purpose, as near to the edge of the column flange as possible.

The distribution of stress in the butt welds between the beam flanges and the column, Figures 19 to 21, is of much interest to the designer in the case of no stiffeners (Series A) or of the vertical plate stiffeners (Series C & D). However, it is noted that in both this investigation and in that by Brandes-Mains (Figure 32), no weld failures occurred until after excessive rotation had taken place.

The criterion has been proposed that the plastic moment shall be realized. To show this condition in the same manner as in previous investigations a different plot of moment-rotation curves is presented, Figure 31, in which the rotation plotted is the total joint rotation, that is, the total angular rotation from the butt welds, connecting the beams and columns, to the center line of the column. This plot is similar in nature to Figure 32 which summarizes three pertinent tests from the Brandes-Mains investigation. It should be obvious by inspection that all beams in this investigation except A-1 and A-4 pass this criterion with an ample margin.

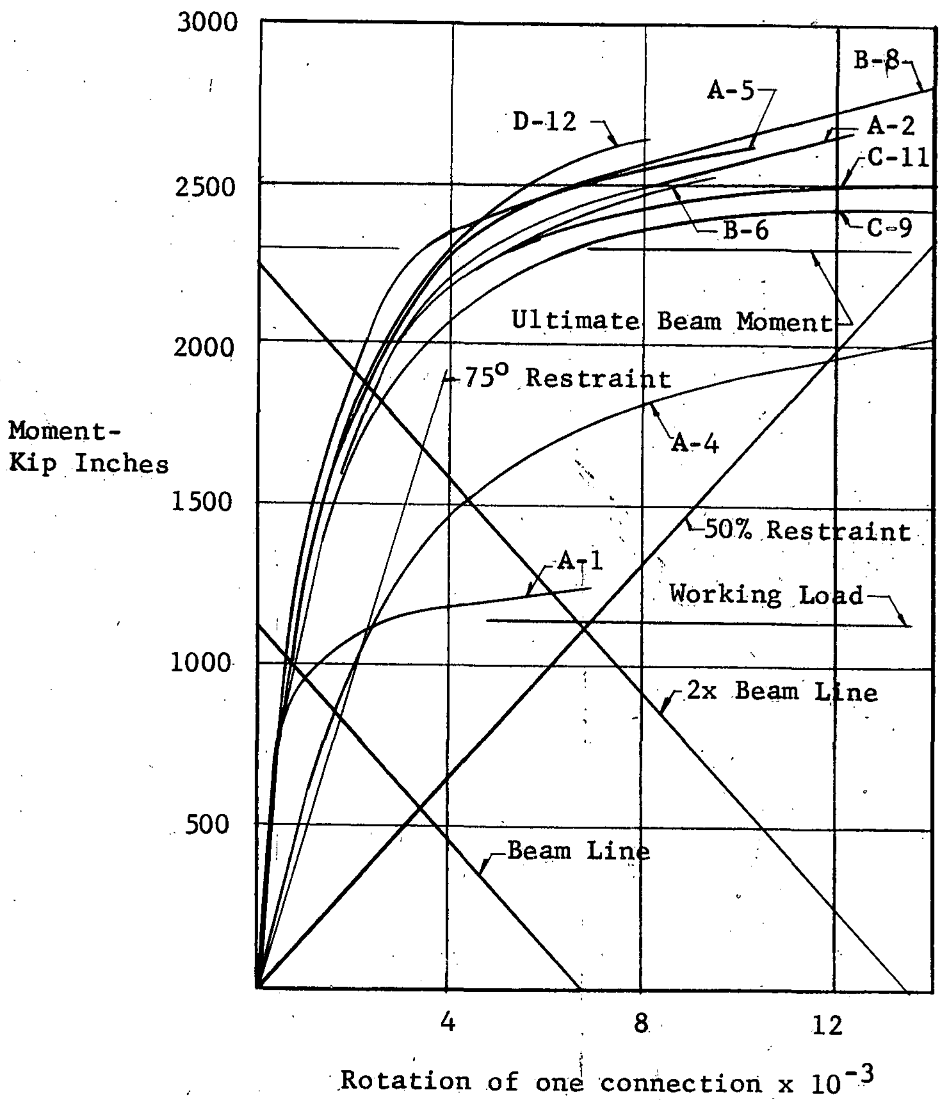


FIG. 31 SUMMARY CHART: MOMENT VS. TOTAL JOINT ROTATION

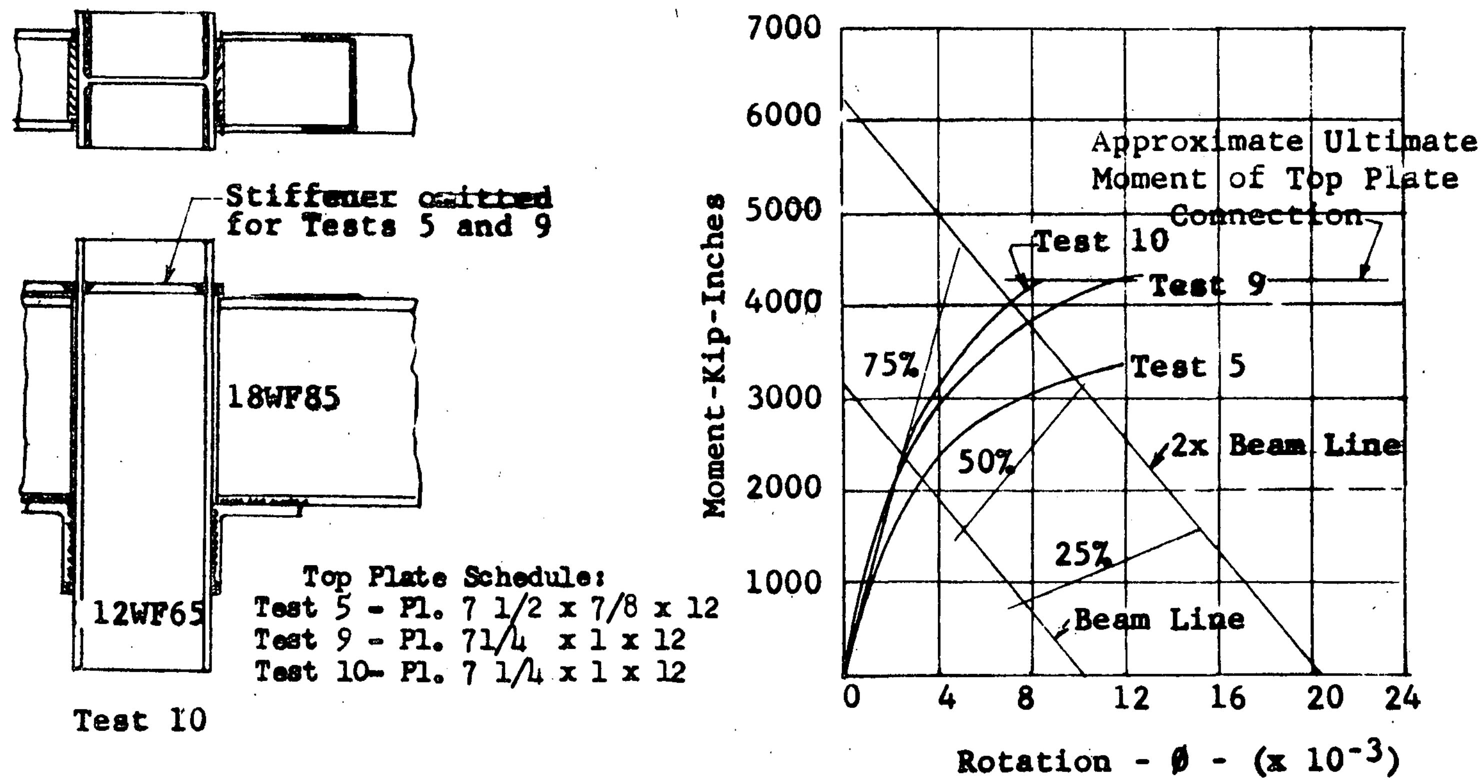
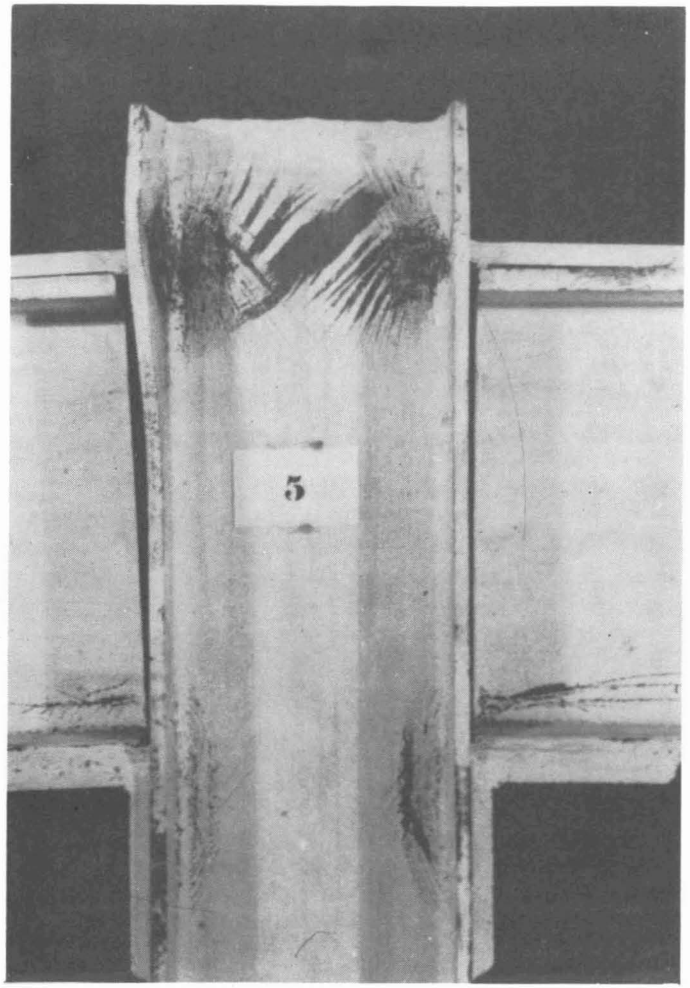


