Welded Continuous Frames

and

Their Components

Progress Report No. 32

CORNER CONNECTIONS

LOADED IN TENSION

by

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This work has been carried out as part of an investigation sponsored jointly by the following:

American Institute of Steel Construction
American Iron and Steel Institute
U.S. Navy Department (Contract 39303 and 610(03))
Office of Naval Research
Bureau of Ships
Bureau of Yards and Docks
Welding Research Council

Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania
April, 1959

Fritz Laboratory Report No. 205C.23
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I. ABSTRACT

A series of straight welded corner connections, having diagonal and half-depth vertical stiffeners, were tested in a manner which produced moments that tended to open the connection legs. This simulates a rather infrequent type of loading, which increases the possibility of weld fracture due to the stress conditions at the re-entrant corner.

These tests were carried out on connections tested previously in compression, that is, with forces and moments tending to close the connection legs.

The results presented here show that the tension form of loading does not constitute a possible limitation on the application of plastic analysis to structural design. With proper welding procedures, the desirable strength, stiffness, and rotation capacity can be realized.
II. INTRODUCTION

1. Purpose and Scope of Investigation

A series of tests was carried out at Lehigh University as part of a program on welded corner connections, a study which is a part of a general investigation of the plastic behavior of welded continuous frames and their components.

Earlier studies covered the testing of a number of corner connections of different designs, all connecting 8B13 members. (1) Later studies were made concerning the effect of size of member on connection behavior. (2)

In the phase being reported here, the tension behavior of knees is discussed. Some tension tests were conducted earlier on thirteen corner connections joining 8B13 members and the results were reported in Reference 3. This report will deal with tension tests of larger corner connections previously tested in compression for a series of tests to study a possible "size effect" and two 12WF36 corner connections which were taken from a simple portal frame. (4) One of the 12WF connections was in the "prime" or untested condition while the other was previously strained beyond the elastic limit in compression.

Under "compression loading" the forces acting on a knee tend to close it as in Fig 1a. "Tension loading" refers to that condition in which the forces tend to open a joint as in Fig 1b.
Common loadings on continuous frames require that the corner connections be subjected to combinations of bending moment and compressive thrust. Certain special cases require that the connections withstand bending moment and tensile forces rather than the normal bending moments and compressive forces that would occur under most loading conditions. These special cases would include buildings subjected to blast, flat roofs with light dead load and live loads but large lateral forces, and crowns of gable roofs. A discussion of possible tension type loadings is contained in Reference 3.

A few additional tension tests were desirable on larger specimens than previously studied to establish the behavior of connections when subjected to this type loading as a guide to design, especially since the possibility of weld failure is increased due to constraints which produce a triaxial state of stress.

Measurements made during the tests were used to determine whether the knee would meet the basic requirements for knees set forth in Reference 1. The requirements are the same for both tension and compression loading of knees in structures.

These requirements are:

a) The knee must be capable of resisting at the corner the full plastic moment $M_p$, of the rolled section joined.

b) The stiffness should be at least as great as that of an equivalent length of the rolled section joined.

c) The knee should have sufficient rotation capacity; that is, it should be capable of absorbing further rotations at near-maximum moments after reaching the plastic hinge condition.

d) The knee should be economical to fabricate.
Tension at the re-entrant corner of the knee imposes a more severe requirement on the performance of the weld in the vicinity of that point because of the tri-axial stresses that are present. Hence, the interest of these tests is primarily on the possibility of weld failure and whether or not any such failures would limit the load or deformation capacity of welded connections subjected to tension loading.

III. DESCRIPTION OF TEST SPECIMENS AND APPARATUS

2. Test Specimens

Each knee consisted of two identical members originally joined at right angles. Since all of the knees but one had been previously tested in compression, they were distorted somewhat from the earlier tests, having buckled previously, and were permanently deformed both locally and laterally. The 12WF36 "prime" connection, which was cut from a full-sized single-bay rectangular rigid frame, had a very small amount of permanent set remaining from local yielding during the frame test. However, this deformation was negligible because the moment in this knee was at all times below the yield moment during the frame test.(4) As is shown in Fig 2, the end of the web of the column was joined to the lower flange of the beam. A complete description of the test specimens is contained in Reference 2.

