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STATICAL BEHAVIOR OF CROPPED-END CONNECTIONS IN TUBULAR TRUSSES

BY

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ABSTRACT

The statical behavior of seven Pratt-type tubular truss connections was investigated experimentally. Each connection specimen comprised a square hollow-structural-section chord segment and two round hollow-structural-section web segments. The ends of the webs were cropped and welded in the plane of the truss.

The ultimate strengths of the connections were found to be comparable to those of conventional connections. End-cropped connections with no web member overlap were found to be unacceptably flexible. However, lapping of the web members was found to substantially increase the connection stiffness.

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Materials for all test specimens were provided by the Steel Company of Canada, Limited.

Welding of all specimens was done at the Dominion Bridge Plant in Winnipeg.

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NOTATION

B outside diameter (width) of web tube

D outside depth (diameter) of chord tube

d outside diameter (width) of web tube

d_c outside diameter of compression web tube

d_t outside diameter of tension web tube

e joint eccentricity (distance from chord axis to intersection of web axes)

F_y specified minimum yield point or yield strength

T thickness of chord tube

t thickness of web tube

U_c ultimate load of compression web tube

W_c working load of compression web tube

W_v vertical force component in web tube on chord tube

CHAPTER 1

INTRODUCTION

1.1 General

In recent years, Hollow Structural Sections (HSS), or tubes, have been used extensively in truss construction, because tubular trusses are very economical and their behavior has become known through research and practical experience.[1] However, unless a special profiling machine is available, the fabrication cost of tubular truss connections is usually high if the ends of webs require profiling. The cost may be substantially reduced by cropping (flattening and cutting symmetrically in one stroke) the ends of the webs and welding them along the axes of the chords. Although such cropped joints have been successfully used for secondary joints carrying light loads,[1,3,5] their behavior requires a thorough investigation before they can be safely used in primary joints.

As a result, a program of investigation of cropped connections in tubular trusses has been recently undertaken at the University of Manitoba, under the sponsorship of CIDECT and the Steel Company of Canada, Limited. Tentatively, the program has been divided into three phases.

The first phase, which has been recently completed,[7,8] compared somewhat qualitatively, the statical behavior of cropped-end tubular truss joints with the geometries shown in Fig. 1.1. The different type numbers designate different parameters.

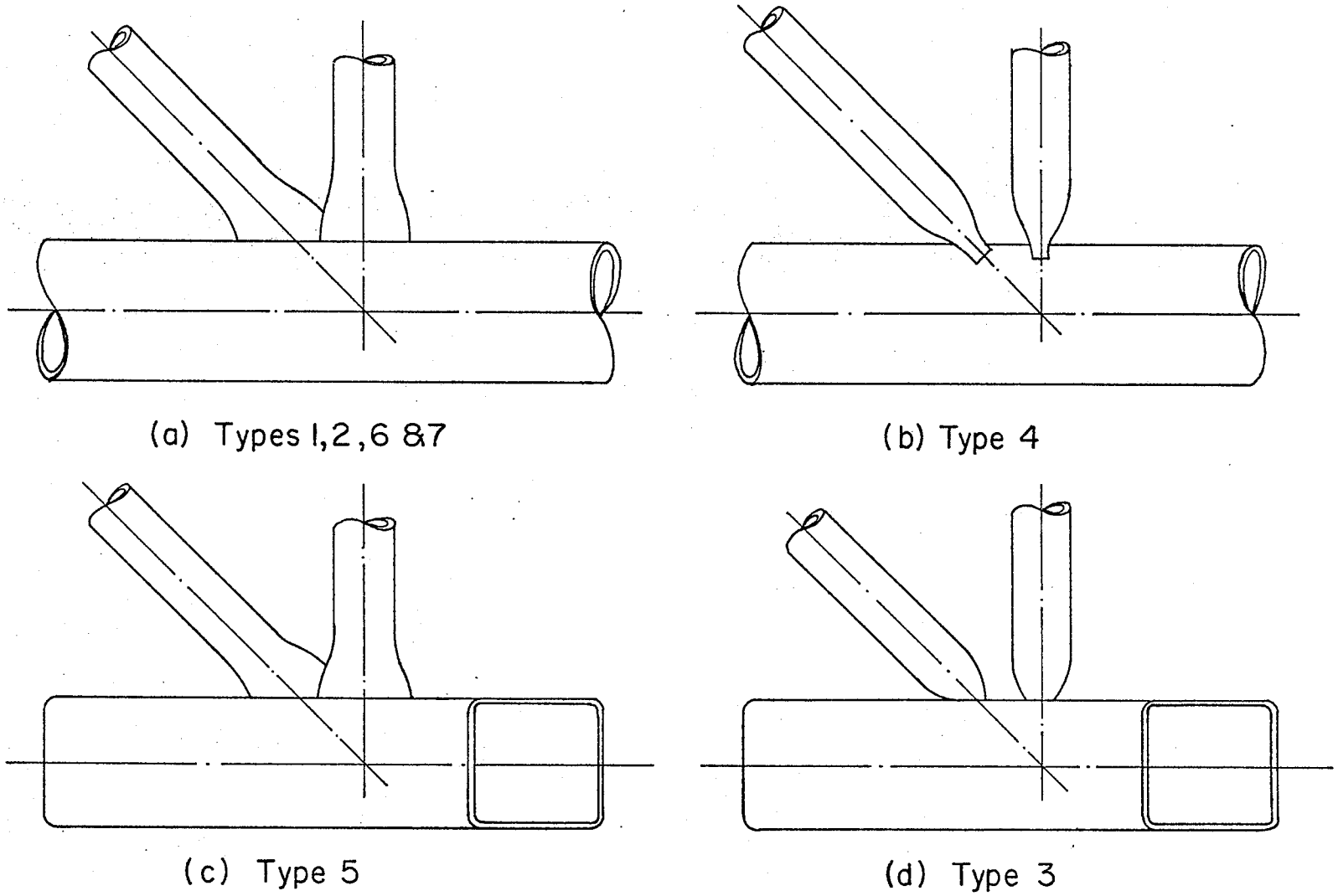


FIG. 1.1 CONFIGURATIONS OF SPECIMENS TESTED IN PHASE I

The second phase, which is in progress, is intended to investigate in more detail the statical behavior of joints of the types 1,2,5,6, and 7 shown in Fig. 1.1(a) and (c).

It is intended that the third phase involve a study of the behavior of the most efficient joints, under repeated loading.

The present study constitutes an initial part of the second phase.

1.2 Object of Investigation

The object of this experimental investigation was to determine the statical behavior of tubular truss connections composed of round webs with cropped ends welded along the axes of square chords, as shown in Fig. 1.1(c). The majority of specimens tested had zero overlap of the webs.

1.3 Limitations of Investigation

The limitations of this investigation were:

1) the investigation was entirely experimental, no analytical study of joint behavior was undertaken,
2) static loading only was considered,
3) a Pratt-truss joint configuration was considered,
4) each test specimen was composed of a chord section and two webs,
5) all chords were square HSS, all webs were round HSS; all materials had a minimum yield strength of 50 ksi,
6) the connected ends of all webs were cropped, not flattened

and sawn,

7) no preload, representing an axial force, was applied to the chord.

CHAPTER 2

TUBULAR TRUSS CONNECTIONS

2.1 Introduction

In this chapter, the advantages and disadvantages of tubular trusses are discussed in detail. Some methods of overcoming the disadvantages are presented. Special emphasis is given to end-cropping as an alternative to end-profiling. The measures and parameters of joint behavior are also discussed.

2.2 Advantages and Disadvantages of Tubular Trusses

Hollow Structural Sections (HSS), or tubes, have many advantages over other types of rolled sections when used as truss members. A weight saving of the order of 20 per cent is quite common for tubular trusses.[21] This is not only because HSS have high compressive, bending, and torsional resistance as well as good lateral stability, but also because they are available in a wide range of sizes and strength grades. Moreover, the closed cross-sectional shape of HSS minimizes the exposed area, thus dirt collection, the costs of fire and corrosion protection, and painting. Finally, tubular trusses with exposed structural members usually are aesthetically pleasing.

The economy of tubular trusses is often reduced by two main disadvantages - both of which are related to the joints.

The first of these is the relatively high cost of fabrication of some joints, especially those involving intersections of curved surfaces, which require multiple cutting or profiling.

The second is a possible reduction of member working load due to excessive deformation of the loaded faces of the chord wall.[1,7,11]

2.3 Substitutes for End-profiling

Some methods that can be used to avoid profiling the ends of webs are the following.

1) Gusset plates may be used for connecting the webs to the chords. However, the use of gussets is expensive, as it requires extra material and slotting of the ends of webs. Gussets cause stress concentrations and should be avoided particularly for joints subject to fatigue loading.[1,3,9,10]
2) Rectangular or square HSS, which have recently begun to be manufactured, may be used instead of round HSS. However, these sections are structurally less efficient than round HSS.
3) The ends of webs may be flattened in the plane of the truss and welded to the chords.[1-8] This eliminates profiling but requires an extra operation.
4) The webs may be cropped and welded along the lengths of the chords.[4-8] The cropping not only eliminates profiling, but also minimizes the cutting time, as the tubes can be rapidly squeezed and cut in one operation in a cropping machine, which essentially comprises two V-shaped jaws attached to a hydraulic

press. Moreover, the welding is simplified, since only fillet welds along straight lines are required. Finally, a greater combination of HSS forming the joint is possible, as the welding space on the chord face is increased by cropping the ends of the webs.

However, the economy in fabrication cost of cropped joints is expected to be offset somewhat by the requirement for slightly larger webs due to the flexibility of the joints and an increase in the buckling length of the compression web.

2.4 Local Deformation of Chord Wall

If the wall of the chord in a tubular truss joint is relatively thin, the local deformations of the connected face of the wall at the joint may become excessively large at the working loads of the members.[1,5,7,11,12] The local chord-wall deformation occurs because the chord wall is relatively flexible, and because relatively small webs are usually connected, with a moment arm between them, along the middle of the chord face.

In order to prevent the chord-wall deformation from becoming excessive at the working loads of the webs, thicker chord walls can be used.[1,5,7,11,14] But this is obviously not an economical solution from the standpoint of material cost. An alternative solution is to provide an overlap between the webs, as this has been found to decrease the chord-wall deformation.[1,5,7,11,14] Although this solution requires multiple cutting, it is not expensive if special multiple cutting machines are available. Another method of reducing the chord-wall deformation is to

use webs of larger diameters (widths) so that the normal force components of the webs tend to be transferred to the side walls of the chord.[1,11,14] But this solution is not applicable to connections involving webs flattened in the plane of the truss.

2.5 Modes of Joint Failure

A tubular truss joint may fail in one of several modes, depending on which joint component - chord, compression web, tension web, weld - is the weakest. Different modes of joint failure are:

1) excessive local deformation of chord wall,[5-8,11-13]
2) buckling of compression web,[5-8,12,13]
3) fracture of weld,[5-7,11]
4) rupture of tension web,[6,10,11]
5) collapse of walls of compression web.[6,13]

Excessive local deformation of the chord wall is the most common type of failure for a typical, economical tubular truss, which usually has relatively thin, and thus flexible, chord walls.

Excessive chord-wall deformation is defined differently by different investigators. For example, Eastwood et al[12] arbitrarily defined a joint failure as a deflection of the compression web into the chord of 0.05 inch, because such a deformation is very clearly visible.

Although a chord-wall deformation of 0.05 inch is excessive for a chord of a certain width, it becomes less excessive for a chord of larger width. Therefore, the maximum allowable deformation of the chord wall should be limited to a certain proportion of the chord width. This is analogous to limiting the maximum allowable deflection of a beam to a proportion of the span.

However, an appropriate percentage to be used for defining the maximum allowable chord-wall deformation is somewhat arbitrary. It is proposed that the maximum allowable deformation of chord wall be limited to one per cent of the chord width. Thus, for 6 x 6 and 4 x 4 chords, which are used in this study, the local deformation limits are 0.06 and 0.04 inches, respectively; close to 0.05 inch used by Eastwood et al.

Incidently, the same percentage is used for the maximum allowable variation in cross-sectional dimension of HSS.[21] This is suitable because it sets a reasonable limit on the variation of the cross-sectional area of HSS. However, the significance of the maximum allowable chord-wall deformation thus defined depends on how much this local deformation affects the overall deflection of the truss.

In general, the chord-wall deformation is not uniform; therefore it causes a rotation at the ends of the webs and a buckling failure of the compression web usually results.

Sometimes a fracture of the weld at the lap between the webs or under the tension web close to the compression web also occurs.

If the tension web is the weakest joint component, the joint may fail by rupture of the tension web.

On the other hand, if the walls of the compression web are relatively thin, failure by collapse of the walls of the compression web may occur. However, this type of failure may be prevented by requiring that the diameter of the compression web not exceed $3300/F_y$ times the thickness of the section.[20]

2.6 Measures of Joint Performance

The performance of a tubular truss joint, as for other structural elements, is usually measured by three criteria - strength, stiffness and stability.

Two different definitions of joint strength are commomly used. Because of different methods of load application, some investigators have defined joint strength in terms of the maximum load in the compression web,[13] while others have used the maximum tension web load.[3,5,7,11]

In this study, although the load was applied to the tension web, the strength of the joint is defined as the maximum normal force component of the webs on the chord. The normal direction

was used because the force in this direction was the main cause of chord-wall deformation. Furthermore, the ultimate strengths of the test specimens in this study were governed by the buckling strengths of the compression webs, which were normal to the chords.

In order to compare the strengths of different joints, the joint strength is generally expressed in non-dimensional form. Two forms are commonly used: one is joint efficiency; the other, joint load factor.

The joint effiency has been defined as the ratio of the strength of the joint to the tensile strength of the diagonal member.[3,7,11] Sometimes the efficiency is calculated for the member that yields first by considering the total axial load in that member at the proportional load, assuming a yield stress of 16 tons per square inch.[13]

The joint load factor can be defined as the ratio of the ultimate load to the working load in the tension web.[1,5,7] Alternatively, the ratio of these loads for the compression web can be used.[12,14] It was found from the results of the present tests that the joint load factors based on the two different definitions differed by less than ten per cent.

The values of joint load factor based on the ultimate loads and working loads of the tension webs are relatively easy to calculate, because no slenderness ratios need be taken into account. However, the values are not very accurate in cases where the ultimate strengths are governed by the buckling

strengths of the compression webs. In such cases, the joint load factor so defined changes if the size of tension web is changed - even when the ultimate load of the compression web is kept approximately constant by using the same compression web.

Therefore, the joint load factor to be used in this study is defined as the ratio of the ultimate load to the working load for the web that fails first. The working load of the tension web is calculated by multiplying its nominal cross-sectional area by 0.6 of the minimum yield strength. The working load of the compression web is calculated by assuming an effective length factor of 0.7 and using the allowable stress given in Reference 20.

The joint load factor, rather than the joint efficiency, is used in this study because of the present trend toward ultimate strength design. Moreover, the joint load factor is based on the minimum yield strength, which is known by the designer; whereas the joint efficiency is based on the ultimate strength of steel, which may vary somewhat.

In addition to strength, a joint should have sufficient stiffness. Two types of joint stiffness are considered. One is the local stiffness, the other is the overall stiffness. Both of these can be expressed in terms of load per unit deflection. Stiffness can also be discussed in terms of moment per unit rotation but this definition is not applicable here.

The local stiffness of a joint is represented by the minimum ratio of the transverse load on the chord to the transverse deflection of the connected face of the chord wall relative to the opposite wall.

The overall stiffness of a joint can be defined as the load per unit deflection of the joint in the direction perpendicular to the chord.

The two types of joint stiffness can be shown graphically by plots of the deformation of the wall and the deflection of the joint, versus the normal loads. The stiffnesses of the joints should be sufficiently high that the maximum allowable deflection at the mid-span of the truss is not exceeded and the local deformation of the chord wall is not excessive.

The load-deflection curves not only show the stiffness of the joints, they also indicate the loads at which the chord walls and the joints become unstable and yield or fail by local or overall instability.

2.7 Parameters Affecting Joint Behavior

The behavior of a tubular truss joint is affected by the following joint parameters,[1,3,5-8,11-15] with reference to Fig.2.1,

1) the ratio of web diameter (width) to chord width (diameter), $\frac{d}{B}$,
2) the ratio of chord width (diameter) to chord thickness, $\frac{B}{T}$,
3) the joint eccentricity, e; gap and lap,
4) the angles of inclination of the webs,
5) the value of the axial stress in the chord,
6) the ratio of web thickness to web diameter (width), $\frac{t}{d}$, and

7) the length, degree of flattening and arrangement of the flattened ends - if these are used.

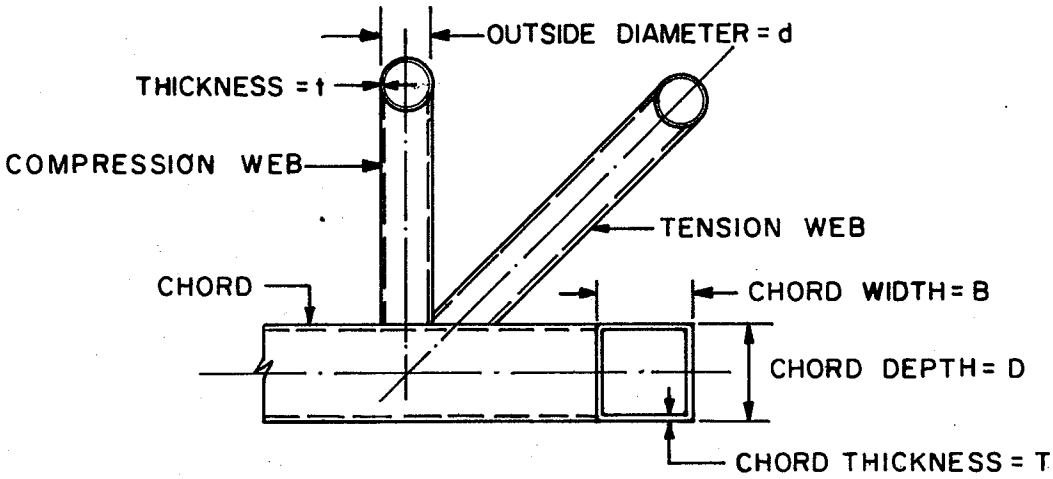
It has been found[3,5,6,9,12-15] that the first three parameters are more significant than the others. The effects of the parameters on the joint behavior are as follows.

1) The ratio of web diameter to chord width denotes how close the side walls of the web and chord are to each other and thus how well the normal load component from the web can be transferred to the lateral chord walls. Therefore, the larger this ratio the higher the buckling or tear-out strength of the chord.[6,11,12]

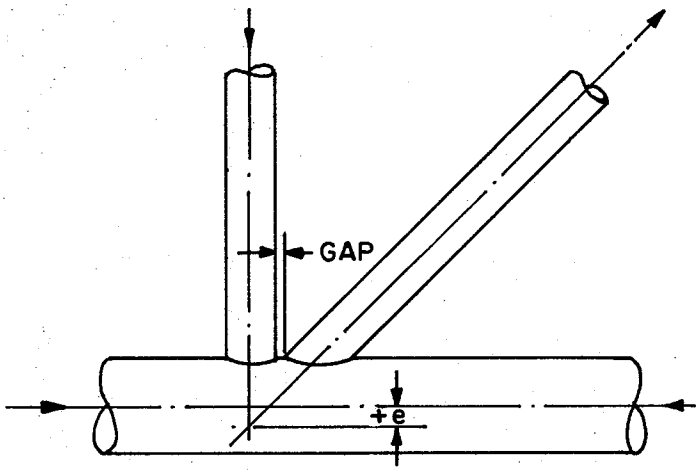
2) The width-thickness ratio for the chord represents the flexibility of the joint. The larger this ratio, the smaller the resistance to local buckling deformation.[6,11,12]

3) The joint eccentricity is the distance from the chord center line to the point of intersection of the web center lines. If the point of intersection lies on the chord center line, the eccentricity is zero. If it is inside the chord center lines, the eccentricity is negative; otherwise the eccentricity is positive. The three types of joint eccentricity are illustrated in Fig. 2.1. In this figure, the definitions of gap and lap between the webs, and the notation for the dimensions of truss members are also given.

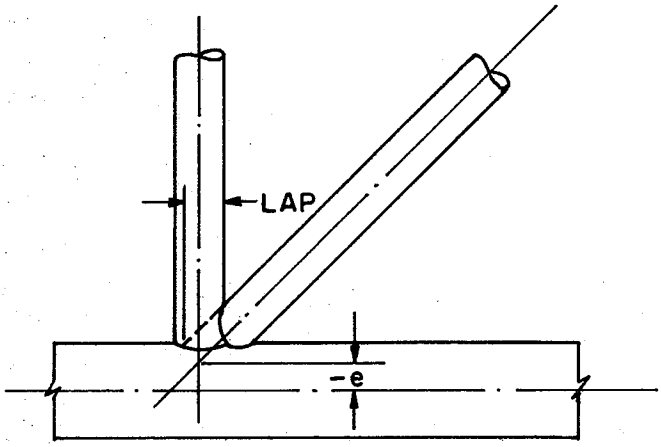
A joint with negative eccentricity is usually stronger than a joint with zero eccentricity, which in turn is stronger than a positive eccentricity joint. This is because a negative eccentricity joint usually has either a lap or at worst,



(a) ZERO ECCENTRICITY



(b) POSITIVE ECCENTRICITY



(c) NEGATIVE ECCENTRICITY

FIG. 2.1 JOINT ECCENTRICITY

a small gap between the webs. This allows a portion of the loads in the webs to be directly transferred between them instead of through the chord wall.[3,6,11-13]

4) The angle of inclination of the web affects the contact length between the web and the chord tube. If the angle of inclination is decreased, the contact length is increased and therefore the normal loading intensity of the web on the chord is decreased. Hence, the lesser the inclination of the diagonals, the higher the strength of the chord.[6]

5) The axial stress in the chord arises from the action of the truss as a whole and is additive to the stresses produced by the forces transferred from the webs to the chord. Therefore, the larger the axial stress in the chord, the smaller the strength of the chord wall.[6] However, tests have shown that the axial stress in the chord does not significantly affect the joint strength.[11,12]

6) The ratio of web thickness to web diameter represents the rigidity of the web. Hlavacek[6] reasoned that the larger the web rigidity, the smaller the strength of the joint because there arise higher secondary stresses from the bending of the joint.

7) If the ends of the webs are flattened or cropped, the length, degree of flattening and orientation of the flattened parts become joint parameters.

