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Welded beam-column connections, June 1953

R. F. Pray

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Professor C. D. Jensen  
Department of Civil Engineering and Mechanics  
Lehigh University  
Bethlehem, Pennsylvania

Dear Professor Jensen,

Attached hereto please find my final report on the research work which I conducted under the course title CE 404, Structural Research. I hope that it is complete and adequate.

Needless to say, I wish that I had more time to work with you and Professor Beedle on this subject, Welded Beam-Column Connections. I found the work extremely interesting in all respects, and was particularly happy to have the chance to conduct some actual testing to include with my other work. I regret that I will not be able to stay on at Lehigh to continue this work as I am very curious to see what the final results are going to be.

Anyway, here is "the report" which I hope may prove to be of some value to you in the future.

Very truly yours,

Reuel Ford Fray, III  
Candidate for M. S.  
June, 1953

P.S. Time spent on this project was 248 hours including conferences, etc.
WELDED BEAM - COLUMN CONNECTIONS

By

R. Ford Pray
BSCE, Lehigh, 1950
Graduate Student in Civil Engineering

June 1953

As Part of A
Research Program

F.L. Project No. 233

Conducted By

Professor Lynn S. Beedle
Assistant to the Director of
Fritz Engineering Laboratory

Professor Cyril D. Jensen
Professor of Civil Engineering

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania
THE ELEMENT ON WHICH ALL STRUCTURES DEPEND
# TABLE OF CONTENTS

Front piece
Contents and Important Illustrations  
Introduction

I. Object  

II. Theory and Discussion  

III. Proposed Design Procedure  
1. General  
2. Top Plate Design  
3. Seat Design  
4. General Comments on Design  
5. Design Check

IV. Tension Tests of the Plates  

V. Test of a Built Up Beam-Column Connection

VI. Comments

VII. Appendix

### IMPORTANT ILLUSTRATIONS

<table>
<thead>
<tr>
<th>Fig.</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Details of the Three Proposed Plate Designs</td>
<td>18</td>
</tr>
<tr>
<td>10</td>
<td>Details of the Tension Test Set-up</td>
<td>25</td>
</tr>
<tr>
<td>12</td>
<td>Average Load - Elongation Curves (Tension Test)</td>
<td>28</td>
</tr>
<tr>
<td>13</td>
<td>Average Load - Elongation Curves (Tension Test)</td>
<td>29</td>
</tr>
<tr>
<td>18</td>
<td>Details of the Beam-Column Connection Specimen</td>
<td>36</td>
</tr>
<tr>
<td>23</td>
<td>Moment - Rotation Curves</td>
<td>42</td>
</tr>
<tr>
<td>26</td>
<td>Variation in the Location of the Center of Rotation</td>
<td>46</td>
</tr>
</tbody>
</table>
Due to the limited time available to the author to work on this project for credit hour requirements, the review of literature has been kept to a minimum. This phase has been covered quite thoroughly by both Professors Jensen and Beedle. Note should be made of the articles, "Introduction", "Summary", and "Theory" written by Professor Jensen as part of the general program and included in this report in the Appendix.

As well as reviewing the articles mentioned above, the author has investigated several papers by Brandes & Mains\(^1\) and Johnston & Deits\(^2\) published previously on this subject. References to other material will be made as necessary throughout this report.

Of course the many discussions with Professors Jensen and Beedle on theory, analysis, and application have been very valuable. Also, the author will rely on a general knowledge of welding problems, theoretical and practical, as obtained in the Graduate Course in Structural Welding at Lehigh and from discussions with men from industry actually connected with these problems.

\begin{enumerate}
\end{enumerate}
OBJECT

The general object of the overall program is stated in Mr. Jensen's "Introduction" under the sub-title, 'Proposal'. (See Appendix).

The specific object of this author's work will be several fold:

1. To set up a design procedure for a typical assumed set of conditions which will be applicable in engineering design practices.

2. To analyze said design and predict the action of the actual joint.

3. To run tests on various actual plates fitting the design requirements in 1. In testing these plates alone, the set up will be designed to fit the actual conditions existing in the real joint as closely as is possible.

4. To investigate and test several actual beam-column connections based on above design.

5. To correlate items 3 and 4 looking for a possible prediction of joint action from the results of the plate tests alone thus eliminating the expenses of testing actual built-up beam-column connections in future research.

It should be noted that the foregoing work will be conducted primarily as a guide to further research - pilot tests in effect. This is due to the fact that the author will not be at Lehigh long enough to see the whole research program through.
THEORY AND DISCUSSION

It is most common in building design today to pick beam sizes based on simple supports at the columns. In riveted connections there is a certain slippage which is somewhat unpredictable and which gives the joint little or no rigidity. Hence, simple supports probably is a good assumption for the common riveted connection.

However, in welded joints there is no chance for movements other than due to actual yielding in the steel.

In usual welding practice, a seat angle connection alone, Fig. 1, is assumed to be a simple support since the outstanding leg of the angle tends to bend allowing rotation of the beam end.

\[\text{Seat angle connection before loading beam.} \]

\[\text{When beam is loaded, the outstanding leg of the angle yields producing rotation.} \]

Fig. 1

If the beam is welded directly to the column, Fig. 2, no rotation is possible unless the column itself rotates or deforms in the flange region. In the case where beams frame into the column from opposite sides, the rotational effects will be balanced, or nearly so, for dead loads at least. Research indicates the definite need for stiffeners between the column flanges to reduce their deformation and to transmit the beam flange loads. Thus we have a fully rigid connection permitting us to design the beam for a moment of $\frac{WL}{12}$ instead of the usual $\frac{WL}{8}$ used for a simple beam. It is seen that a considerable
savings in beam weight may be effected. However, fabrication difficulties, such as cutting to more exact lengths, fitting, field welding (overhead and vertical as well as down hand), etc., increase the cost of such a joint somewhat offsetting some, if not the major portion, of the economy of the lighter beam.

The next step might be obvious. Is it not possible to easily design a connection which will give a savings in the beam selection due to some percent of full rigidity in the joint, and, at the same time, be as easy (or nearly so) to fabricate and erect as the simple connection? This has been tried using a top plate and seat connection, Fig. 3, and various procedures for design have been suggested and used.
It will be noted that in this connection, Fig. 3, in contrast to the direct welded connection, Fig. 2, all the field welding would be in a downhand position and that fabrication and erection tolerances would be greater.

The action of the seat in this connection is primarily to carry the vertical reaction. However, some provision must be made to transfer the thrust of the bottom flange of the beam into the column allowing for possible reversal of the moment causing the thrust if wind is involved. This is usually done by fillet welding the beam flange to the seat or butt welding it directly to the column flange using the seat as a back-up plate. The problem of the seat will be discussed more fully later as part of the second phase of this project.

The question might now be raised as to what is happening in the top plate under the expected loading conditions. For discussion it will be simpler to divide the expected loadings into two groups: 1. Vertical loadings, and 2. Wind moment loadings in combination with the vertical loadings.

1. Vertical loadings.

There are two basic design procedures proposed in considering the top plate action due to vertical loadings on the beam.

In the first case, the plate stress is considered to be within the elastic range (usually 20 to 24ksi) under normal expected vertical loadings. Hence, when the beam is loaded, the plate, due to rotational tendency of the beam end, will elongate a small amount elastically. When part of the load is removed, the plate will simply shorten a relative amount. Thus,
we are continually within the elastic range, and the only problem which might be encountered over a period of many years would be fatigue. However, it is hard to believe that fatigue will enter the picture when we consider the relative magnitudes of the dead and live loads and the fact that the number of repetitions of load in the lifetime of the building usually would not approach the fatigue limit.