It was necessary to put extensions on the ends of the 12WF36 knees since they were cut from a frame tested earlier. These extensions were made from a 14WF30 rolled section (Fig 3).
The members were all rolled from A-7 structural steel, and their physical properties were measured in conjunction with the earlier test programs in which they were used. The summary of coupon test results and the measured cross-sectional properties of the 12WF36 members are given in Tables 1 and 2 of Reference 4. The same information for the 14WF30, 24WF100, 30WF108, and 36WF230 members are given in Tables 1 and 2 of Reference 2. In general, these results indicate that the average yield strength of the coupons was from 35 to 40 ksi, and that no measured dimension varied by more than 6% from handbook values.

In Table 1 of this report are listed the chemical properties of the steel used in the specimens. The chemistry is such that even with the thick flanges of the larger sections, it was considered appropriate to weld the connections without preheating or using special electrodes in accordance with the suggestions of Reference 5.

3. Loading System

In the connection tests, the knees were set up in an 800,000 lb. screw-type testing machine in the position shown in Fig 4. The knees were set with legs at 45 degrees with the horizontal and concentrated loads were applied along the hypotenuse of the triangle. The effect of this loading was to cause bending moments which were maximum at the corner, plus equal shear and tensile forces.

Steel pins twelve inches long, welded perpendicular to the plane of the web transmitted the load from plate links which were secured
to each head of the machine by 6 inch pins. Figure 5 is a diagrammatic sketch of the links and specimen. These pins were able to rotate in the plane of the connection, thus a condition of zero moment existed at the points of load application. The end pins and web were stiffened by double plates and stiffeners, as shown in Fig 2, which were also designed to carry part of the end reaction to the flanges.

4. Lateral Support

Lateral support was provided for the 12WF36, 14WF30, and 24WF100 connections by a system of four tie rods attached to the inner and outer corners of the knees at the tips of the flanges. The tie rods were arranged so as to restrain the knees from any deflection except in a vertical plane. A description of the lateral support system is contained in Reference 2.

Since the connections were permanently deformed both locally and laterally during the previous compression tests, there was a tendency to induce large forces in the lateral support system as the connection tended to straighten out when the load was applied. This necessitated frequent adjustments in the lateral support system. Since the magnitude of the lateral forces so measured was meaningless, it was decided not to provide lateral support for the 30WF108 and 36WF230 knees.

5. Rotation Measurements

Rotation indicators of the types indicated in Figs. 6 and 7 were used to determine the rotation of the knee. Seven indicators of the type shown in Fig 6 were used on the 12WF36 connections: one across the knee and three across each leg. The other connections used five
rotation indicators of the type shown in Fig 7: one across the knee and two across each leg. A detailed description of the development and use of this type rotation indicator is given in Reference 6.

6. Deflection Measurements

The relative deflection of the ends of the legs was measured by a single Ames dial, attached to a rod which spanned the hypotenuse of the triangle formed by the legs of the knee (See Fig 5). The plunger of the dial was set in a punch mark set directly in line with the pin of the lower leg of the knee.

Changes in the average length of the moment arm were measured by means of a "mirror gage". This consisted of a plumb bob attached by a wire or string to the outside corner of the knee and suspended in a bucket of water placed at the base of the testing machine, thus preventing excessive swaying. A 0.01 inch scale equipped with a mirror was used to measure the horizontal displacement of the string or wire.

7. Test Procedure

In the conducting of each test, four phases of the test required slightly different procedures. These phases were:

1. Initial adjustment of the test apparatus and taking of zero reading of the instrumentation.
2. Elastic range loading, with load and deformation readings controlled by a load criterion.
3. Plastic range loading, with load and deformation readings controlled by a deflection criterion.

4. Plastic range loading, after maximum load, with a minimum of deformation readings.

A complete description of each of the individual phases is included in Reference 2 and is directly applicable to this report. It was necessary to watch for the development of a possible weld fracture in addition to the points outlined in the test phases.

IV. ANALYSIS AND DESIGN OF SQUARE KNEES

8. Theoretical Analysis

The theoretical analysis of the connections reported herein was presented in earlier reports. There is no significant difference between the theoretical analysis for either tension or compression loading upon a knee. Hence, much of the procedure and theory reported in Reference 2 is directly applicable to this companion report. The results that apply to both reports are summarized in Tables 2, 3 and 4.

The only characteristic of behavior that is changed, except for the possibility of weld fracture, is local buckling. Local buckling of the compression flange is delayed during the tension test in comparison with the compression test. Figure 1 indicates the reason why local buckling is more critical for compression than for tension. Under compression loads the local buckle would occur at lower moments because the compression flexural strains are additive to the direct
strains resulting from the axial compressive force. However, under tension loading, the compression flexural strain is reduced by the superimposed direct tensile strains. Therefore, local buckling occurs at higher moments when the connection is subjected to "tension" loadings. Table 5 shows the moments at which local buckling occurred for both tension and compression tests.

