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Connections for welded continuous portal frames part 3, Progress Report No.4, Discussion of Test Results and Conclusions

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FOREWORD TO PART III

Members of the Committee have received draft copies of Parts I and II of this paper which were subsequently approved and have been published in the July and August, 1951, issues of the Welding Research Supplement.

The numbering system for the figures, tables, references, and equations continues the sequence of Parts I and II. At the present writing it is not anticipated that figures appearing in Parts I and II will be reproduced in Part III.
I. DISCUSSION OF TEST RESULTS

The objectives of the investigation, outlined in Part I, form the basis for the arrangement of this discussion of test results. First, the results are examined to see if the connections satisfy the requirements of elastic design. Are the assumptions of present design practice consistent with safety? Secondly, it is of interest to observe the behavior beyond the elastic limit and up to collapse in order to evaluate the possibilities and limitations of new concepts of plastic design and analysis.

1. ELASTIC STRAIN DISTRIBUTION IN STRAIGHT KNEES

The portions of Connection P outside the knee behaved according to the ordinary beam theory, Fig. 32** and 33**. At sections close to the knee, within 2" of the vertical stiffener, the stress distributions become irregular due to end disturbances, this being usual in the case of any end connection or bearing support. However at sections of the beam and column removed from the connection a distance of half the depth of the section, the stresses were in reasonably good agreement with the computed values. As was pointed out in Part II the tensile stresses in the exterior beam flange are transmitted to the knee by shear (Fig. 62***) and such shear stresses must be con-

*This treatment of elastic stress distribution is only partially complete, and it is expected that further data will be presented in a separate report.

**Fig. 1 to 57, inclusive, will be found in Part I as published in the July 1951 Welding Journal, pp 359s to 384s; Fig. 58 to 73, inclusive, are in Part II, August 1951 Welding Journal, pp. 397s to 405s. Fig. 52 is included in this report as well as in Part I.
sidered. In Connection P, for example the maximum web shear stress was larger than that at the critical section for bending. Later equations give the required web thickness to prevent these undesirable deformations due to shear force.

Fig. 34 and 35 show that there are local strains that differ from the values predicted but the trend is to confirm the assumption for Type 7 connections that bending moment decreases linearly from a maximum at the critical section where the knee joins the rolled section to zero at the exterior corner. In the case of the beam, as shown in figure 34, no correction was made for bending in the outer flange since SR-4 gages were mounted on but one side. From Fig. 21, noting the shape of the deformed top flange in the region of the knee, it is seen that the measured strains are consistent with the deflected shape. As expected, the stress is a maximum at the face of the column.

The theoretical distributions of stress shown in Figs. 32-35 take into account both bending moment and direct stress, although the influence of the latter is quite small. Since the lengths of members are short, additional moment due to column deflection is negligible and has not been considered.

2. ELASTIC STRENGTH OF CONNECTIONS

In this section will be discussed the behavior of the connections at loads in the vicinity of the yield point. The evidence and influence of residual stresses and the formation of "yield lines" as revealed by mill scale will also be discussed.
2A. Shear Yield of Straight Knees Without Diagonal Stiffeners.

Equations have been developed in Section I, Part II to predict the moment at which yielding of Type 7 connections due to shear force should commence and a comparison has been made with theoretical yield moments due to flexure. Connection P provided an opportunity to compare theory with experiment, Fig. 19 of Part I containing both theoretical and experimental curves. As is evident from this figure, non-linear behavior of the connection commenced at a moment somewhat lower than the predicted moment at which yielding due to shear force should commence. Subsequent rotation was developed well beyond that which could be tolerated in most engineering structures. The first yield line was observed at 311 in-kips, non-linear behavior was observed visually from the plotted curve at 493 in-kips, and the maximum moment carried was 1150 in-kips. The theoretical "shear yield" values are 724 in-kips assuming a uniform distribution of shear stress in the knee and 630 in-kips assuming a non-uniform shear distribution (See Part II of the paper). The predicted initial yield moment in flexure is 1195 in-kips.

Comparing theory with experiment, the first yield line was observed at about 50% of the predicted "shear yield" load, non-linear behavior was observed about 20% below the predicted value, and the rate of increase of deflection increased markedly at a load as low as 50% of the calculated flexural yield load. The latter relationship is the one requiring attention since the computation of moments at which shear yielding com-

* See footnote next page
mences is not a part of routine analysis procedures. In spite of the large rotations, the connection did not develop the predicted flexural yield strength of the weakest adjacent member.

The assumption of uniform shear distribution in the knee web gives an approximate prediction of actual behavior, but the assumption of non-uniform distribution of shear stress provides a much better indication of the load at which inelastic deformation of connections with unstiffened webs will commence.

Following the usual steps for proportioning a Type 7 connection the designer would check the shear in the beam and column; but it has been demonstrated here that it is most important for him to check the shear in the knee web. To make sure that the "shear" type of failure does not occur, the moment at which shear yield occurs, $M_{h(t)}$, must be equal to or greater than the moment which results in flexural yield, $M_{h(\sigma)}$.

These expressions, developed in Part II, are,

$$M_{h(t)} = \frac{\sigma_y}{2} \frac{w d^2}{(1-d/L)}$$  \hspace{1cm} (3)

$$M_{h(\sigma)} = \sigma_y \left[ \frac{(1-d/L)}{5} + \frac{1}{A_L} \right]$$  \hspace{1cm} (4)

* These reductions are due to stress concentrations and residual stresses and are consistent with observations made in continuous beam tests. (35).

** It is assumed that the proportions are such that web buckling does not occur in the elastic range of stress.
By equating these two expressions the required web thickness is obtained,

$$W^2 = \frac{2}{5\alpha L(1-\frac{d}{2L})^2} \left[ \frac{1 - \frac{d}{L}}{1 - \frac{d}{2L}} \right]$$

$L$ is the distance from the center of the knee (the haunch point) to the load points on the connection arms (Fig. 74). In an actual frame $L$ is the distance between the point of inflection and the haunch point.

Equation 36 may be simplified to a convenient expression for required web thickness. If the proportions of connection $P$ are assumed ($L/d = 6$), the third term in Equation 36 is $0.91$. For 14WF30, 8B13, 21WF82, 6B12, 24WF110, and 8WF31 shapes the second term in Equation 36 ranges from 1.870 to 1.888; the average product of the two terms in brackets is 1.71. Of course, the selection of the single $L/d$ ratio is an arbitrary one. Instead of $L/d = 6$, if the basis for comparison had been selected as the "$I_u$" value for each member (L corresponding to $Ld/bt = 600$), then the product of the last two terms in Equation 36 is 1.82. Taking an average value, then, of 1.76, a suggested expression for web thickness $w$ is obtained,

$$w = 1.76 \frac{S}{d^2}$$

This result is based on an assumption of uniform distribution of shearing stress in the knee web. A more conservative relationship than that given by Equation 37 would be obtained if the required web thickness to prevent premature inelastic shear deformation were increased by about 15% to take into account the actual nonuniform shear distribution. In this case,
By the rule of Equation 38 none of the WF shapes would provide sufficient web thickness to give an adequate Type 7 connection without additional stiffening. One WF shape (12WF16\(\frac{1}{2}\)) is very nearly adequate. One shape of every nominal depth has been checked by computation and several shapes within one series have also been studied. The lighter members in each nominal size more nearly meet the web thickness requirement and would require the least amount of additional stiffening.

Computations have also been made for American Standard I shapes. For each nominal depth the heaviest section theoretically has adequate web thickness without requiring additional stiffening material. Most of the lighter sections in each series have insufficient web thickness.

Additional tests should be conducted specifically for the purpose of checking the validity of Equation 38 as a rule for specifying the required thickness of doubler plates or diagonal stiffeners. However, since most rolled shapes are deficient in web thickness for Type 7 connections, it is recommended that design rules require diagonal stiffeners or doubler plates.

The 8B connection uses the former detail. If the diagonal plate is objectionable, extra web thickness may be obtained with doublers using Equation 38 as a guide. Connections of this type were not tested in the program, although such studies are planned.

In the case of Type 8B connections, Equation 38 and the assumptions of Part II may be used to specify the required thickness
of diagonal stiffeners. In Equation 29 the "effective" area of the web was assumed to be made up of two parts: the actual web area, \( A_w = w \times d \), and an equivalent stiffener area, \( A_s \), arrived at by assuming the stiffener material uniformly distributed over the web plate. From Equation 29,
\[
A_s^e = \sqrt{2} (b_s \ t_s)
\]
where
\[
t_s \quad \text{stiffener thickness}
\]
\[
b_s \quad \text{total width of stiffener.}
\]
Since this equivalent stiffener area makes up the deficiency in web thickness, then
\[
A_s^e = (w_r - w_a) \ d
\]
where \( w_r \) is the required web thickness according to equation 38 and \( w_a \) is the actual web thickness of the rolled shape, and \( d \) is the depth of section. Equating the two expressions for \( A_s^e \), the required thickness of diagonal stiffener is given by
\[
\frac{t_s}{\sqrt{2} b_s} = \frac{(w_r - w_a) \ d}{\sqrt{2} b_s}
\]

An examination of Fig. 20 shows that the experimental deflection curve deviates from a straight line at approximately the same moment as that of the rotation curve (Fig. 19). This indicates that knee deformations cause the non-linear behavior at the low loads. This is clear from Fig. 21 as well. Fig. 19 indicates that there is no factor of safety against yielding at a moment corresponding to a working stress of 20 ksi. Thus, Connection P is "inadequate" from the point of view of elastic design. As is evident from Fig. 22 and 52 sufficient diagonal stiffening was provided in Connections A, K, L, and M to prevent serious shear deformation. The photographs of the con-
Connections at failure (Figs. 26, 29) show that in spite of the use of diagonal stiffeners, yield due to shear force still occurs at high loads, but with a satisfactory margin of safety for elastic design.

2B. "Yield Lines" and Residual Stress

Coating the connections with whitewash revealed the flaking of mill scale at yield zones. "Yield lines" were observed at loads between 31 and 81% of the calculated initial yield load as indicated on the experimental curves (Figs. 22, 36, 37, 44). This yielding at less than calculated values is usually due to a combination of residual stresses and stress concentrations. For the larger built-up connections (B, N, G, H) the resultant increase in measured rotations or deflections due to formation of these first yield lines is slight indeed (Fig. 44). However, in the remaining connections where the connection length is shorter, then formation of the first yield line may be associated with the commencement of non-linearity of the load-deformation curve (Fig. 22). The consequences of non-linear behavior at loads lower than the predicted yield point are usually not serious in the case of members designed to resist flexural loads. However, when one considers column action, wherein yielding with a corresponding reduction in effective bending stiffness aggravates buckling, the possible seriousness of residual stress becomes important.