In the second case, the plate stress is considered to be at, or close to, the proportional limit under normal expected vertical loadings. A common assumption is 30 ksi in the plate. Thus we are still within a range below the so called plastic zone, but only barely so, and a small permanent set in the plate might be induced. Naturally, any additional load above that assumed will cause a permanent set in the plate, which, within limits, has been proven not to be dangerous.

2. Wind moment in combination with vertical loadings.

Now, after the brief discussion of the top plate action under vertical loadings, let us look into the case of wind moments in addition to these vertical loads.

Consider wind in combination with the first case of vertical loading. The plate is within the elastic range for the vertical loads and produces a certain percent end restraint, giving a typical moment diagram as shown in Fig. 4a. (Next page) Consider an additional wind moment at the joint which has a moment diagram as in Fig. 4b. Since the plate is designed to be in the elastic range it will take more moment than $M_v$ (the end moment due to vertical load) before reaching the yield point in the plate at $M_{yp}$. From our present knowledge we
we must assume that once the joint moment reaches the $M_{yp}$ for the plate, no additional moment can be carried by the joint.

Now, if the wind moment, $M_w$, is such that $M_v + M_w < M_{yp}$, the plate will carry the full wind moment without yielding and the resulting moment diagram may be obtained by simply adding the two individual diagrams as shown in Fig. 4c. The center moment will remain the same and will be close enough to the maximum for normal design purposes. The maximum ordinate might shift slightly to the right. The curve itself will flatten to the right of the center and become sharper and steeper to the left. Assuming the plate to be designed for 20 ksi and yield
point at 33ksi, the \( M_w \) must be less than the \( M_v \) (\( M_w = 13/20 \ M_v \)). Therefore, at the right end of the beam, the plate tension will be simply reduced to a lower tension value. Naturally, this will change according to the working stress in the plate.

If now \( M_v + M_w \geq M_{yp} \), the plate will only take a part of the wind moment, \( M_w' \), such that \( M_v + M_w' = M_{yp} \). The remaining wind moment, \( M_w'' \), will be transferred to the opposite end of the beam. For the sake of discussion, consider the worst situation of a wind moment in combination with the case 2 vertical loading (where the plate stress is assumed at or near the yield point under normal vertical loadings). This will be the extreme in which \( M_v \) will equal \( M_{yp} \). See Fig. 5. Since no additional moment can be carried by the plate at the left end, the combined moment diagram must be constructed as shown in Fig. 5c. In effect,
the wind moment diagram itself will assume the shape shown
dashed in Fig. 5b, which, when combined with the vertical load
moment diagram, will simply rotate it as shown. The left or-
dinate remains equal to \( M_v = M_{yp} \). The right ordinate will
become \( 2M_w - M_v \). The point of maximum moment in the span
(low point of the curve) will actually shift some small amount
from the center toward the right and will vary some in magni-
tude. However, for design purposes, the center moment can be
assumed to approximate the maximum and taken equal to \( \left( \frac{W L}{8} - M_v \right) + M_w \) as shown. Dependent upon the magnitude of \( M_w \) with respect
to \( M_v \), a reversal of stress may be set up in the top plate at
the right end.

Theoretically, the wind moment may increase until a second
"plastic hinge" (the lefthand top plate acts as the first one)
is created. This may occur at one of two places. Somewhere in
the beam itself, usually close to the center, when the moment
creates stresses in the plastic range in the flanges, or in
the right hand top plate if, in design, due consideration is
not taken of the possibility of its acting like a strut in
compression.

Mr. LaMotte Grover describes what happens in the top plate
under similar loading conditions. Since his description is very
clear, liberty will be taken in quoting it directly here.³

".... it is assumed that the beams will tend to
act first as though they were rigidly connected; but the
details of design provide for the occurrence of a limited
amount of non-elastic deformation in the connection

---

³. LaMotte Grover - "Manual of Design for Arc Welded Steel
Structures", pp 55-56, Air Reduction Co., New
York, 1949.
material, without real structural damage to it (as determined by research), so that as the loads come onto the structure," wind or vertical, "this permanent deformation prevents the building up of excessive beam-end moments, limiting them to the values assumed in design.

"Upon the removal of part of the load, in the case of a top-plate connection, the unattached mid-portion of the top plate shortens elastically from its length after yielding, to some shorter length determined by the remaining load. Obviously, a complete removal of the load would produce a compression in the top plate, which would be a function of the permanent deformation that had occurred under previous loading.

"If a loading greater than the previous one should then be applied, further plastic deformation would occur, resulting in a greater residual compression stress in the top plate after complete removal of loading. When the maximum combined loading has once been reached, no further permanent deformations will take place. Subsequent variations in wind or gravity loading produce deformation changes entirely within the elastic range.

"As mentioned previously in this manual, such plastic deformation as that described above, actually occurs in practically all structural details that have been used in riveted construction; but the recognition of plastic deformation and exercising control over it by a rational design method is a rather new idea to many structural engineers."

In our case, if the assumed wind moment is reversed, the combined moment diagram in Fig. 56 will simply reverse end for end. Thus the top plate at either end of the beam may be
stressed in the plastic range and may also get a reversal of stress. If the wind is great enough to eliminate the $M_v$ we have, basically, the case that Mr. Grover describes as "complete removal of the load", and the compression due to the increment of plastic deformation will be in effect.

The foregoing discussion has been concerned with the extreme cases, but with these in mind, the analysis of an intermediate case should be fairly simple.

From this discussion we see that many factors may enter the problem of design: 1. effect of vertical loads; 2. effect of additional wind loads in combination - overstressing the beam itself, possible reversal of stresses in the plate, etc.; 3. selection of the plate to fit the needs. Another important point not mentioned previously is the A.I.S.C. specification that the welds can not be over stressed.

Thus we are confronted with several possible procedures to use in designing the plate:

1. Design the top plate at an assigned stress for gravity loading and a certain percent restraint, and then check it for the additional wind moment allowable.

2. Design the top plate at an assigned stress for a combination of gravity and wind loadings.

3. Design the top plate for some combination between the above two procedures.

To design and test specimens for all the mentioned possibilities would involve a long range research program. Therefore, this paper will be concerned with one case, the case of designing the top plate for a stress within the elastic range for normal gravity loading. In the discussion of results, the problem of wind moments in combination with the gravity loads
will be discussed for this particular design. This design procedure is perhaps the most conservative (other than a top plate designed for a fully "fixed" joint for all conditions of loading) of those discussed, and should, therefore, be the starting point for research on all the procedures. In other words, the conservative case will be tried and proved, if possible, through research and then be used as a foundation for further research on the "less conservative" ideas.
PROPOSED DESIGN PROCEDURE

I. GENERAL

Consider the moment diagrams shown in Fig. 6, below, for a beam with uniform load (the usual case in building design) and various degrees of end restraint. \( W \) is the total distributed load and \( L \) the span length.

a. Simple beam
   No end restraint

b. Partially fixed-end beam
   50\% end restraint

c. Partially fixed-end beam
   75\% end restraint

d. Fully fixed-end beam
   100\% end restraint

Fig. 6

As indicated by previous research\(^1\,\,^2\), a logical design procedure would be to assume 50\% end restraint for the beam design and 75\% end restraint for the connection design. As seen in Fig. 6, if the end restraint provided by the connection varies by 25\% in either direction from the assumed 75\%, the beam will not be over-stressed since the point of maximum moment merely shifts from the centerline to the end. It is deemed unlikely that a properly designed semi-rigid welded connection would show a decrease of more than 25\% from the 75\% assumed. Contrarily, previous research indicates that actual restraints in the range of working loads will be, generally, greater than 75\%. 