9. Design of the Knees

The detailed procedure for the design and fabrication of the knees is presented in References 2 and 7. The stiffening required to prevent excessive distortion, the design of the welds, and the welding sequence that was used in order to minimize distortion and prevent weld cracking are outlined and discussed in Reference 2. Reference 7 contains a sample analysis and design of the 24WF100 connection and shows the development of the required stiffening.

V. TEST RESULTS

The results of the experimental investigation and their correlation with the data obtained from the compression test are next presented. The discussion follows.

10. Moment-Deflection Results

In Fig 8 the non-dimensional curves of moment versus deflection are given for the connections tested in this series. They are compared with the curves obtained from the earlier compression tests. The values of $M_y$ and $\delta_y$ are computed from the theoretical considerations
as presented in Reference 2, using coupon data and actual cross section measurements. The haunch moment, \( M_h \), is computed at the intersection of the neutral lines of the adjoining members of the connection. In all experimental curves, the moment has been corrected for the measured increase or decrease in moment arm due to the deflection between end pins. Numerical results of the non-dimensionalized curves can be obtained from Tables 2 and 3.

11. Moment-Rotation Results

In Fig. 9, the non-dimensionalized curves of moment versus rotation are presented. Each curve compares the results obtained from the tension test with those obtained from the compression test. \( \Theta_T \) is the total rotation experienced by the connection in the vicinity of the junction of column and beam. It includes the rotation of the legs over the portion to which the rotation indicators are attached.

It should be pointed out that erratic behavior of the elastic portion of the moment-rotation results must be expected for the tension tests. This is primarily due to the buckled configuration of the connection resulting from the prior compression tests. As load was applied, the connections tended to align and straighten out causing the uncertain behavior in the rotation indicators.

12. Strength and Rigidity of the Connections

Figure 8 shows the strength of connections subjected to tension loads. Table 2 presents a summary of the numerical values of the theoretical strength of the connections. The maximum moment values shown in the non-dimensionalized curves can be obtained with the aid of Table 2.
Figure 9 indicates the rigidity of the connections tested in tension. The earlier non-linear behavior exhibited at lower loads than in the previous compression tests is primarily due to the Bauschinger effect. (A phenomenon that occurs when a specimen is loaded in tension or compression above the proportional limit and then reloaded in the opposite direction. It is observed that yielding of the specimen as a whole occurs at a much reduced stress.) A more comprehensive explanation of the effect can be found in Reference 8.

VI. DISCUSSION OF TEST RESULTS

Before beginning discussion of the individual test results, the significance of the moments $M_p$ and $M_h(p)$ will be interpreted for the general case.

In the analysis of a flat-roofed rigid frames, the ultimate load would be based on the assumption that the moment at the intersection of the beam and column centerlines was $M_p$. This assumption neglects the fact that the column and beam sections have depth. Any test result in which the moment at the haunch $M_h$ equals or exceeds $M_p$ indicates a connection adequate to be used in a rigid frame.

Observation of previous tests has shown that, in square corner connections, the plastic hinge will ordinarily form in the rolled section outside the corner.\(^1\),\(^2\),\(^3\) Therefore, the moment, $M_r$, at the edge of the rolled section may reach $M_p$ (corrected for axial load) and the moment, $M_h$, at the haunch may reach a greater value $M_h(p)$, depending on the moment gradient. (see sketch in Table 2). This greater moment $M_h(p)$ is ordinarily reached without requiring additions to the design or fabrication procedure. It is of interest to the
experimenter to compare the maximum haunch moment achieved in a test with a predicted value for $M_p$ merely to gauge the accuracy with which such predictions may be made. Let it remain clear, however, that reaching $M_p$ is all that need be required of a haunch to satisfy a design standard for strength.

Of course, it is not inconceivable that a designer might elect to re-evaluate a design on the basis of this small increase in moment supplied and reduce the size of sections used. However, this procedure is not recommended because it increases the complexity of the analysis. It is probably more practical to have this small reserve of strength available as an additional safety measure against instability.