All of the connections, a number of which were built up by welding (i.e., B, C, G, H, I) were tested in the as-delivered,
as-welded condition. The observed yield line patterns indicate that welding introduced residual stress patterns somewhat similar to those formed due to cooling after rolling. Fig. 75 indicates schematically the possible distribution of residual stresses in web and flange material at a cross-section through a haunch fabricated by welding. In the tests, yield lines were observed at the edges of the compression flange and at the center of the tension flange.

26 Yield Strength of Connection Types

As in the case of most structural members in bending, the transition from elastic to plastic behavior was very gradual in these connection tests; a well-defined yield point was not observed (Fig. 52). As a consequence, the "initial yield load" or the "yield strength" of the various connections are to be compared, then definite criteria must be adopted. Prior to describing the criteria available, the terminology will be defined:

(a) Moment - position of critical section along the member:

\[ M_h = \text{"Haunch" moment - the moment at intersection of neutral lines of girder and column extended.} \]

Fig. 74

\[ M_r = \text{"Rolled Section" moment - connection moment at junction of rolled beam and knee.} \]

\[ M = \text{Moment at any position.} \]

(b) Theoretical or computed moments; (subscripts h and r have been omitted)

\[ M_i = \text{"theoretical initial yield moment" of the connection for a particular loading condition. (Fig. 74)} \]
(c) Experimentally observed moments: (subscripts h and r omitted)

\[ M_{(1)} = \text{"Yield Line" moment - the moment at which the first yield line is observed (Fig. 74)} \]

\[ M_{(2)} = \text{"Visual Yield" moment - the moment at which the plotted curve becomes non-linear as observed visually.} \]

\[ M_{(3)} = \text{"General Yield" moment (this criterion is defined below).} \]

Some of the available yield strength criteria are summarized in the following paragraphs. Not all of them have been used in this paper.

(a) **Yield Line Moment:** The moment at first yielding, \[ M_{(1)} \], described above is a value recorded during the test, and is determined by careful examination of the test member after each load increment. Fig. 76.

(b) **Visual Yield Moment:** This has also been described above and is designated in Fig. 76 as \[ M_{(2)} \]. This is an approximate method dependent upon the scale to which the curve is plotted and the judgement of the observer, but the moment-rotation curves are all plotted to the same order of magnitude and the results should be of value for comparing one connection with another.
(c) General Yield Moment: \( M_y \) is determined by the graphical method shown in Fig. 76. Originally suggested by one of the authors, (33) it has been termed the "limit of structural usefulness" since it corresponds to a point at which connection deformations would begin to affect structural behavior elsewhere in a continuous frame, and the structure will no longer serve the purpose intended.

(d) Scatter band: After drawing the experimental curve to a large enough scale to indicate the scatter, the yield strength of the structure or member is defined as the intersection with the experimental curve of a line parallel to the elastic part and offset by one-half the scatter band width. This method has been used by others, but has not been employed in this paper. It is one of the most sensitive criteria.

(e) Slope Factor: Another method used by one of the authors (34) involves the drawing of a line tangent to the test curve at a slope one-third the original elastic slope. The point of tangency is a measure of the yield strength. Comparisons using this method have not been made, but the method is indicated in Fig. 77a.

(f) Deformation Increment: Progress Report 5 (35) compares the behavior of continuous beams on the basis of the percent increment of centerline deflection beyond the predicted deflection at the elastic limit moment \( M_{el} \). The scheme is shown in Fig. 77b; it is necessary to compute the initial yield moment, \( M_{y} \), and the deformation increment is read from the curve. In the case of curved and haunched connections, for which deflection computations are complex, the same information may
be obtained by extending the elastic line to the computed initial moment, $M_{(i)}$. The corresponding "theoretical" as well as the experimental rotations may be read from the graph. Included in a table below is a comparison of connection behavior on this basis.

(g) Reduction in Moment: Another criterion previously used in conjunction with (f) above is the percentage reduction in moment at the computed initial yield deflection. The method is also illustrated in Fig. 77-b but has not been used in this paper.

Having described the terminology and discussed the various possible criteria, these latter will now be used to compare the behavior of the various connections with one another and, in some cases, with theoretically predicted values.

In Table 2 the moment at yielding, as defined by the "Yield Line", "Visual Yield", and "General Yield" are criteria compared with the initial yield moment, $M_{h(i)}$. The calculated initial yield moments $M_{h(i)}$ shown in column 4 of Table 2 are computed from the wedge or flexure theory (as suitable) and take into account the influence of axial thrust as well as bending moment. Table 2 also includes in column 11 the results of calculations using the "Deformation Increment" criterion (see above). In two cases (connections B and P) the predicted yield moment was not reached by the connection.
Neglecting connection P, summarizing all the tests, the three yield strength criteria give the results shown in Table 3.

**TABLE 3: YIELD STRENGTH OF CONNECTIONS**

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Ratio</th>
<th>Maximum Ratio</th>
<th>Minimum Ratio</th>
<th>Deviation (Max - Min)</th>
<th>Average of all readings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Line Moment</td>
<td>$\frac{M_h(i)}{M_h(i)}$</td>
<td>.80</td>
<td>.31</td>
<td>.49</td>
<td>.55</td>
</tr>
<tr>
<td>Visual Yield Moment</td>
<td>$\frac{M_h(i)}{M_h(i)}$</td>
<td>1.07</td>
<td>.39</td>
<td>.68</td>
<td>.70</td>
</tr>
<tr>
<td>General Yield Moment</td>
<td>$\frac{M_h(i)}{M_h(i)}$</td>
<td>1.24</td>
<td>.84</td>
<td>.40</td>
<td>1.08</td>
</tr>
</tbody>
</table>

Although the scatter is considerable this shows that:

1. The first yield line was observed on the average at about 50% of the computed initial yield value.
2. At about 70% of the computed moment, the departure of the experimental curve from a straight line could be detected visually.
3. The "General Yield" averaged 8% greater than the computed initial yield.
4. The "General Yield" criterion gives the least scatter and will be the basis of conclusions regarding the yield strength of the connections tested. It requires no preliminary calculations and is readily determined using the experimental curve.

Thus with respect to the predicting of the initial yield (elastic) strength it is concluded that present theories are
adequate for most of the connections. As indicated by column 10 of Table 2 most of the connections have "General Yield" values greater than the calculated initial yield moment, lower ratios being observed in connection Types 2B, 7, and 8B. This indicates that most of the connections did not yield a significant amount until a load was reached greater than the predicted value.

The points on the experimental curves that correspond to the "Yield Line" and the "General Yield" values are indicated in Figs. 19, 22, 36, 37, and 44 by "Y" and "YS" respectively.

According to the Deformation Increment criterion, less scatter was observed in these tests than in the case of recent continuous beam tests. In the latter, the range of Deformation Increment was from 13 to 88%. In these connection tests the range of increase in deflection over the computed value at $M_{h(1)}$ is from 5 to 24%, the average value being 14% neglecting connections B and P which did not develop strengths as high as the initial yield value.

(1) **Straight Knees** (Connections A, K, L, M, P)

The straight knees indicate non-linear behavior at relatively lower loads than the rest of the connections. Since the rotation measurement includes a relatively short length when compared to the larger connections, the results of accumulated local yielding are in evidence at relatively lower moment values.

Examining the "General Yield" ratio (column 10 in Table 2) and noting the construction details, it appears that residual stress and stress-concentrations may affect the results. The
number of stiffeners and hence the amount of welding increases in the order, A-K-L-M. In the case of Connection M, welding the vertical stiffener near the top would introduce tensile residual stresses. As a consequence the application of bending moment causing tension in the outer flange, would result in local yielding at a load lower than predicted. In the experiments the "General Yield" moment ratios (Table 2) increase in the order, M, L, K and A.

Difficulties were experienced with the measurement of rotation in connections A, K, and M in the later stages. This accounts for the heavy dashed curves in Fig. 22.

(2) Tapered Haunch Knees

Connections B, C, and N are the largest of the tapered haunch knees tested in the program. Connection B had lower yield moment ratios and connections C and N showed higher values than the average for all connections. Yield lines in C and N were observed at very nearly the same moment. In Connection C these lines were in the haunch web (Fig. 41) and subsequently widened as shown.

For Connections C and N, yielding in the rolled sections just outside the knee caused non-linear behavior in the moment deformation curves. In the case of Connection B, however, the experimental curves deviated from a straight line almost immediately after the formation of the first yield line. The first yielding occurred at about 41% of the computed initial yield moment and "General Yield" at about 84% of that value. Residual stress probably contributed to this earlier yielding of connection since, due to the size of the assembly there probably were large residual stresses built up due to welding.
Connections D, E, and F were close to the average for all of the connections in their behavior. As in the case of the straight knees, those connections with full-depth stiffeners (D and F) yielded at lower moment values than the one with half-depth stiffeners (Connection E), Fig. 36. This is so because the welding of the half-depth stiffeners would induce relatively low residual stress and would introduce a less severe stress concentration than the full-depth stiffener.

As is indicated in Figs. 38 and 39 the bracket and stiffeners were obviously sufficient to prevent any large-scale yielding in the knee area, forcing it to occur in the rolled section. Referring also to these same two figures and to Fig. 36, the full-depth tapered stiffener as an extension of the column flange is adequate, and the extra expense of welding a full stiffener is not justified. Use of the tapered stiffener does permit some additional yielding but this is insignificant.

(3) Curved Knees

For the curved knees G, H, I and J, Table 2 shows that the first yield line occurs at relatively lower moment than in the other connection types. However, the local yielding did not influence seriously the deformation (note that the "General Yield" ratio was higher than the average, 1.19 compared to 1.08). Due to welding, the curved inner flange should have compressive residual stresses at the edges. The formation of yield lines observed in the tests was consistent with this pattern.

As may be noted in Figs. 15 and 41, many of the built-up connections gave evidence of yielding due to shear force similar to
that causing the unsatisfactory behavior of Connection P. However, such yielding was local in character and did not influence the moment-rotation curves.