The reader is referred to Reference 1, pp 147s - 149s, and to Mr. Grover's discussion, pp 53 - 59, on this subject.

Knowing the design moment at the connection, the pull in the top plate and the thrust in the bottom flange may be calculated by dividing the moment by the depth of beam. Plate size, welds, etc. are designed in the usual manner remembering that welds may not be overstressed as set up in the A.I.S.C. Specifications.

With these ideas in mind let us proceed to design several top plates for a typical situation. Then we will analyze these designs, predicting results, and tension test them. From these results we will predict and then test an actual built-up beam-column connection.

II. TOP PLATE DESIGN

1. Assumed typical design situation:

   15' span and 50K total distributed load.
   20 ksi allowable stress for full penetration butt welds
   and normal working stress for steel.
   13.6 ksi allowable shear in steel and on throat of
   fillet welds (1200#/in per 1/8 in weld size.)
   75% end restraint for connection design.
   50% end restraint for beam design.

2. Beam design:

   \[ M = \frac{WL}{3} - \frac{0.5WL}{12} = \frac{WL}{12} = \frac{50 \times 15}{12} = 62.5 \text{ k-ft} \]
   \[ I/c = \frac{M}{S} = \frac{62.5 \times 12}{20} = 37.50 \text{ in}^3 \]
   Most economical section: 14 WF 30 with \( I/c = 41.8 \text{ in}^3 \)
   Check: \( S = 62.5 \times 12/41.8 = 17.95 \text{ ksi} < 20 \text{ ksi} \text{ OK} \)
   Web shear = \( \frac{50}{0.27 \times 13.8} = 6.69 \text{ ksi} < 13.6 \text{ ksi} \text{ OK} \)

   Hence: USE A 14 WF 30 BEAM

3. Plate design stresses:

   \[ M = 0.75WL/12 = \frac{WL}{16} = 50 \times 15 \times 12/16 = 562 \text{ k-in} \]
   Plate design load = \( T = \frac{M}{d} = \frac{562}{14} = 40.2 \text{ k} \)
Note: The actual % end restraint will be greater than 75% approaching a value of around 90%. This is based on previous research. Therefore, assume actual end restraint to be 90% for design of welds, etc.

Actual plate load = \( T_a = 40.2 \times 90/75 = 48.3 \text{ k} \)

4. PLATE DESIGN "A":

We will call this the standard wide end plate and use it as a basis for comparison in this report.

a. Required plate cross-sectional area at 75% end restraint:

\[ 40.2/20 = 2.01 \text{ in}^2 \]

Hence: USE A 5-1/2 x 3/8 Plate (2.06 in²)

b. Actual stress at working load assuming 90% restraint:

\[ 20 \times 90/75 = 24 \text{ ksi} \]

c. Butt welds cannot be over-stressed, hence widen end of plate: 5.5 x 24/20 = 6.6 in; USE 6-3/4 in end width

d. Fillet welds designed to carry actual \( T_a = 48.3 \text{ k} \)

Maximum weld size = plate thickness - 1/16 in (AISC 24d)

3/8" plate, 3/8" beam flange, so use 5/16" weld at an allowable of \( 5/16 \times 1200 = 3000 \text{#/in} \)

Required length: \( 48.3/3 = 16.1" \)

End width of plate: \( 5.5 \)

Length required per side: \( 2 \times \frac{10.6}{5.3} = 5.5" \); USE 5.5 in

Note: The maximum length of fillet weld on each side is usually limited to 0.67 to 1.00 times the plate width. If necessary, one or more slot welds may be used. See Reference 3, p 54 & 79.

e. The unwelded length of the plate, \( L' \), is made about 20% greater than the width to provide sufficient length of plate metal, within which the desired deformations may occur.

\[ 1.20 \times 5.5 = 6.6 \text{ in} \]; USE 7 in
f. A gradual transition from the widened end should be provided to reduce stress concentrations. A curved fillet will be used in this design with the radius approximately equal to the difference between the two widths.

\[ R = 6-3/4 - 5-1/2 = 1-1/4 \] : USE 1-1/2 in

It is anticipated that this should be satisfactory.

g. For final details of Plate Design "A" see Fig. 7.

In normal shop practice this plate would be entirely flame cut by automatic machines, an economical process.

5. PLATE DESIGN "B":

On consideration of design "A" it will be noted that there will be a tendency toward concentration of stresses at the end of the fillet welds. This might reduce the effectiveness of the reduced section of the plate forcing it not to act as desired. We wish to limit the zone of deformation and keep the stresses in the welds as low as possible.

Hence, for Plate Design "B" it was decided to increase the width of the plate at the fillet welded end to 6-3/4" also providing a curved transition. No calculations given in Design "A" need to be changed except for the length of fillet welds along the side of the plate.

a. Fillet welds

Length required: 16.1" (as before)

End width of plate: 6.75

\[ \frac{2}{2} = 9.35 \] : USE 9.35

Length req'd per side: 4.875" : USE 5.0 in.

b. For final details of Plate Design "B" see Fig. 7.

Automatic machine flame cutting would again be employed, and it is anticipated that the relative cost would not increase very much above that for design "A".
6. PLATE DESIGN "C":

It is questioned whether a simpler design could not be devised eliminating flame cutting curves, etc. A reduced section in a plate of standard width could be accomplished by drilling holes. Thus the plates could be cut to length by standard flame cutters or metal saws (shearing a surface which will be welded is not good practice due to the local damage to the steel), stacked in convenient numbers, and placed in a multiple drill. It is the author's opinion that this would be a feasible method of fabrication.

For the sake of comparisons, maintain the 3/8" plate thickness and 6-3/4" end width.

a. Required plate area at 75% end restraint:
   \[ \frac{40.2}{20} = 2.01 \text{ in}^2 \] (as before)

b. So that butt welds will not be over stressed, area required at the assumed actual restraint of 90%:
   \[ 2.01 \times \frac{90}{75} = 2.42 \text{ in}^2 \Rightarrow \text{USE } 6\frac{3}{4}" \times 3/8" \ (2.533 \text{ in}^2) \]
   (Note: a 6\frac{1}{8} x 3/8 - 2.44 \text{ in}^2 or a 5\frac{1}{2} x 7/16 - 2.41 \text{ in}^2 would be better design)

c. Reduce area to 2.01 \text{ in}^2 by drilling one hole:
   \[ 2.53 - 2.01 = 0.52 \times 3/8 = 1.39 \Rightarrow \text{USE } 1-3/8" \text{ hole} \]

d. Use fillet welds as in design "B".

e. To provide sufficient length for inducing deformations in the reduced area try three holes spaced longitudinally at 2 x diameter, minimum. Make the end distance from the center of the end hole to the butt weld or end of fillet weld slightly greater than \( \frac{1}{3} \) the plate width allowing for the common 45° yield line assumption.
   \[ 0.6 \times 6-3/4 = 4.05 \Rightarrow \text{USE } 4.0 \text{ in} \]
   \[ 2 \times d = 2 \times 1-3/8 = 2-3/4 \text{ in between centers of holes}. \]
f. For final details of Plate Design "C" see Fig. 7.

The ideal for this design would be a slot in the center of the plate, however, this would be basically the same as design "B". Also, it is felt that multiple drilled holes possibly would be easier to fabricate than the slot, and, if proper design can be developed, just as effective.