Fig. 32 - THREE TESTS BY BRANDES AND MAINS

Three test specimens from the Brandes-Mains investigation are particularly pertinent and the results are reproduced here (Figure 29). Specimens 5, 9, and 10 consisted of stub 12WF65 columns with 18WF85 beams framing onto the column flanges. No column stiffening was provided for Specimens 5 and 9, while a pair of plate stiffeners was added opposite the beam tension flanges for 10, as shown in the figure. The photograph, Figure 30, shows Specimens 5 and 10 after failure. Note in the moment-rotation curves that Specimen 5 failed to develop the full plastic moment, while 9 and 10 just reached it. Specimen 5 had the handicap of a weak seat angle which deformed under the shearing load.



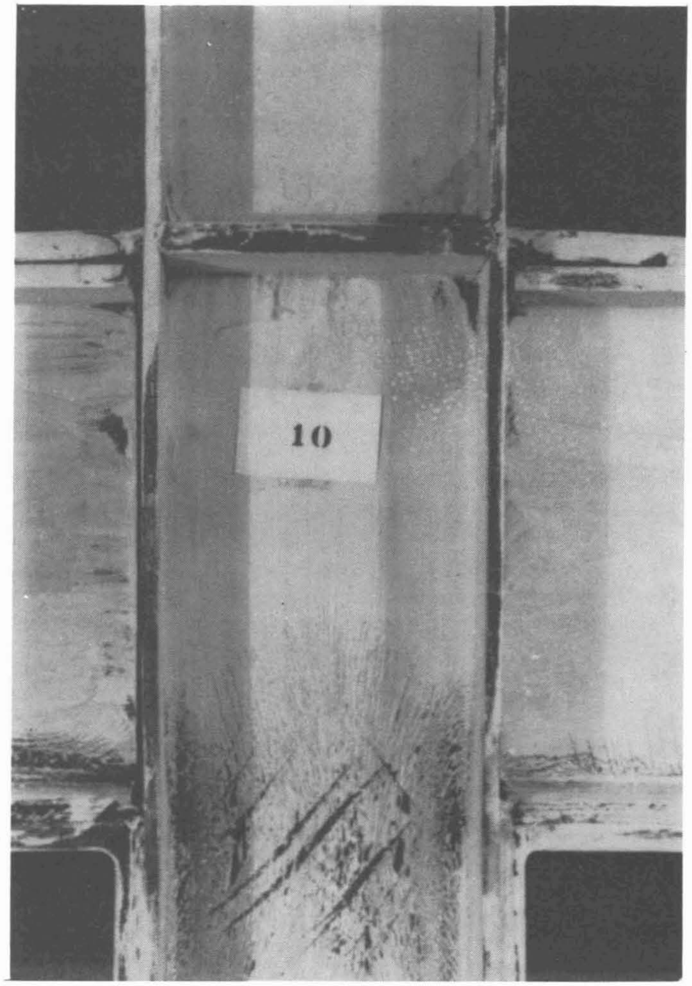


Fig. 33 Photographs of Tests 5 and 10, Brandes and Mains

These analyses to determine whether or not stiffeners are required in the column web in the region of high compression, when applied to the above three specimens for the Brandes-Mains investigation, indicated a state of overstress for Specimens 5 and 9 in both the tension and compression zones and for Specimen 10 a state of overstress in the compression zone; therefore, since the connections were satisfactory except for No. 5, there is good indication that both methods are conservative.

7. SUMMARY OF TEST RESULTS

1. Stress concentrations were found in the unstiffened column webs opposite the beam flanges as shown in Figures 27 and 28. These concentrations caused the columns (in the A series) to have less stiffness against rotation than the test beam. However, these stresses in themsleves did not cause the connection to fail to meet the criteria of developing the full plastic moment of the beams except for A-1 and A-4 where the column web failed to carry the required beam moments partly because of inelastic yielding of the column web and partly through buckling.

2. The columns showed no particular signs of distress when subjected to an axial load of 1.65x working load (an average axial stress of 24 ksi) at a time when beam working-load moments were also applied to them. Note that this conclusion is limited to rolled WF columns. ($^{d}/_{w}$ is maximum of about 40 and in this condition will develop almost the yield stress without buckling.)

3. Further, at the end of each test, with the final beam loads still applied, 2x working column load (29 ksi) was applied with no marked evidence of distress to the column.

4. The horizontal plate stiffeners yielded efficient and satisfactory service when used in these two-way beam-column connections. Formulas (6) and (13) appear to be entirely safe when used with A.I.S.C. Spec. 18 (c) which limits the thickness of a diaphragm to $1/40$th of the clear distance between flanges.

