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A. A. Topractsoglou

L. S. Beedle

Welded Continuous Frames and Their Components

Progress Report No. 15

FURTHER STUDIES

OF

WELDED CORNER CONNECTIONS

(Tension Loading)

by

A. A. Toprac
(University of Texas)

Lynn S. Beedle
(Lehigh University)

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1. A B S T R A C T

A series of typical welded corner connections for structural frames were tested in a manner that would simulate a relatively infrequent form of loading - a moment loading that tends to open the connecting arms. The possibility of weld fracture is thus increased.

Does this form of loading constitute a possible limitation on the application of plastic analysis to structural design? The report shows that it does not. With sound welding and suitably-designed joints, the desirable strength, stiffness, and reserve in ductility may be achieved; thus a satisfactory connection for welded rigid frames is assured.

2. INTRODUCTION

The problem being considered in this report concerns the ability of welded knees in rigid frames to meet one of the possible design requirements. If the loading on a structural frame is such that the forces tend to "open" the joint, will the connection exhibit adequate strength, stiffness, and reserve of ductility?

The studies were carried out as part of a program being sponsored at Lehigh University by the American Institute of Steel Construction, Bureau of Ships, Bureau of Yards and Docks, and Welding Research Council. Several previous progress reports (1, 2) have described the results of studies into the strength of welded continuous frames and their components with particular reference to plastic action. A portion of that program has dealt with portal frames of the type shown in Fig. A1. In particular, Ref. 2 has described studies of straight, tapered,

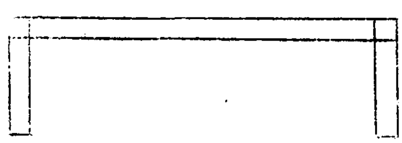


Fig. A1: Portal Frame

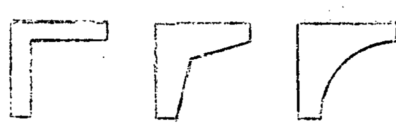
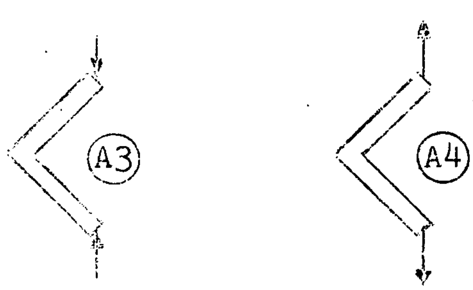


Fig. A2: Connection Types



Compression Loading

Tension Loading

and curved corner connections for these frames (Fig. A2) tested in such a way that the joints closed under the action of the forces. This is termed "compression" loading and is shown in Fig. A3. The present report considers the case in which the forces open the joint (Fig. A4) and is called "tension" loading.

2.1 RIGID FRAME BEHAVIOR

The current studies at Lehigh University, mentioned above, together with the work at Brown University[3] and at Cambridge University in England[4] reflect the present interest in the strength of steel frames loaded beyond the elastic limit. The maximum strength of a steel frame may quickly be determined by a knowledge of the plastic behavior of components. Plastic theory assumes that as the load increases a plastic hinge* forms at the point of maximum moment; as load is further increased, hinges form at other critical sections until the entire structure (or a segment of it) is unable to carry more load due to the formation of plastic hinges at a sufficient number of cross-sections. Application of plastic analysis to structural design results in a balanced design of a more economical frame, proportioned with less design office effort[5].

Further discussion of plastic analysis and design may be found in the references previously cited. The important point to be kept in mind is that the ability of a steel frame to reach its computed maximum load depends upon the ability of the component parts to form plastic hinges. Some of these hinges may be caused by "tension" loading of corner connections.

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* A plastic hinge is characterized by rotation of a cross-section through a considerable angle while the moment remains substantially constant.

2.2 LOADS ON RIGID FRAMES

A study of typical frames and their loading shows that in almost all cases, the typical loading is compression loading (joint being closed). "Tension" loading is highly unlikely in portal frames, and even if it does exist, the ability of the knee to carry high values of this tension loading is not required for the development of maximum strength.

For gravity loads the joint in a portal frame tends to close under increasing loads. Due to lateral load acting alone on a frame, moments of opposite sign exist at the two knees of the frame; one of the knees is under "tension" loading and the other is under compression. However, when the two loading systems are combined, then in most structures the final moments are a high compression at the one knee and a lesser compression or modest tension at the other.

The crown of a sharply-peaked gabled roof is an example of a tension-loaded connection. Heavy structural U-shaped hangers are another example, and of course blast forces could create tension loading in ordinary building structures.

A few specific examples will now be considered. In Fig. 1 a frame is shown with a span equal to twice the column height and loaded with vertical and side load. For Case I in which H is less than V (typical for wind loading), the re-entrant corner at B is in compression. Not until H = V does full tension loading develop there. The formation of a tension hinge is shown at Joint B for Case III (H > V). Thus, the necessary condition for tension hinge action for this frame is H = V, a value for these particular proportions that is ten times greater than that which may be expected for normal combination of wind and roof loading.

The frame of Fig. 2 has, perhaps, a more realistic ratio of span length to column height (3 to 1). The concentrated loads shown at the third points simulate uniform loading. For these proportions and for normal wind and roof loads, the ratio of horizontal load, H, to vertical load, V, is about 1 to 9. The moment diagram is shown in Fig. 2(b) at first yield (dotted) and at the ultimate load (solid). For the latter case the plastic mechanism is as shown in Fig. 2(c). For normal wind load the windward corner is still loaded in compression. A tension hinge will not form until H/V = 1.5, or in other words, until the side load is nearly fifteen times that normally due to wind!

Even for the rather unusual frame proportions shown in Fig. 3, the windward corner is subjected to only a moderate tension. From the bending moment diagrams of this frame it is apparent that the second plastic hinge will occur in the beam away from the knee under "tension".

Although the occurrence of plastic "tension" loading in structures is improbable, a few tests are desirable because of the special cases already mentioned in which the attainment of maximum plastic strength of the structure depends upon proper performance of knees loaded in tension.

2.3 REQUIREMENTS FOR CONNECTIONS

The general requirements imposed on continuous connections by implied or stated assumptions have been discussed previously(2). The basic requirements for connections to resist "tension" are the same:

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

"(b) For straight knees the stiffness (or "rigidity") should be at least as great as that of an equivalent length of the rolled sections joined. Depending upon the proportions of the frame and the deflection limitations, additional flexibility in the knee may not be objectionable so long as requirement (a) is satisfied within the limit established in (c) below.

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

"The above requirements may be summarized for a square knee by stating that it must be as stiff and as strong in bending as an equivalent length of rolled section, with adequate rotation capacity as governed by degree of restraint, loading, and proportions of members."

Tension at the re-entrant corner of the knee imposes a more severe requirement on the performance of the welds at that point because combined stresses in tension are present. Whereas under compression it would be expected that the strength of the knee would be limited by instability, the limit of tension loading might be the onset of weld fracture.

2.4 PURPOSE AND SCOPE

With the above requirements in mind, the objectives of this investigation were the following:

a. To obtain information on the strengths and rigidities of various types of knees when loaded in tension.

b. To compare the results with those obtained in the former compression tests(2).

c. To investigate whether or not weld failure would be a limitation in the design of knees when subjected to tension loadings.

The previous analyses (2) for elastic-limit strength, for deformation in the elastic range, and for plastic moment capacity are as valid for tension loading as they are for compression.

3. DESCRIPTION OF TESTS

The following is a short description of the specimens used, the test set-up, instrumentation, and test procedure.

The knees tested in this program are of Type 2, 2B, 4, 5A, 8B, 15, and 16. Table 1 summarizes the tests conducted. Specimens A to M are the same specimens that were tested in compression as described in Ref. 2 and as shown in Fig. 8 and 9 therein. These specimens were tested in tension, without being straightened after the prior compression tests. Since all the connections had buckled previously and were permanently deformed both locally and laterally, one would expect their behavior to be somewhat different from a prime connection. Specimens V and W (which are identical to specimen K[2] and of type 8B) are prime connections that were tested in compression and tension in order to:

(a) determine any differences between prime and previously strained connections when tested in tension, and,

(b) compare the behavior of identical prime members under compression and tension loads.

Connection V was first tested in compression and then in tension, whereas Connection W was tested in tension only. To distinguish between the compression test and the subsequent tension test of the same connection V, the compression test is labeled as V_c and the tension test as V. The prime tension test is labeled W.

All connections were proportioned to join two 8B13 rolled shapes. Since there were two sets of tests (A-M and V_c, V, and W), there were two lots of material (designated I and II in Table 1). Average section properties for the two lots of 8B13 shapes are shown in Table 2.

The results of the physical property tests are shown in Table 3. These were standard tension tests carried out at a slow laboratory strain rate. Dimensions of the coupons were determined with the aid of a micrometer.

Connections A to M were tested at Lehigh University in an 800,000-lb. screw-type machine. The specimens were set with legs at 45 degrees with the horizontal and loaded through interchangeable loading fixtures bolted at each end of the specimen. The loads were transmitted through a 3-in. pin welded to the loading fixture to a pair of plate links with 3 1/4-in. and 6 1/4-in. holes. A 6-in. pin was secured to each head of the machine to pull the specimens through the links. Fig. 4 is a diagrammatic sketch of the links and the deflection dial. Fig. 5 shows the links and a specimen after testing. No lateral support was provided to the specimens tested at Lehigh.

Identical experimental test set-ups were used for Connections V and W which were tested at the University of Texas in a 400,000-lb. screw-type machine. For specimen V_c, tested in compression, the test set-up is the same as that described in a prior paper(6). Specimens V_c and W (tested in compression and in tension, respectively) were laterally supported by two pairs of flexible bars which were anchored to rigid supports furnished by two 8WF17 beams clamped to the outside of the testing machine columns at the level of the knee. Specimen V (which had already been tested in compression) was tested in tension without lateral supports.

The tests were carried out under static loading conditions at room temperature. An Ames dial gage was used to measure deflection between the arms of the connection specimens A to M and V (see Fig. 4). For Connections V and W both deflection and corner rotation data were taken. The specimens were whitewashed to reveal the flaking of the mill scale when yielding occurred.

The usual practice was to apply loads in 2-kip increments in the elastic range. This increment was decreased when yielding commenced. Beyond the elastic limit "plastic creep" occurred, and therefore deformation gage readings were not taken until the connection had stabilized.

In certain cases, the test set-up was somewhat crude. The use of lateral support was generally abandoned; if lateral support had been provided, the strength of the connections probably would have increased. However, it was considered that should the tests indicate satisfactory performance when tested in this simple manner, the procedure would be justified since compression loading is the more usual and critical case.

4. TEST RESULTS AND DISCUSSION

The results of the tests are shown in Figs. 6 to 18 and are summarized in Table 4 and in Fig. 19. The data has been arranged primarily according to connection type. Figs. 6 to 11 cover the straight connections, Figs. 12 to 15 show the results of tests on the tapered haunches, and Figs. 16 to 18 include the curved knees.

4.1 STRAIGHT CONNECTIONS

(1) Connections V_c, V, W

In Fig. 6 the curves of moment at the knee vs deflection measured across the assembly are given for specimens V_c, W, and V. These are all type 8B connections with web stiffener omitted over the column flange. The tests were carried out at Texas to supplement the prior Lehigh tests A to M. A simplified theoretical deflection curve is also drawn, based on methods already developed in Part II of Ref. 2.

Comparison of the prime compression test (V_c) with the prime tension test (W) shows almost identical behavior. The strengths are substantially the same (555 and 565 in-kips). The tension test is only slightly stiffer than the compression test and even this small difference probably would have disappeared had the moment values been corrected for change in distance from the line of force to the center of the knee as the specimen deflected. Both V_c and W were somewhat more flexible than the simple theory predicts. The moment at the knee is obtained by multiplying the load, P, by the prependicular distance measured from the center of the knee (intersection of center of gravity lines of column and girder) to the line of force. This distance gradually becomes greater for specimens under compression loads and becomes smaller for those under tension loads.

The difference in behavior of a "prime" tension test and one tested first in compression and then in tension is shown in Fig. 6 by a comparison of V and W. Specimen V, tested in tension after having been previously tested in compression as V_c, carried a higher maximum moment (M_h = 625 in-kips against 565 in-kips for W), and was considerably more flexible than prime member W. The curve for Connection V departs from linearity sooner than the other two connections due to the Bauschinger effect (localized residual stresses due to the prior compression yielding). The increase in strength may be due, in part, to hardening by virtue of the prior compression.

In both Connections V and W the first weld failures* occurred at about the same moment value; but V went on to carry increased load, whereas W collapsed rather quickly. The most probable cause for this difference is better welding on V than on W. Another contributing factor might be the presence or absence of lateral support. V was not supported laterally; W was. By increasing the general stiffness of the entire assembly, fracture tendency is increased. It seems, however, that difference in weld quality is the more likely explanation.

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* As will be discussed later, in some cases weld cracks were scarcely visible and did not alter the behavior; in others they progressed and led to final collapse.

These connections were detailed just like Connection K. By comparison with Fig. 23 of Ref. 2 it will be seen that the behavior of the compression test (V_c) was almost identical to Connection K.

All three connections shown in Fig. 6 reached the computed M_p value. There was a considerable reserve of strength beyond the first weld crack.

In the figures the points of first yield (Y), local buckling (L), weld crack (C), and weld fracture (F) have been indicated by the appropriate symbols. With regard to yielding, the two prime connections show the typical behavior of local yielding at about 1/3 to 1/2 of the computed yield load. Local buckling of flange edges was observed by eye.