(4) **Moment Strength of Rolled Section at Junction with Connection**

The moment $M_r$ at the end of the rolled section is a factor in the design problem of specifying the location of the splice or joint between the haunch and the rolled shape. As a basis for discussion and illustration the Type 2B connection will be used. Fig. 78-a shows a knee proportioned to attach to a particular beam. In Fig. 78-b the initial yield moment capacity is plotted diagramatically as a function of distance from the intersection of the neutral lines of the girder and column extended.

In a frame for which the connection is proportioned, assume that the points of inflection do not move during application of the load. This corresponds to the test condition. If the point of inflection (or load point) were at A, Fig. 78-c, then first yielding would occur when the haunch moment equals $M_h(i)$; but the strength of the beam at section 1-1 would not have been developed. In order for the girder yield strength to be reached, the connection would have to yield and strain-harden until $M_h = M_h(y)$ as shown.

If the point of inflection were moved to position B, Fig. 78-d, then the knee would yield when $M_h = M_h(i)$, yield occurring at a section such as 2-2. Unless a plastic hinge were developed at section 2-2, with subsequent strain-hardening, the theoretical yield strength could not be developed at section 1-1. Only when the point of inflection is at C (Fig. 78-c) is there justification for assuming that the connection will develop the elastic limit strength at the end of the rolled section.
In testing the curved and the tapered haunch knees the length of arm "a" was adjusted so that initial yielding should occur more or less uniformly along the length of the haunch. (This corresponds to position B of Fig. 78-d). A load-point at A was ruled out because, for static load, good elastic design would call for haunch depth to be proportioned according to the moment diagram, the connection being stressed as uniformly as practicable along its length. It seems obvious that if the connection were to develop its yield strength at section 1-1 with the load at position B, it would certainly do the same if the point of inflection were removed to point C or beyond.

The question then to be answered is: Did the various connections develop the initial yield moment \( M_r(i) \) at the junction between the haunch and the rolled section? The results of a comparison on this basis are shown in Table 4 column 7. N is not included because of the unequal flange widths. The data in column 7 is obtained by dividing the observed "General Yield" moment (column 5, defined previously) by the haunch moment at theoretical yielding of the rolled section, column 6. The data in column 6 has also been corrected for variation in material properties between the material used to fabricate the haunches and the 8B13 rolled shape. The variation is shown in Table 5. Two values are shown for connection CJA. This is because the connection is not symmetrical and yielding would occur sooner at one end of the connection than at the other.

The following observations are made with respect to the data in column 7 of Table 4.

(a) Nearly all of the connections developed
"General Yield" strengths greater than a moment corresponding to initial yield at the end of the connection.

(b) The straight knees have lower ratios than most of the remaining connections.

(c) Connection B yielded at a load which was only 78% of that corresponding to rolled section yield, and similar performance is seen for section a-a of Connection C. The latter case is not unexpected; under the symmetrical loading system used, section b-b would be the critical cross-section of the connection.

(d) As a group, the curved knees give the best performance.

In column 8 of Table 4 the connections have been compared neglecting the influence of axial load in the computation of $M_h(y)$. In all but two cases significant yielding occurred at loads lower than the initial yield load in flexure at the junction of rolled beam and connection.
3. STIFFNESS OF CONNECTIONS IN THE ELASTIC AND PLASTIC RANGE

This section will cover the load-deformation aspects of connection behavior, including the moment-rotation and moment-deflection curves. Both the elastic and initial inelastic behavior will be considered.

In Fig. 52 moment at the knee has been plotted against the average unit rotation (total rotation in the knee divided by the equivalent length). These are all experimentally-determined curves. The solid line is the curve determined from the control-beam test (simply-supported beam under third-point loading).

This figure shows that all of the so-called built-in connections exhibit an average stiffness greater than that of the rolled section (8B13) in the elastic region. Only the straight knees (Types 2 and 8B) are less rigid. It has been observed in tests of continuous beams (35) that residual stresses and stress-concentrations may cause an increase in deflections above predicted values in the so-called elastic region; but the increased deformations in the connection tests are somewhat greater than those usually found in beam tests.

The increased rotations in the straight knees will increase the bending moments and deflections elsewhere in any frame of which the knee is a part. Using the 8B13 section in a frame with a column height of 10' and a beam span of 24' loaded at the third-points, calculation has been made of the resultant increase in deflection at the frame centerline. The experimental connection rotation of connection L at the theoretical initial yield moment was used in the analysis. The deflection
was found to be about 10% greater than predicted due to the increase in rotation above the value predicted on the basis of complete continuity.

A. **Straight Knees**

**Connection P, (Type 7)**

In Fig. 19, describing the behavior of connection P, it is evident that the "computed elastic stiffness" considering both shear and flexure, is greater than the measured slope of the experimental curve (solid line). The actual rotations are about 13% greater than predicted according to the assumptions in Part II. The theoretical moment-rotation curves for the 8-in. and 14-in. members are shown by broken lines. On the basis of "equivalent length" and the assumption which is implicitly assumed for continuous connections, the elastic curve should lie between these two curves, and is shown in Fig. 19 as a long dashed line computed according to the theory of Part II. The connection fails by a factor of two to develop elastic stiffness equivalent to that assumed in elastic design; thus elsewhere in a structure of which it were a part stresses would be higher than the computed values.

At a moment of about 600 in.-kips, the connection commences to yield rapidly (due to shear in the knee panel) but at the same time it continues to carry increased bending moment. The increase in load is due to the fact that

(a) there is strain hardening in the web and additional load is required to cause yielding in those parts of the knee removed from the haunch point, and
(b) the flanges provide restraint by bending as shown in Fig. 21.

From the discussion in Part II, it is evident that a calculation could be made to obtain the required thickness of web material such that a Type 7 knee will meet the stiffness requirements. This requirement for knees joining rolled sections of equal depth was that the rotation, \( \varphi_A \), measured over an equivalent length \((dL = d)\) be no greater than that given by the expression,

\[
\varphi_A = \varphi \cdot d \quad \text{.................................................(40)}
\]

where \( \varphi = \frac{M_r}{EI} = \frac{M_h}{EI} \) \((1 - \frac{d}{2L})\). From Equations (16) and (17) of Part II, the sum of the shear deformation, \( \gamma_7 \), and the bending deformation, \( \beta_7 \), is given by

\[
\gamma_7 + \beta_7 = \frac{M_h}{wd^2G} (1 - \frac{d}{L}) + \frac{M_h}{2EI} (1 - \frac{d}{2L}) d \quad \text{...............(41)}
\]

The required web thickness is obtained by equating expressions 40 and 41. Further, neglecting the influence of the terms \((1 - \frac{d}{L})\) and \((1 - \frac{d}{2L})\),

\[
\frac{M_h}{EI} d = \frac{M_h}{wd^2G} + \frac{M_h}{2EI} d \quad \text{...............(41)}
\]

Replacing \( I_F \) by \( I \) (an assumption on the unsafe side by a small amount),

\[
\frac{d}{EI} = \frac{1}{wd^2G} + \frac{d}{2EI} \quad \text{...............(41)}
\]

Then the required web thickness, \( w \), is given by

\[
w = \frac{2EI}{d^3G} = \frac{ES}{d^2G}
\]
This expression gives the minimum web thickness, \( w \), to provide adequate stiffness in the elastic region.

By comparing measured rotations with computed values for Connection P it has been possible to make one check on equation 42, although it was necessary to change the terms somewhat due to difference in depth of shape. Agreement within 10% was observed.

**Connection A (Type 2)**

The moment-rotation curve for Connection A is shown in Fig. 22. The two theoretical curves in the elastic range are for the two different assumptions made in Part II of the paper. The dot-dash computed elastic line takes into account both shear and flexure; the dotted "equivalent length" line considers only flexure of an equivalent length of rolled beam. It is evident from the rotation calculations of Part II that both the deformation due to shear and flexure must be considered when computing knee rotations.

Below 160 in-kips test connection A deformed only 9% above the predicted value. Larger than predicted rotations would be expected due to the absence of stiffeners to transmit the reaction at the reentrant corner of the connection. The measured rotation is 38% greater than the theoretical value at a moment of 400 in-kips. It was noted previously that rotation measurements for Connection A were unreliable in the higher ranges.
Connections K, L, M (Type BB)

Using Connection L as the basis for comparison, the experimental curve indicates rotations only about $1\%$ greater than that predicted considering both shear and flexure (dot-dash line in Fig. 22). Thus excellent agreement in the elastic region was obtained.

The simplest computation of rotation, that based on "equivalent length" as developed in Part II, predicts rotations that are about $14\%$ less than the observed values. Thus this connection type does not have sufficient effective web thickness to prevent elastic rotations greater than those assumed by the simple theory. It would be of value to see how well the simple theory (which neglects shear) agrees with tests of other sizes and shapes of cross-section.

In Fig. 22 in the inelastic region, two "theoretical" moment-rotation curves are drawn, one for uniform moment throughout the equivalent length of the knee, the other based on the assumption that no inelastic rotation occurs within the knee (See Part II). The first assumption (dotted curve) provides the best agreement with experiment over the greatest range. At the end of the elastic portion and in the initial plastic range experimental values are greater than predicted by theory. Factors which contribute to this are residual stresses, stress concentrations, and a certain amount of yielding due to shear force at the higher moments.
Deflections

Fig. 23 shows the deflection curves of the 4 straight knees (A,K,L,M). The theoretical curve (dashed line) is based on assumptions outlined in Part II resulting in Equations 33-35. The experimental deflections are greater than predicted by the theory used since the latter is based on so-called minimum requirements that do not take shear deformation into account. This deflection computation assumes uniform moment over an equivalent length of knee. A more exact comparison with the deflection curve in the elastic region could undoubtedly be obtained by the use of the "exact" predicted rotation developed in Part II.

Due to stress concentrations, residual stress, and plastic deformation due to shear force the experimental curves become non-linear at relatively low loads. However, the increase in deflection does not become uncontrolled until a load is reached corresponding approximately to the plastic hinge moment at the end of the rolled section.

Fig. 23 also shows the theoretical curve in the strain-hardening region. Connection L was provided with the best lateral support and the agreement between theory and experiment is good.
B Tapered Knees

A theoretical analysis of the tapered and curved knees is not included in this report. A few comments on stiffness of the two types in the elastic and plastic range will be made. The use of any of the tapered knees tested in this program as part of a frame would assure a continuity at least as great as that implied in the assumption of complete continuity in straight knees, (Fig. 52) Neglecting connections N and D, the measured stiffness of the connections fall within a narrow band more than twice the stiffness of the rolled section.

