STRESS DISTRIBUTION IN SOME COMMON WELDED
BEAM-TO-COLUMN CONNECTIONS

by

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ABSTRACT

This is a report on tests of welded beam-to-column connections carried out in the Materials Testing Laboratory, Department of Civil Engineering, University of British Columbia.

The test series included four basic types of connection commonly used in practice, and for each type, a set of stress distribution curves has been given for each part of the connection. Realizing also, that for a connection to be of any practical use, it must have adequate rotation capacity, moment-rotation curves have also been given. From the various stress curves given, an attempt has been made to tabulate magnification factors, which are described in the text, and in this way correlate the peak stress values obtained with the theoretical linear stresses assumed. In order to do this for the column of each connection, a linear stress distribution was assumed, using information obtained from the tests carried out. Finally, the calculated values for each connection are compared in the concluding section.
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NOTATIONS

\( M \) = applied moment
\( M_R \) = resisting moment at the connection
\( Z \) = section modulus
\( \sigma \) = normal stress
\( \sigma_t, \sigma_c \) = normal tensile and compressive stresses
\( \sigma_a \) = allowable design stress

\( E \) = modulus of elasticity
\( \epsilon \) = normal strain
\( \delta \) = deflection or elongation
\( T \) = tensile force \((\sigma_tA)\)
\( C \) = compressive force \((\sigma_cA)\)
\( P, Q, R, W, \) = applied loads

\( A \) = area of cross-section
\( A_w, A_F \) = areas of web and flange of structural section
\( D_b, D_c \) = depth of beam and column sections
\( B_b, B_c \) = breadth of beam and column sections
\( t_b, t_c \) = flange thickness of beam and column sections
\( w_b, w_c \) = web thickness of beam and column sections
\( k_b, k_c \) = \( k \)-distance of beam and column sections
SECTION A

INTRODUCTION

(i) General:

From the design viewpoint, welded connections are classified as "rigid", as opposed to "flexible" or "pinned" connections which have little or no resistance to end rotation. Due to this resistance to end rotation in rigid connections, a resisting moment is developed, which manifests itself in the beam in the form of direct stress in the flanges and web, with the flanges more highly stressed than the web. Providing this stress is within the elastic range and assuming plane sections to remain plane, a linear stress distribution, as shown in figure 1 is assumed to exist across the beam web and flanges.

This stress distribution will not necessarily be obtained under practical conditions, and it is the object of this paper to determine the distribution of these stresses in both the beam and column sections under static loading. The four basic types of connection tested are those shown in figure 2. In all four connections, the beam was welded to the column flange.
To follow p. 1

Fig. 1  Theoretical Linear Stress Distribution in Beam Flanges and Web
Connection A:

Here the beams were welded directly to the column flanges. Two connections were tested.

A-1 - Fig. 2(a): had a continuous fillet weld around the periphery of the beam section.

A-2 - Fig. 2(b): had 1/4" single butt welds across all the beam flanges, and 6" of 1/4" fillet weld on both sides of the beam web.

Connection B:

Fig. 2(c): had 1/4" single butt welds across all beam flanges, and 1/4" fillet welds on both sides of the beam web. This connection also had boxing plates in the column section, which were welded into place with 1/4" fillet welds on both sides of the boxing plates.

Connection C:

Fig. 2(d): here the beams were welded to the column flanges through moment plates. The tension moment plate was welded to the column flange with a 5/16" single butt weld, and to the beam tension flange with 1/4" fillet weld. The compression moment plate was welded to the column flange with 1/4" fillet welds on both sides of the moment plate, and to the compression beam flange with 1/4" fillet welds on each edge of the moment plate as shown in the figure. The "web connecting plate" was welded to the column flange with 1/4" fillet welds on both sides of this plate, and to the beam web by 1/4" fillet welds.

Connection D:

Fig. 2(e): basically this connection was the same as connection C. Here, however, there were boxing plates welded into the column section using
To follow p. 2

**Connection A-1.**

- \( \frac{1}{8} \)" fillet weld continuous around beam periphery.

**View A-A.** (a)

**Connection A-2.**

- \( \frac{1}{8} \)" fillet welds both sides of web.
- \( \frac{1}{4} \)" single butt welds across all beam flanges.

**View B-B.** (b)

**Connection B.**

- \( \frac{1}{8} \)" fillet weld both sides of beam web.
- \( \frac{1}{4} \)" single butt welds across all beam flanges.
- \( \frac{1}{4} \)" fillet welds both sides of boxing plates.

**View C-C.** (c)

*Fig. 2*
**Connection C.**

- **Tension Moment Plate**
- **Web Connecting Plate**
  - on one side only of beam web.
- **COMP Moment Plate.**

- **d** View A-A.
  - 1/4" fillet welds connecting tension moment moment plate to beam flanges.
  - 1/4" fillet welds connecting 'web connecting plate' to beam web, and column flange.

**Connection D.**

- **Tension Moment Plate**
- **Web Connecting Plate**
  - on one side only of beam web.
- **COMP Moment Plate.**
  - 1/2" fillet welds both sides of boxing plates.

- **e** View E-E.
  - 1/4" fillet weld connecting tension moment plate to beam flange.
  - 1/4" fillet weld connecting 'web connecting plate' to column flange, and beam web.
  - 1/4" fillet weld connecting 'web connecting plate' to column flange.
1/4" fillet welds on both sides of the boxing plates as shown.

There are, of course, many factors which can affect the stress distributions in both the beam and column sections. The following is a discussion of some of these.

(ii) Some Factors Affecting Stress Distribution:

1. Residual Stresses:

Residual stresses are those present in a solid body when no external forces are acting on the body, or they may be considered as the stresses that remain in a structure as a result of localized deformations. Residual stresses introduced by welding are caused by localized heating associated with the welding process, and to a lesser degree by shrinkage of the weld material.

In measuring the applied stresses with electrical resistance strain gauges, it has been assumed that under zero external loading no stress exists in the specimen, and hence the residual stresses have been neglected completely. The effect of neglecting these residual stresses does not seem to have been very consequential, since, for each test carried out, strain gauges in similar positions on both beams of the same connection, have indicated similar stress values. In particular, the resulting stress distribution curves show a fairly symmetrical form for all applied moments.

2. Beam Not Welded at Right Angles to Column:

This is a very common occurrence in welding practice, and one which has significant features under the test conditions. To understand fully the resulting distortion of the stress pattern, figure 3 has been drawn.
Fig. 3  Beam Not Welded at Right-Angles to Column
Figure 3, part (a) shows the beam framing into the column, and the applied moment $M$ acting on the connection. Part (b) shows the beam in true section, twisted through an angle $\alpha$ with respect to the column centre-line and part (c) shows the components of $M$, the applied moment, parallel to and at right angles to the beam web. These components may be stated as:

(a) $M_1 = M \cos \alpha$ - in plane at beam web
(b) $M_2 = M \sin \alpha$ - at right angles to beam web

and they induce the following effects:

$M_1$ gives the linear theoretical diagram which normally exists in the I-beam section when subjected to a moment in the plane of the web.

$M_2$ tends to cause a rotation of the beam in the direction as shown, and hence the stresses in the section are now modified.

Consider the points $A_1$, $A_2$, $B_1$ and $B_2$.

Due to $M_1$ - tension at $B_1$ and $B_2$; compression at $A_1$ and $A_2$

Due to $M_2$ - tension at $A_1$ and $B_1$; compression at $A_2$ and $B_2$.

It can be seen now that in the compression zone of the beam, $A_1$ and the half flange on that side of the web will have a reduced stress, whereas $A_2$ and the half flange on that side of the web will have an increased stress. Similar conditions exist in the tension zone. This then accounts for the trapezoidal stress diagram shown at the two flanges. Using this same argument, a distortion of the practical stress distributions due to this twist in welded position of the beam, can also be deduced.

3. Allowable Tolerances in Rolled Sections:

All rolled structural sections must pass a tolerance test before
being supplied to the steel fabricator. Having passed these tests, however, does not necessarily mean that the section is not distorted, quite the contrary. Apart from variations in weight, length and thickness of cross-sectional detail, a flange distortion as shown in figure 4 can exist.

![Diagram of beam flanges in rolling distortion](image)

**Fig. 4** Distortion of Beam Flanges in Rolling

This type of distortion would lead to a trapezoidal flange stress pattern similar to that shown above in figure 3, and once again, a direct influence on the stress pattern for the section.

The above three features are perhaps the most significant in distorting the stress patterns from their true shapes. There are others also, such as discontinuous welds, which can introduce these stress concentrations.¹

(iii) Stress Measurement Considerations:

Since "welded" connections are dealt with in this test series, the stresses required are those actually developed in the weld sections. In order to do this electrical resistance strain gauges were proposed, and used successfully. These strain gauges, however, require a smooth surface to be glued to, such that accurate results can be obtained. Now any weld surface is very irregular, and since any action, such as filing, or grinding a section to the required surface conditions is liable to change the stress characteristics, placing gauges on the weld section was thus eliminated. This left the other possibility of placing small gauges directly opposite the weld section. Small gauges, such that the stress measured would be extremely close, if not equal, to that actually existing in the weld itself. Two types of gauge were used,

(a) SR4 type A8

(b) Phillips type PR9214

these gauges, have coil dimensions of approximately \( \frac{1}{8}'' \times \frac{3}{16}'' \).

Another major feature to be considered was range of the gauges. Maximum strain measurement required for the test, was that of yield stress i.e. in the case of steel this value is 0.1% to 0.2% strain. Since the gauges are linear up to approximately 1.0% strain, then strain measurements on steel could be accomplished with these gauges.

Yield stress was required for all steel sections, for which stress curves were to be drawn. In order to obtain these values, tension test specimens, 18 in number, were selected from the various specimens after testing, and the yield point established by tensile tests. The values obtained varied widely, and generally were quite high, therefore a standard value of 35 kip/in.\(^2\) was adopted as yield stress, for all specimens. The actual values indicated by the tension tests were above this value.
(i) **Modified Stress-Strain Diagram:**

Subjecting a steel test specimen to a tension test reveals a graph of stress versus strain as shown by the curve OABCDE in figure 5(a). Initially there is a steep linear section called the "Elastic Region," up to point B which is the "Upper Yield Point." Then the curve drops abruptly to point C, the "Lower Yield Point," and from here continues in a straight line without stress increase to point D, this range now being called the "Plastic Region." After point D, the stress begins to increase once again, until "Ultimate Stress" is reached, point E, and finally failure at point F. This latter range DE is generally called the "Strain Hardening Region."

As with these tension specimens, so too, do the fibres of a structural element act, under tensile or compressive forces. Figure 5(b) shows an I-beam subjected to a moment M, and three stress diagrams developed in the beam, for three different magnitudes of M. Figure 5b(i) indicates that yield stress has developed at the outer beam fibres, i.e. a strain in the outer fibres corresponding to point A in 5(a). Figure 5b(ii) indicates a yield
Fig. 5
stress developed over a greater section of the beam due to an increased moment, and hence a strain increase in the outer fibres, with no increase in stress accompanying it. This, then, would correspond to a strain in the outer fibres, in the region CD. If now the strain in the outer fibres was allowed to increase until it reached a value in the strain hardening region, the stress in the outer fibres would increase once again, but in the theory of "Limit Design," these strain hardening stresses are usually neglected, and this being so, the stress is assumed to remain at the yield value until ultimate failure, at which stage a "Plastic Hinge" is said to form. This condition has been shown in 5b(iii), the whole section having now developed yield stress, and an increase in moment over and above this condition cannot be equalized by a stress increase, hence failure. Usually, as well as the strain hardening stresses, the peak at A is neglected, and finally the modified stress-strain curve OACDG is left. The moment corresponding to collapse conditions is usually designated $M_p$ or "Plastic Moment".

In the following chapters where stress diagrams have been drawn, the modified curve has been used as a basis for stress analysis, strain hardening stresses have been neglected.

(ii) Failure Theory:

Having developed the theory of stress build-up in the I-section, it is now possible to investigate the failure modes of the connection, from the point of view of both beam and column failures.

Application of moment to the connection results in the formation of stresses, as shown above, which will be greatest in the beam flanges, and directly opposite these flanges, in the column section. These highly stressed
Fig. 6  Beam Failure

Fig. 8  Column Failure
sections then, lead to the possibility of three types of failure:

1. Beam Failure
2. Column Failure
3. Weld Failure.

Consider the mechanism of each in turn.

1. Beam Failure:

In this failure mechanism, the applied moment is increased until a plastic hinge is formed in the beam section, at which time, theoretically, yield stress \( f_y \), is developed over the total beam section. In practice, however, due to the fact that without boxing plates the column flanges are not completely rigid, but will deflect as cantilevers, there will be a stress relief at the outside edges of the beam flanges. It seems likely then that full theoretical plastic moment need not necessarily be developed in the beam of this connection for a beam failure to occur. Consider figure 6 above. Diagram (a) shows the connection with a moment applied to one of the beams, producing tensile and compressive forces in the beam flanges of T and C respectively. Diagram (b) represents the position of the beam at failure, with a rotation during loading of \( \alpha \) as a result of plastic yielding in the beam. Diagrams (c) and (d) represent sections through the beam showing tension and compression column flanges respectively, at the point of formation of full plastic moment in the beam. The column flanges in (c) and (d) are bowed inwards and outwards respectively, opposite the compression and tension flanges of the beam, indicating the possible stress relief mentioned earlier.

This possible stress relief would induce plastic stresses only in the central section of the beam flanges, and would produce stresses in the beam
section as shown in figure 7(a). From this, only a section of the beam as shown cross-hatched in figure 7(b) would have yield stresses induced.

Fig. 7 Possible Stress Relief for Connections Without Boxing Plates

The width "x" shown in figure 7(b) would depend upon such features as:

(a) thickness of column web;
(b) $k_c = k$-distance for column section,

where the $k$-distance of an I-beam is defined as the thickness of the flange plus the length of web which contains the fillets. The $k$-distance for the beam section has been dimensioned in figure 7(b). Generally for the two sections of the connection:

$k_b = k$-distance for the beam

$k_c = k$-distance for the column.
2. Column Failure:

In a column failure, the applied moment is increased until the plastic hinge is formed in the web of the column. The formation of this plastic hinge induces yield stress ($\delta_y$) over a length of the column web directly opposite the beam. The total length of the column web stressed to yield stress in a failure of this type, includes a length equal to the total depth of the beam, plus a length of column web either side of this which has been deduced by Beedle\(^2\) as three times the k-distance of the column section (refer figure 9). This plastic hinge formation is accompanied by tensile yielding, and compressive buckling of the column web opposite the tension and compression beam flanges respectively. Figure 8 represents the failure mode encountered. Diagram (a) shows the connection with a moment applied producing compressive and tensile forces of C and T respectively in the beam flanges. Diagram (b) represents the position of the beam at failure, with a rotation of "a" caused by the plastic deformations in the column web. Diagrams (c) and (d) represent sections through the beam showing the tension and compression beam flanges at failure, and in particular the tensile yielding of the column web opposite the tension flange of the beam, and the buckling of the column web opposite the compression flange of the beam. This type of failure would be exhibited by connections without boxing plates in the column section.

From the limit design viewpoint, Beedle\(^3\) has formulated a criterion for the collapse condition in a column section under applied moment. This condition is shown in figure 9.


\(^3\) Ibid.
Fig. 9 Beedle's Criterion for Column Failure
Neglecting strain hardening stresses, Beedle⁴ has specified the length of column web over which yield stress is induced at failure to be

\[ \left[ d_b + 6k_c \right] \]

where \( d_b \) = depth of beam section
\( k_c \) = k-distance for the column section.