2.8 Flattened-end and Cropped-end Connections

Jamm et al[2] have suggested that the flexibility of pipe sections used as truss chords should be preserved by minimizing the weld - a common location of joint failure at the time the suggestion was made - so that it runs over a relatively narrow part of the chord surface. If the web pipes are relatively large, this can be achieved by flattening their ends, longitudinally, to a maximum of 0.20 to 0.25 of the chord pipe diameter. The end flattening may result in an overlapping of the webs, causing only partial shear force transfer through the chord pipe wall. It was claimed that if the depth of the direct connection between the compression and tension webs is greater than 0.15 of the chord pipe diameter, the full joint efficiency (force transfer through the joint equal to the maximum allowable load) can be realized.

Hlavacek[6] found that the strengths of tubes with flattened ends, no matter whether connected longitudinally or transversely to the chord tube, were decreased by 20 per cent on the average. He suggested that the flattened lengths of the webs should not be more than 0.8 to 1.2 inch, for otherwise sideway buckling would take place at about 50 per cent of the tube strength. He

also stated that cross-wise flattened ends were less safe against sideway buckling than were the length-wise flattened ones, since they could not transmit bending moment in the plane of the joint.

Mouty[22] stated that, in order to obtain the normal effective length of 0.7 in all planes for a circular web tube with circular chords, the degree of end flattening must not be greater than one-third of the web tube diameter and one-quarter of the chord tube diameter; while the web tube thickness must be at least equal to that of the chord.

According to the German Specification and the British Standard, the minimum allowable value of the ratio of web diameter to chord diameter, $(\frac{d}{D})$, is 0.25 and 0.33, respectively.[9] For a flattened-end connection, Bouwkamp[9] suggested that the web diameter, d, be replaced by the reduced width of the flattened end of the web.

Anderson[5] found that the maximum local chord stresses in connections involving cropped-end webs were much greater than those with profiled webs. The intensity of stress in the chords with cropped webs appeared to be reduced by increases in the amount of negative joint eccentricity (as defined previously) and the degree of robustness of the direct connection between strut and tie. (The two properties are directly interrelated.) The adverse stress distribution of cropped-end connections did not in general lead to lower ultimate load factors, because (i) the joint could relieve overstress by

redistribution, (ii) the compression web collapsed before the full plasticity of the chord wall was developed, (therefore the ultimate-load factors obtained were to some extent a measure of the strength of the strut under eccentric load), (iii) the normal loads at the joint were distributed over greater axial lengths of chord, (iv) part of the transfer of normal load at the joint occurred through the direct connection between the webs, (v) the chord stress was mitigated by the incorporation of negative joint eccentricity. Anderson recommended cropping as a safe and economical substitute for profiling in joints composed of round tubes of similar proportions to those investigated, provided that the loads were static. Further, a negative joint eccentricity should be provided so that a robust direct connection between the webs results. Since the direct connection contributes to a maldistribution of stress, such a connection should be subjected to static loading only.

It has been found[7] that the stiffnesses and yield loads of flattened-end connections involving square chords were approximately one-third and one-quarter, respectively, lower than those of connections involving round chords. Furthermore, their ultimate loads were within ten per cent of each other. The joint load factors for flattened-end joints with overlapping web members were not significantly lower than those for similar conventional connections. The stiffness, yield load and ultimate load were not significantly affected by either the

direction, or the method, of end flattening. The joint stiffness, but not the yield load or ultimate load, was significantly affected by the joint eccentricity.

CHAPTER 3

TEST PROGRAM

3.1 Joint Configuration

Although Warren (K-type) truss joints are more commonly used in truss construction, Pratt (N-type) truss joints were selected for this study. There are several reasons for the selection.

Firstly, the Pratt truss Joint provides a more general joint configuration, as it has both vertical and inclined webs.

Secondly, the joint also provides a more severe test situation, since the force in the compression web distributes over a smaller surface area of the chord. In addition, it is more likely to require a positive joint eccentricity in order to avoid overlapping of the webs. Therefore, design criteria for the Pratt truss joint may be conservatively applied to the Warren truss joint.

Finally, the selection facilitates comparisons with results from other investigations, as most of them were based on the Pratt truss joint configuration.[1,5,7,8,10,12-15]

Although connections with square chords were found to be weaker than those with round chords,[7] square chords were investigated in this study because they facilitate attachment of other roof components in roof truss construction. Since the joint behavior is not significantly affected by different methods

of end flattening (flattening and sawing, shearing, and cropping), and since cropping is the most efficient method, the webs for all specimens were cropped. As the direction of end flattening does not significantly affect joint strength and because it involves simpler fabrication, a direction of end flattening in the middle plane of the truss was used for all connections in this study.

3.2 Specimen Design

The design of the test specimens was influenced by several factors, including the geometry and loading of a typical prototype truss, joint parameters to be investigated, available member sizes and loading equipment.

The geometries and loadings of the two Pratt trusses shown in Fig. 3.1 were used for the proportioning of the specimens and selection of member sizes. The ratios of span to depth of the trusses shown are quite typical. The loadings were chosen to suit available member sizes and the capacity of the loading equipment (200 kips).

Member sizes were selected by following the method presented in References 1 and 20. However, provisions regarding maximum permissible normal loads on the chord faces were not considered. An effective length factor of 0.9 was assumed for chords, and 0.7 for webs.

Relatively large HSS with thin walls were selected for chords, in order to represent an economical design and a relatively severe loading condition, and to facilitate deformation measurements.

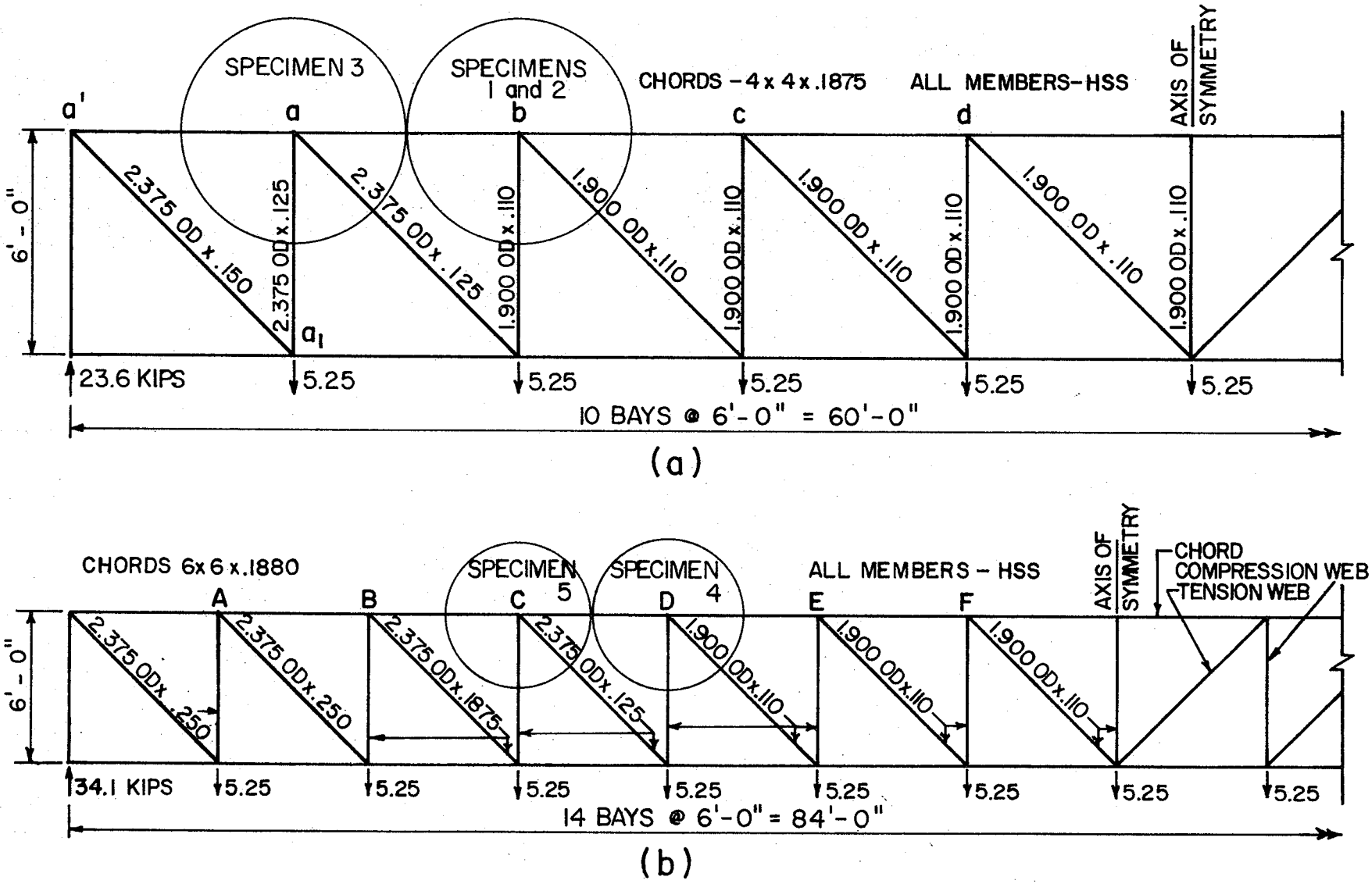


FIG. 3.1 TRUSSES ASSUMED IN DESIGN OF TEST SPECIMENS

Webs were chosen such that several different ratios of web diameter to chord width resulted.

3.3 Joint Parameters

The effects of different joint parameters on the behavior of tubular truss joints can be separately investigated by varying one parameter at a time. A complete investigation of these effects would involve a large number of specimens. Since different parameters were expected to influence joint behavior to different degrees, it was felt desirable to determine, as a start, their relative significance.

Ten specimens were initially proposed for determining the relative significance of four different parameters - ratio of chord width to thickness, ratio of web diameter to chord width, ratio of lap (and/or eccentricity) to chord depth, and ratio of web diameter to web thickness.

Unfortunately, some of material sizes required were not immediately available and the proposed specimens had to be modified to use available materials. In addition, it was felt that a maximum of five specimens could be tested in the present study.

Because of the small number of tests, the variations of parameters were necessarily limited. It was decided that joints with no lap would be investigated first, as they need no multiple cutting, and also represent relatively severe loading conditions.

The different values of joint parameters used in this study

are given in Table 3.1 .

It will be seen that two different ratios of width to thickness of chord were used. Four different ratios of web diameter to chord width were involved. Using a lap of one inch and zero, the joint eccentricities of the specimens were positive and varied from zero to 2.16 inches. The ratios of web diameter to web thickness were different by only about ten per cent.

With these variations of parameters, it was possible to quantitatively determine the effect of joint eccentricity and lap by comparing specimens 1 and 2. A direct quantitative measurement of the effects of other parameters was not possible, as more than one parameter was varied at a time. However, an indication of the relative importance of the several parameters was obtained. Furthermore, the test results indicated the statical behaviour of five different cropped joints with and without laps, thus showing the performance of a variety of geometries for this type of joint.

Specimen 1, the only one with a lap, was tested for comparison not only with specimen 2, which had a different lap (eccentricity), but also with specimens 5A1 and 5A2, which had been tested in an earlier study.[7]

3.4 Specimen Details

The member sizes and the different ratios of joint parameters are given in Table 3.1.

TABLE 3.1 DESCRIPTION OF SPECIMENS

Specimen	Chord HSS-BxDxT (in)	Webs HSS-d O.D. x t (in)	Lap (in)	Joint Ecc., e (in)	$\frac{B}{T}$	$\frac{d}{B}$	$\frac{e}{D}$	$\frac{d}{t}$
1	4 x 4 x .1875	1.900 OD x .110	1	0	21.3	0.475	0	17.3
2	"	"	0	1.33	21.3	0.475	0.332	17.3
3	"	2.375 OD x .125	0	2.16	21.3	0.594	0.540	19.0
4	6 x 6 x .1880	1.900 OD x .110	0	0.34	31.9	0.317	0.057	17.3
5	"	2.375 OD x .125	0	1.16	31.9	0.396	0.193	19.0

Each test specimen was composed of a square HSS chord with two round HSS webs with clipped ends welded parallel to the center line of a chord face, as illustrated in Fig. 3.2 .

Removable bearing plates were used at the lower end of the chord and at the end of the compression web. The distance from the bottom of each bearing plate to the intersection of the center lines of chord and compression web was three feet. This length of chord and compression web was used because it was assumed, as in other studies[7,8,12-15], that truss members tend to deform with points of inflection at approximately the mid-length of each member.

The length from the other end of the chord to the panel point was two feet, and no prestressing force was applied to the chord.

Because of the size of the loading frame, the length from the loading pin of the tension web to the intersection of the center lines of the webs was limited to the range of 2 feet - 8.3 inches to 2 feet - 9.9 inches.

The member sizes for specimens 1 and 2 were designed to represent joint b shown in Fig. 3. 1 (a), while specimen 3 corresponded to joint a. Specimen 4 was intended to represent joint D of Fig. 3. 1 (b), and specimen 5, joint C. Joints A and B of Fig. 3. 1 (b) were also designed for testing but the web material was not available.

3.5 Materials

All round Hollow Structural Sections (HSS) used for webs

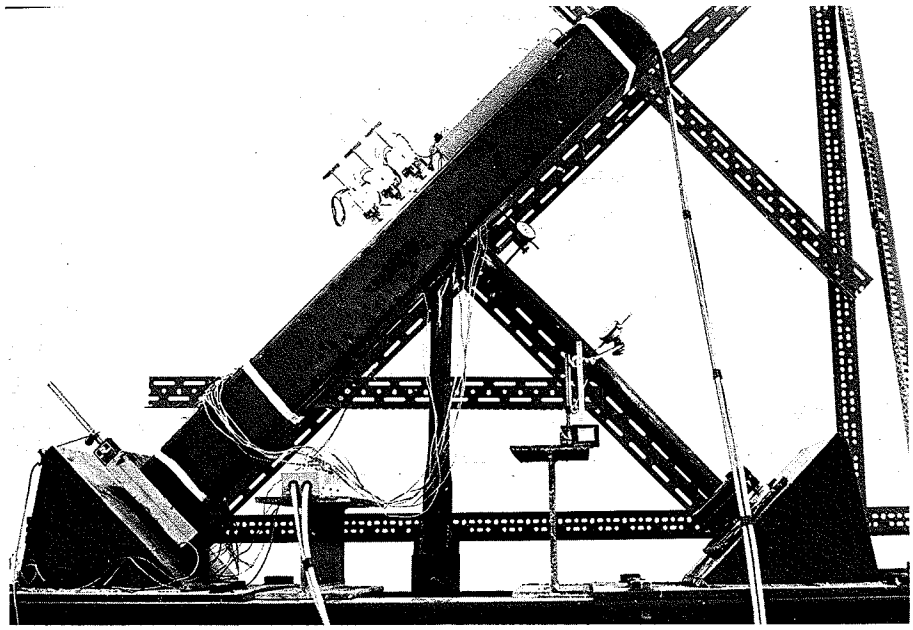


Fig. 3.2(a) General Configuration of Test Specimen (No. 4)

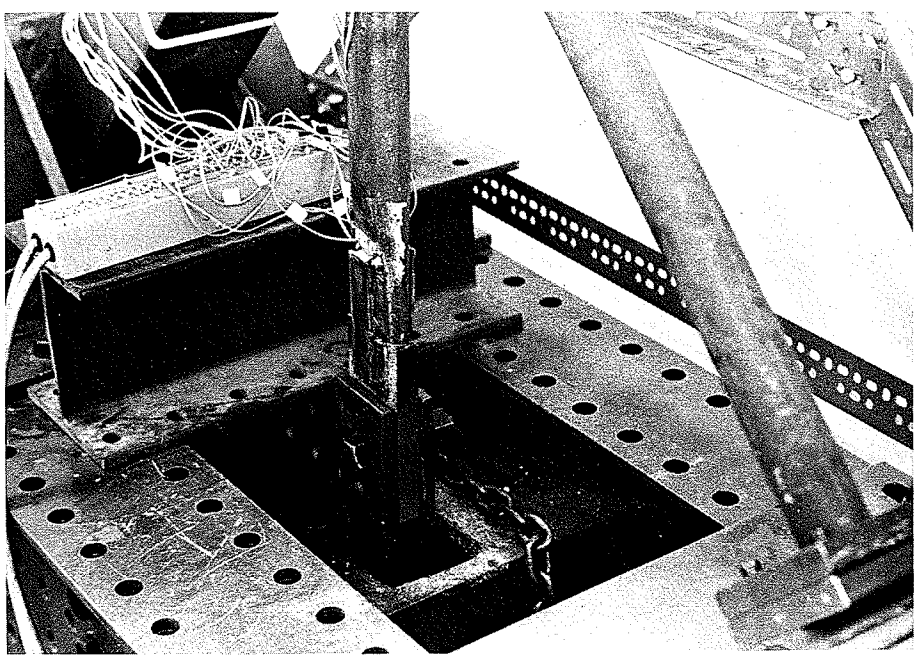


Fig. 3.2(b) System for Loading Tension Web

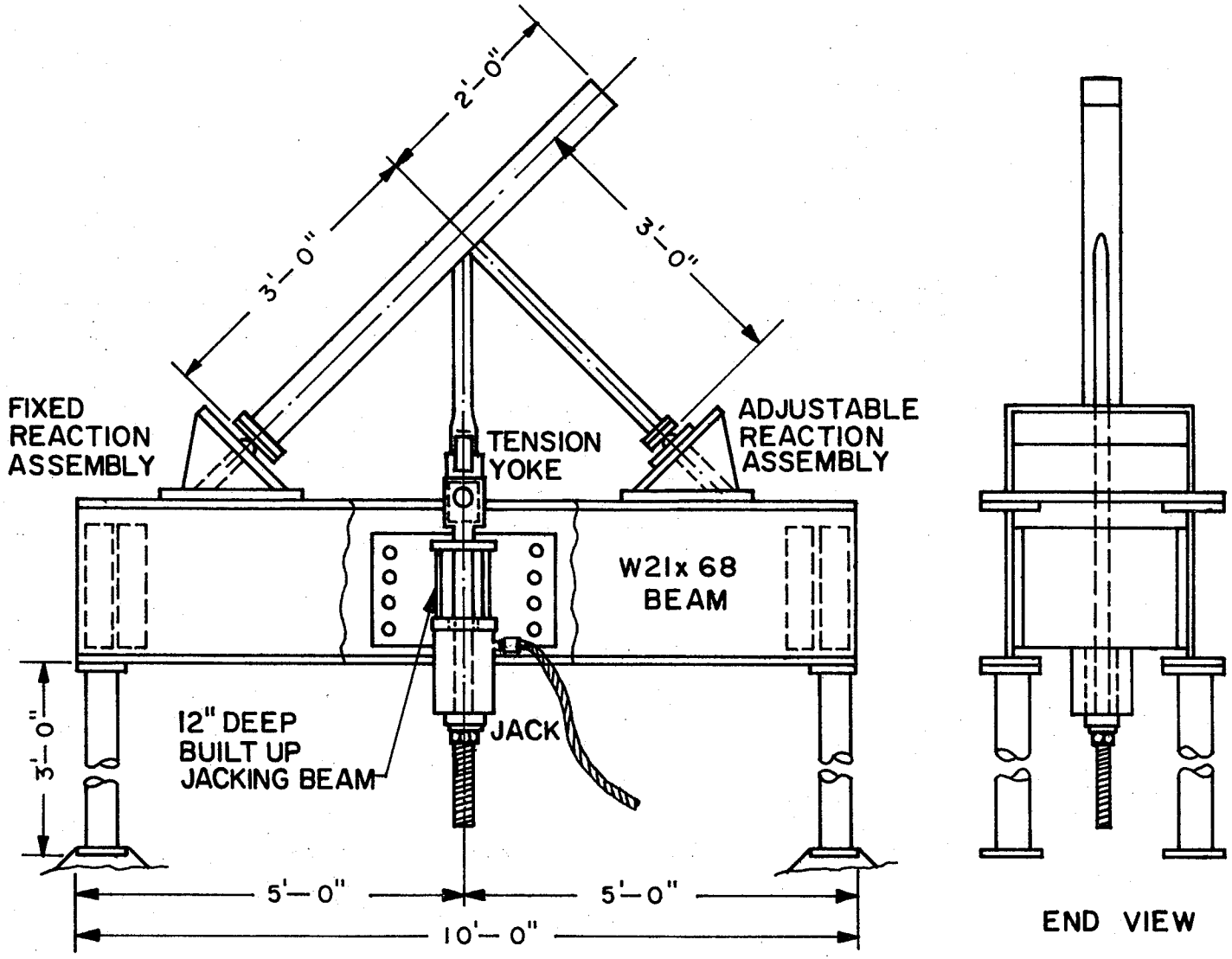


FIG. 3.2(c) LOADING FRAME

were hot-formed, conforming to CSA Specification G40.16-1969, Grade 50. Mill reports and tension tests conducted at the University of Manitoba indicated yield strengths in the range of 53.2 ksi to 56.3 ksi and ultimate strengths in the range of 72.5 ksi to 78.8 ksi.

The HSS-4x4x.1875 used as chords were cold-formed, conforming to CSA Specification G40.17-1969. Tension tests showed a yield strength of 64.0 ksi and an ultimate strength of 70.0 ksi.

The HSS-6x6x.1880 used as chords were cold-formed, conforming to CSA G40.21 Hollow Structural Steel Sections Grade 50W Class H. According to Mill reports, these tubes had yield strengths in the range 63.7 ksi to 64.9 ksi and ultimate strengths in the range 80.2 ksi to 81.7 ksi.

3.6 Specimen Supports

The supports for the chord and compression web at the points of inflection were designed as rockers, whereas the load was applied to the tension web through a pin, as shown in Fig. 3.2(b). Since the positions of the ends were kept approximately constant relative to each other, the boundary conditions would approximate the conditions in a rather stiff truss. Had the ends been permitted to move relative to each other by means of rollers and a pin, the boundary conditions would have approximated the conditions in a truss where the members are relatively slender. The rockers and pinned end conditions were chosen because of their simplicity and because the effects of the different boundary conditions on the joint behavior were found to be insignificant.[12]

3.7 Loading Plate Design

Before the specimens were fabricated, a series of tension tests was performed on tension web loading plate assemblies. The object was to obtain a simple and effective loading system for the tension webs to ensure that failure would occur in the specimen rather than the loading system.

Some of the assemblies failed prematuredly in the heat affected zone,as shown in Figure 3.3 .

In the course of the tension tests, it was found that the ultimate tensile strengths of the hollow structural sections were not affected even by complete end flattening.

Thus, since the strength of the tension web was found to be unaffected by end-flattening, a simple and effective connection between a pin-hole plate and a round hollow section was obtained by flattening one end of the hollow section to the thickness of the loading plate (3/4 to 1 inch thick) and using two side plates, as shown in Figs. 3.2, 3.3d. A small piece of flat bar was inserted and tack welded inside the tube beyond the flattened part to help keeping the tube in the cylindrical shape when it was loaded. This loading system was used for all specimens.