13. Discussion of Each Connection Tested

12WF36 (Prime) (242T1)

In Figure 8a the moment-deflection curve of the knee is shown. The curve shows that the connection followed very closely the predicted theoretical curve to yield. The ultimate moment was about 35% greater than the theoretical yield moment. The connection was capable of carrying a moment equal to or greater than the theoretical plastic moment through a deflection, of ten times the predicted theoretical yield deflection. In Fig 9a, a comparison is made between the moment-rotation curves for the compression and tension test of the prime connection. It can be noted that very little difference exists between the two curves and the tension curve followed very closely the predicted theoretical curve. The connection was able to sustain large rotations at moments greater than the plastic moment. There is little if any variation in the elastic range that can be attributed to the Bauschinger effect since the connection was not loaded beyond the elastic limit during the compression test.
Failure was brought about primarily by lateral-torsional buckling, although some local buckling was present. There were no weld cracks observed at any time during the test.

12WF36 (Pretested)(242T2)

This knee was deformed from the prior frame test and a limited amount of local and lateral deformation remained. From the moment-deflection curve (Fig 8a) the effect of the prior compression test shows little if any influence on the elastic behavior of the connection. It can be observed from Fig 9b that the connection was not required to sustain as large an amount of deformation during the compression test as in some of the larger connection tests. Hence, it is probable that this would not influence the elastic behavior as in the larger size connections, and the Bauschinger effect might not be as pronounced for this connection.

Figure 9b shows a comparison between the moment-rotation curves of the tension and compression tests. Note that very little deviation is evident in the elastic portion of the curves. The connection was able to sustain a much greater rotation under the tension loading at moments greater than the plastic moment since local and lateral buckling was delayed.

The connection was able to develop an ultimate moment which was 40% greater than the theoretical yield moment. This ultimate moment was about 15% higher than the ultimate moment of the prior compression test. Failure was again brought about by lateral-torsional buckling similar to that of the prime knee.
In Fig 8b, the moment-deflection curves for both the tension and compression tests are shown just above the results for the 12WF36. There was very close agreement between the two curves until about 75% of the theoretical yield was reached. Then the curve of the tension test started deviating quite rapidly due to the effect of prior loading.

The apparent deviation did not prevent the connection from developing an ultimate moment which was about 10% greater than the ultimate moment obtained during the compression test.

In Fig 9c, the moment-rotation curves are shown. Again there is a slight discrepancy during the elastic portion of the tension curve due to the configuration of the knee prior to testing. The knee was able to sustain much larger rotations when tested in tension at moments equal to or greater than the plastic moment.

Failure of the connection was brought about primarily by lateral-torsional buckling. During the compression test of this knee, failure was primarily from local buckling. The reason for this difference in failure mode was mentioned in Section 8.

A comparison of the moment-deflection curves for the tension and compression tests is shown in Fig 8d. The knee started deviating from the theoretical curve at very low loads. The ultimate moment was about 25% higher than the ultimate moment of the compression test.
There is a marked discrepancy of the moment-rotation curve for the tension test shown in Fig. 9e. It shows the experimental rotation being considerably less than predicted until the yield moment is reached. This does not coincide with the results obtained in Fig. 8d. This discrepancy is probably due to the lateral and local deformation that remained from the previous compression test. As the connection tended to straighten, erratic behavior occurred in the rotation indicators. At times they indicated rotation in the sense opposite to that expected, presumably due to the lateral deformation.

Once the load had passed the predicted yield, the connection behaved as expected; it sustained a moment greater than or equal to the plastic moment through a rotation of approximately twice that obtained during the compression test.

Eventually, after the connection had rotated far beyond any value that would be required in plastic design, lateral-torsional buckling occurred (Fig. 10). No weld cracks were observed during the test.

The moment-deflection results of the tension and compression tests are compared in Fig. 8c. The departure from linearity started at about 40% of the theoretical yield moment. The ultimate moment attained by the knee is very nearly the same for both the tension and compression tests. The early departure from linearity had no adverse effect upon the strength of the connection.

In Fig. 9d the moment-rotation results are compared for the tension and compression test. The departure from linearity is evident from
the onset of the test. The connection was able to sustain rotation at moments equal to or greater than the plastic moment. This rotation exceeded the rotation obtained during the compression test by about 30%. Final failure of the knee was primarily due to lateral-torsional buckling.

36WF230 (T43)

This was the largest connection tested in this series. In Fig 8e the moment-deflection results are presented for both the compression and tension test. Again, the tension test shows greater flexibility. Figure 8e shows that the connection exceeded the predicted maximum moment by about 8% and the plastic hinge moment by about 20%.

After the connection had deflected an amount equal to about 8 times the predicted yield deflection, a fracture occurred. The moment reached was slightly below the ultimate moment which had been reached in compression, but was still 18.6% greater than that required by design. The deflection sustained in the tension test was about 60% of the deflection in the compression test.