For this condition, the collapse moment can be calculated as

\[ M_p = \frac{6u \cdot t_w}{4} \left[ d_b + 6k_c \right] \]

where \( t_w \) = thickness of the column web.

3. Weld Failure:

This third type of failure occurs at points where a high stress concentration exists. It is regarded, however, as the most serious of all three, because once a fracture is initiated in the weld material, there is an immediate redistribution of stress which will allow the fracture to enlarge until complete rupture of the weld becomes possible. The material of the weld indicates a brittle nature under a tensile test, and hence this rupturing process can be very rapid indeed.

In the beam and column failures described earlier, this abruptness of collapse would not be expected. This is so, since the sections which are highly stressed and deform to define the failure mode, do not exhibit a brittle nature under the tensile test, but have a stress-strain curve of the type shown

⁴ *Ibid.* p.177-178
in figure 5. From this then, the stresses produced due to strain hardening would allow the connection to deform slowly under increasing applied moment.
(i) General:

All specimens were fabricated with the same beam and column sizes, so that a direct comparison of the measured test values could be made, for the four different connections.

The quantities measured during the test were:

1. Applied load
2. Connection rotation
3. Strain at predetermined points.

From these measured values moment-rotation curves have been drawn, which indicate a rigidity comparison, and from the measured strains, a distribution of stress has been computed and plotted. These results then, comprise the basis for comparison of the four connections tested.

(ii) The Test Specimen:

Figure 10 shows the basic test specimen. It consisted primarily of 4 beams, 8" x 5 1/8" W.F. at 17.0#/ft, welded to the flanges of a central column
Fig. 10  The Test Specimen
6" x 6" W.F. at 15.0이며/ft. There were two loads applied to the specimen:

1. Compression load "Q" applied by the testing machine, causing direct compression stress in the column.
2. Loads "P" applied at the ends of the beam by turnbuckles, causing a moment M to develop at each connection.

To avoid scoring the machined surfaces of the Olsen tester, 8" x 8" x 1" plates were welded to the ends of the central column. Also, at the end of each beam two 5 1/4" x 3 1/2" x 1/2" bearing plates were welded, one on each side of the beam web, to provide bearing resistance against the applied P-loads.

Initially, due to the limited amount of electrical strain measurement switching gear available, and the number of strain gauges required for each test, only two of the four beams could have test data recorded from them. In order to economize, a method of stiffening two beams during the test was finally adopted so that now two separate tests could be carried out on each specimen, and test data gained from all four beams on the one specimen.

To do this a stiffening beam, as shown in figure 11, was connected through two 5/8" rods to the top two beams of the test specimen. The effect of this was to stress the lower connections to failure, allowing only elastic strains in the top connections. An analysis of the forces produced has been given below. Since only two connections of each type were proposed for the test series, a further economy was adopted in the form of two different connections on the one specimen, one for the top beams and the other for the bottom beams. Finally, a total of four test specimens was used.
Fig. 11  Stiffening Beam in Position

Fig. 12  Deflection Analysis at the Top Connection
(iii) Load Analysis for Stiffening Beam:

In figure 12 let:

P = load applied by turn-buckle.
R = load in the two rods together.
\( \delta_1 \) = deflection of stiffening beam at load R.
\( \delta_2 \) = elongation of the two rods under applied R-load.
\( \delta_3 \) = deflection of bottom beam at load R.
E = Youngs Modulus.
I = moment of inertia of beam about axis of bending.
A = cross-sectional area of two rods.

From geometry then,

\[ \delta_1 + \delta_2 - \delta_3 = 0 \]  \hspace{1cm} (1)

From "Strength of Materials",

\[ \delta_1 = \frac{R b^3}{3EI_1} \] \hspace{1cm} \[ \delta_2 = \frac{R L A}{AE} \]

\[ \delta_3 = \frac{P b^2}{6EI_2} (3a - b) - \frac{R b^3}{3EI_2} \]

Substituting these values into the above deflection equation (1), and assuming E to be the same for all elements, leaves:

\[ R \left[ \frac{b^3}{3I_1} + \frac{L}{A} + \frac{b^3}{3I_2} \right] = \frac{P}{6I_2} \left[ 3ab^2 - b^3 \right] \]  \hspace{1cm} (2)
Finally, substituting the known values into (2) leaves:

\[ R = 0.53P \]

Thus the moment in the top connection was reduced to approximately half the applied value in this manner, e.g.

actual applied moment = \( Pa - Rb \)
\[ = Pa - 0.53Pb \]
\[ = 18.7P \text{ kip-ins.} \]

moment applied without stiffening beam = \( Pa \)
\[ = 33P \text{ kip-ins.} \]

For each specimen, the weaker of the two connections was always tested first, and in this way, due to the lower failure load for this connection, the smallest moment possible was applied to the top or second connection, which still had to be tested. When the second or stronger connection for this specimen was tested, the moment produced at the top connection did not matter since the test on it had already been carried out.

1. Consider the first specimen, with connections A-2 and B.
   A-2 was tested first.
   Failure load on A-2 = 11.7 kips.
   Moment produced at B = 18.7P
   \[ = 218.8 \text{ kip-ins.} \]

Values of yield stress were not reached until an applied moment of approximately 350 kip-ins. was reached in the test on connection B.

2. Consider the second specimen, with connections C and D.
   C was tested first.
Failure load on C = 12.5 kips.

Moment produced at D = 18.7P = 233.8 kip-ins.

Values of yield stress were not reached until an applied moment of approximately 400 kip-ins. was reached in the test on connection D.

This indicates then, that the stiffening beam held the stresses in the untested connections to a value below that of yield stress.

(iv) Load Application and Measurement:

Test P-loads were applied to the specimen by two hand operated, double-acting turnbuckles, connected between the points A and B shown on figure 10. Since the distance between the beams was 410" c/c, these turnbuckles had to be made specially for the tests. Load measurement was carried out by means of strain gauges on the turnbuckle rods. To eliminate any stress effects from the threaded section, a clear rod length of approximately 9" was required. Considering these features then, the final adopted design is as shown in figure 13(a).

On the turnbuckle, the strain gauges were placed diametrically opposite each other, in order that the average strain would be recorded. In this way any stresses due to bending of the rod would not register on the strain indicator, since one gauge would be in compression and the other in tension. The section on which the gauges were placed was reduced in diameter, such that a higher sensitivity could be attained, i.e. greater strain for same load. The turnbuckles were connected to the beam ends through a linkage system, as shown in figure 13(b). This system allowed a vertical pull to be applied, regardless of beam deflections.
Fig. 13(a)  Detail of Turn-Buckle

Section reduced to \( \frac{1}{4} \) \( \phi \).

two strain gauges diametrically opposite.

Fig. 13(b)  Detail of Load Connection at Beam Ends

Bottom flange width reduced to \( 1'' \) for clearance.

Bearing plates.

13\( \frac{1}{2} \)’’ \( \times \) 4\( \frac{1}{4} \)’’ \( \times \) 1’’

Turnbuckle.

14\( \frac{1}{2} \)’’ \( \phi \) pins.
Knowing the applied load and the distance from the loaded point to the connection, the applied moments could be calculated directly.

(v) Angular Measurement:

Figure 14 shows the system used to measure the angle change at the connections. The light aluminum bracket was suspended from the centre of the column web by four pointed bolts set into punch marks. Since under load the centre of the column web would have negligible or no movement, this bracket could be considered as fixed, relative to the column. Two 1/8" rods with wing nuts, one each on the top and bottom sections, acted as "clamps" to hold these four bolts firmly in position. Finally, to stop the whole system from rotating about the centre line of the column, two "stays" were connected to the bottom clamp, which in bearing against the sides of the column flange, held the bracket against rotation. To read the angle change, dial gauges, connected to the top of magnetic bases, were placed on the beam flanges as shown. The scale of these gauges could be extended by adjusting the brackets at the sections shown. It should be noted that the top dial gauge is outside the bracket and the bottom dial gauge inside the bracket. The reason for this is that in the event of a sudden failure, such as a weld failure, the dial gauge would always be moving away from the bracket, and thus there is less likelihood of breaking the dial gauge.

Rotation at the connection occurred due to the elongation of the tension flange and shortening of the compression flange, since angular rotation of the cross-section at this point would be negligible. Considering the rotation to occur about the centre of the beam, the following equations will be true since $\phi_1$ and $\phi_2$ are very small angles:
View A-A.

Fig. 14  Details of Beam Rotation Measurement
\[
\begin{align*}
\rho_1 &= \frac{d_1}{r} \\
\rho_2 &= \frac{d_2}{r}
\end{align*}
\]

where \(d_1\) and \(d_2\) are the changes in dial readings and \(\rho_1\), \(\rho_2\) and \(r\) as in the diagram. \(\rho_1\) and \(\rho_2\) are measured in radians. The final angle change was taken as the average of these two "half beam rotation" values, \(\rho_1\) and \(\rho_2\).

(vi) Strain Measurement:

This was done directly, using electrical resistance strain gauges, and strain measuring equipment. From these measured strains, the stresses induced could be calculated from the equation

\[
\frac{\varepsilon}{E} = \epsilon
\]

or \(\varepsilon = \epsilon E\)

Assuming a value for \(E\) then, stress distributions could be plotted for all loads. There are many features which can create distortion in these stress distributions. It can be realized then, that for the same applied moment on connections of the same type, stress distributions would not be the same in every detail. However, experiment showed that if "check" gauges were placed at pertinent points on the second connection, then very close values could be obtained, when both connections were under the same loading conditions. This was the system adopted during testing. The first connection had gauges at all points where a strain measurement was required, and the second connection had gauges which acted only as a check. This method, as well as economizing on the number
of gauges used, showed the results of similar connections to be very close indeed, even though distortions in the final curves are obvious.

One major correction to be made to the measured strains was a correction for gauge factor. In all of the tests there were more than 20 gauges to be read, their factors being different. Since only one gauge factor was set on the strain measuring bridge, an error was introduced. To correct this error the following equation was used:

\[
\text{corrected reading} = \frac{G_s}{G} \times \text{indicated reading}
\]

where:

\[ G_s = \text{Gauge factor set on bridge.} \]
\[ G = \text{True gauge factor of strain gauge.} \]
SECTION D

TEST PROCEDURE

(i) Calibration of Turnbuckles:

As shown previously, the load was applied by turnbuckles at the ends of the test beams, creating moments at the connections. In order to be able to calculate these applied moments, however, the value of the load applied by these turnbuckles had to be measured. This was done with the use of strain gauges on the turnbuckle rods. Thus, before any actual testing could be carried out, these strain gauges had to be calibrated in terms of a known load, i.e. the strain in these strain gauges corresponding to a known load on the turnbuckle had to be measured. To do this, the turnbuckles were connected, one at a time, to the upper and lower heads of a Tinius Olsen tension tester. The tensile loads applied to the turnbuckle were measured directly from the load indicator on the testing machine, and the corresponding strains induced in the turnbuckle, were measured with a strain indicator connected to the strain gauges on the turnbuckle rod. Both turnbuckles were calibrated in this manner and the results of these calibrations have been plotted on graphs number 1 and 2. With these graphs, the loads, and hence moments applied to the connections under test,
could be calculated. It will be noticed that two curves are given on each graph, one for "load" and the other for "unload". These two graphs correspond to strain and load readings taken during the calibrations, when the turnbuckle was first loaded, and then unloaded. To calculate the applied moments in test, the "load" curves were used.

(ii) Procedure for Testing:

Initially, with no moment applied to the connections, i.e. in figure 15, \( P = 0 \), the load \( Q \) on the column would be increased to 30 kips (6.5 k/in.\(^2\)). Loads \( P \) would then be applied, creating moments at the connections. In the case of connections A and C, since a small failure moment was predicted, the P-loads were given small increments between each set of recordings. In the case of connections B and D, however, a larger failure moment was predicted, and correspondingly larger increments were used.

Besides creating a moment at the connections under test, the P-loads also caused a compression in the section of the column between the beams, as shown in figure 15. This meant that the longitudinal stress in this section of the column was increasing with every load increment, i.e. load at any time during the test would be \( (Q + 2P) \). For the purposes of the test, however, this load of \( (Q + 2P) \) was required to remain constant, so the compressive load \( Q \) on the column was reduced with every increase in \( P \). In the case of connections B and D the failure load was so high that this decrease in \( Q \) was stopped at a load of \( Q = 5 \) kips, such that the specimen could still be kept steady.

This loading system continued until the onset of failure, at which point, the specimen was left to creep for about 12 hours before the final set of readings was taken.
Equilibrium of Vertical Forces on Test Specimen

Fig. 15
(i) General:

The test results are presented here in four sections, each section being devoted to a single connection. The presentation within each section is similar and has been written under the following sub-headings.

A. Description of Connection.
B. Failure Mechanism
C. Moment-rotation Characteristics.
D. Stress Distribution
   a) Gauge Layout
   b) Discussion of Curves
   c) Failure Load and Moment.
E. Calculations.

Values for $E$ of $(30 \times 10^3)$ kip/in.$^2$, and for $f_y$ of 35 kip/in.$^2$ were adopted throughout, since the values obtained from test specimens varied widely.
In plotting the stress distributions in the various sections of the connections for both beam and column webs, the average stress from the two sides has been plotted as a single curve. However, in the case of beam flanges, moment plates and boxing plates, two curves for the same applied moment have been plotted for each, one for the "outer" surface, and one for the "inner" surface. As a definition, the inner surfaces of the beam flanges face each other and have the beam web connecting into them. In order to save space and time, selected stress curves for increasing applied moment have been plotted on the same sheet. Also, in the case of the beam flanges, moment plates and boxing plates, these curves for inner and outer surfaces have been plotted on the same sheet. The applied moment, corresponding to these plotted curves, has been indicated in the accompanying written material. Although two connections of each type were tested, only one set of stress curves has been plotted for each type.

In determining the gauge layout for each test specimen, consideration was given to both the type of failure expected and a layout which would yield the required stress distributions economically. Also, for simplicity, the layout to be used was to be adaptable to all specimens, with minor modifications where required. For example, the case of a connection with-or without boxing plates. The layout adopted finally for each connection has been shown diagrammatically under "Gauge Layout."

As mentioned earlier, to economize on the number of gauges used, one connection of the two in each test had strain gauges at all points where stress was required, and the second connection had check gauges at the most important points only. This system allowed a check on the measured strain values. Since, in the strain gauge layout diagrams, many sectional views are required to describe the layout completely, in order to reduce this number of
views on the beam and column webs, diagonal lines have been inscribed on those gauges which have a second gauge on the opposite side of the web. This diagonal line then, indicates that there is a gauge on both sides of the web at this position.

As a final check on the resulting curves, the resisting moments developed under elastic conditions were calculated using a planimeter to measure the area beneath each curve set. These measured values were checked in two ways

1. applied moment = resisting moment
2. $\sum$ horizontal forces = 0

where in (2) the horizontal forces arise as tension and compression forces due to the application of moment to the connection. For all connections tested, these two checks were carried out on both beam and column sections for the plotted stress curves, and the results of these checks have been given in the "Calculations" at the end of each connection write-up. This check has been given regardless of whether stresses above yield stress ($6_y$) exist in the individual sections of the connections, assuming a maximum stress of $6_y$ is developed. This check can be carried out with very little induced error due to neglecting the strain hardening stresses, as the increase in stress above the yield point is very small.

From the given curves of stress distribution in the beam flanges, moment plates and column web, three factors have been calculated for elastic conditions:

1. "$a$" = stress magnification factor for stress peaks on outer surfaces of both beam flanges.

2. "$D$" = width of outer surface of beam flange across which this peak stress, over and above the theoretical linear stress, acts.