As shown in Fig. 36 there is a difference in behavior of Connection E when compared with D and F. The latter with the full-depth stiffeners at the bracket ends show increased deformations in the initial plastic range as compared with Connection E which has half-depth stiffeners. It is suggested that this is due to residual stresses induced due to welding of the full-depth stiffeners (D & F) in the vicinity of the tension flange as has been discussed earlier. The half-depth stiffener thus appears to have a slight advantage.

A rough comparison of the rotational stiffness of tapered knees in the elastic range indicates the sequence N - B - C - F, N being the most rigid (Fig. 52). D and E are not listed because of the difficulty with rotation measurement. N is stiffer than the others because of the wider plate acting as "tension" flange. The sequence mentioned is more or less in accordance with the minimum distances measured between the external corner and the inner flange which are as follows:
Connection F is so short that, proportionately, a greater length is under higher moment and this results in increased deformation.

The comparison mentioned above suggests that there may be a rough correlation (based on the simplest of assumptions) between average unit rotation and minimum distance, \( d_h \), measured from external corner to the inner flange at the haunch, (Fig. 79). The average unit rotation, \( \varphi \), of a beam under constant moment and with constant modulus of elasticity varies inversely as \( I \). If the web is neglected, then \( I \) varies as \( d^2 \) and,

\[
\frac{\varphi_1}{\varphi_2} = \frac{I_2}{I_1} = \frac{d_h^2}{d_1^2}
\]

where

\( \varphi_1 \) = average unit rotation of beam with depth \( d' \)

\( (d' = 1.414d, \text{ Fig. 79} ) \)

\( \varphi_2 \) = average unit rotation of beam with depth \( d_h \)

\( (\text{Fig. 79}) \)

Comparing connection B with the rolled section curve, the computed ratio of stiffness is

\[
\frac{\varphi_1}{\varphi_2} = 2.22
\]
Examining Fig. 52 at M = 270 in-kips, the observed unit rotation \( \phi_a \) of rolled beam of depth \( d \) is

\[ \phi_a = .0002 \text{ rad.} \]

Then \( \phi_b \), the computed rotation of connection B should be

\[ \phi_b = \frac{\phi_2}{\phi_1} \phi_a = .00009 \text{ rad.} \]

The experimental average unit rotation for connections B, C, and F from Fig. 52 is

\[ \phi_b = .000075 \text{ rad.} \]

This discrepancy between computation and experiment is 17%.

(Taking the influence of the web into account the difference between experimental results and calculated value is about 5%). Thus, the stiffness of haunched connections compared to the rolled section varies roughly as the square of the minimum distance measured between the external corner and the inner flange. Such a relationship is a convenience when it is desired to compute frame reactions and "center moments" more precisely when haunched knees are used. Over the length of the knee, the stiffness can be modified by a constant factor arrived at by the above comparison. Where the length of haunch is greater, more precise methods may be used. See for example, Griffiths (1).

C. Curved Knees

Connection J was intended for comparison with connections D, E, and F. The curves in Fig. 36 show that the elastic stiffness of all four connections is almost identical. The stiffness of curved connections as indicated by total rotation measurements (not shown in the figures) is in decreasing sequence I, H, and G.
The deflection measurements (Fig. 44) show similar behavior since the lengths of the rolled sections vary about the same as the total length of test specimen. This behavior is expected since the connections with larger equivalent lengths and thinner curved flanges would be expected to give larger total rotations. If, however, the total rotation is divided by the equivalent length to give the average unit rotations, connections G, H, and I have very nearly the same elastic stiffness, G being somewhat more rigid. Connection J is considerably more flexible than the other curved knees as indicated by average unit rotation (Fig. 52). The three connections G, H, and I, proportioned approximately according to the AISC procedures, (1) have a stiffness averaging about three times that of the rolled section.

The correlation between stiffness and haunch depth, \( d_h \), suggested earlier for tapered knees is fairly good for the curved knees. Assuming that \( I \) varies as \( d_h^2 \), at constant moment \( M = 270 \) in-kips the measured and computed values agree within 10% as shown in Table 6. Only the two connections with equal thickness of inner and outer flanges are included.

### Table 6

**Comparison of Connection Stiffness on the Basis of Haunch Depth**

<table>
<thead>
<tr>
<th>Connection</th>
<th>Average Unit Rotations, ( \phi ), in Radians</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observed</td>
<td>Computed</td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>( 9.6 \times 10^{-5} )</td>
<td>6.2</td>
</tr>
<tr>
<td>G</td>
<td>( 4.9 \times 10^{-5} )</td>
<td>8.9</td>
</tr>
</tbody>
</table>
Thus, for the curved and tapered knees with equal thickness of inner and outer flanges, the average unit rotation, $\phi_h$, may be determined approximately by the expression

$$\phi_h = \frac{2d^2}{d_h^2} \cdot \frac{M_h}{EI} \cdots \cdots \cdots \cdots \cdots (43)$$

where

- $d$ = depth of rolled beam
- $d_h$ = minimum distance from external corner to inner flange
- $M_h$ = haunch moment.

It is emphasized that the above expression is developed from the grossest of assumptions and may be coincidental to these tests.

4. PLASTIC STRENGTH AND LOAD-DEFORMATION CHARACTERISTICS BEYOND THE MAXIMUM LOAD

4A. Introduction

In Part I of this paper the requirements for connections were discussed from the point of view of plastic design. Three such requirements were noted as follows:

1. Straight connections must be capable of resisting at the corner the full plastic moment, $M_p$, of the rolled sections joined.

2. For straight knees the stiffness (or "rigidity") should be at least as great as that of an equivalent length of the rolled sections joined.

3. The connection may be required to absorb further rotations at near-maximum moments after reaching the plastic hinge condition. This property has been termed "rotation capacity".
No special requirement was listed for haunched knees since it is not clear that plastic design is suited to frames with built-up connections. To give adequate factor of safety and to justify assumptions in analysis, requirements (1) and (2) above are also desirable properties for elastic design.

The stiffness of connections has been discussed in the previous section. If deflection is not a matter of great concern, increased flexibility could be allowed in the elastic range so long as the connection eventually developed the required strength. A special examination of a frame might be required in this case to make certain that at the last plastic hinge point there was adequate rotation capacity to counteract the influence of large deformations at a connection. On the other hand, if deflection is critical in the design, (as may well be the case) then rotations beyond those implicitly assumed in design computations could have an adverse effect.

The observed behavior of all the connections follows the same general pattern. The initial elastic range is followed by the elastic-plastic stage (an initial plastic region) in which the rotations and deflections gradually become larger for equal increments of load. After a gradual increase in the amount of yielding, local plastic instability of the compression flanges is observed, tending to cause the knee to buckle in a direction normal to the plane of the knee. With increasing loads the local buckling becomes more pronounced, and the connection collapses soon after plastic buckling occurs in the web.

In Table 7 the maximum plastic moments are compared with various computed moments for the different connections. As

* The problem of plastic design and deformation has been discussed in Progress Report No. 5(3)
this table is studied with Fig. 74 which describes the terminology, it will be seen that each criterion of strength (plastic hinge at the haunch, initial yield, rolled section yield, and plastic hinge at the splice) is successively more severe. In considering the plastic strengths, then, these four criteria form a basis for the discussion.

(a) How does the observed maximum haunch moment $M_h(a)$ compare with the plastic hinge moment of the rolled section, $M_h(a)$?

Column 5 of Table 7 shows that all of the connections except Connection P will develop the predicted plastic hinge moment $M_h(a)$ at the intersection of the neutral lines of column and girder. It is rather obvious that built-up connections would be more than adequate in this respect since so much additional material is supplied at the haunch. It would certainly be wasteful of material if haunches were used only to assure the development of plastic hinge strength.

(b) How does the maximum observed haunch moment compare with the computed initial yield moment, $M_h(i)$?

This important general comparison, suggested by Mr. T. R. Higgins, is shown in column 7 of Table 7. Excepting Connection P (whose deficiencies have already been discussed) and Connection B, the knees exhibited a reserve strength beyond the elastic limit greater than that of a simply-supported beam under pure bending. The ratios range from 1.21 to 1.47; for the pure bending of a beam, the ratio of the maximum load carried compared to the load at which the test member is computed to yield (this ratio is called the "shape factor") ranges from 1.12 to 1.20 and for the 8B13 shape is 1.15.
(c) How does the maximum observed haunch moment compare with the predicted yield moment of the rolled section, $M_{h(y)}$?

The results of this comparison are shown in column 9 of Table 7. Although it has been seen in an earlier part of this paper that all of the connections yielded at moments less than a value equivalent to the yield moment at the splice, all of the connections eventually developed strengths greater than this value except Connection B and one arm of Connection C.

The behavior of Connection B requires particular attention, since it is rather commonly used. In Fig. 81 the initial yield moment capacity of the connection at each cross section is given by line "a". The load was at position S and thus the theoretical moment distribution corresponding to the initial yield condition at the splice, $M_{r(y)}$, is shown by line "b". The actual moment diagram at collapse is given by line "c", where $M_{h(4)}/M_{h(y)} = 0.94$. According to these results, if a knee of this type had been designed rather closely to the moment diagram, a re-design would be necessary, extending the haunch splice point towards the center of the frame until the moment at the end of the rolled section was only 94% of the yield moment. An alternate is to design the haunch with a thicker inner flange plate.

Two other factors probably influenced the behavior of Connection B: the method of lateral support and residual stress. As will be remembered, Connection B was the first of the larger built-up knees to be tested and the vertical guide system of lateral support was used (Fig. 15). This scheme lacks the stiffness inherent in the direct tie bars used on the later curved knees (Fig. 16). Fig. 80-b shows the lateral deformation that
occurred normal to the plane of the knee after the connection had yielded, indicating insufficient support stiffness. Further, the tendency toward lateral buckling will be greatest in this connection type. Because of the loading and shape of knee a considerable length of it yields at the same load. On the other hand, in the curved knees yielding is concentrated in one area, (compare Fig. 40 with 49 for an example). Buckling of the inner compression flange should occur when the flange becomes plastic. The longer the length of haunch that is yielded, the more severe will be the buckling tendency.

