THE UNIVERSITY OF MANITOBA

STATICAL BEHAVIOUR OF TUBULAR TRUSSES WITH CROPPED-END CONNECTIONS

by

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A dissertation submitted to the Faculty of Graduate Studies of the University of Manitoba in partial fulfillment of the requirements of the degree of

MASTER OF SCIENCE

TO MY PARENTS

MR. AND MRS. S.R. GHOSH

CONTENTS

ABSTRACT

Tests of two similar Pratt trusses of twenty-four foot span, each involving four cropped-end tubular steel web members and square hollow structural section chord members are reported and results are compared with those obtained on isolated joint tests done at the University of Manitoba. Trusses designed in such a way that the four test joints in a truss would be consecutively loaded to failure are described. Each time a joint failed, it was suitably reinforced and the loading was resumed to cause failure of the next joint. The joints having slender web members failed due to buckling of the compression web members out of the plane of the truss. The joints with stocky web members failed due to excessive deformation of the chord face at the joint. The corresponding joints when tested in isolation failed primarily due to excessive deformation of the chord face at the joint. The failure loads of the joints in truss tests were twenty-five to thirty percent lower than in the isolated joint tests.

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NOTATION

d = outside diameter of web tube

d_o = outside diameter of chord

b_o = chord width

t_o = thickness of chord tube

t = thickness of web tube

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

F_c = load in compression member

F_T = load in tension member

CHAPTER I

INTRODUCTION

1.1 General

In recent years, Hollow Structural Sections (H.S.S.) have been used extensively in truss construction, partly because of an increase in the availability of a wide range of section sizes and strength grades. Moreover, hollow structural sections have many advantages over other types of rolled sections when used as truss members. A weight saving of the order of 20 percent is quite common for tubular trusses[26]. This is because H.S.S. have high compressive, bending and torsional resistance as well as good lateral stability. The closed cross-sectional shape of H.S.S. minimizes the exposed area and hence makes corrosion protection and painting relatively inexpensive. Finally, tubular trusses with exposed structural members are aesthetically pleasing.

A major disadvantage to the use of hollow structural sections in trusses is the high cost and difficulty of fabrication of the joints. The ends of the web members require careful profiling in order to obtain a proper fit between the webs and the chord face. The welding of the members, especially in the acute angles between them, becomes fairly difficult.

End cropping, a recently developed fabrication procedure which involves simultaneously flattening and shearing the ends of the web members, permits an economical web-chord connection. The cropping not only eliminates profiling, but also reduces the cutting time, as the tubes can be rapidly squeezed and cut in one operation in a cropping machine. Moreover, the welding is simplified, since only straight-line fillet welds are required.

In the past, cropped joints have been successfully used for secondary joints carrying light loads[1,3,5]. Their behaviour requires extensive experimental investigation before they can be safely used in primary joints.

A testing program was recently begun at the University of Manitoba, [21] to investigate the properties of cropped connections for use in tubular

Twenty-nine specimens, each consisting of a single Pratt truss joint and the portions of the truss webs and chords extending half way to the adjacent joints, were tested to destruction. While the results obtained in the isolated joint tests were consistent and reasonable, it was felt desirable to compare the isolated joint behaviour with that of similar joints incorporated into tubular trusses.

1.2 Object of Investigation

The object of this study was therefore to examine the behaviour of cropped joints incorporated into trusses and to compare it to that of similar joints previously tested as isolated joint specimens. A range of isolated truss joints tested by Morris and Thiensiripipat[27] was examined and four types of joints were chosen to be incorporated into each of two similar full scale trusses. The trusses were designed in such a way that the four test joints in a truss would be consecutively loaded to failure. Each time a joint failed, it was suitably reinforced and the loading was resumed to cause failure of the next joint.

1.3 Limitations of Investigation

The limitations of this investigation were:

(1) The investigation was entirely experimental; no analytical study of joint behaviour was undertaken.

(2) Static loading only was considered.

(3) The truss chords were all H.S.S. 4 x 4 sections and all webs were round H.S.S.

CHAPTER II

TUBULAR TRUSS CONNECTION

2.1 Introduction

In this chapter, the failure modes and the parameters affecting joint behaviour are discussed. A brief summary of past work on both conventional and end-cropped (or end-flattened) joints is also presented.

2.2 Failure Modes

A tubular truss joint may exhibit one of several modes of failure, depending upon which joint component - chord, compression web, tension web or weld, is the weakest. Different modes of failures are:

(1) Excessive local deformation of chord wall.[1,6,9,11,17,21]

(2) Buckling of compression web.[1,6,9,10,11,17,21]

(3) Fracture of weld.[1,6,9,17,21]

(4) Rupture of tension web.[5,6,9,17,21]

(5) Collapse of wall of compression web.[11,17]

The most commonly observed type of failure has been excessive rotational deformation of the chord face, caused by the adverse load condition at the joint.

It has been found in most cases that chord face deformation has been excessive and has rendered the structure useless long before the ultimate failure of the joint. Consequently, joint deformation has been used by many investigators as a failure criterion.

Eastwood et al.[10] used a chord face deformation of 0.05 inches as a failure criterion. Morris and Thiensiripipat[22,27] suggested that the allowable chord face deformation be a value proportional to the chord size, the rationale being that allowable beam deflection is proportional to beam length. They recommended an allowable deformation of one percent of the chord width.

Other investigators used alternative methods to determine failure. Anderson[1] and Bouwkamp[4] used ultimate failure criteria such as buckling, tension failure, and weld fracture in the evaluation of their test specimen.

Blockley et al.[2] defined failure according to the rate at which deflection was occurring. A movement of the loading ram of 0.045 inches per minute under no increase in loading was assumed to indicate failure of the joint.

In general, the chord wall deformation 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 occurs at the lap between the webs.

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

On the other hand, if the wall of the compression web is relatively thin, it may buckle locally and produce failure. However, this type of failure may be prevented by requiring that the diameter of the compression web not exceed 3000/Fy times the thickness of the section[8], where Fy is the specified minimum yield strength for the web material.

2.3 Measures of Joint Performance

The most common measures of joint performance are strength, stiffness and stability.

In order to compare strengths of different joints, previous investigators have represented joint strength in dimensionless form as either joint efficiency[6,21,28] or joint load factor[1,2,7,12,21].

The joint efficiency has been defined as the ratio of the failure load for the joint to the strength of the tension web, or as the yield strength of the first member to yield.

The joint load factor has been defined as the ratio of the failure load for the joint to the design load of the tension web or the compression web. Hence, the joint strength may be represented either as a tension load factor or a compression load factor, depending which member governs the design.

In this study, the joint load factor based on the tension webs is relatively easy to calculate, because no slenderness ratio need be taken into account. However, the values are meaningless when ultimate strength is governed by the buckling strength 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. 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[24] as has been recommended for webs in conventional tubular trusses.

While joint stiffness is usually defined as the initial slope of the load-deformation plot for the joint, in this study, the average slope in the elastic region was used.

Another measure of joint performance usually considered is the stability of the joint. The load-deflection curve indicates the load at which the chord walls and the joint become unstable and yield or fail by local or overall instability.

2.4 Parameters AFfecting the Joint Performance

There are a number of parameters which affect performance of a tubular truss joint[1,6,10,17,21,28]. As illustrated in Figure 2.1, these are:

(1) The ratio of chord thickness to chord width (diameter), t_o/b_o.

(2) The joint eccentricity, e, or the gap or lap.

(3) The ratio of web diameter (width) to chord width (diameter), d_1/b_o.

(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), t_1/d_1.

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

2.5 Behaviour of Conventional Tubular Truss Joints

Several investigators[1,10,11,12,17,28] have found that the behaviour of conventional tubular truss joints is governed primarily by the first three of the parameters, listed above.

(1) Ratio of chord thickness, t_o to chord diameter, d_o, for a round chord, or chord thickness to chord width, b_o, for a rectangular chord. The smaller t_o/b_o, the more flexible the joints and the smaller the resistance to buckling of the compression web.

(2) The spacing of the web members where they meet the chord face. It has been expressed in terms of the lap of the webs, the gap between

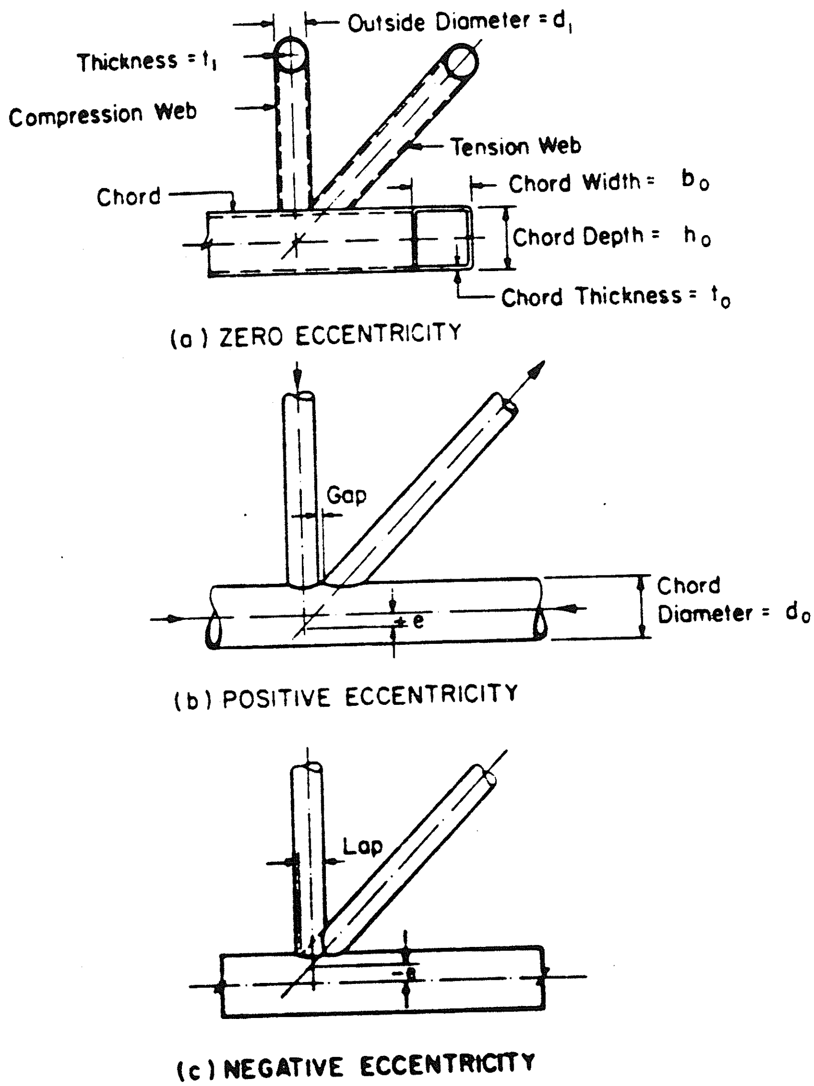


FIG. 2.1 PARAMETERS AFFECTING JOINT BEHAVIOR

hem, or an eccentricity, as illustrated in Figure 2.1. Lap joints, which often have negative eccentricities, exhibit greater strength and stiffness than do gap joints, which usually have positive eccentricities.

(3) The ratio of compression web diameter, d_1, to chord diameter, d_o, (width b_o, for a rectangular chord). For a conventional H.S.S. truss joint, this parameter is significant because the larger d_1/b_o, the more directly the web member forces are transmitted to the side walls of the chord. Hence, the larger d_1/b_o, the stiffer and stronger is the joint.

Bouwkamp[6] found that an increase in the t_o/b_o ratio relieved the radial bending stress in the chord tube wall and resulted in a stronger joint. Furthermore, he found that the effects of varying the d_1/b_o and t_1/b_o ratios became less significant as the degree of overlap of the web members increased.

Eastwood et al.[11] confirmed Bouwkamp's findings and in addition, found that when the load transfer between the webs occurred through the chord, the local joint deflections were reduced by increasing the ratio d_1/b_o. This was due to the fact that more load was transmitted through the side walls of the chord.

The effect of t_o/b_o and d_1/b_o on joint efficiency, for joints involving round chords with varying degrees of lap, are illustrated graphically in the Stelco H.S.S. design manual for connections[26]. The joint efficiency increases with an increase in d_1/b_o. Changes in d_1/b_o significantly affect the joint performance, especially for joints involving thick chords.

For joints involving a gap between the web members, increases in d_1/b_o and t_o/b_o tend to increase the overall stiffness of the joint and to reduce local bending stresses.

Problems arise in joints involving square chords when the web members are welded to a relatively thin chord face, as commonly occurs in economical truss design. A connection of this type usually experiences relatively large local deformation of the chord face at the joint, unless stiffening members are welded to the chord face or the members overlap. Furthermore, this deformation occurs at a load very much below the elastic limit of the members, rendering elastic design useless.

Eastwood and Wood[13] have expressed the ultimate load of a gap joint as a function of chord thickness. Design tables and charts have been compiled to relate the maximum vertical load that can safety be applied to the rectangular chord member, to the parameters t_o/b_o and d_1/b_o[26].

2.6 Flattened-end and Cropped-end Joints

Jamm et al.[19] suggested that flexibility of pipe sections used as truss chords should be maintained by localizing the weld, a common location of joint failure, so that it runs over a relatively narrow part of the chord surface. If the web tubes are relatively large, this can be achieved by flattening their ends. The end-flattening may result in only partial overlapping of the webs, causing only partial transfer of shear forces through the chord tube wall. It was also claimed that 100% joint efficiency can be achieved if the depth of the connection between the compression and tension web members is greater than 0.15 of the chord tube diameter.

Hlavacek[17] found that the joint efficiency is 20 percent lower for joints with flattened-end tubular web members, whether connected longitudinally or transversely to the chord, than that for conventional joints. He suggested that flattened lengths of the webs should not be more than 0.8 to 1.2 inches, otherwise the compression web may buckle at a lower load than expected. He also stated that crosswise flattened ends were less safe against buckling than were lengthwise flattened ends, since the former could not transmit bending moment in the plane of the joint.

Mouty[24] stated that, in order to obtain an effective length factor of 0.7, as is permitted for webs of conventional tubular trusses, the degree of end flattening must not be greater than one-third of the web 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 ratio of web diameter to chord diameter, d_1/d_0 is 0.25 and 0.33 respectively[4]. Bouwkamp suggested, in this context, that the web diameter d_1, be replaced by the reduced width of the flattened end of the end-flattened webs.

Investigations involving cropped-end web members were performed by Anderson[1], who found that joints involving cropped-end members had local stresses which were much higher than those with profiled ends. The intensity of the stress was reduced with an increase in lap of the web members. It was noted, however, that the increased stresses in joints involving cropped-ends did not lead to lower ultimate loads. Anderson[1] attributed this to the following factors:

(i) The joints could relieve overstress by stress 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 compression web under eccentric load.

(iii) The normal loads at the joint were distributed over a greater axial length of the chord than those in profile joints.

(iv) Part of the transfer of normal load at one joint occurred through direct connection between the webs.

(v) The chord stress became less severe by incorporation of negative joint eccentricity.

Anderson[1] recommended cropping as a safe and economical substitute for in joints composed of round tubes of similar proportion to those investigated, provided that the loads were static. He further recommended that a positive joint eccentricity should be provided to incorporate a robust direct connection between the webs. Since, however, direct connection contributes to the mal-distribution of stress, such a connection in structures subjected to major dynamic load might be dangerous.

Morris, Frovich and Thiensiripipat[21] found that stiffness and yield loads of flattened-end connections involving square chords were approximately one-third and one-quarter, lower respectively, than those of connections involving round chords. Furthermore, their ultimate loads were within ten percent 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.

2.7 Summary of Previous Work

2.7.1 Conventional Joints (Webs not End-cropped)

From the findings of previous work on conventional tubular truss joints (ones with profiled webs), the effect of overlapping of webs, and of the ratio of chord thickness, t_o, to chord diameter, d_o, for a round chord, or chord thickness to chord width, b_o, for a square chord, can be summarized as follows:

(1) Lap joints are stronger than gap joints.

(2) Load factors for cropped-end joints increase with an increase in lap between the webs.

(3) An increase in lap between the webs results in an increase in joint stiffness.

(4) The intensity of the local stresses can be reduced by increasing the overlap of the web members.

(5) A joint efficiency of 100% was achieved when the depth of connection between the webs was greater than 0.15 times of the chord diameter.

(6) Joints with small t_o/b_o ratio are relatively flexible and hence have relatively small resistance to buckling of the compression web.

(7) When the ratio of diameter of the web, d_1, to the width of the chord, b_o, is increased there is an increase in the stiffness of the joint.

(8) An increase in the ratio of t_o/b_o(or t_o/d_o) relieves the bending stress in the chord tube wall and results in a stronger joint.

(9) The minimum allowable value of the ratio of web diameter to chord diameter has been specified as 0.25 (German Code) and 0.33 (British Codc).

2.7.2 End-cropped and End-flattened Joints

The following is a summary of the findings of previous research on end-cropped and end-flattened joints:

(1) Joints with crosswise flattened webs are less safe against sideways buckling than are those with lengthwise flattened ones, since the former could not transmit bending moments in the plane of the truss.

(2) The stiffness, yield load and ultimate load are not significantly affected by either the direction or the method of end-flattening.

(3) While Hlavacek[17] found that the joint efficiencies are generally 20 percent lower than for conventional joints, Morris et al.[21] found that joint load factors for overlapping end-flattened joints are close to those of conventional joints.

(4) Cropped-end joints produce higher local stresses than do profiled joints.

(5) The intensity of the local stresses can be reduced by increasing the overlap of the web members.

(6) The increased local stresses in the cropped-end joints do not lead to a lower ultimate load.

From the findings of past work on both conventional and end-cropped joints, it can be seen that lap joints are more favorable than gap joints. However, according to Morris and Thiensiripipat[22], an increase in lap of more than 50 percent has virtually no effect on the joint strength. Both the strength and stiffness of the joint increase with the ratio of t_o/d_o and d_1/b_o.

It can be concluded from previous work that statically loaded cropped-end connections with overlap between the web members can be used as an economical, safe substitute for the profiled-end joints.

2.8 Comparison of Behaviour of Isolated Joints with that of Joints in Actual Truss

Dasgupta[9] performed a series of tests on full scale trusses incorporating ten joints similar to ones that had been tested as isolated truss joints at Sheffield University. He found that the ultimate loads of the isolated joints were higher than those of corresponding joints in full scale trusses. He explained the differential as follows:

The basis of the isolated joint tests was the assumption that points of contraflexure develop at the mid-points of all members in a truss under the action of secondary moments. Therefore, it was thought that if the part of the truss around a joint and between the points of contraflexure were isolated, the truss conditions would be properly represented. However, this arrangement is not capable of simulating the boundary conditions completely as the inflection points were not allowed to undergo relative displacements. The secondary truss moments are, therefore, not developed in the isolated joints. Dasgupta also showed that the points of contraflexure develop near the mid-points of web members only, and the positions of the points of contraflexture in the chord members, if the points exists at all, may not necessarily be near the mid-points. Therefore, the force boundary conditions at the ends of the chord members may not be simulated in the isolated joints.

Dasgupta analysed the truss using a semi-rigid analysis computer program. He compared the bending moments with those obtained from full scale truss analysis. The secondary bending moments in the branch members of the isolated joints were negligible in comparison to those in the full scale truss.

He concluded that these secondary moments cause high stress concentrations at the joint and as a consequence, the trusses fail at relatively low loads. On the other hand, the absence of these moments increases the local deflection and it was found that all of the Sheffield isolated joints failed due to excessive local deflection.

Finally, he suggested that the results of isolated joint tests should be reviewed in the light of additional moments which would be imposed on the joints if they acted as a part of a complete structure.

CHAPTER III

EXPERIMENTAL PROCEDURE

The objective of the experimental work was to study the behaviour of four joints in a full truss setting, and to compare it with that of similar joints that had previously been tested in isolation by Morris and Thiensiripipat[22]. Since all four joints were incorporated into a single truss, the latter was designed in such a way that the four joints would be consecutively loaded to failure. Two identical trusses, each incorporating the four test joints, were loaded to destruction. This chapter deals with choice of test joints, details of design of the truss, measurement of displacement and strain, and also a brief description of the testing and reinforcing sequence of the joints.

3.1 Choice of Joint

It has been concluded by a number of investigators[6,9,10,13,22,28] that overlapping H.S.S. truss joints are stronger and stiffer than similar lap joints. Morris and Thiensiripipat[22] found that joint strength increased by 10 to 15 percent, as web member lap was increased from zero to 50 percent. A further increase in lap had virtually no effect on joint strength. Thus, when selecting joint configurations to be incorporated into the test trusses, it was decided to use four joints, all with 50 percent lap, representing two extreme values of the other geometric parameters considered in the isolated joint tests. The configurations of the joints chosen are given in Table 3.1 and illustrated in Figure 3.1

The joint designations indicated in Table 3.1 are those that were

TABLE 3.1 SPECIFICATION OF THE EXPERIMENTAL JOINTS

Joint Designations	Chord HSS-b_oxd_oxt_o (in.)	HSS-d_1(O.D.)xt_1 (in.)	Lap (in.)	Joint Ecc.,e (in.)	$\frac{b_o}{t_o}$	$\frac{d_1}{b_o}$	$\frac{e}{d_o}$	$\frac{d_1}{t_1}$
3A50	4x4x0.188	1.66 O.D. x 0.125	0.88	0.25	21.3	0.415	0.063	13.28
4A50	4x4x0.250	1.66 O.D. x 0.125	0.88	0.25	16	0.415	0.063	13.28
3C50	4x4x0.188	2.875 O.D. x 0.250	1.44	0.50	21.3	0.718	0.125	11.5
4C50	4x4x0.250	2.875 O.D. x 0.250	1.44	0.50	16	0.718	0.125	11.5

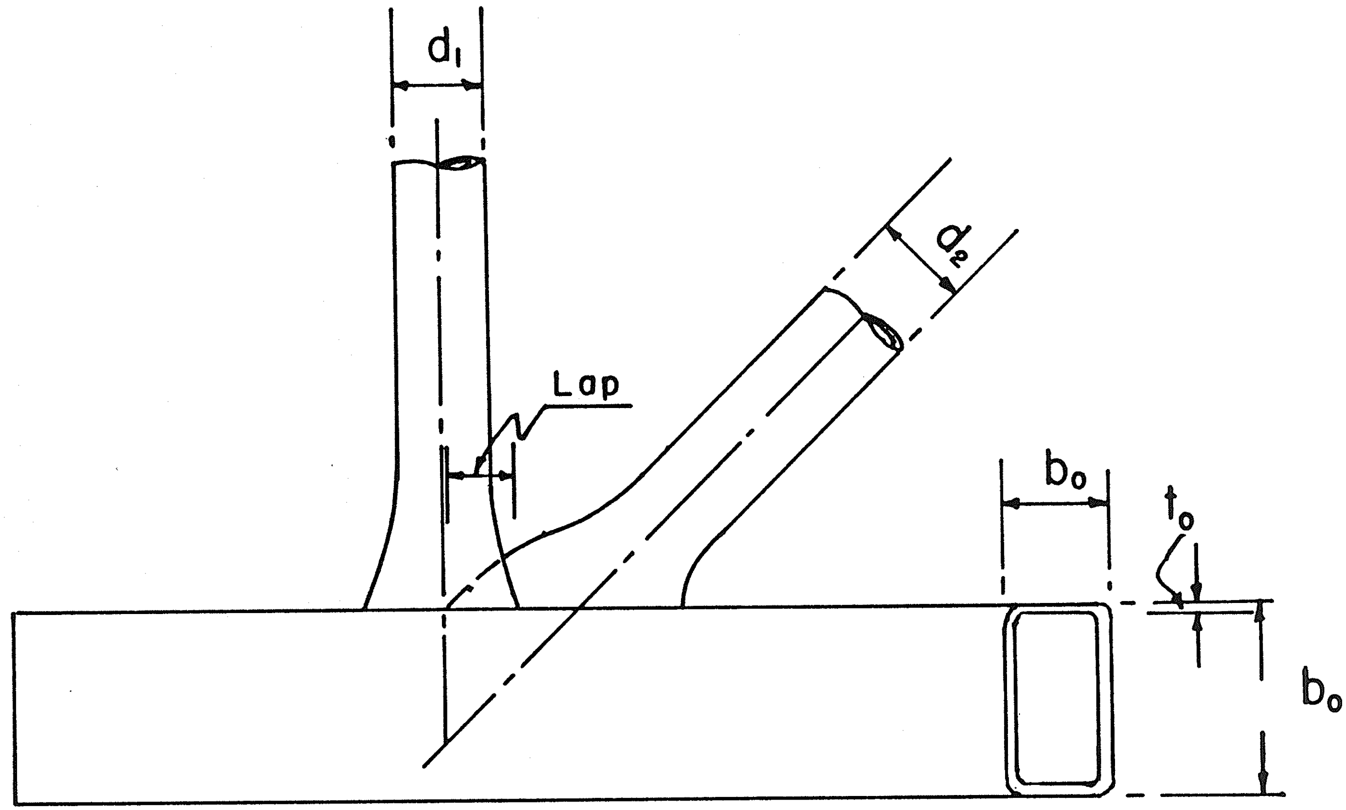


FIG. 3.1 TYPICAL JOINT GEOMETRY

used in the isolated joint tests[22]. The first character used in the joint designation was a digit indicating the corresponding chord thickness/width ratio. The next character was a letter indicating a particular web diameter/chord width ratio. The last two characters, two digits, corresponded to the lap of the web members as a percentage of the web diameter.