7. General comments on the plate designs above.

A 14 WF 30 beam was picked for this particular situation. The flange width of this beam is 6-3/4" which will not influence the width of the butt welded end of the plate which is limited by the column dimensions only. However, in the case of Designs "B" and "C", the fillet welded end of the plate is 6-3/4" and it is obvious that this will not fit on the above beam section. As mentioned before through, this plate dimension was used to maintain similar dimensions for all designs for comparison of the results of the tension tests, particularly the relative actions of the plates.

For the built up beam-column tests, as in actual practice, the thickness of the plate would be varied as necessary to provide the proper plate width as limited by the column dimensions and the beam flange width. It should be mentioned that in design common standard plate thicknesses and widths would be chosen wherever possible.

III. SEAT DESIGN

Standardization of details and tabulation are a blessing to the designer in saving time, etc. Mr. Grover has presented a complete tabulation of "Standardized Welded Connections" which have a variety of uses. The tables include allowable loads for all common beam sizes for various spans, the deflections resulting, and
PLATE DESIGN "A"

PLATE DESIGN "B"

PLATE DESIGN "C"

Scale: 3" = 1'-0"

Notes: R's to be flame cut or sawn.

£ of £'s to be parallel to the direction of rolling in mill.

FIG. 7
piece marks for standardized connections - seat angle, T seat, and double "clip" angles. The tables are set up for simple beams throughout. It is readily seen that such a table is very valuable.

Eventhough we are not designing a simple beam, the tables can still be used for the connections by simply entering the table with known beam size and load and then picking the seat detail to suit disregarding the span length. This will be used here and our subsequent tests will also prove the worth of these standardized connections.

1. Beam size: 14 WF 30
   Total distributed load: 50k

2. Enter table on page 187, Ref. 3, with above data:
   - Double angle connection R9
   - Seat angle connection 363
   - T seat connection 3A8

3. Some method must be provided to carry the thrust in the bottom flange of the beam (which corresponds to the tension in the top plate) into the column. Hence, a seat type connection must be used. The thrust may be transmitted by fillet welds along the flange edges to an extended outstanding leg of the seat or by filling the gap between the end of the flange and the column with a square butt weld. The former will usually entail a built up T seat with a very long outstanding leg. T's are usually undesirable in building construction because the vertical stiffening leg interferes with the interior wall line, etc. Also, the seat angle will usually be simpler to fabricate in both the shop and the field. This is what we will use here.

4. USE a 363 seat (see Fig. 8 for details) with a square butt weld between the end of the beam flange and the column.
   If this butt weld is carefully placed it should be as strong as the beam as far as carrying the thrust.
Because of availability, it was decided to use an 8 WF column section in the tests. It was also decided to make the connection to the flanges of the column, using stiffeners equal in size to the beam flanges and placed between the column flanges, where it would be more readily accessible for observance during testing and for picture taking.

IV. GENERAL COMMENTS ON DESIGN

The design procedure here outlined is, in this author's opinion, a feasible and economical one. The calculations given in this paper are, of course, expanded for completeness, and, in a design office, could be simplified a good deal.

As can be readily seen, all the welds are economically positioned. The seat angle would have flat or horizontal fillet welds in the shop. The top plate would have flat fillet and butt welds, easily accessible to a welder sitting on the beam, in the field. Likewise, the square butt weld on the bottom flange would be in a flat position and readily accessible. The only work to be done on the beam itself would be the punching of the erection bolt holes unless some other method of erection is devised.
V. DESIGN CHECK

It is possible to check the capacity of this connection by using the theory presented and formulas derived by Professor Jensen and included in the appendix of this paper. Time will not be taken to repeat these derivations here. Formulas used will be referred to the appendix by numbers corresponding to those used by Professor Jensen.

1. Predicted % and restraint assuming rotation about the bottom:

\[
\%R = \frac{100 Ax d^2 x L}{L' I L'' + A x d^2 x L} \quad \text{------(6)}
\]

Where:

- \(A\) = 2.06 in\(^2\)
- \(L\) = 15 x 12 = 180 in
- \(L'\) = 7 in
- \(d\) = 14 in
- \(I\) = 289.6 in\(^4\) for a 14 WF 30

\[
\%R = \frac{100 (2.06)(14)^2 (180)}{2(289.6)(7) + 2.06(14)^2 (180)}
\]

\[
= \frac{100 (72,800)}{4050 + 72,800} = 94.8\% \text{ (pivot @ bottom)}
\]

2. Predicted % and restraint assuming rotation about mid-depth:

\[
\%R = \frac{100 A x d^2 x L}{4 I L' L'' + A x d^2 x L} \quad \text{------(7)}
\]

\[
= \frac{100 (72,800)}{8100 + 72,800} = 90.6\% \text{ (pivot @ mid-depth)}
\]

3. Predicted "Plastic Moment":

Plastic Moment = \(M_p\) = 33 Ad

Assuming 35 ksi = stress at yield point

\[
M_p = 33 (2.06)(14) = 952/12 = 79.4 \text{ K}\cdot\text{ft}
\]

4. Construct a Moment-Rotation Curve using the above data.

For the "Beam Line":

\[
\begin{align*}
\varphi &= 0 \text{ (fully fixed end)} \\
M &= WL/12 = 50x15/12 = 62.5 \text{ K}\cdot\text{ft} \\
\varphi &= 2/3 \times WL/8 \times L/2EI \text{ (moment area)} \\
&= 2/3 \times 50 \times 15/8 \times 15/2 \times 30,000 \times 289.6 \\
&= 7.78 \times 10^{-3} \text{ radians}
\end{align*}
\]

See next page for M-\(\varphi\) Curve constructed from Design assumptions.
One criterion to check is the location of the intersection of the predicted curve with the 50% end restraint line. If this intersection falls to the right of the "Beam Line" for two times the design requirements, it indicates that the beam is more likely to fail before the connection material does. At any rate the above curves indicate that we have designed a fairly conservative semi-rigid connection which will provide 50% end restraint (the assumed design condition for the beam) at more than twice the beam design requirements. The plates were designed to carry a moment of 562 k-in or 46.9 K-ft and their indicated capacity is some 79.4 K-ft.
In other words, the connection has a reserve capacity of 32.5 K-ft. This reserve capacity may be utilized to carry wind moments if desired. Actually, the reserve will not be so great since the connection is actually providing some 90% restraint carrying a corresponding moment of $46.9 \times 90/75 = 56.2$ K-ft. This still provides a reserve of 23.2 K-ft, which may be sufficient to carry wind design moments. In actuality, the yield stress in steel as it is manufactured is more likely to be on the order of 36 to 37 ksi instead of the 33ksi that we assumed in this design.

It should be noted here that there has been some confusion in the past in using the symbols $\Theta$ and $\Phi$ to stand for the connection rotation. $\Phi$ is more commonly associated with discussion of beam properties such as $M/EI$, and it is felt that a distinction should be made between the two uses. Hence, $\Theta$ will be used in this report to stand for end rotation on the beam with respect to the column expressed in radians.

We have now gone through a rather complete design procedure for a welded semi-rigid beam-column connection. The rest of the work included here will be concerned with testing procedures and results for connections of this type. In particular, we will wish to establish an actual $M-\Theta$ Curve for this connection as designed by test and compare it with that predicted.
TENSION TESTS OF THE PLATES

As mentioned earlier in this paper, one of our objects is to investigate the possibility of predicting the actual connection action (constructing a \( M - \theta \) Curve, etc.) by the results of a tension test on the plate alone. If this is possible, the expense of the built up specimen can be practically eliminated in future work.