It was found that, in order to flatten the end of the tube to the thickness of the loading plate, it was better to insert a bar of appropriate size into the tube to obtain the correct depth,than to place the loading plate close to the tube, because of a spring-back action of the flattened part.

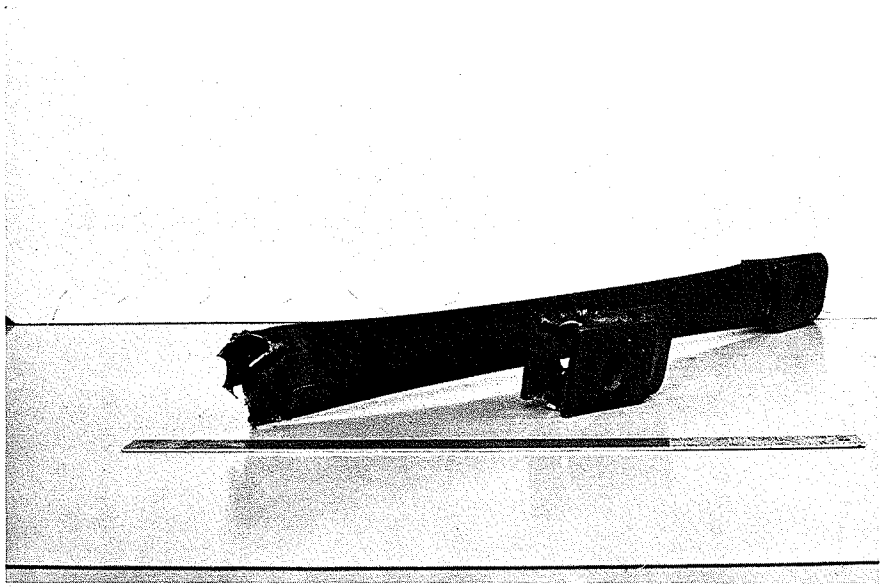


Fig. 3.3(a) Premature Failure in Heat-affected Zone

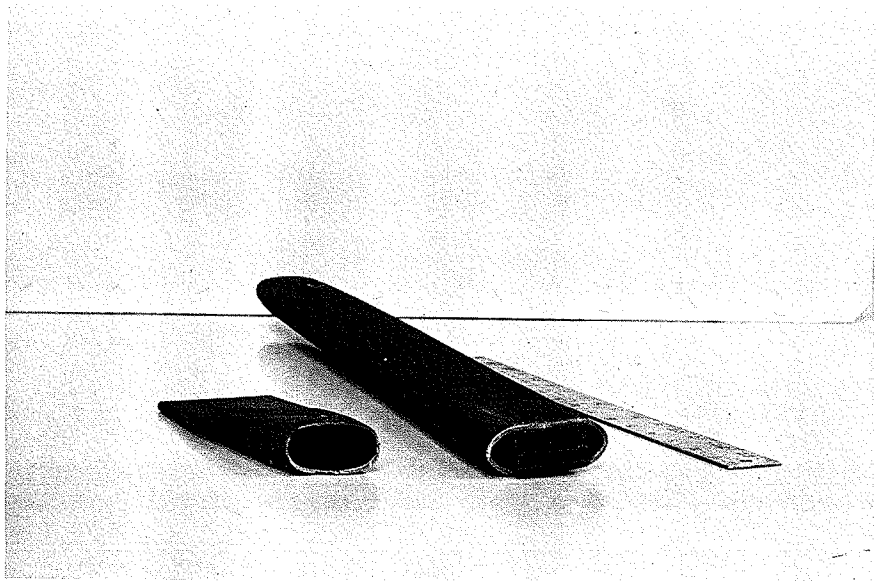


Fig. 3.3(b) Tensile Failure of Tube with Unequal Flattened Ends

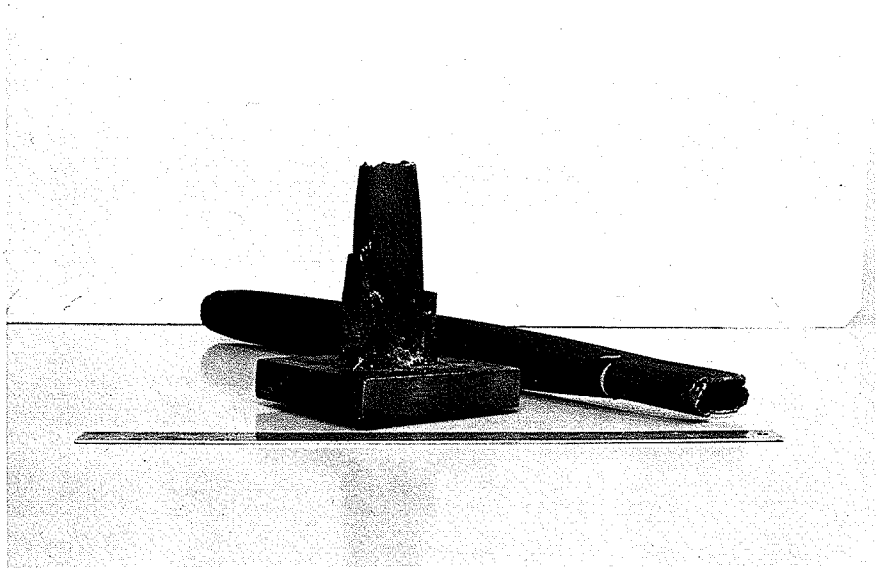


Fig. 3.3(c) Tensile Failure of Tube with Unflattened and Flattened Ends

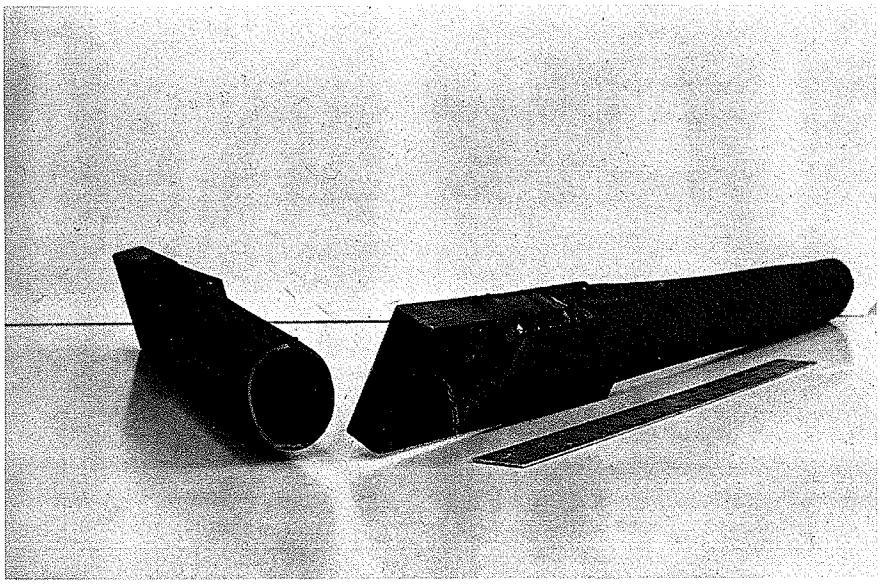


Fig. 3.3(d) Tensile Failure of Loading System Used in This Study

Test results on flattening of tubes are shown in Fig. 3.4.

It was found that a complete flattening of hot-formed seamless HSS at room temperature usually produced cracks at the edges and seams. If the tubes were flattened moderately (say, to half their diameters), no cracks occurred. Cracks that occurred in flattening hot-formed tubes were normally smaller than those in cold-formed tubes. It is clear that tubes with large diameters and thin walls can be flattened more easily and with smaller cracks than relatively small tubes with thick walls.

3.8 Fabrication

Each chord section was cut to size using a mechanical hacksaw. To provide for deformation measurement devices, three 1/4-inch diameter holes were drilled and tapped at each of three cross-sections on the face opposite the connected face of the chord. It was ensured that neither of these two faces had the seam.

The web sections were sawn a few inches longer than the design length. They were then cropped to size in a cropping machine at the Dominion Bridge Company Winnipeg fabricating shop. The cropping machine essentially consisted of two V-shape steel teeth attached to an ordinary hydraulic press. The teeth were designed for cropping relatively small tubes such that the cut surfaces were unsymmetrical and a good

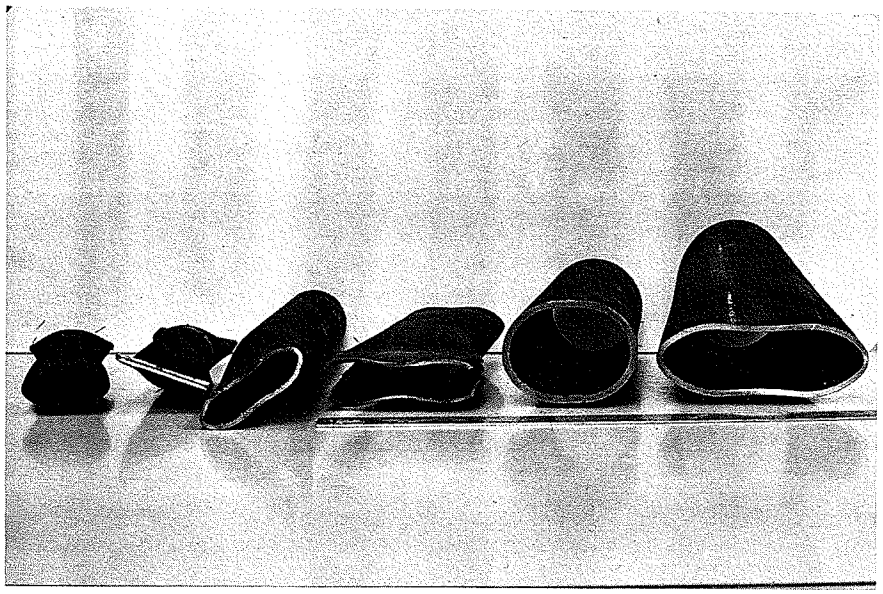


Fig. 3.4(a) Tubes with Flattened Ends

Fig. 3.4(b) Tubes with Cropped Ends

welding surface resulted on one side only (Fig. 3.4 b). Although welding web tubes on one side only may be satisfactory for joists with small webs, it may not be strong enough for trusses with relatively large member forces. Therefore the cropped surfaces were ground symmetrical for proper fitting and welding on both sides of the webs.

The sawn end of each tension web was then flattened parallel to the cropped line and connected to a loading plate as previously described.

Each specimen was tack welded in a jig, as shown in Fig. 3.5. To simulate typical shop practice, all final welding was performed by the same qualified welder at the Dominion Bridge Co. fabricating shop. An A.C. machine and 5/32-inch Lincoln Type E7018 electrodes were used for all welds.

3.9 Displacement Measurements

Two types of displacement measurement were employed. One involved the measurement of local deformation of the connected face of the chord wall, as this was usually the primary cause of joint failure. The other involved the overall displacement of the joint and the transverse displacements of the compression webs, as they indicated the overall stability and final failure of the specimen.

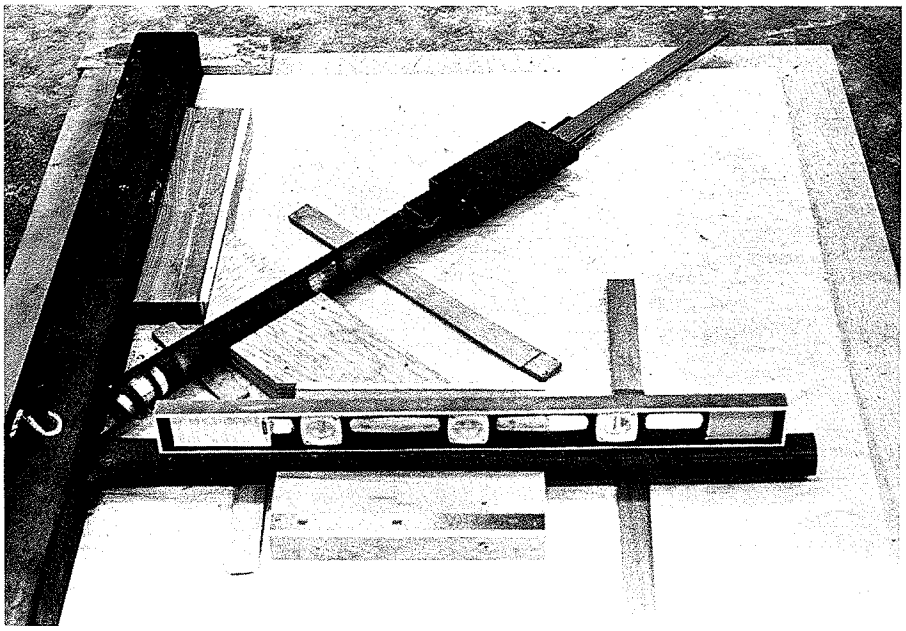


Fig. 3.5(a) Jig Used for Proper Assembly of Members

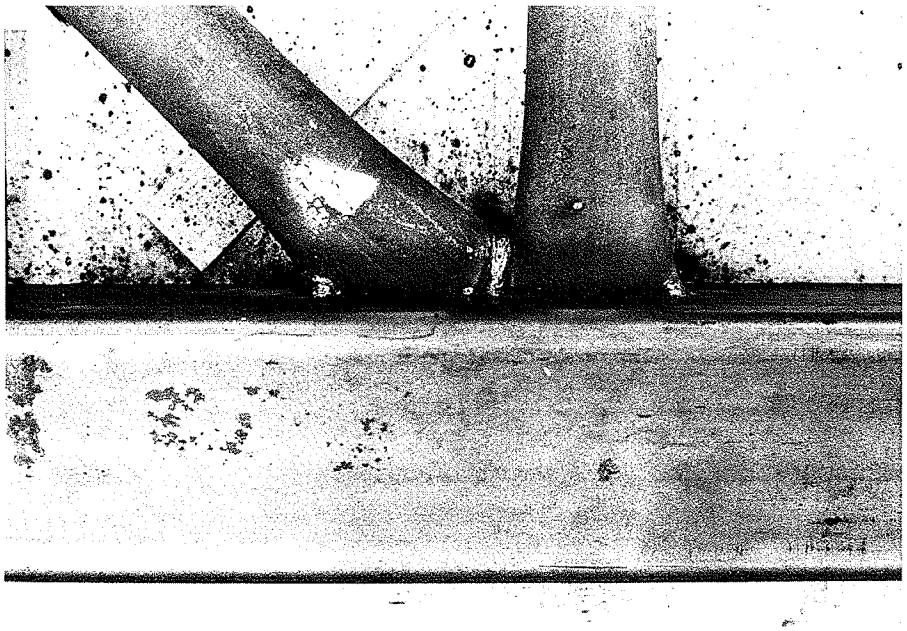


Fig. 3.5(b) Example of Tack Weld of Specimen (No. 1)

In order to measure the displacements of the connected face of the chord wall, relative to the opposite wall, three displacement transducers, as shown in Fig. 3.6, were used. The transducers were Hewlett Packard Series 7DCDT with stroke of ± 0.250 inch. Each transducer essentially consisted of a ferrite core (style number B12) and a cylindrical body with an electrical circuit inside. The transducer body was rigidly held by a mounting block. Both ends of the core were connected to extension rods. The top rod was attached to a tension spring, and the bottom rod to a stainless steel wire. The other end of the wire had been cemented to a 3/16"-diameter nut, which provided a surface for cementing the wire to the chord wall at the location where displacement was to be measured.

The cement used for connecting the nut to the twisted stainless steel wire of 0.015-inch diameter and with a thin plastic coating, was strain gage cement KYOWA EP-18. The nut was cemented to the chord wall, after cleaning the wall with acetone solution, with a drop of strain gage cement CC-15A KYOWA, which dried rapidly and adhered very well.

The procedure of cementing the nut to the chord wall and mounting the transducer on the chord face, as well as the nuts and wire after installation in the chord section, is shown in Fig. 3.6.

The transducers with proper lengths of wires and nuts were

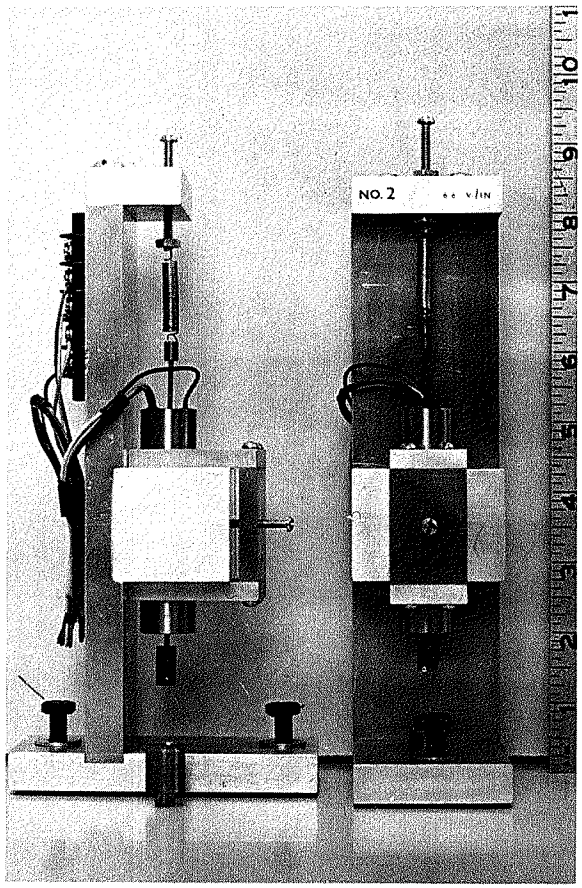


Fig. 3.6 (a) Displacement Transducers and Mounting Blocks

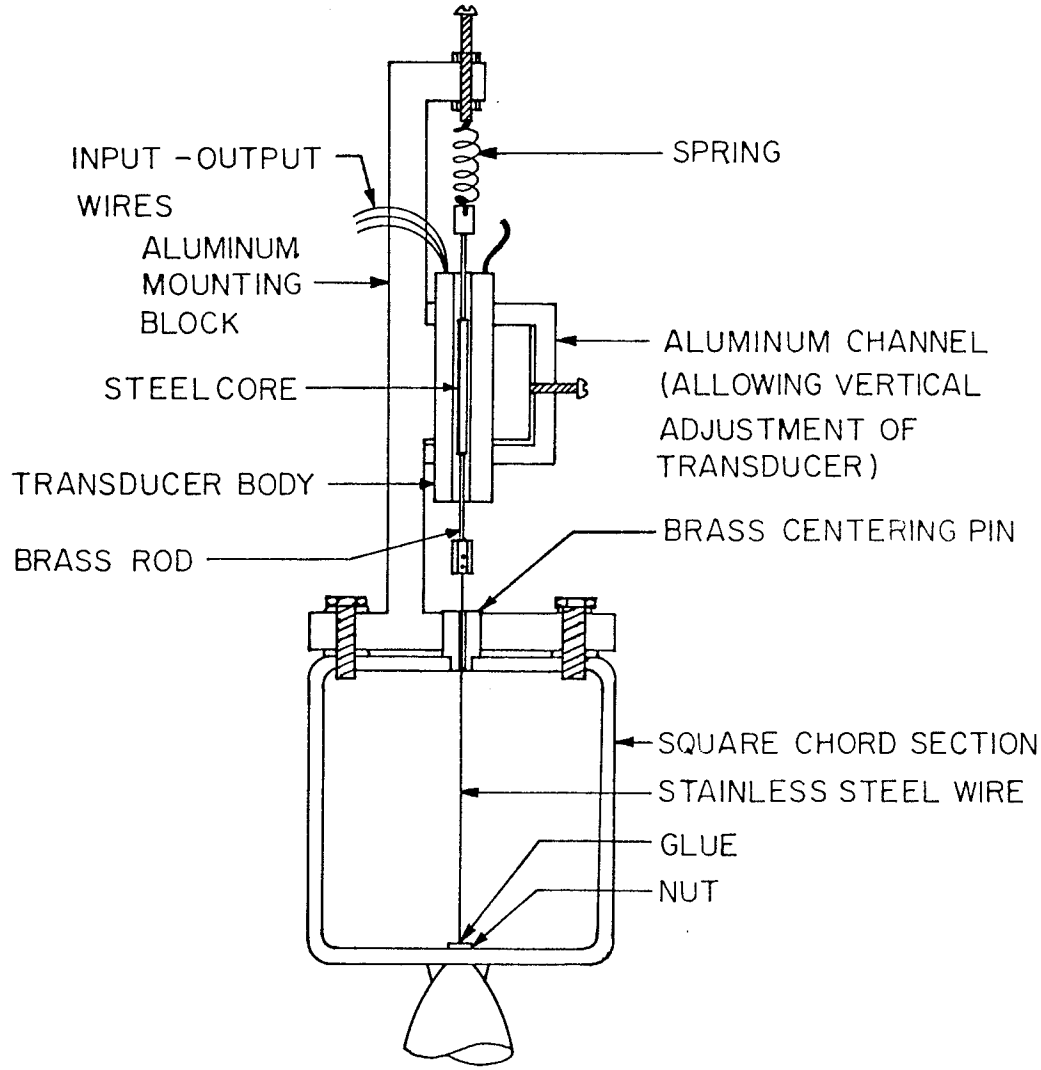


Fig. 3.6 (b) Transducer Assembly

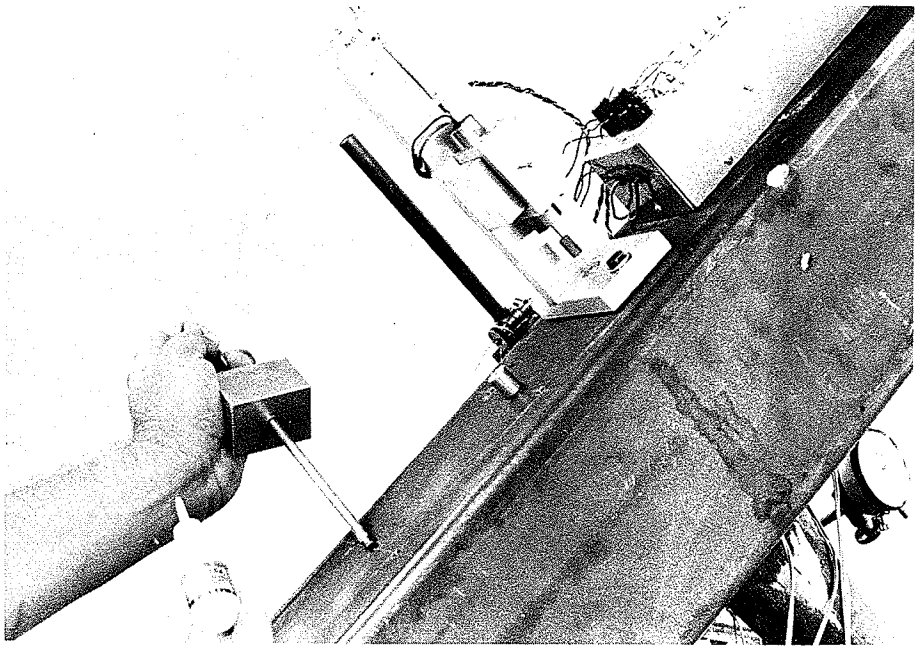


Fig. 3.6(c) Transducer and Mounting Process

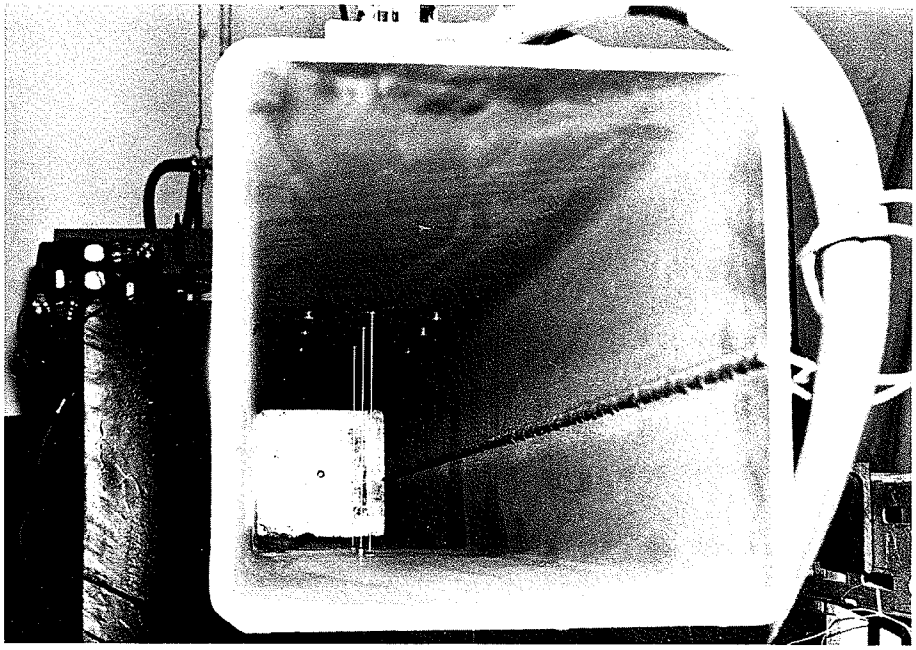


Fig. 3.6 (d) Nuts and Wires in Position

calibrated against a depth micrometer with an accuracy 0.001 inch. The displacement-voltage curves obtained were very linear, as expected. For a stainless wire of $5\frac{1}{4}$" length, as used for a 4x4 chord section, the sensitivity of the transducers ranged from 6.58 to 6.62 volts/inch; the average value of 6.60 volts/inch was used for conversion. When a stainless steel wire of 7 inch length was used for a 6x6 chord section, the factors varied from 6.36 to 6.38 volts/inch; therefore a value of 6.37 volts/inch was used. With the transducer in position over a 48 hour period, the factor exhibited only a minor increase from 6.36 to 6.38 volts/inch.