3.2 Design of Truss

A Pratt truss was selected, so that the joint geometries would be similar to those of the isolated joints tested by Morris and Thiensiripat[22].

The isolated test joints were designed to represent joints in a Pratt truss with a depth of 72 inches and a panel length of 72 inches. The joint was supposed to represent the part of the truss between points of contraflexure, which were assumed to occur at the mid-points of all of the members. In view of this, it was decided to test trusses with a height of compression member of approximately 68 in. The depths of the trusses were 72 in. centre to centre of chord members. The truss geometry is shown in Figure 3.2. The top and bottom truss chords were of the same outside dimensions, but they had different thicknesses in order to represent two different types of joint at the two ends of the truss verticals.

The trusses were tested under simply supported end conditions, provided by hold-down connections to a structural floor, and the loads were applied vertically upward at midspan. Although the self weight of the truss acted in the direction opposite to that of the applied load, it was decided to neglect the self-weight effects, the weight of the

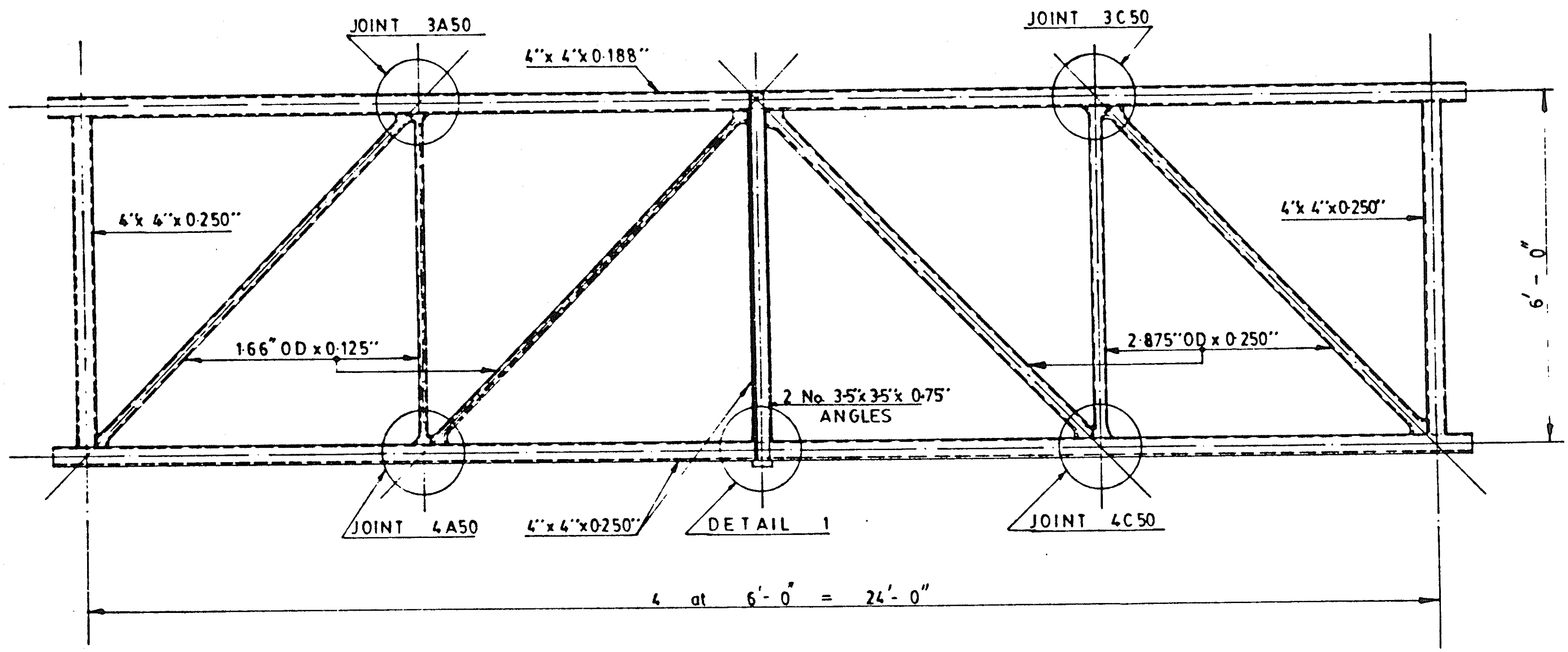


FIG. 3.2(a) TRUSS GEOMETRY

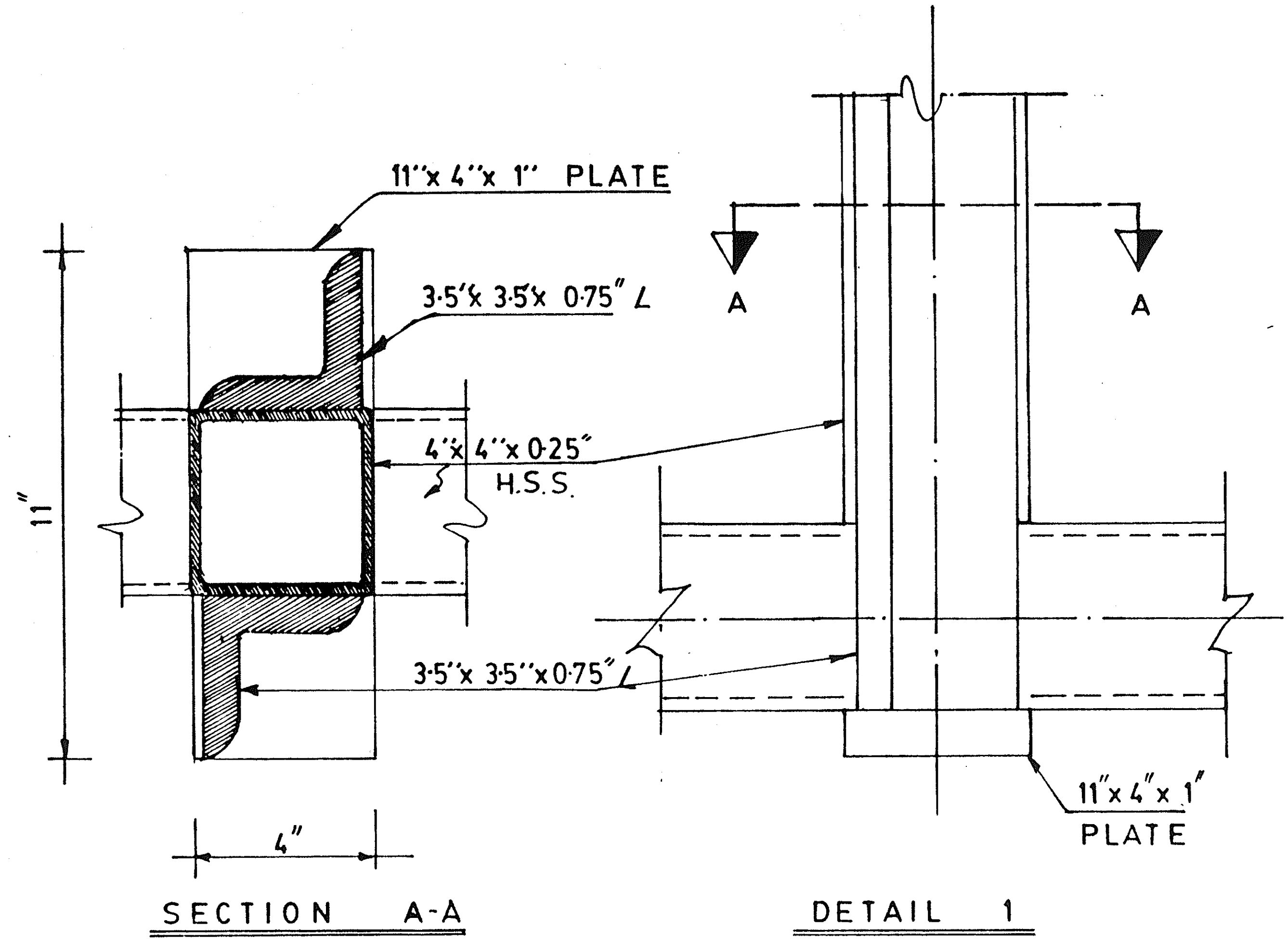
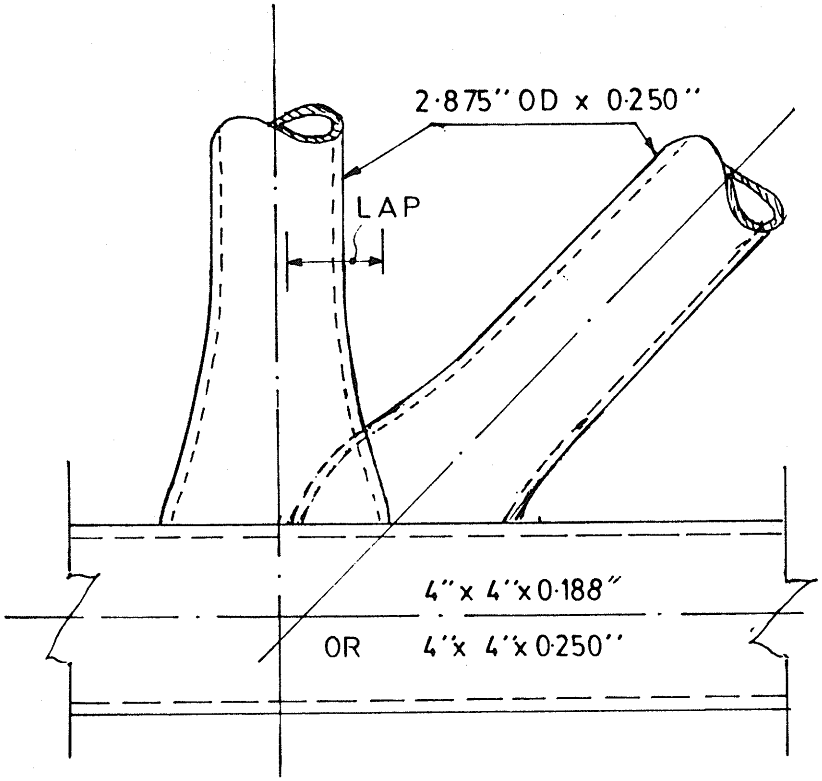
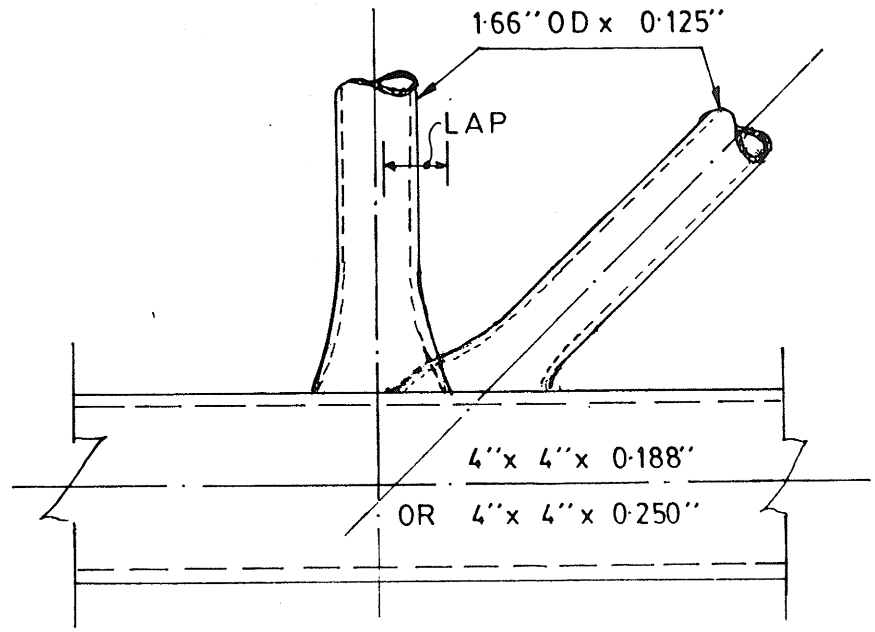


FIG. 3.2(b) TRUSS GEOMETRY



DETAIL OF JOINTS 3C50 AND 4C50



DETAIL OF JOINTS 3A50 AND 4A50

FIG. 3.2(c)

trusses being very small. To avoid premature failure of the central compression web, it was fabricated from a 4" x 4" x 0.250" H.S.S. reinforced with two 3-1/2" x 3-1/2" x 3/4" angles, as illustrated in Figure 3.2(b). To ensure the transmission of loads to the truss without collapsing the chord where the loading was applied, a 1 in. thick plate was welded to the chord face, as illustrated in Fugure 3.2(b), detail 1.

3.3 Materials

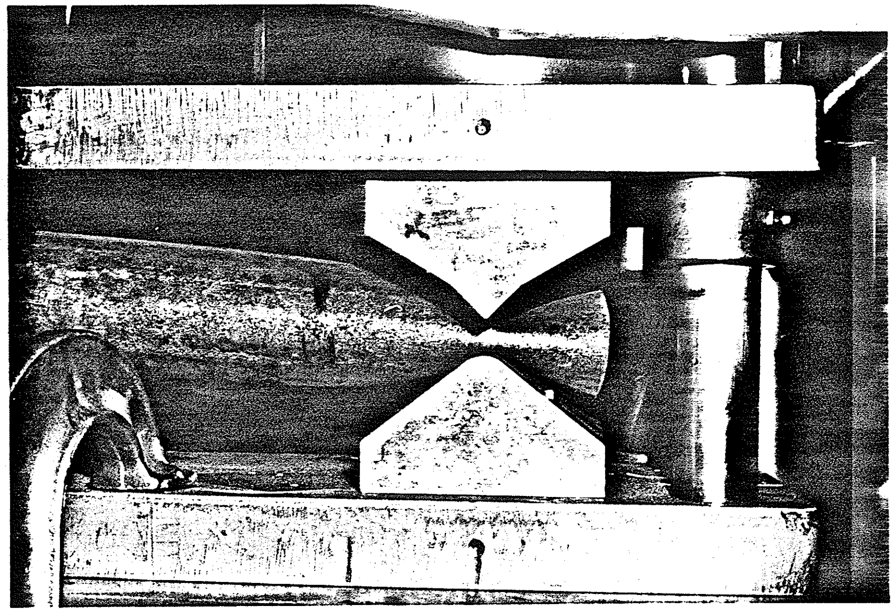
Round hollow structural sections (H.S.S.) used for the webs were hot formed, conforming to CSA specification G40.21 Grade 50W. The H.S.S. 4 x 4 x 0.188 and 4 x 4 x 0.250 used as chords and webs, conformed to specification G40.21 Grade 50W Class H. Tension tests conducted at the University of Manitoba indicated yield strengths in the range of 47 ksi to 54 ksi, as indicated in Table 3.2.

3.4 Fabrication of Trusses

The cropping of web members was carried out using a specially designed cropping machine, in the University of Manitoba structures laboratory. The cropping of web members is illustrated in Figure 3.3. The cropping machine consited of two V-shaped tungsten chromium tool steel blades attached to alignment guides. The cropping force was supplied by a 600,000 lb capacity Universal Testing Machine. The blades were designed for a cropping force of up to 200 kips. This force was sufficient for 1.66 OD webs. However, for the 2.875 OD webs, it was necessary to simulate the end-cropped geometry by first squeezing the tube with the cropping machine, then cutting it at the appropriate location with a mechanical

TABLE 3.2 YIELD STRENGTH OF CHORDS AND WEB MEMBERS OF THE TRUSSES

Specimen	Sample	Area (in^2)	Yield Load (Kips)	Yield Strength (Ksi)	Average Yield Strength (Ksi)
1.66 O.D. x 0.188 (web)	1	0.062	3.36	54.5	53.9
	2	0.062	3.29	53.4	
2.875 x 0.250 (web)	1	0.125	6.50	52.0	51.6
	2	0.125	6.42	51.4	
4 x 4 x 0.250 (chord)	1	0.120	5.76	48.0	46.5
	2	0.120	5.40	45.0	
4 x 4 x 0.188 (chord)	1	0.093	4.39	47.5	48.1
	2	0.093	4.49	48.6	



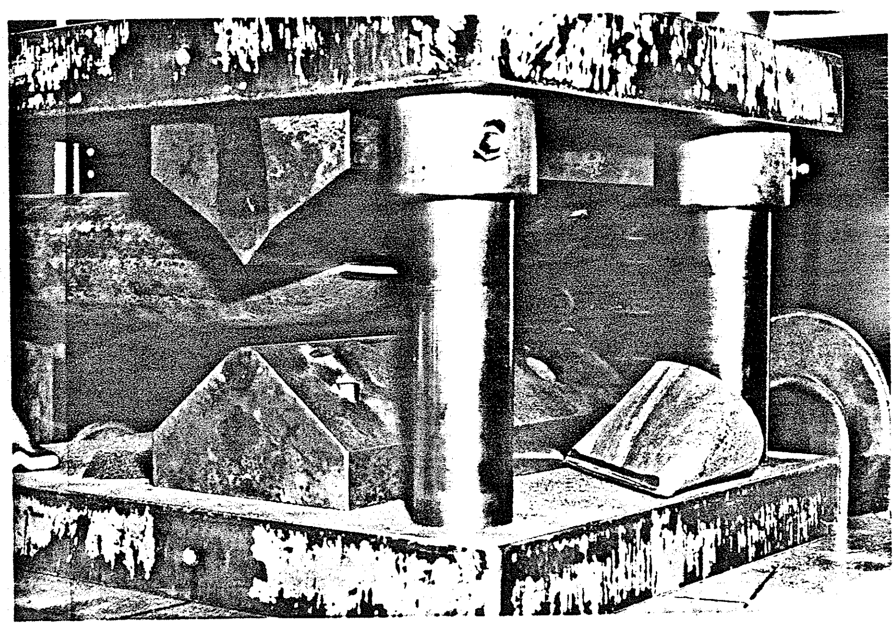


Fig. 3.3 Cropping of Web Members

hack saw. This method also avoided the formation of longitudinal cracks at the welded seam of the H.S.S. Cracks are usually produced at the seam when hot formed H.S.S. with thick walls are completely flattened. The cropped surfaces were ground symmetrical for proper fitting and welding on both sides of the webs.

The truss was tack welded in a jig, as shown in Figure 3.4. To simulate typical shop practice, all final welding was performed in the University of Manitoba structures laboratory by the same qualified welder.

3.5 Truss Supports

A detail of one of the truss supports is shown in Figure 3.5(a) and (b). The support consisted of a W 6 x 6 x 15.5 section welded to a 3/4 in. thick plate, which was bolted to a 1½ in. thick plate by eight A-325 ½ in. diameter bolts. The total assembly was then bolted to the floor by four high strength bolts. The truss was connected to the W-section by two 6 x 3/4 x 24 long plates using eight 3/4 in. diameter bolts. Between the plate and the truss a 3 in. x 1 in. x 10 in. long plate was welded on either side of the truss.