Consider the values shown in figure 16(a):

\[
\alpha = \frac{\text{Actual peak stress at outer surface}}{\text{Theoretical stress at outer surface}} = \frac{\sigma_2}{\sigma_c}
\]  

(1)

These values of \(\alpha\) are calculated only for the outer surfaces of both beam flanges, since it is this outer surface stress which is used in the design of steel members.

Also, "D" is the width of beam flange across which the actual stress is greater than the theoretical value \(\sigma_c\) at the outer surface. Since the more highly stressed section occurs over the centre as indicated in figure 16(a), then this width \(D\) has been calculated as

\[
D = \left[ w_c + Xk_c \right]
\]

(2)

where

- \(X\) = a dimensionless constant
- \(w_c\) = thickness of column web
- \(k_c\) = flange thickness plus fillet width of column,

(Shown in figure 16(a)). The distance \(D\) has been calculated in this form as, without boxing plates, the web thickness of the column plus the width of the column flange containing the fillets either side of the web, is the strongest section against which the beam flanges have to bear. Consequently, this stronger central section will induce a peak stress value in both tension and compression beam flanges, since the column flanges outside this central region are relatively flexible and will deflect under applied load.
Theoretical stress across beam section.

Actual stress pattern across outer surface of beam flanges.

\[ D = \text{width of beam flange across which stress is greater than theoretical.} \]

\( \sigma_0 \) — Theoretical stress at outer surface of beam flanges.

\( \sigma_2 \) — Actual peak stress at outer surface of beam flanges.

**Fig. 16(a): Actual and Theoretical Beam Flange Stress Distributions**
To follow p. 27

To follow p. 27

**Stress Patterns Without Moment Plates.**

- 6 = Actual peak stress value.
- 6 = Theoretical peak stress value.

**Fig. 16b(1)**

<table>
<thead>
<tr>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>d_b</td>
</tr>
<tr>
<td>t_b</td>
</tr>
<tr>
<td>k_c</td>
</tr>
<tr>
<td>w_c</td>
</tr>
<tr>
<td>w_b</td>
</tr>
<tr>
<td>t_b</td>
</tr>
<tr>
<td>t_c</td>
</tr>
<tr>
<td>t_c</td>
</tr>
<tr>
<td>( \frac{d_b + t_b + t_c}{2} )</td>
</tr>
</tbody>
</table>

**Fig. 16b(ii)**

**Symbol Notation Used**
From figure 16(b):

\[ \beta = \frac{\text{Actual peak stress in column web}}{\text{Theoretical peak stress in column web}} = \frac{6}{6}, \]

where now the stresses shown are the average stresses in the column web at the section considered. Here, in order to simplify calculations, the theoretical stress diagram has been assumed to have the triangular shape shown, and also is assumed symmetrical about the neutral axis shown. For the connections without moment plates this neutral axis would be at the \( \frac{d}{2} \) of the beam, and the column web stress pattern would truly be symmetrical about this line. For connection C, (with moment plates, without boxing plates), this assumed neutral axis is in the same position as that indicated by the actual column web stress patterns given later. For connection D, (with moment plates, with boxing plates), this assumed position of the neutral axis would be approximately 0.2" closer to the centre line of the compression moment plate than that indicated by the stress diagrams given for the column web.

Assuming that the peak stresses for the theoretical stress distribution occur at the positions indicated in figure 16b(i), the values of \( \beta \) can be calculated directly by equating the actual applied connection moment to the value of the resisting moment given in terms of \( \beta \), as below.

Connection A: Without moment plates and without boxing plates:

\[ M = \frac{6_{1w_c}}{6} \left[ d_b^2 + 9k_c(d_b + 2k_c) \right] \]

Connection B: Without moment plates but with boxing plates:

\[ M = \frac{6_{1w_c}}{6} \left[ d_b^2 + 9k_c(d_b + 2k_c) \right] + 6_{1BwB}(d_b - t_b) \]
where for simplicity it has been assumed that there is a uniform stress of
acting across the thickness $t_B$ of the boxing plates.

Connection C: With moment plates and without boxing plates:

$$M = \frac{6 \cdot w_c}{3} \left[ 2Y^2 + 9k_c(Y + k_c) \right]$$

where $Y = \frac{1}{2}(d_b + t_T + t_C)$

Connection D: With moment plates, with boxing plates:

$$M = \frac{6 \cdot w_c}{3} \left[ 2Y^2 + 9k_c(Y + k_c) \right] + 2 \cdot 6 \cdot l_B \cdot w_B(Y - \frac{t_B}{2})$$

where $Y$ has the same value as given for connection C, and where again for
simplicity, it has been assumed that there is a uniform tensile stress of $6$, acting across the thickness $t_B$ of the boxing plates.

In all these forms given, $M$ is the value of the applied moment,
and $6$, the theoretical value of the peak stress in both tension and compression
regions of the column web since the diagrams are assumed to be symmetrical, and
also, since the thickness of the tension and compression boxing plates are the
same.

The symbols used are as defined in figure 16b(ii). From these
equations, $6_x, 6_T$ can be calculated directly and the magnification factor $\beta$
computed as $6$ is calculated from the strains measured during test.

Values of $\alpha, \beta$ and $X$ have been calculated for the stress peaks in
all connections.

For comparison, the moment-rotation curves for each connection of
the same type have been drawn on one sheet, assuming rotations to occur about
the centroid. Since the beams were connected in two ways, i.e.

(i) beams welded directly to column flanges

(ii) beams welded to column flanges using moment plates,

then the position of the centroid would be different for both cases. In the first case above, the centroid was considered to be at the centre of the beam web, but in the second case, as the moment plates had different thicknesses, the position of the centroid was calculated, and the results of this calculation have been shown in figure 23. From the fact that dial gauges were placed on both the tension and compression beam flanges or moment plates, and hence elongation of both these sections measured independently, then values of rotation were calculated for both tension and compression sections of the beam. These "half-beam rotation" values were compared to one another for the same applied moment, as a check, and the final beam rotation taken as the average of these two values. This half-beam rotation check showed almost identical results for the connections without moment plates but indicated a difference for those with moment plates. The average value of the two, however, was still regarded as the final value of the beam rotation in all cases.

Finally, the resulting error for the planimeter check on the stress distributions drawn for reference was calculated in terms of:

1. \[ \frac{\text{Applied Moment} - \text{Resisting Moment}}{\text{Applied Moment}} = - \% \]

2. \[ \sum \text{Horizontal Forces} = - \text{kips} \]

For (1) the "applied moment" is that actually applied by the turnbuckles, and the "resisting moment" is that measured by the planimeter moment check. For (2) the \[\sum\text{Horizontal forces}\] is the algebraic sum of the compressive and tensile forces set up in the beam or column section as a direct result
of the applied moment. Considering tensile forces as positive and compressive forces as negative, according to the law of statics this sum should be zero, as there are no other horizontal forces acting on these sections. These errors can be attributed to many factors, some of which are:

1. assumed value of $E$,
2. allowable error in gauge factor,
3. assumption of a linear stress variation between inner and outer surface stress distributions,
4. reproduction of stress distributions from given point values.

From this planimeter check also, values of resisting moment carried by the beam flanges, moment plates and boxing plates in the various connections, as a percentage of the total applied moment, have been given for reference in the "Calculations" at the end of each section. In this section also, under the sub-heading of "Web Crippling," for connections without boxing plates, the percentage of the horizontal beam flange or moment plate force distributed into a length of column web of $(N + 2k_c)$ has been calculated,

$$\text{Horizontal force in beam flange (or moment plate)} = \frac{\text{Horizontal force in length } (N + 2k_c) \text{ of column web}}{\text{Horizontal force in length } (N+2k_c) \text{ of column web}}$$

where the "horizontal force in the length $(N + 2k_c)$ of column web" is calculated using the planimeter.

---

**Connection A-1.**

$\frac{1}{4}$ filler weld, continuous around the periphery of the beam section.

**Section A-A.**

<table>
<thead>
<tr>
<th></th>
<th>Beam.</th>
<th>Column.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Section</strong></td>
<td>8&quot; WF at 170#</td>
<td>6&quot; WF at 155#</td>
</tr>
<tr>
<td><strong>b</strong></td>
<td>5.25&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td><strong>d</strong></td>
<td>8&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td><strong>t</strong></td>
<td>0.308&quot;</td>
<td>0.269&quot;</td>
</tr>
<tr>
<td><strong>w</strong></td>
<td>0.24&quot;</td>
<td>0.24&quot;</td>
</tr>
<tr>
<td><strong>k</strong></td>
<td>0.625&quot;</td>
<td>0.563&quot;</td>
</tr>
</tbody>
</table>
A. Description of Connection

The beams were welded directly to the column flanges, using 1/4" fillet welds, which were to have been continuous around the periphery of the beam sections. This, however, was not so. Gaps were left in the welding at the fillets between beam web and beam flange, as shown in figure 17.

These weld discontinuities had an effect on the measured stress in the beam web, near the fillets, which is discussed later under "Discussion" of stress curves.
B. Failure Mechanism

The two separate tests carried out on this connection both revealed the same failure mechanism. At failure both specimens showed excessive deformation caused by a buckling of the column web in the compression zone. All welds remained intact with no visible sign of deformation. With the onset of this column web buckling, horizontal rotation of the beams about a vertical axis became very evident. Because resistance to rotation in this direction is small, and as there are no boxing plates in the column section, a large horizontal beam rotation was finally realized in both tests. This horizontal beam rotation was produced by the method of loading.

C. Moment Rotation Characteristics: Graph No. 3

In each of the two tests carried out measurements of rotation on two separate beams were taken, and the results of these have been plotted separately in graphs No. 3(a) and 3(b) for each test. Each individual curve shown, therefore, represents the average rotation at the connection of a single beam. Since two separate tests were carried out, four curves have been plotted, two from each test.

D. Distribution of Stress

a) - Gauge Layout:

In order to obtain a complete stress distribution across all sections of this connection, gauges were placed on the second specimen in the positions shown in figure 18. The first specimen tested had only a limited number of gauges which were insufficient to give a complete set of the required stress distributions.
Moment Rotation Curves
Connection A-1 (Test No1)

Graph No. 3(a)

0 — Test beam.
Δ — Check beam.

Angle change in Radians x 10^-3
Moment Rotation Curves
Connection A-1 (Test No. 2)

Graph No. 3(b)

O — Test beam.
Δ — Check beam.
Fig. 18  Gauge Layout for Connection A-1
The possible failures considered for this connection in relation to the strain gauge positions were:

1. Column web failure in compression zone.
2. Weld failure at beam tension flange (as fillet welds were used).

From the first test on this connection, since the moment-rotation curves were close up until the buckling failure of the column web, it was assumed that the stress distributions across the two beam flanges were almost the same. From this reasoning then, and the possibility of the weld failure, the tension flange of the "test beam" had five gauges symmetrically placed on the outer surface and four at the inner surface, whereas the compression flange had only one gauge at the centre of the outer surface and two gauges on the inner surface close to the beam web. As a check, the "second beam" also had five gauges on the outer surface of the tension flange and one on the outer surface of the compression flange. This was done in case of a weld failure in the second beam.

Earlier, a criterion for column web failure was given having a stress distribution at failure extending over a length of column web of \((d_b + 6k_c)\). In order that a stress distribution could be measured over this length as a check on this value, gauges were placed on the column web as follows: one on either side of the column web opposite the beam flanges, (this is indicated by the diagonal lines on these gauges) and at a spacing of 3 kc. either side of these, on one side of the column web only.

For the beam web, four gauges were placed on one side symmetrically about the centre-line, and two gauges on the other side as indicated by the diagonal lines on two of the gauges shown. The total number of gauges used in this test was:
Beam section = 24
Column section = 12
Total = 36

From this gauge layout, stress distributions corresponding to load numbers 2, 4, 10 and 15, as shown in the table of applied moments, have been plotted.

b) - Discussion:

1. Tension Beam Flange - Graph No. 4

Both the inner and outer surfaces of the tension beam flange show fairly symmetrical curves of stress distribution. Also, both surfaces indicate a very definite central peak, even for small applied moments, indicating the stress relief caused by the bending of the column flanges. To indicate the effect of the horizontal beam rotation, stress curves No. 4 have been drawn. Here, both the inner and outer surfaces of the flange show a one-sided distortion caused by this horizontal rotation of the beam. Initially, the stresses at both surfaces are close, but with increasing applied moment the difference between them increases rapidly at the centre where the stresses are high, and at the edges only a small change is noticeable.

2. Beam Web - Graph No. 5

Initially the curves are fairly linear but finally become very distorted near the fillets. At the compression end of the web the stress values rise, then begin to decrease with increasing moment, and at the tension end, the stresses remain very small. These low stress values are caused by the gaps in the welding shown in figure 17. Since the ends of the web are not welded at these points, theoretically, where there is no weld there will be no flexural stress induced by the applied moments. However, a strain would be indicated
due to local deformations set up by shear, and it is these values which have been recorded by the strain gauges in these positions. In drawing the curves in the beam web then, the stresses in these unwelded regions have been neglected and the curves approximated, assuming a similar shape to those drawn for connection A-2.

3. Column Web - Graph Nos. 6 & 7

These two graphs represent the induced strains and corresponding stresses respectively, in the column web. Here, the build-up of strain in the web opposite the beam flanges was so rapid that a strain diagram, as well as a stress diagram, has been drawn. Comparison of curve No. 5 with the criterion set forward by Beedle shows that plastic stress values exist over a region a little wider than \((d^b + 6k_c)\), and in particular, at \((d^b + 12k_c)\), the stress values are still approximately 50% of yield stress. Outside this region however, they go to zero rapidly as indicated by the planimeter moment check. This comparison shows then that Beedle's assumption is on the safe side.

c) - Failure Moment:

Values of failure moment were recorded for the two tests carried out and the values obtained are tabulated below:

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Failure Load (kips)</th>
<th>Failure Moment (kip-ins.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.1</td>
<td>333</td>
</tr>
<tr>
<td>2</td>
<td>11.4</td>
<td>376</td>
</tr>
</tbody>
</table>

6 Beedle, "Plastic Design of Steel Frames", p.177-178
### Connection A-1

#### Table of Applied Moments

<table>
<thead>
<tr>
<th>Load No.</th>
<th>Applied Moment (kip/ins)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>Q-load 30&lt;sup&gt;k&lt;/sup&gt; on column</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>Plotted curve No. 1</td>
</tr>
<tr>
<td>3</td>
<td>69</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>116</td>
<td>&quot;  &quot;  &quot; 2</td>
</tr>
<tr>
<td>5</td>
<td>167</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>178</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>194</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>206</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>220</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>233</td>
<td>&quot;  &quot;  &quot; 3</td>
</tr>
<tr>
<td>11</td>
<td>246</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>260</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>276</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>282</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>290</td>
<td>&quot;  &quot;  &quot; 4</td>
</tr>
<tr>
<td>16</td>
<td>303</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>317</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>326</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>376&lt;sup&gt;+&lt;/sup&gt;</td>
<td>Maximum moment</td>
</tr>
</tbody>
</table>

<sup>+</sup> Stress distribution for maximum moment, load No.19, has been plotted for column web, as curve No. 5.
Connection A-1

Stress Distribution in Tension Beam Flange.

Graph No. 4.

Yield stress 35 kip/in².

- Outer surface.
- Inner surface.
**Connection A-1.**

**Stress Distribution in Beam Web.**

*Graph No. 5.*

Plotting points outside the "welded length" have been disregarded. The numbers given to these points correspond to the curve numbers. The tension values begin to decrease, as indicated by 3 being higher than 4.
Connection A-1.