The locations where the relative chord wall displacements were measured are shown in Fig. 3.7. The transducers were denoted T1, T2 and T3. The three locations were selected because it was seen from the results of the Phase I study[7] that the deflections usually were largest at the locations selected for T1 and T3, whereas T2 indicated, together with T3, the difference in displacement (thus rotation) of the chord wall under the compression web.

Fig. 3.7 also shows the locations of four dial gages, (D1, D2, D3, D4), used for measuring the overall displacements of the members. Gages D1 and D2 measured the in-plane and out-of-plane displacements of the joint. Gages D3 and D4 measured the in-plane and out-of-plane displacements of the mid-point of the compression web of the specimen. Each dial

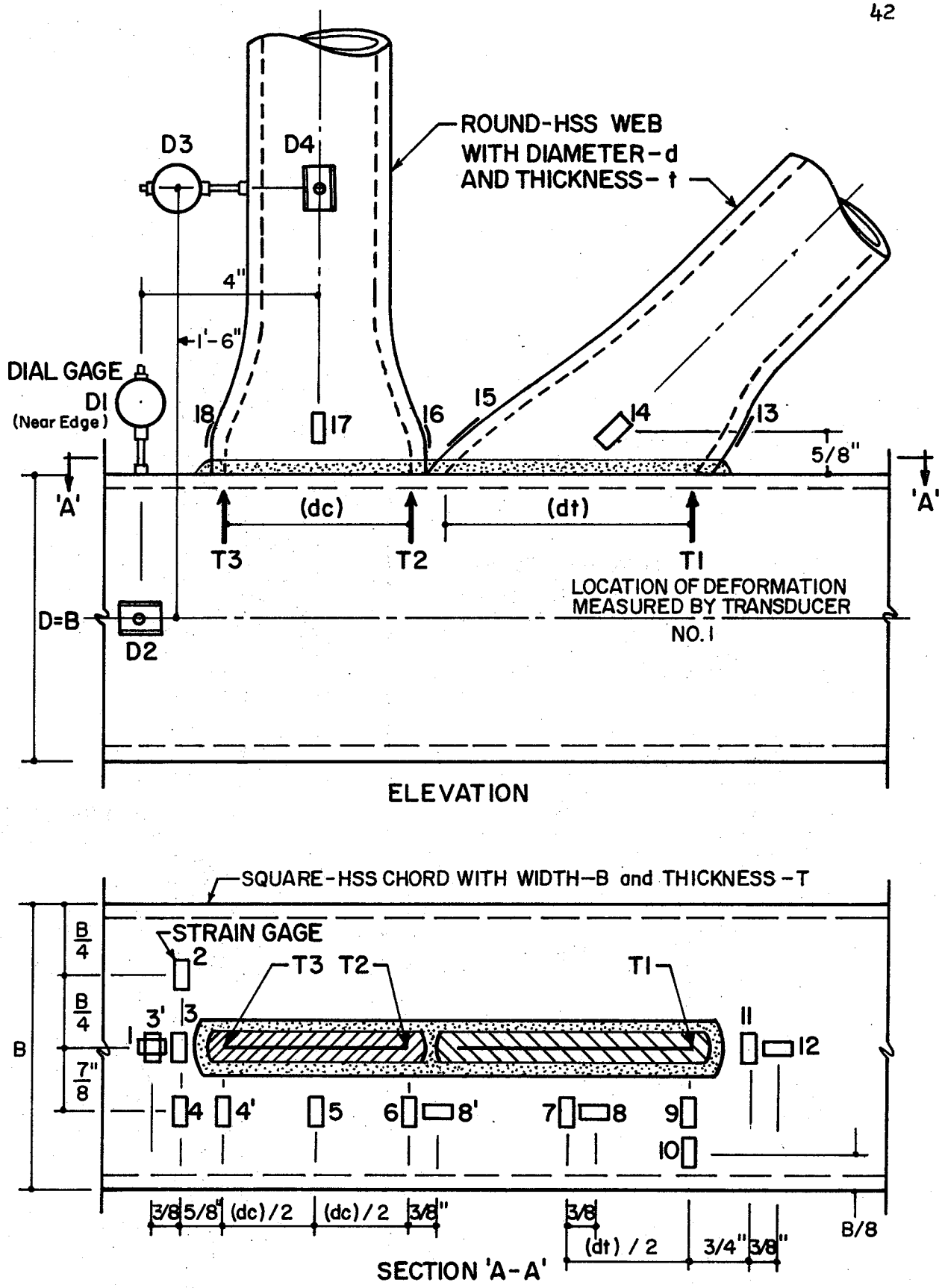


FIG. 3.7 LOCATIONS OF TRANSDUCERS, DIAL AND STRAIN GAGES.

gage measured to an accuracy of 0.001 inch.

3.10 Strain Measurement

Electrical resistance strain gages were used for measuring the strain distributions in the members in the vicinity of the joint. The strain gages were Micro-Measurements Precision Strain Gages MM Type EA-06-187BB-120 with gage length of 0.187 inch, gage resistance of (120.0±0.15%) ohms, and gage factor of 2.09±0.5%. The input voltage was 4 volts. The number of strain gages used for each specimen varied from 18 to six. A total of 60 gages was used on the five specimens.

The gage locations are shown in Fig. 3.7. It was assumed that the joints were symmetrical with respect to the center lines of the connected faces of chords. Therefore strain gages were normally installed on only one side of the chord axis. Since it has previously been found that the strains at some distance from the joint were quite uniform and nominal[7], and that strains close to the joint were relatively large,[5,11-15] the strain gages in this study were located close to the welds.

3.11 Data Acquisition

A Hewlett-Packard Data Acquisition System and a Hewlett-Packard calculator model 9830A, as illustrated in Fig. 3.8, were used to acquire and record on paper and magnetic tape

the output voltages from the transducers and strain gages. A program in the 9830A calculator was used to convert the voltage readings from the tape to displacements and strains and to print them on a Hewlett-Packard Printer.

3.12 Loading Frame

The loading frame was the same one that had been employed in a previous study.[7,8] It is shown in Figs. 3.2 and 3.8. It consisted of two W21 x 68 beams supporting two reaction assemblies inclined at 45 degrees to the horizontal. Each reaction assembly had a two-inch diameter semi-cylindrical bearing surface with a 1/4-inch diameter centering pin to receive the chord and compression web of the specimen. One of the reaction assemblies had provision for horizontal adjustment and adjustment of the bearing surface parallel to the sloping face.

A 12-inch deep built-up jacking beam was bolted to the main beam and permitted horizontal adjustment by increments of three inches. The bottom of the beam provided a bearing surface for a 200-kip hydraulic jack which was used for loading the tension web. The jack and the web were connected by means of a tension yoke, which passed through an opening in the beam. The bottom of the yoke was threaded into the jack; whereas the top was connected to the tension web by a bolt, as shown in Fig. 3.2.

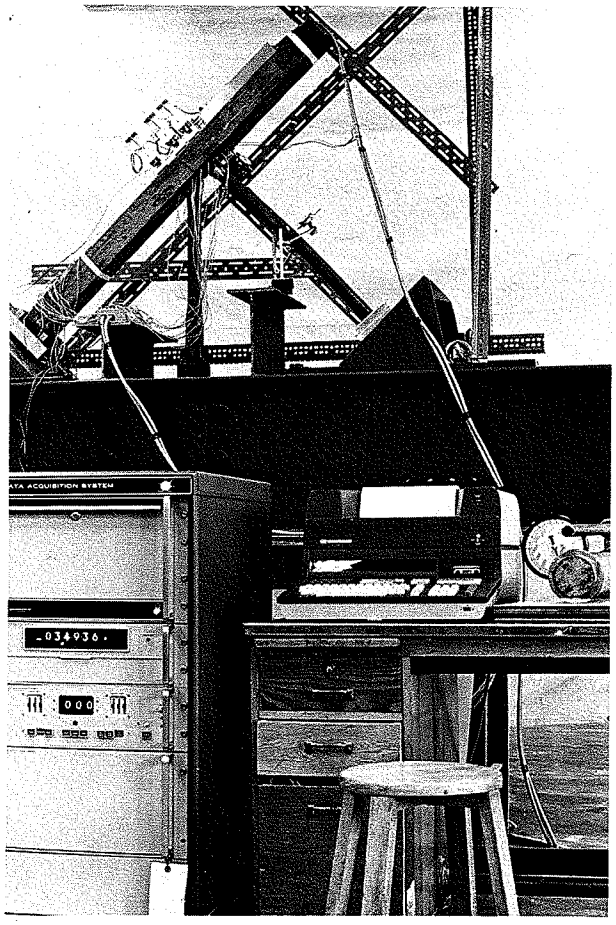


Fig. 3.8 Hewlett-Packard Data Acquisition System and Calculator

3.13 Jacking System

The hydraulic jack used for loading the tension web was connected to a 10,000 psi Enerpac pressure gage (Model 30-102). The jack and gage were calibrated against a 200-kip Riehle testing machine. The conversion factor was found to be 2.09 kips per 100 psi.

The pressure gage was subdivided into 100-psi divisions and had an accuracy of one half per cent of full scale accuracy. Therefore the accuracy of the applied load was approximately ± 1.0 kip.

CHAPTER 4

TEST RESULTS AND DISCUSSION

The test results are presented and discussed by considering the strength, stiffness, stability, failure mode, and strain distribution of the cropped joints in turn.

4.1 Joint Strength

The test results are summarized in Table 4.1. The joint strength is given in terms of the ultimate load and load factor of the web that fails first. Since all specimens finally failed by buckling of the compression webs, the ultimate loads tabulated are the maximum loads in these webs, and the load factors are the ratios of the ultimate loads to the working loads of these webs. The working load of a compression web is calculated by assuming an effective length factor of 0.7, which is recommended for uncropped webs,[1] and following the CSA standard S16-1969.[20]

The load factors of the cropped joints range from 1.65 to 1.91; they are generally greater than 1.70, which is the value normally used in limit design.

The effect of an overlap between the webs on the joint strength can be seen by comparing the first two specimens which have idential sections but different lap. With an increase in lap from zero to 53% of the web diameter, the load factor

TABLE 4.1 TEST RESULTS

Specimen	Chord B x D x T (in)	Webs d OD x t (in)	Lap (in)	Joint Eccentricity (in)	Working Load of Comp. Web W_c (kips)	Ultimate Load of Comp. Web U_c (kips)	Load Factor U_c/W_c	Elastic Stiffness of Connection (kips/in)	Approx. Yield Load (⊥chord wall) (kips)	Failure Mode
1	4 x 4 x .1875	1.900 OD x .110	1.0	0	12.75	24.4	1.91	700	13	X Y Z
2	"	"	0	1.33	"	21.1	1.65	467	11	X Y
3	"	2.375 OD x .125	0	2.16	20.5	38.4	1.87	294	16	X Y
4	6 x 6 x .1880	1.900 OD x .110	0	0.34	12.75	23.6	1.85	276	9	X Y
5	"	2.375 OD x .125	0	1.16	20.5	35.1	1.71	256	12	X Y

Notations: X = Excessive deformation of connected face of chord wall

Y = Buckling of compression web

Z = Fracture of weld at connection between webs

Note: 1 kip = 1,000 lbs.

increases by 16 per cent. This is because the lap permits a direct force transfer between the webs; thus the chord wall deforms less and applies a smaller moment to the ends of the webs. Consequently the compression web buckles at a higher load.

The effects of prestress in the chord, and different methods of end flattening, on the joint strength can be seen by comparing specimen 1 to specimens 5A1 and 5A2, which were tested in Phase One[7] and were practically identical to specimen 1 except that they had a prestress of 14 kips (19% of the working load of chord) applied to the chords and that their webs were flattened and sawn instead of cropped. The ultimate load in the compression web of specimen 1 is within two per cent of the corresponding loads of specimens 5A1 and 5A2. Therefore the effects of the prestress in the chord and different methods of end flattening appear to be insignificant.

A comparison of any pair of specimens other than the first pair involves more than one joint parameter. Therefore such a comparison gives the combined effect of several joint parameters on the joint strength. However, the separate effect of a joint parameter can be obtained by comparing two pairs of specimens that have two joint parameters varying such that one of them is the same in both pairs.

A comparison of specimens 2 and 3, which have different web sizes, shows a 13% increase in the load factor with a 25% increase in the ratio of web diameter to chord width (d/B), and

a 62% increase in the ratio of positive joint eccentricity to chord depth (e/D). A similar comparison of specimens 4 and 5 shows an 8% decrease in the load factor with a 25% increase in the d/B ratio, and a 240% increase in the e/D ratio. A comparison of the two pairs of specimens thus indicates that the load factor decreases by about 21% with a 180% increase in the e/D ratio.

Specimens 2 and 4, which have different chord sizes, show a 12% increase in the load factor with a 50% increase in the ratio of chord width to chord thickness (B/T), and an 83% decrease in the e/D ratio. However , a similar comparison of specimens 3 and 5, which have a 50% increase in the B/T ratio and a 64% decrease in the e/D ratio, shows a 9% decrease, instead of an increase, in the load factor. These illustrate the complexity of the relationship of the load factor to various joint parameters.

The load factors for the cropped joints with one-inch lap and those without lap are comparable to the load factors of corresponding,conventional joints that were reported in References 12, 14, and 15. The load factors ranged from 1.62 to 2.40 for joints with laps of 0.55 inch to 6.23 inches, and from 0.65 to 2.12 for joints with gaps of 0.14 inch to 1.89 inches.

References 14 and 15 also give a plot of the ratio of ultimate

load to the product of chord thickness and yield strength against the ratios of mean web diameter to chord width, for joints with weld gap, as illustrated in Fig. 4.1. Also shown are the lower bound of ultimate load and the suggested working load curve, which is obtained by dividing the lower bound by a safety factor of 1.6.[1]

In order to compare the ultimate loads of cropped joints with those of corresponding, conventional joints, similar variables were graphically superimposed as shown in Fig. 4.1. It can be seen that the ultimate strengths of cropped joints both with and without a lap compare favorably with the ultimate strengths of conventional joints with gaps.

Since the range of the ratio of the mean web diameter to chord width, in the present tests, varies from 0.317 to 0.594, and only 7 specimens were tested, it is clear that further tests are needed before the formulas given in the figure can be confidently applied, especially for the ratios outside the range.

4.2 Joint Stiffness

The stiffness of a tubular truss joint with cropped webs welded along the center line of a chord face can be expected to be adversely affected by the rotational deformation of the chord wall.

The stiffness of the chord wall of such a joint can be shown graphically by a plot of loads applied to the tension web vs local deformation of chord wall, as presented in Fig. 4.2.

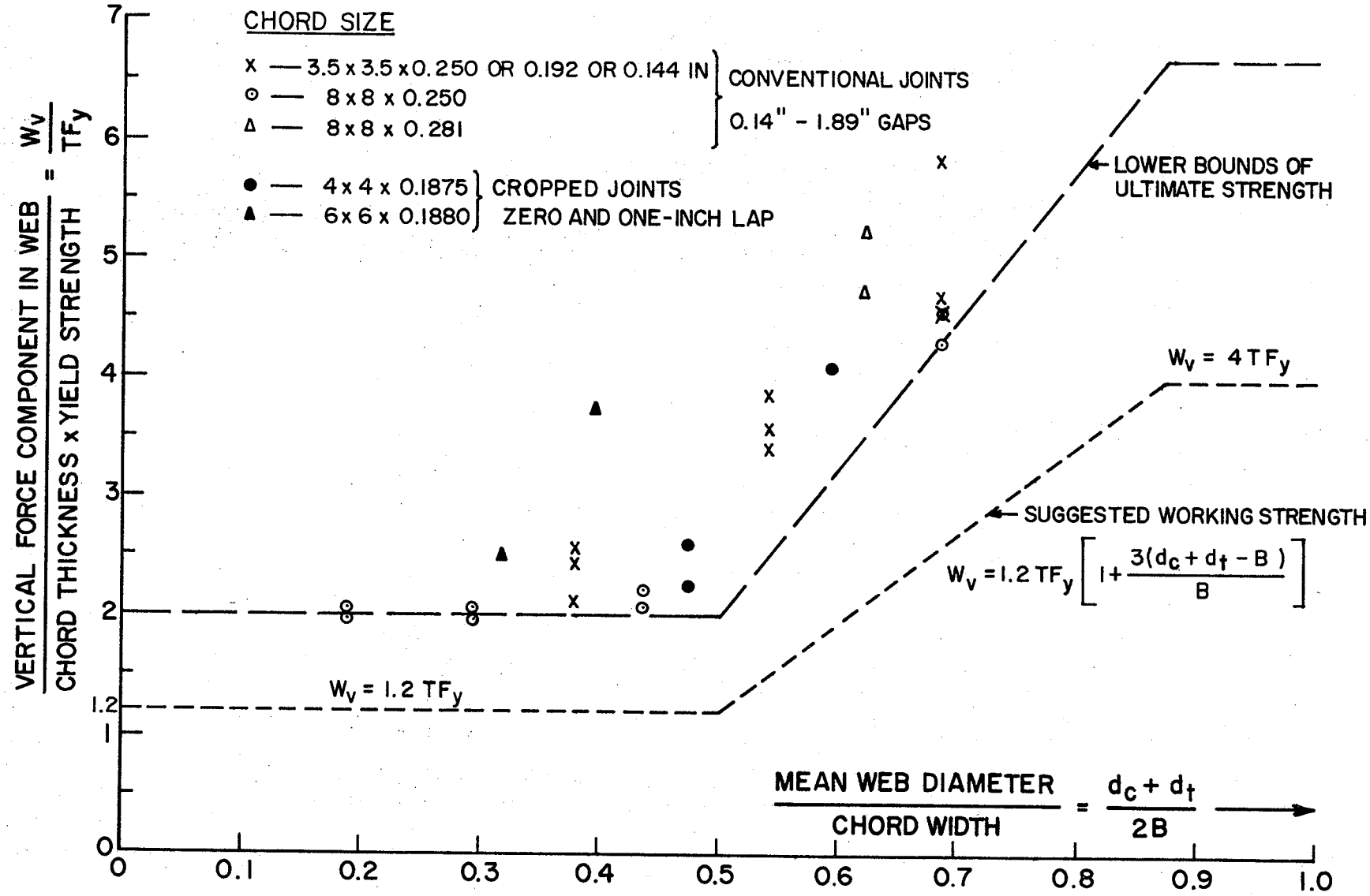


FIG. 4.1 TEST RESULTS FOR CONVENTIONAL & CROPPED JOINTS

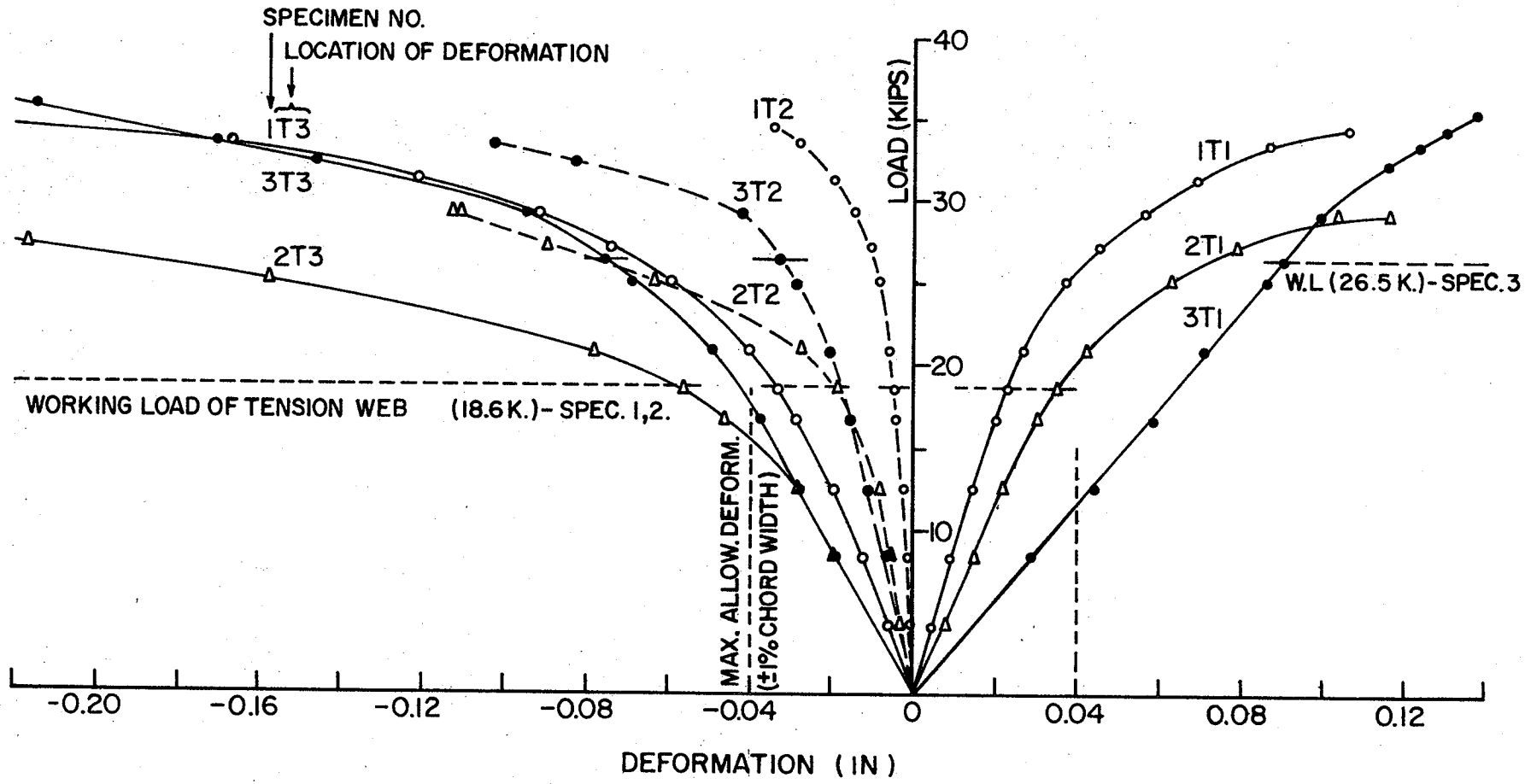


FIG. 4.2(a) LOAD IN TENSION WEB vs LOCAL DEFORMATION OF CHORD WALL - SPECIMENS 1,2,3.