Both the bottom and top chords of the truss were supported laterally at the supports, near midspan and at two other locations, to prevent possible out-of-plane chord buckling. Photographs of the lateral support assemblies are shown in Figure 3.6. The lateral support for the bottom chord was provided by 2 x 2 x ½ angles bolted at one end to the chord and at the other to a heavy W-section at the same height as the lower chord. For the upper chord the lateral support angles were bolted to stationary steel columns. This enabled the truss chords to deflect in

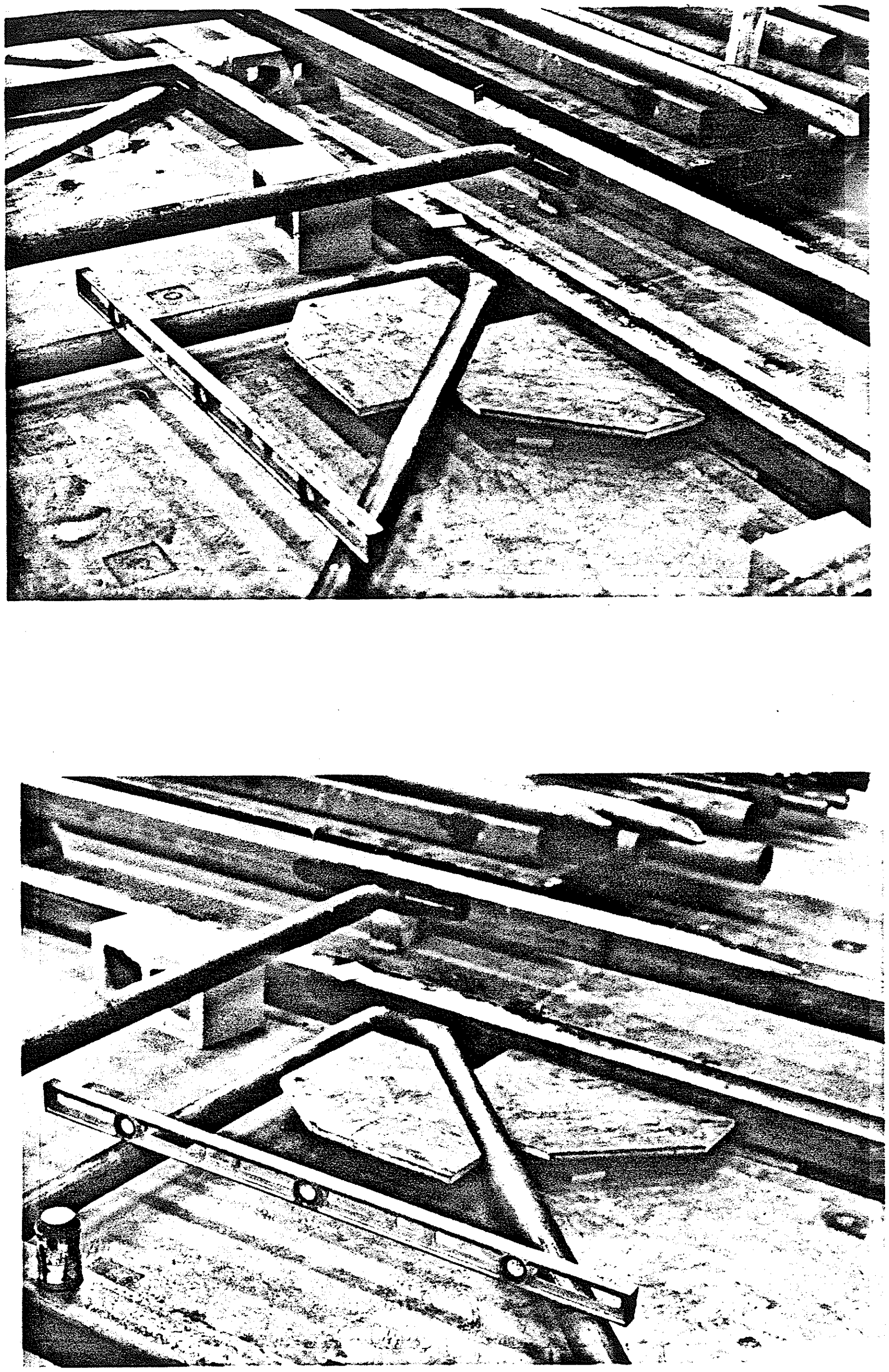


Fig. 3.4 Fabrication of the Truss

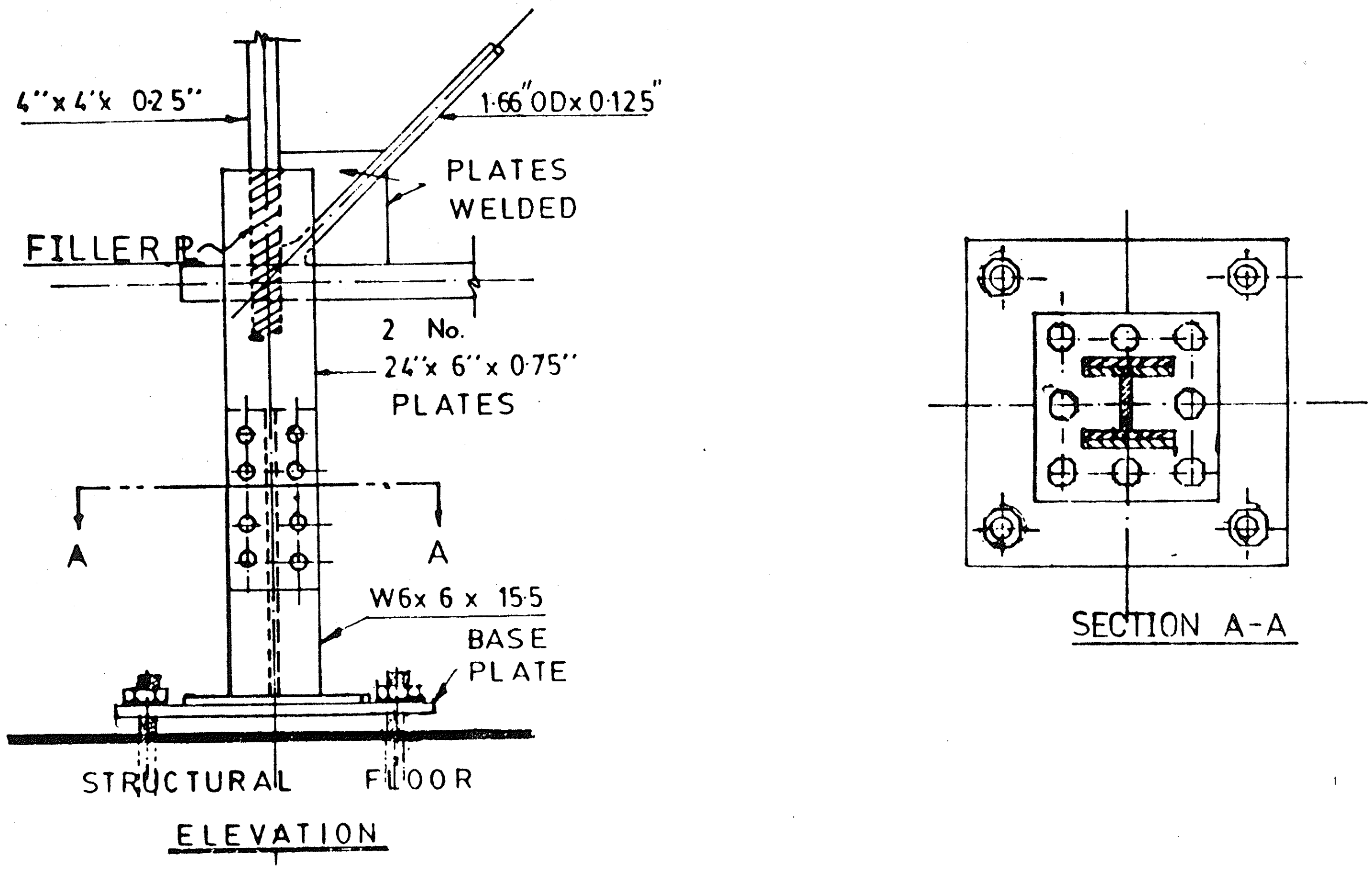


FIG. 3.5(a) TRUSS SUPPORT

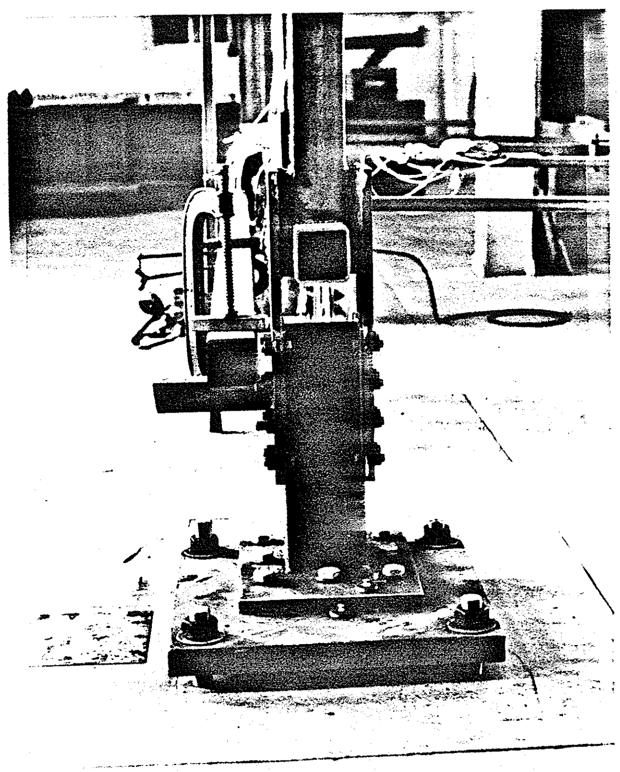
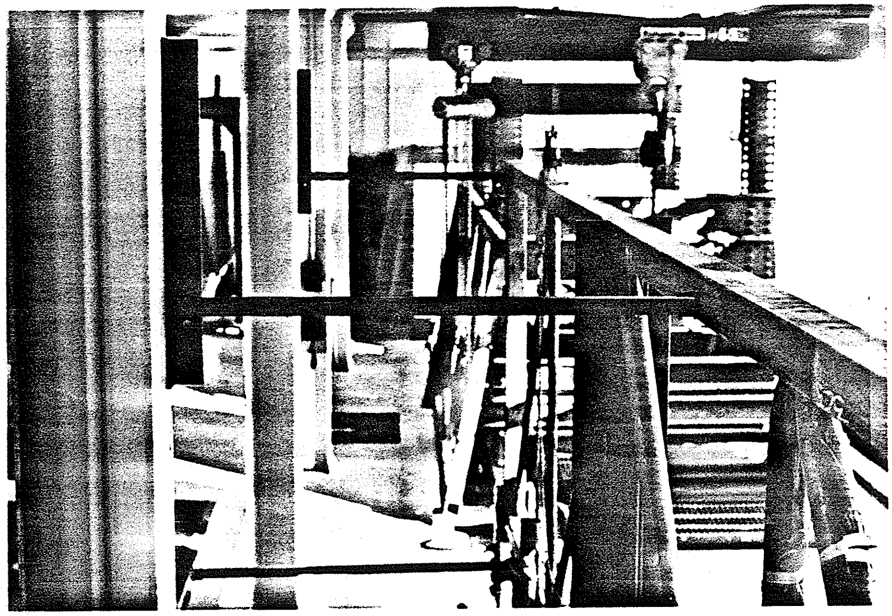
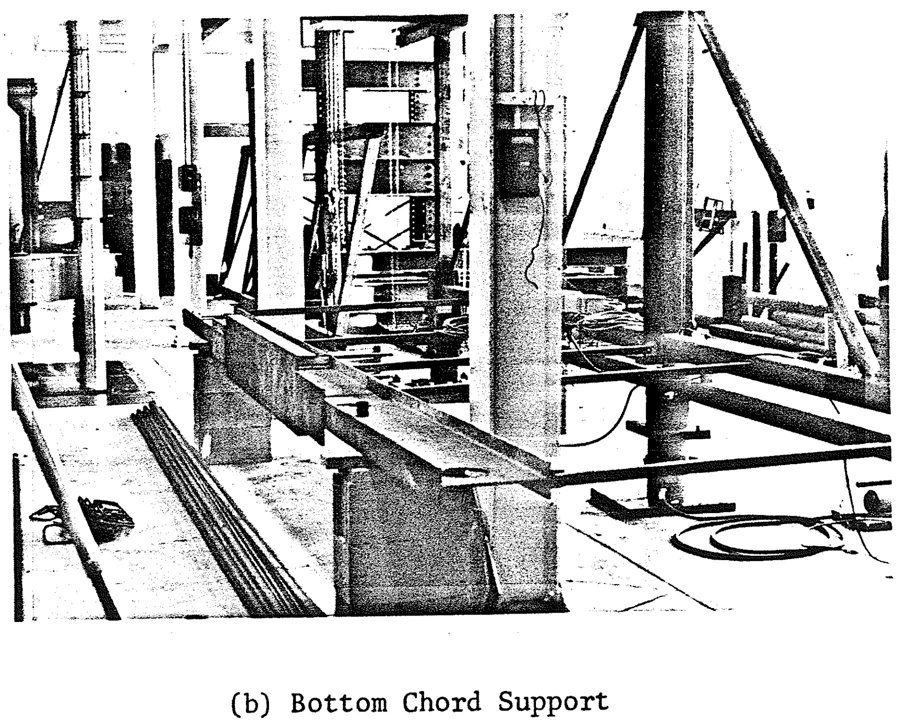


FIG. 3.5(b) TRUSS SUPPORT



(a) Top Chord Support



(b) Bottom Chord Support

FIG. 3.6 LATERAL SUPPORT ASSEMBLY

the vertical plane, but not the horizontal one.

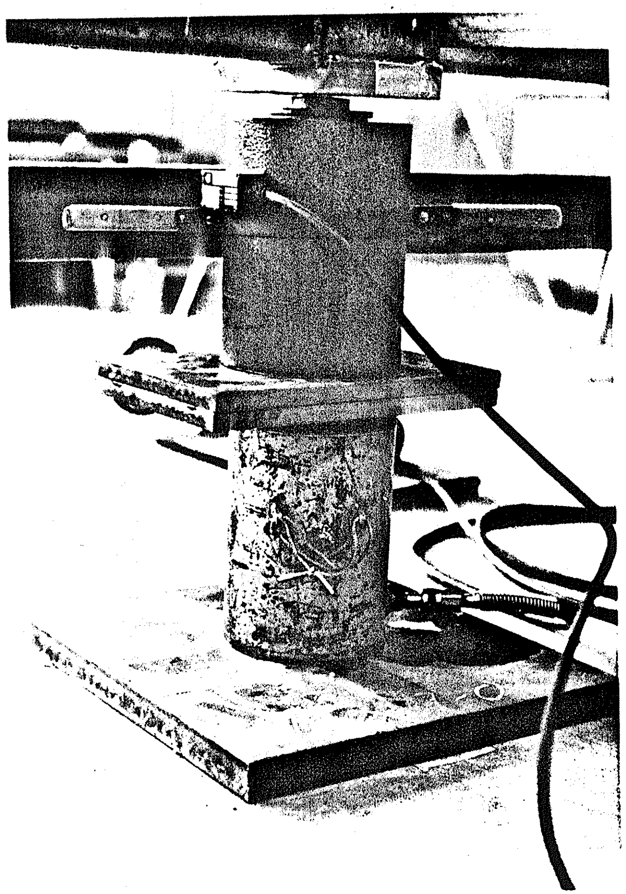
3.6 Loading Arrangements

The truss was loaded at mid-span through a 200 kip hollow core load cell, by a 200 kip capacity jack connected to a 10,000 psi Enerpac Pressure Gauge (Model 30-102). The loading device is shown in Figure 3.7. 3.7. The load cell was calibrated against a 600,000 lb Universal Testing Machine.

3.7 Measurement of Displacements

Two types of displacement measurement were used. One involved the measurement of local deformation of the connected chord wall, as this deformation was very significant. Secondly, the mid-span deflection of the truss and the transverse displacements of the compression webs were measured. The latter measurements were used to detect out-of-plane buckling of the compression web.

In order to measure the deformation of the connected face of the chord, six displacement transducers were mounted on the back of the chord face and connected to wires which passed through the centre of the chord and were in contact with the inside surface of the loaded chord face, as shown in Figures 3.8 and 3.9. The relative positions of the transducers, as illustrated in Figure 3.10, were chosen on the basis of several criteria. Firstly, it was observed by previous investigators that maximum chord face displacements occurred at the extreme edges of the web members, on the longitudinal centre line of the chord face. Hence, as these deflections are of importance, a transducer was located at each edge. Secondly,



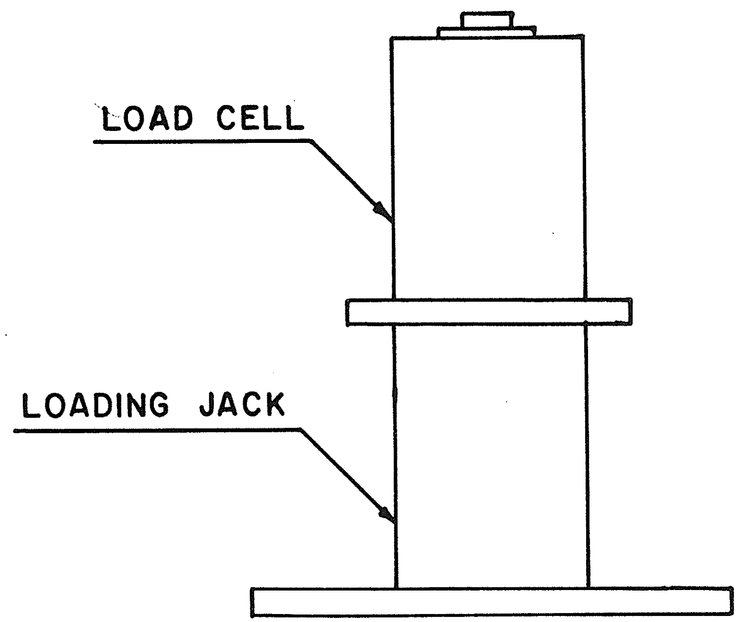
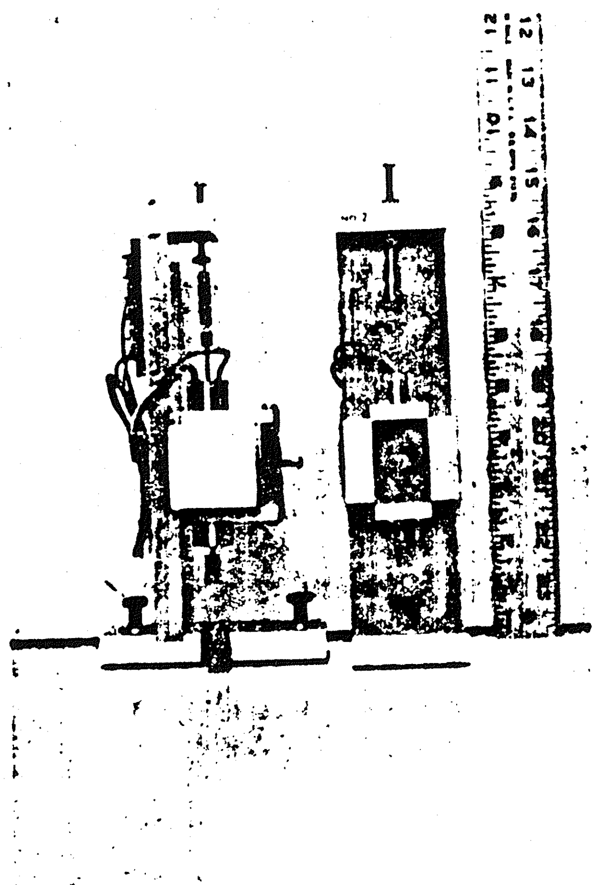
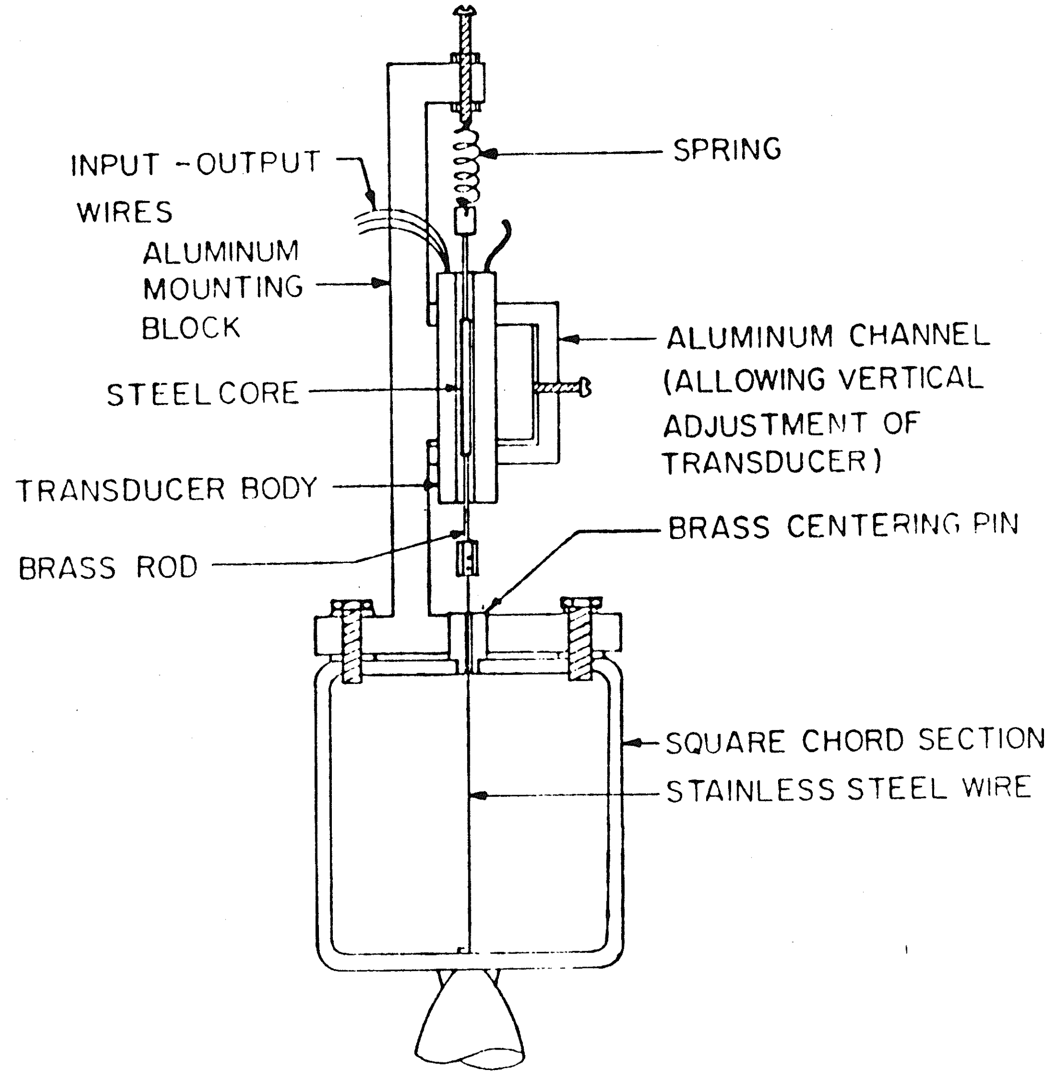