Strain Distribution in Column

WEB.

Tension Beam Flange.

Compressive Strain in inches x 10^-6/ inch.

Tensile Strain in inches x 10^-6/ inch.

Comp' Beam Flange.

Graph No. 6.

N.B.: Curve numbers refer to "load numbers" for test on connection A-1.
Failure moment stress distribution has been shaded in order that a comparison with Beedle's criterion may be made visually.
E. Calculations

a) - Check on Statics:

The results of the statics check on both beam and column sections are tabulated below.

### Moment Check on Beam Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Flange Moment</th>
<th>Web Moment</th>
<th>Total Moment</th>
<th>Applied Moment</th>
<th>Error %</th>
<th>% Resisted by Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>20.6</td>
<td>-</td>
<td>6.7</td>
<td>-</td>
<td>49.5</td>
<td>- 85</td>
</tr>
<tr>
<td>2</td>
<td>48.6</td>
<td>-</td>
<td>15.4</td>
<td>-</td>
<td>115.5</td>
<td>- 84</td>
</tr>
<tr>
<td>3</td>
<td>96.9</td>
<td>-</td>
<td>31.4</td>
<td>-</td>
<td>232.7</td>
<td>- 86</td>
</tr>
<tr>
<td>4</td>
<td>116.8</td>
<td>-</td>
<td>48.5</td>
<td>-</td>
<td>290.4</td>
<td>- 83</td>
</tr>
</tbody>
</table>

(Moments in kip-ins.)

### Horizontal Force Check on Beam Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Flange Force</th>
<th>Web Force</th>
<th>Σ Horiz. Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
</tr>
<tr>
<td>1</td>
<td>+ 4.6</td>
<td>-</td>
<td>+ 1.9</td>
</tr>
<tr>
<td>2</td>
<td>+10.2</td>
<td>-</td>
<td>+ 4.0</td>
</tr>
<tr>
<td>3</td>
<td>+25.2</td>
<td>-</td>
<td>+ 7.8</td>
</tr>
<tr>
<td>4</td>
<td>+30.3</td>
<td>-</td>
<td>+12.0</td>
</tr>
</tbody>
</table>

(Forces in kips.)

Since there were not enough strain gauges on the compression beam flange to draw stress distributions for the inner and outer surfaces, the statics check above has been left incomplete. These values given above, however, indicate from both moment and horizontal force checks, that the average area beneath
the compression flange curve is close, if not equal, to that for the tension flange. In order to calculate the percentage of the applied moment resisted by the beam flanges, the web moment was subtracted from the applied moment and this value, as a percentage of the applied moment, was considered to be that resisted by the flanges.

**Moment Check on Column Section**

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Calculated Moment</th>
<th>Applied Moment</th>
<th>% Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>49.0</td>
<td>49.5</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>116.5</td>
<td>115.5</td>
<td>0.9</td>
</tr>
<tr>
<td>3</td>
<td>229.3</td>
<td>232.7</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>285.5</td>
<td>290.4</td>
<td>1.7</td>
</tr>
<tr>
<td>5</td>
<td>377.4</td>
<td>376.2</td>
<td>0</td>
</tr>
</tbody>
</table>

*Moments in kip-ins.*

**Horizontal Force Check on Column Section**

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Web Tension</th>
<th>Force Compression</th>
<th>Σ Horiz. Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+6.4</td>
<td>-6.4</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>+15.4</td>
<td>-15.3</td>
<td>0.1</td>
</tr>
<tr>
<td>3</td>
<td>+29.7</td>
<td>-30.0</td>
<td>0.3</td>
</tr>
<tr>
<td>4</td>
<td>+31.8</td>
<td>-31.4</td>
<td>0.4</td>
</tr>
<tr>
<td>5</td>
<td>+50.7</td>
<td>-50.3</td>
<td>0.4</td>
</tr>
</tbody>
</table>

*Force in kips.*
Here the moments and horizontal forces contributing to statical equilibrium in the column web have been tabulated for the plotted curves. The above values show only small errors for both checks, indicating that the curves drawn approximate the true stress distributions very closely.

b) - Modification of Theoretical Elastic Stresses:

Under conditions of elastic stress in the beam or column sections, values of $\alpha$ and $\beta$, the magnification factors, have been calculated.

"$\alpha$" = Stress Mag'n Factor at Centre-Line of Beam Flanges

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Flange</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.1</td>
<td>2.1</td>
<td>2.1</td>
</tr>
<tr>
<td>Comp'n Flange</td>
<td>0.8</td>
<td>0.9</td>
<td>1.2</td>
<td>1.4</td>
<td>1.4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

"$\beta$" = Stress Mag'n Factor for Column Web Peak Stresses

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>0.9</td>
<td>1.3</td>
<td>1.4</td>
</tr>
<tr>
<td>Comp'n</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
</tbody>
</table>

From the previous statical check on the beam it was deduced that for the same applied moment, the average area beneath the compression flange stress curves was greater than that beneath the tension flange curves. This point, together with the above values of $\alpha$, lead to the conclusion that the peak stresses are more centrally localized in the tension beam flange than in the compression beam flange. Similarly, for the column web the peak stresses in the tension
region are more localized than those in the compression region for the same applied moment. This effect is very noticeable in the given stress curves for the column web.

For the tension beam flange, the width "D" has been calculated for all stress curves from which the values of \( \alpha \) have been taken.

\[
x\text{-values From } "D" = (w_c + Xk_c)
\]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Flange</td>
<td>4.2</td>
<td>4.4</td>
<td>4.6</td>
<td>5.1</td>
<td>5.4</td>
<td>5.8</td>
<td>5.9</td>
<td>6.0</td>
</tr>
<tr>
<td>Comp'n Flange</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Here, since there was only one gauge on the compression flange, values of \( X \) could not be calculated for this flange.

c) - Failure Moment in Column Web:

Considering Beedle's\(^7\) criterion for column web failure, as given earlier, and using the assumed value of yield stress of 35 k/in.\(^2\), the theoretical failure moment is

\[ M_F = 273 \text{ kip-ins.} \]

Failure moment in the tests were:

Test No. 1 = 333 kip-ins.
Test No. 2 = 376 kip-ins.

It would seem then, that Beedle's assumption is on the safe side.

\(^7\) Ibid, p.177-178
This fact is emphasized when it is remembered that the measured failure moments in test were considered to be influenced by rotation of the beam about a vertical axis. For this reason, these test moments are regarded as being lower than the actual value for a beam restrained from this sideways motion, which is more the practical case when a beam frames into a column at both ends of its span.

d) - Web Crippling:

According to the American Institute of Steel Construction specifications, the compressive stress at the web toe of the fillets in the column, resulting from interior loads not supported by bearing stiffness shall be:

\[
\frac{R}{t(N+2k_c)} \geq 24.0 \text{ k/in.}^2
\]

where \( R \) = concentrated interior load in kips,
\( t \) = web thickness in inches,
\( N \) = bearing length in inches,
\( k_c \) = k-distance for column section.

This formula allows the load to be distributed into a length of the column web of \((N + 2k_c)\), where \( N \) in this case is the thickness of the beam flange. Calculations show that while elastic conditions exist in the column web, only approximately 50% of \( R \) is distributed into this length of web. The values obtained by measurement are

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>50%</td>
<td>55%</td>
<td>56%</td>
</tr>
</tbody>
</table>
With higher applied moments plastic stresses are induced, and the above percentage reduces when these strain hardening stresses are neglected.
Connection A-2.

\[ \frac{1}{2} \text{ single butt welds across all beam flanges.} \]

\[ \frac{1}{2} \text{ fillet welds, both sides of beam web.} \]

**SECTION A-A.**

<table>
<thead>
<tr>
<th>Section</th>
<th>Beam</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>5.25''</td>
<td>6''</td>
</tr>
<tr>
<td>d</td>
<td>8''</td>
<td>6''</td>
</tr>
<tr>
<td>t</td>
<td>0.306''</td>
<td>0.269''</td>
</tr>
<tr>
<td>w</td>
<td>0.24''</td>
<td>0.24''</td>
</tr>
<tr>
<td>k</td>
<td>0.625''</td>
<td>0.563''</td>
</tr>
</tbody>
</table>
A. Description of Connection

The beams were welded directly to the column flanges with 1/4" single butt welds across all beam flanges. For shear resistance, two 6" runs of 1/4" fillet weld, one on either side of the beam web were used to connect the beam web to the column flange.

B. Failure Mechanism

One test only was carried out on this connection giving the same type of column web failure as was the case for connection A-1. At failure, the buckled column web allowed excessive vertical beam rotation and once again, due to lack of rigidity against rotation about a vertical axis, horizontal rotation of the beam accompanied this.

C. Moment-Rotation Characteristics: Graph No.8

Values of rotation were taken from the two beams in the single test carried out, and from these results, two curves have been plotted. Each curve shown therefore represents the rotation of a single beam.

D. Distribution of Stress

a) - Gauge Layout:

The positioning of the gauges on this connection has been indicated in figure 19. Since connection A-1 failed by a buckling of the column web, and also, since the welds holding the beam flanges to the column section in A-2 were more substantial than those for A-1, the only failure mode considered possible
Moment Rotation Curves
Connection A-2.

Graph No. B.

O - Test beam.
Δ - Check beam.

Connection Moment in kip-inches

Angle change in Radians x 10^-3
To follow p. 44

**Fig. 19 Gauge Layout for Connection A-2**
for A-2, was that of column web crippling in the compression region. Gauges were placed on the column web section then, in exactly the same pattern as was the case for connection A-1. Two gauges were placed opposite each beam flange, one on either side of the column web, and the rest on one side of the web only at a spacing of 3 k\textsubscript{c} centre to centre, from those opposite the beam flanges.

For the beam section, five gauges were spaced symmetrically across the outer surface and four across the inner surface of the tension flange, as shown in the figure. For the compression flange, one gauge only was placed in a central position on the outer surface and two only on the inner surface as shown. Finally, for the beam web, four gauges were placed on one side, and two more on the second side, as indicated by the diagonal lines drawn on two of the gauges in the layout diagram. The second beam in the connection did not have gauges attached to it, as results produced by these check gauges on the second beam of connection A-1 indicated values very close to those recorded from gauges on the test beam for the same connection. The total number of gauges used for the test was:

Beam section = 18
Column section = 12
Total = 30

From this gauge layout stress distributions corresponding to load numbers 2, 4, 6 and 10, as shown in the table of applied moments, have been plotted.

b) - Discussion:

1. Tension Beam Flange - Graph No. 9

The shape of these curves is very similar to those shown for connection
A-1, but here the distortion is not as great. Horizontal rotation of the beam during this test was reduced considerably as compared to the tests on A-1, and this seems to have reduced this distortion. It should be noticed that the central peak stress is still a predominant feature of these curves.

2. Beam Web - Graph No. 10

Since the total height of the beam web was not welded to the column, the imposed flexural stresses due to the applied moments, would be developed within this welded length only. The shape of the curves for this connection are more symmetrical than those indicated for connection A-1. Here, the areas beneath tension and compression loop are very nearly equal, a fact which points toward equally stressed beam flanges.

3. Column Web - Graph No. 11

As was the case for connection A-1, so here too, peak stress values exist in the column web opposite the beam flanges. These stress peaks realize plastic values very rapidly. Here again, as for A-1, the width of column web highly stressed opposite the tension beam flange, is much less than that opposite the compression beam flange. As a result of this, for the same applied moment, the peak stress values opposite the tension beam flange are higher than those opposite the compression beam flange.

c) - Failure Moment:

The values of failure load, and failure moment recorded for this test are tabulated below.

<table>
<thead>
<tr>
<th>Failure Load (kips.)</th>
<th>Failure Moment (kip-ins.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.7</td>
<td>386.0</td>
</tr>
<tr>
<td>Load No.</td>
<td>Applied Moment</td>
</tr>
<tr>
<td>---------</td>
<td>----------------</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>69</td>
</tr>
<tr>
<td>4</td>
<td>103</td>
</tr>
<tr>
<td>5</td>
<td>139</td>
</tr>
<tr>
<td>6</td>
<td>175</td>
</tr>
<tr>
<td>7</td>
<td>206</td>
</tr>
<tr>
<td>8</td>
<td>244</td>
</tr>
<tr>
<td>9</td>
<td>271</td>
</tr>
<tr>
<td>10</td>
<td>311</td>
</tr>
<tr>
<td>11</td>
<td>342</td>
</tr>
<tr>
<td>12</td>
<td>369</td>
</tr>
<tr>
<td>13</td>
<td>386</td>
</tr>
</tbody>
</table>
CONNECTION A-2.

STRESS DISTRIBUTION IN BEAM TENSION FLANGE.

GRAPH No. 9.

Yield Stress 35 kip/in.²

○ Outer surface.
△ Inner surface.
CONNECTION A-2

STRESS DISTRIBUTION IN BEAM WEB

GRAPH No. 10.

[Diagram showing stress distribution with axes for Tensile Stress in kip/in² and Compressive Stress in kip/in², with yield stress at 35 kip/in².]
**Connection A-2**

**Stress Distribution in Column Web.**

Graph No. II.

- **Yield Stress** 35 kip/in²
- **Compressive Stress in kip/in²**
- **Tensile Stress in kip/in²**

Peak stress values are more localised in tension region than in compressive region.

To follow p. 47
E. Calculations

a) Check on Statics:

The results of the statics check on both beam and column sections are tabulated below.

**Moment Check on Beam Section**

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Flange Moment Tension</th>
<th>Web Moment</th>
<th>Total Moment</th>
<th>Applied Moment</th>
<th>Error %</th>
<th>% Resisted by Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11.1</td>
<td>2.5</td>
<td></td>
<td>24.0</td>
<td>-</td>
<td>90</td>
</tr>
<tr>
<td>2</td>
<td>48.1</td>
<td>9.2</td>
<td></td>
<td>103.0</td>
<td>-</td>
<td>92</td>
</tr>
<tr>
<td>3</td>
<td>78.3</td>
<td>15.7</td>
<td></td>
<td>175.0</td>
<td>-</td>
<td>91</td>
</tr>
<tr>
<td>4</td>
<td>133.8</td>
<td>24.9</td>
<td></td>
<td>311.0</td>
<td>-</td>
<td>92</td>
</tr>
</tbody>
</table>

(Moments in kip-ins.)

**Horizontal Force Check on Beam Section**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+ 2.9</td>
<td>+0.6</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>+12.5</td>
<td>+2.3</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>+20.4</td>
<td>+4.7</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>+34.7</td>
<td>+7.6</td>
<td>-</td>
</tr>
</tbody>
</table>

(Forces in kips.)

As for connection A-1, there were insufficient gauges on the compression beam flange to draw stress distributions for the inner and outer surfaces, hence the statics check above is once again incomplete. Using the above
results, however, both checks indicate that the average area beneath compression flange curves is close, if not equal, to that for the tension flange curves. Once again, in calculating the percentage of the applied moment resisted by the beam flanges, the beam web moment was subtracted from the applied moment, and this value, as a percentage of the applied moment, considered to be that resisted by the beam flanges.

<table>
<thead>
<tr>
<th>Moment Check on Column Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curve No.</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

(Moments in kip-ins.)

<table>
<thead>
<tr>
<th>Horizontal Force Check on Column Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curve No.</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

(Force in kips.)
The errors indicated for this connection, in the above statical check, are slightly higher than those given in the same check for connection A-1. However, they are still small, indicating that the curves drawn approximate closely the true stress distributions.

b) - Modification of Theoretical Elastic Stresses:

For those stress curves which lie wholly within the elastic range of the beam and column sections, values of $\alpha$ and $\beta$, the magnification factors, have been calculated.