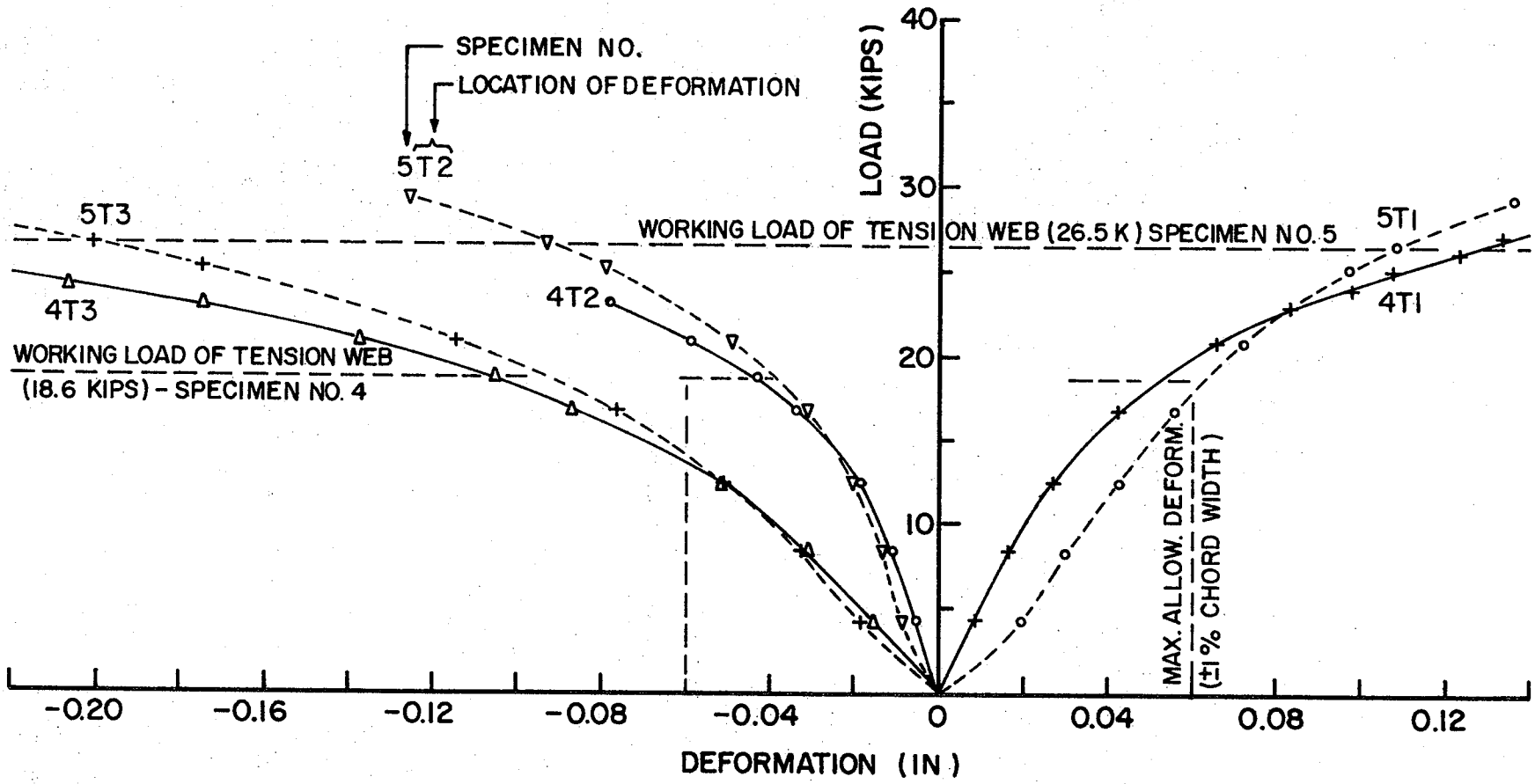


FIG. 4.2(b) LOAD IN TENSION WEB vs LOCAL DEFORMATION OF CHORD WALL - SPECIMENS 4&5.

The three locations of deformation measurement, denoted by T1, T2 and T3, are shown in Fig. 3.7.

The measured load-deformation behavior of all specimens was very similar in that the chord wall deformation at location T3 was always greater than that at T2, thus indicating an in-plane rotation at the end of the compression web. The chord wall at T3 deflected more than that at T2 because the wall at T2 was pulled up somewhat by the tension web.

The maximum deformation usually occurred at location T3, as illustrated in Fig. 4.2 and 4.4 . This is partly because of the rotation deformation under the compression web, and also because the force per unit length along the chord was greater under the compression web than under the tension web - since the normal force components of the two webs were equal and opposite but the length along the chord under the compression web was smaller.

The only exception was specimen 3, for which the deformation at T1 was slightly larger than that at T3 up to about the working load of its tension web. This is probably because the webs of this specimen were relatively stiff when compared to the chord wall and thus the deformations at T1 and T3 were comparable until the compression web started to buckle.

The elastic stiffness of the connection can be measured by the minimum ratio of the normal load component to the corresponding deflection of the chord wall, in the elastic range. The elastic stiffnesses of the connections are presented

in Table 4.1. It can be seen from this table and Fig. 4.2 that the elastic stiffnesses of the connections decrease consecutively from specimens 1 to 5.

Although specimens 1 and 2 had identical member sections, specimen 1 was one and a half times stiffer than specimen 2. In other words, at any given load in the elastic range, the maximum chord-wall deformation of specimen 2 was about 50 per cent larger than that of specimen 1. This is because specimen 1 had a lap of one inch between the webs, whereas specimen 2 had no lap. The lap permitted some direct transfer of force between the webs. Consequently, the normal force components, which caused the chord-wall deformation, were reduced.

A quantitative direct comparison of the elastic stiffnesses of other pairs of connections is not possible because several significant parameters were involved. However, qualitative comparisons will be made.

The elastic stiffness of the connection (as previous defined) of specimen 3 was smaller than those of specimens 1 and 2. This is probably because the webs of specimen 3 were relatively stiff and thus could exert normal forces on the chord wall without bending easily. As a result, the rotational deformation of the chord wall of specimen 3, due to the compression web buckling, was smaller. A similar behavior was also found in specimens 4 and 5. The elastic stiffnesses of the connections of specimens 4 and 5 are relatively small because their chord walls are very flexible.

It can be seen from Fig. 4.2 that the chord walls of some specimens yielded at loads below the working loads of their tension webs. Yielding below the working loads could cause the truss to become unstable, and impose a permanent set (deformation) on the chord wall when the loads are removed.

Approximate yield loads, defined as the web forces normal to the chord axis when the chord face deformation curves started to deviate from straight lines, are tabulated in Table 4.1. The values given are approximate only, as several of the load-deformation curves become non-linear at very small loads.

It can be seen from Table 4.1 that all of the approximate yield loads are lower than the working loads of compression webs of joints without lap, but the yield load and working load of the lap joint are approximately equal.

Therefore, if a permanent set is to be avoided, the working loads of the webs have to be reduced such that the normal load components do not exceed the approximate yield loads.

If a certain percentage of permanent set (such as $\pm$ 1 per cent chord width) can be tolerated then a higher value of yield load can be obtained by a procedure similar to that used in obtaining a 0.2% - offset yield strength.

In order to estimate the effect of the chord-wall deformation on the overall truss deflection, a typical tubular truss, as shown in Fig.3.1(a), was analysed. Assuming pin connections,

the mid-span deflection due to elastic deformation alone was found to be 1.97 inches. The analysis was then repeated, incorporating the effects of local chord-wall deformations of specimens 2 and 3 (obtained from Fig. 4.2(a)) for all appropriate joints, and the mid-span deflection was found to be 3.21 inches. If the local chord-wall deformations of specimen 1 (lap joint) were used instead of those of specimen 2 (joint without lap), the mid-span deflection would be 3.09 inches (1/230 times the span). Thus, the effect of local chord-wall deformations of the cropped joints was to increase the mid-span deflection by about 60 per cent. Approximately half of this increase was contributed by joint a', a and a_1 (Fig. 3.1(a)). This is because the forces in the webs at these joints are relatively large.

In calculating the mid-span deflection, the local chord-wall deformation curves of specimen 3 (joint a) were assumed to be the same as those of joints a' and a_1. At the working load of the web of joint a', the chord wall at the joint deformed plastically and this would correspond to a local instability failure.

Thus, it is clear that although the ultimate strengths of cropped joints without lap and of the types considered in this study are satisfactory, the flexibility of these joints makes them unsuitable for carrying relatively large forces.

However, the stiffnesses of these cropped joints can be

improved by overlapping the webs or increasing the thickness of chord walls.

The overall deflection of the joint was measured by a dial gage positioned at the location D1, as shown in Fig. 3.7. Plots of loads in tension webs vs overall deflections of joints are presented in Fig. 4.3.

The overall deflection behavior of all joints was very similar. In the elastic range, the joint deflections of specimens of identical chord sections were approximately equal. For example, at a load of 18.6 kips, the joint deflections of the five specimens were 0.090, 0.089, 0.100, 0.115, and 0.108 inch, respectively. These values and Fig. 4.3 indicate that specimens 4 and 5, with more flexible chord walls, deflected more than specimens 1,2 and 3.

4.3 Joint Stability

A truss joint is unacceptable on grounds of safety if it becomes unstable locally, or if any of its members buckles at a load less than the prescribed load factor times the working loads of its members.

It has been shown in the preceding section that most of the chord walls at the joints deformed excessively and yielded at the working loads of the tension webs, thus the joints became unstable locally. This local instability also induced a moment at the ends of the webs; the moment caused the compression web to finally fail by buckling.

The overall deflection and stability of a joint and its

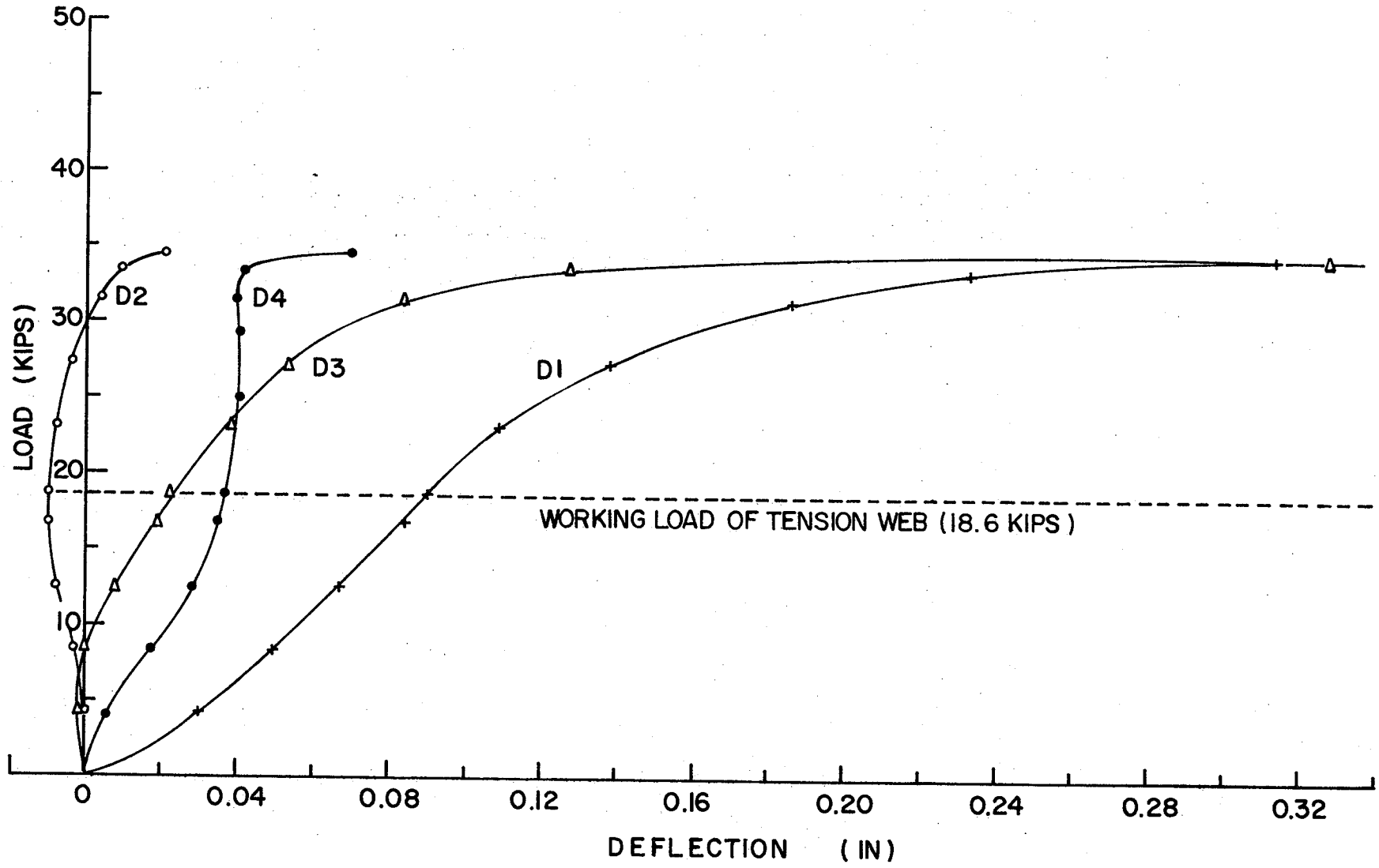


FIG. 4.3 (a) LOAD IN TENSION WEB vs DEFLECTION OF MEMBERS - SPECIMEN NO.1.

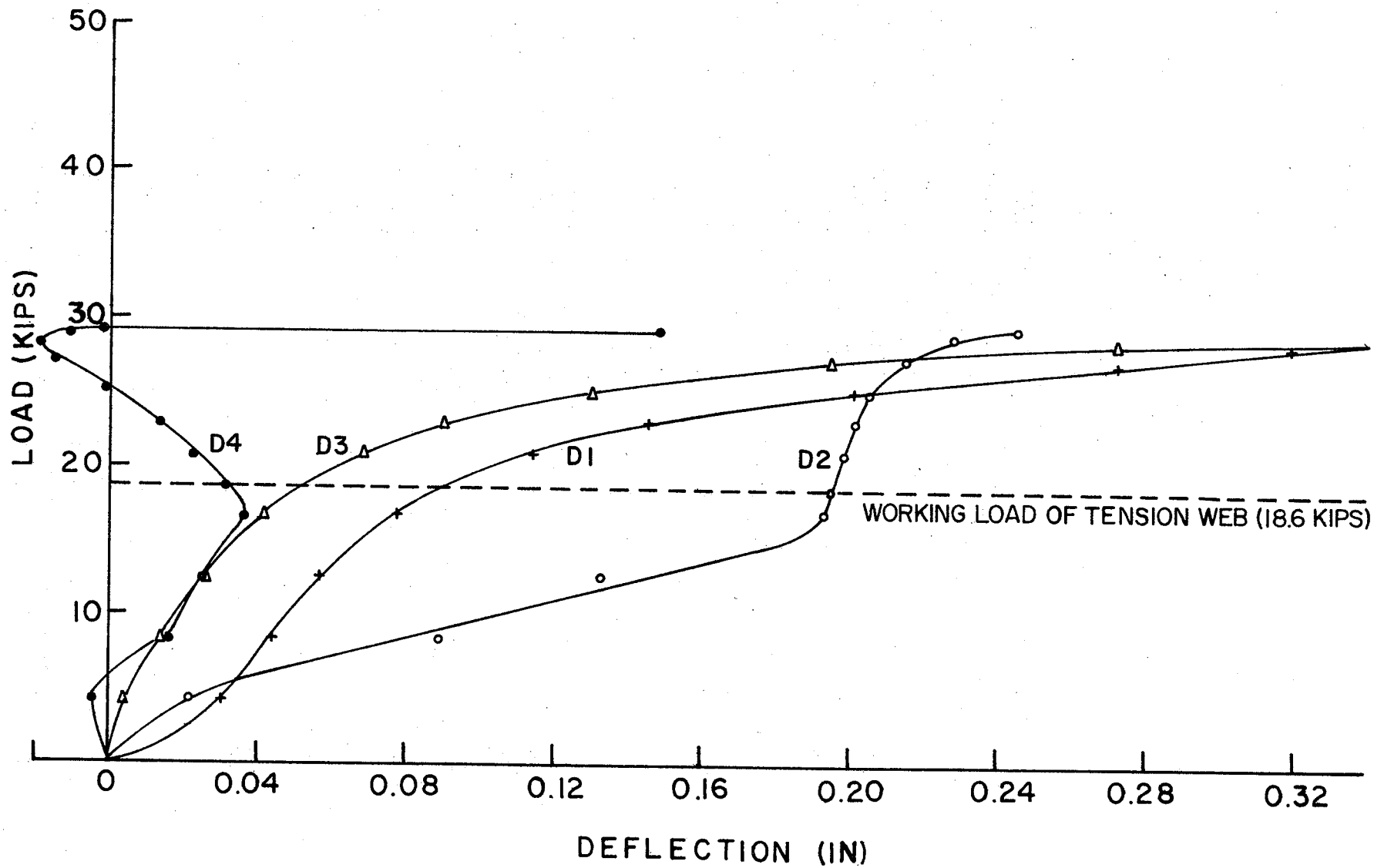


FIG.4.3 (b) LOAD IN TENSION WEB vs DEFLECTION OF MEMBERS – SPECIMEN 2

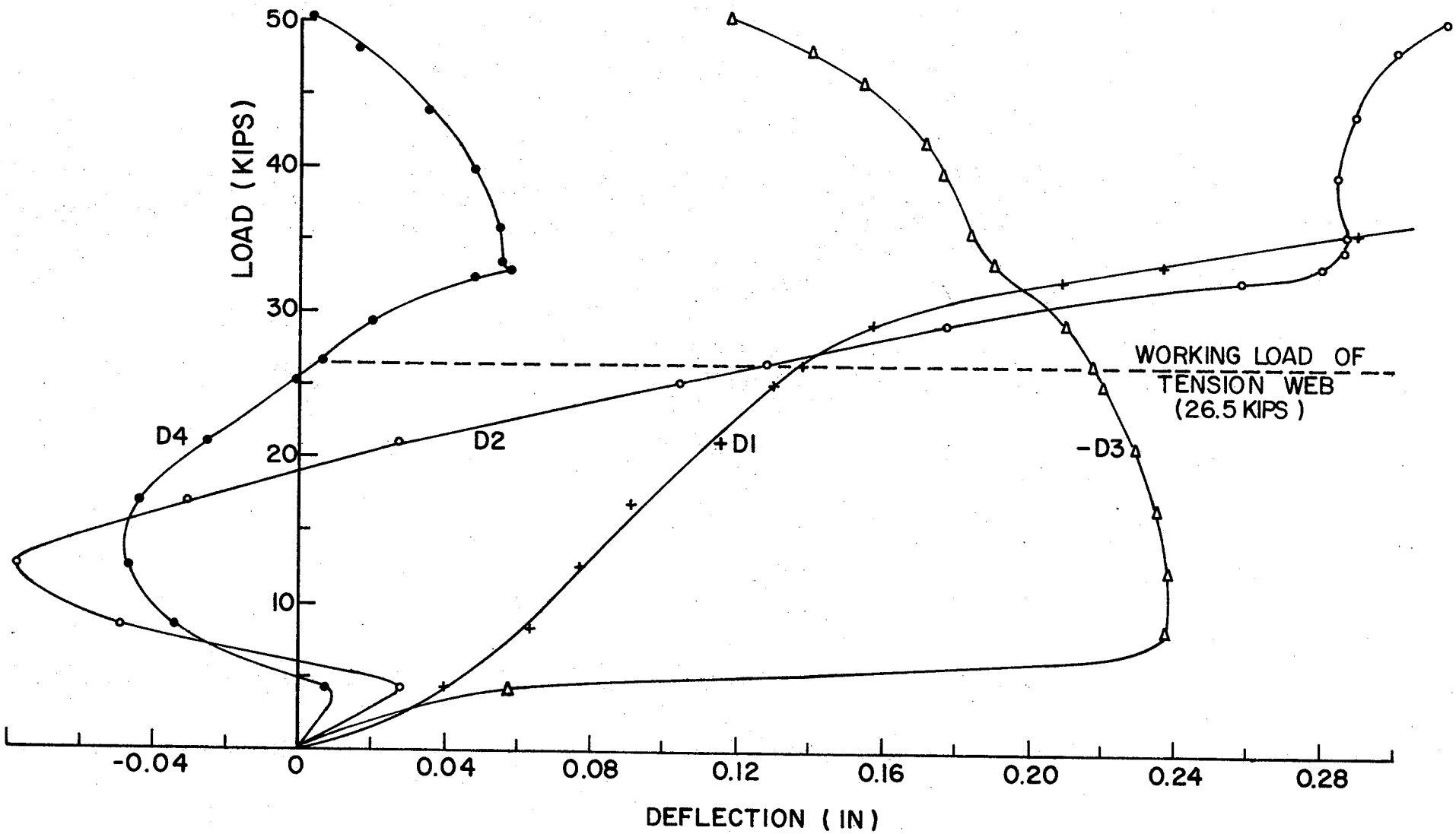


FIG. 4.3 (c) LOAD IN TENSION WEB vs DEFLECTION OF MEMBERS – SPECIMEN NO. 3.