5. These tests on the vertical plate stiffeners (Series C & D) must be considered as preliminary ones only, since the vertical plate stiffeners were conceived for use with three or four-way connections. However, the vertical plate stiffeners tested did serve satisfactorily. Of the two types of vertical plates, the split beam type (D-12) showed the greater promise since the stem of the ST6WF 32.5 effectively prevented both the column web and the stiffeners themselves from buckling in the latter stages of the tests. Moreover, in contemplated use in a four-way connection the stem of the tee would serve to stiffen and strengthen the connection between the column and the beams framing to the column webs.

6. The present criterion, whether column stiffeners are required or not, A.I.S.C. specification 26 (h), that the computed stress, from $\frac{R}{t(N+2k)}$, must not exceed 24 ksi., - is much too conservative and could well be replaced by the same general formula but replacing the denominator by $t(N+4k)$.

7. A plastic analysis approach, expressed by formulas (10), (13) and (15), shows much promise of being a practical, slightly conservative criterion.

8. The beam-column connections tested had several points of stress concentrations including those existing in the butt welds connecting beams welded to the flanges of unstiffened columns, but it has been shown that these are not the critical items in determining stiffening requirements.

8. DESIGN RECOMMENDATIONS

From experience obtained in this test program a review is made of current design formulas and techniques which might usefully be employed in designing direct-welded two-way column connections.

A Series-Unstiffened Connections

This is probably the most difficult connection to analyse due to a high localized concentration of stress in the column web at the beam flanges. Also there is always the possibility of web buckling if the column web is thin. The current A.I.S.C. specifications clause 26 (h) can be used to give some idea as to when stiffening may be required, but the results obtained are very conservative. A more reasonable determination of stiffening requirements may be obtained by changing the 2k in the A.I.S.C. formula to 4k. Thus the recommended formula is: $\frac{R}{t(N+4k)}$ must not exceed 24 ksi. It is recommended that this apply to both tension and compression areas, pending further tests.

The plastic analysis approach is also recommended for use in the design of direct-welded two-way beam-column connections. This approach in practical form is expressed by equations (10), (13) and (15).