"$\alpha$" - Stress Mag'n Factor at Centre-Line of Beam Flanges

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Flange</td>
<td>2.1</td>
<td>2.1</td>
<td>2.5</td>
<td>2.6</td>
<td>2.6</td>
</tr>
<tr>
<td>Comp'n. Flange</td>
<td>1.4</td>
<td>1.7</td>
<td>1.9</td>
<td>1.9</td>
<td>2.1</td>
</tr>
</tbody>
</table>

"$\beta$" - Stress Mag'n Factor for Column Web Peak Stresses

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>0.8</td>
<td>1.0</td>
<td>1.1</td>
<td>1.2</td>
</tr>
<tr>
<td>Comp'n</td>
<td>0.7</td>
<td>0.8</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Once again, as for connection A-1, it can be concluded that since the average area beneath the beam flange stress curves is almost the same, for the same applied moment, then the peak stress values in the beam flanges are more localized in the tension flange than in the compression flange. These peak stress values in the column web are also more localized in the tension region.
than in the compression region as indicated by the values of $\beta$.

For the tension beam flange, the width $D$ has been calculated for all stress curves from which $\alpha$-values have been obtained.

$$X\text{-values From } D = (w_c + X_k_c)$$

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Flange</td>
<td>2.3</td>
<td>3.4</td>
<td>4.1</td>
<td>4.4</td>
<td>4.9</td>
</tr>
<tr>
<td>Comp'n Flange</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Comparison of the above values of $\alpha$ and $D$, with those calculated for connection A-1, indicates an increase in $\alpha$, and a decrease in $X$. A point which may have influenced this difference is the amount of welding on the beam web. In connection A-1, apart from the weld discontinuities mentioned, the total height of the web was operative in resisting the applied moment. Under these conditions approximately 84% of the applied moment was resisted by the beam flanges. In connection A-2 the beam web had only 6" of weld along its length, thus reducing the moment carried by this section (compare values in moment check for A-1 and A-2). This automatically increases the percentage moment resisted by the beam flanges, and hence in A-2 approximately 91% of the applied moment is resisted by the beam flanges. This 7% increase in moment resisted by the beam flanges, it seems, has influenced the values of $\alpha$ and $D$ as indicated previously.

c) - Failure Moment in Column Web:

Once again, comparing the failure moment developed in test to that
given by Beedle\textsuperscript{8} shows:

1. Failure moment from test = 386 kip-ins.
2. Beedle's failure moment = 273 kip-ins.

a comparison which again indicates that Beedle's assumption is on the safe side.

d) - Web Crippling:

From the American Institute of Steel Construction specification on interior loads not supported by bearing stiffeners, the following values are those calculated for connection A-2, as the percentage of the applied load distributed into the length of the column web \((N + 2k_c)\), where again, \(N\) is the thickness of the beam flange.

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>63</td>
<td>62</td>
<td>60</td>
</tr>
</tbody>
</table>

The values indicated here are higher than those indicated for connection A-1.

\textsuperscript{8} Beedle, "Plastic Design of Steel Frames", p.177-178


**Connection B.**

\[ \frac{1}{4} \text{" single butt welds across all beam flanges.} \]

\[ \frac{1}{4} \text{" fillet welds both sides of beam web.} \]

\[ \frac{3}{8} \text{" boxing plates.} \]

**Section A-A.**

<table>
<thead>
<tr>
<th>Section</th>
<th>Beam. 8&quot; WF. at 170#</th>
<th>Column. 6&quot; WF. at 15.5#</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>5.25&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td>d</td>
<td>8&quot;</td>
<td>6&quot;</td>
</tr>
<tr>
<td>t</td>
<td>0.308&quot;</td>
<td>0.269&quot;</td>
</tr>
<tr>
<td>w</td>
<td>0.24&quot;</td>
<td>0.24&quot;</td>
</tr>
<tr>
<td>k</td>
<td>0.625&quot;</td>
<td>0.565&quot;</td>
</tr>
</tbody>
</table>
A. Description of Connection

The beams were welded directly to the column flanges using 1/4" single butt welds across all beam flanges. On each beam web there were two 6" runs of 1/4" fillet weld, one on either side of the web and symmetrically placed with respect to the centre-line of the web. In the column section, opposite the beam flanges, there were 3/8" thick boxing plates welded into position with 1/4" fillet welds. This welding was continuous around the edges of the boxing plates, and in particular, these welds were placed on both sides of each boxing plate.

One defect which had a definite effect on the measured stresses was the positioning of these boxing plates.

Fig. 20 Displacement of Tension Boxing Plates With Respect to Tension Beam Flange

For the "test beam", the centre-lines of the compression beam flange and compression boxing plates were in line, but the centre-lines of the tension boxing plates and tension beam flange were as shown in figure 20. The gap $G$
shown in this figure was 3/16" for one side and 1/4" for the other, (these values have been indicated with respect to the stress curves on the stress distributions for the tension beam flange and tension boxing plates). The effect of this gap G would be to induce somewhat higher stresses at the inner surface of the tension beam flange and the outer surfaces of the tension boxing plates.

B. Failure Mechanism

One test only was carried out on this connection type, with stress and rotation measurements being measured from the two beams on the specimen. The failure was in the beam at the tension flange with one beam failing before the other. However, prior to failure, both beams showed excessive creep under load, indicating that failure conditions were near for both. At the onset of failure the tension flange of the second beam began to tear away from the butt weld holding it to the column flange, i.e. this was not a weld failure but a tearing of the parent metal of the beam flange at the weld. This separation was initiated in the centre of the tension beam flange where the stress peak exists. Although the beam could still withstand applied moment, this point was considered as failure since the most highly stressed section of the butt weld was now inoperative.

Slight horizontal rotation of the beams was noticeable at failure but not to the extent of that in connections A-1 and A-2. Evidence of this rotation was indicated by a buckled form on one side only of the compression flange of the beam.
C. Moment-Rotation Characteristics: Graph No. 12

For both the beams tested, values of beam rotation were measured and curves of applied moment against connection rotation have been plotted. Comparison of these curves with those for A-1 and A-2 indicates a definite increase in rigidity due to the addition of the boxing plates to the column section. Indeed, the failure moment has been increased by a factor of 2.5 approximately.

D. Distribution of Stress

a) Gauge Layout:

Figure 21 indicates the positioning of all strain gauges for stress measurement in connection B.

The two failures considered in relation to gauge placement were:

1. Beam Failure
2. Weld Failure

   (i) at beam tension flanges,
   (ii) at boxing plate / column flange fillet welds.

Since from connections A-1 and A-2 it was concluded that the stress distributions across tension and compression flanges of the same beam were not identical, both flanges of the test beam had five gauges placed on the outer surface and four gauges at the inner surface as shown. Also, the test beam web had four gauges on one side symmetrically placed and two gauges on the other side, as indicated by the diagonal lines on two of the gauges on this section. For the column web, gauges were placed as close as possible to both sides of both boxing plates as none could be placed directly opposite the beam flanges.
Moment Rotation Curves.

Connection B.

Graph No. 12.

- Test beam.
- Check beam.

Angle change in Radians x 10^{-3}
Gauge Layout for Connection B
as in A-1 and A-2. Externally to the boxing plates, a gauge was placed at a distance of $3 k_c$ from the centre line of both beam flanges, the object of these being to compare stress values at these points with those indicated in the same position from connections A-1 and A-2. Inside the boxing plates, gauges were spaced 2" from those directly opposite the boxing plates. Diagonal lines have been drawn on gauges which have a check gauge on the opposite side of the column web. For the boxing plates, as higher stresses were expected on the outer surface of the tension boxing plates, four gauges were placed on the outer surfaces and two on the inner surfaces of both tension and compression sets as shown.

On the second beam, three gauges were placed on the outer surface of both tension and compression flanges as a check on the values indicated by the gauges on the test beam.

The total number of gauges used in this test were:

<table>
<thead>
<tr>
<th>Section Type</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam sections</td>
<td>30</td>
</tr>
<tr>
<td>Column sections</td>
<td>12</td>
</tr>
<tr>
<td>Boxing plates</td>
<td>12</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>54</td>
</tr>
</tbody>
</table>

From this gauge layout, stress distributions corresponding to load numbers 2, 4, 6 and 10, as indicated in the table of applied moments, have been drawn.

b) Discussion:

1. Beam Flanges: Graphs No. 13 and 14

The compression flange shows very good symmetry, while the tension flange, due to the welded position of the boxing plates at this point, indicates a higher stress on one side. This feature was mentioned earlier. Initially,
the stress distributions are fairly linear, agreeing with the elastic theory, but very rapidly stress concentrations begin to appear at the centre and both edges of both flanges. For the same applied moment the measured stress values are higher in the compression flange than in the tension flange, and it is thought that the welded position of the boxing plates has affected the tension flange stresses, producing these lower values. The values indicated by the compression flange are considered here to be more correct.

2. Beam Web: Graph No. 15

Stresses in the beam web are initially linear, as in the case of connections A. The tension side has a larger enclosed area than the compression side, and this is once again attributed to the welded position of the boxing plates opposite the tension flange. As for connections A, the stress distributions in the beam web have been drawn only for the welded length of beam web.

3. Column Web: Graph No. 16

Again, as in connections A, the column web indicates high stress concentrations opposite both beam flanges. Due to the presence of the boxing plates however, these peak stresses are smaller in connection B. There is a rapid decrease in stress outside the region of the boxing plates and, in particular, at a distance of 3 $k_c$ outside the boxing plates, the stress is virtually zero. Without these boxing plates, as indicated in connections A, stresses at this point would have reached yield value.

4. Boxing Plates: Graphs No. 17 & 18

Stress distribution across both boxing plates is similar to that in the beam flanges. Both inner and outer surface curves show distorted forms indicating high edge stresses on opposite edges. This arises from the tendency

* See "Conclusions"
of the beams to rotate about a vertical axis when the moments are applied. This horizontal beam rotation is resisted directly by the boxing plates, and due to this, very distorted stress patterns could be expected for both surfaces of the boxing plates.

c) - Failure Moment:

The values of failure load and failure moment recorded for this test are:

<table>
<thead>
<tr>
<th>Failure Load (kips)</th>
<th>Failure Moment (kip-ins.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>27.8</td>
<td>917.4</td>
</tr>
<tr>
<td>Load No.</td>
<td>Applied Moment</td>
</tr>
<tr>
<td>---------</td>
<td>----------------</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>33</td>
</tr>
<tr>
<td>3</td>
<td>66</td>
</tr>
<tr>
<td>4</td>
<td>99</td>
</tr>
<tr>
<td>5</td>
<td>132</td>
</tr>
<tr>
<td>6</td>
<td>165</td>
</tr>
<tr>
<td>7</td>
<td>198</td>
</tr>
<tr>
<td>8</td>
<td>264</td>
</tr>
<tr>
<td>9</td>
<td>296</td>
</tr>
<tr>
<td>10</td>
<td>330</td>
</tr>
<tr>
<td>11</td>
<td>396</td>
</tr>
<tr>
<td>12</td>
<td>465</td>
</tr>
<tr>
<td>13</td>
<td>548</td>
</tr>
<tr>
<td>14</td>
<td>640</td>
</tr>
<tr>
<td>15</td>
<td>683</td>
</tr>
<tr>
<td>16</td>
<td>726</td>
</tr>
<tr>
<td>17</td>
<td>766</td>
</tr>
<tr>
<td>18</td>
<td>812</td>
</tr>
<tr>
<td>19</td>
<td>917</td>
</tr>
</tbody>
</table>
Connection B.

Stress Distribution in Tension Beam Flange.

Graph No. 13.

Col. flange.

Beam web.

Tension boxing plate.

Col. web.

Tension beam flange.

Gap G = \frac{3}{16}

Yield Stress 35%.

Gap G = \frac{1}{4}

Tensile Stress in kip/in.

○ Outer surface.

△ Inner surface.
CONNECTION B

STRESS DISTRIBUTION IN COMPRESSION BEAM FlANGE.

GRAPH No. 14.

Yield Stress 35 ksi.

O — Outer surface.
A — Inner surface.
Overall stress distribution reduced as compared to A-1 and A-2.
To follow p. 59

**Connection B.**

**Stress Distribution in Column Web.**

**Graph No. 16.**

Tensile Stress in kip/in²

- Stress peaks are below boxing plate & as tension beam flange and tension boxing plate are not in line.

- The peak stresses at the boxing plate positions have been calculated using the planimeter moment check.
**Connection B.**

**Stress Distribution in Tension Boxing Plates.**

(BOXING PLATES OPPOSITE TENSION FLANGE.)

**Graph No. 17.**

- **Yield Stress:** 35 kips/in²
- **Gap G = 5/16**
- **Gap G = 1/2**

- Peak values for both surfaces taken from graph no. 16.
- High tension on inner surface of beam corresponds to that.
- Column web thickness.

O - Outer surface.
△ - Inner surface.
connection B.

Stress Distribution in Compression Boxing Plates.
(Boxing plates opposite comp. beam flange.)

Graph No. 18.

Yield Stress: $35,000$ psi.

Higher stress values on this side are probably due to horizontal beam rotation.

Peak values for both surfaces taken from graph no. 16.

Column web thickness.

○ — Outer surface.
△ — Inner surface.
E. Calculations

a) - Check on Statics:

The results of the statics check on both beam and column sections are tabulated below.

### Moment Check on Beam Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Flange Moment</th>
<th>Web Moment</th>
<th>Total Moment</th>
<th>Applied Moment</th>
<th>Error %</th>
<th>% Resisted by Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
<td>Comp'n</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>14.7</td>
<td>14.9</td>
<td>2.9</td>
<td>32.5</td>
<td>33</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>43.7</td>
<td>45.6</td>
<td>9.3</td>
<td>98.6</td>
<td>99</td>
<td>0.4</td>
</tr>
<tr>
<td>3</td>
<td>71.3</td>
<td>74.9</td>
<td>16.5</td>
<td>162.7</td>
<td>165</td>
<td>1.3</td>
</tr>
<tr>
<td>4</td>
<td>139.4</td>
<td>147.7</td>
<td>38.5</td>
<td>325.6</td>
<td>330.0</td>
<td>1.3</td>
</tr>
</tbody>
</table>

(Moments in kip-ins.)

### Horizontal Force Check on Beam Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Flange Force</th>
<th>Web Force</th>
<th>Σ Horiz. Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
</tr>
<tr>
<td>1</td>
<td>+3.8</td>
<td>-3.8</td>
<td>+0.5</td>
</tr>
<tr>
<td>2</td>
<td>+11.3</td>
<td>-11.8</td>
<td>+2.0</td>
</tr>
<tr>
<td>3</td>
<td>+18.3</td>
<td>-19.2</td>
<td>+3.8</td>
</tr>
<tr>
<td>4</td>
<td>+36.2</td>
<td>-38.3</td>
<td>+10.8</td>
</tr>
</tbody>
</table>

(Forces in kips.)

The above two checks on the beam section indicate that the tension and compression beam flanges have similar average areas beneath the stress curves for inner and outer surfaces. Also, here, as in the case of connection A-2, only 6" of the beam web was welded to the column flange. and, in both cases, the beam
flanges resist approximately 91% of the applied moment.