The moment-rotation results are shown in Fig 9f. As with the 24WF100 connection, the measured rotation in the elastic range was less than the theoretical predicted value. However, at about 60% of the predicted yield the knee showed a rather rapid departure from linearity. The connection was able to maintain moments equal to or greater than the plastic moment through a rotation of five times the predicted yield rotation. Additional deformation was not possible due to failure of the beam flange adjacent to the butt weld
at the re-entrant corner. The failure of the knee was due to brittle fracture occurring in the beam flange at the re-entrant corner of the inside flanges. The crack propagated to the midpoint of the web of the beam, and the load-carrying capacity of the connection immediately dropped.

Since the connections subjected to tension-type loadings are seldom encountered in practice, and since when they do exist they require much less deformation capacity than those subjected to compression-type loading, no limitation to plastic design is implied from the fracture. Eventually a component must fail, and due to the geometry and type of loading it happened that this one fractured instead of failing in the usual buckling mode.

Figures 12 and 13 are photographs showing the extent of fracture in the beam web and the fracture surfaces. The main body of the fracture is typically "brittle" as evidenced by the small lateral contraction at fracture, the absence of shear lip, and the cleavage appearance of the fractured surface.

Note that the weld itself had sufficient strength to force the failure to occur adjacent to it.

14. Performance of Welds

In five of the six connections tested, there were no cracks or weld failures observed. Only the 36WF230 knee exhibited a fracture which brought about failure of the connection. However, it is clear from Figs 8e and 9f that the connection was able to sustain some plastic deformation prior to this failure.
As indicated in Reference 2 it was not considered necessary to use a preheat or low hydrogen electrode on any of the connections fabricated. According to Ref. 9, with carbon less than or equal to 0.25% and the thickness being welded over one inch but less than two inches, it is generally recommended that some control be made over the heat input and that a low hydrogen electrode or preheat be used in order to prevent a brittle type failure. This was not done on the 36WF230 connection which had an average flange thickness of 1 1/4 in. When this connection was tested in compression, satisfactory behavior was obtained and no weld cracking was observed. However, when tested under a tension-type loading, a brittle type failure did occur. Where extremes in the service temperature of a building are anticipated, it would be necessary to take precautions in order to prevent any possibility of brittle failure.

The chemical properties of the ladle analysis and an analysis made on a core specimen after the fracture occurred are shown in Table 6. It can be noted that the carbon content is 0.04% greater in the core specimen. However, as pointed out in Reference 5 there may be as much as 0.04% more carbon in the steel as delivered as compared to the ladle analysis.

It is interesting to note that the recommendations of References 5 and 9 put the welding of the 36WF230 connection in a borderline category. That is, Reference 9 recommends the use of preheat or low-hydrogen electrodes, and Reference 5 does not. At the same time, the test results must be interpreted as also being in a borderline category. The connection reached its ultimate load and withstood sufficient deformation to allow it to do its job satisfactorily.
Other observations with regard to the welding procedure followed are:

(1) If the forty-five degree fillet welds on the end of the diagonal stiffener are made by down hand passes instead of the vertical fillets that were made, sounder welds would result. It is very difficult to get the proper penetration with vertical fillet welds at that angle.

(2) If notches had been placed at the ends of the diagonal stiffeners they would have helped relieve the stress concentrations that exist at the corners. In other words, had the weld details been designed for tension loading, it is quite likely that its performance would have been much better.

VII. SUMMARY AND CONCLUSIONS

The following observations can be made from the results of this study of square knees under "tension loading", in which the re-entrant corner of the knee is in tension.

1) All connections reached the predicted plastic moment and exceeded it by an amount that varied from 4 to 33% (Fig 8 & 9).

2) Local buckling occurred at higher moments in the tension test than it did in the previous compression test (Table 5).

3) The ultimate moments reached during the tension tests were higher than the ultimate moments that were obtained during the prior compression test, except for the 30WF108 and 36WF230 connections. The ultimate moment attained by the 30WF108 connection was approximately the same in both tests. The ultimate moment of the 36WF230 connection was about the same as that of the prior compression test when failure of the beam occurred. (Figs 8 and 9)
4) Sufficient rotation was attained at moments equal to or greater than the plastic moment to have allowed redistribution of moments had the knee been part of an indeterminate structure (Fig. 9). Since the plastic hinge would form last at the tension corner, very little rotation is required at that point.

5) All connections but the 36WF230 knee failed by lateral torsional buckling with some local buckling in the web and flanges. The 36WF230 failed due to fracture of the flange adjacent to the butt weld at the re-entrant corner (Figs. 10 through 13).