FIG. 3.7 LOADING DEVICE

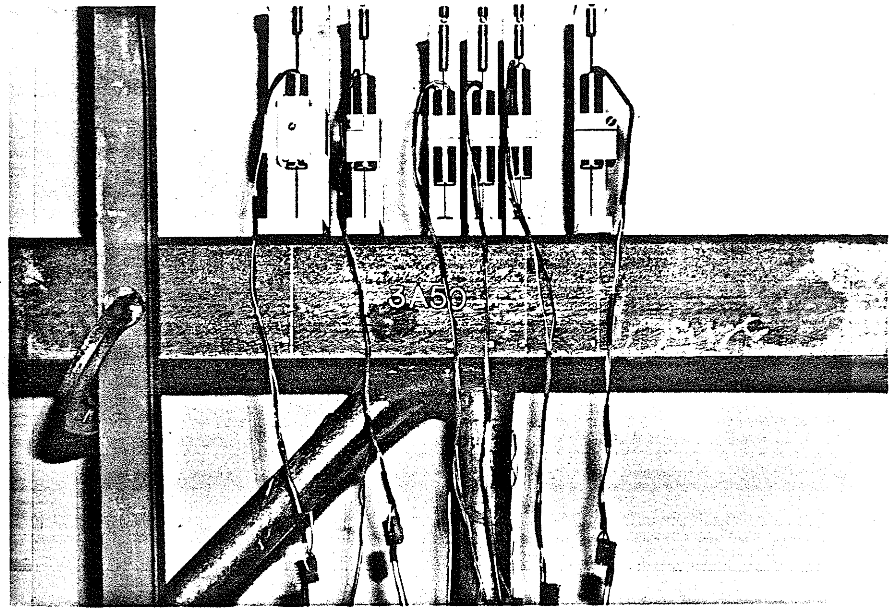


(a) Displacement Transducers and Mounting Blocks



(b) Transducer Assembly

FIG. 3.8



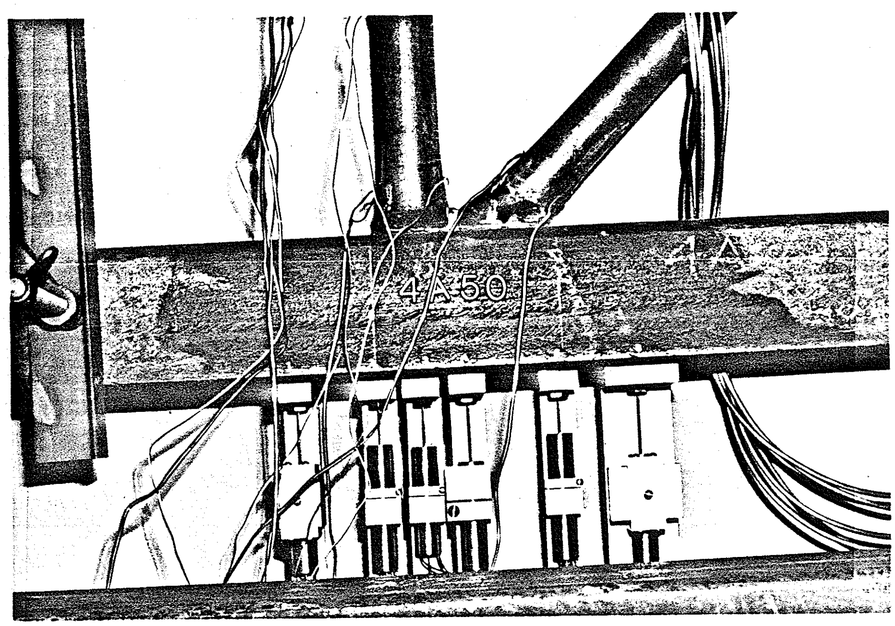


FIG. 3.9 TRANSDUCERS IN POSITION

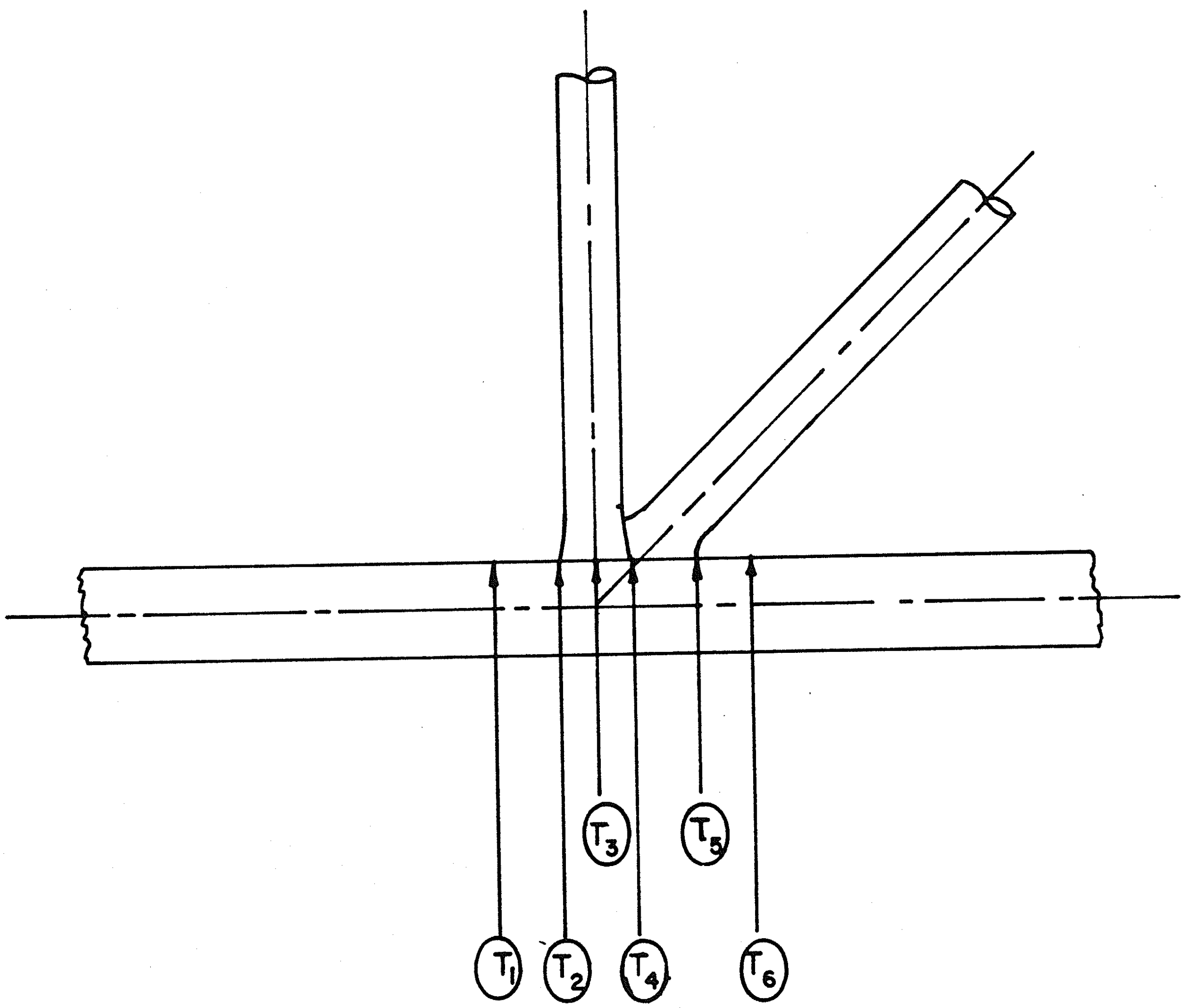


FIG. 3.10 TRANSDUCER LOCATION

it was of importance to measure the deflection of the chord face at the intersection of the web members. Thus a third transducer was placed at this position. Thirdly, to obtain average displacements of the compression web members into the chord, a transducer was located where the centre line of the compression web intersected the chord. Lastly, the remaining two transducers were positioned immediately to the left and right of the incident web members to complete the collection of data and permit accurate plots of the deformation of the loaded chord face at various lead levels. The transducers were Hewlett-Packard Series TDCDT, with a stroke of ± 0.250 inches. Each transducer essentially consisted of a ferrite core (style number B12) surrounded by a cylindrical coil. 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 compression spring, and the bottom rod to a stainless steel wire. The other end of the wire was connected to the chord wall at the location where deformation was to be measured.

The cement used for connecting the wire to the chord wall, was strain gauge cement CC-15A KYOWA.

The transducer assembly was calibrated against a depth micrometer with an accuracy of 0.0001 inch. The displacement-voltage curves obtained were linear, as expected. For a stainless steel wire of 5¼ in. length as used for the 4 x 4 chord section, the sensitivity of the transducer ranged from 4.0 to 4.1 volt/inch, the average value of 4 volt/inch was used for conversion.

To measure the out-of-plane displacement of the midpoint of the compression web and to measure the centre point deflection of the truss, two dial gauges were used. Each gauge measured to an accuracy of 0.001 inch. The positions of the dial gauges are shown in Figure 3.11.

3.8 Measurement of Strain

Electrical resistance strain gauges were used for measuring strain distribution in the vicinity of the joint in the web members and at the mid-point of the compression web. The strain gauges were Micro-Measurement precision strain gauges, type EA-06-250BG-120, with a gauge length of ¼ in. gauge resistance of (120.0 ± 0.3%) ohms, and gauge factor of 2.06 ± 0.5%. The input voltage was 4 volts. Twenty-four strain gauges were used on each truss, four at each joint and four at the mid-point of each compression web.

The gauge locations are shown in Figure 3.11. Since it was found in earlier investigations that the strains at some distance from the joint were quite uniform and nominal [21], and that strains close to the joint were relatively large [1,6,10-13], the strain gauges were placed close to the weld in the plane of the truss and those at the mid-point of the compression member were placed two in the plane of the truss and two out of the plane.

3.9 Data Acquisition

A Hewlett-Packard Data Acquisition System, consisting of an HP9825A calculator with HP9871A Printer and Scanner 3495A, was used to acquire and record on paper and magnetic tape the output voltages from the transducers and strain gauges. A program generated in the Civil Engineering Department of the University of Manitoba was used to convert the voltage readings

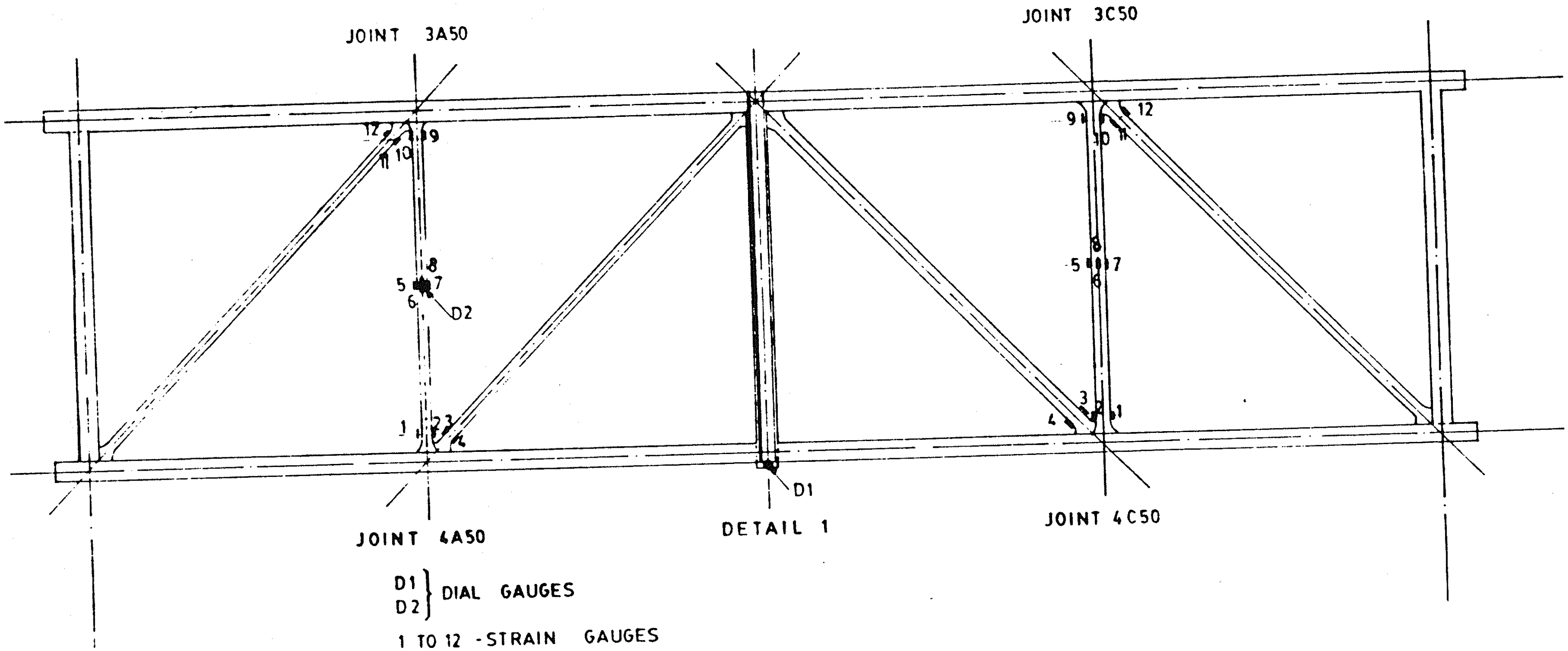


Fig. 3.11 Location of Dial Gauges and Strain Gauges

from the tape to displacement and strain values and to print them. Figure 3.12 shows the data acquisition system.

3.10 Testing and Reinforcement Sequences

The testing program consisted of a series of sequential tests on each of the two identical trusses, failing each of the four test joints in turn. Every time a joint failed it was suitably reinforced and the truss was reloaded for the next test.

A two part identification system was used to identify each of the tests. The first two characters, T1 or T2, indicated the truss number. The last character, 1,2,3 or 4, designated the specific joint under test, according to the following schedule:

1 = 3A50

2 = 4A50

3 = 3C50

4 = 4C50

In each truss, the compression web connecting joints 3A50 and 4A50 failed prematurely; a retest was done on joint 3A50 after suitable reinforcement of the web. In these cases, the designations T1.1A and T2.1A were used to differentiate between the initial test and the retest.

The sequence of testing and reinforcement was the following:

TRUSS - 1

TEST T1.1

The loading was controlled to fail joint 3A50 first, because this was the weakest of the four test joints. As the loading began, the compression member connecting joints 3A50 and 4A50 began to bend into an S-

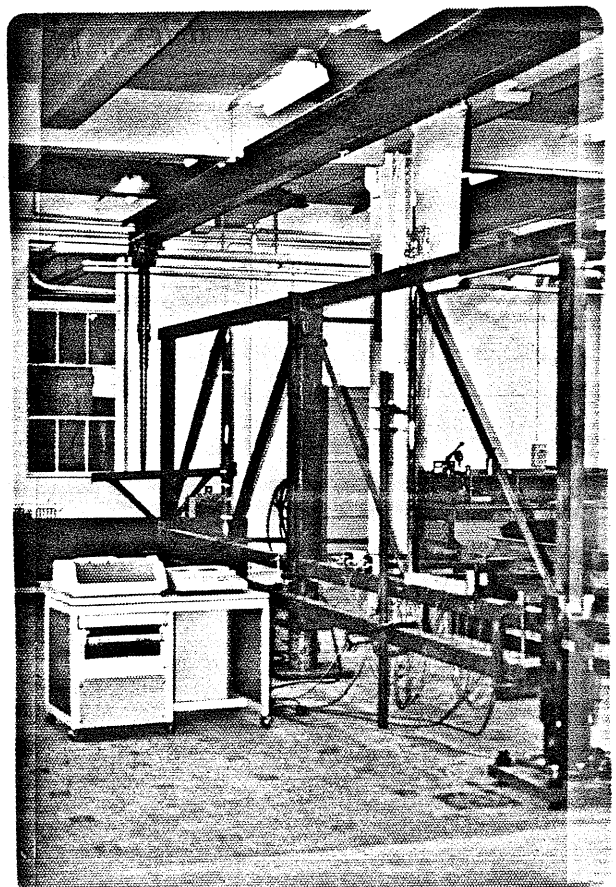


FIG. 3.12 DATA ACQUISITION SYSTEM

shape in the plane of the truss, with a point of inflection near the mid-point of the member. After considerable deformation of the two test joints had occurred, the compression web member suddenly buckled out of the plane of the truss, thus producing an abrupt drop-off in the truss loading. During this entire testing period the deformation of joints 3A50 and 4A50 and any out-of-plane bending of compression member were measured. The buckled compression member is shown in Figure 3.13.

TEST T1.1A

The compression member was next straightened and reinforced by welding two 34 in. x 2 in. x ¼ in. plates along the middle part of the compression web in a plane normal to that of the truss. The reinforced compression member is shown in Figure 3.14. The loading was then repeated and the reinforced compression member again began to bend into an S-shape, in the plane of the truss. However, again it eventually buckled out of the plane of the truss, with a sharp kink forming at the end of the reinforcing plates closest to joint 3A50, as shown in Figure 3.15. The deformations of joints 3A50 and 4A50 were again measured during the entire test period.

TEST T1.2

The buckled compression member was again straightened and further reinforced with two plates of dimension 51 in. x 2 in. x ¼ in. used in the plane of the truss and another two plates, 17 in. x 2 in. x ¼ in., used in the out-of-plane direction. All plates were welded to the compression member and extended up to the top chord of the truss, in order to reinforce joint 3A50. The purpose was to reinforce joint 3A50 while attempting to measure the deformation of joint 4A50 during subsequent loading. The reinforced

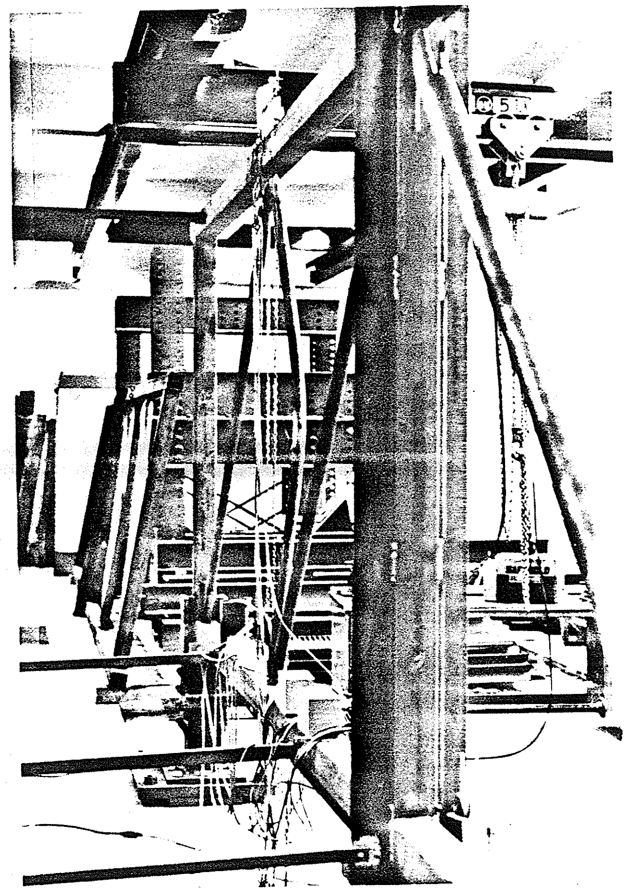


FIG. 3.13 BUCKLING OF COMPRESSION MEMBER

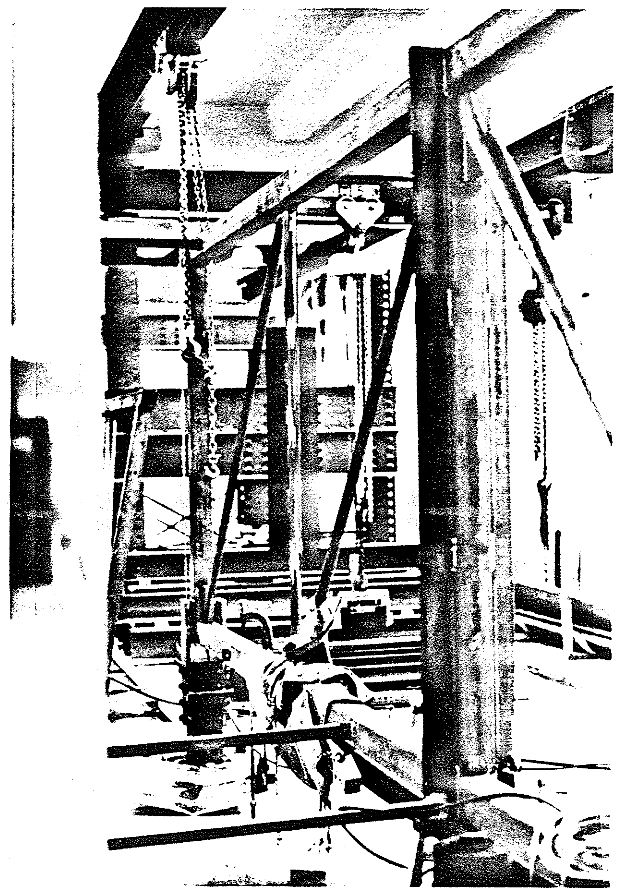


Fig. 3.14 Reinforced Compression Web

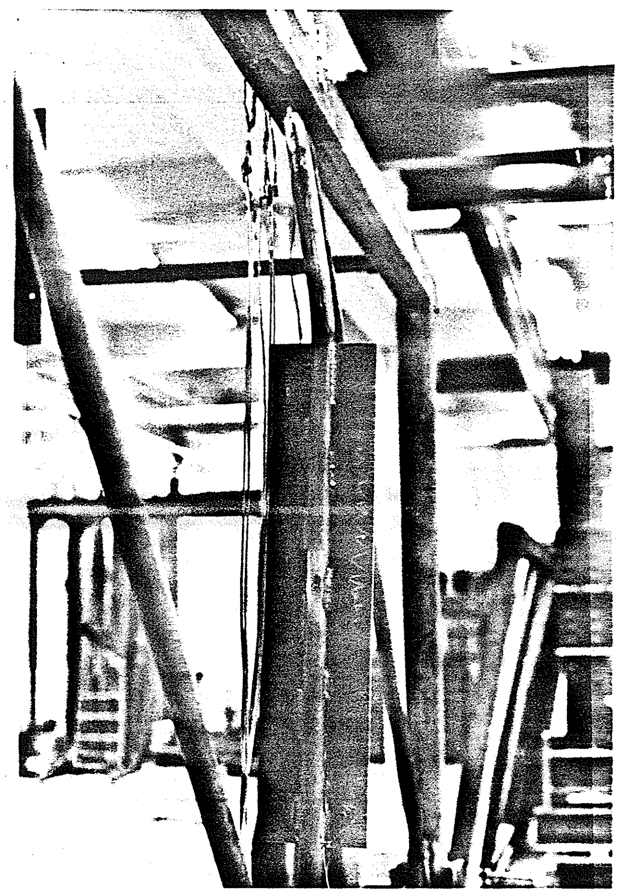


FIG. 3.15 BUCKLING OF REINFORCED COMPRESSION WEB

compression web is shown in Figure 3.16. As the loading was continued in this test, due to over reinforcement of the compression web, it did not rotate at the joint and loading was stopped at a level of one and a half times that at which joint 4A50 failed in the isolated joint test.

TEST T1.3

After test T1.2 had been completed the half of the truss containing joints 3A50 and 4A50 was reinforced, the compression web by two 3½ in. x 3½ in. x ½ in. angles and the tension members each by one angle of the same dimensions. The reinforced truss is shown in Figure 3.17.

The truss was then loaded until joint 3C50 failed. In this test also, the compression member began to bend into an S-shape in the plane of the truss. This bending continued as the loading was increased, until joint 3C50 failed by excessive deformation of the chord face. This subsequently caused the tension member to tear off near the weld at the joint. Failed joint 3C50 is shown in Figure 3.18.

The deformation of both joints 3C50 and 4C50 was measured during the entire period of loading. Since, after the failure of joint 3C50, the compression member was greatly deformed and since adequate reinforcement of joint 3C50 would have been difficult, no further testing of Truss-1 was performed.