**Moment Check on Column Section**

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Boxing Plate Moment</th>
<th>Web Moment</th>
<th>Total Moment</th>
<th>Applied Moment</th>
<th>Error</th>
<th>% Resisted by B. Pl.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
<td>Comp'n</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>13.4</td>
<td>9.5</td>
<td>9.8</td>
<td>32.7</td>
<td>33.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>40.8</td>
<td>35.4</td>
<td>25.6</td>
<td>101.8</td>
<td>99.0</td>
<td>3.0</td>
</tr>
<tr>
<td>3</td>
<td>49.6</td>
<td>60.8</td>
<td>56.0</td>
<td>166.4</td>
<td>165.0</td>
<td>0.8</td>
</tr>
<tr>
<td>4</td>
<td>108.0</td>
<td>120.7</td>
<td>109.0</td>
<td>337.7</td>
<td>330.0</td>
<td>2.3</td>
</tr>
</tbody>
</table>

(Moments in kip-ins.)

**Horizontal Force Check on Column Section**

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Boxing Plate Force</th>
<th>Web Force</th>
<th>Σ Horiz. Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
</tr>
<tr>
<td>1</td>
<td>+ 3.6</td>
<td>- 2.5</td>
<td>+ 0.6</td>
</tr>
<tr>
<td>2</td>
<td>+10.9</td>
<td>- 9.5</td>
<td>+ 2.7</td>
</tr>
<tr>
<td>3</td>
<td>+13.2</td>
<td>-16.3</td>
<td>+ 8.4</td>
</tr>
<tr>
<td>4</td>
<td>+28.9</td>
<td>-32.2</td>
<td>+15.0</td>
</tr>
</tbody>
</table>

(Forces in kips.)

From the moment check on the column section it can be seen that, since the boxing plates resist approximately 2/3 of the applied moment, then the stress peaks in the column web would be greatly reduced in magnitude, as is shown on the stress distributions for the column web.
b) - Modification of Theoretical Elastic Stresses:

Using the stress distributions from moments that produce wholly elastic stresses in the beam or column section, the following magnification factors were calculated.

\[ a_1 = \text{Stress Mag\'n Factor at Centre-Line of Beam Flanges} \]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( a_1 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Comp'n</td>
<td>1.1</td>
<td>1.2</td>
<td>1.5</td>
<td>1.5</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
</tbody>
</table>

\[ a_2 = \text{Stress Mag\'n Factor at Edges of Flanges} \]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( a_2 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>0.7</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>Comp'n</td>
<td>1.1</td>
<td>1.2</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.4</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Values of \( a \) here have been tabulated for the stress peaks in both beam flanges at the centre line \( (a_1) \), and also at the edges of the flanges \( (a_2) \). The values indicated above for \( a_2 \) are average values taken from the two edges of the tension or compression flange. These values of \( a_2 \) calculated above show that with the higher applied moments the stress at the edges becomes greater than that at the centre of the compression flange, but it is felt that distortions due to horizontal beam rotation has caused this. Values of \( a_1 \) and \( a_2 \), calculated for the check beam, agree very well with those tabulated above.
These tabulated values of $\beta$ are only approximate since the curves of stress opposite the beam flanges could be drawn in only with the use of the planimeter moment check. Due to the presence of the boxing plates no gauges could be placed opposite the beam flanges in order that an accurate stress measurement be made.

Values for the width $D$ have been calculated for both tension and compression flanges of the test beam and the values of $X$ corresponding to these are tabulated below.

Here, the stress at the outer surface of the tension flange shows a peak, which alone, is greater than the actual design stress, whereas with the compression flange the stress across the total outer surface is greater than the actual design stress. The stress value at the two minimum points on either side of the central peak stress for the compression flange, however, are equal.
in magnitude to the actual design stress.

Once again, these factors indicate a more localized stress in the central region of the tension flange, although the welded position of the boxing plates may have affected the values.
**Connection C.**

To follow p. 64

![Diagram showing connection details](image)

- **Tension moment** $P$
  - $\frac{3}{16}$ single butt weld across tension moment plates.
  - $\frac{1}{8}$ fillet weld connecting plate to beam web.
  - $6'' \times \frac{1}{2}'' \times \frac{5}{16}''$ web connecting plate.
  - $\frac{1}{8}$ fillet welds on both sides of compression moment plates.

- **Compression moment** $P$
  - $\frac{3}{8}''$ fillet weld, both sides of web connecting plate.
  - $5\frac{1}{2}'' \times \frac{1}{4}''$ fillet weld.

**Section A-A.**

- **Section B-B.**
  - $5\frac{1}{2}'' \times \frac{1}{4}''$ fillet weld.

<table>
<thead>
<tr>
<th>Section</th>
<th>Beam</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b$</td>
<td>5.25''</td>
<td>6''</td>
</tr>
<tr>
<td>$d$</td>
<td>8''</td>
<td>6''</td>
</tr>
<tr>
<td>$t$</td>
<td>0.308''</td>
<td>0.269''</td>
</tr>
<tr>
<td>$w$</td>
<td>0.24''</td>
<td>0.24''</td>
</tr>
<tr>
<td>$k$</td>
<td>0.625''</td>
<td>0.563''</td>
</tr>
</tbody>
</table>
A. Description of Connection

The beams were welded to the column flanges through the moment plates of the shape shown in the initial diagram. Clearance of $\frac{3}{8}$" was left between the beam and column sections. The tension moment plates were welded to the column flanges with 5/16" single butt welds, and to the beam tension flanges with 12 1/2" of 1/4" fillet weld. The compression moment plates were connected to the column flanges with 1/4" fillet welds on both sides of the moment plate, and to the beam compression flanges with two 5 1/2" lengths of 1/4" fillet weld, one at each edge of the moment plate. The web connecting plates were welded to the column flanges with 3/16" fillet welds on both sides of this plate, and to the beam web with 1/4" fillet weld around the edge as shown. It should be noted here that there was only one web connecting plate to each beam.

B. Failure Mechanism

One test only was carried out on this connection, and in this single test only one beam had strain gauges on it in order to measure the induced stresses. This was so as this test was not originally planned, and only a limited number of strain gauges were available for the test. Failure moment induced the same type of column web buckling in the compression zone as exhibited by connections A. The value of the failure moment however, was higher than those given for connections A.

C. Moment Rotation Characteristics: Graph No. 19

Rotation values were recorded from beams in the single test carried
out, but due to the different thicknesses of the moment plates, in order to calculate the actual beam rotation, rotation was assumed to take place about the centre of gravity of the cross section. Figure 23 shows the position of this centre of gravity with respect to both moment plates. The beam rotations as calculated from the values indicated on the dial gauges on the tension and compression moment plates, did not agree as well as in the previous connections. This indicates that the centre of rotation was not actually at the centre of gravity calculated. However, for each applied moment, the average value of these two was used to plot the resulting curves.

D. Distribution of Stress

a) Gauge Layout:

Figure 22 represents the gauge layout used for measurement of stress in this test. The only failure considered possible, in the light of previous tests, was that of column web buckling in the compression region. The outer surfaces of both moment plates had five gauges symmetrically placed across them, but since the inner surfaces of both moment plates were concealed by the beam flanges, no gauges were placed on these surfaces. Also, as there were no boxing plates in the column section, it was considered that at the welds connecting both moment plates to column flange, the outer surface of each moment plate would be more highly stressed than the inner surface, a point which was shown to be true by the statical check. In order to measure stresses in the connecting plate, two gauges were placed on the plate, and two more on one side only of the beam web, and in this way it was hoped to be able to approximate the stresses in the connecting plate. For the column web, the same gauge layout was used as for connections A. Two gauges were placed on each side of the column web opposite
MOMENT ROTATION CURVES
CONNECTION C.

GRAPH No. 19.

O — Test beam.
Δ — Check beam.

Connection moment in Kip-inches.

Angle change in Radians x 10^-3.
Fig. 22  Gauge Layout for Connection C
the centre line of each moment plate, and the rest at a spacing of $3k_c$ from these on one side of the web only.

The total number of gauges used in this test was:

- Beam section = 14
- Column section = 12
- Total = 26

From this gauge layout, stress distributions corresponding to load numbers 2, 4, 6 and 8 as indicated in the table of applied moments, have been drawn.

b) - Discussion:

1. Moment Plates: Graphs No. 20 & 21

Both moment plate stress distributions indicate the stress peak at the centre line due to the presence of the column web. These peaks are very prominent even in the curves for small applied moments. The shape of the curves is very symmetrical about the central peak in both moment plates. Due to the different thicknesses of the two moment plates, the peak value on the outer surface of the tension plate is less than that for the outer surface of the compression plate for the same applied moment. Stress values at both edges of each moment plate are considerably smaller. It is felt that the stress distributions at the inner surface of each moment plate would have the same general shape as those at the outer surface, but the stress values would be smaller. This assumption is based on the curves obtained for the stress distributions at the inner and outer surfaces of the beam flanges in connections A-1 and A-2. It is to be remembered here of course, that the beam web will not directly affect the stresses at the inner surfaces of the moment plates.
2. Web Connecting Plate: Graph No. 22

The stress curves plotted for this plate have been approximated from the values of stress obtained at each point where gauges were placed, and the assumption of zero stress at the centre of gravity of the section (see figure 23). In order to plot the average stress at the two points where stress values have been given on the curve, stresses at the two surfaces were required. Stresses at the one surface were measured directly, and for the surface concealed by the beam web, the stress values were assumed to be those indicated by the gauges on the beam web, since the thicknesses of the web connecting plate and beam web were not too different. These stress values indicated then are an average of stresses measured by the two gauges at each section. As would be expected, it was found that the concealed surface was more highly stressed.

3. Column Web: Graph No. 23

Here again, the peak stresses are opposite the centre-line of each moment plate, and in particular, the tension peaks are more localized than the compression peaks. In comparison with connections A-1 and A-2 however, the magnitude of these peaks is reduced. This feature could probably be caused by two facts:

a) width of moment plates greater than width of beam flange,
b) increased moment arm between the moment plates.

(a) above would spread the applied load over a greater length of column web, and (b) would decrease this applied load due to the moment plates which overall, would have the effect of decreasing the magnitude of the peak stresses at both tension and compression regions of the column web.

c) Failure Moment:

The values of failure load and failure moment recorded for this test
Failure Load (kips) | Failure Moment (kip-ins.)
---|---
12.5 | 412.0

Table of Applied Moments

<table>
<thead>
<tr>
<th>Load No.</th>
<th>Applied Moment</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>Q-load 30k on column</td>
</tr>
<tr>
<td>2</td>
<td>33</td>
<td>Plotted curve No. 1</td>
</tr>
<tr>
<td>3</td>
<td>68</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>103</td>
<td>&quot; &quot; &quot; 2</td>
</tr>
<tr>
<td>5</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>173</td>
<td>&quot; &quot; &quot; 3</td>
</tr>
<tr>
<td>7</td>
<td>211</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>244</td>
<td>&quot; &quot; &quot; 4</td>
</tr>
<tr>
<td>9</td>
<td>279</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>314</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>349</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>385</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>412</td>
<td>Maximum moment</td>
</tr>
</tbody>
</table>
CONNECTION C.

STRESS DISTRIBUTION IN TENSION MOMENT PLATE.

(t = \frac{1}{2}"

GRAPH NO. 20.

Yield Stress 35 ksi

Curves for the inner surface have been approximated using the planimeter moment check as a guide.

0 - Outer surface (true).
--- Inner surface (assumed)
Connection C.

Stress Distribution in Compression Moment Plate

\( t = \frac{3}{8} \)

Graph No. 21.

Yield Stress 35 kips/in.²

Curves for inner surface have been approximated using the "statics check" as a guide. For curve no. 4, no curve of stress at inner surface has been given.

\( \bigcirc \) - Outer surface (true).

\( \cdots \) - Inner surface (assumed).
**CONNECTION C.**

**STRESS DISTRIBUTION IN BEAM WEB CONNECTING PLATE.**

\[ t = \frac{5}{16} \]

**GRAPH No 22.**

Tensile Stress in kip/in²

Comp. Stress in kip/in²

Theoretical neutral axis.
Tension peak stress is again more localised than the comp. peak stress.
E. Calculations

a) - Check on Statics:

Although curves of stress distribution have been given for both surfaces of both moment plates, the stress distributions at both inner surfaces have been assumed, and thus no static check is given for the beam section. It should be noted however, that these given curves are a statically possible set.

% Moment Resisted by Moment Plates

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>95</td>
<td>93</td>
<td>92</td>
</tr>
</tbody>
</table>

These values of percentage moment resisted by the moment plates can only be regarded as approximate, since a true statical check on the beam section could not be made. The values indicated however, are close to those given for the beam flanges of connections A-2 and B.

Moment Check on Column Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Web Moment</th>
<th>Applied Moment</th>
<th>Error %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>32.8</td>
<td>33.0</td>
<td>0.6</td>
</tr>
<tr>
<td>2</td>
<td>102.4</td>
<td>103.0</td>
<td>0.6</td>
</tr>
<tr>
<td>3</td>
<td>174.0</td>
<td>173.3</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>253.6</td>
<td>244.0</td>
<td>3.9</td>
</tr>
</tbody>
</table>

(Moment in kip-ins.)
Horizontal Force Check on Column Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Web Force</th>
<th>Σ Horiz. Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
</tr>
<tr>
<td>1</td>
<td>+3.9</td>
<td>-3.8</td>
</tr>
<tr>
<td>2</td>
<td>+12.2</td>
<td>-12.3</td>
</tr>
<tr>
<td>3</td>
<td>+20.5</td>
<td>-20.7</td>
</tr>
<tr>
<td>4</td>
<td>+30.1</td>
<td>-32.2</td>
</tr>
</tbody>
</table>

(Force in kips.)

For the column section, the values indicated in the above statical check show the stress distributions are very close to the true distribution.

b) - Modification of Theoretical Elastic Stress:

![Diagram of column section with calculated position of centre of gravity.]

- Tension Moment Plate: 5\(\frac{1}{2}\) \(\times\) 1\(\frac{1}{2}\)
- Web Connecting Plate: 6\(\frac{3}{8}\) \(\times\) 3\(\frac{5}{8}\)
- Comp'n Moment Plate: 6\(\frac{3}{8}\) \(\times\) 5\(\frac{3}{8}\)

\[
I_{c.g.} = 93.3\text{ in}^4 \\
C_{tension} = 4.18\text{ in.} \\
C_{comp'} = 4.70\text{ in.}
\]

Fig. 23 Calculated Position of Centre of Gravity
Using the values calculated in figure 23, the following magnification factors were calculated for stress distributions wholly within the elastic region for both beam and column sections.

\[ \alpha = \text{Stress Mag'n Factor at Centre-line of Moment Plates} \]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>2.0</td>
<td>1.8</td>
<td>1.9</td>
<td>2.3</td>
<td>2.1</td>
<td>2.4</td>
</tr>
<tr>
<td>Comp'n</td>
<td>2.9</td>
<td>2.9</td>
<td>3.0</td>
<td>2.9</td>
<td>2.9</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Values of \( \alpha \) tabulated here show lower values for the tension moment plate than for the compression moment plate. It should be remembered however, that the compression moment plate is 2/3 the thickness of the tension moment plate.

\[ \beta = \text{Stress Mag'n Factor for Column Web Stress Peaks} \]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>0.9</td>
<td>1.0</td>
<td>1.1</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Comp'n</td>
<td>1.0</td>
<td>1.0</td>
<td>1.1</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The above tabulation shows the values of \( \beta \) for the tension and compression regions of the column web to be equal. Inspection of the stress distributions for the column web however, still tends to show a more localized tension stress peak as compared to the compression stress peak.