The second factor influencing the behavior of Connection B is residual stress. Knees of this type, completely built-up by welding, contain residual stresses which are probably of similar form to those induced when a rolled shape cools after rolling. Tensile stresses would be formed along the line of the weld and compressive stresses would be present at the flange edges. When the inner compression flange yields due to the residual stress, then its buckling strength is reduced and lateral deformation such as that shown in Fig. 80-b would occur. More studies of the Type 2B connection are planned.

(d) How does the maximum observed rolled section $M_{cp}(4)$ compare with the computed plastic hinge moment at that point, $M_{r(p)}$? This comparison is shown in line 11 of Table 7.

The straight knees (Types 2 and 8B) and the connections with the 45° haunch are satisfactory. The Type 7 straight knee does not quite develop the yield strength (although it has good rotation capacity), and one of the curved knees and the tapered Connection B are unable to deform plastically and still maintain their cross-sectional form sufficiently to develop the rolled beam hinge moment at the splice. Thus, it is not possible to arbitrarily extend the plastic design method to such an extent
that the position of the splice between beam and connection is
selected on the basis of the predicted plastic hinge moment at
that point. If such a procedure were to be followed, sufficient
thickness of material must be placed in the inner flanges
so that yielding will occur in the rolled section at the splice
and not within the knee itself.

As noted earlier, it is not clear at this time that plastic
design is suited to frames with haunched connections, although
there are certain possibilities which will be mentioned later on.

Before discussing in detail the strength of the various con-
nections the problems of plastic instability and lateral support
and of rotation capacity will be mentioned.

4B Plastic Instability and Lateral Support

Commercially available cross-sectional shapes such as those
used in this program are proportioned in such a manner that local
buckling of flange or of web elements does not occur in the
elastic range. Similarly the members and complete connections
were of short enough lengths that elastic lateral buckling was
not possible.

However, once the elastic limit is passed (and residual
stresses may cause this situation at lower than expected loads)
both local and lateral buckling follow since, in the yielded area,
the tangent modulus is reduced from about 30,000,000 psi to a
value approaching zero. The fact that immediate collapse does
not occur may be attributed to the yielding process for struc-
tural steel. The subject of local inelastic instability is
being currently studied at Lehigh University.

It is emphasized, however, that plastic instability was in-
volved in the collapse of every connection tested, and in most
cases brought about final collapse. Similar connections joining
members of other WF shapes would be expected to perform at least as well since the light 8B13 section has low resistance to local instability in the inelastic range of stress.

The onset of yielding results in marked reduction in stiffness; this of course necessitates the use of lateral support in a test program. The three methods used for providing lateral support were described in Part I of this paper. The results of these tests, together with others, indicate that for members with cross-sectional form similar to the 8B13 shape, the flex-bar support system is much better than the vertical guide system. The latter introduces friction when the tendency to lateral deformation begins. Furthermore, the vertical guides, being more flexible, allow more lateral deflection which in turn further aggravates the buckling tendency. Local and lateral inelastic buckling start at very nearly the same time. For straight knees this may be seen by reference to Figs. 30 and 31.

The improved support provided by the flex-bars as against the vertical guide system is clear from a comparison of the final buckled shape of Connections B and G. Lateral support on G (flex-bars) was adequate to cause collapse to occur in an S-shaped pattern (Fig. 46); Connection B however, buckled in a single half-wave, Fig. 80. Fig. 52 shows that the result of improving the lateral support is to increase the rotation capacity (compare D and E with F, and compare A and K with L and M).

It was confirmed in this investigation that a rather small force is required initially to prevent sidewise deformation. However, when the flange elements buckle locally, tending to cause lateral buckling, then this force increases rapidly,
Bracing at the knee for connections of this type would be adequate in the plastic range if it could carry about 10% of the total thrust on the knee and were tied to a rigid supporting structure.

Connections M and L, having about the same strengths, provide further opportunity for examining the influence of lateral support. Whereas M commences to lose the ability to carry load after a deflection of about 0.7" (Fig. 23), L continues to carry increased load. The importance of adequate lateral support is evident since Connection L has less stiffening than Connection M. Comparing Connections F and D, Fig. 36, the latter has the most effective web stiffening, but F develops greater strength. Effective lateral support is the only explanation, particularly since the load-deflection curves are identical in the elastic and early plastic region.

It is concluded that effective lateral support is more significant than variation in fabrication details insofar as plastic strength of connections is concerned. Although, for the two connection types compared in the above paragraph, inferior lateral support did not prevent the members from reaching the predicted load, the more positive support in each case increased carrying capacity and in particular, the rotation capacity.

To be most effective it is evident that lateral support must be provided as close to the expected point of yielding as possible. Referring to Fig. 52, the built-up knees may be compared as a group with the straight knees (A, K, L, M). In the latter the point of lateral support, which is the reentrant corner,
coincides with the point at which flange yielding will first occur. Support is thus provided at the location at which it is needed the most. However in the haunched knees (B, G, H, I, J) lateral support cannot be provided along the whole length of inner flange. In the tests local buckling occurs at locations remote from the point of lateral support and as a consequence collapse is relatively rapid. This behavior is also seen within the group of curved knees themselves. Connections I and J have the shortest effective lengths and possess better plastic rotation characteristics than connections G and H which buckle more rapidly. In haunched connections it would appear that lateral support should be provided at the end of the haunches (splice points) and also at the mid-length of the inner flange. Use of a channel shape formed to fit the inner flange has been suggested to improve resistance to lateral buckling.

4C Local Buckling and Rotation Capacity

The moment at which visible local buckling of flange elements occurred in each of the connections has been shown in Fig. 52. The designation of two observations for Connection N corresponds, first, to buckling of the column flange and, second, to local deformation of the beam flange. In the straight knees (A, K, L, and M) the moment at first observed local buckling was nearly identical in all the tests. (Fig. 22 shows this data to a larger scale). Local buckling occurred at an average unit rotation of about 0.00033 radians per inch in curved and haunched knees.
Referring to Fig. 22, local buckling is indicated at two places for Connection L. One is at a moment at 520 in-kips which corresponds to local buckling of the girder flange. At a moment of 588 in-kips the column flange buckled, a higher moment being required due to the extra stiffening supplied to this flange by the end plate. Beyond this point, rotations increased very rapidly although the connection continued to carry load.

These tests show that local buckling is followed almost immediately by a significant increase in deformation per unit of load increment and in some cases by almost immediate collapse. If plastic design is ever to be adopted then it is essential to develop a specification for proper geometric proportions of rolled shapes to prevent premature inelastic buckling. A study of this is included in the current program at Lehigh University mentioned earlier.

In Part I of this report, in the section "Requirements for Connections" the importance of adequate rotation capacity in the case of straight knees was emphasized as follows:

"The knee (straight) may be required to absorb further rotations at near-maximum moments after reaching the plastic hinge condition. This property has been termed 'rotation capacity'. The precise requirement depends on degree of restraint, the loading and the length-depth ratio of the portal beam".

* Plastic local buckling of outstanding flanges is a problem that apparently has not concerned investigators in the British Isles and on the continent of Europe. The reason probably lies in the fact that the I-shape has inherent resistance to local buckling that the WF shape does not possess. Since the latter is not in general use in those areas, the problem has not required serious attention.
A rotation capacity of about eight times the total rotation at initial yield was suggested for a uniformly loaded beam fixed at the ends with a length-depth ratio of 30. For portal frames under vertical loading this constitutes a rather severe requirement since the girder ends are not completely restrained. Indeed, if the loading were such that in the elastic region the center and connection moments were nearly equal, then theoretically no rotation capacity would be required to develop the predicted ultimate load-carrying capacity. In elastic design there is no requirement for a reserve rotation capacity although ductility is desirable as a safeguard against brittle failure. A specified rotation capacity is a requirement peculiar to plastic design.

Local buckling followed by lateral instability is the phenomenon that most directly limits the ability of a connection to rotate under constant moment. If local stability did not occur, then, after reaching the plastic hinge condition, the connection would merely rotate under nearly constant moment, a condition assumed by the simple plastic theory and the plastic design method based upon that theory. On the other hand, the occurrence of the first local buckling does not necessarily result in immediate collapse and loss in moment capacity of a connection. As confirmed by the tests, the two factors of predominant importance are the geometric proportions of the cross-section and the efficiency of the lateral support system. Connections with the deepest haunches and thinnest flanges (G and B) have the poorest rotation capacity (Fig. 52). The rotation capacity of Connections H and I with relatively thick flanges of small radius is better in each case than that of
Connection G. See Table 8.

**TABLE 8**

**RADIUS AND THICKNESS OF CURVED INNER FLANGES**

<table>
<thead>
<tr>
<th>Connection</th>
<th>Radius</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>32&quot;</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>H</td>
<td>22&quot;</td>
<td>3/8&quot;</td>
</tr>
<tr>
<td>I</td>
<td>16&quot;</td>
<td>1/2&quot;</td>
</tr>
</tbody>
</table>

Connection J has good rotation capacity and the intermediate flange stiffeners are probably helpful in this respect. The straight knees with the smallest haunch depth have the best rotation capacity. None of the connections have rotation capacities as large as straight knees L and M.

The efficiency of the lateral support system was the second factor mentioned as governing the rotation behavior of connections after local buckling occurs. In the straight connections the maximum flange stress is at the reentrant corner. As was mentioned earlier this is consequently the region at which local buckling will first occur. This is also the point at which lateral support is provided (this was done most efficiently in the case of Connection L), and it will be seen that local buckling was not followed by immediate lateral buckling and collapse. (Fig. 22). If adequate lateral support is provided at the point at which local buckling is expected, then local buckling is symmetrical about the web as shown in Fig. 82. There is little tendency toward lateral buckling. However, when later plastic deformation occurs at cross-sections removed from the point of lateral support, local buckling commences on one side of the
flange (Fig. 82) inducing lateral motion, the web deforms, and the connection collapses. Connection L is an example of this behavior. Fig. 29 shows the pattern of local buckling on one side of the connection. Referring also to Figs. 82 and 22, symmetrical local buckling in the girder flange was observed at a moment of 520 in-kips and in the column flanges at a moment of 588 in-kips. Unsymmetrical local buckling did not occur until the moment reached 616 in-kips at a rotation greater than the maximum shown in Fig. 22. This was near the maximum moment and collapse followed shortly thereafter. As was stated earlier, it is not feasible to provide lateral support in built-up connections at every point at which local buckling is to be expected. Thus the local buckling is not symmetrical in the haunched connections and they are unable to maintain their maximum moment strength through further rotations.