Consequently, it was deemed desirable to adhere to the following two criterion in setting up the tension test specimen:

1. The tension test specimen (the top plate) must be mounted in such a way as to duplicate the situation in the actual connection as closely as possible;

2. The testing "rig" should be designed in such a way as to permit repeated usage in future tension tests.

With these criterion in mind and a glance back at Fig. 3, Typical Top Plate and Seat Beam-Column Connection, the problems encountered may be visualized. One of the main concerns was to duplicate the eccentric loading on the plate caused by the fillet welding. It was decided to use two duplicate plates in each test assembly which would permit close duplication of the actual conditions. It was felt that if care was taken in fabrication and assembly, the two plates should share equally in carrying the test load and a plot of the average load vs. the average elongation would closely approximate a similar curve for each plate individually.

The final design of the test set-up is shown in Fig. 10, page 25.

Fig. 11, page 26, is a photograph of the test set-up in the 300,000# hydraulic testing machine in Fritz Engineering Laboratory at Lehigh University.
TENSION TEST SETUP
Showing reusable mounting
Units I & II & Specimen Location.

FIG. 10
Standard gage points were placed along the center-line of the plate at 2" intervals starting at a point $\frac{1}{2}$" from the butt welded end of the plate. A 2" Berry gage was used to determine the relative strains at various points along the plate, and a 10" Whittemore gage was used on the first 10" of the plate starting at the butt welded end. When the Whittemore gage went out of range, a direct reading 10" dial gage was substituted. The dial gage shown in Fig. 11 was used to estimate the strain increments when in the plastic zone, determining when readings of the strain gages should be taken.

The welders were instructed to do all welding of the plate to the mounting units just as they would do in the field. The
welder was fully experienced. A 5/32" E 6012 electrode was used for the first pass of the butt weld and the remaining welding was done with E 6020 electrodes. Straight polarity (the electrode positive) was used throughout.

The results obtained from three standard ASTM Designation E3-46 8" gage length coupon tests were typical for steel. The average "lower" yield stress was 37.5 ksi and the proportional limit about 33.8 ksi. The modulus of elasticity was 29,500 ksi, ultimate stress - 56.3 ksi, and the breaking stress - 46.1 ksi. The coupons were cut from a piece of the same steel plate used for the top plates with their center-lines parallel to the direction of rolling as required for the top plates.

It was anticipated that all three specimens would follow the coupon curve rather closely. However, plate design "A" would probably vary from the typical straight line in the elastic zone at a point below the expected proportional limit due to the high concentrations of stress at the ends of the fillet welds which may tend to induce premature yielding. All the tension tests were run at as slow a speed as practical.

The results of the tension tests of Plate Designs "A" and "B" are shown in Figs. 12 and 13, pp 28 and 29. It will be noted from these load-elongation curves that the results of the plate tension tests followed the curve predicted from the coupon test rather closely. However, there are several points of interest which will be described here. Due to insufficient time available to this author it was impossible to test Plate Design "C" as previously proposed.

In the tests of Plate Design "A" the following things should be noted. Throughout the elastic zone one of the plates (we will call it plate #1) showed twice as much strain as the other.
WELDED BEAM-COLUMN CONNECTIONS

AVERAGE LOAD-ELONGATION CURVES
Initial Yield Zone

TENSION TESTS OF PLATES

- Plate Design 2" Av. Area = 2.065 in²
- Plate Design 3" Av. Area = 2.136 in²

Projected Results of Coupon Tests (Average)

FIG. 12
WELDED BEAM-COLUMN
CONNECTIONS

AVERAGE
LOAD-ELONGATION CURVES

TENSION TESTS OF PLATES

**FIG. 13**

Average Load Per Plate - kips

Average Elongation Per Plate - in x 10^-1 (10" Gage Length)
Once plate #1 started yielding it appeared to carry all of the load until the plastic zone was reached. At this point plate #2 yielded quite quickly and went through the initial plastic zone rapidly until it "caught up" with plate #1 which appeared to be more or less idle. From this point on the two plates seemed to act as a unit sharing the load almost equally.

There could be several causes for this phenomenon. The plates could have been of different cross-sectional areas, but they were measured and found to be almost identical. It was finally determined that most of the trouble was due to bad fitting prior to welding. Looking at Fig. 10, p. 25, it is seen that the plates are not parallel to the center line in the edge view. This deviation is due to the backup plates for the butt weld. The specimen was fabricated in a flat position and fitted with this deviation as it should be. However, when the welder placed the fillet weld on the first plate the contraction of the colling weld pulled the mounting unit surface up against the bottom of the plate. Consequently, the other plate ended up with twice the deviation that it should have. The net result is that the center lines of the two mounting units did not line up and made an angle with each other. Thus, as the test was started the mounting units attempted to straighten out putting more load into one plate than the other.

This bad feature was eliminated in the fabrication of Plate Design "B" by the use of wedges to hold the mounting unit and the two test plates in proper relation to each other during the welding.

Even with this phenomenon, the maximum strain increment between the curves for the two plates was only about equal to their average strain; and it is felt that the curve resulting from taking the average of the two plates still is close to that which
would be theoretically obtained for each plate alone.

As predicted, the proportional limit was somewhat lower than that for the coupon tests. The first yield lines appeared in the region of the ends of the fillet welds at the typical 45° angle. They proceeded from this point upward toward the butt welded end of the plate. Prior to failure, considerable necking down was noticed at the end of the fillet welds. All this shows that the stress concentrations in that region were real and followed our predictions.

Initial failure occurred in the butt weld on plate #2 and was wholly unpredicted. It was felt that the welds would prove to be stronger than the plate material. The reason for this failure soon became evident. The butt weld was completely lacking in penetration, the weld metal coming to within 1/8" plus of the back up strip. The author watched the weld being made and felt at the time that it was a good weld, however, it proved otherwise.

The weld was specified as a 45° butt following normal standards, and the welder was told to place the plate with the root gap that he would use in the field. He used a 1/8" root gap. The end of the plate was shaped as shown in Fig. 14 below, having a square end. To try to correct this situation when welding plate "B", the plate was tapered to a point and a 3/16" to 1/4" root gap specified. This seemed to do the trick since there was no sign of failure in the butt weld on plate "B".

![Diagram of plate A and B](image)

**Fig. 14**

Showing the details of the ends of the plates.
After the failure by tearing in the butt weld on plate #2, the test was continued and plate #1 tore on a line across the ends of the fillet welds as initial yield and necking down indicated it would do. Its butt weld did not show signs of failing due to the fact that it had considerable reinforcement. However, it was later found that it lacked in penetration by about the same amount as that on plate #2. Fig. 15, below, is a picture showing the two failures described above. The failure in Plate #1 was typically ductile.

Plate design "B", unlike "A", showed first signs of yielding in the mid-region of the reduced section, and it wasn't until quite late in the test that yield lines began to appear near either of the welds. This indicated that the stress concentrations were very
effectively reduced as was desired in making the design different from that for plate "A". The initial yield lines are shown in Fig. 16, below, and the initial failure is shown in Fig. 17. The two plates followed each other very closely throughout the test and failure commenced in both almost simultaneously. The failures were ductile tears in the plates at the ends of the curved fillets leading into the reduced section of the plate. As can be see in Fig. 17, tears occurred at both the upper and lower ends of the reduced section.

Initial Yield Lines in Plate Design "B"  
**Fig. 16**

Initial Failures in Plate Design "B"  
**Fig. 17**

It will be noticed in Figs. 12 and 13 that the curve for Plate Design "B" indicates a slightly higher yield stress than that for "A". This is due probably to the inherent characteristics of the steel itself although plate design "B" had a slightly greater cross-sectional area.
The results of the 2" Berry gage readings proved to be so erratic that they will not be presented here. The human element plays too important a part in the use of Berry gages in a situation such as this were many different readings are obtained with one gage. Generally speaking, however, the Berry gage readings did confirm the facts indicated by the yield lines as to what parts of the plates were yielding most, etc.