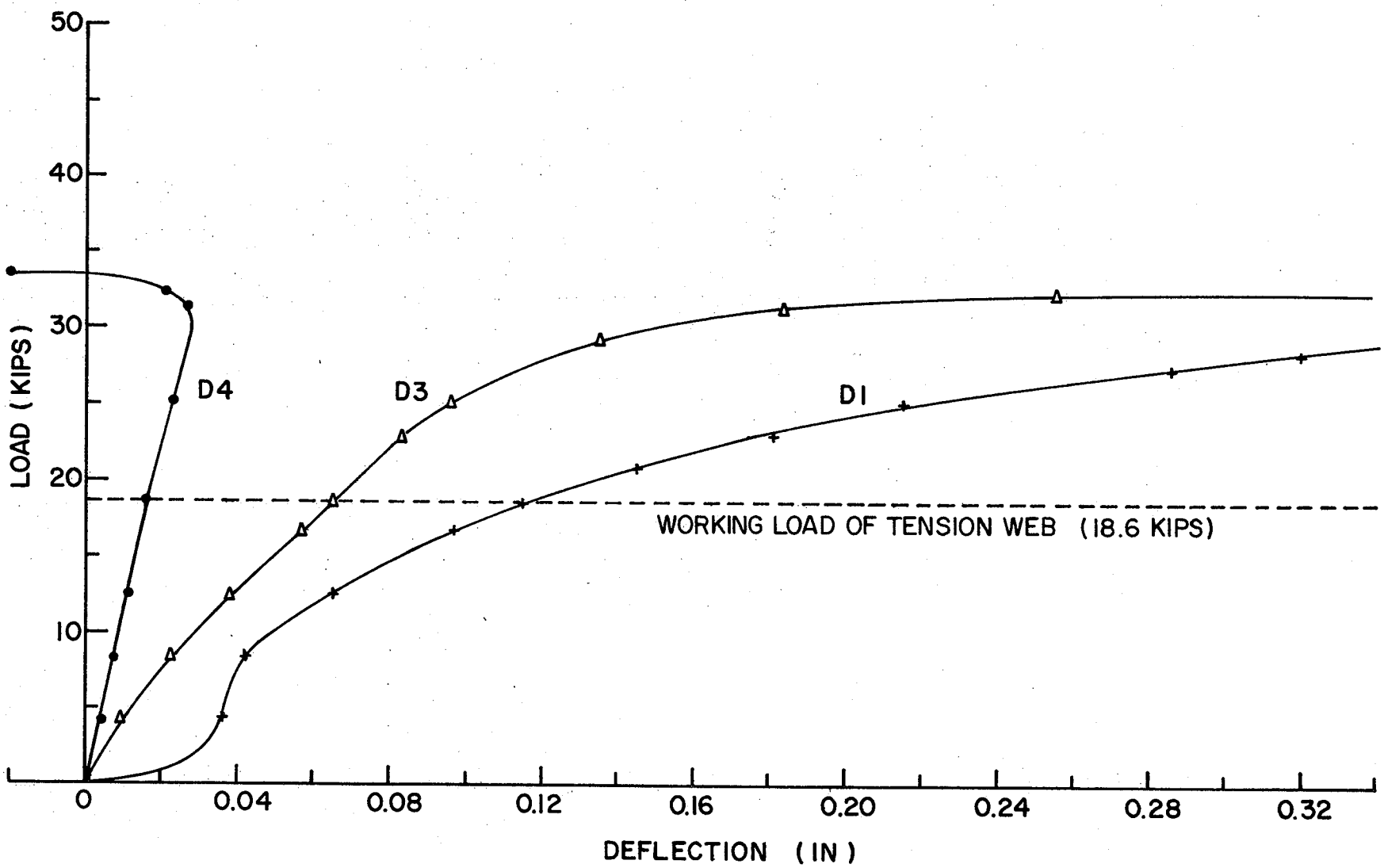


FIG. 4.3 (d) LOAD IN TENSION WEB vs DEFLECTION OF MEMBERS - SPECIMEN NO. 4

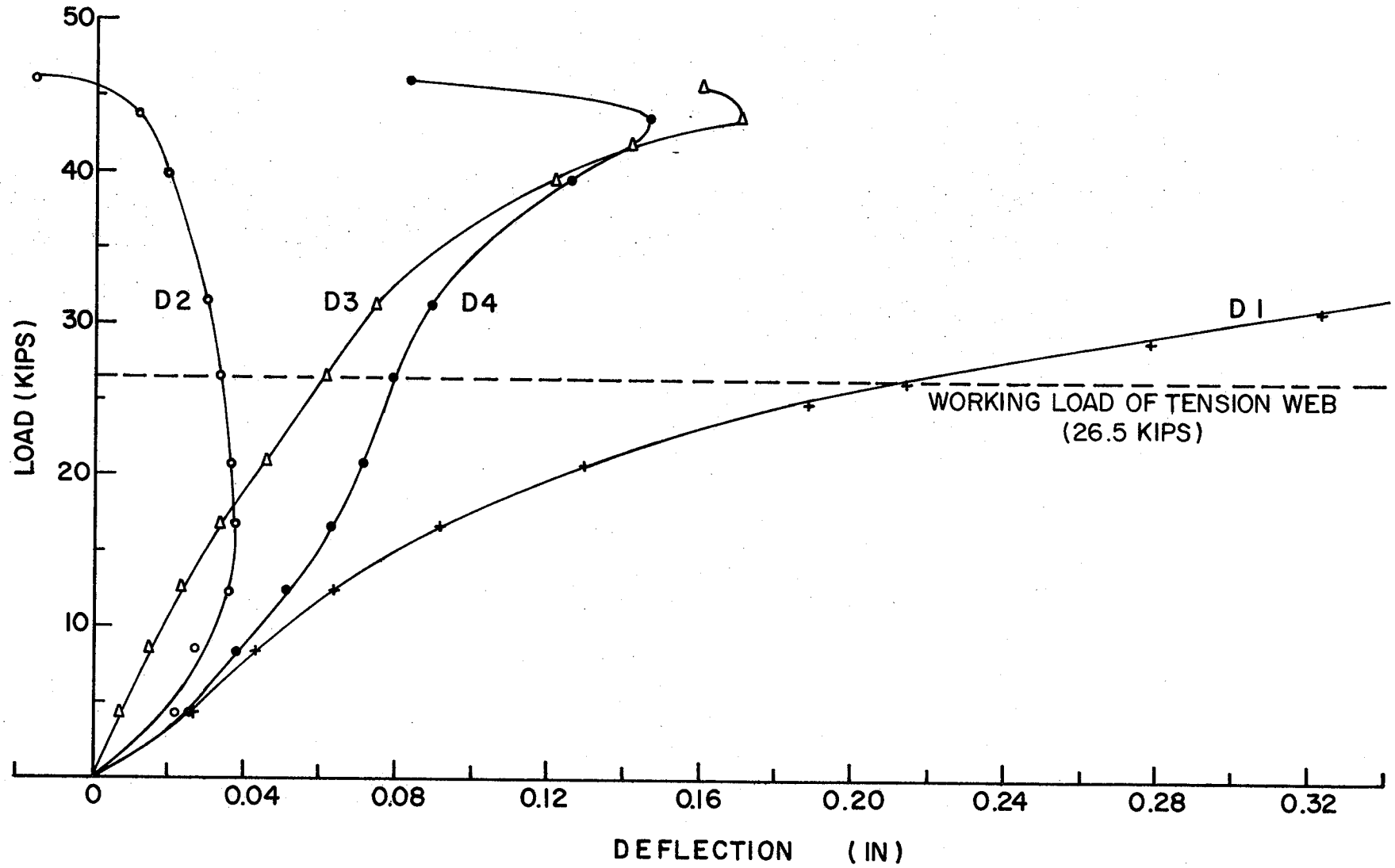


FIG. 4.3 (e) LOAD IN TENSION WEB vs DEFLECTION OF MEMBERS – SPECIMEN NO. 5.

compression web was measured by four dial gages, positioned at the locations shown in Figs. 3.2 and 3.7 . Dial gage 1, which has already been discussed, measured the overall deflection of the joint in the plane of the truss; dial gage 2, measured the out-of-plane deflection of the joint. Dial gages 3 and 4 measured the buckling deflection of compression web in the in-plane and out-of-plane direction, respectively.

It can be seen from Fig. 4.3 that the transverse stability of chords, as represented by the curves labelled D2, was generally good. Specimens 2 and 3 had erratic movements of chords at the joints, probably because shim plates under the base plates of the chords came out. However, the chords finally become quite stable under heavy loads. It will be seen that a transverse movement of the chord at the joint of 0.32 inch (specimen 3) over a length of approximately 36 inches would not be very significant.

The curves D3 and D4 are of special interest since they indicate the relative magnitudes of buckling in two perpendicular directions, at approximately the quarter points of compression webs. It can be seen from Fig. 4.3 that, although sometimes the deflections of the compression webs perpendicular to the truss plane (D4) were initially larger than the in-plane deflections (D3), the latter deflections always became larger when the specimens reached their ultimate loads. This is because initially it was relatively easy for the compression web to buckle in the plane perpendicular to the truss since the cropped

ends of the webs had relatively small bending stiffness in that plane. However, there was virtually no bending moment in that plane - if the joints were symmetrically fabricated and the bearing plates were aligned properly.

As loading increased, a relatively large bending moment, due to rotational deformation of chord wall, was applied to the compression webs in the plane of the truss. This finally caused the compression web to buckle in the plane of the truss.

Deflection measurements (D3) also indicated that the direction of compression web buckling was always away from the tension webs. This is because the deformation of the chord wall under the far end of the compression web was always larger, as discussed earlier.

4.4 Modes of Failure

The failure modes of the clipped joints tested, as indicated in Table 4.1, were very similar. Every joint initially failed, as was expected, by an excessive deformation of the connected face of the chord wall. Every specimen finally failed by buckling of the compression web, due to an end moment primarily caused by the rotational deformation of the chord wall.

Specimen 1, which had a lap of one inch between the webs, failed not only by the two modes of failure, but also by a fracture of weld at the lap. The weld failure was probably caused by an improper tack weld, as shown in Fig. 3.5(b).

Photographs of the specimens after failure are presented in Fig.4.4.

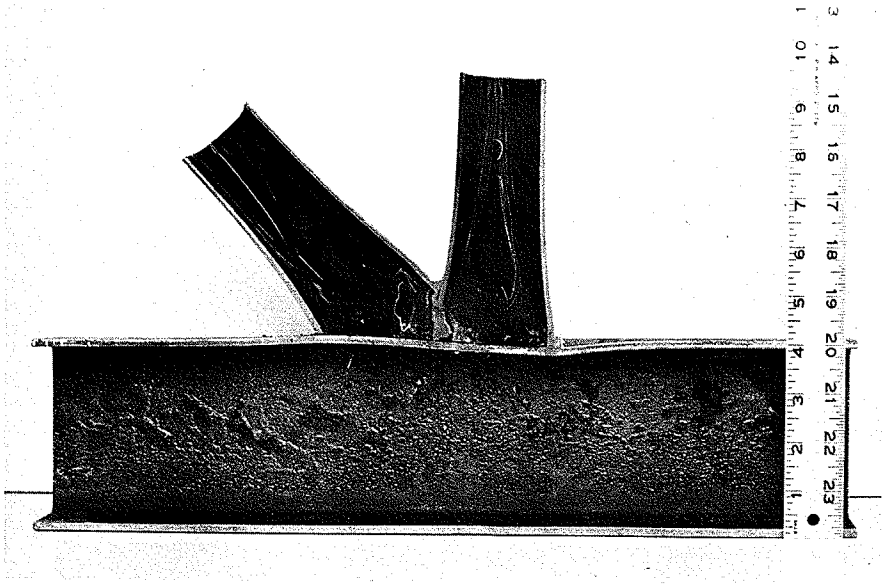


Fig. 4.4(a) Deformation of Specimen 5A2 (Phase I)