TRUSS - 2

TEST T2.1

The truss was loaded in a manner similar to that for Truss-1. As the load was applied, the compression web began to bend into an S-shape in the plane of the truss. Again, after substantial deformation had occurred, the compression web connecting joints 3A50 and 4A50 abruptly

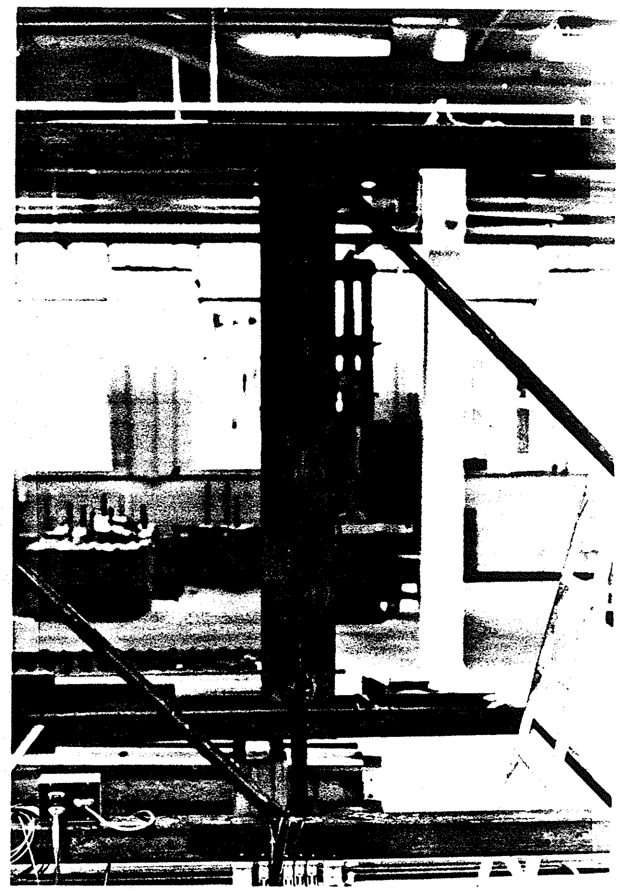


Fig. 3.16 Compression Web Reinforced in Both Planes

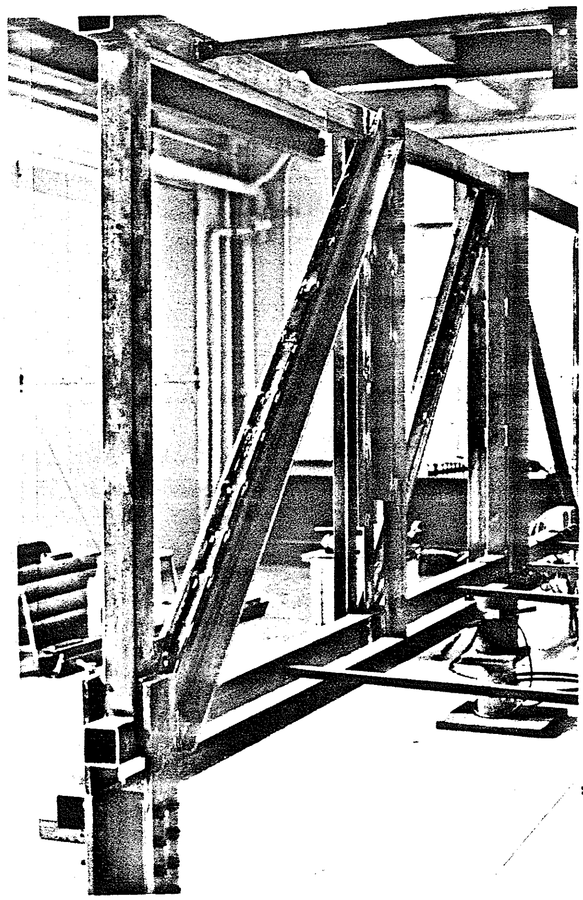


Fig. 3.17 Complete Reinforcement of Joints 3A50 and 4A50

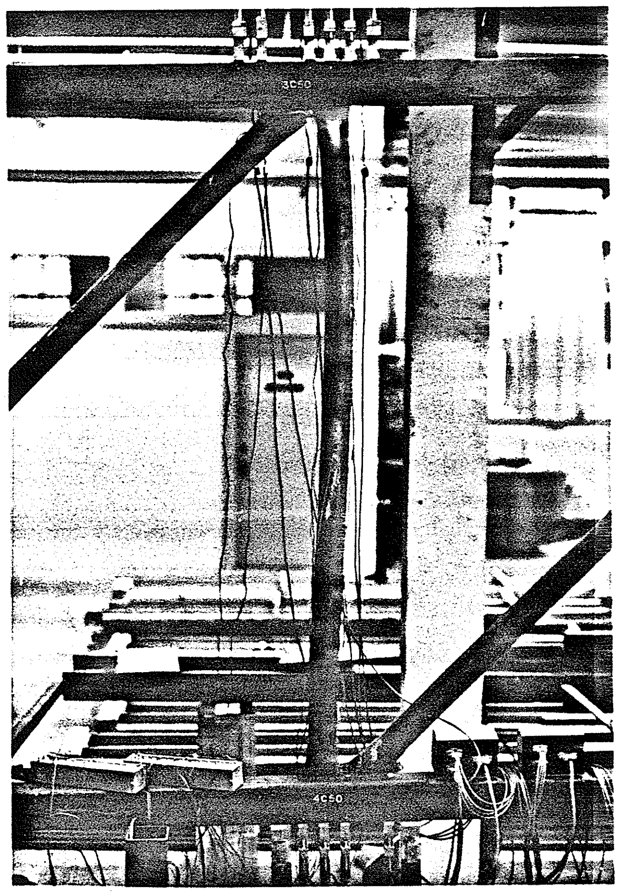


Fig. 3.18 Joint 3C50 After Failure

buckled out-of-plane. This led to an immediate reduction of truss loading.

TEST 2.1A

The compression web was straightened and then reinforced by two 44 in. x 2 in. x ¼ in. plates in the plane normal to the truss, to prevent a recurrence of out-of-plane buckling of the compression web as had occurred in test T1.1A.

The reinforced compression member is shown in Figure 3.19. The loading was then re-applied and the compression web again began bending into an S-shape in the plane of the truss. Eventually joint 3A50 failed by excessive deformation of the chord face and tearing of the tension web near the weld at the joint. The buckled shape of the compression member is shown in Figure 3.20.

TEST T2.2

Subsequent to test T2.1A, the compression member was again straightened and joint 3A50 was reinforced with ¼ in. thick gusset plates, as shown in Figure 3.21. The joint was reinforced so that loading could be continued, to study the behaviour of joint 4A50, which did not show significant deformation.

The loading was again continued until the chord face at joint 4A50 had experienced large deformation and the loading ram moved substantially with a decrease in the recorded load.

TEST T2.3

The half of the truss containing joints 3A50 and 4A50 was then completely reinforced as was done for Truss-1.

The loading was again begun and was continued until joint 3C50

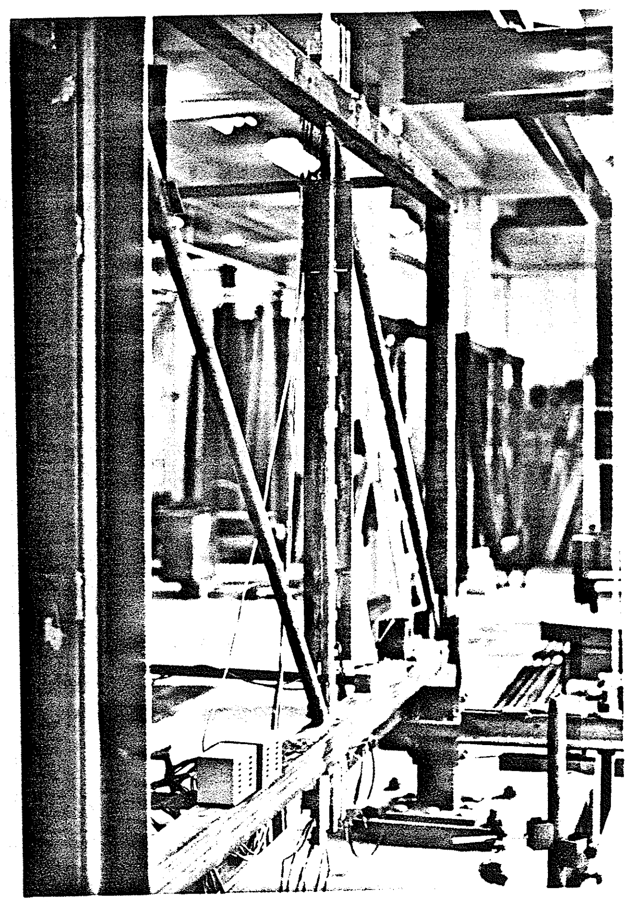


FIG. 3.19 REINFORCED COMPRESSION MEMBER

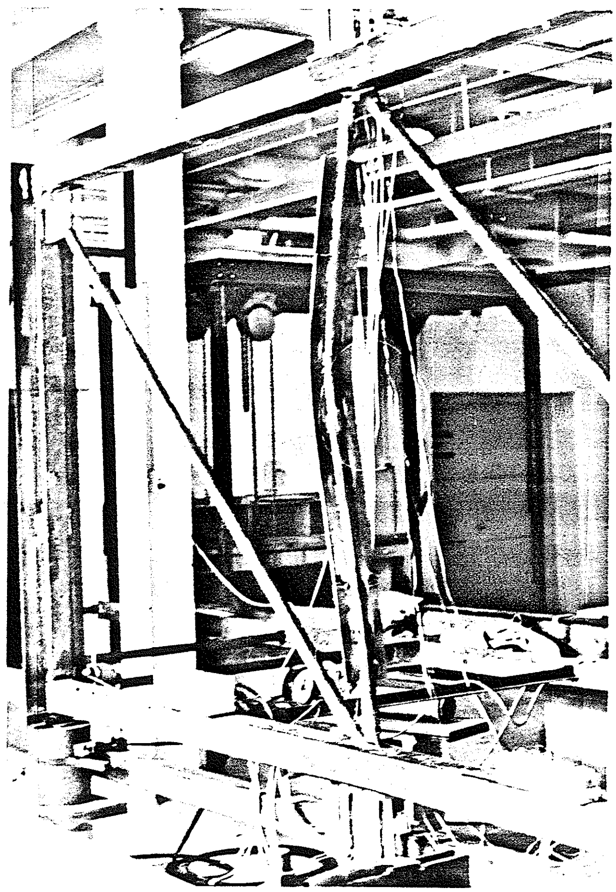
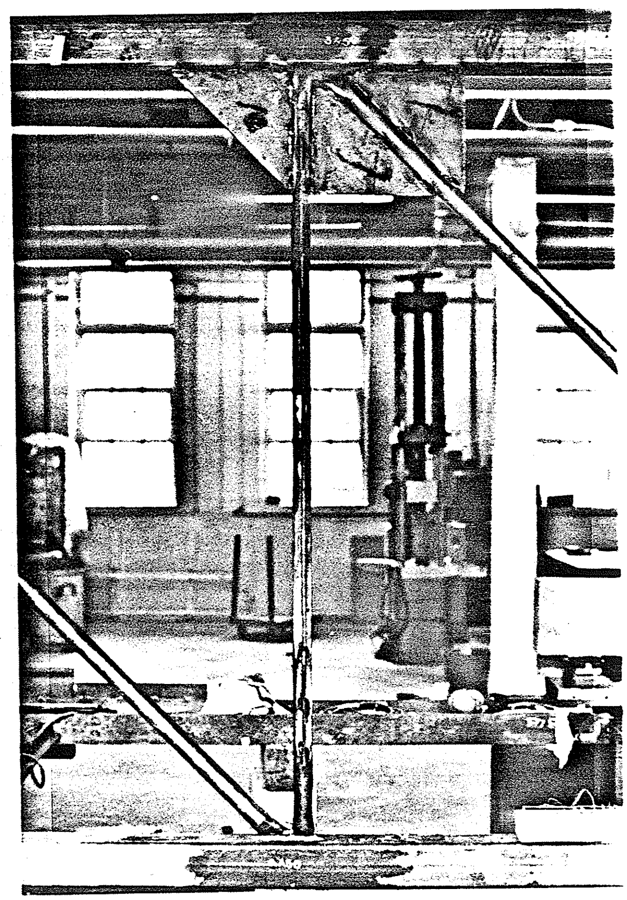


FIG. 3.20 FAILURE OF REINFORCED COMPRESSION WEB

(a)



(b)

FIG. 3.21 JOINT 3A50 REINFORCED

failed, as it did in Truss-1, by excessive deformation of chord face, which eventually led to tearing of tension web near the weld at the joint. The joint deformation was measured throughout the entire range of loading. Although joint 3C50 failed in a manner similar to the corresponding joint in Truss-1, this time the compression member connecting joints 3C50 and 4C50 did not bend as much and joint 4C50 was not visibly deformed.

TEST T2.4

Joint 3C50 was next reinforced by ¼ in. gusset plates and 2 in. x 2 in. x ¼ in. x 20 in. long angles, as shown in Figure 3.22(b).

The loading was then resumed until the weld of the 3½ in. x 3½ in. x 3/4 in. angle reinforcing the compression member connecting joints 3A50 and 4A50 failed. At this point, the loading was stopped, the weld was repaired and the loading was continued. The loading was continued until the weld of the angle reinforcing the tension member at joint 3C50 failed. At this load, the bottom chord of the truss had deflected more than 2 in. from its original position, as illustrated in Figure 3.23. As large deformation of the chord face had occurred at joint 4C50, it was decided to terminate the loading at this point.

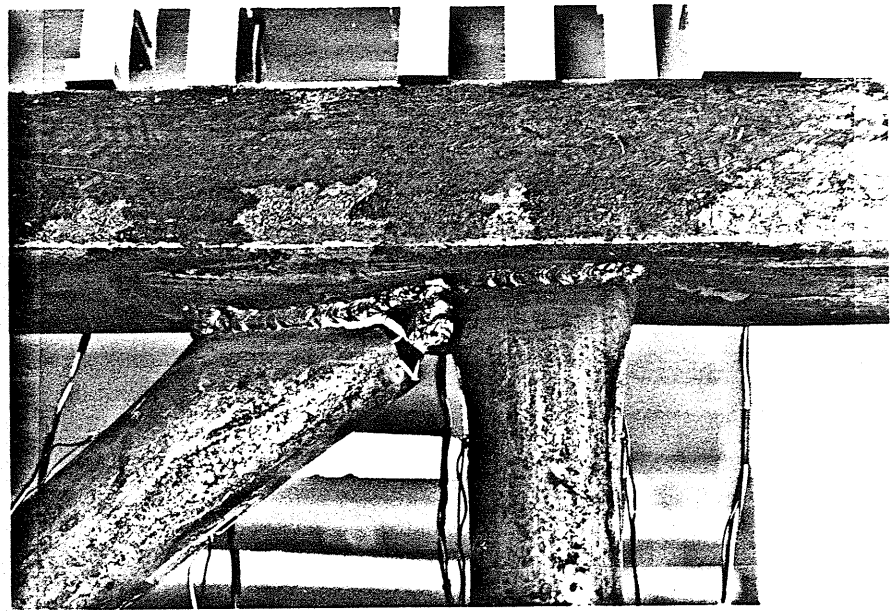


FIG. 3.22(a) JOINT 3C50 AFTER FAILURE

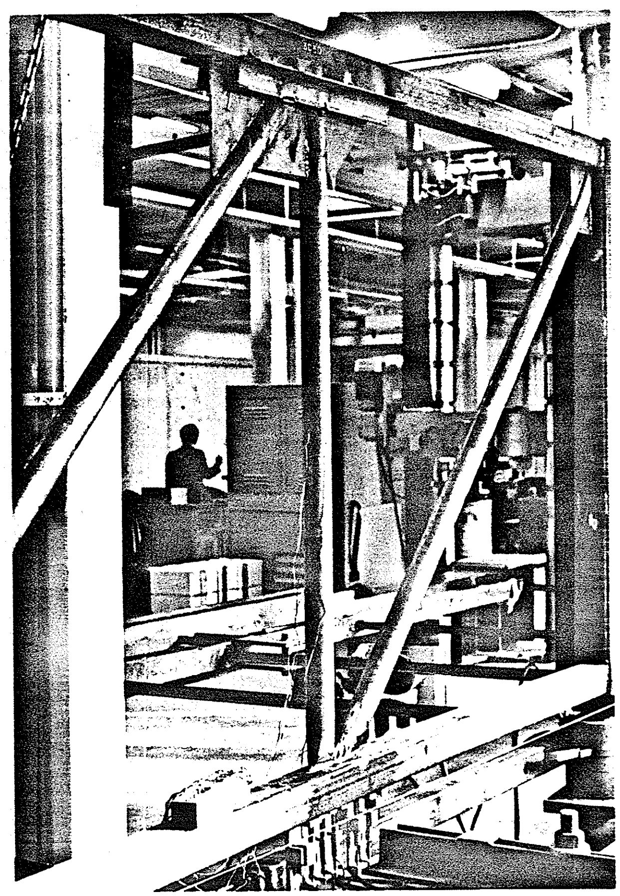


FIG. 3.22(b) JOINT 3C50 REINFORCED

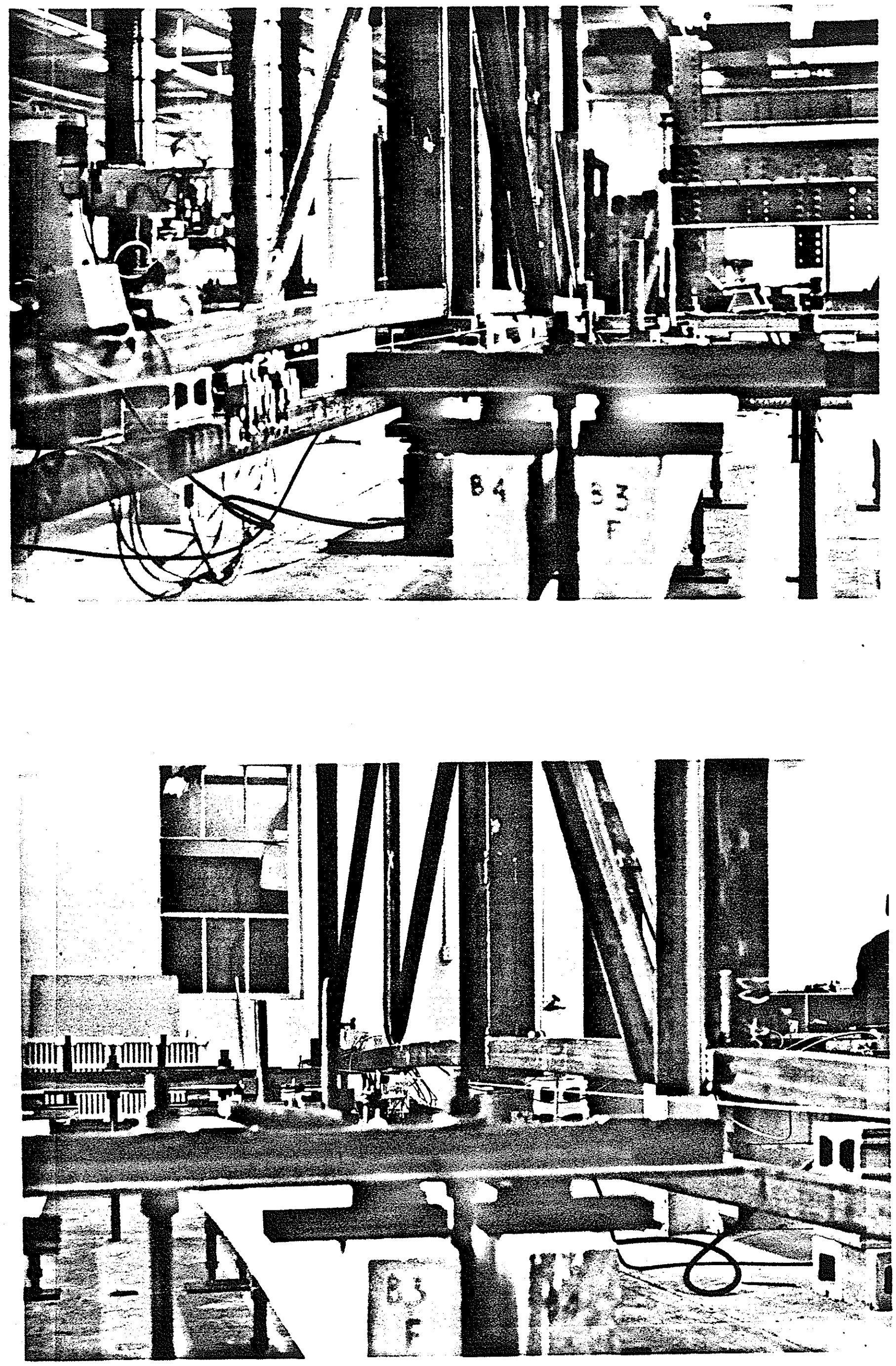


FIG. 3.23 DEFLECTED SHAPE OF THE CHORD

CHAPTER IV

TEST RESULTS AND DISCUSSION

In this chapter, the test results are presented and discussed, considering in turn, load-deformation behaviour, joint strength, joint stiffness, failure modes, chord face deformation, effect of joint deformation on overall truss deflection and strain distribution.

4.1 Load-Deformation Behaviour

The plots of load applied to the compression web vs. its movement relative to the chord axis at the joint, for both truss joints and corresponding joints previously tested in isolation, are presented in Figure 4.1. For the truss joints, deformation was measured as the average of the readings of the three transducers located on the chord face under the compression web. For the isolated joints it was the average of the readings of two transducers located on the chord face at the two edges of the compression web.

The significant performance criteria: ultimate load, yield load and stiffness, will be discussed with reference to the load-deformation plots.

Ultimate loads, load factors and stiffnesses for the test joints in trusses 1 and 2 are presented in Tables 4.1 and 4.2 respectively. Table 4.3 gives a comparison of corresponding values for trusses 1 and 2. The tabulated ultimate load is that load at which the truss was incapable of sustaining a load increase, either due to failure of the web or fracture of the weld at the test joint. This was indicated by substantial movement of the loading ram without any increase in the recorded load.