Weld cracks occurred in specimens W and V. However, in the case of W, weld failure started after the initiation of local buckling. On all of the connections, weld quality left something to be desired. The connections performed so satisfactorily, however, that the effect of defective welds would not have limited the ability of a frame to carry the computed loads.

Fig. 7 shows Connection W at the end of the test. One crack started at the inside column flange and propagated considerably into the web of the column; the other failure was in the diagonal stiffener attachment at the inside corner. This was the only connection to fail at the diagonal stiffener connection. The fact that yielding due to shear force occurred in the web that had been stiffened with the diagonal plate is further evidence for the need of this element. Fig. 8 shows connection

V at the end of the test. The extensive yield in the web and local buckling of the exterior flange shows that the fracture did not limit the carrying capacity of the knee. Fig. 21 shows by sketch the location of weld fractures.

(2) <u>Connections A, K, L, M</u>

Fig. 9 gives the moment-deflection curves for the four square knees A, K, L, and M. For these specimens (as well as those that follow) the moment arms used were corrected for the change in distance between the center of the knee and the line of applied forces due to deformation. Table 4 summarizes the strength of the four knees.

Fig. 9 shows that the behavior of the four connections was similar up to the point of fracture. By comparison with the theoretical curve it is seen that no fracture occurred at a moment less than the plastic hinge moment, M_p, except for Connection A (the one involving the most difficult weld). The strength in tension was greater than the compressive strength (excepting A). The maximum strength of K compares rather well with its companion, V.

The best characteristics were exhibited by Connection L, although there is little difference from M. Yielding at relatively low loads due to the Bauschinger effect is also in evidence in these tests.

Local buckling (see "L" in Fig. 9) is delayed in comparison with the compression tests. Fig. 20 indicates why local buckling is more critical for compression than for tension.

Under compression loading (20-a) local buckling would occur at lower moments because compression flexural strains are added to direct strains in compression; under tension loading (20-b), the compression flexural strain is reduced by the superimposed tensile direct strain. Local buckling should therefore occur at a higher resultant moment when the loading is in tension. The table in Figure 20-c confirms this.

Figs. 10 and 11 show Connections A and K at the end of the test. Cracks which developed in both connections at the re-entrant corner are visible. As was usually the case, first fracture did not cause immediate collapse. This came at higher load as the crack opened up and propagated to the web of the beam. As summarized in Table 4, and as described in more detail in Table 5, the weld quality was at best only fair in all of the connections. There was poor penetration and there were numerous inclusions. With regard to Connection K (Fig. 11) the external weld was good; but the penetration was poor on the weld at the opposite side.

Summarizing the behavior of these straight connections, if high "tension" load is expected, then Connection A seems the least desirable detail of the group. The rest of the connections exhibit no limitation to plastic design of ordinary structures due to the fractures that subsequently occurred.

4.2 TAPERED HAUNCHES

(1) Connections D, E, F

Fig. 12 shows the moment-deflection curves for these three connections. Concerning the theoretical values, the haunch moment at first yield, $M_h(y)$, includes the effect of axial thrust. The haunch moment at plastic yield of the rolled section, $M_h(p)$, does not.

As shown in Table 4, the strength in tension was considerably greater than the strength in compression. On the average the increase amounted to more than 25%. The weld quality seemed somewhat better (Table 5). No fractures occurred below the yield load nor below $M_h(p)$; thus there is no limitation to plastic design due to these fractures. Connection F did not fracture at all.

In Fig. 12, the moment value at observed lateral buckling is designated by "LA". Actually in any given test it is often difficult to distinguish between local and lateral buckling. Does local buckling cause lateral buckling or is local crippling the result of lateral deformation? Both lead to collapse, and in these specimens, buckling of one type or the other preceded fracture.

Due to the fact that the "notch" is less severe in the "45°-bracket" type than in the straight connections, fracture tendency is less pronounced. There was no sudden collapse due to weld failures in these specimens, the maximum load always being greater than the cracking load.

In Fig. 13 the weld failure and the local buckling of the compression flange of Connection D are shown. Connections E and F failed primarily by local and lateral buckling; Fig. 14 shows Connection F at the end of the test.

(2) Connections B and C

Fig. 15 shows the moment vs deflection curves for haunched connections B and C. As with the previously-discussed connections, the strength in tension was greater than the compression strength (Table 4). Connection C developed a moment greater than that necessary to meet plastic hinge requirements. Although the strength of Connection B exceeded the computed yield value, it did not quite reach the plastic moment value; it did not fracture.

Both connections failed by local and lateral buckling of the outer (compression) flange. The weld crack did not affect the strength of Connection C since buckling preceded it.

4.3 CURVED KNEES (SPECIMENS G, H, I, J)

Fig. 16 gives the moment-deflection curves for these four connections. The comparison in Table 4 shows that the curved knees developed strength in tension just about equal to those in the prior compression tests (within 4% for G, H, and I). There is a most adequate reserve of strength above first yield. Comparison of Fig. 16 with Fig. 44 of Ref. 2 indicates that the deformation behavior in tension was also quite similar to that in compression.

All four curved knees failed by local and lateral buckling. Some cracks of minor importance developed in H and I at the end of the knees where the rolled sections were welded to the built-up portions, and in J at the radial stiffeners; but all these weld "failures" were insignificant and did not contribute to the failure. As Fig. 16 shows, buckling occurred prior to fracture.

Fig. 5 shows Connection G at the end of the test. The localized yielding and the lateral buckling which brought about the final collapse of this connection are evident. Fig. 17 shows specimen I after failure and at the end of the test. Connection J is shown in Fig. 18.

4.4 STRENGTH OF CONNECTIONS

As mentioned previously, Table 4 presents a summary of the strength of the connections tested in this series and subjected to tension loads. Fig. 19 is a summary of the moment vs deflection curves for all of the connections tested and is presented on a non-dimensional basis. In every case the maximum observed haunch moment was greater than the computed yield value; as Fig. 19 shows, the reserve above first yield is more than adequate in most cases. Further, in every case except Connection A, the strength in tension was at least equal to and in most cases greater than the strength in compression (Table 4). The average increases were 11% for the straight connections, 27% for the 45°-bracket connections, 8% for haunched connections B and C, and 3% for the curved knees.

A comparison with the computed haunch moments to give the plastic moment at the extremities of the haunch (Table 6, Part III, Ref. 2) shows that except for Connections A and B, the full plastic moment strength was developed.

The limitation on strength was either local and lateral buckling or weld fracture. The results show that the design of Connection A is poor and that Connection B should be proportioned with a thicker inner flange; but otherwise neither the local buckling nor the weld fractures could be considered as a limitation on use of these knees. The necessary strength was developed prior to "failure".

4.5 RIGIDITY OF CONNECTIONS

Most of the members tested in tension exhibited non-linear behavior at lower loads than in the previous compression tests. This is due to the Bauschinger effect; in no way does it constitute any limitation in the use of these connections in design.

With regard to rotation capacity, the performance meets all normal design requirements. In the curved and haunched connections none is required. Some of the straight connections show little rotation capacity, but in ordinary structures rotation capacity "in tension" is not mandatory as such a hinge is usually one of the last to form.

4.6 PERFORMANCE OF WELDS

The chief difference between the tension and the compression behavior is that fractures of the weld occurred in the former. This fracture is due in part to stress concentrations and the presence of multi-axial stresses; but primarily it was due to poor welding. Table 5 summarizes observations made in connection with the welding of the various joints.

In three of the connections there were no weld failures whatever (B, F, G). In six of the connections cracks occurred (designated by "C" in some of the figures -- the connections involved were V, E, C, H, I, J). In no case did these cracks develop further nor did they influence the behavior in any way. Fractures occurred in the remaining six tests (A, K, L, M, W, D). Five of these were straight connections and one (D) was a bracketed joint. Of these six joints, fracture was not accountable for the final failure of L and D (Fig. 13) because local and lateral buckling were also involved in the collapse. The sudden collapse due to fracture of A, W, K, and M is evident from Fig. 19. As may be seen from Fig. 19 only Connection A fractured at a moment less than the yield moment. There is a "reserve" strength above the fracture moment in most cases.

As is clear from Table 4 and Table 5 there is a direct correlation between weld quality or soundness and the tendency to fracture. Sound welds did not fail in spite of severe stress concentrations. Poor welds led to fractures. Had all of the welds been sound, it is reasonable to suppose that <u>none</u> of the connections would have failed due to weld fracture.

In those connections which failed due to weld fracture, the prime cause was lack of penetration. All of the failures were of the shear (ductile) type except for those cases where penetration was apparently poor.

As was mentioned above, only in the case of Connection A was weld failure a limitation in the use of the knee. In all other cases of fracture, the necessary design requirements had been achieved prior to weld failure.

More recent tests on 12- and 14-in. straight connections (to be reported upon later) have been conducted with no fractures whatever.

5. SUMMARY

The following summarizes the results of this study of connections loaded in such a way that the inside, or re-entrant, corner is in tension (that is, "tension loading"):

1. The formation of hinges due to tension loading in usual engineering structures is highly unlikely. (Figs. 1-3). Thus the requirements of strength and reserve ductility are not as stringent for tension connections as for those loaded in compression.

2. The stiffness of connections in tension is comparable to that in compression.

3. Except for Connection A, all of the connections were stronger in tension than in compression, even though cracks or fractures occurred in many of them. (Table 4)

4. The first crack did not cause immediate collapse. In almost every case there was a reserve of strength after initiation of a crack (up to 36%).

5. With but a few exceptions the mode of failure of the connections under tension loading was similar to that observed in the compression tests. Ordinarily fractures were of less pronounced influence than the local and lateral buckling that produced final failure.

6. The weld quality on the straight knees (A, K, L, M, W, V) was generally poor, and there was a direct correlation between weld soundness and fracturing. It is reasonable to expect that had the welding been sound, none of the fractures would have occurred. In more recent tests on larger connections, no fractures whatever have occurred.

7. Only Connection A fractured at a moment less than the yield moment (Fig. 19). In most cases there is a considerable and adequate "reserve" above computed yield. There is a correlation between fracture tendency and magnitude of stress concentration.

8. These results show that, even with poor welds, plastic design of rigid frames would be no more limited by the fractures than would elastic design. Except for Connection A, neither would the fractures be a limitation to elastic design. With good quality of welding, these connection types would constitute suitable designs even for those special cases that require the formation of a tension hinge.

6. ACKNOWLEDGEMENTS

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A part of the work on this "tension connection" study was done at the University of Texas as an outgrowth of studies commenced at Lehigh.

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7. REFERENCES

1. Johnston, B. G. / Yang, C. H. / Beedle, L. S. — "AN EVALUATION OF PLASTIC ANALYSIS AS APPLIED TO STRUCTURAL DESIGN" Progress Report No. 8, Welding Journal, 32(5), p. 224-s. — 1953

2. Beedle, L. S. / Topractsoglou, A.A. / Johnston, B. G. — "CONNECTIONS FOR WELDED CONTINUOUS PORTAL FRAMES" Progress Report No. 4, Part I - "Test Results and Requirements for Connections", 30(7), Welding Journal 539-s (1951); Part II - "Theoretical Analysis of Straight Knees", 30(8), Welding Journal, 397-s (1951); Part III - "Discussion of Test Results and Conclusions", 31(11), Welding Journal, 543-s (1952). — 1951

3. Symonds, P. / Neal, B. — "RECENT PROGRESS IN THE PLASTIC METHODS OF STRUCTURAL ANALYSIS" Journal of the Franklin Institute, 252(5), p. 482. — 1951

4. Baker, J. F. — "A REVIEW OF RECENT INVESTIGATION INTO THE BEHAVIOR OF STEEL FRAMES IN THE PLASTIC RANGE" Journal of the Institute of Civil Engineers, No. 3 (1948-49). — Jan. 1949

5. Beedle, L. S. — "PLASTIC STRENGTH OF STEEL FRAMES" Proceedings, Structural Engineers Association of California, Fritz Laboratory Report No. 205.26. — Oct. 1954

6. Toprac, A. A. — "AN INVESTIGATION OF WELDED RIGID CONNECTIONS FOR PORTAL FRAMES" Welding Journal, 33(1), p. 40-s to 56-s. — 1954

8. NOMENCLATURE AND TERMINOLOGY

A	=	Area of Cross-section
b	=	Flange width
"C"	=	Weld Crack
d	=	Depth of section
E	=	Young's modulus of elasticity
E_{st}	=	Strain-hardening modulus $= \left.\frac{d\sigma}{d\epsilon}\right\|_{st}$
"F"	=	Weld Failure
H	=	Horizontal load
I	=	Moment of inertia
"L"	=	Local buckling
"LA"	=	Lateral buckling
M	=	Moment
$M_{h(i)}$	=	Computed yield moment
$M_{h(p)}$	=	Haunch moment at plastic yield of the rolled section
$M_{h(y)}$	=	Theoretical haunch moment at first yield
$M_{h(4)}$	=	Maximum observed haunch moment
M_p	=	Full plastic moment
M_y	=	Moment at which yield point is reached in flexure
S	=	Section modulus, $\frac{I}{c}$
t	=	Flange thickness
V	=	Vertical load
w	=	Web thickness
"Y"	=	First yield
Z	=	Plastic modulus
ϵ_{st}	=	Strain at strain-hardening
ϵ_y	=	Strain at upper yield point
σ	=	Normal stress
σ_{uy}	=	Upper yield point
σ_y	=	Yield stress level

Table 1

Test Program

Connec-tions	Type (a)	Where Tested	Type of Test	Prior Condition	Rolled Section Joined	Lateral Support	Material Designation
A	2	Lehigh	Tension	Tested in compression	8B13	No	I
B	2B				8B13	No	I
C	15				8B13	No	I
D	4				8B13	No	I
E	4				8B13	No	I
F	4				8B13	No	I
G	5A				8B13	No	I
H	5A				8B13	No	I
I	5A				8B13	No	I
J	5A				8B13	No	I
K	8B				8B13	No	I
L	8B				8B13	No	I
M	8B				8B13	No	I
V_c	8B	Texas	Compres-sion	Prime	8B13	Yes	II
V	8B	Texas	Tension	Tested in compression	8B13	No	II
W	8B	Texas	Tension	Prime	8B13	Yes	II

(a) Figs. 8 and 9 of Ref. 2 classify connections into Types.