B Series - Horizontal Plate Stiffening

This is the most efficient means of stiffening a column in that it eliminates web and flange distortion. Two methods of design may be used as follows:

1. Employ stiffeners of thickness equal to the beam flanges. This may be quite uneconomical of material.

2. Determine the thickness of the stiffeners from a consideration of the modified A.I.S.C. or the plastic analysis approaches from formulas given therein, but no stiffener should be thinner than 1/40th of the clear distance between column flanges.

C & D Series - Vertical Plates or Tee-Type Stiffeners

Since these stiffeners are suggested for three or four-way connections no recommendations are made at present except that if they are used for two-way connections the applicable formulas may be used except that the t (thickness) should be checked for minimum thickness as given above for the B-type stiffeners.

This investigation is by no means complete. The suggested modification of the web-crippling formula and the new plastic analysis approach are not based on sufficient tests. They are however, believed to be safe. No tests have been made with column web stiffeners in the compression area but omitting those in the tension area. This has interesting possibilities. Further, the beam moments in the present two-way tests have been of equal magnitude. It is presently believed that recent tests on columns subjected to combinations of axial load and moment adequately cover this point of the design procedure when the beam moments are unequal. The influence of wind moment is another factor. Finally, there is the behavior of the three and four-way connections when subjected at the same time to an axial load. It is hoped that this progress report will be useful and will give designers the needed information on determining whether or not column stiffeners are required, and if required--how to design them.

Bill Added note on
[illegible] 7 [illegible]

Acknowledgements

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Summary of Coupon Test Results

2/23/56

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
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

Remarks

Notes

E in range $25000 < E < 30000$ ksi
Measured on automatic S/S Recorder
t = tension coupon
c = compression coupon
tf = tension flange coupon
tw = tension web coupon
ϵ_{st} = strain at strain hardening
σ_{pL} = Stress at prop. limit
σ_{uy} = Upper yield pt.
σ_{yL} = Yield level
σ_{ult} = Ultimate
E_{st} in range $300 < E_{st} < 700$ ksi

APPENDIX

Calculations for Design of Specimen

Columns

Working stress on columns = 14.5 ksi

derived from AISC eq $\left(\frac{P}{A}\right) = 17000 - 0.405 \left(\frac{L}{r}\right)^2$ for $L/r = 72$

Structural Section Details

Column Size	Area □" *	Area as measured	Pw kips	1.65 Pw	2 x Pw	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[(a)]	A5
* AISC handbook value. (a) Testing Machine capacity = 800 k						

Analysis of Beams and Beam-Column flange welds

All dimension of sections as measured on specimens

Beams 16WF36

$\sigma_{working} = 20$ ksi

Bending

$M_W = \sigma_W S = V_W L \qquad V_W = \frac{\sigma_W S}{L} = \frac{20 \times 56.4}{48} = 23.5$ kips

use 24 kips

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

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

$V_u = \frac{\sigma_y Z}{L}$, Z = plastic Modulus

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

Elastic Analysis of welds at working beam load

Butt welds to carry applied moment

Fillet weld to carry applied shear

$M_W = 24 \times 48\ k''$, $\quad d_F = 15.91 - 0.43 = 15.48''$

$$F_W = \frac{M_W}{d_F} = \frac{24 \times 48}{15.48} = 74.5 \text{ kips}$$

$$\sigma = \frac{F}{A\ \text{weld}} = \frac{74.5}{7.09 \times 0.431} = 24.4 \text{ ksi} > 20 \text{ ksi} \quad [\text{overstressed}]$$

$V_W = 24$ k $\qquad$ L welds = 26" $\qquad$ 1/4" fillet weld

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

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

Shear (in plastic range) = 18 wd[4] = 18(0.29)15.91 = 83 kips > 52.5 k
V_u (predicted) = 52.5 kips

(in elastic range) = 13 wd = 13(0.29)15.91 = 60 kips > 24 k [OK]

Influence of shear on V_u may be neglected if

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

$$\frac{63.76}{48} < \frac{(14.94)(0.29)}{1.732} \quad \text{or} \quad 1.33 < 2.5 \quad [\text{OK}]$$

Lateral Buckling (in the elastic range) $\qquad \dfrac{Ld}{bt} = \dfrac{96 \times 15.91}{7.09 \times 0.431} = 500 < 600$

$\sigma_{allow} = 20$ ksi

Local Buckling $\quad$ elastic range $\quad$ See 18(b) AISC spec.

$$\frac{b}{t} \leq 32, \quad \text{Actual } \frac{b}{t} = \frac{7.09}{0.431} = 16.45 < 32 \quad [\text{OK}]$$

To reach strain hardening $\dfrac{b}{t} \leq 17$[4]

∴ beams critical for local flange buckling in plastic range

To reach strain hardening $\dfrac{d}{w} \leq 55$[4] for web

$$\frac{d}{w} = \frac{15.91}{0.29} = 54.8$$ ∴ 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 σ-ϵ and M-ϕ relationship.