Rotation capacity is improved by the use of stiffeners which maintain the cross-sectional shape and prevent deformation of the web. Connections A, K, and M were all tested with the vertical guide lateral support system. Connection M exhibited good rotation capacity; although a "collapse curve" was not determined for Connections A and K, it was observed that the load was dropping off rapidly. Of these three connections, only M was supplied with a stiffener in way of the column flange, so that the advantage of such a stiffener in improving the rotation capacity is evident. Fig. 26 shows the manner in which yielding and web crippling tend to occur directly opposite the flange extensions. Thus the effect of vertical stiffeners is to prevent local web buckling directly over the flange, the cross-sectional
shape being maintained, increasing the rotation capacity. The use of numerous stiffeners normal to the flange accounts at least in part for the ability of Connection J to maintain its moment strength through a considerable rotation after the elastic limit was passed (Fig. 52).

In summary, then, lateral support should be provided where local buckling is expected, the latter occurring at points of maximum stress. The provision of stiffeners normal to the compression flange at points where buckling is expected will assist in improving rotation capacity.

The collapse of the tapered and curved haunched connections after reaching the maximum moment is more sudden than the rest of the connections. This need not constitute a difficulty since such knees are a product of one of the basic principles of elastic design, i.e., provide material as nearly as possible in proportion to the expected bending moments. As mentioned in Progress Report 3, whenever a structure can be efficiently designed by elastic methods (where depth of section may be varied to suit the moment diagram) there is doubtful advantage of plastic methods.

However, if the built-up connections were sufficiently stable after the maximum moment were reached, it is conceivable that one might make use of this behavior in design. A frame might be proportioned for one loading condition, and the plastic behavior at the connections might result in a structure just as safe for some other distribution of load. As a result appreciable savings of material might be realized. Suppose, for example, that two critical conditions of loading on a portal frame result in the moment diagrams of Fig. 84-a and 84-b. The sections BCD
are reproduced to a larger scale in Figs. 84-c and d respectively. According to elastic design procedure the frame would be proportioned on the basis of a rolled shape adequate for axial thrust plus a bending moment of magnitude \( M_r \). The haunch would extend into the frame a sufficient distance to make \( M_3 = M_4 \). In the example, this would require a haunch extending about 6\( \frac{2}{3} \) feet along the girder (distance \( Y \) in Fig. 84). If, now, the haunch had adequate rotation capacity it might be shortened in length, the moment diagram of Fig. 84-c being used as the basis for determining the distance \( X \) such that \( M_2 \leq M_4 \). Providing that

\[
M_4 + M_5 \leq M_1 + M_2,
\]

then the frame proportioned for the moment diagram of Fig. 84-a would carry loads which result in the diagram of 84-b. There would be a resultant savings of material and fabrication cost due to the shortened haunch length which amounts to about three feet in this example. In the above expression, if \( (M_4 + M_5) \) is greater than \( (M_1 + M_2) \) then the deficiency could be made up by selecting a slightly heavier rolled shape or selecting the haunch length, \( X \), such that an equality is realized.

Further illustration of the possible use to be made of plastic behavior is in the case of foundation settlement. Consider the two-span frame shown in Fig. 86 loaded uniformly. According to elastic design, haunches would extend to positions "a" and "b". In the event of settlement of the center support, a redistribution of moment would occur similar to that shown in Fig. b. There would be overstress at the haunch at B. However, if this haunch had adequate rotation capacity, then the
frame would not be in danger; under the "full load"* there would merely be a further redistribution of motion that would tend to increase again the moment at C. Similar behavior would be observed in case the foundation at A were the one to settle.

If haunches in multiple span single-story frames could be demonstrated to develop plastic hinges, this would be of further advantage since design procedures might thereby be simplified. Such a procedure could take the form of an almost arbitrary proportion of haunch length for architectural considerations with the rolled shape selected to carry the remaining simple span moment diagram (c - e = d in Fig. 86).

If rotation capacity were required at a haunch, it seems that the most economical approach would involve making the haunch sufficiently strong so that yielding would first occur at the splice point where adequate plastic characteristics can be more easily assured.

Among the factors that would require special attention are the following:

(a) Rotation capacity of the haunch. It would be necessary to specify the required amount of rotation capacity. A study would be necessary to indicate how it is to be obtained.

(b) Deflection limitations. The additional rotation at the plastic hinge would mean additional deflection elsewhere in the frame.

* Working load multiplied by factor of safety.
(c) Economic considerations. Additional expense would be involved to assure adequate rotation capacity. This would have to be offset by savings resulting from the shorter haunch length and/or from the simpler design approach.

Even if a haunched knee had no rotation capacity, the plastic behavior of the girder would still allow the frame designed for the moment diagram shown in Fig. 85-a to carry the moments of Fig. 85-b (assuming that other possible limitations to plastic design were met). A plastic "hinge" would form in the girder at point B, after which the haunch moments would increase to $M_1$. It would be required that $(M_3 + M_4) \leq (M_1 + M_2)$ and that the beam have adequate rotation capacity. Thus the extra material required by $M_3 > M_2$ might be saved.

4D Summary of Plastic Behavior of Various Connection Types

(i) Straight Knees (A, K, L, M, P)

The four straight knees A, K, L & M show an adequate reserve of strength beyond the elastic limit as shown by the comparison in column 7 of Table 7. Although A and K are slightly deficient, they also develop the full plastic strength of the rolled section at the end of the knee (column 11, Table 7).

Connections L and M have adequate rotation capacity due in part to improved lateral support and to the stiffening of the girder web in the case of Connection M. For some applications the indicated rotation capacity of connections A and K might be insufficient.
Connection P develops neither the required strength, stiffness nor rotation capacity.** This is due to the insufficient web thickness, although in the final stages of the test local buckling occurred together with the formation of a fracture. Both of these phenomena were probably factors leading to collapse. The test was stopped as soon as it had been established that the load had passed the maximum.

Although the fracture did not lead to serious trouble, the test of Connection P indicates a detail which should be avoided. Referring to Fig. 24 the design requires that the tension load in the column flange be transferred to the girder web. Since in the vicinity of the cut-away portion of the stiffener only the web itself is available to transmit a proportion of this flange load, there is a stress concentration which led to the failure*. Therefore, use of the end plate detail as in Type 8B connections is to be preferred, accomplishing a more even transfer of load and in addition providing further economy.

(2) TAPERED HAUNCHES

Connections D, E, and F have an adequate reserve of strength beyond the elastic limit (column 7 of Table 7) and develop the

* The beam end was rough-finished, creating additional stress concentrations. However, the fracture did not occur until considerable local necking had taken place.

** Depending on the proportions of a frame of which Connection P was a part and upon the deflection limitation, Connection P might be considered adequate if it were the first plastic hinge to form.
full beam strength at the end of the rolled section (column 11). The improved behavior of F over D (the latter possesses the greater stiffening) is attributed to more effective lateral support which also probably accounts for the improved rotation capacity of Connection F. If such connections were to be used in lieu of straight knees then lateral support should be provided at the ends of the haunch if good rotation capacity is to be obtained, rather than at the center of the compressive flange as was done in these tests.

Connections C and N, as tested under symmetrical loading, developed adequate plastic strength at one end of the haunch and not at the other. This is to be expected since one end is subjected to more severe bending moment than the other. If lateral support were provided at both ends of the haunch, improved behavior might be expected.

Connection B does not develop either the yield strength or the plastic strength of the beam at the end of the haunch. Neither does it develop the theoretical initial yield strength, and the reasons for this have been discussed earlier. Under the loading used, the flanges yield simultaneously throughout their length. Just as in the case of the control beam under uniform moment (Fig. 68) the connection collapses rapidly after such yielding. On the other hand, when the same member has a steep moment gradient as in the case of straight knees (A,K,L,M) it has no difficulty in carrying a moment that is even greater than the maximum control beam moment.
Probably the simplest means of improving the design of this connection is to increase the inner flange thickness. With this improvement it would be an effective connection for design use; it is not desirable to allow a connection with the proportion of Connection B to yield at the "full load".

(3) CURVED KNEES

Of all the knees tested, the curved connections show the greatest reserve strength beyond the elastic limit (column 7, Table 7). The three connections, G, H, and I develop the yield strength of the rolled section at the splice and also develop there the plastic hinge strength of the rolled section (column 11, Table 7). The procedures recommended by the AISC for the design of curved knees thus appear to have additional merit in the plastic range of stress. The rotation capacity improves with decrease in radius of curvature and increase of flange thickness. In fact, at a slight decrease in moment, Connection I satisfies the requirements mentioned earlier for rotation capacity of straight knees.

Prior to collapse connection J develops the yield strength of the rolled section but just fails to develop the full strength of the rolled section. Thus a system of vertical stiffening appears to be somewhat less effective than increased flange thickness as specified by the AISC rules. Compared with Connections D, E, and F, both the strength and rotation capacity of Connection J are greater.
5. **ECONOMY**

The bending moment diagram for a rigid frame usually falls off sharply from the center of the knee; the greater the length of haunch the greater the reduction. Thus the splice moment may be materially less than the haunch moment. As a consequence, the members joined by a haunched knee may be considerably lighter in weight than those required for a straight knee. Therefore, the greater the length of knee the greater the economy of rolled beams.

An increase in haunch length, however, is accompanied by an increase in cost of fabrication. Since the selection of the members of a frame is dependent on the moment diagrams, which vary for each span, height, and loading condition, no general comparison can be made in this paper. Attention is called, however, to the relation between knee lengths and the relative time of cutting and welding as shown in Fig. 83. Except for Connections N and J, it is evident that fabrication time (exclusive of handling) increases in almost direct proportion to the total length of the connection. This length is measured along the neutral lines from the haunch point to the splice between the connection and the rolled shape.

Examining the straight knees as a group it is evident from Fig. 10 that Connection A is the most economical. However an additional factor to be considered is that special plant equipment might be required to make the 45° cut in large quantity production. Comparing Connections K, L, and M, it is not evident from the tests that the increased cost of inserting
vertical stiffeners is warranted unless the full plastic strength is to be developed and unless rotation capacity is specifically required. If this is the case then the use of vertical stiffeners is essential. The difference in cost between L and M is a measure of scatter since L should actually involve less welding time.