Values for the width \( D \) have been calculated for the stress curves of both tension and compression moment plates, and the corresponding \( X \)-values have been tabulated below:
X-values From $D = (w_c + Xk_c)$

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>3.7</td>
<td>4.4</td>
<td>4.7</td>
<td>4.4</td>
<td>4.5</td>
<td>4.2</td>
</tr>
<tr>
<td>Comp'n</td>
<td>3.1</td>
<td>4.1</td>
<td>4.4</td>
<td>4.2</td>
<td>4.2</td>
<td>4.2</td>
</tr>
</tbody>
</table>

These values of $X$ tabulated above indicate that the highly stressed sections for both moment plates are approximately the same width, which would be expected in a connection of this type. Since the beam web does not affect the stress distributions in the moment plates, this highly stressed width of the moments plates is affected directly by the action of the column web alone.

c) - Failure Moment in Column Web:

Comparing Beedle's failure criterion for failure in the column web to the actual failure moment yields:

Actual failure moment = 412 kip-ins.

Beedle's criterion = 314 kip-ins.

This again shows that Beedle's criterion is on the safe side.

d) - Web Crippling:

As for connections A, using the formula set forth in the steel handbook, the following values were found to be the percentage of the applied load distributed into the length of column web of $(N + 2k_c)$

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>4</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>%</td>
<td>46</td>
<td>49</td>
<td>48</td>
</tr>
</tbody>
</table>

These values are slightly lower than those calculated for either A-1 or A-2.
**Connection D.**

Tension and compression moment plates have same dimensions as shown for Connection C.

### Dimensions for Connection D.

<table>
<thead>
<tr>
<th>Section</th>
<th>BeamWF at 170#</th>
<th>ColumnWF at 15.5#</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
<td>5.25&quot;</td>
<td></td>
</tr>
<tr>
<td>d</td>
<td>8&quot;</td>
<td></td>
</tr>
<tr>
<td>t</td>
<td>0.308&quot;</td>
<td>0.269&quot;</td>
</tr>
<tr>
<td>w</td>
<td>0.24&quot;</td>
<td>0.24&quot;</td>
</tr>
<tr>
<td>k</td>
<td>0.625&quot;</td>
<td>0.503&quot;</td>
</tr>
</tbody>
</table>

**Diagram Notes:**
- Tension moment P
- Compression moment P
- 5/16" fillet welds both sides of plate.
- 1/4" fillet welds on both sides of compression moment plate.
- 5 x 1/4" fillet weld.
- 2 1/2 x 1/4" fillet weld.
- 5 x 1/4" fillet weld.
A. Description of Connection

The beams in this connection were welded to moment plates which in turn were welded to the column flanges as was the case for connection C. The tension moment plates were welded to the column flanges with 5/16" single butt welds, and the compression moment plates were welded to the column flanges with two 1/4" fillet welds each, one on either side of the moment plate. The tension moment plate was welded to the tension beam flange with a total of 12 1/2" of 1/4" fillet weld as shown, and the compression moment plate was welded to the compression beam flange with two 5 1/2" lengths of 1/4" fillet weld as shown. The web connecting plate was welded to the column flange with 3/16" fillet welds on both sides, and with 1/4" fillet weld connecting it to the beam web as shown. The 3/8" thick boxing plates were welded into position with 1/4" fillet welds on both sides of all boxing plates. One defect which had an effect on the measured stress was the welded position of these boxing plates.

Figure 24 represents a side view of part of connection D, showing the welded position of both tension and compression boxing plates with respect to the moment plates. The gaps $G_1$ and $G_2$ shown, had the following measured values for the two sides of the connection:

$$G_1 = \frac{1}{2}" \quad \text{and} \quad \frac{7}{16}"$$

$$G_2 = \frac{15}{16}" \quad \text{and} \quad \frac{13}{16}"$$

Considering the effects shown in connection B due to the positioning of the boxing plates, it would be expected here that the outer surfaces of all boxing plates would be highly stressed, and the inner surfaces of the tension
Fig. 24  Gaps Left Between Moment Plates and Boxing Plates on Connection D
moment plate would have one side more highly stressed than the other due to the unequal values of \( G_2 \).

B. Failure Mechanism

In the test carried out on this connection values of rotation and induced stress were recorded from the two beams. At failure moment the fillet welds holding both tension boxing plates to the column flange broke in tension. The failure was very sudden, and due to the high value of the moment applied at this point, the tensile stress induced in the column web was so great as to cause the column web to be torn from the column flange. This tearing effect allowed a large value of beam rotation and in particular, considerable column flange distortion. Prior to this failure it should be noted that higher values of applied moment were reached, but a high creep rate, indicating large plastic deformations, allowed only the given failure moment value to be resisted. This creep is well indicated by the long horizontal section of the moment-rotation curves for this connection.

A second form of failure was being induced in the latter stages of this test, and this was a buckling of the compression moment plate and beam flange.

Figure 25 shows two views of the compression moment plate and beam flange with approaching failure moment. This buckling of the compression beam flange reduced the resistance of the beam to horizontal rotation, with the result that the beam rotated about the vertical axis \( X-X \) shown in figure 25. This vertical axis \( X-X \), it was found, cut the tension flange at a point near the edge of the tension moment plate. The effect of this horizontal beam rotation was to reduce the above mentioned buckled form on one side of the compression
Fig. 25  Mode of Secondary Buckling Failure  
Connection D
beam flange and increase it on the other until finally at failure, this buckled form was noticeable on one side only of the compression beam flange.

C. **Moment Rotation Characteristics: Graph No. 24**

Values of rotation were measured for both beams in the test carried out and the resulting curves plotted for each beam. As for connection C, so here too, the moment plates differed in thickness, so in order to calculate the vertical beam rotation values, rotation was assumed to take place about the centre of gravity of the section (see figure 23). Values of rotation calculated about this point, using the indicated readings from the dial gauges on both tension and compression moment plates, showed a much closer agreement than those for connection C. The final plotted values of beam rotation were taken as the average of these two calculated values.

D. **Stress Distribution**

a) - **Gauge Layout:**

Figure 26 represents the gauge layout used on this connection. The two failures considered possible in this connection with respect to strain gauge layout were:

1. buckling failure in the beam section,
2. weld failure in tension region of beam and column.

The outer surface of both moment plates of the test beam then, had five gauges symmetrically placed on them and the outer surfaces of the check beam had three gauges each spaced as shown. The inner surfaces of the moment plates however, were concealed by the beam flanges. In order then to obtain stress distributions across the inner surface of the tension moment plate, a
MOMENT ROTATION CURVES

CONNECTION D

GRAPH No. 24.

Connection moment in kip-inches

Angle change in Radians x 10^-3

○ — Test beam.
△ — Check beam.
To follow p. 76

Fig. 26  Gauge Layout for Connection D
A section of the tension beam flange 1" wide was removed and five gauges were placed on this inner surface. Due to the position of the welds holding the compression moment plate to the compression beam flange, the removal of a section of the compression beam flange could not be done. For this reason there were no gauges on the inner surface of the compression moment plate. For the web connecting plate, two gauges were placed on the connecting plate and two more on one side of the beam web as shown. These gauge positions are exactly the same as those used for connection C in this plate. The final readings obtained from the two gauges in the tension or compression region were considered as the surface stresses in the web connecting plate, the gauges on the beam web being considered to indicate a stress not too different from that on the concealed side of the web connecting plate. The final readings then were taken as the average value of the two gauges at each point. Due to the relative positions of the moment plates and boxing plates indicated earlier, two gauges were placed on inner and outer surface of all four boxing plates as indicated. This gauge placement, it is hoped, would clearly indicate the effect of the gaps between the boxing plates and moment plates. Finally, for the column web, two gauges were placed as close as possible to the inner and outer surfaces of the boxing plates on one side of the column web. In order to measure the length of column web stressed, a single gauge was placed at a distance of $3k_c$ from the centre-line of the tension and compression moment plates on one side only of the column web, and outside the region of the boxing plates. Inside the region of the boxing plates gauges were placed at a distance of 2" from those directly below the boxing plates. The four gauges inside the region of the boxing plates have diagonal lines on them indicating that there is a second gauge on the opposite side of the column web at this point.
The total number of gauges used in this test was:

- Beam section = 25
- Column section = 12
- Boxing plates = 16
- Total = 53

From this gauge layout stress distributions corresponding to load numbers 2, 4, 6 and 8, as indicated in the table of applied moments, have been drawn.

b) - Discussion:

1. Moment Plates: Graphs No. 25 & 26

Consider the tension moment plate. The curves for the outer surface are symmetrical about the centre of the plate showing a peak at the centre and much lower values at the edges. The curves for the inner surface generally exhibit larger values at the edges than at the centre, and in particular, these edge values are higher than those for the outer surface. It is considered here that the welded position of the tension boxing plates has affected these edge values and it seems very likely that had the boxing plates been welded in a position opposite the tension moment plate then these edge stresses at the outer surface would have been much higher.\(^\text{X}\)

The compression moment plate exhibits this same type of curve on the outer surface but here the edge stresses are very small indeed. In this case it is felt that the stresses have definitely been affected by the position of the compression boxing plates, since it is in these regions that the direct compressive thrust due to the applied moment is transferred from the beam com-

\(^{\text{X}}\) See "Conclusions"
pression flange to the compression moment plate, and hence very high stresses would be expected here. It is with this in mind, and the fact that the centre of the plate will have a stress peak due to the action of the column web, that the stress curves shown for the inner surface have been approximated. A further assumption of equal stress at the edges and centre of the moment plate was used. It should be noted that these inner surface curves, although statically possible, are only approximate, and have been drawn only to give some idea of the area required beneath these curves for statical equilibrium.

2. Web Connecting Plate: Graph No. 27

These curves are for the gauges shown on the beam web and connecting plate, and represent at the points plotted, the average value of the two gauges at each section. As can be seen the area beneath the curves is very small and hence the value of the resisting moment is also small. Thus the assumption made earlier of averaging the stress values from the two gauges at each section would not involve any great error. Assuming zero stress at the centre of gravity, initially the distribution is linear, but later loses this property and shows a curve typical of all the preceding tests in this region.

3. Column Web: Graph No. 28

These curves show two features. First, the tension stress peak seems more localized than the compression stress peak for the same applied moment and second, comparison of these curves with those for connection C, for the same applied moments, indicates a reduction in area beneath the stress curves. The centre lines of both the boxing plates and moment plates have been drawn on these curves, and in particular, stress values have been read from these curves and plotted on the curves shown for the boxing plates as indicated.
4. Boxing Plates: Graphs No. 29 & 30

A major feature of these graphs is the difference in area beneath the stress curves on the inner and outer surfaces for the same applied moment. The outer surfaces are more highly stressed in both cases. The central regions of the inner surfaces, close to the column web, of both tension and compression plates indicate a stress peak at the column web, which would be expected after seeing the stress distributions for the moment plates, and in particular, that for the column web. Indeed, plotted values were taken from the column web as indicated earlier. This large difference in stresses on the inner and outer surfaces is one again assumed to be due to the welded position of the boxing plates.

Table of Applied Moments

<table>
<thead>
<tr>
<th>Load No.</th>
<th>Applied Moment</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>Q-load 40k on column.</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>Plotted curve No. 1</td>
</tr>
<tr>
<td>2</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>122</td>
<td>&quot; &quot; &quot; 2</td>
</tr>
<tr>
<td>4</td>
<td>201</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>274</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>336</td>
<td>&quot; &quot; &quot; 3</td>
</tr>
<tr>
<td>7</td>
<td>434</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>532</td>
<td>&quot; &quot; &quot; 4</td>
</tr>
<tr>
<td>9</td>
<td>615</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>740</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>812</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>917</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>1011</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>1089</td>
<td>Maximum moment</td>
</tr>
</tbody>
</table>
STRESS DISTRIBUTION IN TENSION MOMENT PLATE

(T = \frac{3}{16}"")

Graph No. 25.

Tension boxing plate

Column flange

Gap \( G_2 = \frac{15}{16} \) in

Yield Stress 35 ksi

O - Outer surface

\( \Delta \) - Inner surface

To follow p. 80
STRESS DISTRIBUTION IN COMPRESSION MOMENT PLATE

Column flange.

`\frac{t}{B} = \frac{3}{8}`

Graph No. 26.

Yield Stress 35 ksi.

Curves of stress distribution at inner surface have been approx. using the moment check as a guide.

0 - Outer surface (true).

---

- 0: Outer surface (true).
- - Inner surface (assumed).
STRESS DISTRIBUTION IN WEB CONNECTING PLATE.  
\( t = \frac{5}{16} \)
**Connection D.**

**Stress Distribution in Column Web.**

**Graph No. 28.**

- **Comp. moment plate.**
- **Comp. boxing plates.**
- **Compr. Stress in kip/in.²**
- **Tension boxing plates.**
- **Tension moment plate.**

- **Yield Stress 35 ksi.**
- **Tensile Stress in kip/in.²**

To follow p. 80
SPRING DISTRIBUTION IN TENSION BOXING PLATES

(t = 3/8" opposite tension moment plate.)

GRAPH No. 29.

Yield Stress 35 kN/m²

Stress values at centre for both surfaces taken from graph no. 28.

Gap G₂ = 1/16.

Gap G₂ ± 1/32.

Tensile Stress in kN/m²

○ Outer surface.
△ Inner surface.

Column web thickness.
Connection D.

Stress Distribution in Compression Boxing Plates.

\( t = \frac{3}{8} \) - opposite comp. moment plate.

Graph No. 30.

Yield Stress 35 ksi

Stress values at centre for both surfaces taken from graph no. 28.

Column web thickness.

- \( \bigcirc \) - Outer surface.
- \( \bigtriangleup \) - Inner surface.
E. Calculations

a) - Check on Statics:

The results of the statics check on both beam and column sections are tabulated below:

### Moment Check on Beam Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Moment Plate Tension</th>
<th>Moment Plate Comp'n</th>
<th>Web Plate</th>
<th>Total Moment</th>
<th>Applied Moment</th>
<th>Error %</th>
<th>% Resisted by K-Plates</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31.7</td>
<td>-</td>
<td>5.6</td>
<td>-</td>
<td>77.9</td>
<td>-</td>
<td>93</td>
</tr>
<tr>
<td>2</td>
<td>71.1</td>
<td>-</td>
<td>15.5</td>
<td>-</td>
<td>201.4</td>
<td>-</td>
<td>92</td>
</tr>
<tr>
<td>3</td>
<td>126.7</td>
<td>-</td>
<td>24.5</td>
<td>-</td>
<td>336.7</td>
<td>-</td>
<td>93</td>
</tr>
<tr>
<td>4</td>
<td>234.0</td>
<td>-</td>
<td>39.0</td>
<td>-</td>
<td>531.5</td>
<td>-</td>
<td>93</td>
</tr>
</tbody>
</table>

(Moment in kip-ins.)

### Horizontal Force Check on Beam Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Moment Plate Tension</th>
<th>Moment Plate Comp'n</th>
<th>Web Conn. Plate Tension</th>
<th>Web Conn. Plate Comp'n</th>
<th>Σ Horiz. Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+7.9</td>
<td>-</td>
<td>+0.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>+18.1</td>
<td>-</td>
<td>+2.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>+32.2</td>
<td>-</td>
<td>+4.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>+59.5</td>
<td>-</td>
<td>+6.8</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

(Force in kips.)

Here again, since a full set of curves for the compression moment plate could not be obtained, the above statical check on the beam section is incomplete. The moment resisted by the web connecting plate was used to compute the percentage moment resisted by the moment plates.
Moment Check on Column Section

<table>
<thead>
<tr>
<th>Curve No.</th>
<th>Boxing Plate Moment</th>
<th>Web Moment</th>
<th>Total Moment</th>
<th>Applied Moment</th>
<th>Error %</th>
<th>% Resisted by B. Pl.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
<td>Comp'n</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>20.5</td>
<td>25.7</td>
<td>32.4</td>
<td></td>
<td>78.6</td>
<td>77.9</td>
</tr>
<tr>
<td>2</td>
<td>49.5</td>
<td>62.6</td>
<td>90.6</td>
<td></td>
<td>202.7</td>
<td>201.4</td>
</tr>
<tr>
<td>3</td>
<td>82.5</td>
<td>108.0</td>
<td>151.9</td>
<td></td>
<td>342.4</td>
<td>336.7</td>
</tr>
<tr>
<td>4</td>
<td>140.3</td>
<td>185.2</td>
<td>202.4</td>
<td></td>
<td>527.9</td>
<td>531.5</td>
</tr>
</tbody>
</table>

(Moment in kip-ins.)