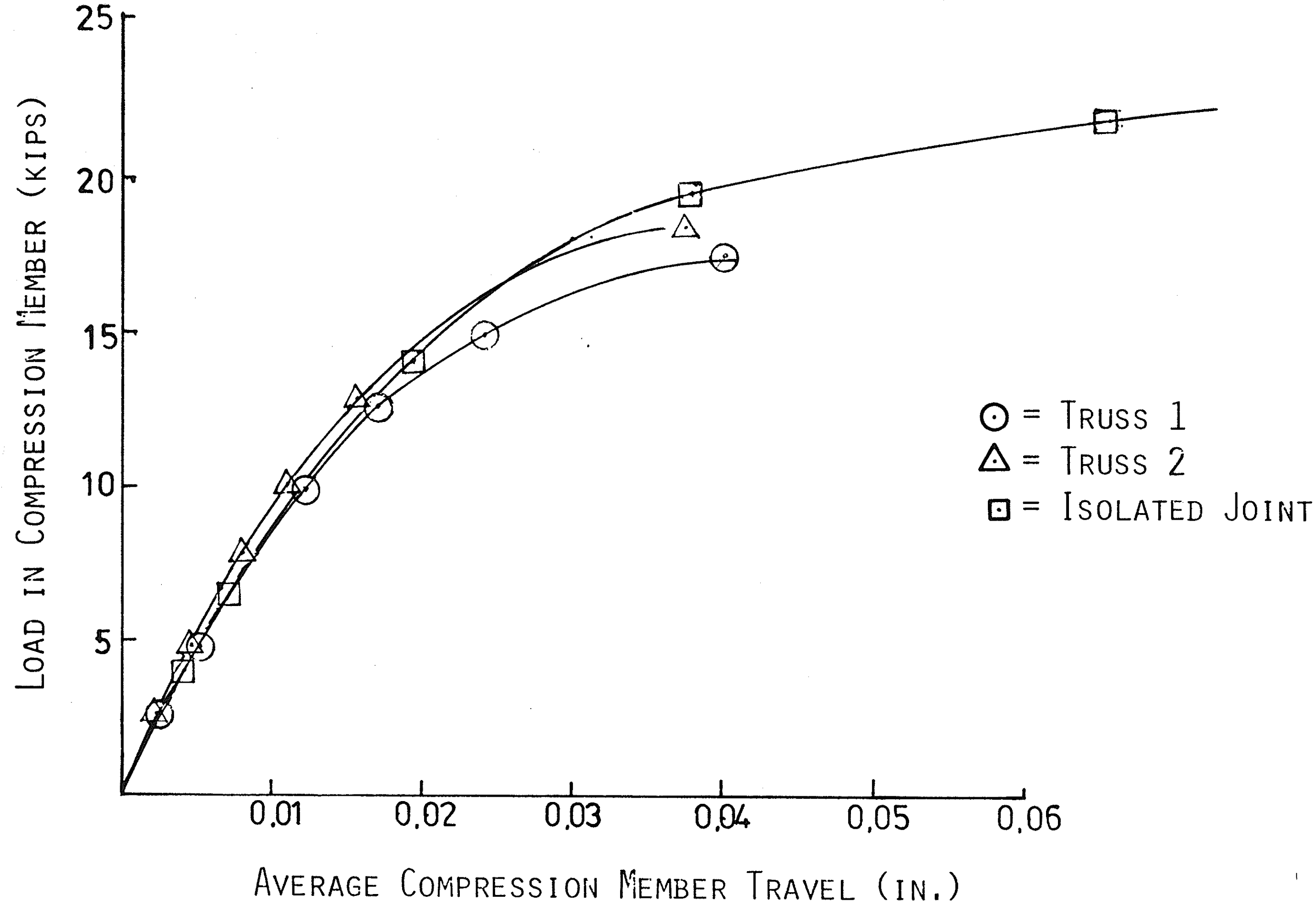


FIG. 4.1(a) LOAD-DEFORMATION JOINT 3A50

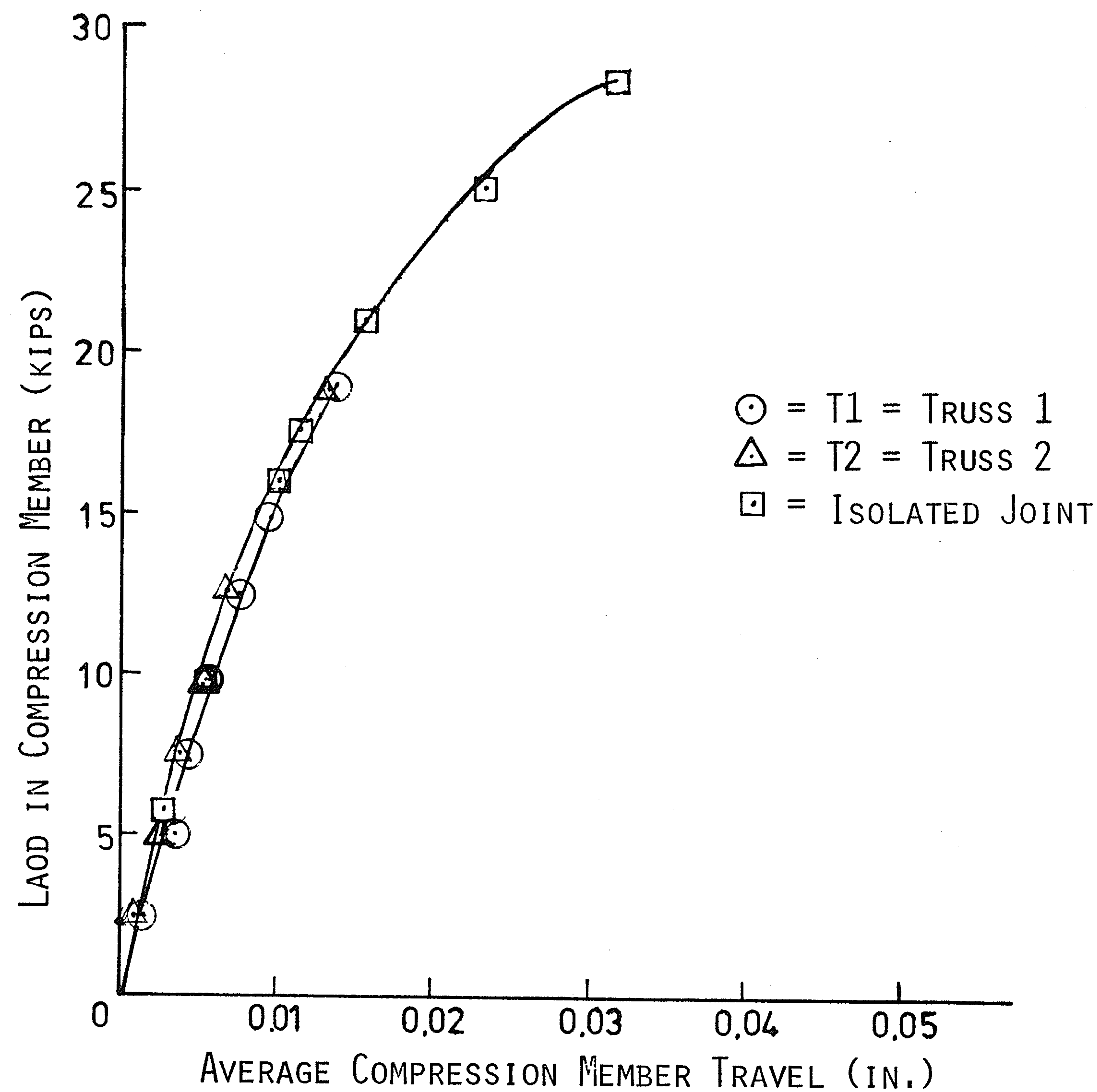


FIG. 4.1(b) LOAD-DEFORMATION JOINT 4A50

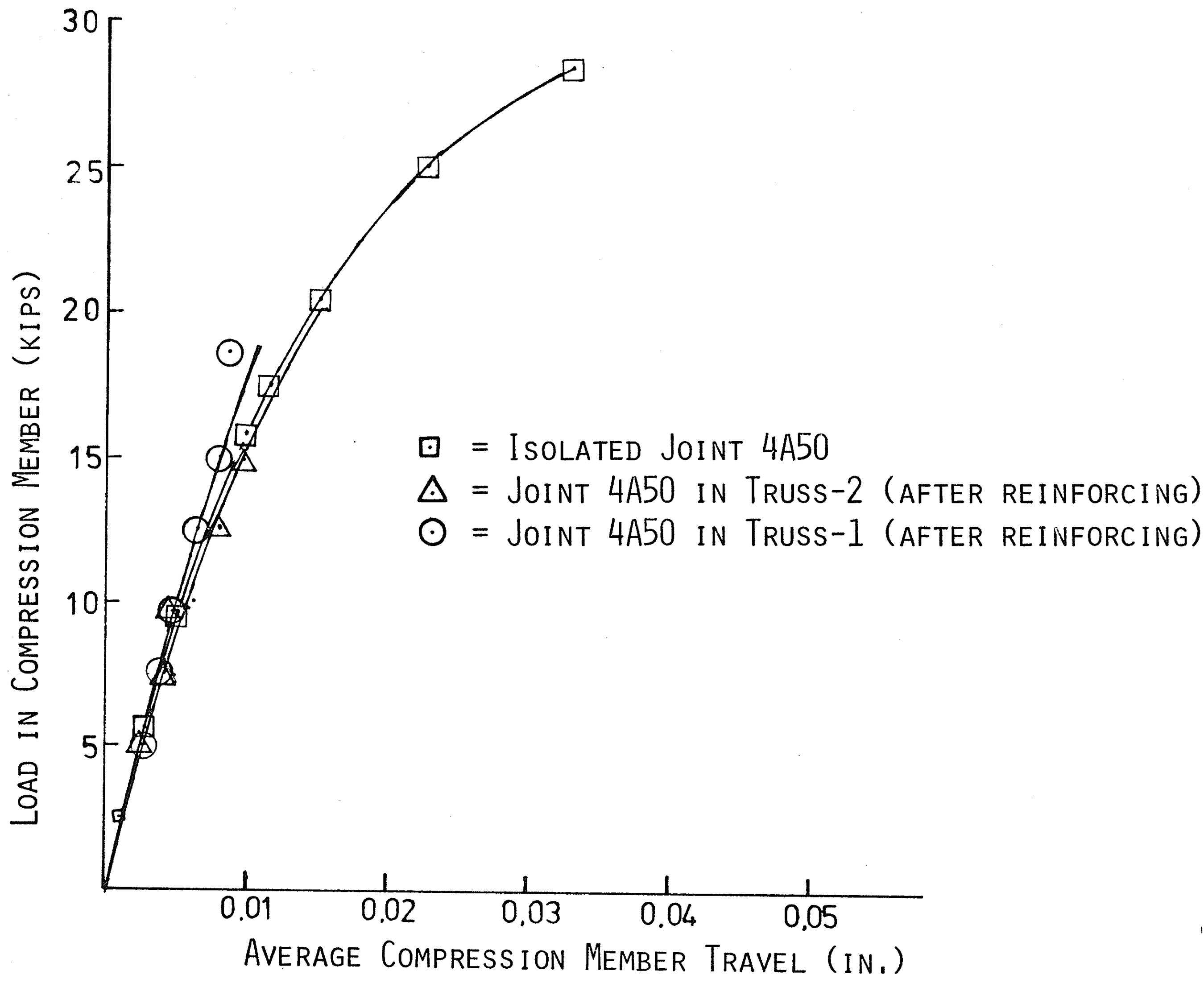


Fig. 4.1(c) Load Deformation Retest of Joint 4A50

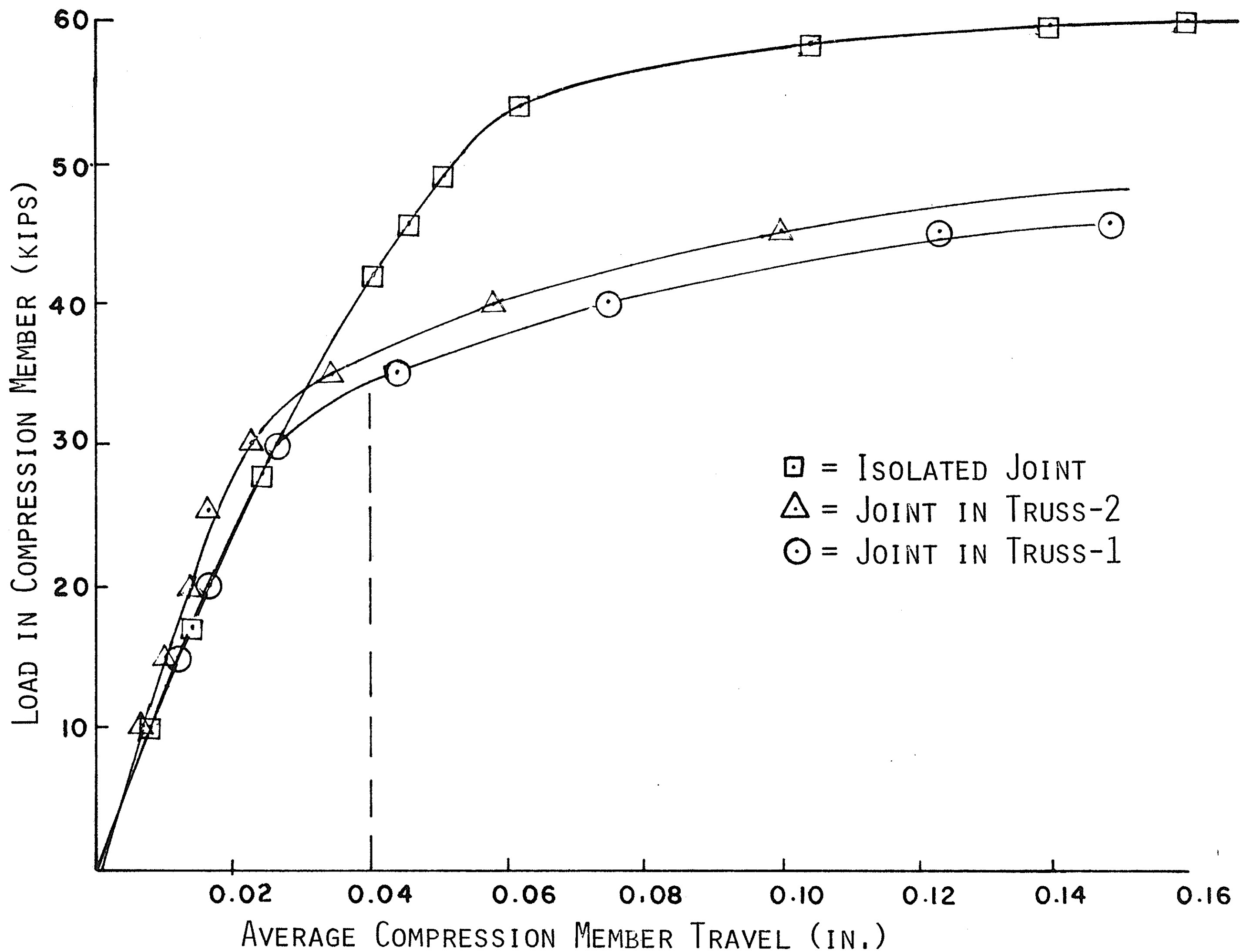


Fig. 4.1(d) Load-Deformation Joint 3C50

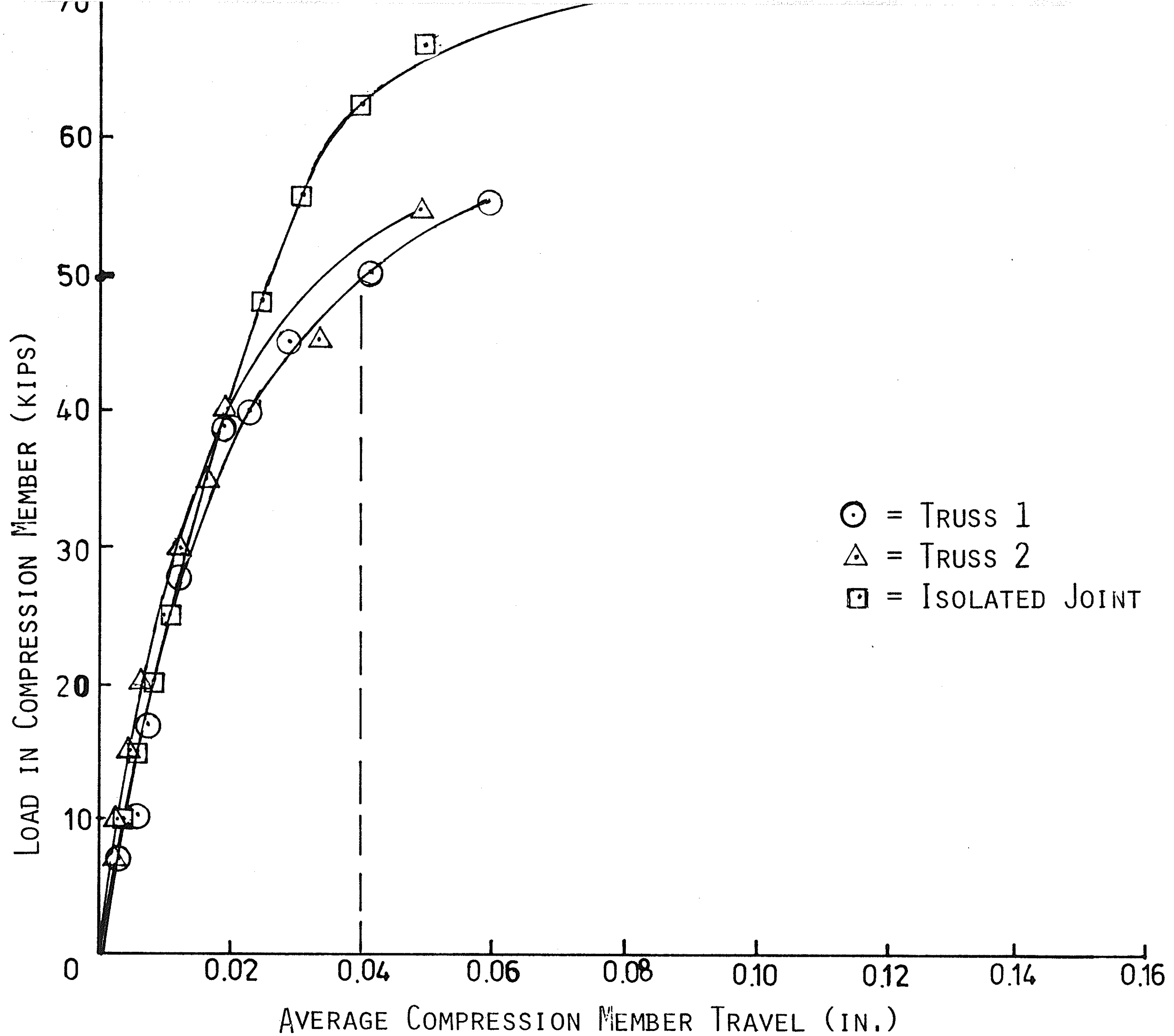


Fig. 4.1(e) Load-Deformation Joint 4C50

TEST 4.1 TEST RESULTS - TRUSS 1

Test No.	Test Joint	Chord $b_o x d_o x t_o$ (in.)	Web $d_1 \times t_1$ (in.)	Ultimate Load (Kips)	Compression Load Factor	Tension Factor	Translation Stiffness (Kip/in)	Failure Mode
	3A50	4x4x0.188	1.66 OD x 1.25				980	W,Z,X
T1.1				34.9	1.49	1.36		
	4A50	4x4x0.250	1.66 OD x 1.25				2000	W
	3A50	4x4x0.188	1.66 OD x 1.25					W,Z
T1.1A				36.95				
	4A50	4x4x0.250	1.66 OD x 1.25					X,Z
	3C50	4x4x0.188	2.875 OD x 0.250				1500	X,Y,Z
T1.3				111.34		1.28		
	4C50	4x4x0.250	2.875 OD x 0.250				2400	X,Z

Failure Modes

W = Buckling of compression web out of plane of truss.

X = Reverse curvature bending of compression web.

Y = Tearing of tension web.

Z = Large deformation of chord face.

TABLE 4.2 TEST RESULTS - TRUSS 2

Test No	Test Joint	Chord $b_o x d_o x t_o$ (in.)	Web $d_1 x\ t_1$ (in.)	Ultimate Load (Kips)	Compression Web Load Factor	Tension Web Factor	Translation Stiffness (Kip/in)	Failure Mode
T2-1	3A50	4x4x0.188	1.66 OD x 0.125	36.80	1.57	1.43	1000	W,X,Z
	4A50	4x4x0.250	1.66 OD x 0.125				2100	W
T2-1A	3A50	4x4x0.188	1.66 OD x 0.125	40.70				W,Y,Z
	4A50	4x4x0.250	1.66 OD x 0.125					W,Z
T2-2	4A50	4x4x0.250	1.66 OD x 0.125	56.68				Z
T2-3	3C50	4x4x0.188	2.875 OD x 0.250	104.52		1.20	1600	X,Y,Z
	4C50	4x4x0.250	2.875 OD x 0.250				2500	X,Z
T2-4	4C50	4x4x0.250	2.875 OD x 0.250					Z

Failure Modes

W = Buckling of compression web out of plane of truss.

X = Reverse curvature bending of compression web

Y = Tearing of tension web

Z = Large deformation of loaded chord face

TABLE 4.3 COMPARISON OF RESULTS, TRUSSES 1 AND 2

Test Joint	ULTIMATE LOAD			LOAD FACTOR						TRANSLATION STIFFNESS		
				Compression Web			Tension Web					
	Truss-1 (Kips)	Truss-2 (Kips)	% Difference	Truss-1	Truss-2	% Difference	Truss-1	Truss-2	% Difference	Truss-1 (Kip/in)	Truss-2 (Kip/in)	% Difference
3A50	34.95	36.8	5.7	1.49	1.57	5	1.36	1.43	4.9	980	1000	2.0
4A50										2000	2100	4.7
3C50	111.34	104.52	6.0	-	-	-	1.28	1.20	6.3	1500	1600	6.2
4C50				-	-	-				2400	2500	8.0

The load-factor is the ratio of the measured ultimate load to the design working load for the web. Load-factors for both the tension and compression webs are given because, while the tension web load factor had been used by other investigators, most of the failures of the joints tested in this study involved buckling or flexural yielding of the compression web. For tests 1.3 and 2.3, the compression web load factor is not tabulated because the joint failed before the load in the compression web reached the design value. The design load for the compression web was based on a web length of 68 in., with an effective length factor of 0.7.

The translational stiffness was calculated as the ratio of the force in the compression web to the displacement of its centre line, normal to the chord axis. The stiffnesses of the truss joints and those of the corresponding isolated joints are compared in Table 4.5.

Various modes of failures are also tabulated in Tables 4.1 and 4.2.

4.1.1 Joint Strength

From the load-deformation plots, it can be seen that the yield loads for corresponding joints 3A50, 3C50 and 4C50 for the two trusses are virtually identical. Here the yield load is defined as the web force normal to the chord axis when the load-deformation plot begins to deviate from the initial straight line.

The load deformation plots for joints 3C50 and 4C50, which were at opposite ends of the same relatively stocky compression web in the truss tests are given in Figures 4.1(d) and (e).

It can be seen that very pronounced yielding occurred at joint 3C50 in both trusses, while yielding was just beginning at joint 4C50, when failure occurred. This was to be expected, as the joints were identical, except that the chord for 4C50 was 0.250 inch thick, while that for 3C50 was only 0.188 inch thick.

From the load deformation plots for joints 3C50 and 4C50, it can also be seen that the joints in the trusses yielded at lower loads than did the corresponding isolated joints. Hlavacek[17] noted that because the axial stress in the chord, arising from the action of the truss as a whole, is additive to the local chord bending stress at the joint, the larger axial stresses in the chord, the smaller the strength of the chord wall. This phenomenon is particularly noticeable when the deformation patterns of the stronger joints, e.g. joints 3C50 and 4C50, are compared with those of weaker joint 3A50. Joints 3C50 and 4C50, which sustained much higher load than did joint 3A50 also had higher axial stress. Hence the chord wall at joints 3C50 and 4C50, in the test trusses, yielded at a much lower load than that which produced yielding in the corresponding isolated joints. Joint 3A50 failed at a relatively low load with very little axial stress in the chord. Hence the joint yielded at almost the same load level as that in the isolated joint tests.

In the case of joint 4A50 there was little apparent yielding. The reason was that the compression web at this joint buckled before much yielding of the joint took place. It is interesting to note that after test 1.1, when the buckled compression member was repaired, reinforced and reloaded to test the deformation behaviour of joint 4A50, the compression member again buckled out of the plane of the truss. The load-deformation plot after reinforcement, illustrated in Figure 4.1(c), shows that the load-deformation behaviour was very similar to that before reinforcement, illustrated in Figure 4.1(b). The load-deformation plot for isolated joint

4A50, which is shown superimposed on those for the corresponding joints in the trusses, in Figure 4.1(b), indicates some yielding. The isolated specimen sustained an ultimate load 20 percent higher than those which caused buckling of the compression web in the truss tests. In both the truss tests and the isolated joint tests, failure of joint 4A50 occurred by buckling of the compression web with little local deformation of the loaded chord face.

From Table 4.3 it can be seen that the load factors for corresponding joints in trusses 1 and 2 are within 6.0 percent of each other. Load-factors could not be determined for joints 4A50 and 4C50 because these joints had comparatively weaker joints (3A50 and 3C50) at the far ends of the compression webs. The failure of the latter joints precluded the loading of the joints 4A50 and 4C50 to their ultimate capacity. The load factors for joints 3A50 and 3C50 were lower than 1.70, which is generally used for limit states design.