In nondimensional form:

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

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

Beam Rotations

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

Applying the Moment Area Theorems We Obtain:

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

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

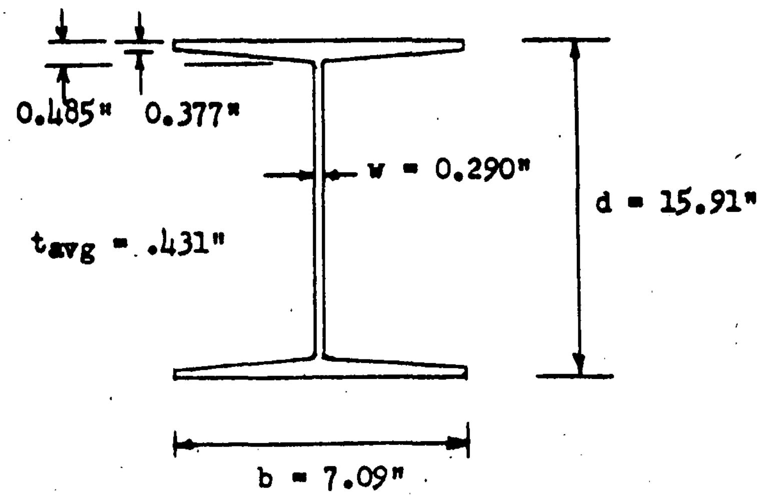
But $V_y = \frac{3EI}{L^3}\,\delta_y$

$$\therefore \theta_y = \frac{3EI}{L^3}\,\delta_y \cdot \frac{L^2}{2EI} = \frac{3}{2}\,\frac{\delta_y}{L}$$

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

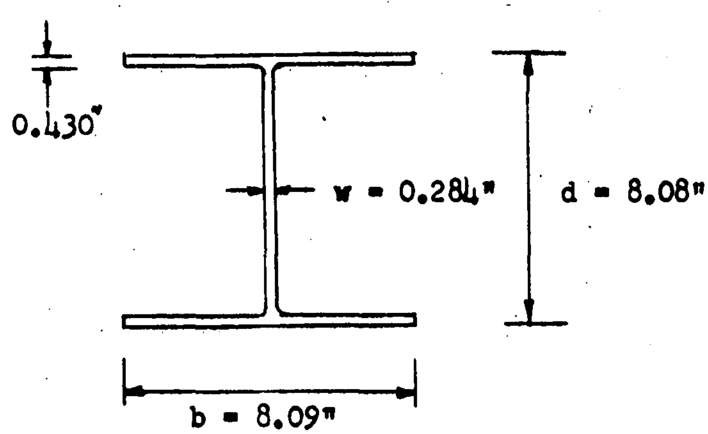
<u>Material Dimensions & Properties</u> -average values

<u>16WF36</u>
beam



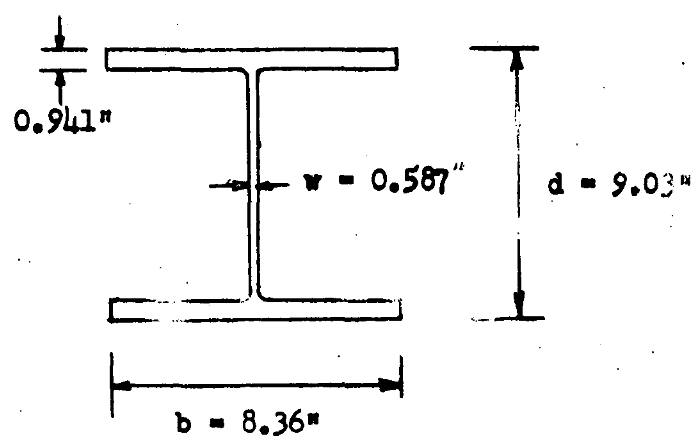
tapered flanges

<u>8WF31</u> (A1)
Column (B6)
(C9)