Several important conclusions may be drawn from the tension tests and their set-up:

1. Careful fitting and welding is essential. Use of wedges is recommended in the fitting operation. For the butt weld it is recommended that the plate be tapered to a point and a 3/16" to 1/4" root gap be used.

2. If the above is followed, the average curve obtained from the test will closely approximate the curves for the individual plates.

3. Plate Design "B" appears to eliminate most of the stress concentration problems encountered in Design "A". It might be deemed wise to increase the radius of the curved fillets somewhat.

4. Special care must be used to obtain rational results from the 2" Berry gages. It is suggested that it might be easier and more accurate to use several SR-4 gages instead if the information is deemed necessary or desireable.

All in all the tests proved to be easy to run and easy to interpret. Excluding the initial cost of fabricating the mounting units, the tests should prove to be economical to fabricate and run.

As to the adaptability of the tension tests to predicting the M - θ Curve, we are now concerned. The next part of this program will show the results.
TEST OF A BUILT UP BEAM-COLUMN CONNECTION

Due to the time factor involved, the designs of the tension specimens and the beam-column connection specimen had to be submitted at the same time. It was decided to use Plate Design "A" in the connection specimen since the results of the tension tests, which indicated that Design "B" might be better, were not available as yet. However, our basic point, the prediction of the M - θ curve from the tension test results, should still be able to be proven irregardless of the top plate design used.

A predicted M - θ curve is easy to derive from the results of the tension tests. It will be remembered from the section of this report on design that the tension in the plate is a function of the moment on the connection and the elongation in the plate is a function of the beam end rotation, θ. It can be readily seen that the moment may be obtained by multiplying the tension indicated in the plate by the beam depth, and the rotation obtained by dividing the corresponding elongation by the beam depth. The resulting values may be plotted as a predicted curve on the M - θ diagram as shown in Fig. 23 as part of the results of this test.

Details of the specimen design are shown in Fig. 18, page 36. Wherever possible, shop and field practices were duplicated, both in fabrication and welding. The welding of the top plates followed the procedure used for Plate Design "B" in the tension tests. As would be done in the shop, the seat angles were position welded. The square butt weld at the bottom of the beam was placed in a flat position as it would be in the field. The erection bolts were included in the specimen to see if they would have any effect on the connection even though this possibility was doubted.

The reaction loads in the test were placed at 3' - 1" from the center line of the column which was the approximate inflection points for the beam with 90% end restraint.
FIG. 18

Note: x - Location of rotation bar attachments to fillet between web and flanges. (as close as practical to flanges.)

Top Plate Design "A" (See Work Order #1)

Load carrying Stiffener

8x8 WF Column

14WF30 - 3'3" long

8x8 WF 3/8" Stiffeners, clipped corners, butt welded to flanges and fillet welded to web of column

2 1/4" x 3/4" erection bolts (holes may be flame cut, ream if possible)

7" x 4" x 3/4" Seat is 9 - 9" long

Mill end of Column

Testing Machine Head

12WF10G Base Beam

Testing Machine Bed

Position of Specimen in the Testing Machine

Seat Detail
Detail of the Welds on the Top Plate of the Beam-Column Connection Test Specimen

Fig. 19

Detail of the Seat, Stiffener, and Welding of the Beam-Column Test Specimen

Fig. 20
Figs. 19 and 20 on the previous page show some of the details of the top plate and the seat of the test specimen.

Rotation bars were placed as indicated in Fig. 18, page 36, with the distance between their legs being 12" in all cases. Round bars were used for the legs which were spot welded to the beam (and column) web, the weld being on the side of the leg toward the center line of the beam in all cases. Light angle sections were welded to the legs and were about 24" long, providing ample room for variation of the distance between dial gages. The gages were mounted on bars welded to the rotation bars were necessary to span a considerable distance, and other gages were attached directly to the rotation bars.

A dial gage was mounted on each top plate with a gage length of 10" from the column flange. These gages were used primarily to check the results of the elongation in the plate for comparison with the tension test results. A running plot was kept.

In addition, a Huggenberger gage was mounted on the top and bottom flanges of the beam at a point ten inches from the support. These gages were used to check the actual bending moment in the beam against the assumed bending moment for a particular load on the specimen.

As was done in previous research, the specimen was tested upside down to facilitate the use of the available testing machine. Fig. 18 shows the set up for testing in the 300,000# machine, used for the tension tests, which would have required the base beam as shown. However, the test was actually run in the 800,000# mechanical testing machine, the bed of which was long enough to accommodate the specimen without using a base beam.

Fig. 21 on the next page shows the specimen as it was
in position in the 800,000# machine ready for testing.

The Beam-Column Connection Test Specimen as it appeared in the 800,000# machine at Fritz Engineering Laboratory

Fig. 21

As before, the test was run at as slow a speed as practical. Throughout the elastic range readings were taken at predetermined load increments. In the plastic range, readings were taken at predetermined strain increments as indicated by the elongation dial gages on the top plates. This method proved to be very satisfactory and provided ample points to plot a good curve. Elongation in the top plate vs. the load was plotted as the test proceeded and netted a curve in very close agreement with one interpolated from the results of the tension test data. The results
of the Huggenberger gage readings indicated that the actual moment in the beam was in very close agreement with the moment predicted from the test set up and loading.

One question was raised concerning the interpretation of the results. This question was what should be considered the rotation of the connection. It will be noted from Figs. 18 and 21 that rotation bars were placed to measure the rotation between the column flanges, between the end of the beam and the column, between the two ends of the top plate, and between two points beyond the top plate. Fig. 22, below, will help to clarify this discussion.

Illustration showing the location of several of the Rotation Bars and the relative rotations that they indicated.

Fig. 22

It is logical to assume that the connection rotation is the sum of the rotations indicated by bars #2 and #4 minus any rotation which may occur between the column flanges, bar #0. In our case the rotation indicated by bar #0 was negligible and had no influence on the connection rotation. It will be noted in the
illustration that bar #4 will indicate rotation in the beam itself plus possible rotation due to the connection. The beam rotation can be computed for any loading by \( \theta_b = \phi (1) = M \frac{1}{EI} \), where \( \theta_b \) is the rotation in radians, \( \phi \) is the unit rotation throughout the area enclosed by the bars #4, \( l \) is the distance between the bars (14" in this case), \( M \) is the average moment on the section for which \( \phi \) is determined, and \( EI \) is a property of the beam itself. This calculation is based on a beam assumed fixed at the end and the normal concepts of unit rotations and \( M/EI \). Thus the rotation due to the connection will be the rotation indicated by the rotation bar #4 minus the rotation normally expected in the beam itself if it were fully fixed at the end, \( \theta_b \). These calculations were run for many points throughout the test range and the following interesting fact was noted. In all cases the rotation indicated by bar #4 was equal to that calculated for the beam itself and, therefore, the entire connection rotation was indicated by bar #2, as would normally be assumed before making investigations into the \( M/EI \), etc. In effect this means that the connection was as strong as the beam itself in this respect.

The results of the test are plotted on the \( M - \theta \), Fig. 23 on the next page. It will be noted that the curve is superimposed on the curves predicted by design assumptions and the results of the tension test on Plate Design "A". The curve plotted from the results of the beam-column connection test is the average curve for the two connections which appeared to follow each other rather closely throughout the test.