The ultimate load of joint 3A50 is 184 percent less than that of joint 3C50. This is because joint 3A50 had weaker webs than those of joint 3C50. Ultimate loads given in Table 4.4 are the loads in the compression webs at failure. These loads are presented in order to facilitate the comparison between the corresponding joints in the trusses and those tested in isolation.

It can be seen from the table that the ultimate loads for the corresponding joints in the two trusses were 17 to 23 percent lower than that of the corresponding isolated joint. It was assumed in the isolated joint tests that, during loading, the truss members tended to deform with points of inflection at the mid-length of each other. Hence the boundary con-

TABLE 4.4 COMPARISON BETWEEN ISOLATED AND TRUSS RESULTS

Joint	ULTIMATE LOAD IN COMP. WEB(KIPS)			Percentage Difference between (1) & (3)	Percentage Difference between (2) & (3)	Percentage Difference between (1) & (2)
	Truss - 1 (1)	Truss - 2 (2)	Isolated Joint (3)			
3A50	17.45	18.40	22.87	23.0	19.5	5.2
3C50	55.67	52.26	67.72	17.8	22.8	6.2

ditions in the isolated joints did not exactly simulate truss conditions. In the actual truss, rotation of the chord induced moments $-m'_c$ and m'_t in the web members as shown in Figure 4.2, which caused a high stress concentration at the joint. As a result, the joints in the test trusses failed at lower loads than did the corresponding joints tested in isolation where, due to different boundary conditions, these moments did not develop.

Dasgupta[9] found that ultimate loads for conventional joints incorporated into trusses were 25 to 30 percent lower than those obtained in isolated joint tests.

4.1.2 Joint Stiffness

The elastic stiffness of the joint was calculated as the slope of the initial straight line portion of the load-deformation plot for the joint. The stiffnesses of the truss test joints are repeated in Table 4.5, along with those of isolated joints. It can be seen from this table that the elastic stiffnesses of the corresponding joints in the two trusses are within 7 percent of each other. The joints with the thicker chord face are seen to be 60 to 100 percent stiffer than those with the thinner chord face, as the former offers more resistance to deformation. The elastic stiffnesses of joints 3A50 and 4A50 are 20 to 60 percent smaller than those for joints 3C50 and 4C50 respectively. This is because the webs for joints 3C50 and 4C50 were stiffer than those for joints 3A50 and 4A50.

The stiffnesses of the joints incorporated into the trusses were 0 to 8 percent greater than those of the corresponding isolated joints. The difference in stiffness could be due, in part, to slight differences in the sizes of the welds between web and chord members in the various

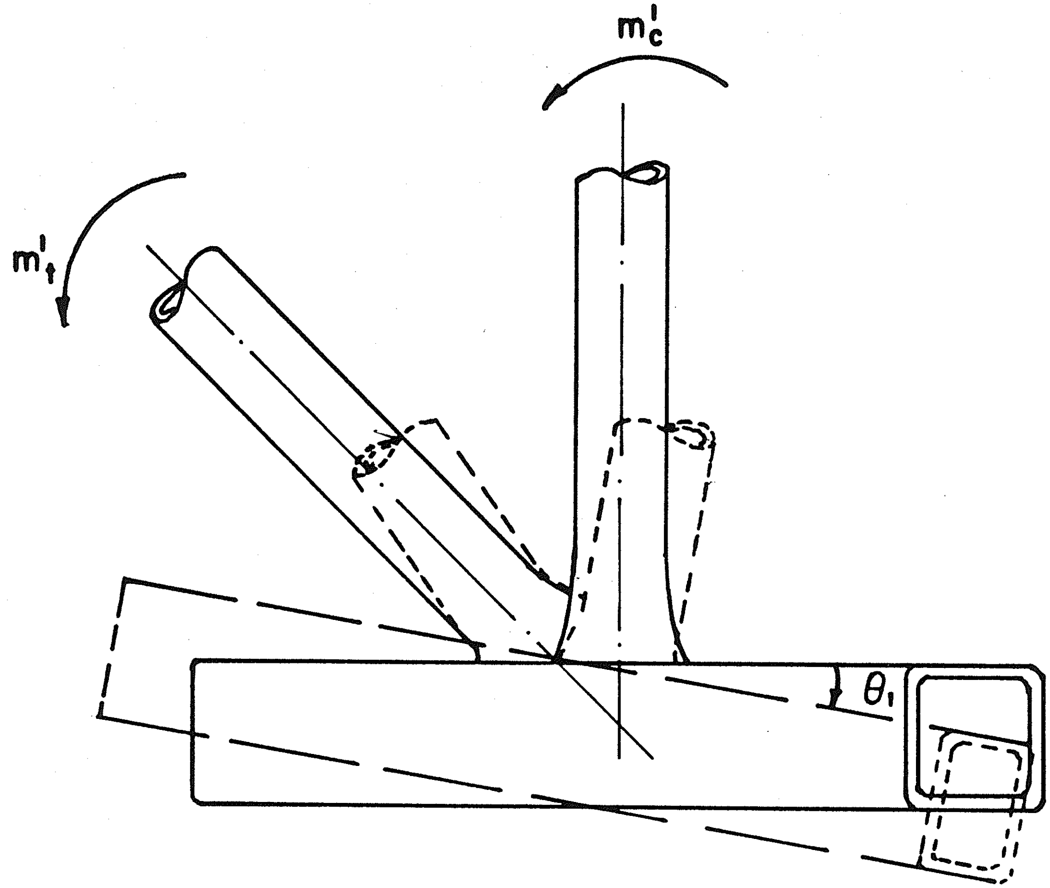


FIG. 4.2 ROTATION OF THE CHORD

TABLE 4.5 ELASTIC STIFFNESS OF TRUSS JOINT AND ISOLATED JOINT

Truss	Test Joint	Elastic Stiffness of Truss Joint (Kips/in.)	Percentage Difference	Elastic Stiffness of Joint in Isolated Test (Kips/in.)	Percentage Difference
1	3A50	980	2.0	980	0
2		1000			2.0
1	4A50	2000	4.8	2000	0
2		2100			4.8
1	3C50	1500	6.2	1500	0
2		1600			6.2
1	4C50	2400	4.0	2300	4.2
2		2500			8.0

specimens. In the truss joints relative rotation between the web members and the chord, due to overall truss deflection, would tend to diminish the local rotational deformation of the loaded chord face. This would, in turn, increase the joint stiffnesses.

4.1.3 Modes of Failure

The modes of failure for the test joints in the two trusses are indicated in Tables 4.1 and 4.2.

The mechanism of failure was consistent. The web member forces, indicated as F_C and F_T in Figure 4.3, developed a moment at the chord face, causing it to undergo rotation. This rotation then induced moments at the end of the web members, which tended to cause them to bend into an S-shape in the plane of the truss.

Joints 3A50 and 4A50, which involved slender webs, consistently failed by compression web buckling. With the increase in load, the compression webs at joints 3A50 and 4A50 began to bend in the plane of the truss. Then, as the load in the compression member reached its critical value, the member failed abruptly by buckling out of the plane of the truss (perpendicular to the direction of cropping).

Joints 3C50 and 4C50, which involved large web members, did not fail by buckling of the compression web. However, yielding occurred in the compression web at joint 3C50 in truss 1. In both trusses, the modes of failure of these joints involved large plastic deformation of the chord face, followed by tearing of tension web, as shown in Figure 4.4.

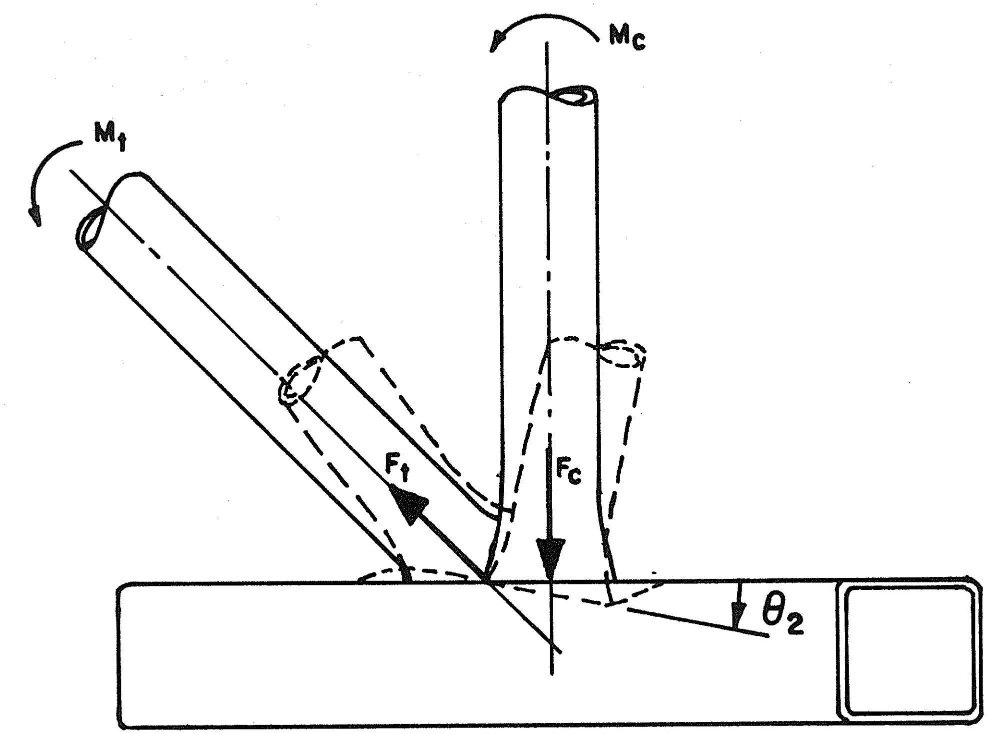


FIG. 4.3 FAILURE MECHANISM

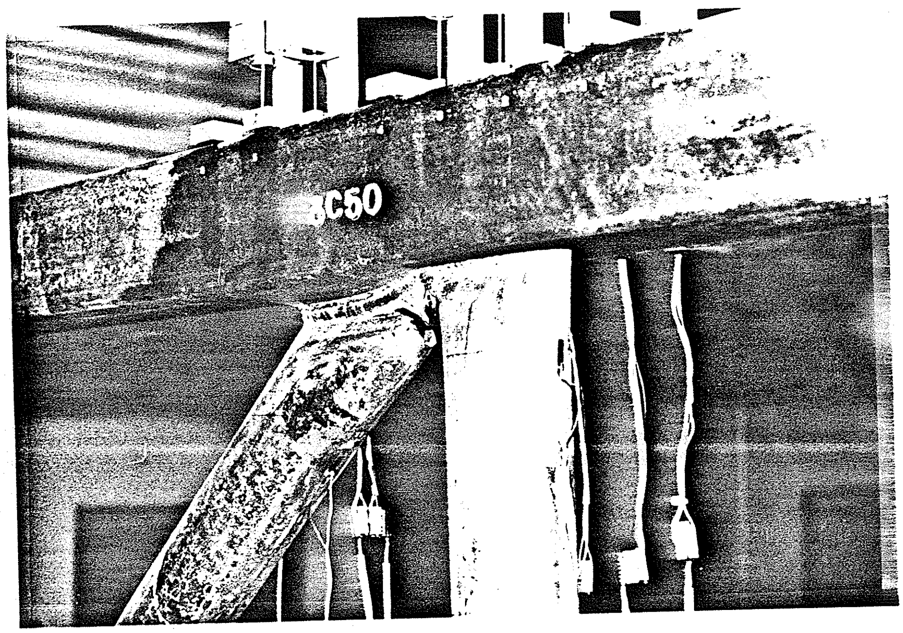


FIG. 4.4 TEARING OF TENSION WEB

4.2 Load-Deformation Behaviour of Loaded Chord Face

Plots of the deformed shapes of the loaded chord face at each of the test joints are presented in Figures 4.5(a) to (h). In the figure, the locations of transducers used to measure chord face deformation are indicated by T_1, T_2 etc.

The deformation of the chord wall at the location of T_2 was always greater than those at T_3 and T_4 because the wall at the latter locations was pulled up somewhat by the tension web. In the elastic range, the deformation of the chord face at T_2 was comparable to that at T_5, in all joints. It can be seen from the proportionality of the curves at different load levels that the deformed shape of the chord face remained approximately constant throughout the elastic region and even up to the ultimate load. Furthermore, the chord face remained approximately linear over the length of the connection to the webs. From the deflection profiles for all tests, it can be seen that the point where deflection changed from upward to downward remained more or less stationary at low load levels.

For the joints with relatively thin chord faces, it was noticed that at loads beyond the working load of the compression web, the deflection of the chord face at location T_2 was greater than that at T_5. This is illustrated in Figures 4.5(a) and (e). It was partly because of the rotational 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.

Comparative plots of chord face deformation for the truss joints and the corresponding joints previously tested in isolation are shown in Figures 4.5(i) to (ℓ). They correspond to a load that is less than the