Examining Connections D, E, and F, the use of half-depth stiffeners indicates an advantage over those of full-depth. From a strength point of view there is little to separate the two designs so that some real economy might be realized by the use of half-depth stiffeners if large numbers of connections were involved. Providing F and D do not differ because of experimental scatter, Fig 10, it is somewhat more expensive to bevel the "inner" stiffener, even though this eliminates a line of welding as compared with D. Undoubtedly, however, if the sniped plates were prepared in mass production, Connection F would be more economical since a special fitting operation is required for the vertical inner stiffener used in Connections D and E. Thus on a cost basis E is to be preferred over F and F over D.

A comparison may be made between Connections B and H. Both are about the same length (Fig. 23). B is about 25% more expensive, although this ratio would be reduced by the extra cost of material for H and the expense of rolling the curved inner flange. As is seen from Fig. 52 and the comparisons contained in earlier sections, the load carrying capacity of Connection H is considerably greater than that of B.
Further general or specific comparisons are difficult to make, since economy of rolled section, carrying capacity of haunch, and expense of haunch fabrication must all be considered.

In arriving at an economical design from the overall viewpoint of cost of members and of fabrication of the joint, one might suggest beginning with a trial selection of straight member using straight knees. To be compared with the above trial design is another in which the members are lighter in weight and not capable of carrying the knee moment. The points near the joint where strengthening must begin can be ascertained from the moment diagram, and thus the knee lengths are established and the joint designed. A comparison of costs may now be made which will include cost of members and cost of knee fabrication.

The introduction of curved knees and tapered haunches into a design also allows for a savings in fabricating costs due to the fact that a smaller rolled shape would be handled, joined, and fabricated than would be required in case straight knees were used.

The disproportionate expense of connection J (Fig. 83) does not appear to warrant this form of construction.
6. **FURTHER RESEARCH**

In the interest of shortening this paper a separate note on proposed research has been prepared.\(^{(36)}\) Generally a means of improving straight knee performance is warranted if it can be done at small extra expense. The use of doubler plates in lieu of diagonal stiffeners is a possibility. The influence of size and shape of cross-section requires further examination. In the built-up connections a considerable amount of analytical work remains to be done and further tests are warranted at the present time. For various knee proportions, dependable splice moments must be established.
II: SUMMARY AND CONCLUSIONS

The statements and conclusions which follow are based on the connections tested and the rolled shapes examined.

1: CONNECTION TYPES

(1) TYPE 2 (Connection A)
   (a) By taking into account both shear and moment the experimental elastic moment-rotation curve can be predicted by theory within about 10%.
   (b) The Type 2 connection develops adequate strength but the Type 8B connection is preferable because of cost factors involved in plant equipment and improved performance in the plastic range.
   (c) Although the difference was not marked, this connection takes less time to fabricate than the Type 8B connections.

(2) TYPE 2B (Connection B)
   (a) This knee does not quite develop the moment at the haunch corresponding to the initial yield condition.
   (b) When compared to the other built-up models, the less desirable behavior of this connection under the "worst loading condition" selected is due primarily to the fact that the whole length of the inner compression flange yields simultaneously. Residual stress is a factor since it causes this yielding to occur at a lower load than predicted, further aggravating the tendency toward lateral buckling.
(c) On the basis of this one test, the initial yield load is the true limiting carrying capacity. Alternate possibilities are to extend the haunch a greater distance into the frame, to increase the size of rolled section and shorten the haunch, or to increase the inner flange thickness.

(d) The average unit rotation, $\phi_h$, for the knee may be determined approximately by the expression

$$\phi_h = \frac{2d^2}{d_h^2} \cdot \frac{M_h}{EI}$$

where $d$ = rolled section depth, $d_h$ = haunch depth, and $M_h$ = haunch moment.

(3) **TYPE 4 (Connections D, E, F)**

(a) The best design of these three knees is Connection E. Further improvement could be realized by using a snipped full-depth stiffener as an extension to the inner column flange and by supporting the connection laterally at the ends of the haunch. Additional economy could be realized by replacing the two external stiffeners with an end plate as in the Type 14 Connection, Fig. 4.

(b) With this connection type (which was not tested under a worst loading condition) the connections indicate adequate reserve of strength above the yield point. The rolled section strength is fully developed at the splice.

(c) Lateral support should be provided at the splices if good rotation capacity is to be obtained in the plastic range.
(4) **TYPE 5A (Connections G,H,I,J)**

(a) The curved knees designed according to the AISC specifications (1) (G, H, and I) performed in excellent fashion. Both the initial yield strength and the full plastic strength at the end of the rolled section were developed (Table 7). The reserve strength above the predicted yield load is more than adequate.

(b) The average unit rotation is about one-third that of the rolled beam, and for the same thickness of curved inner flange may be determined within 10% by the expression given in paragraph 2 (d) above.

(c) A system of vertical stiffening, such as that used in J in lieu of increased flange thickness is not to be recommended on the basis of these tests. For its strength and length, Connection J is relatively expensive. One advantage of this system of stiffening is to improve rotation capacity.

(5) **TYPE (7) (Connection P)**

(a) External to the knee, the elastic stress distribution is in accordance with ordinary beam theory except in the local region within a distance from the knee of half the depth of the section.

(b) Within the knee, flange stresses decrease linearly from the critical section at the splice to the external end of the flange. Shear stresses in the web are larger than those at the critical section for bending.
(c) Yielding due to shear force in the knee web occurs at about 50% of the moment corresponding to the flexural yield point. Agreement with theory was within a few percent. The subsequent additional rotation cannot be tolerated in most engineering structures and means must be taken to avoid such failure. Diagonal stiffeners or web doubler plates are recommended. The expression

\[ w = \frac{2S}{d^2} \]

(in which \( S = \) section modulus and \( d = \) girder depth) gives the required web thickness to prevent premature web yielding.* Using this relationship, all WF and I shapes have been examined; it has been found that no WF shape has adequate web thickness, and in the American Standard I series only the heaviest shapes have adequate web thickness.

(d) As tested, this connection type has somewhat inadequate strength characteristics since it falls short of developing the computed yield strength of the weaker of the two members joined.

(e) In the elastic range this connection is more flexible than predicted by the theory developed. Further, it is twice as great as that implied in ordinary deflection calculations on the basis of complete rigidity at the connections. If the web thickness, \( w \), were such that

\[ w = 2.6 \frac{S}{d^2} \]

then, theoretically at least, adequate elastic stiff-

* Further tests designed specifically to check this expression are advisable.
ness is assured.

(f) An end plate is preferable to intermediate stiffeners for transmitting the tension load in the exterior column flange to the web of the girder. Such a scheme is used in Type 8 and 14 Connections, Fig. 4.

(6) TYPE 88 (K L M)

(a) The diagonal stiffening used was adequate to prevent serious yielding due to shear force.

(b) Excellent agreement between theoretical and experimental moment-rotation curves was obtained in the elastic range. This rotation is somewhat greater (14%) than the equivalent length stiffness implied in the ordinary computation of deflections in continuous structures.

(c) Good agreement between theory and experiment was also obtained for inelastic moment-rotation and moment-deflection curves when it was assumed that the moment was uniform over the equivalent length of the knee.

(d) These connections yield at a considerably lower load than predicted, probably due to residual stress and stress concentrations. However the ultimate strengths developed are very close to the plastic hinge moment.

(e) The half-depth stiffener as used in L is preferred, decreasing the small influence of residual stress. It is doubtful that K has adequate rotation capacity, whereas Connections L and M are excellent in this respect.
(7) TYPE 15 (C)

(a) This knee would normally be designed for a non-uniform moment gradient but was tested with an equal moment gradient on each leg. This amounts to examining the connection under a second loading condition. Although it develops the plastic hinge strength at one end, it does not reach the yield or full plastic strength at the other splice. Little improvement in this characteristic could be obtained with a more rigid system of lateral support, since the strength at section b-b (see sketch) was somewhat greater than obtained in Connection L with its efficient lateral support system.

(b) The elastic average unit rotation is given approximately by the expression of paragraph 2 (d) above.

(8) TYPE 16 (N)

The behavior of Connection N is similar to that of Connection C described above.
2. Structural Behavior

(1) YIELD STRENGTH

(a) The following criteria were selected for evaluating the behavior of the various connections:

1. "Yield Line Moment" \( (M_1) \) - the moment at which the first yield line is observed.
2. "Visual Yield Moment" \( (M_2) \) - the moment at which the plotted curve becomes nonlinear as visually observed.
3. "General Yield Moment" \( (M_3) \) - a moment determined by graphical construction to indicate significant inelastic deformation.
4. "Deformation Increment" - the percent increment of deflection beyond the predicted value at the computed elastic limit moment.

(b) In general the first yielding occurred at about 50% of the computed initial yield moment. This yielding was due to residual stress and stress concentrations. At about 70% of the computed moment the departure of the experimental curves from linearity could be detected visually. The "General Yield Moment" was on the average greater than the predicted initial yield moment, indicating a good reserve of strength beyond the elastic limit. Connections B and P were exceptions. The deformation increment at the theoretical elastic limit ranged from 5 to 24%.

(c) Nearly all of the connections developed "General Yield" strengths greater than a moment corresponding to initial yield at the end of the connection.
(2) **ULTIMATE STRENGTH**

(a) Only the Type 7 connection was unable to carry a haunch moment equivalent to the plastic hinge value of the rolled section.

(b) All of the connections except B and P developed a moment-carrying capacity considerably greater than the calculated initial yield moment.

(c) Except for Connection B each of the knees developed a haunch moment greater than that corresponding to the yield strength of the rolled section at the splice, due account being taken of axial thrust.

(d) Some of the haunched connections did not develop the full rolled beam strength at the haunch ends. This does not imply that the connections were necessarily inadequate; frames using such connections would probably be proportioned in such a way that the yield moment would be reached at the critical sections simultaneously.

(3) **STIFFNESS**

(a) Expressions for elastic stiffness of straight knees have been developed which give fair agreement with Type 7 connections and good agreement with Types 2 and 8B, (diagonally stiffened). The development also includes expressions for the rotational stiffness of straight knees of the three types with equal and with unequal depths of rolled shapes meeting at the connection.

(4) **RESIDUAL STRESS**

(a) Residual stress (combined inseparably in some cases with stress concentration effects) accounts for a lowering of
the initial yield strength. In some flexural members the influence is not permanent and disappears after larger rotations are experienced.