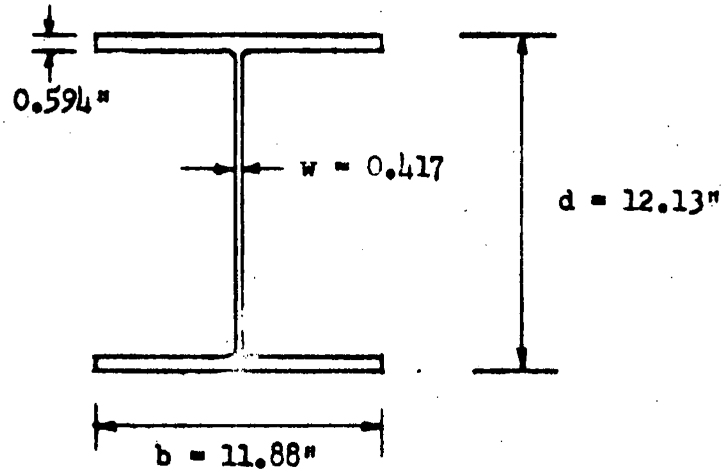
parallel flanged S

<u>8WF67</u> (A2)
Column



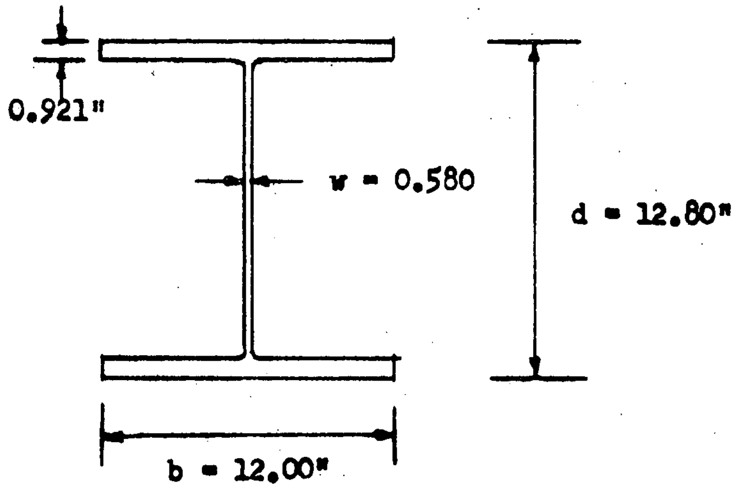
parallel flanged S

12WF65 (A4)
Column



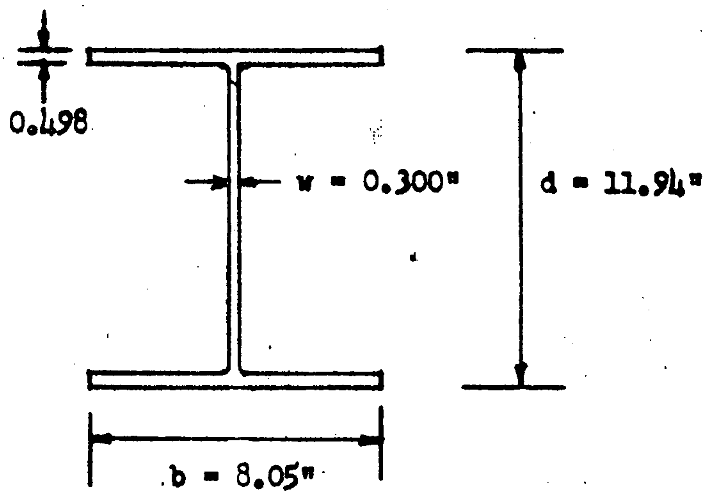
parallel fl nged

12WF99 (A5)
Column



parallel flanged

12WF40 (B8)
Column (C11)



parallel flanged

Plate thickness average values - from coupons & measurements on specimens

B6	0.439"	(7/16)	b = 3.9"
B8	0.264"	(1/4)	b = 3.9"
C9, C11	0.305"	(5/16)	d = 22"

3.545
.431

3545
10635
14180

1.527895
2

3.054

.377
.054
(.431)(3.545)