Horizontal Force Check on Column Section

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Comp'n</td>
<td>Tension</td>
</tr>
<tr>
<td>1</td>
<td>+6.2</td>
<td>-5.7</td>
<td>+3.0</td>
</tr>
<tr>
<td>2</td>
<td>+15.0</td>
<td>-13.9</td>
<td>+8.7</td>
</tr>
<tr>
<td>3</td>
<td>+25.0</td>
<td>-24.0</td>
<td>+15.1</td>
</tr>
<tr>
<td>4</td>
<td>+42.5</td>
<td>-41.2</td>
<td>+22.8</td>
</tr>
</tbody>
</table>

(Force in kips)

From the above moment check on the column section it can be seen that only approximately 57% of the applied moment is resisted by the boxing plates. This figure would be higher if the boxing plates had been positioned opposite the moment plates. Even though these boxing plates are positioned incorrectly the above figures indicate quite a reduction in the moment resisted by the column web.
b) - Modification of Theoretical Elastic Stresses:

The following magnification factors were calculated from the stress distributions producing only elastic stresses in the beam or column sections.

\[ \alpha = \text{Stress Mag'n Factor at Centre-line of Moment Plates} \]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>1.1</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.5</td>
</tr>
<tr>
<td>Comp'n</td>
<td>1.3</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

These tabulated values of \( \alpha \) for both the tension and compression moment plates are the values calculated for the test beam. Calculated values for the check beam are exactly the same for both moment plates. Since the stress curves do not rise at the edges of the moment plates, as was the case for the beam flanges of connection B, no magnification factors have been tabulated for these points. It is felt however, that if the boxing plates had been positioned correctly a stress rise at the edges would be noticeable, and in particular, the above values of \( \alpha \) would be smaller.

\[ \beta = \text{Stress Mag'n Factor for Column Web Stress Peaks} \]

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>1.4</td>
<td>1.5</td>
<td>1.6</td>
<td>1.5</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>Comp'n</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.3</td>
<td>1.4</td>
<td>1.3</td>
</tr>
</tbody>
</table>

The values of \( \beta \) tabulated above are only approximate, as no gauges could be placed on the column web at these sections in order that an accurate

* See "Conclusions"
stress reading be obtained, due to the boxing plates. However, the values as calculated are close. Had the boxing plates been welded in directly opposite the moment plates it is felt that the above values would have been reduced.

Values of the width $D$ have been calculated for both the tension and compression moment plates of the test beam, and the $X$-values corresponding to these are tabulated below.

X-values From $D = (w_c + X_k c)$

<table>
<thead>
<tr>
<th>Load No.</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>$X$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>1.2</td>
<td>2.0</td>
<td>2.3</td>
<td>2.4</td>
<td>2.4</td>
<td>2.7</td>
</tr>
<tr>
<td>Comp'n</td>
<td>1.3</td>
<td>2.0</td>
<td>2.1</td>
<td>2.1</td>
<td>2.1</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The above $X$-values indicate widths $D$ which are approximately the same, as was the case for connection C. This would once again be expected, due to the action of the column web on the moment plates.
(i) General

This test series indicates the type of failure very likely to occur in these welded connections. For connections A and C, a column web buckling would be considered, and with connections B and D, failure of a weld in the tension region, as well as a buckling in the compression region, would be considered. The stress distributions given for the various components of each connection indicate in particular the peak stress values which initiate these failures.

A point which must be emphasized here is that the stress distributions given are for "skin" or "surface" stresses. However, in the case of the column web and beam web these skin stresses at any point are considered to have a constant value across the thickness of the section. These peak values of skin stress can initiate local failures of the type indicated in connection B, i.e. a local tearing of the parent metal at the surface most highly stressed. This indicates that although this peak value is only at the surface, and most probably the stress decreases at the inner fibres, these surface stresses can induce failure.
(ii) Comparison of Test Values

1. Connections A-1, A-2 & B:

These three connections had one feature in common, i.e. no moment plates. The following table indicates an average set of calculated values for these connections:

<table>
<thead>
<tr>
<th>Connection</th>
<th>$\alpha$ Tension</th>
<th>$\alpha$ Comp'n</th>
<th>$X$ Tension</th>
<th>$X$ Comp'n</th>
<th>$\beta$ Tension</th>
<th>$\beta$ Comp'n</th>
<th>% M. by Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>2.0</td>
<td>1.4</td>
<td>5.2</td>
<td>-</td>
<td>1.3</td>
<td>0.9</td>
<td>84%</td>
</tr>
<tr>
<td>A-2</td>
<td>2.5</td>
<td>1.8</td>
<td>4.6</td>
<td>-</td>
<td>1.1</td>
<td>1.0</td>
<td>91%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Connection</th>
<th>$\phi$ Tens.</th>
<th>$\phi$ Comp'n</th>
<th>$\chi$ Edge Tens.</th>
<th>$\chi$ Edge Comp.</th>
<th>$\beta$ whole flange</th>
<th>% M. by Box.Pl.</th>
<th>% M. by Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>1.3</td>
<td>1.4</td>
<td>0.9</td>
<td>1.3</td>
<td>2.3 whole flange</td>
<td>1.3</td>
<td>71%</td>
</tr>
</tbody>
</table>

Since for connections A-1 and A-2 the horizontal force check indicates a small difference between the areas enclosed beneath the stress curves on the tension and compression sides of the beam web, then the average area beneath the inner and outer curves of the beam flanges should also be very nearly equal for $E$ horizontal forces = 0. Also, without boxing plates, the peak stress at the centre of the tension flange is higher than that at the centre of the compression flange, as indicated by the $\alpha$-values for A-1 and A-2. From this it could be assumed then that the peak stress is more localized in the tension flange, and hence the value of $X$ for the compression flange would be higher than that for the tension flange. To further support this, investigation of the column web stresses shows this localized effect in the tension region as compared to the compression region in all connections which do not have boxing plates.
As an explanation for the small values of $\alpha$ for connection A-1, it is considered that since the total height of the beam web in A-1 has been welded, the moment resisted by the beam flanges would be smaller than that for A-2 as indicated above. This would effectively reduce the average area beneath the stress curves in the beam flanges, and correspondingly reduce $\alpha$. With the inclusion of boxing plates, connection B, the values of $\alpha$ for both tension and compression flanges are equal. Also, these peak stresses defined by $\alpha$ are much reduced as compared to those indicated for connections A-1 and A-2. This, of course, is due to the more uniform stress distribution across both surfaces of the beam flanges, as shown in the stress curves given earlier. Values of $X$ for the tension flange of B are approximately half those indicated for A-1 and A-2. For the compression flange of B however, the whole outer surface is stressed above the theoretical design stress, the minimum points on either side of this central peak stress being just slightly greater than the theoretical design stress. The percentage of the applied moment resisted by the beam flanges in B is the same as that for connection A-2. $\beta$-values for the three connections are almost identical for the compression regions but vary for the tension regions. For connection A-1 the tension $\beta$-values seem very high as compared to those for A-2.

Stress curves for the outer surface of the tension beam flange, corresponding to a moment producing 20 kip/ins.$^2$ theoretical linear stress at this surface, have been given for comparison in graph No. 31. These curves have been drawn symmetrical, the plotted values having been approximated from test information, and for this reason they are called "idealized curves." This moment value is:

$$M = 282 \text{ kip-ins}.$$
To follow p. 87

MOMENT ROTATION CURVES.

Connection B - 917 k-ins. max.
Connection A-1 - 376 k-ins. max.
Connection A-2 - 386 k-ins. max.

Graph No. 32.

Connection moment in kip-inches.

Angle change in Radians x 10^-3
CONNECTIONS A-1, A-2, & B.

"IDEALISED CURVES" FOR STRESS AT OUTER SURFACE OF TENSION BEAM FLANGE.

Applied Moment = 282 kip-ins

Graph No. 31.

Yield Stress 35 ksi

Theoretical linear stress 20 ksi

Connection A-1.
No boxing plates.
Fillet welds.

Connection A-2.
No boxing plates.
Butt welds.

Connection B.
Boxing plates.
Butt welds.
As a final comparison of these connections, the moment-rotation curves for the "test" beam of these connections have been drawn on one sheet in graph No. 32.

2. Connections C & D

Both these connections had moment plates. The table below indicates an average set of values calculated from the test information:

<table>
<thead>
<tr>
<th>Connection</th>
<th>α</th>
<th>X</th>
<th>β</th>
<th>% M. by M. Plates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tens. Comp'n</td>
<td>Tension Comp'n</td>
<td>Tension Comp'n</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>2.2</td>
<td>2.9</td>
<td>4.3</td>
<td>4.2</td>
</tr>
</tbody>
</table>

Without boxing plates the values of α are again unequal, but now the stress peak is higher in the compression moment plate than in the tension plate, as shown in connection C. However, the X-values indicate similar widths of the moment plates more highly stressed for connection C. These facts could indicate that the thicker moment plate has a tendency to spread the applied load more uniformly across its width than the thinner moment plate. Values of β as compared with those for connections A-1 and A-2 show identical values for the compression region of the column web, but slightly lower values for the tension region. This low value of β in the tension region would be expected as the tension moment plate was very much thicker than the beam tension flange, and hence the applied horizontal load from this moment plate would tend to be spread more along the column web. The moment resisted by the moment plates is almost the same as that for connections A-2 and B.
With boxing plates, connection D, the values of \( \alpha \) are once again found to be equal as was the case for connection B. The X-values indicate almost similar widths of moment plate more highly stressed, and in particular the values are approximately half those shown for connection C. The values of \( \beta \) are very much higher in this connection, but this is attributed to the welded position of the boxing plates, and it is felt that had they been welded in opposite the moment plates these values of \( \beta \) would have been reduced. In calculating these values of \( \beta \) the positioning of the boxing plates was taken into consideration. The percentage of applied moment resisted by the boxing plates is smaller than that for connection B, and the percentage of applied moment resisted by the moment plates is the same as that for connection C.

As for connections A-1, A-2 and B, stress distributions for the outer surfaces of the tension moment plates for connections C and D have been drawn in the form of "idealized" curves, as shown in graph No. 33. The moment-rotation graphs for the "test" beam of the two connections have been drawn on one sheet for comparison in graph No. 34. For these two connections the moment which produces a maximum theoretical linear stress of 20 kip/ins.\(^2\) is:

\[
M = 398 \text{ kip-ins.}
\]

It is felt that the indicated values of \( \alpha, \beta \) and \( X \) given for connection D have definitely been affected by the positioning of the boxing plates, and this effect will now be discussed.

(iii) Effect of Position of Boxing Plates

This discussion will deal with beam flanges only, but it is to be remembered that the same argument will apply for moment plates as well.

The position of the boxing plates in relation to the beam flanges
MOMENT ROTATION CURVES

GRAPH No. 34.

Connection D = 1089 k-ins. max.

Connection C = 412 k-ins. max.

Angle change in Radians $\times 10^{-3}$
Connections C & D.

Idealised curves for stress at outer surface of tension moment plates.

Applied moment = 398 kip-ins.

Graph No. 33.

Yield stress 35 ksi.

Connection C
No boxing plates.

Theoretical linear stress 20 kip/in².

Connection D
Boxing plates.

Tensile stress in kip/in².
of a welded connection can affect the stress distribution in these flanges markedly. In order to develop the required resisting moment in these sections, the stress is distributed across the connecting welds in such a manner that the more rigid regions are more highly stressed, i.e. opposite the column web in both tension and compression beam flanges. Because the column flanges are relatively very flexible without boxing plates, the stress at the edges of the beam flanges would be very small compared to that at the centre, and under applied moment, a bowing of the flanges would result. With boxing plates however, this bowing effect is much reduced since the flange now has a much greater rigidity. This increased rigidity increases the stress induced at the edges of the beam flanges for the same applied moment and in particular, decreases the peak value of stress at the centre opposite the column web. Reference to graph No. 31, connections A-2 and B, indicates this point well. Thus, with the boxing plates welded into a position directly opposite the beam flanges the stress distribution in these flanges does not exhibit such a high peak stress.

Consider now the case where the boxing plates are displaced to one side of the beam flanges, as was the case for connection D. Here, the edges of the column flanges have a rigidity intermediate between that for no boxing and that for boxing plates positioned opposite the beam flanges. The result of this is that a smaller stress is induced in the edges of the beam flanges and consequently, the stress peak at the centre increases in order that the necessary resisting moment be developed. Also, a bowing of the column flange is noticeable at both the beam flange and boxing plate connections, with the result that a double curve in the form of an "S" is developed as shown in figure 27.
This figure shows the tension beam flange and tension boxing plate framing into the column, and as a result of the forces $T_1$ and $T_2$, the column flange has been bent into the form of an S. As a direct result of this, higher surface stresses than normal would be induced at points A on both beam flange and boxing plate. If now, as in connection D, fillet welds were used to connect the boxing plates to the column section, these higher stresses at A would most likely tend to initiate on earlier failure. It is felt that this was the case for connection D, since these high values of surface stress were measured in the positions given above, and failure of the fillet welds represented the failure mechanism.

To conclude then, it seems most likely that correct positioning of the boxing plates leads to the following:
1. More uniform stress distribution in beam flanges.
2. Smaller stress peak at the centre of the beam flanges.
3. More uniform stress gradation between inner and outer surfaces of beam flanges and boxing plates.
4. Possibility of a stronger connection.

(iv) Factors of Safety

Considering a design stress of 20 kip/ins. and a theoretical linear stress distribution, the design moment for each connection would be

(a) with moment plates = 398 kip-ins.
(b) without " " = 282 kip-ins.

Using the failure moments measured in test, the factors of safety for each connection would be:

<table>
<thead>
<tr>
<th>Connection</th>
<th>A-1</th>
<th>A-2</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor of Safety</td>
<td>1.12</td>
<td>1.33</td>
<td>3.25</td>
<td>1.03</td>
<td>2.74*</td>
</tr>
</tbody>
</table>

From these figures it would seem that about 30% only of this theoretical design moment could be used for connections without boxing plates in order to gain a factor of safety greater than 3.0.

(v) Concluding Remarks

From the four connection types tested, it seems that the stress peak in the centre of the beam flange or moment plate will exist regardless of the use of boxing plates. With boxing plates of course, the magnitude of this stress peak is reduced.

* This value considered to be low due to explanation given earlier regarding the boxing plates.
The values calculated from the test information indicate to the designer of welded structures some idea of the magnitude of the stress and its distribution within the connection. However, the actual values given are correct only for this ratio of beam size to column size. In order to give a general rule, further investigation with varying beam and column sizes would be necessary.
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Failure of Connection D - notice section removed from beam tension flange, and tearing of column web. Also notice S-shape of column flange as indicated in Fig. 27.
Buckling failure shown by Connection C.

Relation between welded positions of boxing plates and moment plates in Connection D.
Buckling of compression flange of Connection B.

Failure of tearing of parent metal of tension flange in Connection B.
Close-up of angle measuring equipment.

Typical buckling failure of Connections A-1 and A-2.
Angle measuring equipment in position.

Stiffening beam and rods in position.