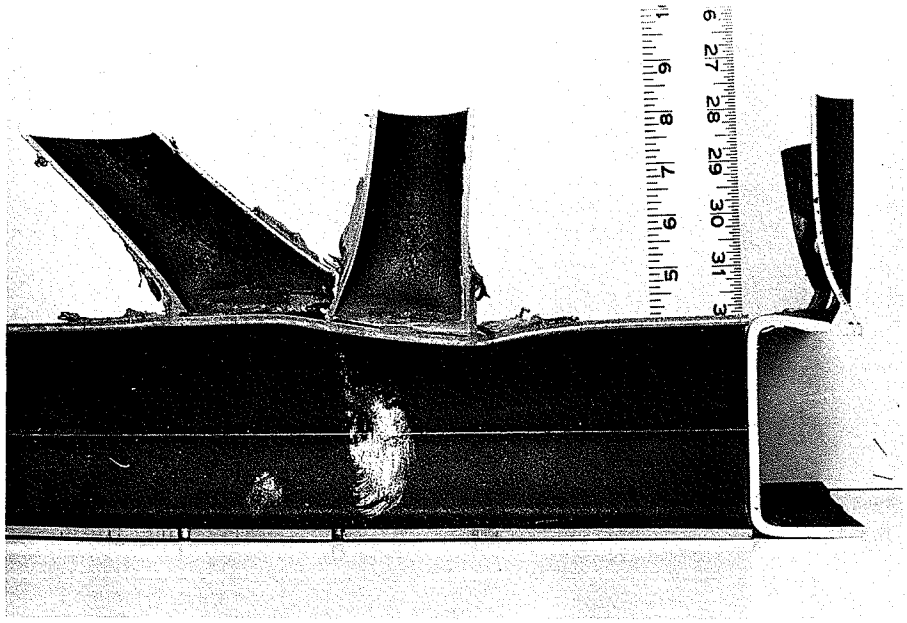


Fig. 4.4(b) Deformation of Specimen 1

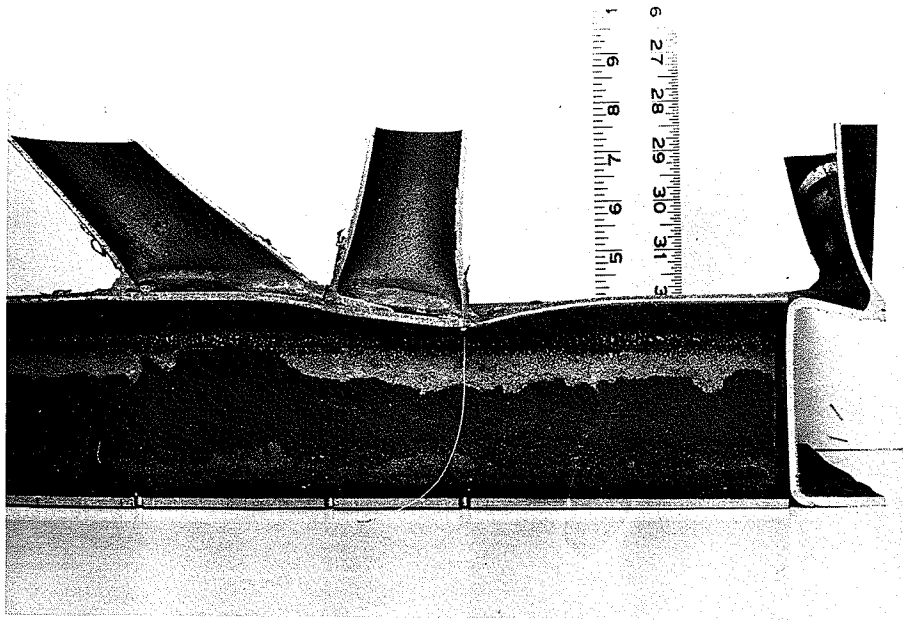


Fig. 4.4(c) Deformation of Specimen 2

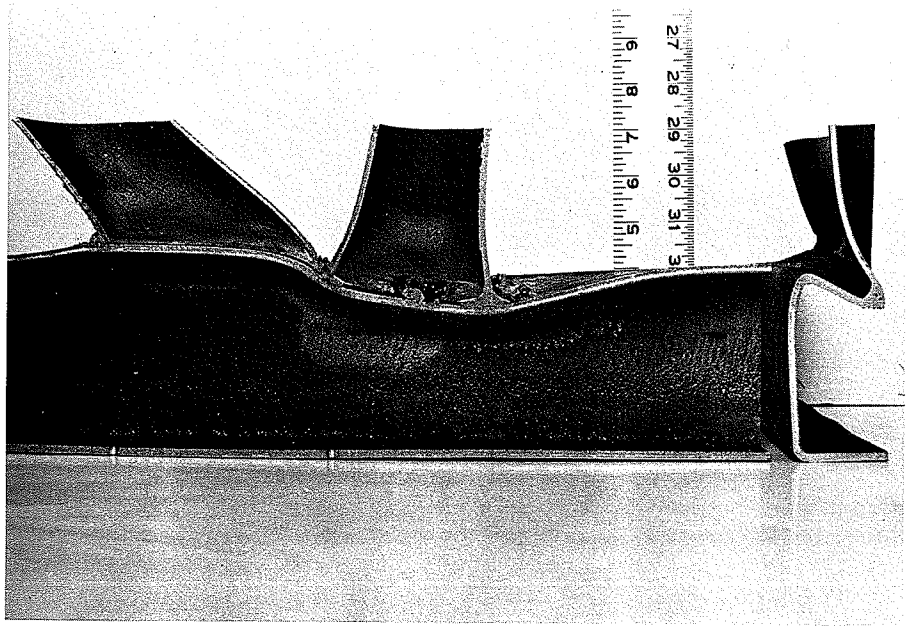


Fig. 4.4(d) Deformation of Specimen 3

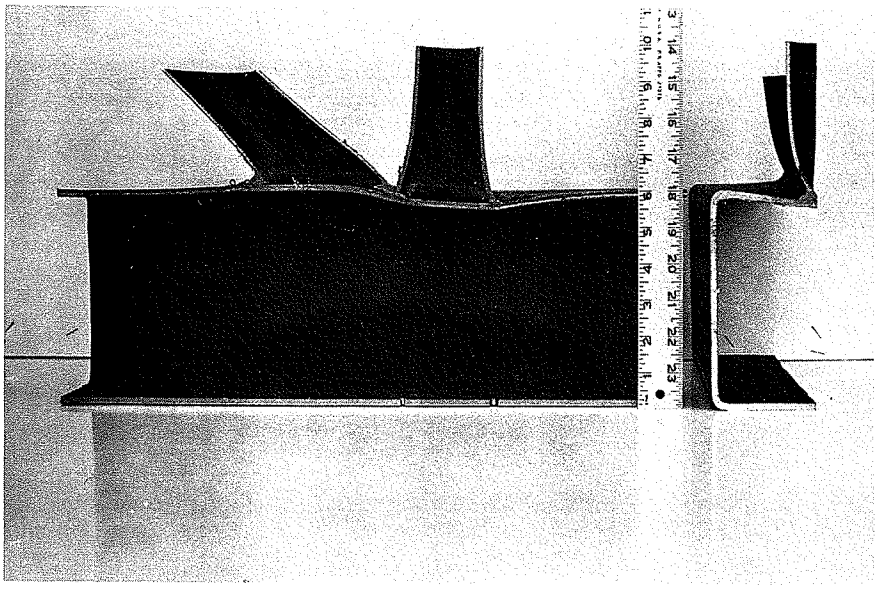


Fig. 4.4(e) Deformation of Specimen 4

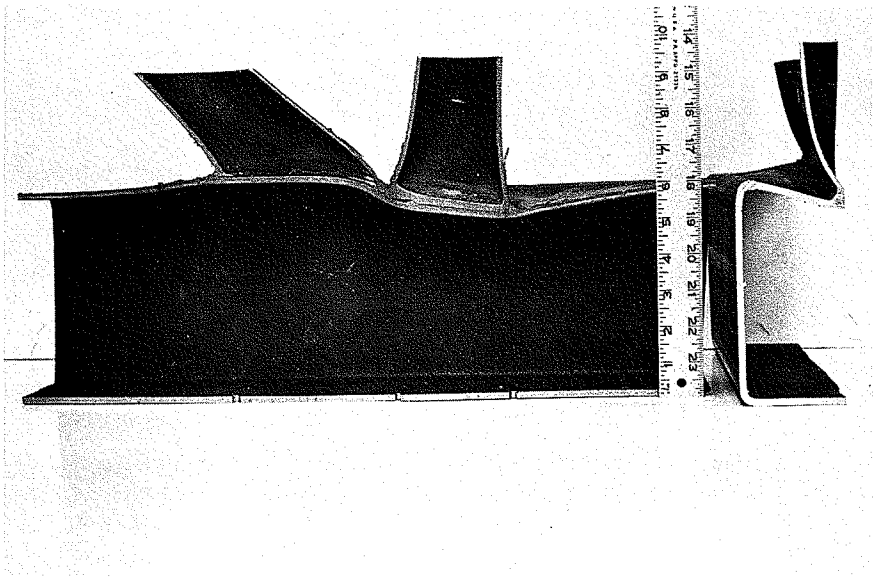


Fig. 4.4(f) Deformation of Specimen 5

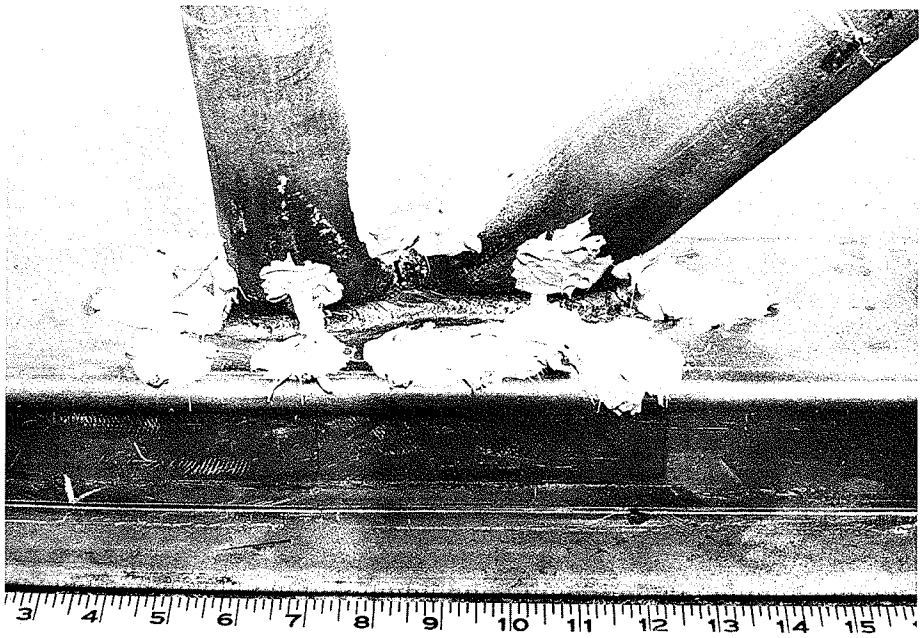


Fig. 4.4 (g) Deformation of Specimen 1

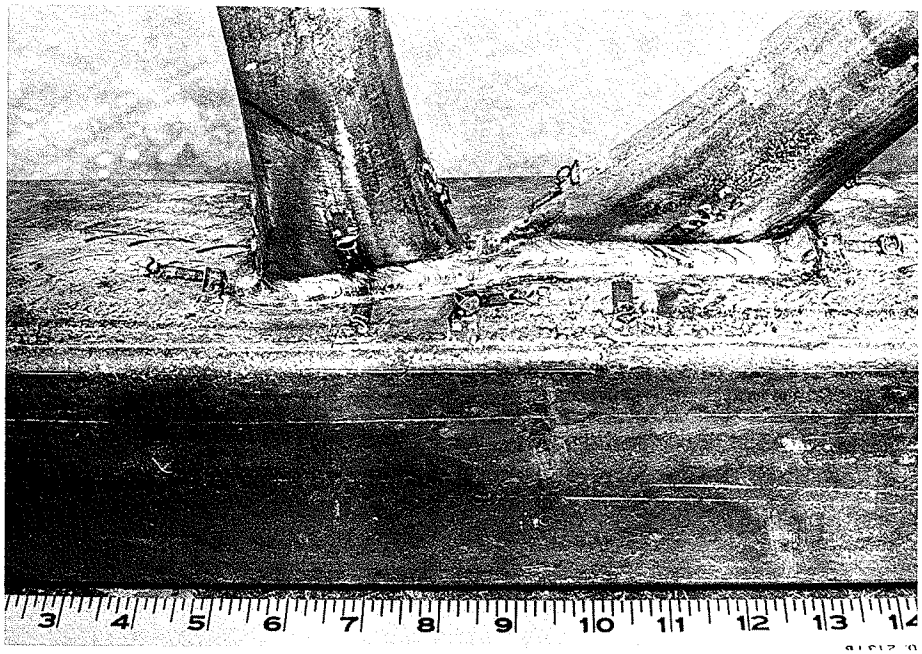


Fig. 4.4 (h) Deformation of Specimen 2

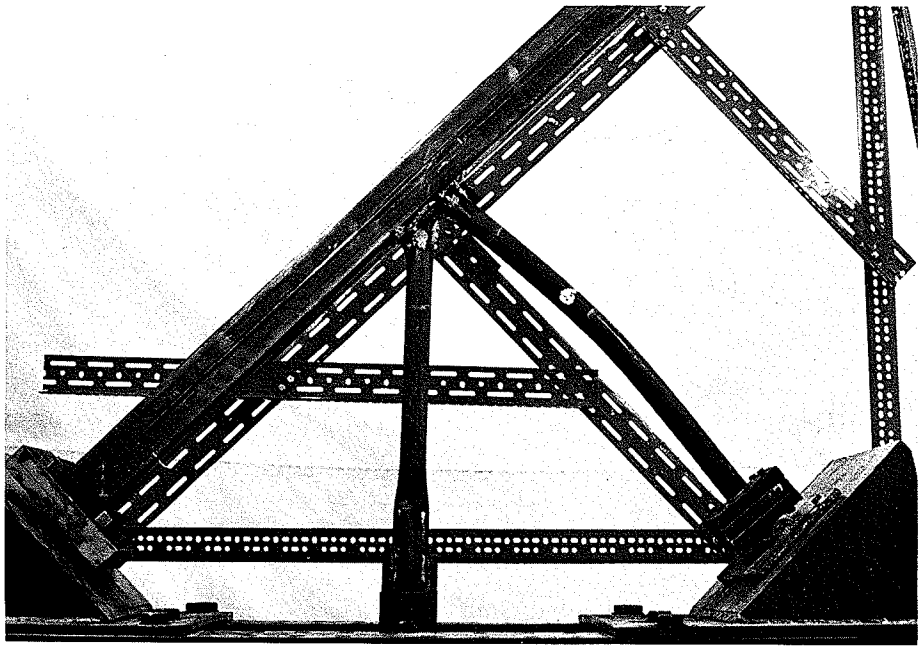


Fig. 4.4(i) Buckling of Specimen 1

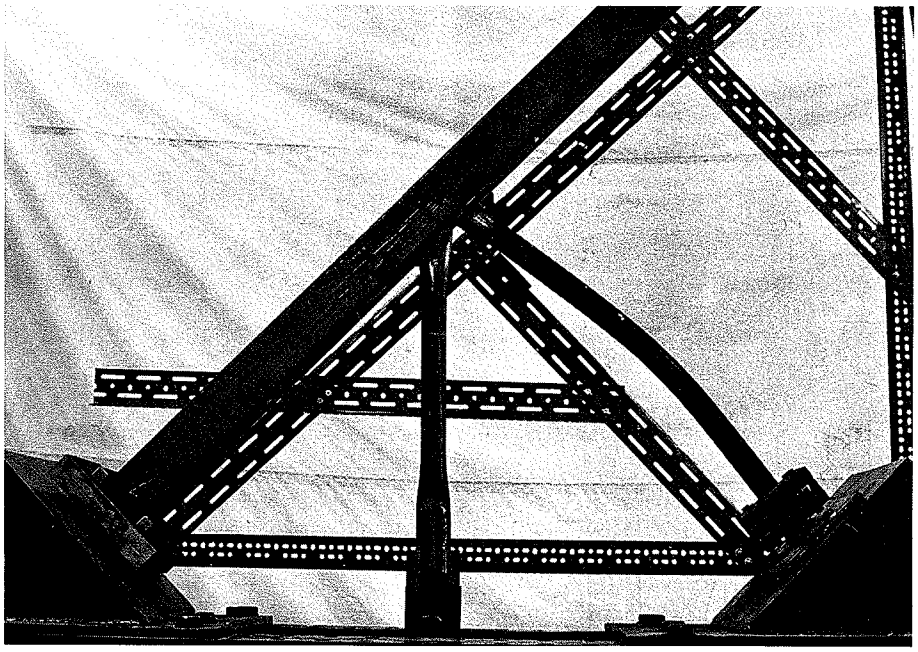


Fig. 4.4(j) Buckling of Specimen 2

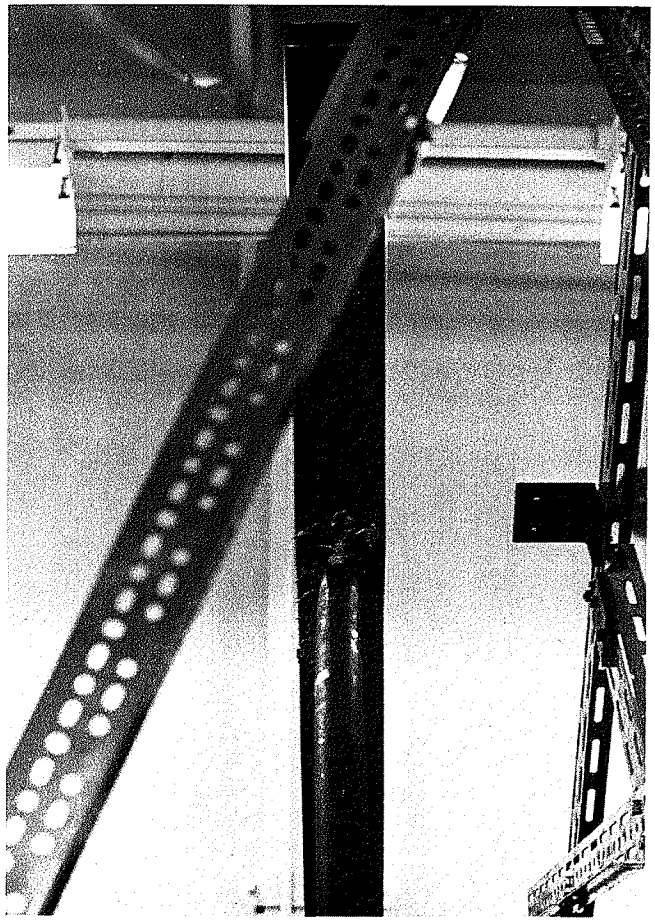


Fig. 4.4 (k) Specimen 1

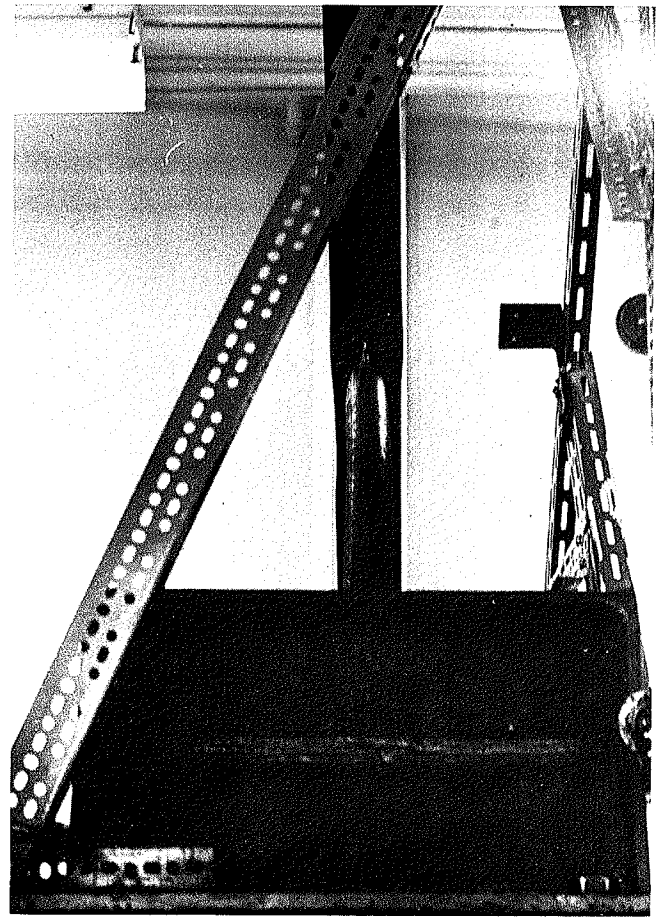


Fig. 4.4 (l) Specimen 2

It can be seen that for specimen 5A2, which was practically identical to specimen 1, the shape of chord-wall deformation was different to that of specimen 1. The chord wall under the webs of specimen 5A2 (and also 5A1) rotated as a unit because no weld fracture occurred.

The general shapes of the cut sections of specimens after failure are very similar. The deformation of the chord wall under the compression web was usually larger than that under the tension web, because of greater load per unit length, as discussed in Section 4.3. The maximum deflection normally occurred under the far end (from the joint) of the inside wall of the compression web, as denoted earlier by T3. This was due to the buckling of the compression web.

The chord-wall deformation and the buckling of the compression web obviously influenced each other. Initially, opposite and equal force components from the webs were applied perpendicular to the chord wall. They caused an inward and outward deformation of the chord wall under the compression and tension web, respectively. Since the distributed forces from the webs were applied eccentrically on the chord wall, they also caused the chord wall to rotate so that the compression web buckled away from the tension web, as shown in Fig. 4.4. The force in the compression web magnified the buckling in the web, which in turn caused the chord wall to rotate more.

Finally, each specimen failed by buckling of the compression

web; which, in the strict sense, was not a failure of the joint. However, the chord wall at the joint had already deformed excessively before the final buckling failure occurred, as shown by Fig. 4.4.

As has been pointed out, a chord-wall deformation of 0.05 inch was assumed in a previous study[12] to represent failure. However, such arbitrary definition of excessive deformation does not give the effect of this local deformation on the overall deflection of the joint. In fact, if a definition of excessive deformation is to be developed, it would probably be more appropriate to define it in terms of a percentage of the chord width (such as 1%, which is approximately the maximum allowable variation in cross sections of HSS) because an absolute value of deflection (such as 0.05 inch) would be less visible as the chord width increased.

The cross sections of specimens 1, 2 and 3 after failure indicated that the deflection under the compression web increased successively from specimen 1 to 3. That was because the magnitude of force distribution acting perpendicular to the chord wall increased from specimen 1 to 3. This was also true for specimens 4 and 5.

The point of maximum bending of the compression web was approximately at the quarter points of the truss depths, except that for specimen 1, the lap joint, it was slightly closer to the chord.

Fig. 4.4 also shows that a slight warping of the lateral

walls of the chord usually occurred at the joint.

4.5 Strain Distribution

The locations of strain gages in the vicinity of the joints are shown in Fig. 3.7 .

Applied loads in tension webs are plotted against measured strains in Fig. 4.5

The measured strains in the chord faces are consistent with the local chord deformation previously presented. The strain curves show that the maximum strains in the chord faces usually occurred at the location of gage 3. These strains were compressive and in the transverse direction.

Relatively large, tensile strains in the chord faces occurred under gage 11 again in the transverse direction.

When each joint reached the working load of its tension web, the strains at locations 3 and 11 were always larger than the member yield strain, calculated on the basis of the minimum yield strength of the steel (50 ksi). This indicates that the chord faces at these locations yielded at loads below the working loads.

Other compressive strains of the chord faces, in the transverse direction and in the order of decreasing magnitude, were at locations 3', 4', 5' and 6'. As would be expected, the transverse strains of the chord faces close to the tension webs were tensile and decreased from location 11 to 9 and from 9 to 7, respectively.

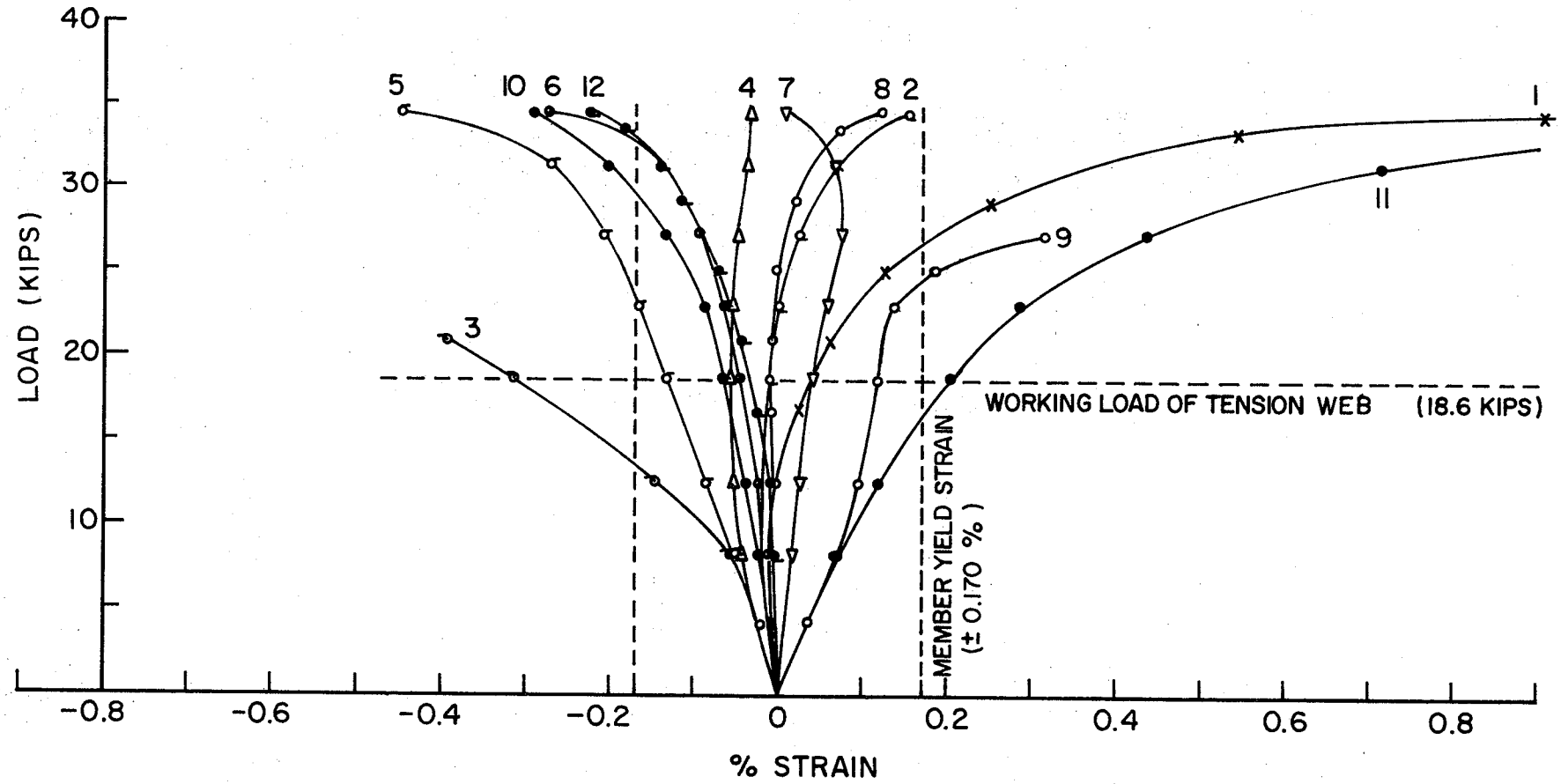


FIG. 4.5(a) LOAD IN TENSION WEB vs STRAIN IN CHORD - - - SPECIMEN NO. 1.

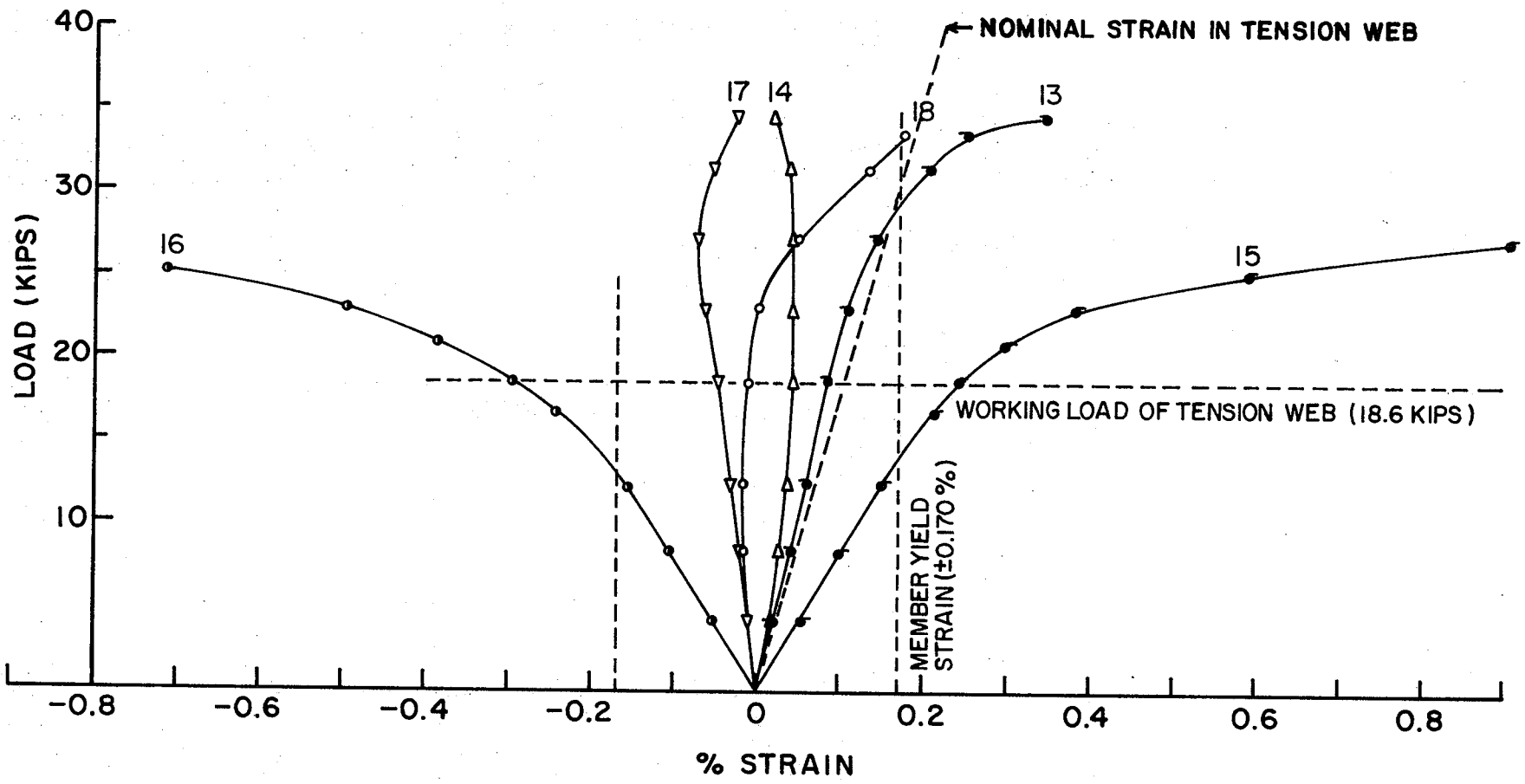


FIG. 4.5(b) LOAD IN TENSION WEB vs STRAIN IN WEBS - SPECIMEN NO. 1.