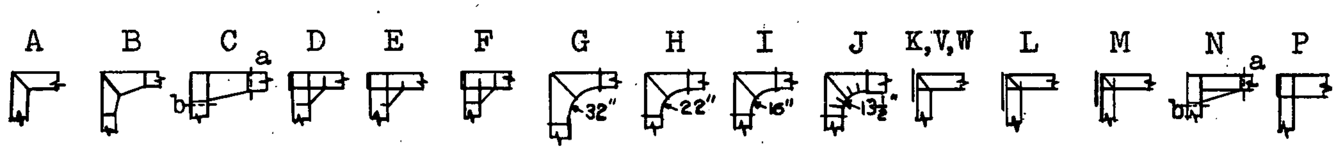


Table 2

Section Properties of Test Specimens (8B13)

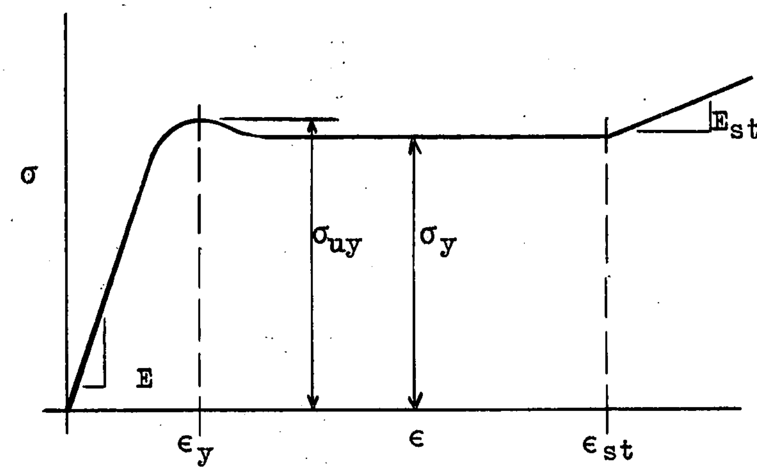
Material Designation		d in.	b in.	w in.	t in.	A $in.^2$	I_x $in.^4$	S_x $in.^3$	Z_x $in.^3$
	Handbook	8.00	4.000	0.230	0.254	3.83	39.5	9.88	11.35*
I	Measured	8.063	4.031	0.237	0.266	3.993	42.07	10.42	12.006
	% Variation	+0.788	+0.775	+3.04	+4.73	+4.26	+0.651	+5.47	+5.78
II	Measured	8.07	4.06	0.231	0.257	3.96	42.1	10.44	11.95
	% Variation	+0.875	+1.50	+0.435	+1.18	+3.39	+6.58	+5.68	+5.28

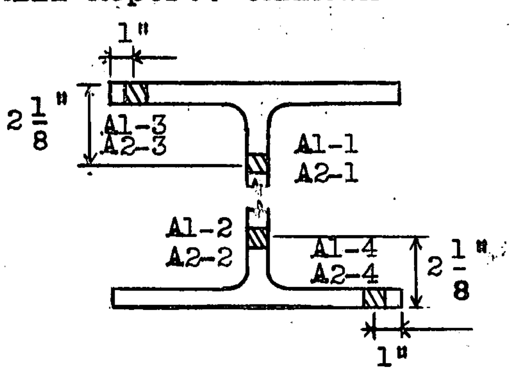
* Z_x, computed from properties of split tees, equals twice the area of the tee times the distance from stem to centroid.

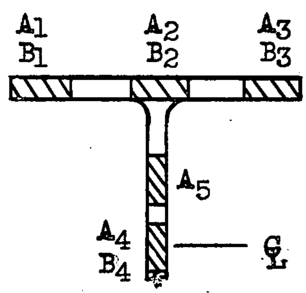
Table 3

Summary of Coupon Test Results

Coupon	Type of Test	Upper Yield Point σ_{uy} (psi)	Yield Stress Level σ_y (psi)	Elastic Strain ϵ_y (in/in)	Plastic Strain ϵ_{st} (in/in)	Strain Hardening Modulus E_{st} (psi)
8B13 Shape Material Designation I* (Lehigh)						
A1-1	Tension	46,600	45,400	0.0021	0.0170	
A1-2	Tension	55,300	51,300	0.0019	0.0171	
A2-1	Tension	43,700	42,600	0.00210	0.0165	
A2-2	Tension	50,300	49,700	0.00195		
Web Average		49,000	47,200	0.0020	0.0169	
A1-3	Tension	41,500	41,300	0.00145		
A1-4	Tension	43,300	42,200	0.00321		
A2-3	Tension	43,300	43,200	0.00190		
A2-4	Tension	42,600	42,600	0.004		
B-1	Tension	40,100	40,100	0.0037	0.0181	431,000
B-2	Tension	41,600	41,300	0.0027	0.0232	385,000
B-3	Tension	41,600	41,600	0.0017	0.0247	594,000
Flange Average		42,000	41,800	0.0027	0.0220	470,000
Mill Report:	Tension	44,470	Tensile Strength - 66,260 psi			
8B13 Shape Material Designation II* (Texas)						
A4	Tension	54,400				
A5	Tension	47,700				
B4	Tension	53,500				
Web Average		51,900				
A_1	Tension	45,800				
A_2	Tension	43,800				
A_3	Tension	47,900				
B_1	Tension	47,000				
B_2	Tension	43,800				
B_3	Tension	45,500				
Flange Average		45,600				
Mill Report: Unknown						







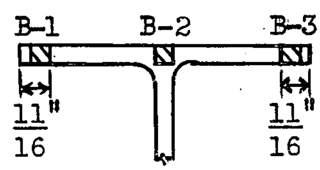


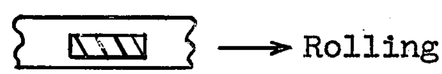
Table 3 (Cont.)

Summary of Coupon Test Results

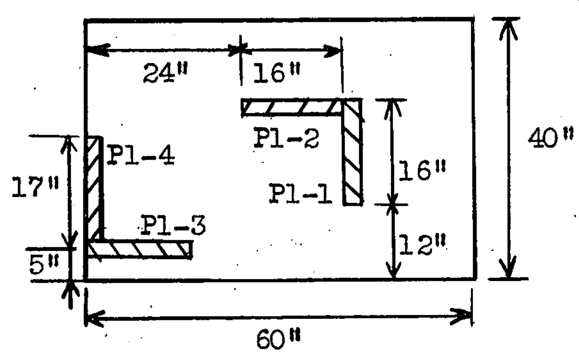
Coupon	Type of Test	Upper Yield Point σ_{uy} (psi)	Yield Stress Level σ_y (psi)	Elastic Strain ϵ_y (in/in)	Plastic Strain ϵ_{st} (in/in)	Strain Hardening Modulus E_{st} (psi)
Plate Material	Material Designation I* (Lehigh)					
P1-1	Tension	35,700	32,400	0.00128	0.0200	472,000
P1-2	Tension	33,600	32,800	0.00126	0.0179	560,000
P1-3	Tension	32,700	32,300	0.00100	0.0180	510,000
P1-4	Tension	33,800	33,200	0.00107	0.0200	465,000
P2-1	Tension	42,700	42,400	0.00180	0.0118	643,000
P2-2	Tension	41,300	40,800	0.00180	0.0185	1,130,000
P2-3	Tension	43,700	42,300	0.00130	0.0185	417,000
P2-4	Tension	44,100	43,300	0.00205	0.0197	728,000
P3-1	Tension	46,700	46,200	0.00260	0.0188	436,000
P3-2	Tension	46,500	45,600	0.00155	0.0195	506,000
C-1	Tension	38,600	38,600	0.0025	0.0168	560,000
C-2	Tension	40,200	39,500	0.0015	0.0210	546,000
D-1	Tension	36,700	35,000	0.0020	0.0162	893,000
D-2	Tension	37,100	35,500	0.0024	0.0160	606,000
Average of all Plate Material		39,500	38,500	0.00172	0.0180	605,000

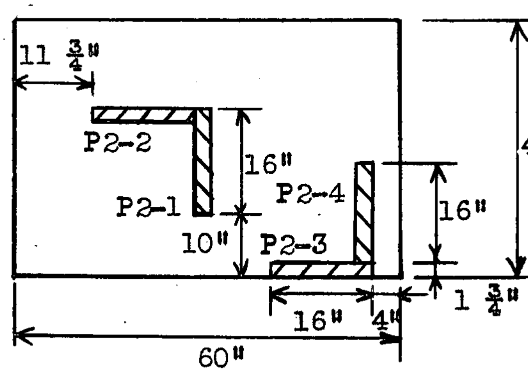
Mill Report: Unknown

* I - Heat Number 34Y532
II - Heat Number Unknown



C-1, C-2, D-1, D-2





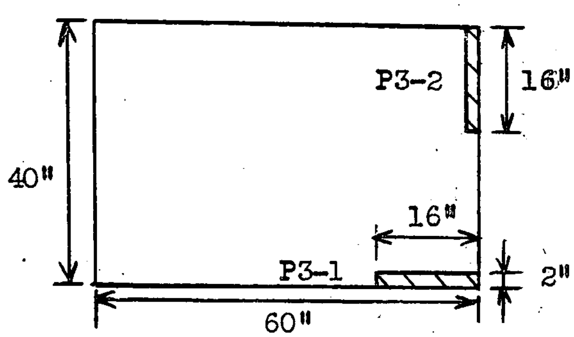


Table 4

Test Results

Connection	Type	Computed Initial Yield Moment $M_h(i)$	Tensile vs Compression Strength: Tensile Strength $M_h(4)$	Compressive Strength $M_h(4)$	$M_h(4)$ Ratio $\frac{\text{Tension}}{\text{Compression}}$	Fracture Strength (1st. Fracture): Nature of Fracture[a]	Moment at Fracture $M_h(F)$	Ratio $M_h(4)/M_h(F)$	Weld Quality
Straight Connections									
A	2	454	486	548	0.89	Fracture	361	1.34	Poor
K	8B	454	613	560	1.09	Fracture	609	1.00	Fair
L	8B	454	686	624	1.10	Fracture	686	1.00	Fair
M	8B	454	674	576	1.17	Fracture	674	1.00	Fair
V, V_c	8B	476	(V)625	(V_c)555	1.13	Crack	552	1.13	--
W	8B	476	565	555	1.02	Fracture	506	1.12	Fair
Average			$\boxed{634}$	$\boxed{574}$	$\boxed{1.11}$				
Haunched Connections (Tapered)									
D	4	597	976	751	1.30	Fracture	960	1.02	Poor
E	4	597	1050	804	1.31	Crack	832	1.26	Fair
F	4	597	1015	844	1.20	None	-	-	Good
Average			$\boxed{1014}$	$\boxed{800}$	$\boxed{1.27}$				
B	2B	750	781	728	1.07	None	-	-	--
C	15	524	793	720	1.10	Crack	*	-	Fair
Curved Connections									
G	5A	970	1358	1296	1.04	None	--	-	--
H	5A	840	1220	1228	0.99	Crack	1170	1.04	--
I	5A	883	1169	1180	0.99	Crack	1168	1.00	--
J	5A	620	1006	915	1.09	Crack	979	1.03	--
Average					$\boxed{1.03}$				

* Fracture occurred after maximum load

(a) Crack - means a very small separation - not usually visible - did not affect strength.
Fracture- means a separation that developed to the point of being visible.