Section Properties

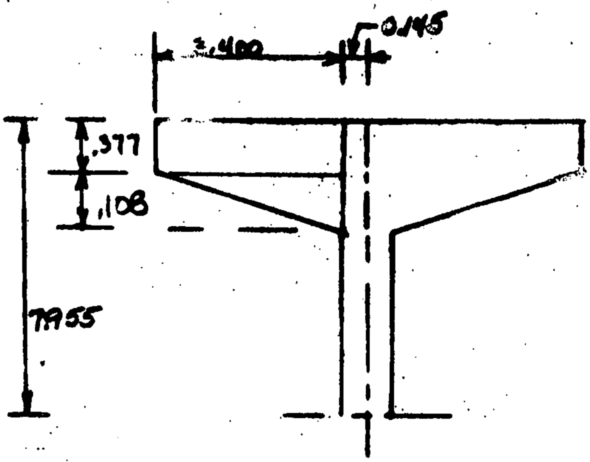
16WF36 Beams

$$\frac{A}{4} = (3.4)(.377) = 1.283$$

$$\frac{.108}{2}(3.4) = 0.184$$

$$(.145)(7.96) = \underline{1.155}$$
$$2.622$$

$$A = 10.488 \ \square''$$



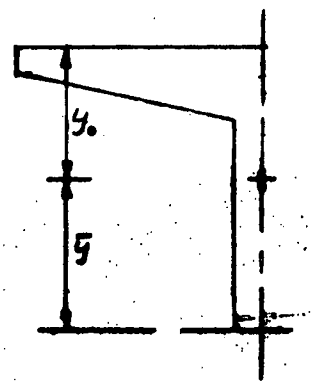
$$\frac{I_{xx}}{4} = (1.283)(7.766)^2 = 77.40$$
$$(0.184)(7.542)^2 = 10.46$$
$$\frac{(0.145)(7.955)^3}{12} = 6.11$$
$$(1.155)(3.978)^2 = \underline{18.27}$$
$$112.24$$

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

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

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

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



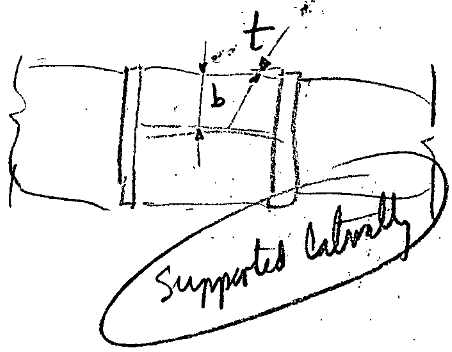
$$\bar{y} = \frac{15.94}{2.622} = 6.08''$$

$$y_0 = \begin{array}{r} 7.96'' \\ \underline{-6.08''} \\ 1.88'' \end{array}$$

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

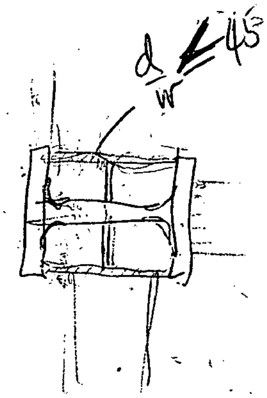
1. b/t limit can be met without increase of area by cutting down on b!

2. $\frac{b}{t} = 16$ would be adequate if no plastic deformation req'd.



3. How to justify stability of Type C connection

4. Look into $\frac{d}{w}$ of "C"

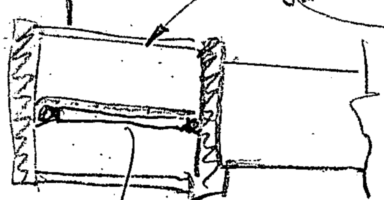


$r = .3b$

$\frac{L}{.3b} = 40$

$\frac{L}{b} = 12$

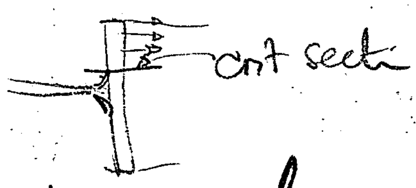
5. web plate



6. Get out that old analysis by a European referred to by Wright in his thesis

7. is $w_s \geq w_c$ too safe?

8. check



9. Report to be read, published?

Jan 15