6) When the thicknesses of sections to be joined exceed one inch, careful attention should be paid to the chemistry of the rolled sections to determine if a preheat or low-hydrogen electrode is required to prevent premature failure of the welds. The recommendations of Greenberg(5) and of Stout and Doty(9) provide data to help determine the proper welding procedure.

7) With proper welding procedure and with careful inspection of welding, the development of plastic hinges in large-size members without fracture of the welds may be assured. These connections are able to absorb sufficient rotation at near-maximum moment after reaching the plastic hinge condition.

8) The prior loading and resulting plastic deformation from the previous "size-effect" studies cause some erratic behavior to occur in the elastic and initial plastic portions of the tests for the tension study. Such things as erratic rotation, deflection and lateral support behavior may be attributed to the prior loading. Hence, the results are probably conservative. Prime specimens would have performed better.
9) The test results show that, even with "improper" welding procedure, plastic design of rigid frames would be no more limited by fracture than would a comparable elastic design. In most structures the hinge subjected to the tension-type load would be the last to form and would not require the additional rotation needed by the other hinges. Thus, the 36WF230 which did attain the plastic moment prior to fracture would be satisfactory.
VIII. ACKNOWLEDGEMENTS

This report has been prepared as a result of research carried out at Lehigh University in the Fritz Engineering Laboratory. William J. Eney is Director of the Laboratory and Head of the Department of Civil Engineering. The work was done as part of a larger program on Welded Continuous Frames and Their Components directed by Lynn S. Beedle. F. W. Schutz supervised the planning and execution of the test program. Technical guidance and supervision for this project was furnished by the Lehigh Project Subcommittee of the Structural Steel Committee, Welding Research Council. T. R. Higgins is chairman of the Lehigh Project Subcommittee. The authors wish to express their sincere appreciation to the Lehigh Project Subcommittee for valuable suggestions received.

Thanks are also extended to Kenneth R. Harpel, foreman, and the Laboratory staff of technicians, who erected the specimens and offered helpful suggestions in regard to test apparatus. Bryan C. Chapman assisted in the tests. The results of the tension tests on the 12WF36 connections were supplied by R. B. Madison and T. E. Gunn. The assistance furnished by members of the Fritz Laboratory staff in the preparation of this manuscript is gratefully acknowledged.
IX. REFERENCES


X. NOMENCLATURE AND TERMINOLOGY

A = Area of section
a = Distance between point of inflection and re-entrant corner of connection.
b = Flange width
d = Depth of section
E = Young's modulus of elasticity
E_{st} = Strain-hardening modulus
F = Flange force
F_s = Force in stiffener
f = Shape factor = Z/S
G = Shearing modulus of elasticity
I = Moment of inertia of section
L = Distance between the point of inflection and the haunch point
M_H = "Haunch" moment, Subscript y refers to moment at yield; subscript p refers to moment at reduced plastic moment
M_p = "Hinge" value of full plastic moment; the ultimate moment that can be reached at a section according to the simple plastic theory = \sigma_y Z
M_r = Moment in a connection at junction of rolled beam and connection
M_y = Moment at which yield point stress is reached in the rolled section \sigma_y L(f)^S
P = Load on knee
P_y = Load when yield stress is first reached in the extreme fibers of rolled section.
r = Distance from end of knee to point of rotation measurement.
S = Section modulus of beam
t = Flange thickness
t_s = Stiffener thickness
w = Web thickness
\[ w_r = \text{Required web thickness} \]

\[ Z = \text{Plastic modulus; the combined statical moments of the cross-sectional areas above and below the neutral axis. Subscript } a \text{ refers to area carrying axial force, subscript } 2 \text{ refers to plastic modulus of web.} \]

\[ \delta = \text{Deflection; subscript } y \text{ refers to deflection at yield.} \]

\[ \varepsilon = \text{Strain; subscript } st \text{ refers to strain-hardening, } y \text{ refers to yield} \]

\[ \Theta = \text{Rotation (subscript } T \text{ refers to total rotation within a rotation indicator, } y \text{ refers to rotation of knee at yield)} \]

\[ \sigma = \text{Direct stress (bending); } \sigma_{yl} \text{ = lower yield - point stress; subscripts } f \text{ and } w \text{ refer to flange and web.} \]

\[ \tau = \text{Shear stress} \]
TABLES

1. Chemical Properties
2. Summary of Theoretical Moments
3. Summary of Theoretical Loads and Deformations
4. Summary of Theoretical Shear Stresses and Web Reinforcement
5. Comparison of Moments at which Local Buckling Occurred
6. Chemical Analysis of Steel for 36WF230 Connection
TABLE 1