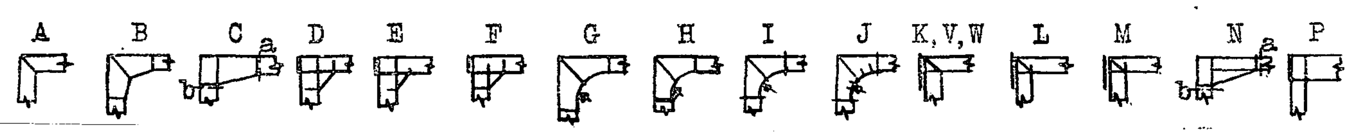
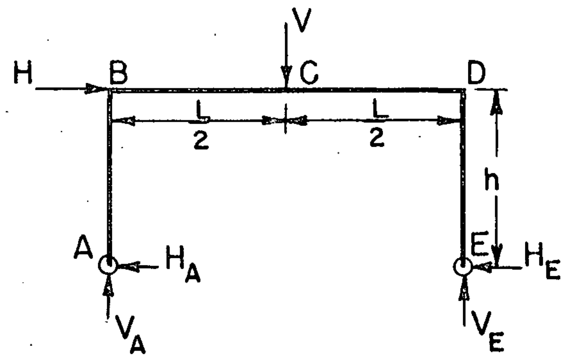


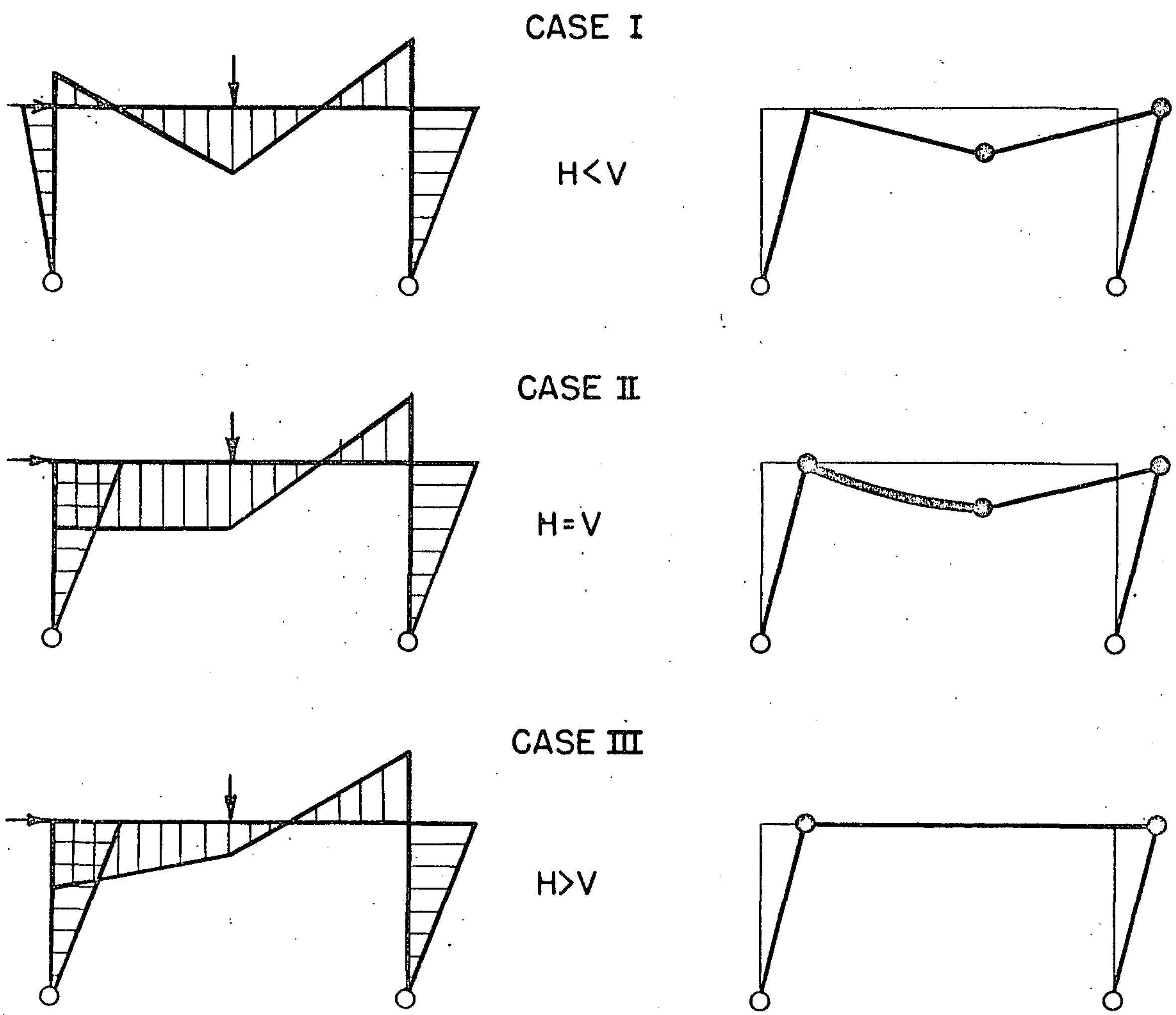
Table 5

Notes on Weld Fractures

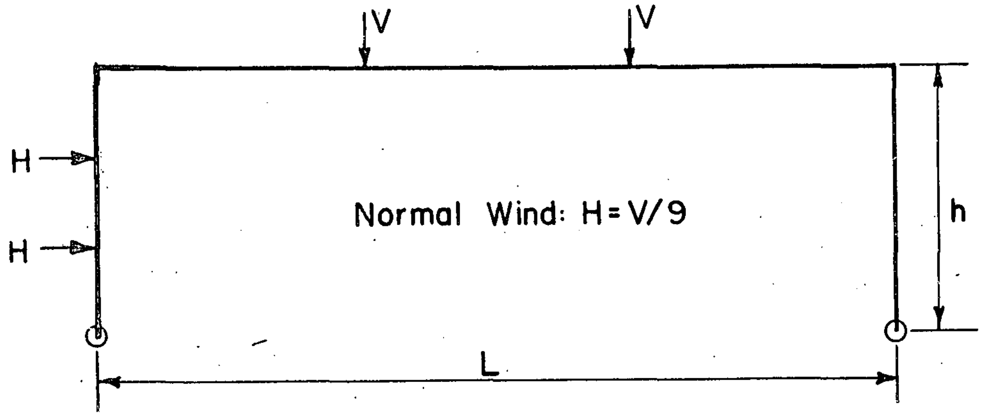
Connection	Type of Fracture	Nature of Weld Failure
A	Exterior: Poor penetration, "shear-tear" failure. Interior: Failure started at one side where there was poor penetration; progressed to the other side of flange as a ductile (shear) tear. Web: "Poor penetration" type of failure; web pulled out from the weld.	Fracture
B	No crack.	None
C	Very poor weld; crack formed but was no longer visible after end of test; weld is irregular and conducive of fracture due to stress-concentration.	Crack
D	The first tear (outside) involved a shear failure in an undersized weld; back-up pass was never deposited; failure in web attachment started in shear, shifted to "shear-tear" (poor penetration), and back to shear; poor weld at the opposite corner; the failure is also ductile with local necking in evidence.	Fracture
E	No cracks visible at end of test; appearance of both critical welds is poor. Much undercutting.	Crack
F	Appearance is the best of the series; no fractures were observed; some undercutting.	None
G	None	None
H	Crack not visible; audible during test.	Crack
I	Cracks visible during test but "closed up" at end of test.	Crack
J	Cracks at junction of radial stiffener and inner flange; "closed up" at end of test.	Crack
K	Penetration pull-out (below the fusion line); shear failure in weld as the secondary crack.	Fracture
L	Weld has better appearance than Connection K; more shear failure, but considerable penetration pull-out below fusion line -- flange.	Fracture
M	Similar behavior to L; some penetration failure at web; welds only fair; some undercutting.	Fracture
V_c	None	None
V	Crack at bottom of diagonal stiffener weld on both sides of web and crack at top of inside column flange.	Crack
W	Crack at bottom of diagonal stiffener weld on both sides of web and crack at top of inside column flange.	Fracture



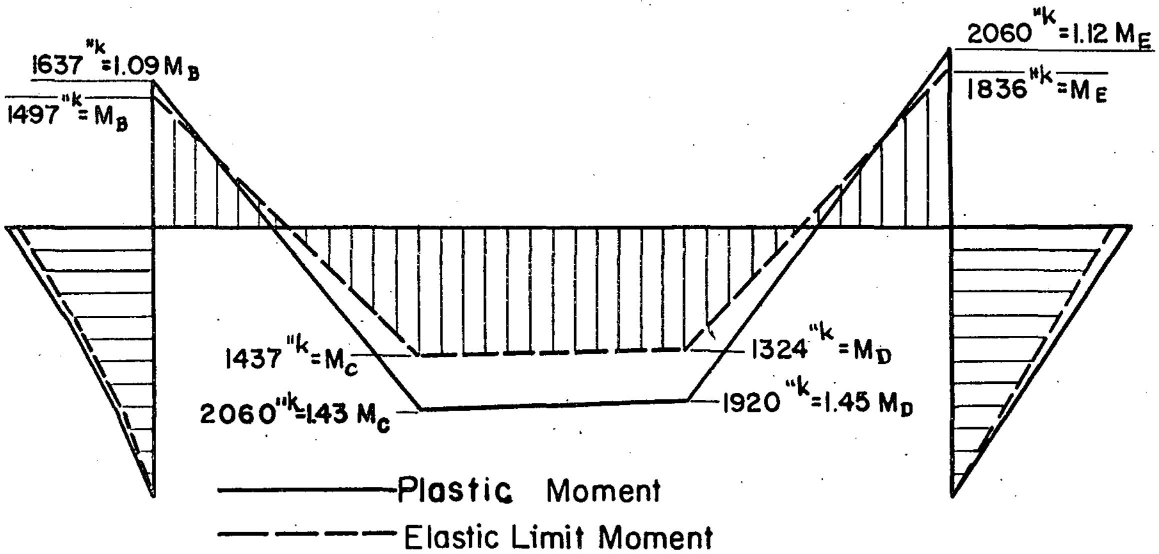
(a)



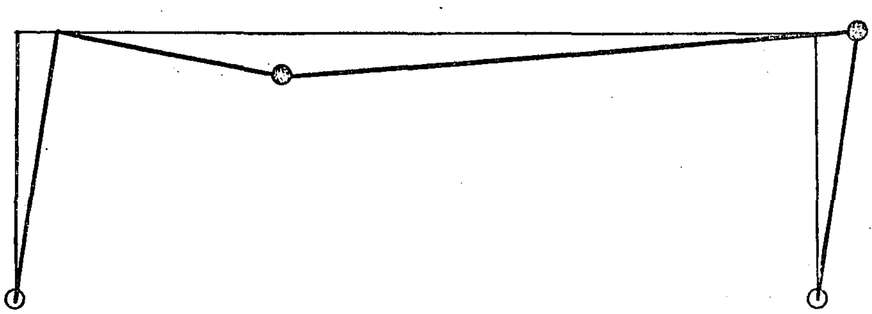
205C.15/2



(a)



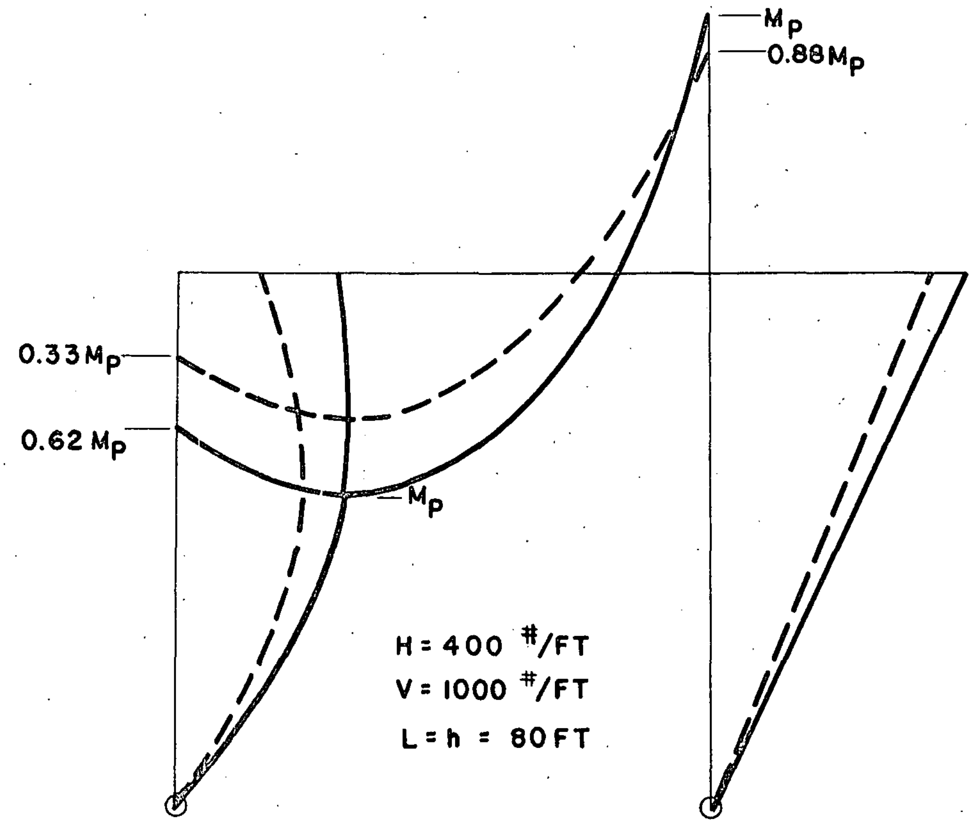
(b)



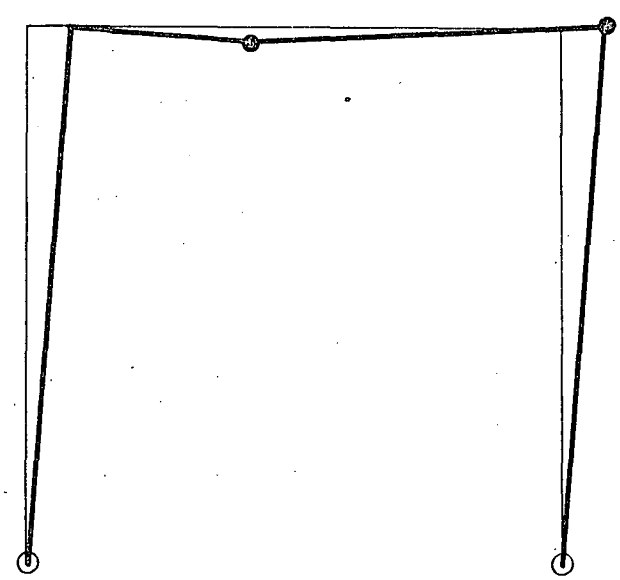
(c)