(b) Residual stress was possibly a factor in bringing about the early collapse of the Type 2B Connection B. The reason is that the haunch compression flange acts as a restrained column and it appears that residual stress will certainly influence the carrying capacity of compression members, although its influence on some flexural members may be negligible.

5 INELASTIC INSTABILITY

(a) Plastic instability was involved in the collapse of every connection and in most cases brought about final collapse. If plastic design is ever to be adopted, then a specification must be developed for proper geometric proportions of rolled shapes to prevent premature inelastic local buckling.

(b) Deformations increase rapidly once local buckling has occurred. In the case of straight knees in which it is possible to provide lateral support at the most critical point, collapse does not occur when the first local buckling becomes evident, since the buckling is symmetric on both sides of the web. However when a half-wave is formed on one side of the web only, then collapse follows rapidly. In the straight connection L, however, this was well beyond the needed rotation capacity.

(c) The seriousness of local buckling is markedly reduced whenever lateral support can be placed at each point of expected yielding.
(6) **LATERAL SUPPORT**

(a) The flex-bar method of lateral support is to be preferred over the vertical guide system.

(b) It is important to place lateral support at points of expected maximum stress. Whenever this can be done at all such positions, then the seriousness of local buckling may be markedly reduced.

(c) A rather small force is required initially to prevent sidewise deformation, the force increasing rapidly after local buckling.

(d) Compression flange lateral support should be provided at the center of built-up haunches and at the splice points between haunch and beam.

(e) Except for variations in cost, so far as plastic strength is concerned, effective lateral support is more important than the variations in fabrication details.

(7) **ROTATION CAPACITY** (as influencing plastic design)

(a) Depending on the proportions and loading of a frame with straight knees, each connection must have "rotation capacity", the ability to rotate through a considerable unit angle change after the plastic hinge moment has been reached.

(b) In the case of a third-point-loaded beam attached to very stiff columns, a rotation capacity of about eight times the rotation at initial yield is required.

(c) With adequate lateral support the Type SB connections are satisfactory in this respect while the behavior of Type 2 is not.
(d) While some of the built-up knees have fair rotation capacities, most of them collapse very rapidly after first local buckling. This includes those that are well-supported laterally. However, rotation capacity is not required of haunched connections, since they result directly from an application of elastic design principles.

(e) If for some reason rotation capacity is desired, then in some haunched connections it could be obtained by increasing the strength of the knee slightly beyond that required elastically so that yield would occur at the splice—where good plastic characteristics could be assured.

(f) Rotation capacity is dependent on an ability of the knee to resist the tendency to local buckling. Thick flanges and effective lateral support are most helpful.

(8) COST OF FABRICATION

(a) There is an almost linear relation between the time required for cutting and welding and the total length of haunch as measured along the neutral lines. Connections N and J are exceptions.

(b) Sniped, half-depth stiffeners should constitute an economical advantage over those of full-depth, and in the last analysis will always be less expensive than the complete stiffener welded to the web and to both top and bottom flange.
III: ACKNOWLEDGEMENTS

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NOMENCLATURE

A = Area of section
A_w = Area of web
A_s' = Equivalent stiffener area
b = Flange width
b_s = Total width of stiffener
d = Depth of section
d_h = "Haunch depth", the minimum distance from the external corner of the haunch to the inner flange.
w = Web thickness
t = Flange thickness
t_s = Stiffener thickness
I = Moment of inertia of section
I_f = Moment of inertia of the flanges only.
S = Section modulus of beam
L = Distance between the point of inflection and the haunch point in an actual frame
ΔL = Equivalent length of a connection
M_h = "Haunch" moment
M_h(τ) = "Haunch" moment at which yield occurs due to shear force
M_h(σ) = "Haunch" moment at which yield occurs due to flexure
M_r = Moment in a connection at junction of rolled beam and connection
M_p = "Hinge" value; full plastic moment; the ultimate moment that can be reached according to the simple plastic theory.
M_a = "Hinge" value at haunch
M_{(i)} = Initial yield moment
M_{(y)} = Moment at which yield point stress is reached at the end of the rolled section
\[ M_1 = \text{Observed yield line moment} \]
\[ M_2 = \text{Observed visual yield moment} \]
\[ M_3 = \text{Observed general yield moment} \]
\[ M_4 = \text{Observed maximum moment} \]
\[ \sigma_y = \text{Average lower yield point stress} \]
\[ E = \text{Young's modulus of elasticity} \]
\[ G = \text{Shearing modulus of elasticity} \]
\[ \phi = \text{Average unit rotation} \]
\[ \phi_A = \text{Rotation measured over equivalent length of connection} \]
\[ \gamma = \text{Rotation in connection due to shear} \]
\[ \beta = \text{Rotation in connection due to bending} \]
**TERMINOLOGY**

**General yield moment:** The moment at which the deformations due to this moment begin to affect the structural behavior elsewhere in a structure. The value is determined graphically as shown in Fig. 74.

**Haunch depth:** The minimum distance from the external corner to the inner flange.

**Haunch moment:** The moment at the haunch point.

**Haunch point:** The intersection of the neutral lines of girder and column, extended.

**Initial yield moment:** A computed moment at which the nominal maximum stress reaches the yield point, excluding the influence of stress concentrations and residual stresses.

**Luder's line:** Wedge or plane of yielding which forms in mild steel in the vicinity of the yield point.

**Plastic hinge moment:** The ultimate moment that can be reached at a section according to the simple plastic theory.

**Rolled section moment:** The moment at the junction of the rolled beam and the knee. (A or C in Fig. 7)

**Rotation capacity:** The ability of a structural member to rotate under near-constant moment.

**Visual yield moment:** The moment at which the plotted curve becomes non-linear, as observed visually. (see Fig. 74)

**Yield line:** Flaking of mill scale following the formation of Luder's line as revealed by whitewash.

**Yield line moment:** The moment at which the first yield line is observed. (See Fig. 74)

**Yield strength:** The load or moment at which a significant amount of yielding occurs as indicated by available criteria.
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</table>

** L = distance from inflection point to haunch point. H = distance from inflection point to junction of beam and connection. Fig. 78

* Adjusted for variation in material properties between haunch and rolled section.
TABLE 2: The Yield Strength of Connections**

<table>
<thead>
<tr>
<th>Type</th>
<th>Connection</th>
<th>Sketch</th>
<th>Computed Initial Yield Moment $N_h(1)$</th>
<th>Yield Line Moment $N_h(1)$</th>
<th>Visual Yield Moment $N_h(2)$</th>
<th>General Yield Moment $N_h(3)$</th>
<th>Deformation Increment at $N_h(1)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
<td>(7)</td>
<td>(8)</td>
</tr>
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<td>A</td>
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<td>369</td>
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<td>380</td>
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<td>15</td>
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<td>524</td>
<td>402</td>
<td>563</td>
<td>645</td>
<td>1.23</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td></td>
<td>597</td>
<td>477</td>
<td>477</td>
<td>640</td>
<td>1.07</td>
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<tr>
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<td>E</td>
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<td>597</td>
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<td>472</td>
<td>680</td>
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</tr>
<tr>
<td></td>
<td>F</td>
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<td>597</td>
<td>264</td>
<td>276</td>
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<td>H</td>
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<td>I</td>
<td></td>
<td>883</td>
<td>526</td>
<td>516</td>
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<td>1.02</td>
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<td></td>
<td>J</td>
<td></td>
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<td>300</td>
<td>475</td>
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<tr>
<td></td>
<td>L</td>
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<td></td>
<td>M</td>
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<td>N</td>
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<td>577</td>
<td>722</td>
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<tr>
<td>7</td>
<td>P (Shear)</td>
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<td>311</td>
<td>493</td>
<td>637</td>
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<td></td>
<td>P (Flexure)</td>
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<td>403</td>
<td>637</td>
<td>0.54</td>
</tr>
</tbody>
</table>

** See Text and Fig. 74 for more complete description of terminology.

* Connection did not develop $N_h(1)$.
<table>
<thead>
<tr>
<th>Material</th>
<th>Yield Point Stress in Tension, (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB13 Flanges</td>
<td>41.8</td>
</tr>
<tr>
<td>1/4&quot; plate (B,C,G,J)</td>
<td>39.1</td>
</tr>
<tr>
<td>3/8&quot; plate (H, inner flange only)</td>
<td>39.0</td>
</tr>
<tr>
<td>1/2&quot; plate (I, inner flange only)</td>
<td>35.2</td>
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</table>
TABLE 7: The Maximum Plastic Strength of Connections

<table>
<thead>
<tr>
<th>Type</th>
<th>Connection</th>
<th>Sketch</th>
<th>Comparison with Computed Plastic Hinge Moment, $M_{h(a)} = 517^*k$</th>
<th>Comparison with Initial Yield Moment, $M_{h(i)}$</th>
<th>Comparison with Computed Haunch Moment at Yield of the Rolled Section, $M_{h(y)}$</th>
<th>Comparison of Observed Maximum Rolled Section Moment, $M_{r(p)}$</th>
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</thead>
<tbody>
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<td>$M_{h(a)}$</td>
<td>$M_{h(1)}$</td>
<td>$M_{h(y)}$</td>
<td>$M_{h(1)}$</td>
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<tr>
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<td>(6)</td>
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<tr>
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<td>(Flexure)</td>
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</tbody>
</table>

*** Adjusted for variations in material properties between haunch and rolled section.
** See Fig. 74 for terminology.
* $M_{r(p)} = 517^*kips$ for all connections except P, in which $M_{r(p)} = 1190^*kips$.
× This ratio is based on 1190 *kips, the $M_{p}$ of 8 WF 31.
Fig. 52 Connection behavior compared with SB13 rolled section (based on moment at the knee).
Fig. 75. Hypothetical distribution of residual stresses due to welding.

Fig. 74. Explanation of the various designations used in the comparison of connection strength.

Experimental Values

Fig. 76. Four criteria used in this paper for comparing the strength of structural members.

Fig. 77. Additional yield strength criteria.
Fig. 79. Haunch depths.

Fig. 78. Rolled section moment strength of Type 2 B connections.

Fig. 80. Lateral deformation out of knee plane.

Fig. 81. Theoretical and experimental moment curves.

Fig. 82. Symmetrical and unsymmetrical local buckling.
Fig. 83. Fabrication time as a function of haunch length.

Fig. 84. Moment diagrams for two portal frame loading conditions.

Fig. 85. Hypothetical moment diagrams for portal frame girders.

Fig. 86. Hypothetical change in moment diagram due to foundation settlement.