It is interesting to note the close agreement between the various predicted curves and the results of the connection test. The deviation of the curve predicted from the tension test may be caused in part by the trouble that was encountered during that test.
WELDED BEAM-COLUMN CONNECTIONS

MOMENT - ROTATION CURVES

M - θ

TEST OF BUILT-UP BEAM-COLUMN CONNECTION USING TOP PLATE DESIGN "A"

FIG. 23
Yielding and failure in the top plate was identical with that in plate #1 in the tension test. Initial yield occurred in the region of the ends of the fillet welds and proceeded from that point toward the center of the plate. There was no sign of stress concentration or yielding in the butt weld zone. The final failure was by tearing across the plate on a line between the ends of the fillet welds. It was of a ductile nature.

The first yield lines to appear in the test were in the web of the beam just above the bottom flange starting near the butt weld. However, this remained a single line and progressed very slowly until near the end of the test. It was not felt that this had any influence on the test at all. As was mentioned before, the rotation bars indicated that normally throughout this zone. A short while before failure in the top plate, the bottom flange of the beam began to show signs of severe bending over the end of the outstanding leg of the angle. This did not occur until the test was out of the range of usefulness for the top plate.

Fig. 24 on the next page shows a close-up view of the failure in the top plate and the yielding in the bottom flange and web of the beam. Fig. 25, also on the next page, is a general picture of the beam-column connection as it appeared after the test was completed.

In connection with the M - θ curve, Fig. 23 on the preceding page, it should be noted that the curve continued for a considerable distance beyond the limits of the plot shown. When the rotation bars were removed from the specimen they indicated a rotation of some 24 x 10^-3 radians, or almost double the maximum shown on the M - θ curve, at a moment of 113 K-ft. The ultimate moment was 116 K-ft and the predicted ultimate rotation from the tension test was 43 x 10^-3 radians. This part of the curve was not included here.
Close-up View of the failure in the top plate and the yielding in the region of the seat angle

Fig. 24

General view of the test specimen after the completion of the test

Fig. 25
as that part that is shown was felt to be more important.

Note that the M - θ curve indicates that the connection
was still providing the minimum 50% end restraint beyond the
"Beam Line" for three times the design requirements, and provided
more than 75% end restraint at the "Beam Line" for two times the
design requirements.

It was interesting to note that the center of rotation
did not remain constant throughout the test. Fig. 26 on the next
page is a plot of the % of beam depth from the bottom to the
center of rotation vs. θ, the connection rotation. This curve
indicates that during the initial rotation the center moves upward
from the bottom flange as it begins to deform, sharing more of the
load with the top plate at each increment of loading. However,
once yielding in the top plate starts its elongation far exceeds
the contraction possible in the bottom flange and the center
moves downward again.

Several conclusions may be drawn from the results of this
test:

1. The top plate will perform similarly to the way it
does in the tension test,

2. The connection rotation for this particular design
seems to be indicated entirely by the rotation bars measuring
the rotation between the column and the end of the beam,

3. Stiffeners selected as comparable in size to the beam
flange are sufficient to eliminate deformations in the column,

4. The seat designed as shown proved to be entirely
adequate and may be said to be somewhat too stiff since the beam
flange was caused to bend over it in the later stages of the test.

Of course there are other points of a more minor nature
which could be thought of at this point.
WELDED BEAM-COLUMN CONNECTIONS

CURVE SHOWING VARIATION IN THE LOCATION OF THE CENTER OF ROTATION vs. CONNECTION ROTATION - $\theta$

FIG. 26
COMMENTS

Again the reader is reminded that the work done for this report and the tests described herein are primarily of a pilot nature for future research. Naturally before any definite conclusions are drawn, considerably more tests need to be run and interpolated. In research it is impossible to derive, formulate, and state indicated facts from the results of a few tests such as those that I have had the opportunity to run. Consequently, the following statements are only indications of the facts which may or may not be proven by further research on the subject.

Indications are that the beam-column connection action may be predicted from the results of the tension tests on the top plates. It must be remembered, however, that the results of these tests seem to prove this for only one design. But it is felt that with further tests for comparisons, as done here, a method of predicting the connection action may be devised.

The tests reported here do indicate the adequacy of the design procedure as outlined earlier in the paper. There are several points on which the designer should be cautioned. Care must be taken in specifying the welds to be used, particularly the butt weld. Also, it seems that the plate widened at both ends, Plate Design "B", may prove to be a more desireable design. The author regrets that he did not have sufficient time to test Plate Design "C" as it would be interesting to compare the results.

The design method for the seat seems to be more than adequate. Actually, there is some question as to whether a seat with an outstanding leg which is less stiff would not be more desireable.
To sum up, it appears that we have a design procedure that proves to be more than adequate in all respects. Secondly, it looks as if a tension test, carefully fabricated, of the plate design will be sufficient to predict the connection action in the future. As mentioned before, if this is proven, considerable time, effort, trouble, and cost may be eliminated in future research.

It will be interesting to see how the results of further research compare with those indicated by the few tests described in this paper. It will be particularly interesting to note the results of tests on the more radical designs for the top plates as described in the section on theory.

Beam-Column Connections are the elements on which all structures depend. Can we design then more efficiently, more economically, and more rationally? Through welding?
THEORY

L. Relation between stress in top plate, of a beam-column connection, and the load on the beam.

A. First assumption: Case where rotation takes place about base of beam (point A in sketch), which is the approximate case for light top plates. Let $A \& L'$ be the area and the effective length of the top plate. Let $W \equiv$ total distributed load on the beam of length $L$.

Now the end slope, $\frac{W L}{B}$, for a simple beam is given by $WL^2/24EI$ (see any Mech. of Mat. text).

When top plate is in tension, the elongation of the plate is given by $T L'/AE$, where $T$ is the total tension in the plate (also given by $SL'/E$ where $S$ is unit stress).

Now the actual end slope may be expressed in two ways, first in terms of the beam load and length as:

$$\theta_{act.} = \frac{WL^2}{24EI} - \frac{T d L}{2EI} \quad \text{(by area-moment, see sketch B)}$$

and also in terms of beam depth and deformation of the top plate as:

$$\theta_{act.} = T L'/AE \quad \text{or} \quad SL'/Ed$$

Therefore

$$T L'/AE \quad \text{or} \quad SL'/Ed = \frac{WL^2}{24EI} - \frac{T d L}{2EI}$$

From which

$$T = \frac{WL^2/24I}{A + dL} \quad \text{(1)}$$

Also

$$A = \frac{WL}{12d} - \frac{2d L'}{d + AD^2} \quad \text{(2a)}$$

Units: Stress in psi, load in lbs., lengths and depth in inches, I in inches$^4$. 
B. Second assumption: Case where rotation takes place about mid-height of beam, which is approximately the case for heavy top plates.

As before \( \phi \) (simple beam) = \( \frac{WL^2}{24EI} \) and deformation of top plate is given by \( \frac{TL^0}{AE} \) or \( \frac{5L^0}{E} \).