2

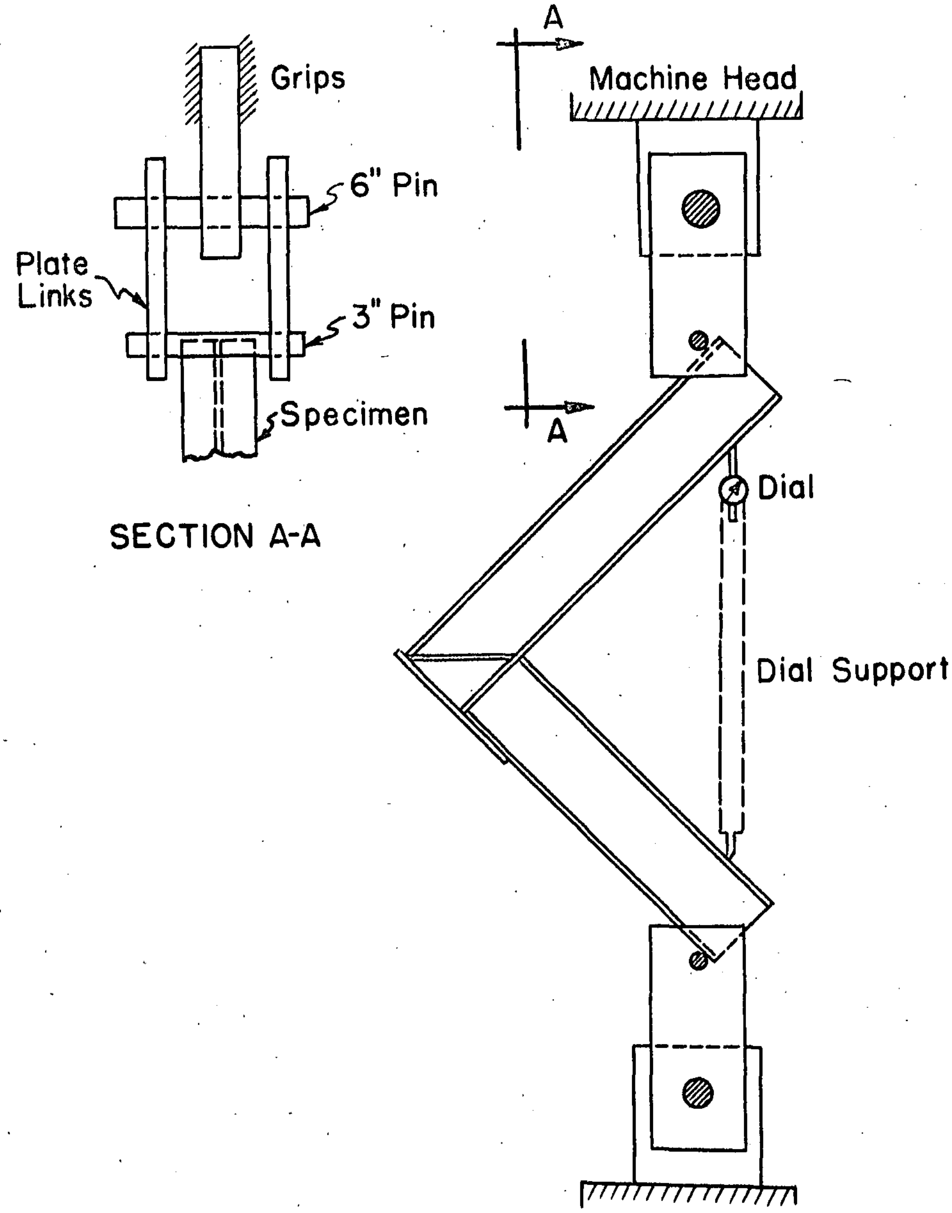
205C.15/3



(a)



(b)


Grips
6" Pin
Plate Links
3" Pin
Specimen
SECTION A-A
A
A
Machine Head
Dial
Dial Support

4

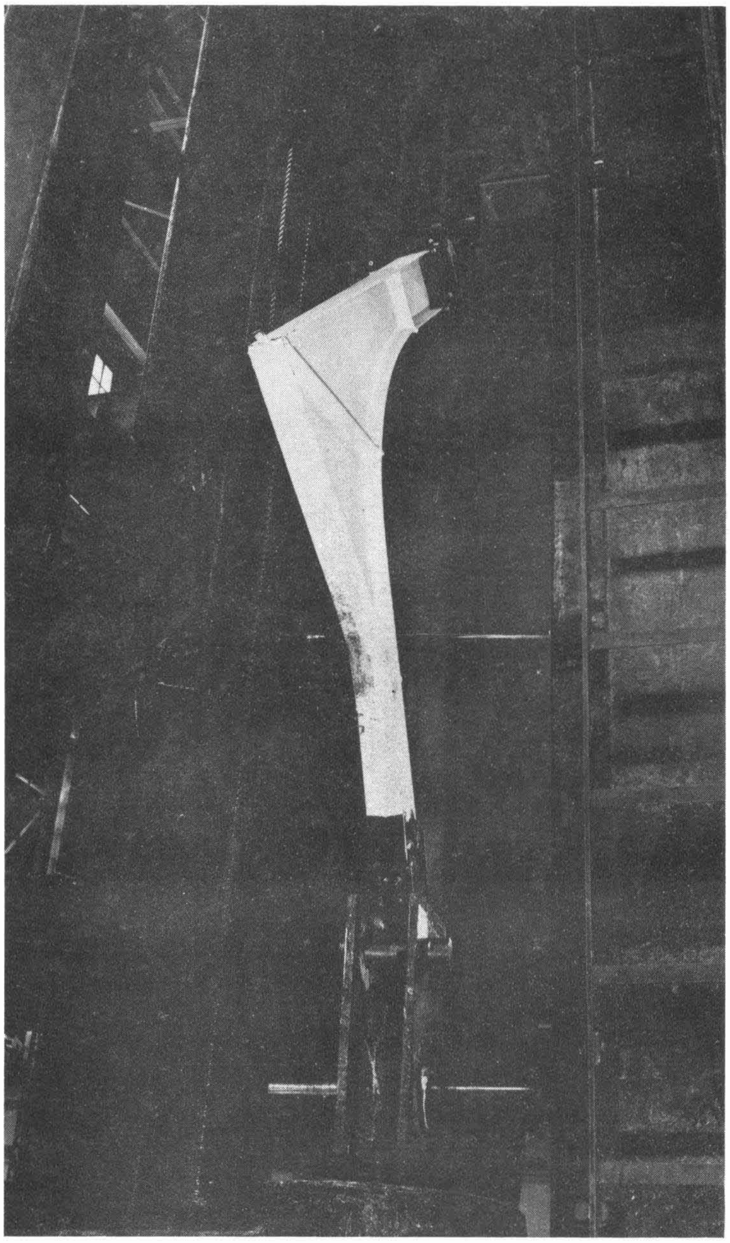


Fig. 5 Typical test set-up (Connection G)

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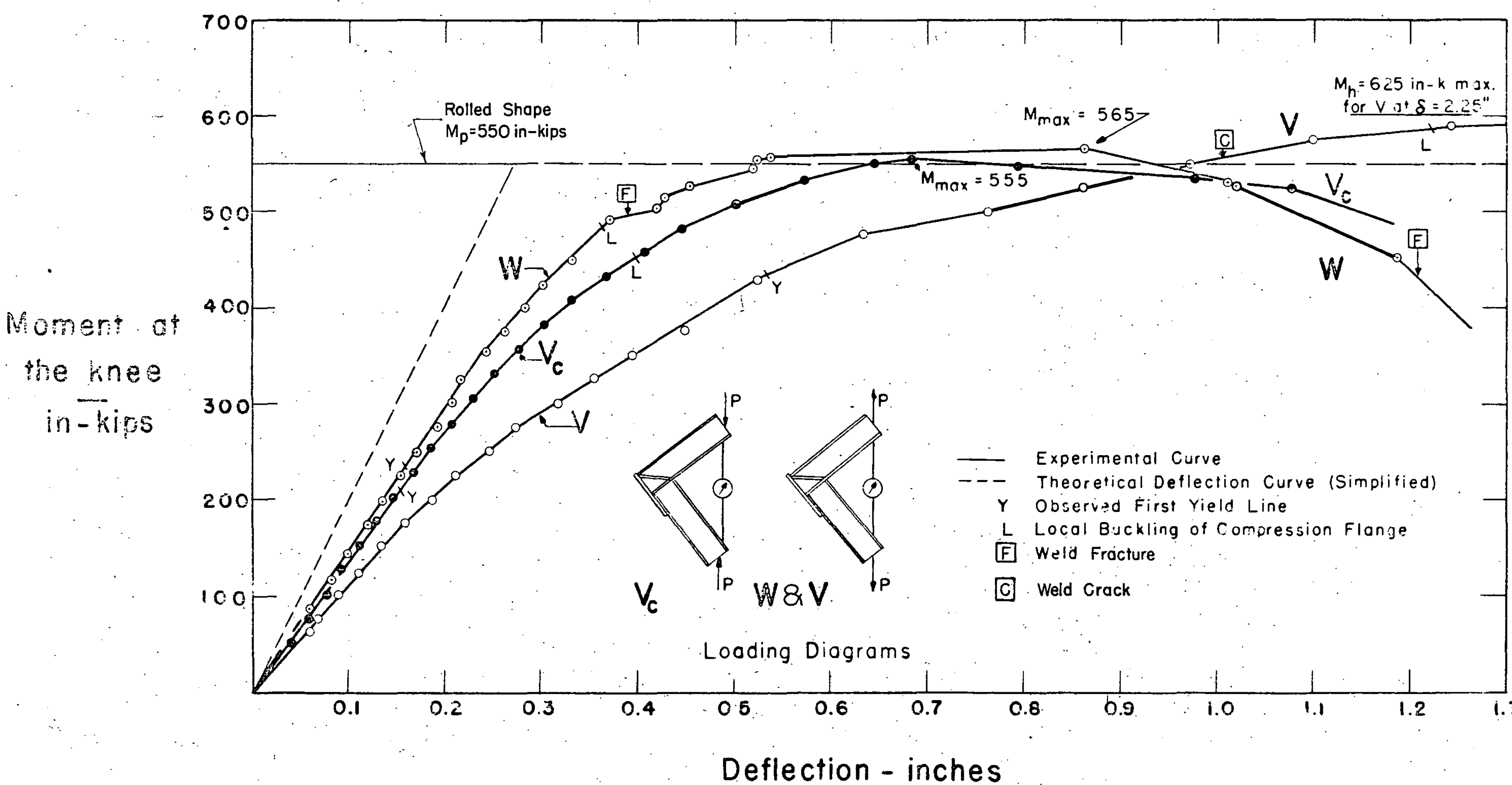


Moment at the knee in-kips
Deflection - inches
700
600
500
400
300
200
100
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
Rolled Shape
M_p=550 in-kips
M_{max} = 565
M_{max} = 555
M_h = 625 in-k max. for V at δ = 2.25"
W
V_c
V
Y
L
F
C
P
V_c
W&V
Loading Diagrams
Experimental Curve
Theoretical Deflection Curve (Simplified)
Y Observed First Yield Line
L Local Buckling of Compression Flange
F Weld Fracture
C Weld Crack