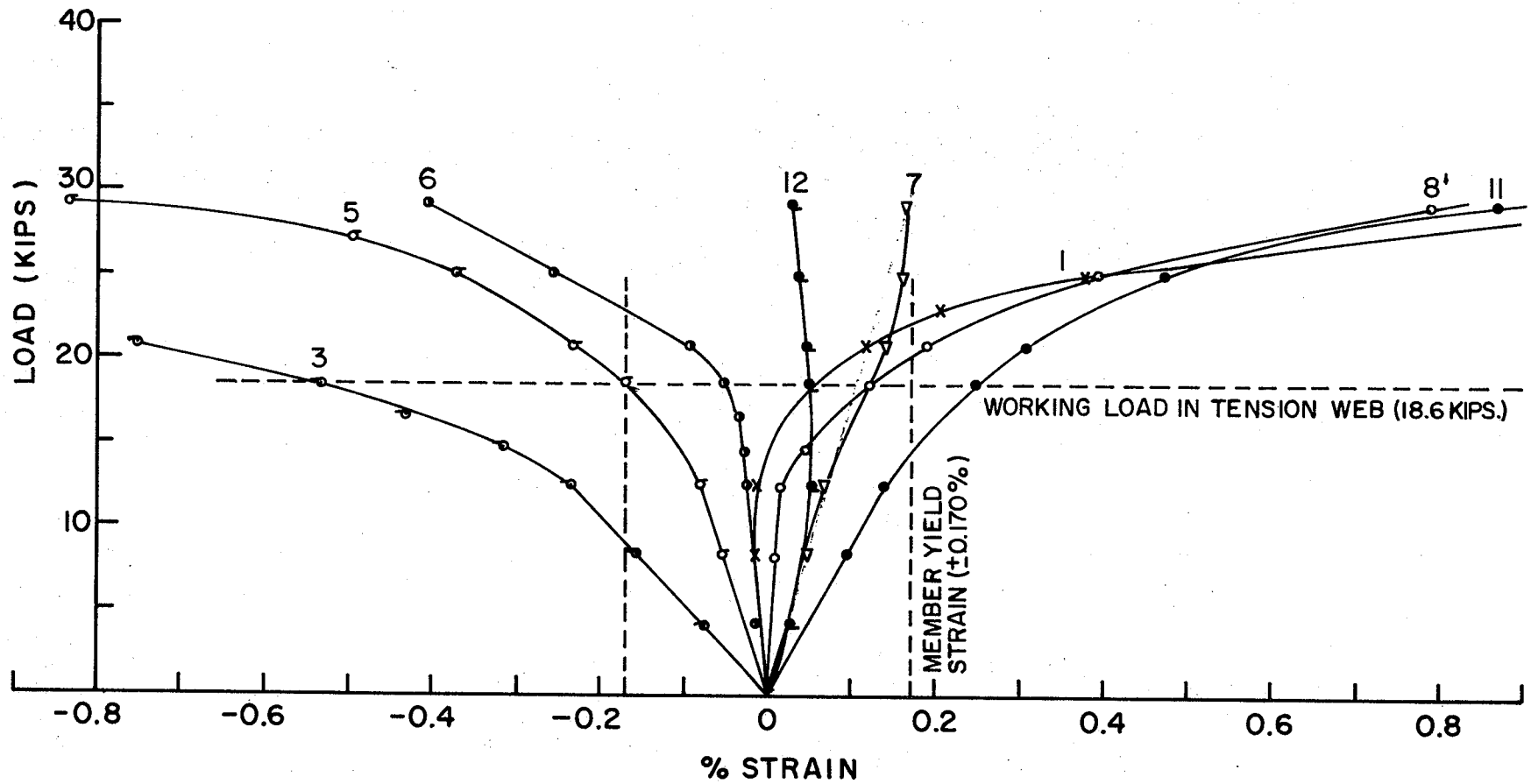


FIG. 4.5(c) LOAD IN TENSION WEB vs STRAIN IN CHORD - SPECIMEN NO. 2.

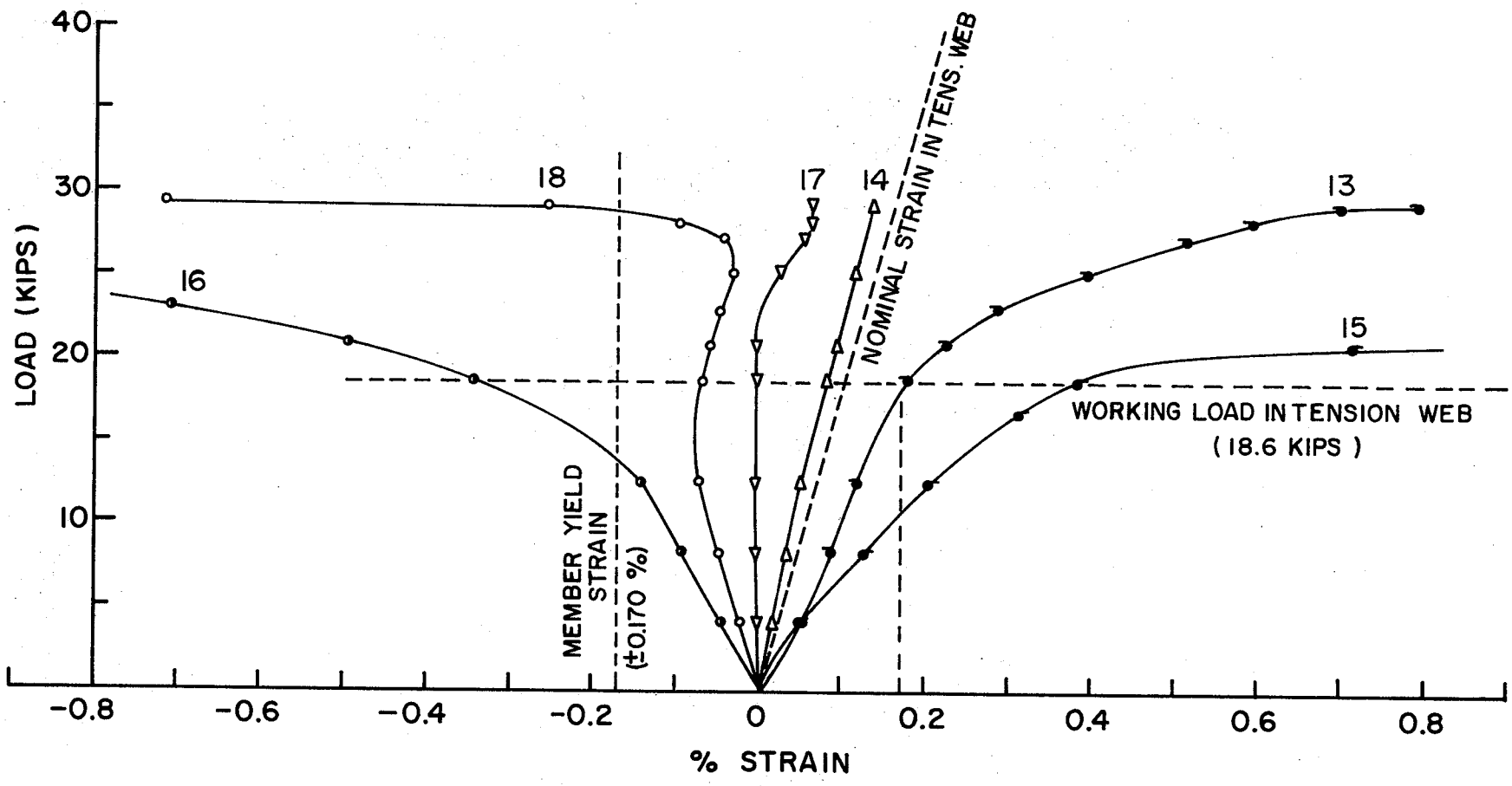


FIG. 4.5(d) LOAD IN TENSION WEB vs STRAIN IN WEBS - SPECIMEN NO. 2.

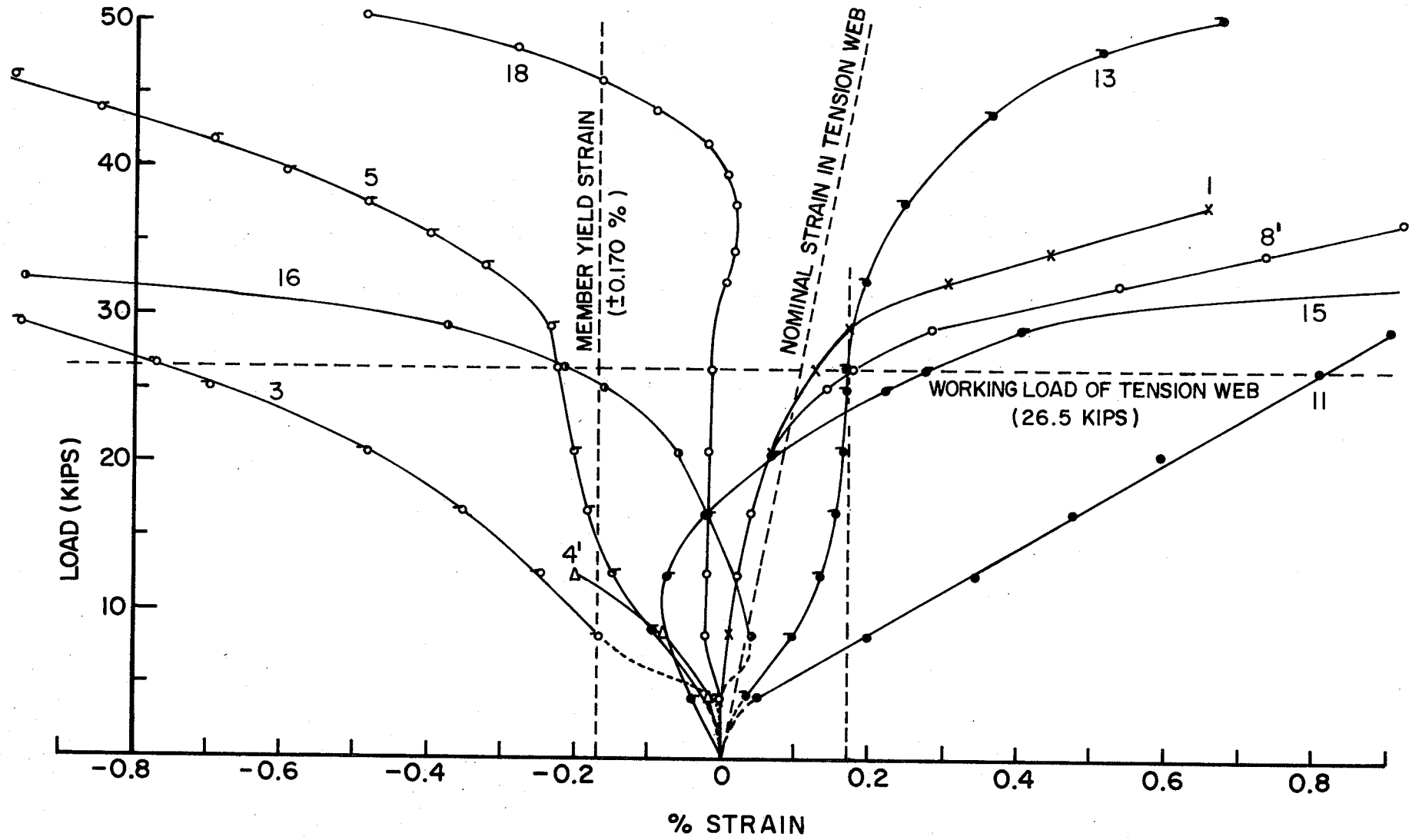


FIG. 4.5(e) LOAD IN TENSION WEB vs STRAIN IN CHORD AND WEBS - SPECIMEN NO. 3.

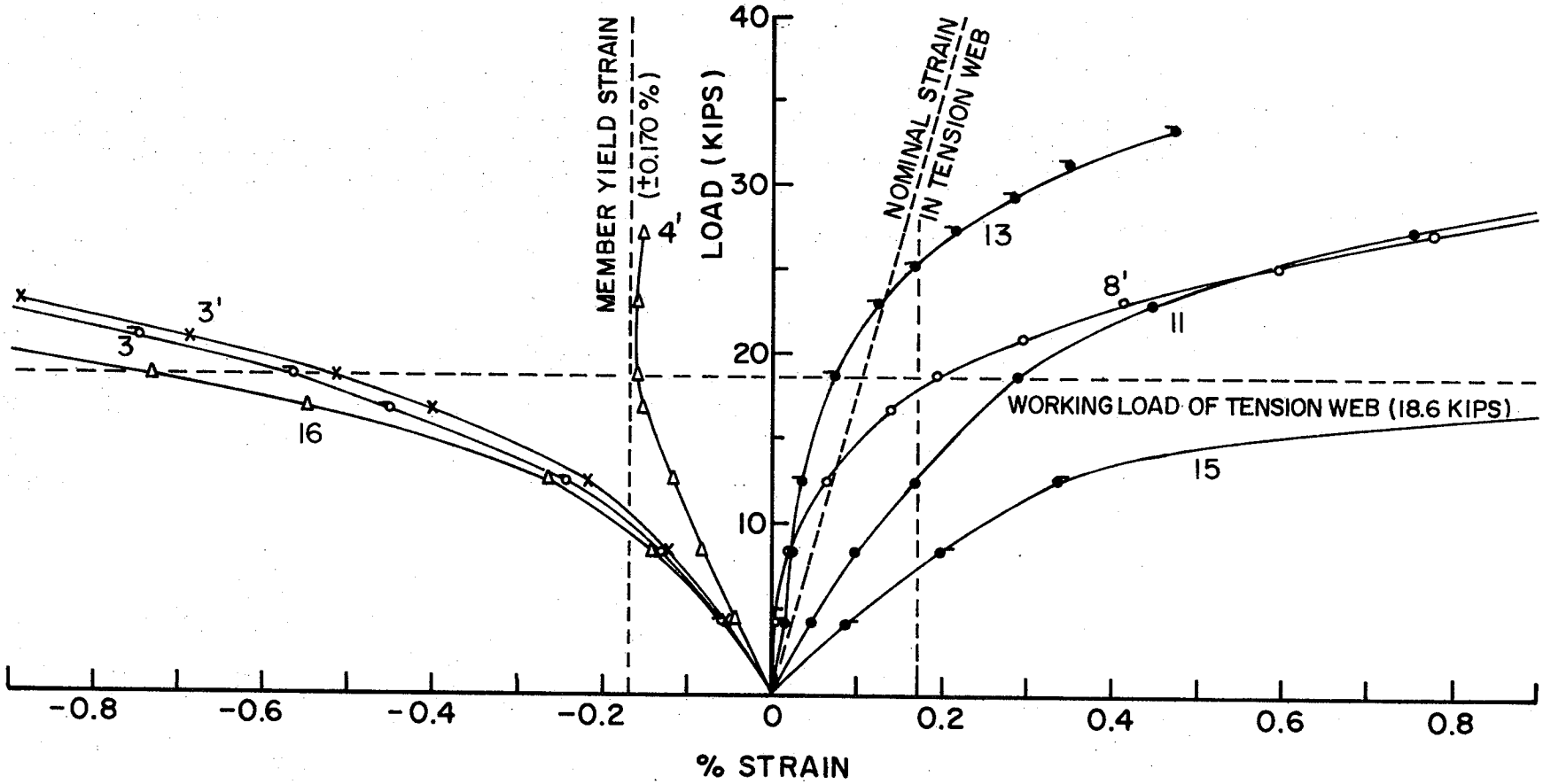


FIG. 4.5(f) LOAD IN TENSION WEB vs STRAIN IN CHORD AND WEBS- SPECIMEN NO. 4.

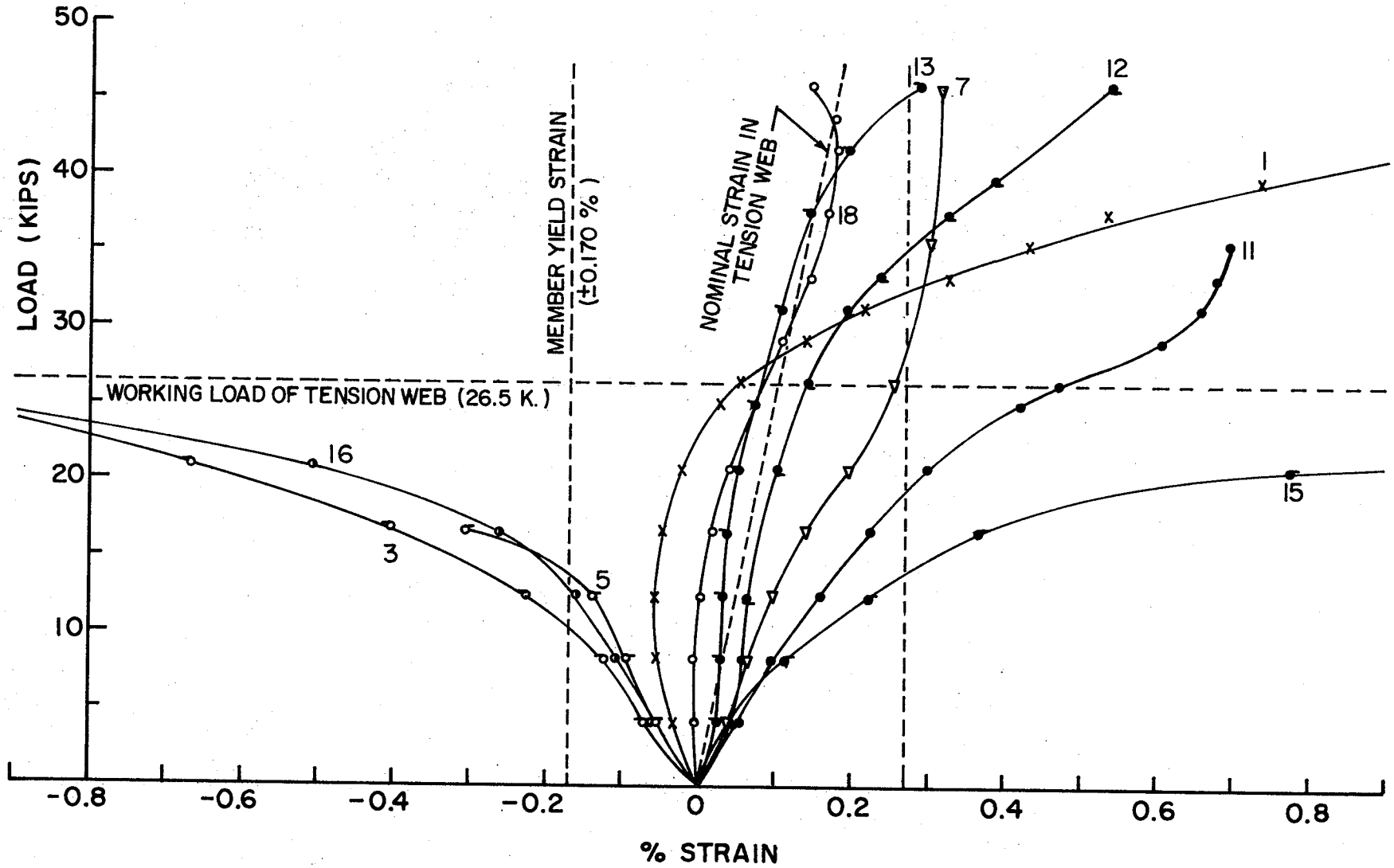


FIG. 4.5(g) LOAD IN TENSION WEB vs STRAIN IN CHORD AND WEBS- SPECIMEN NO. 5.

Thus the transverse strains in the chord faces around the webs decreased from the two far-side edges of the web system to the middle part, which confirmed the local deformation of the chord walls.

Strain gages 2, 3, 4 on specimen 1 indicated the transverse strain distribution across the chord face. The strains varied from compressive at the middle to tensile near the edges of the chord face, as would be expected from a bending moment diagram. A similar behavior, but in the opposite sense, was obtained from gages 9 and 10.

Strain gage 1 was usually in tension, whereas strain gage 12 varied from compression to tension.

The axial strains in the webs near the chord face were rather non-uniform. However, up to the working load, the nominal strain (load divided by nominal cross-sectional area and modulus of elasticity) of the tension web approximated very closely the average measured strains in this web.

The maximum axial strains in the webs occurred at locations 15 and 16. These strains reached the member yield strain at load levels below the working loads. The strain gage readings 15 and 16 were usually comparable to those of gages 11 and 3, respectively.

The axial strains in the webs indicated that some bending occurred near the ends of the webs of all specimens.

A comparison of the measured strains of specimens 1 and 2 showed that the joint generally reached the member yield strain

at a higher load (up to 48% for location 3) when the lap between the webs increased from zero to one inch.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The test results on the statical behavior of seven Pratt Truss (N-type) joints, composed of square hollow chords and round hollow webs with ends cropped and welded along the center lines of the chord faces, show the following.

1) The load factors of the joints (defined as the ratio of the ultimate load to working load of the web that fails first) range from 1.65 to 1.91; therefore the joint strengths appear to be satisfactory.
2) The load factors of the cropped joints with and without lap between the webs are slightly higher than those of similar, conventional joints with weld gaps between the webs.
3) The load factor increases (by 16%) with an increase in the lap between the webs (from zero to 53% of the web diameters).
4) A prestress (of 19% of the working load of chord) in the chord does not significantly affect the ultimate strength of the joint.
5) Cropped joints and flattened-end joints have approximately the same ultimate strength provided that the flattened length is small (less than about 1 inch).
6) The local deformation of the chord wall is decreased by about 50 per cent due to an increase in lap from zero to one inch.

7) The local deformation of the chord wall increases the mid-span deflection of a truss by the order of 60 per cent. Thus, cropped joints without lap and with a ratio of chord width to chord thickness greater than approximately 20 are very flexible. The working loads of the webs of such joints are governed by the deformation of the chord wall.

8) As would be expected, the elastic stiffness (defined as the minimum ratio of the normal load component to the corresponding deflection of the chord wall in the elastic range) of the connection increases with a decrease in the ratio of chord width to chord thickness.

9) The deformation of the connected face of the chord wall is non-uniform because of the couple produced by the normal components of the web forces. This rotational deformation induces an end moment in the compression web and causes it to finally fail by buckling, primarily in the plane of the truss and away from the tension web. The buckling strength of the compression web, therefore, governs the ultimate strength of the joint.

10) Every joint tested initially failed by an excessive deformation of the connected face of the chord wall and finally failed by buckling of the compression web. A weld fracture at the lap also occurred for one of the lap joints.

11) The strain distribution around the connection is non-uniform, relatively large strains occurring in the middle plane of the specimen. The maximum strains in the chord faces are in the

transverse direction close to the webs, whereas the maximum strains in the webs occur near the junction of the webs. The strains in these locations generally reach the yield point at a load approximately half of the working loads of the webs.

5.2 Recommendations

In essence, this study shows that cropped joints without laps between the webs are very flexible, but they can be used for joists with relatively small normal forces on the chord faces. Further, the allowable normal force on the chord wall in a cropped joint, unlike a conventional joint, is not necessarily increased if the web diameters are increased. However, this disadvantage may be compensated by overlapping of the webs, which is not a difficult problem for a cropped joint.

Therefore, any further investigation of cropped joints involving square HSS chords should be limited to joints with some overlap between the webs, otherwise the ratio of chord width to chord thickness should not be greater than approximately 20.

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