Also \( \phi_{act} = \frac{WL^2}{24EI} - \frac{TdL}{2EI} \)

But \( \phi_{act} \) in terms of the deformation of the top plate is given by

\[ \phi_{act} = \frac{2TL^0}{AEEd} \text{ or } 2\frac{L^0}{Ed} \]

Therefore \( T = \frac{WL^2}{24I} \frac{2L^0}{Ad} + \frac{4L^0}{2I} \)

\[ \text{and } s = \frac{WL^2}{24I} \frac{2L^0}{d} + \frac{AdL}{2I} = \frac{L^0}{12} \left[ 4IL^0 + \frac{1}{Ad^2} \frac{L^0}{d^2} \right] \]

Also

\[ A = \frac{WL}{12ds} = \frac{4IL^0}{d^2} \tag{4a} \]

II. Determination of Per Cent Restraint

A. Basic Equations Regardless of Center of Rotation of End of Beam.

Now per cent restraint = \( \frac{100 \ Td}{WL/12} \)

\[ = \frac{1200 \ Td}{WL} \]

\[ = \frac{1200 \ Ad}{WL} \tag{5} \]

Fig. 3. Moment Diagrams

B. First Assumption: Rotation about Base (Point A, Fig.1)

From equation (5) \( \%R = \frac{1200 \ Ad \ s}{WL} \)

Substituting value of \( s \) from (2)

\[ \%R = \frac{1200 \ Ad \ WL^2}{12 \ 2L^0 \ + \ Ad^2 \ L} = \frac{100Ad^2L}{2IL + Ad^2L} \]
C. Second Assumption: Rotation about Mid-Height, C.

As for (B) but substituting value of $S$ from equation (4)

$$\% R = \frac{1200 \text{ AdWL}^2}{\text{dWL}^2} \left[ \frac{1}{4IL^2 + \text{Ad}^2 L} \right]$$

Whence

$$\% R = \frac{100 \text{ Ad}^2 L}{4IL^2 + \text{Ad}^2 L}$$

(7)

III. Estimate of End Moment when Plastic Action Begins.

On the assumption that the top plate yields at 33 ksi and responds as a uni-axially stressed member, the plastic action will begin when the end moment is $T_d$, where $T$ is given by $AS_yp$. ($S_yp$ being the stress at yield point)

Therefore

$$\text{Moment} = AS_yp.$$  

(8)

When $S_yp = 33$ ksi,

$$\text{Plastic Moment} = 33 \text{ Ad} \quad \text{(in kip-inches)}$$  

(8a)
APPLICATIONS

I. Design Top Plate for 14"WF-30, 15' span, Total Distributed Load = 50 kips.

A. Additional Design Assumption: S in Top Plate to be Designed for 20 ksi at Working Load. \( L' = 7'' \)

By (2a)(rotation about base) \( A = \frac{WL}{12ds} - \frac{2IL'}{d^2L} \)

Whence \( A \approx 2.56 \text{ sq. in.} \)

By (4a)(rotation about mid-h.t.) \( A = \frac{WL}{12ds} - \frac{4IL'}{d^2L} \)

Whence \( A \approx 2.44 \text{ sq. in.} \)

Use plate 5"x 1/2" (actual \( A = 2.50 \text{ sq. in.} \))

To find actual % restraint in elastic range use formulas (6) & (7)

\[
\%R \text{ (rotation about base)} = 95.6 \\
\%R \text{ (rotation about mid-h.t.)} = 91.5
\]

To find yield-point moment:

By (8a) \( M = 33 \times 2.5 \times 14 \approx 1157 \text{ kip-inches} \)

For \( M-N \) curve plot see Fig. 4. The shaded area above line \( AB \) indicates that the actual top plate will probably have a yield-point stress above 33 ksi and further, the top plate is subjected to complex stresses, probably triaxial stresses in some regions, and certainly stress concentrations in others, and will not yield as a uniaxially stressed member. If the triaxial stress condition is considered predominant the yield line will not be level as \( AB \) but will probably be a curved line above \( AB \).

At the present time it is admitted that the prediction of the action of the top plate in the plastic range, including the breaking point, has not been mastered.

The design of the welds at the ends of the top plate is accomplished as follows. At the connection to the column use a 100% quality butt weld as it will be stressed to 20 ksi at working load. At the beam end use fillet welds with or without slot welds, to carry a tension of \( 20 \times 5 \times 1/2 = 50 \text{ kips} \).
B. Same 14"WF-30, etc., but Design Top Plate on the Arbitrary Basis of 75% of Full Restraint and at a Unit Stress of 20 ksi.

75% of WL/12 = 7L/16 = 50 x 180/16 = 562 kip-inches. Arbitrary tension in top plate = 562/14 = 40.1 kips. Required A = 40.1/20 = 2 sq. in. Use plate 5.5" x 3/8" (A = 2.06 sq. in.)

Assume actual %R is about 90%, then actual stress in the top plate at working load will be 20 x 90/75 = 24 ksi. Design welds on basis of a true tension of about 40.1 x 90/75 = 48.1 kips. For connection to column, plate must have a minimum width of 5.5 x 90/75 = 6.6", in which case the butt weld will be stressed to 20 ksi which is OK. Design the connection between the top plate and the beam to carry a T of 48.1 kips.

Use formulas (6) & (7) to check on the actual %R:

By (6) \( \%R = 94.6 \% \)
By (7) \( \%R = 90.0 \% \)

Plastic action by formula (8a) is predicted to begin at an M of 33 x 2.06 x 14 = 953 kip-inches.

C. Same 14WF-30, etc., but Design on Basis that Top Plate Reaches 30 ksi at Working Load.

Required A of top plate by (2a) = 1.67 sq. in.
" A of " by (4a) = 1.55 " "
Use top plate 4.5" x 3/8" (A = 1.68 sq. in.)

By (6) \( \%R = 93.5 \% \)
By (7) \( \%R = 88.0 \% \)

Plastic action is predicted at an M of 33 x 1.68 x 14 = 750 kip-inches

Tension in top plate:

By (1) \( T = 50.1 \) kips
By (5) \( T = 46.7 \) kips
Design welds for a tension of 50 kips.

D. Same 14 WF-30, etc., but Design for Full End Restraint.

End M = WL/12 = 50 x 180/12 = 750 kip-inches
T = 750/14 = 53.6 kips
Required A = 53.6/20 = 2.68 sq. in.
Use plate 5" x 9/16" (A = 2.91 sq. in.)

Check on actual conditions by formulas (1) to (8a):

By (6) \( \%R = 96.0 \% \)
By (7) \( \%R = 92.5 \% \)
Plastic action is predicted at an M of $33 \times 2.81 \times 14 = 1300$ kip-inches.

Actual $T$ in top plate:

By (1) $T = 51.6$ kips
By (3) $T = 49.6$ kips

A summary of the above designs is given in Fig. 5, where it is noted that the most radical design, the one for a top plate stress of 30 ksi at working load provides approximately 90% restraint to a point slightly above the working load (the beam line in the figure); then, based on a uni-axial stress condition and on a yield point stress of 33 ksi, the connection is predicted to yield. However, at 2 x beam line (2 x working load on the beam) the connection still provides more than 50% restraint. On the other hand, the most conservative design (for full restraint) provides a connection which will maintain its elastic properties to a point near the 2 x beam line.

Discussion. The above summary must be considered as but the beginning of the study. Other questions which arise include the following:

1. Design and cost of the welding.
2. Cost of plate and plate preparation.
3. Effect of the restraining moments on the columns (design and cost).

To give some notion of the relative welding requirements in the above study, designs are given in Fig. 6. These designs should not be considered as necessarily the best ones but are for comparison purposes.

In regard to the effect of the restraining moment on the column, it is presently felt that there is considerable merit in keeping these moments low, which is accomplished best by Design C. It is assumed that there will be a companion connection on the other side of the column. In cases where there is no such companion connection the present suggestion is to substitute a flexible beam-column connection if design conditions permit.
Fig. 6. Suggested Top Plate Designs for 14 WF-50, 15° Span, 50 Kip Load.
Moment-rotation Curve for 14°WF-30
15' span, Uniform Load of 50 kips.
Top plate 5" x 1/2", Effective length = 7"
Fig. 5. H-M Curves for 14 WF-30, 15' Span, Uniform Load of 50 Kips. Effective Length of Top Plate Taken as 7".