9

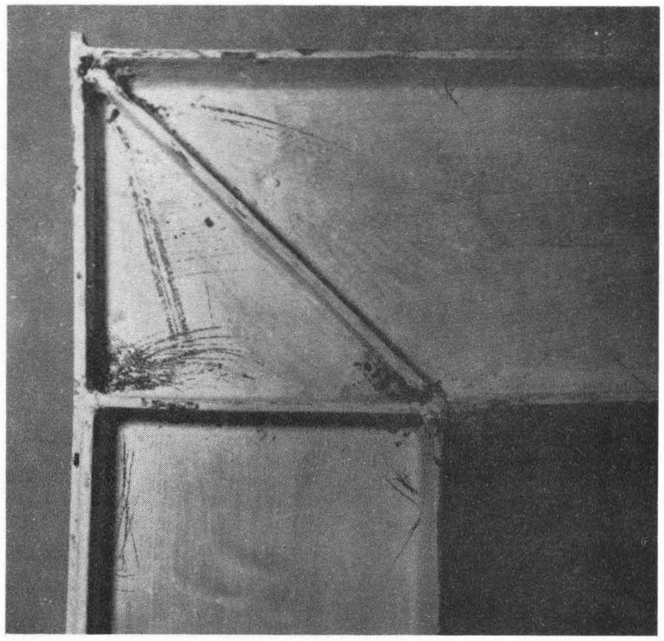


Fig. 7 Connection W. Note crack between diagonal stiffener and inside flange

7

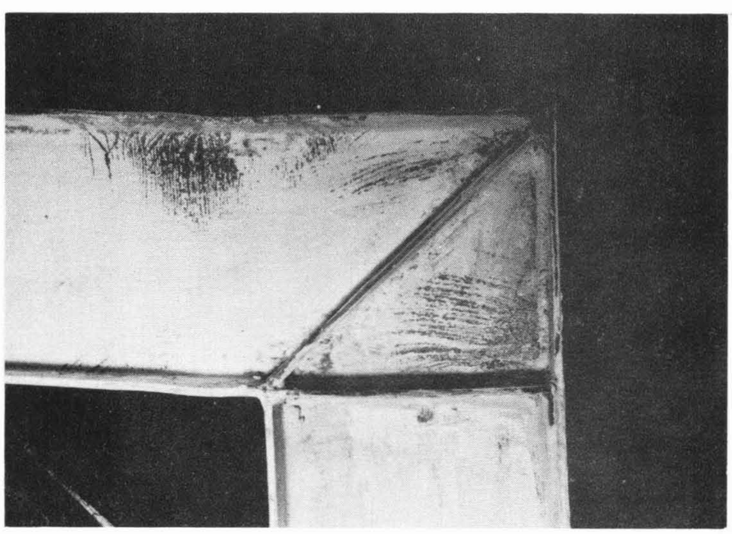
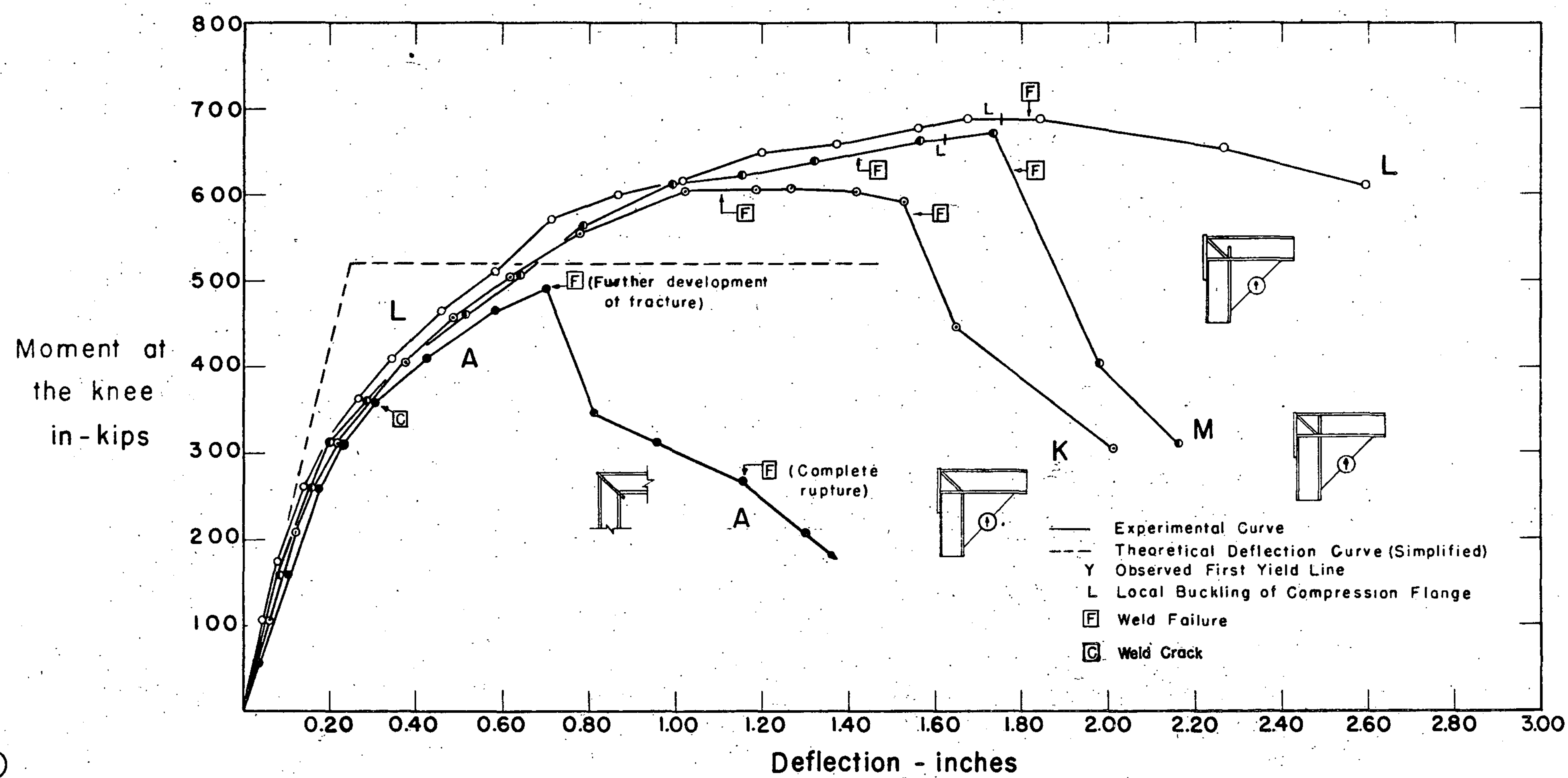


Fig. 8 Connection V at end of test

8

205C.15/9



800
700
600
500
400
300
200
100
Moment at the knee in-kips
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
2.20
2.40
2.60
2.80
3.00
Deflection - inches
L
A
K
M
F (Further development of fracture)
F (Complete rupture)
C
Experimental Curve
Theoretical Deflection Curve (Simplified)
Y Observed First Yield Line
L Local Buckling of Compression Flange
F Weld Failure
C Weld Crack

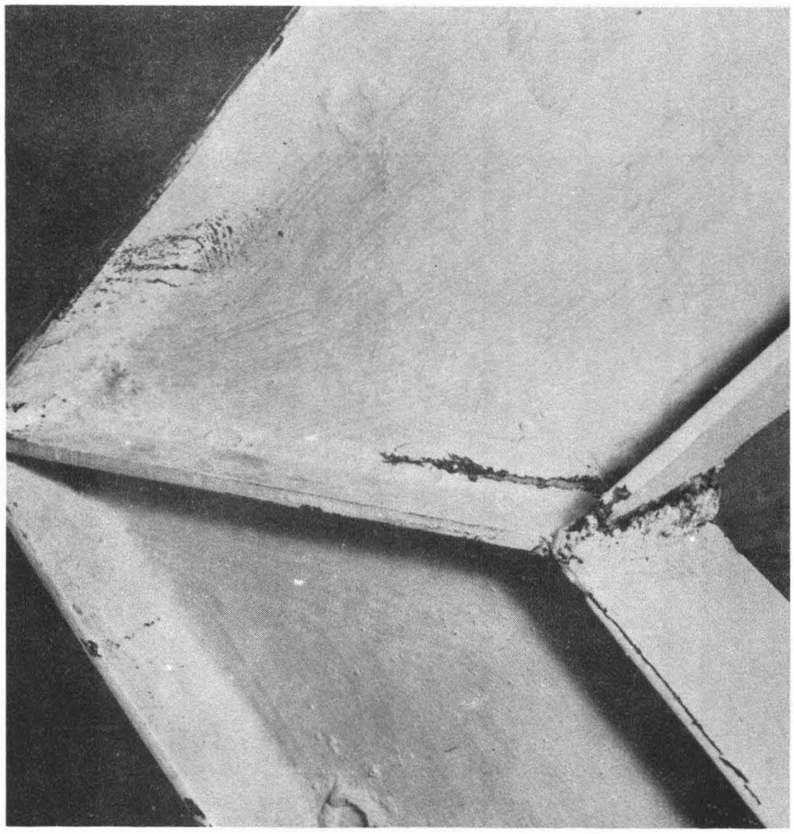


Fig. 10 Connection A. Note crack at re-entrant corner

10

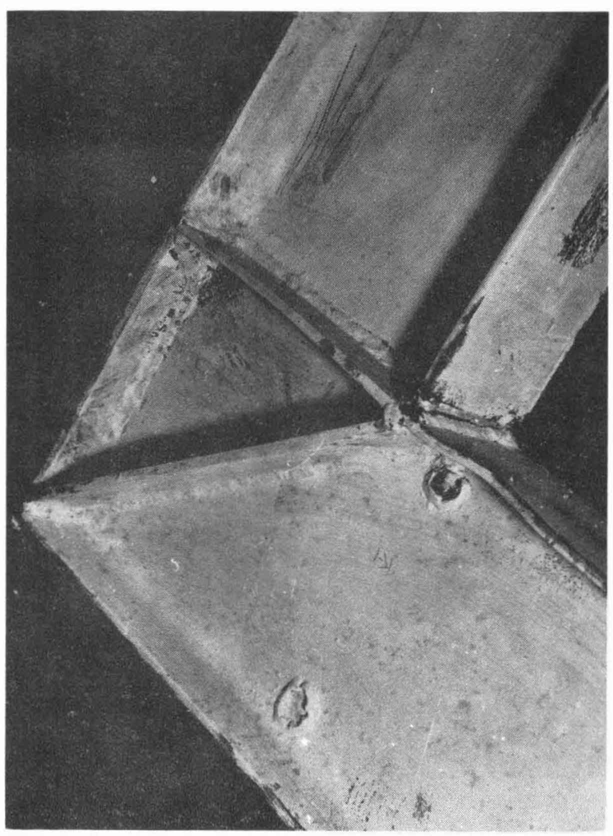


Fig. 11 Connection K. Note crack at re-entrant corner

11

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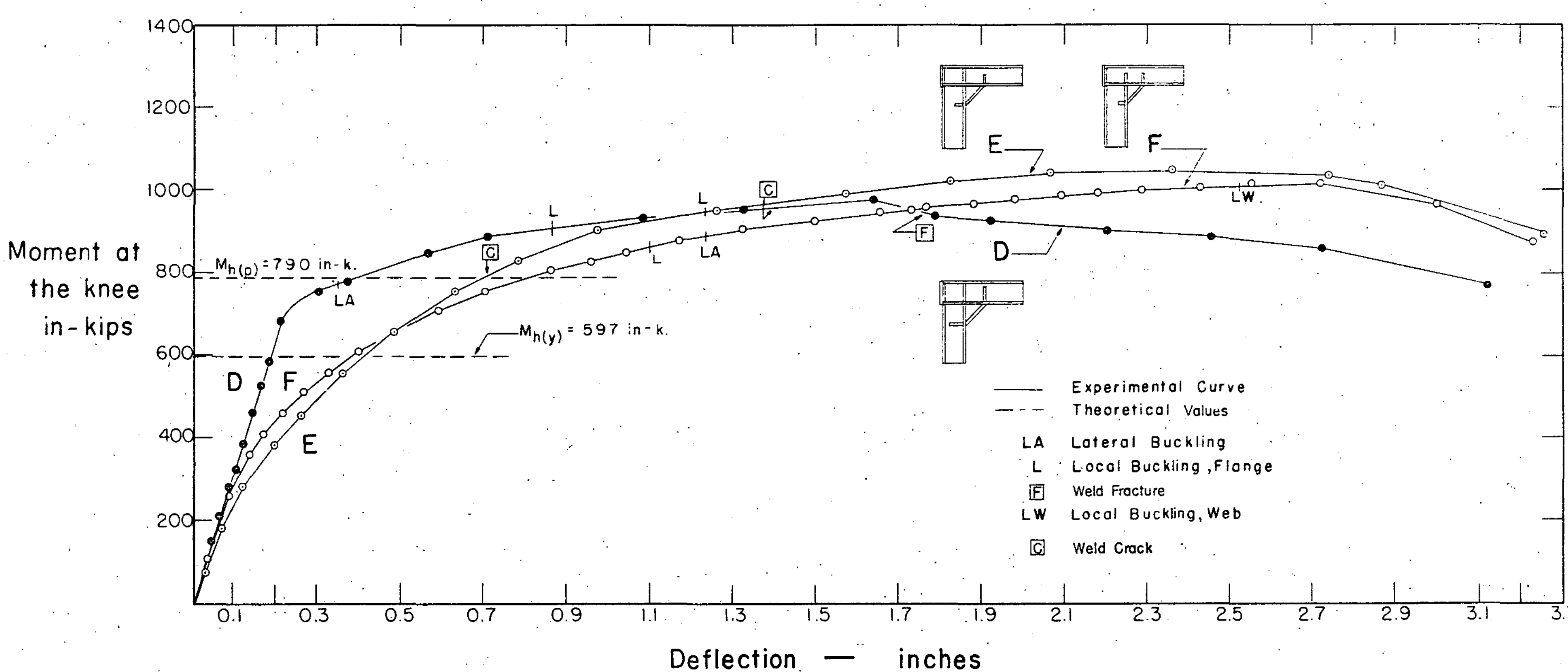


Moment at the knee in-kips
Deflection — inches
1400
1200
1000
800
600
400
200
0.1
0.3
0.5
0.7
0.9
1.1
1.3
1.5
1.7
1.9
2.1
2.3
2.5
2.7
2.9
3.1
3.3
$M_{h(p)}$ = 790 in-k.
$M_{h(y)}$ = 597 in-k.
D
E
F
L
LA
LW
C
F
Experimental Curve
Theoretical Values
LA Lateral Buckling
L Local Buckling, Flange
F Weld Fracture
LW Local Buckling, Web
C Weld Crack

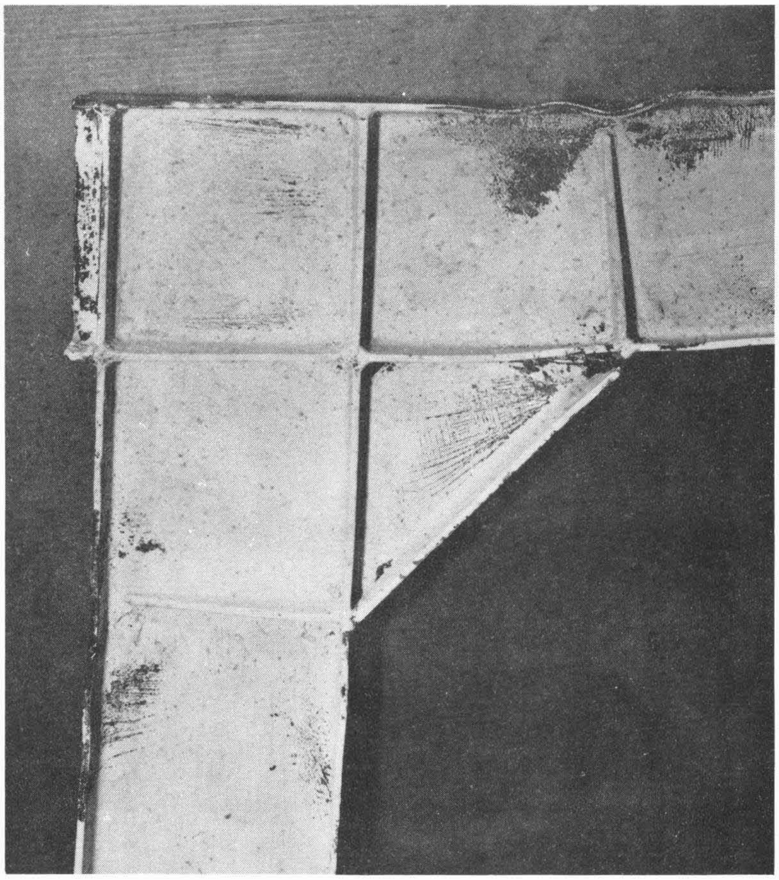


Fig. 13 In Connection D weld failure and local buckling were present at end of test 13