CHEMICAL PROPERTIES

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>HEAT NO.</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>12WF36</td>
<td>---------</td>
<td>0.18</td>
<td>0.65</td>
<td>0.014</td>
<td>0.038</td>
<td>A7-50T</td>
</tr>
<tr>
<td>14WF30</td>
<td>41K525</td>
<td>----</td>
<td>----</td>
<td>0.023</td>
<td>0.044</td>
<td>A7</td>
</tr>
<tr>
<td>24WF100</td>
<td>44D508</td>
<td>0.18</td>
<td>0.56</td>
<td>0.016</td>
<td>0.039</td>
<td>A7-52T</td>
</tr>
<tr>
<td>30WF108</td>
<td>44G451</td>
<td>0.20</td>
<td>0.60</td>
<td>0.01</td>
<td>0.030</td>
<td>A7-53T</td>
</tr>
<tr>
<td>36WF230</td>
<td>35D654</td>
<td>0.19</td>
<td>0.70</td>
<td>0.014</td>
<td>0.032</td>
<td>A7-52T</td>
</tr>
</tbody>
</table>
\[ M_p = \sigma_y Z \]
\[ M_y = \sigma_y S \]
\[ M_{allowable} = \text{AISC Allowable Moment} \]
\[ M_{all} = \text{AISC Allowable Moment (20,000 psi)} \]

**THEORETICAL MOMENTS**

<table>
<thead>
<tr>
<th>Section</th>
<th>Basis of Calculation</th>
<th>Elastic Range</th>
<th>Plastic Range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( M_y ) (parameter)</td>
<td>( M_{allowable} ) (AISC)</td>
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<tr>
<td>12WF36</td>
<td>Measured**</td>
<td>1700</td>
<td>982</td>
</tr>
<tr>
<td></td>
<td>Handbook†</td>
<td>1515</td>
<td>875</td>
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<tr>
<td>14WF30</td>
<td>Measured</td>
<td>1488</td>
<td>757</td>
</tr>
<tr>
<td></td>
<td>Handbook</td>
<td>1380</td>
<td>830</td>
</tr>
<tr>
<td>24WF100</td>
<td>Measured</td>
<td>8,450</td>
<td>4,440</td>
</tr>
<tr>
<td></td>
<td>Handbook</td>
<td>8,210</td>
<td>4,590</td>
</tr>
<tr>
<td>30WF108</td>
<td>Measured</td>
<td>10,100</td>
<td>5,360</td>
</tr>
<tr>
<td></td>
<td>Handbook</td>
<td>9,870</td>
<td>5,540</td>
</tr>
<tr>
<td>36WF230</td>
<td>Measured</td>
<td>29,450</td>
<td>15,100</td>
</tr>
<tr>
<td></td>
<td>Handbook</td>
<td>27,600</td>
<td>15,400</td>
</tr>
</tbody>
</table>

* \( M_{all}, M_h, \) & \( M_r \) were calculated considering the effect of axial load.

**"Measured" quantities calculated using measured dimensions and coupon stresses.

† "Handbook" quantities calculated using AISC handbook dimensions and implied yield stresses. (33ksi)
### THEORETICAL LOADS AND DEFORMATIONS

<table>
<thead>
<tr>
<th>Section</th>
<th>Basis for Calculation</th>
<th>Loads</th>
<th>Deformations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( P_{\text{all}} ) (kips)</td>
<td>( P(y) ) (kips)</td>
</tr>
<tr>
<td>12WF36</td>
<td>Measured Handbook</td>
<td>15.45</td>
<td>25.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.80</td>
<td>22.75</td>
</tr>
<tr>
<td>14WF30</td>
<td>Measured Handbook</td>
<td>12.70</td>
<td>23.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.94</td>
<td>23.0</td>
</tr>
<tr>
<td>24WF100</td>
<td>Measured Handbook</td>
<td>65.5</td>
<td>114.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>67.5</td>
<td>111.4</td>
</tr>
<tr>
<td>30WF108</td>
<td>Measured Handbook</td>
<td>63.2</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td></td>
<td>65.1</td>
<td>107.4</td>
</tr>
<tr>
<td>36WF230</td>
<td>Measured Handbook</td>
<td>148.5</td>
<td>267</td>
</tr>
<tr>
<td></td>
<td></td>
<td>151.0</td>
<td>249</td>
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</table>