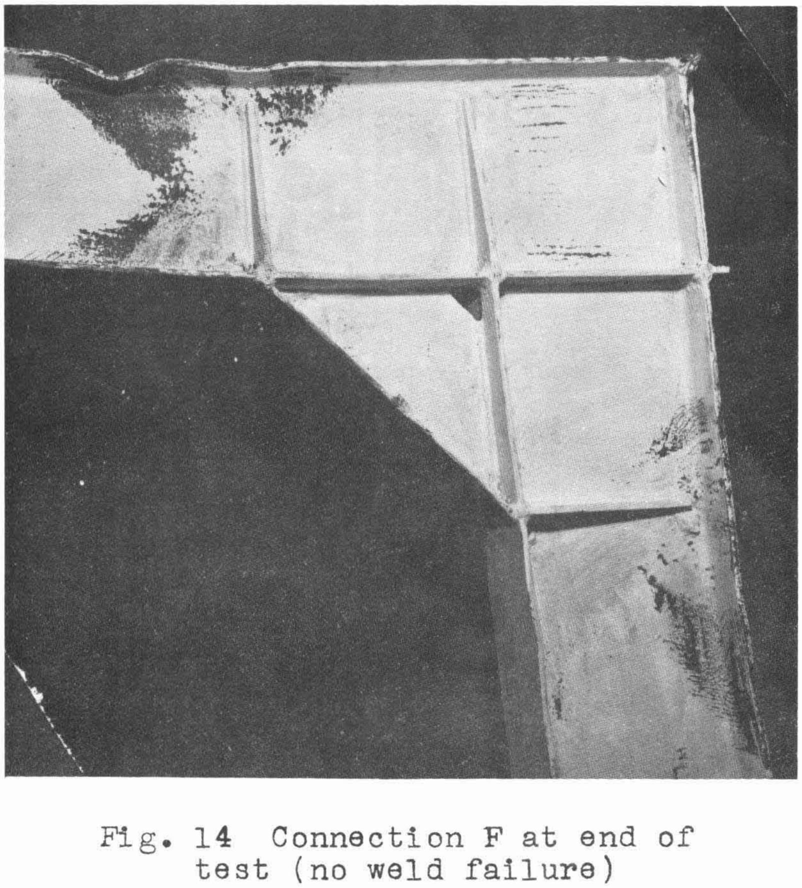
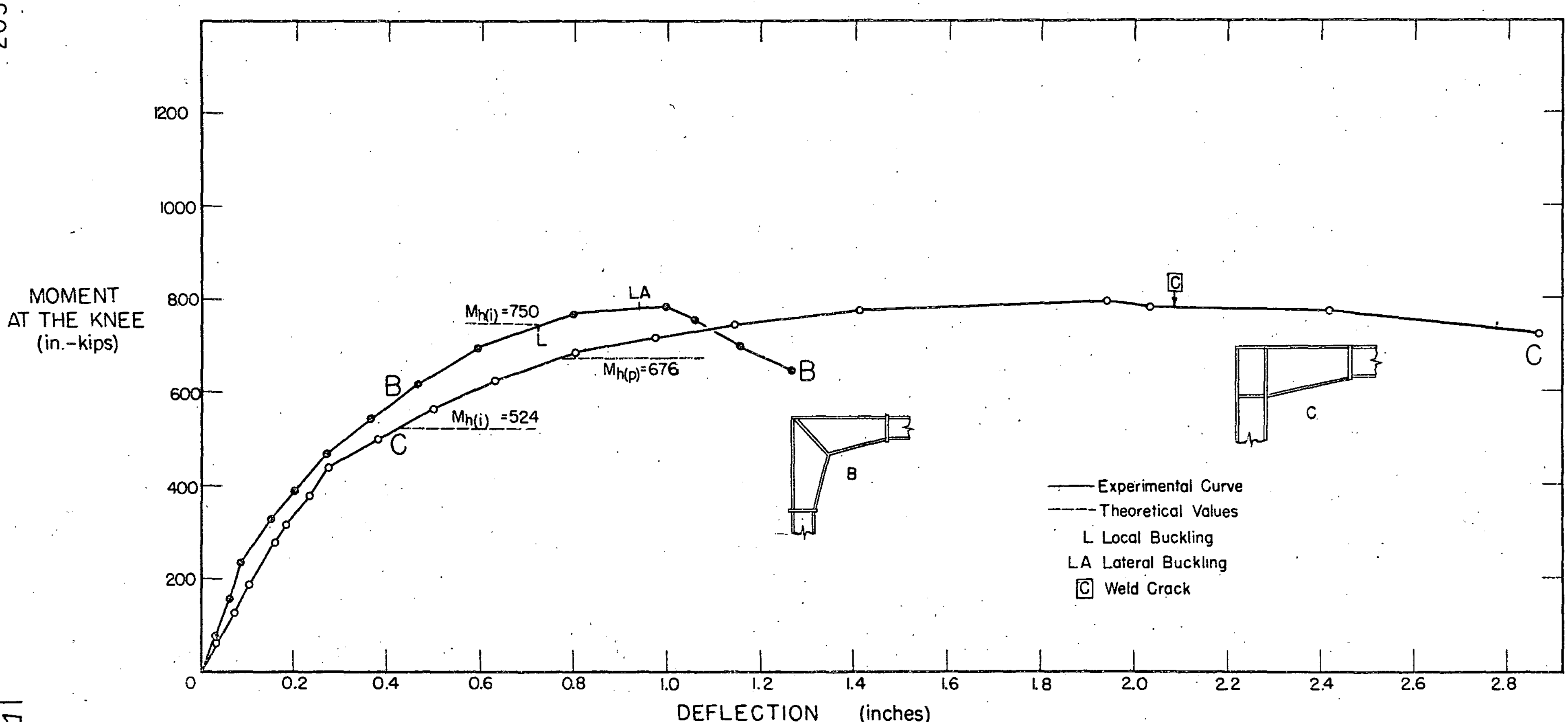
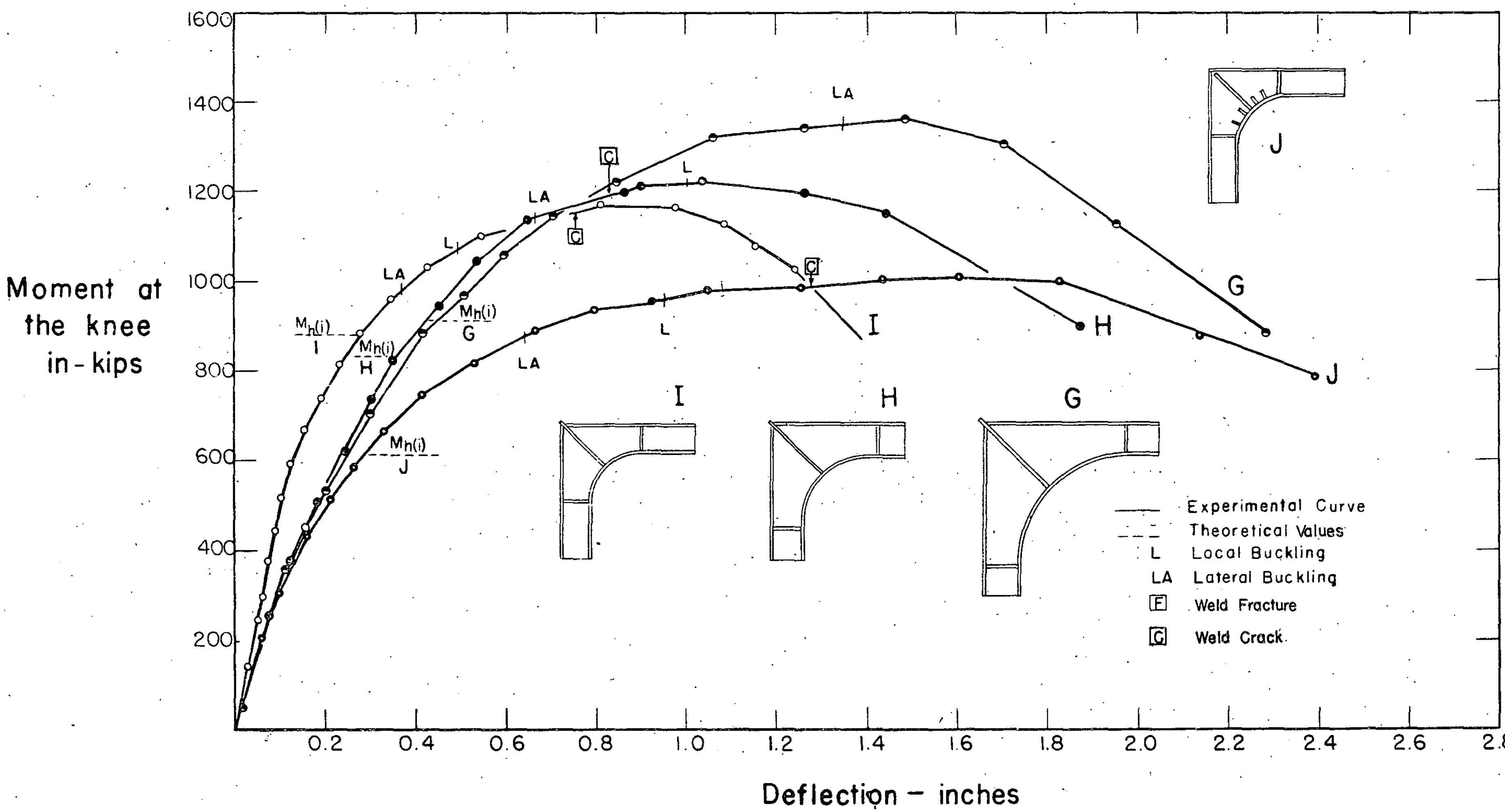


Fig. 14 Connection F at end of test (no weld failure) 14


MOMENT
AT THE KNEE
(in.-kips)
1200
1000
800
600
400
200
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
DEFLECTION (inches)
$M_{h(i)}=750$
L
LA
$M_{h(p)}=676$
$M_{h(i)}=524$
B
C
Experimental Curve
Theoretical Values
L Local Buckling
LA Lateral Buckling
C Weld Crack

15

205C.15/16



Moment at the knee in-kips
Deflection - inches
Experimental Curve
Theoretical Values
L Local Buckling
LA Lateral Buckling
F Weld Fracture
C Weld Crack
I
H
G
J

9

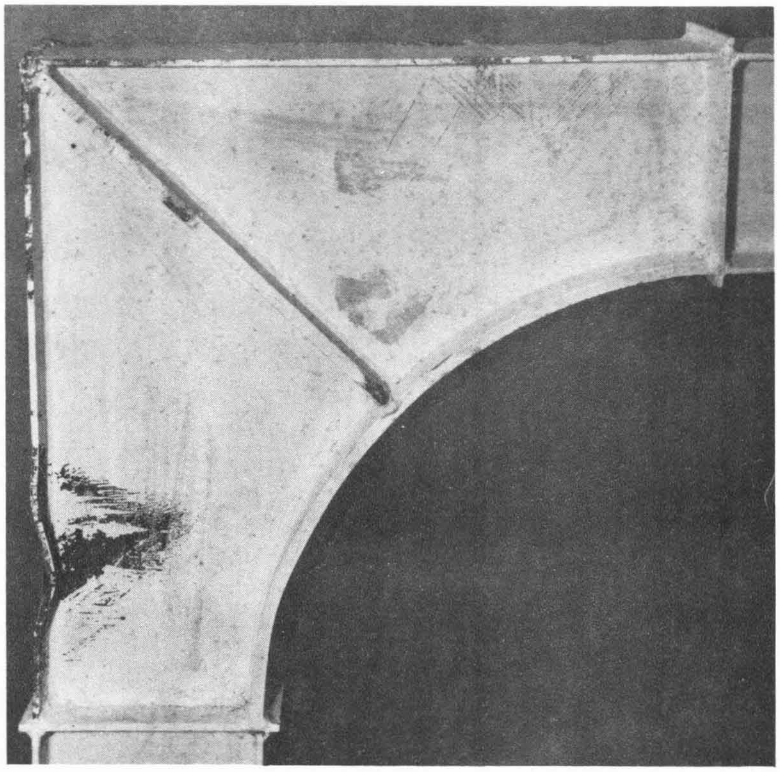


Fig. 17 Connection I

17

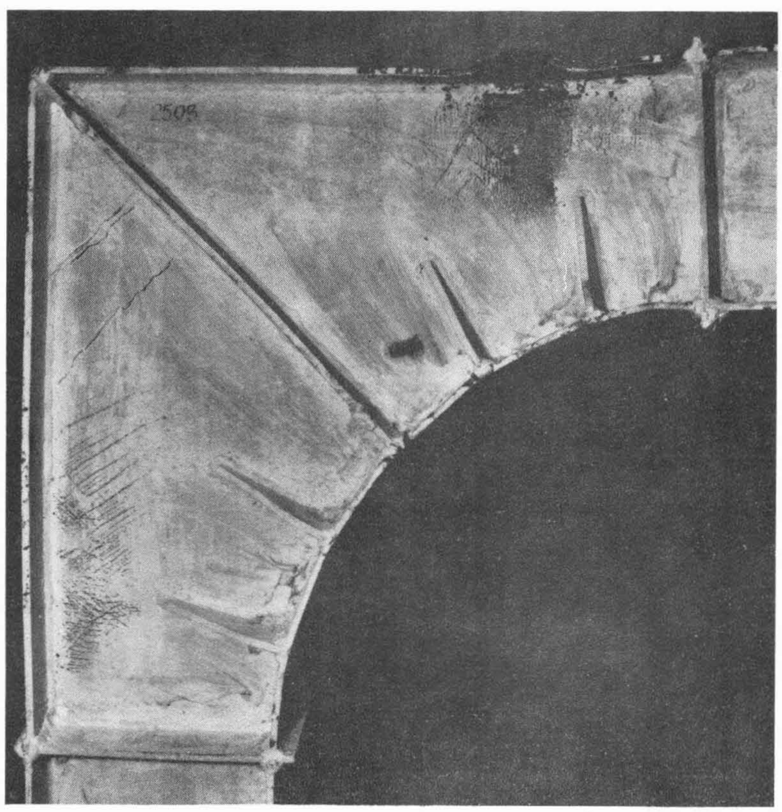
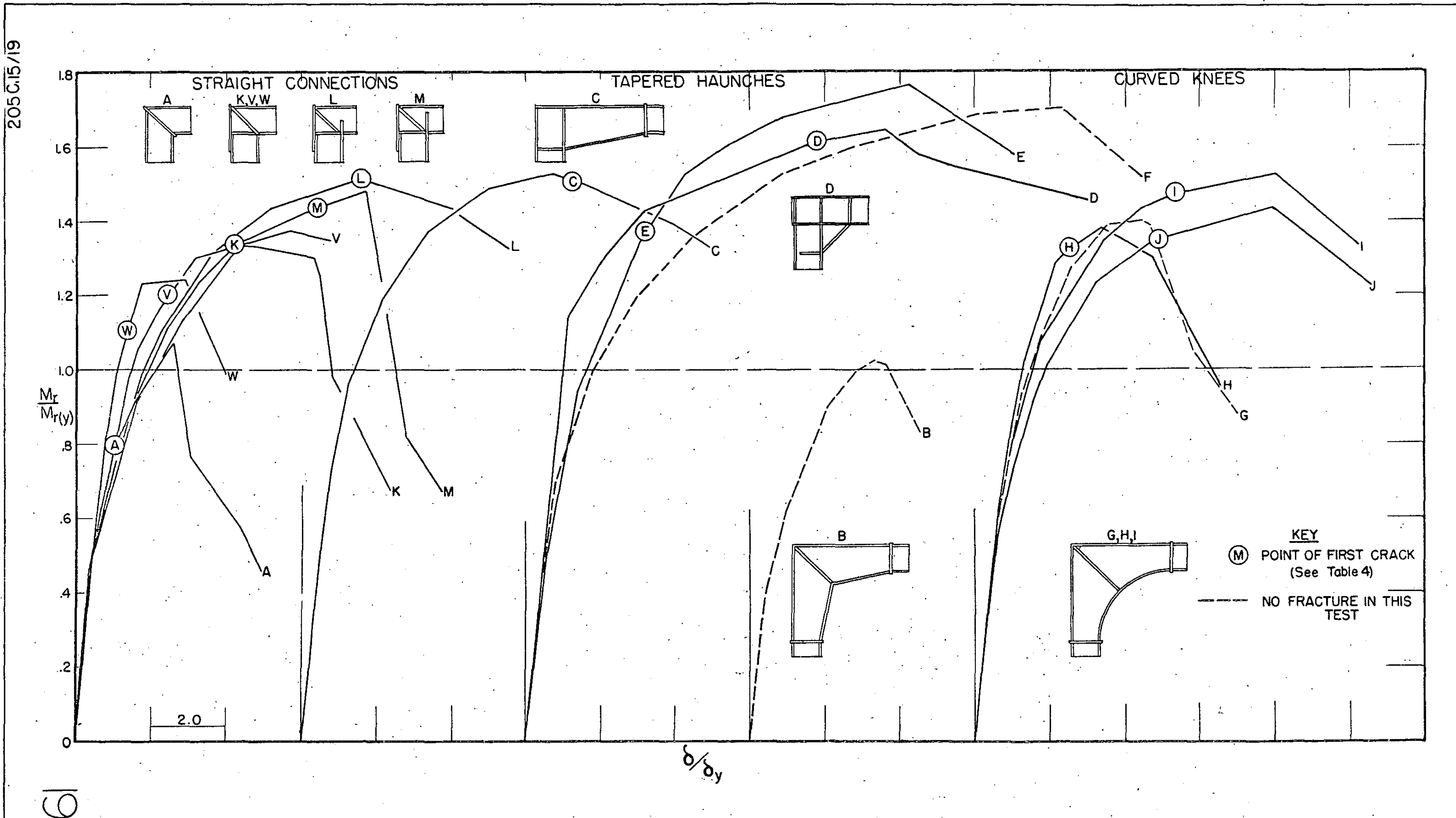


Fig. 18 Connection J

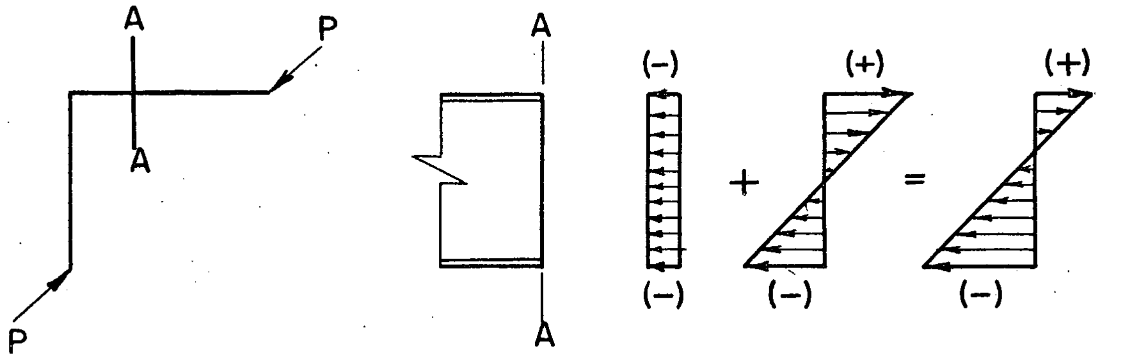
18


205C.15/19
STRAIGHT CONNECTIONS
TAPERED HAUNCHES
CURVED KNEES
A
K,V,W
L
M
C
D
B
G,H,I
1.8
1.6
1.4
1.2
1.0
.8
.6
.4
.2
0
$\frac{M_r}{M_{r(y)}}$
2.0
δ/δ_y
KEY
M POINT OF FIRST CRACK (See Table 4)
NO FRACTURE IN THIS TEST

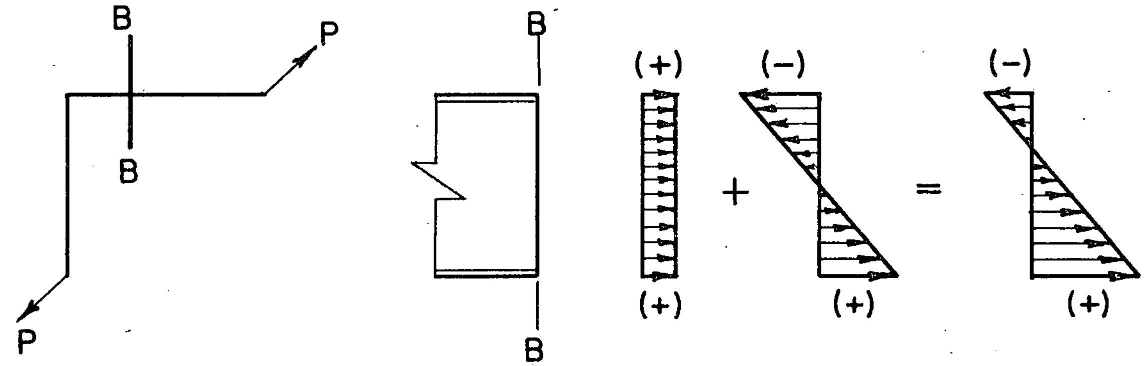
60

205C.15/20

(a) Compression Loading



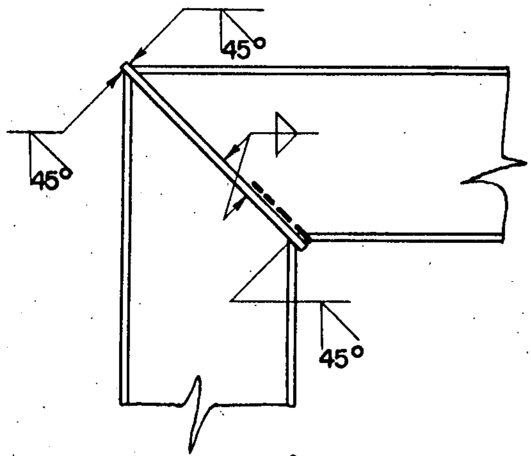
(b) Tension Loading



(c) Comparison of Approximate Moments (in-k.) at Local Buckling

Type of Load	CONNECTION								
	A	K	L	M	D	E	F	V	W
Compression	520	530	590	530	710	785	760	440	(440)
Tension	—	—	685	665	910	950	860	585	485

205C.15/21



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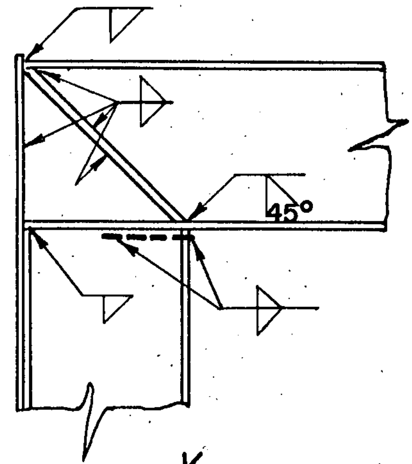

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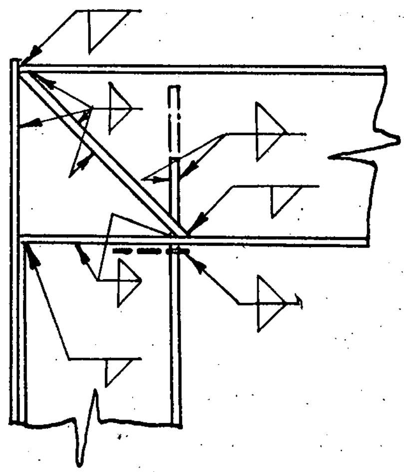
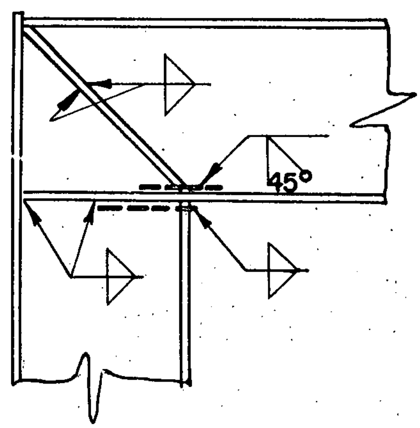


L & M



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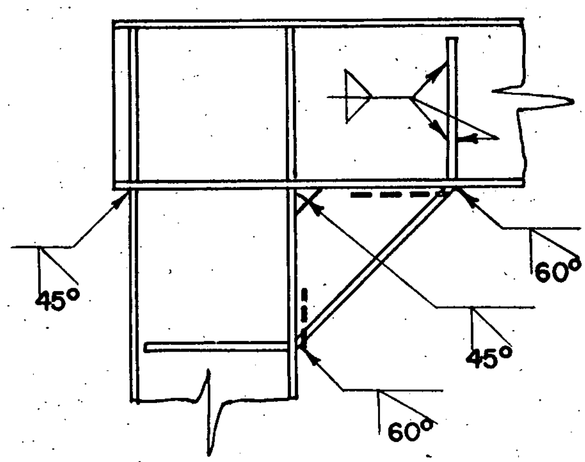
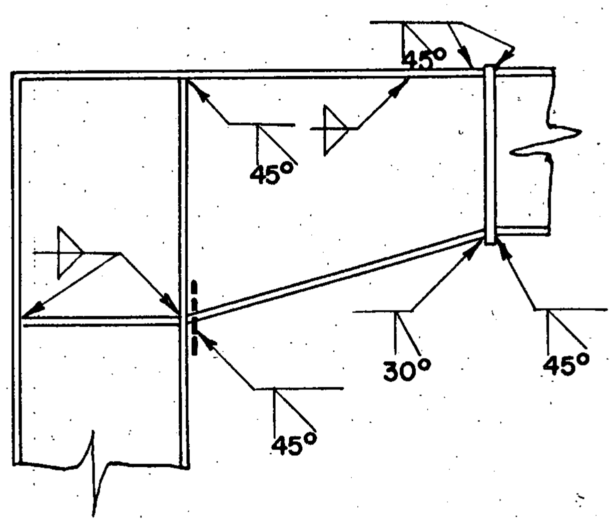


D



C