**TABLE 3**
### Theoretical Shear Stresses & Web Reinforcement

<table>
<thead>
<tr>
<th>Section</th>
<th>Basis of Calculation</th>
<th>Shear Yield Strength of Web (τ_y) (ksi)</th>
<th>Shear Stress (τ) (ksi)</th>
<th>Reinforcement Required (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Without Diagonal Stiffener</td>
<td>With Diagonal Stiffener Used</td>
<td>Web Thickness Required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>τ at M_h=M_h(y)</td>
<td>τ at M_h=M_h(p)</td>
<td>τ at M_h=M_h(p)</td>
</tr>
<tr>
<td>12WF36</td>
<td>Measured Handbook</td>
<td>22.6</td>
<td>33.6</td>
<td>38.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.05</td>
<td>29.10</td>
<td>34.6</td>
</tr>
<tr>
<td>14WF30</td>
<td>Measured Handbook</td>
<td>24.9</td>
<td>25.6</td>
<td>31.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.05</td>
<td>24.2</td>
<td>27.4</td>
</tr>
<tr>
<td>24WF100</td>
<td>Measured Handbook</td>
<td>23.0</td>
<td>24.0</td>
<td>30.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.05</td>
<td>24.4</td>
<td>29.2</td>
</tr>
<tr>
<td>30WF108</td>
<td>Measured Handbook</td>
<td>22.0</td>
<td>17.65</td>
<td>21.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.05</td>
<td>16.40</td>
<td>20.3</td>
</tr>
<tr>
<td>36WF230</td>
<td>Measured Handbook</td>
<td>24.6</td>
<td>23.5</td>
<td>29.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.05</td>
<td>22.6</td>
<td>27.2</td>
</tr>
</tbody>
</table>

* When τ exceed τ_y the stress is meaningless and indicates a stiffener is required.

**Table 4**
TABLE 5

COMPARISON OF MOMENTS AT WHICH LOCAL BUCKLING OCCURRED
(Kip - Inches)

<table>
<thead>
<tr>
<th>TYPE LOAD</th>
<th>CONNECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>12WF36</td>
</tr>
<tr>
<td>COMPRESS</td>
<td>2153</td>
</tr>
<tr>
<td>TENSION</td>
<td>2380</td>
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</tbody>
</table>

TABLE 6

CHEMICAL ANALYSIS OF STEEL FOR 36WF230 CONNECTION

<table>
<thead>
<tr>
<th>Element</th>
<th>Ladle Analysis</th>
<th>Core Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>0.19%</td>
<td>0.23%</td>
</tr>
<tr>
<td>Mn</td>
<td>0.70</td>
<td>0.68</td>
</tr>
<tr>
<td>P</td>
<td>0.014</td>
<td>0.013</td>
</tr>
<tr>
<td>S</td>
<td>0.032</td>
<td>0.035</td>
</tr>
<tr>
<td>Si</td>
<td>*</td>
<td>0.11</td>
</tr>
</tbody>
</table>

* Not recorded
Fig. 1  DIAGRAM SHOWING COMPRESSION AND TENSION LOADINGS
Fig. 2 GENERAL VIEW SHOWING GEOMETRY OF CONNECTIONS
Fig. 3 VIEW OF 12WF36 CONNECTION SHOWING EXTENSIONS
Fig. 4  Overall view of 24WF100 corner connection in testing machine prior to tension test.
Fig. 5 SCHEMATIC DRAWING OF TEST SET-UP.
Fig. 6. Left. Rotation indicators on 14WF30 corner connection. One inch angles were used to support dial gages.

Fig. 7. Below. Rotation indicators on 36WF230 corner connection. One-half inch diameter rods were used to support dial gages.
Fig. 8
MOMENT - DEFLECTION CURVES
Fig. 9
MOMENT - ROTATION CURVES

KEY

\[ \frac{M_h}{M_y} \]

\[ \frac{M_p}{M_y} \]

\[ \frac{M_h(y)}{M_y} \]

Rotation Indicator

Tension

Compression

36WF230 (f)
24WF100 (e)
30WF108 (d)
14WF30 (c)
12WF36 (Prime) (a)
12WF36 Pretested (b)
Fig. 10  Lateral buckling of 24WF100 corner connection

Fig. 11  Lateral buckling of 30WF108 corner connection
Fig. 12  Extent of fracture in 36WF230 Connection

(a) Above: Web of beam
(b) Below: Inner flange of beam
Fig. 13 Fracture Surfaces in 36WF230 Connection

(a) Above: Facing beam.
(b) Below: Facing half-depth vertical stiffener. To the left is a short length of gas-burned surface made to allow photographing.