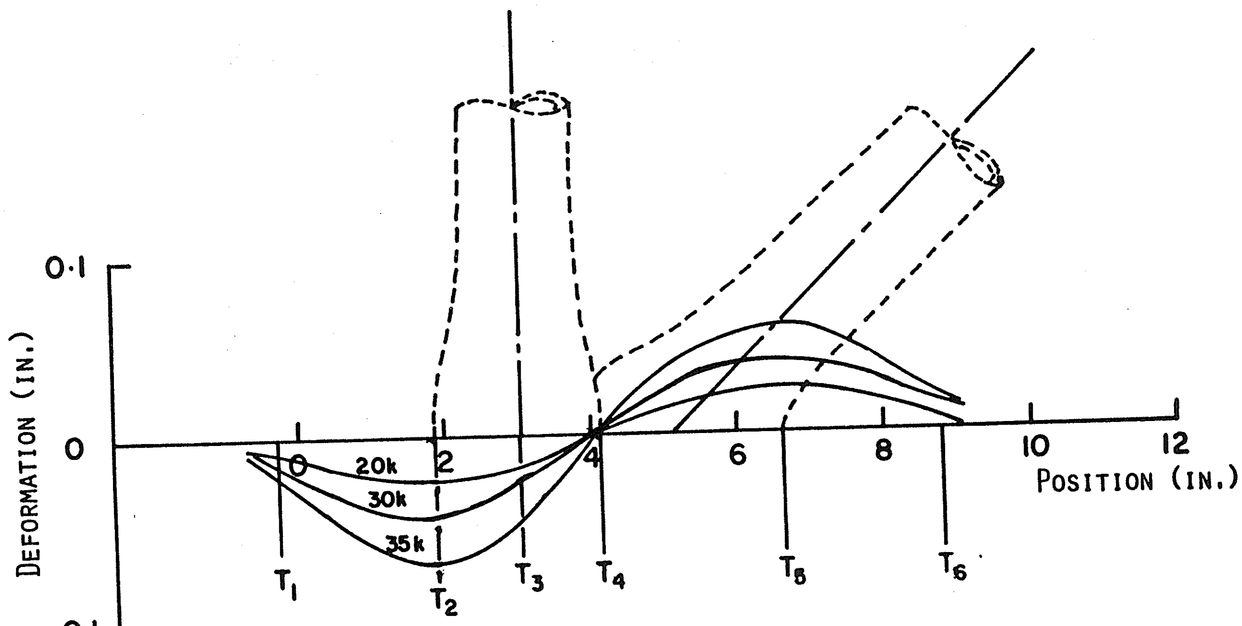


FIG. 4.5(a) DEFORMATION OF LOADED CHORD FACE OF JOINT 3A50, TRUSS-1

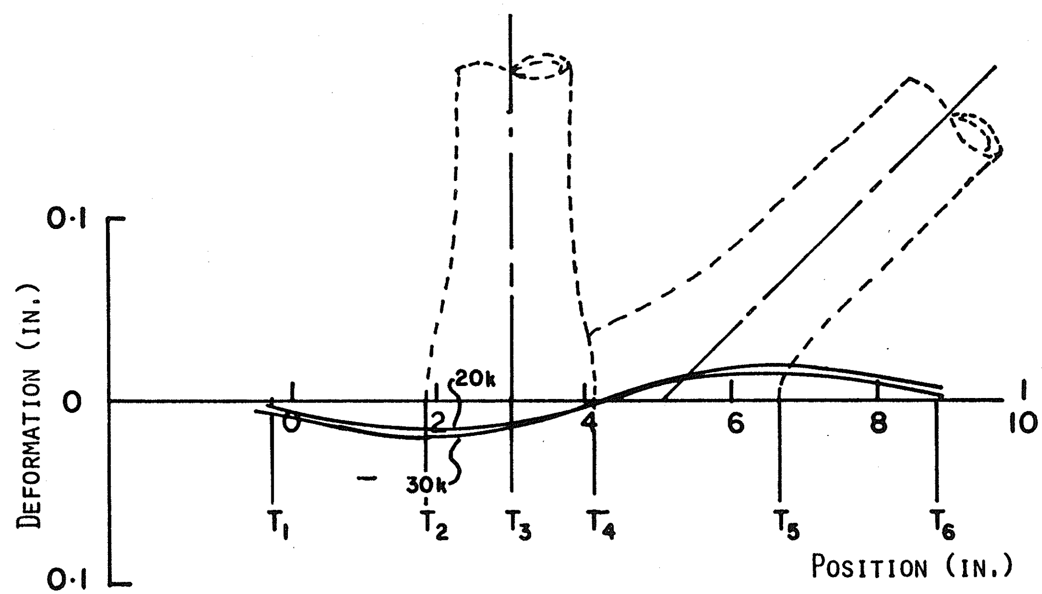


FIG. 4.5(b) DEFORMATION OF LOADED CHORD FACE
OF JOINT 4A50, TRUSS-1

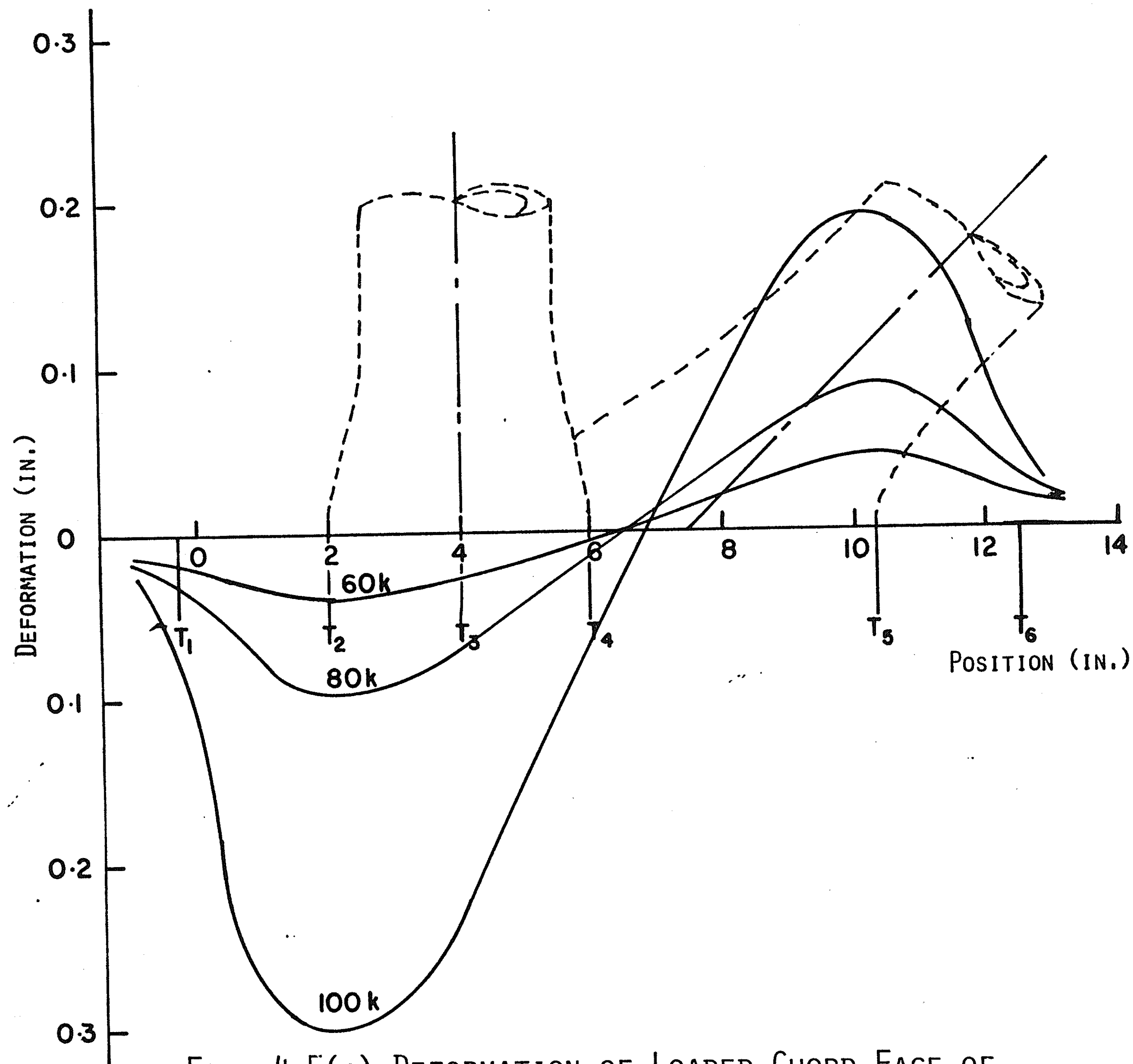


FIG. 4.5(c) DEFORMATION OF LOADED CHORD FACE OF JOINT 3C50, TRUSS-1

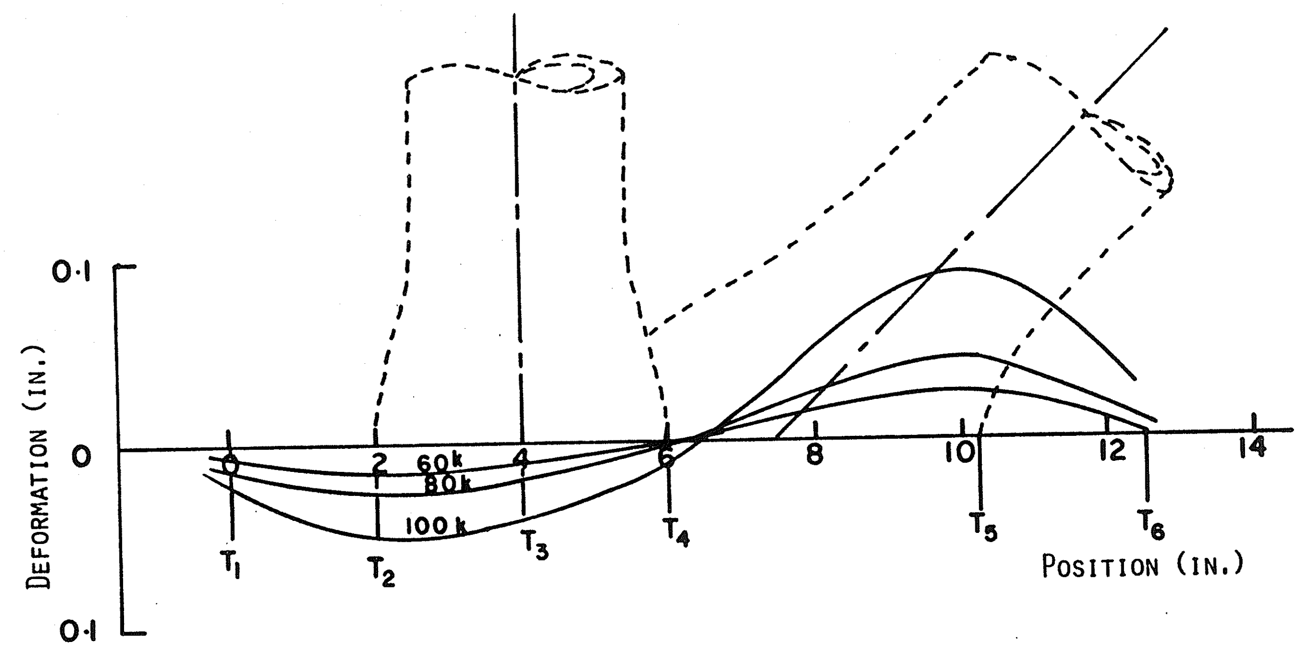


FIG. 4.5(d) DEFORMATION OF LOADED CHORD FACE OF JOINT 4C50, TRUSS-1

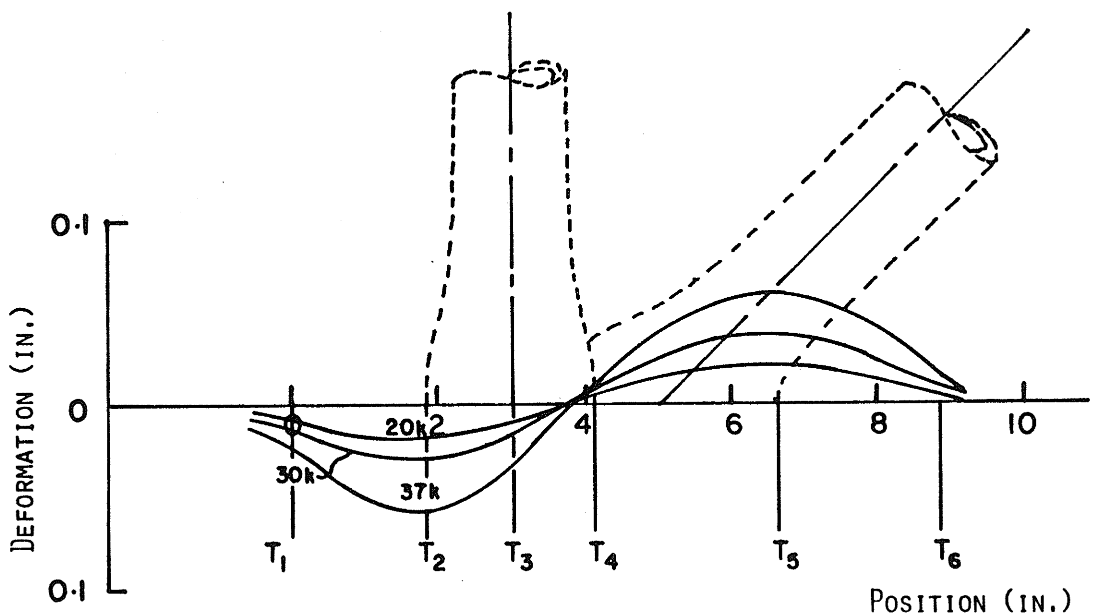


FIG. 4.5(e) DEFORMATION OF LOADED CHORD FACE OF JOINT 3A50, TRUSS-2

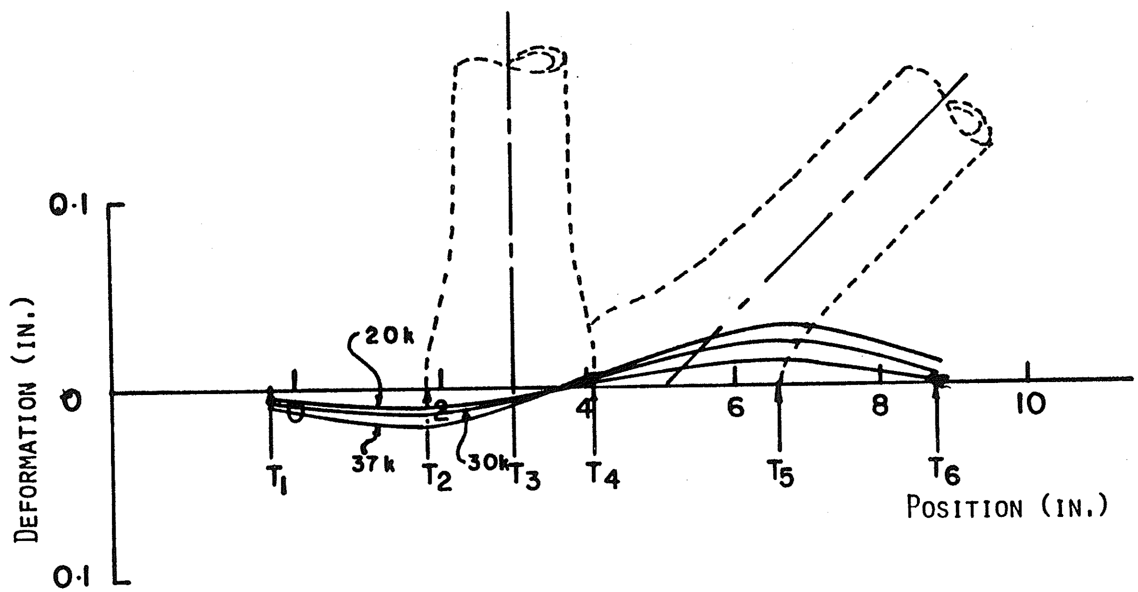


FIG. 4.5(f) DEFORMATION OF LOADED CHORD FACE OF JOINT 4A50, TRUSS-2

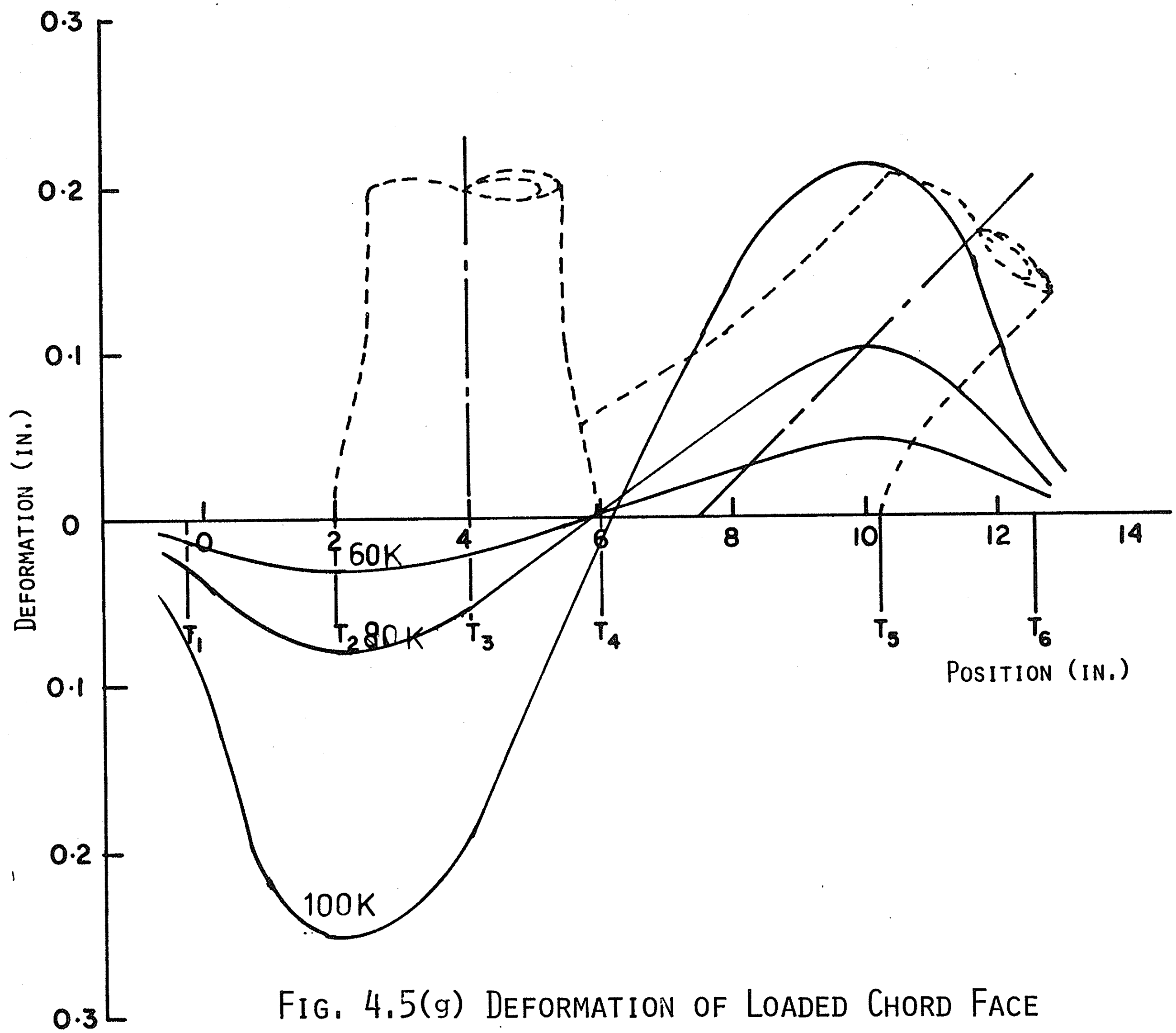


FIG. 4.5(g) DEFORMATION OF LOADED CHORD FACE OF JOINT 3C50, TRUSS-2

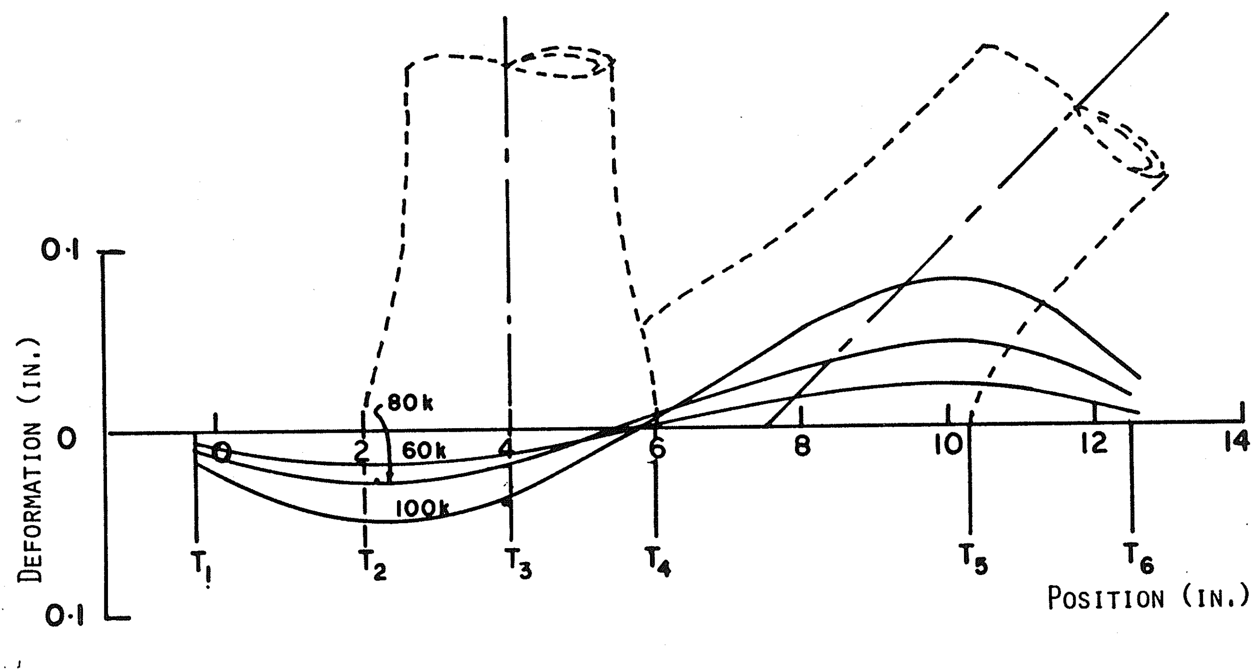


Fig. 4.5(h) Deformation of Loaded Chord Face of Joint 4C50, Truss-2

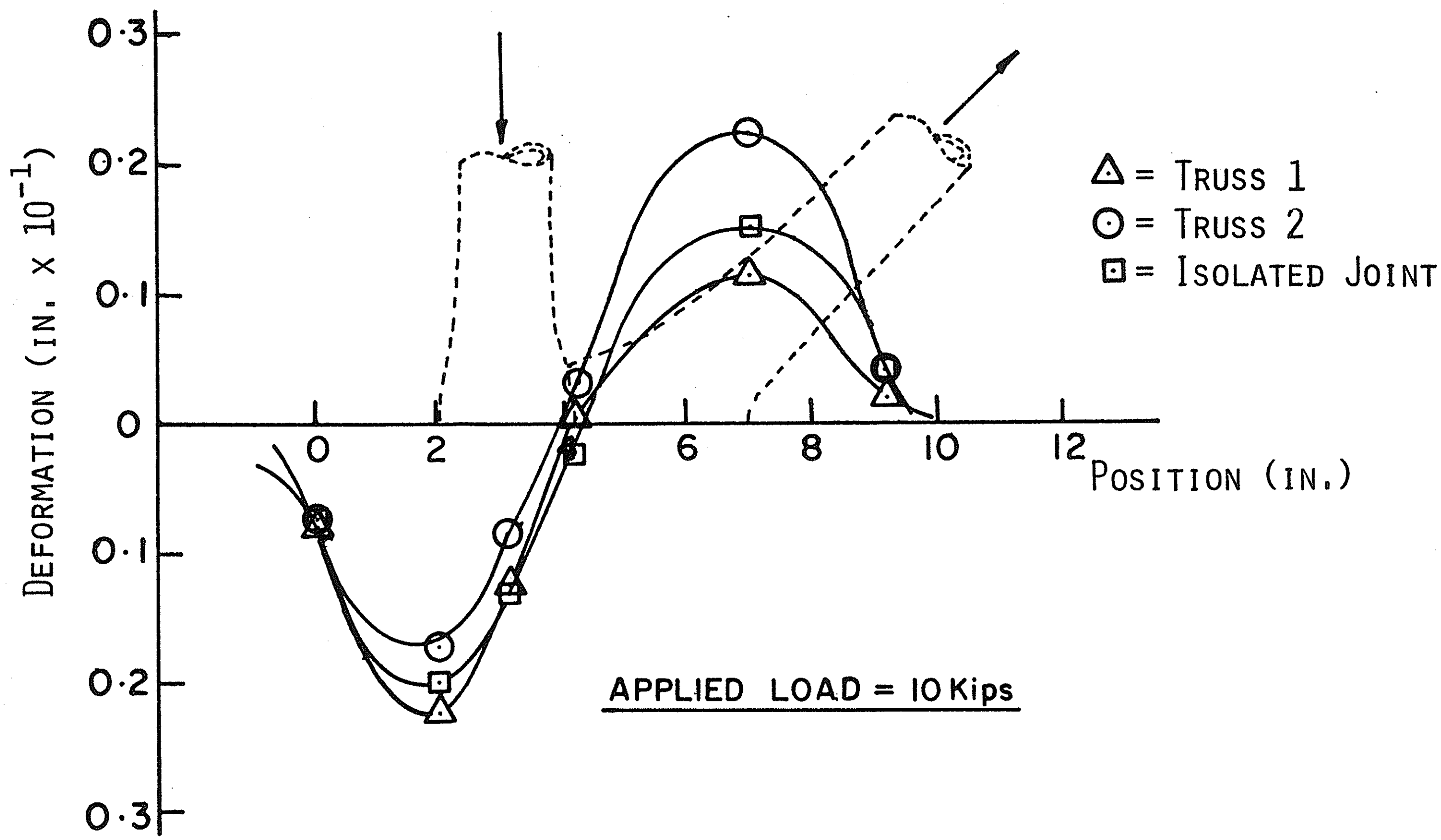


Fig. 4.5(i) Comparison of Chord Face Deformation - Joint 3A50

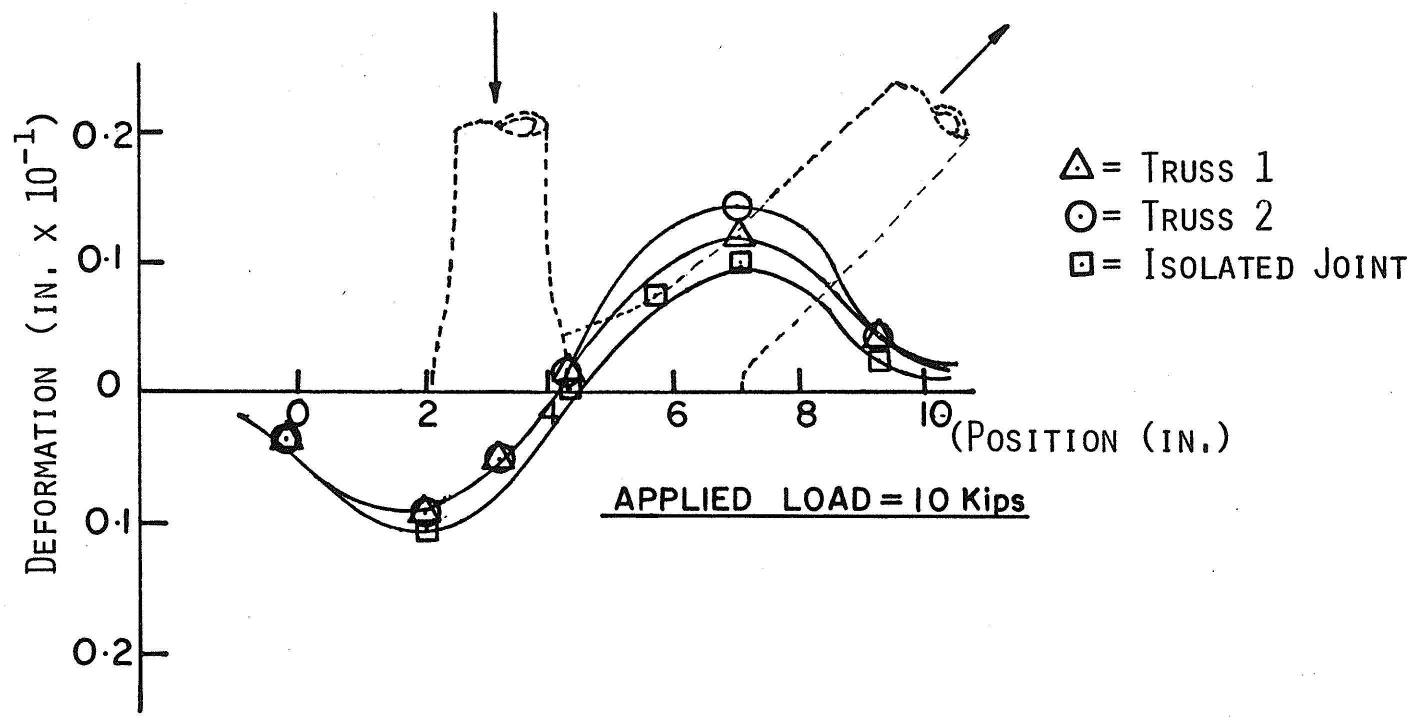


FIG. 4.5(j) COMPARISON OF CHORD FACE DEFORMATION - JOINT 4A50

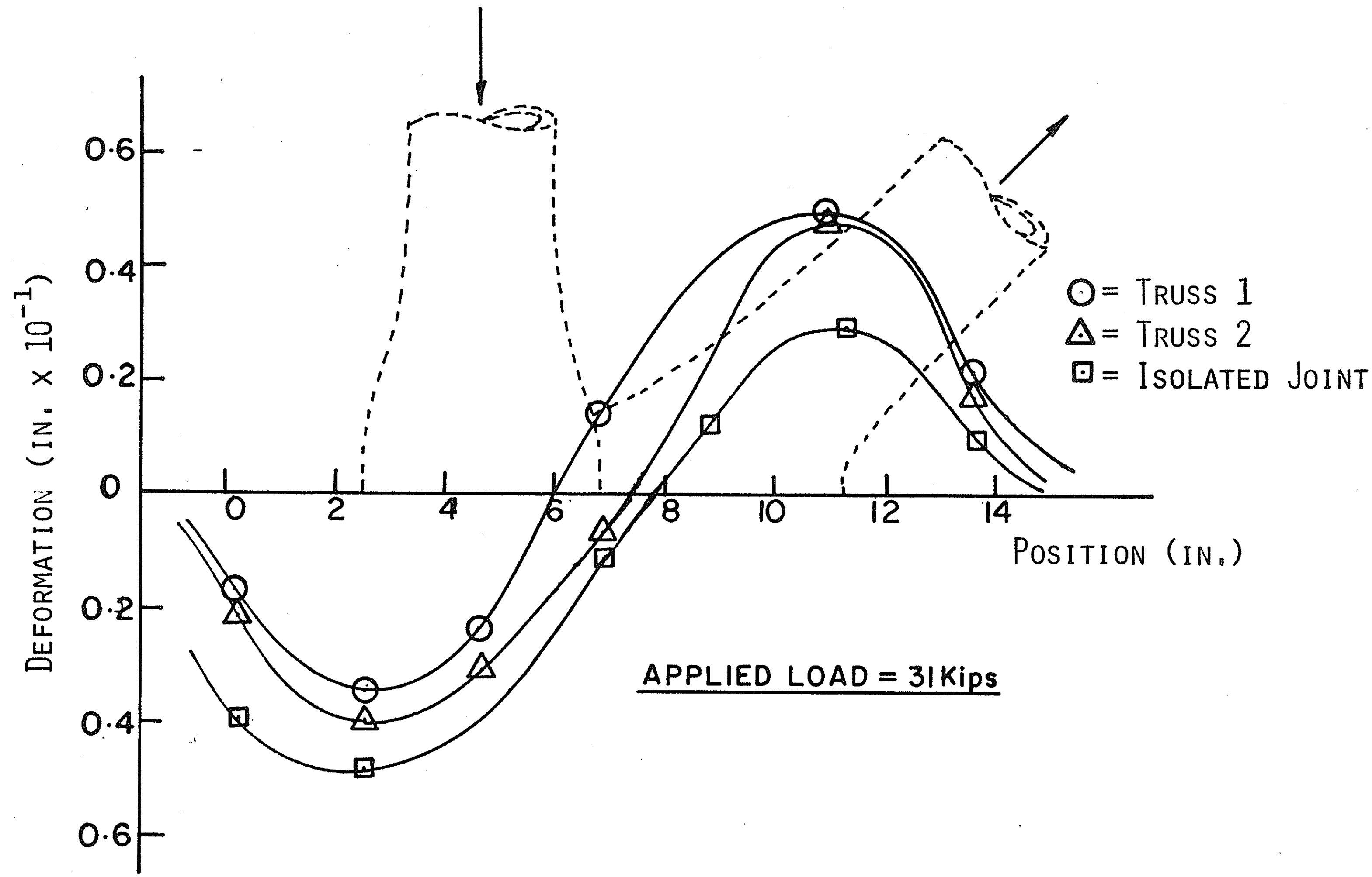


FIG. 4.5(k) COMPARISON OF CHORD FACE DEFORMATION - JOINT 3C50

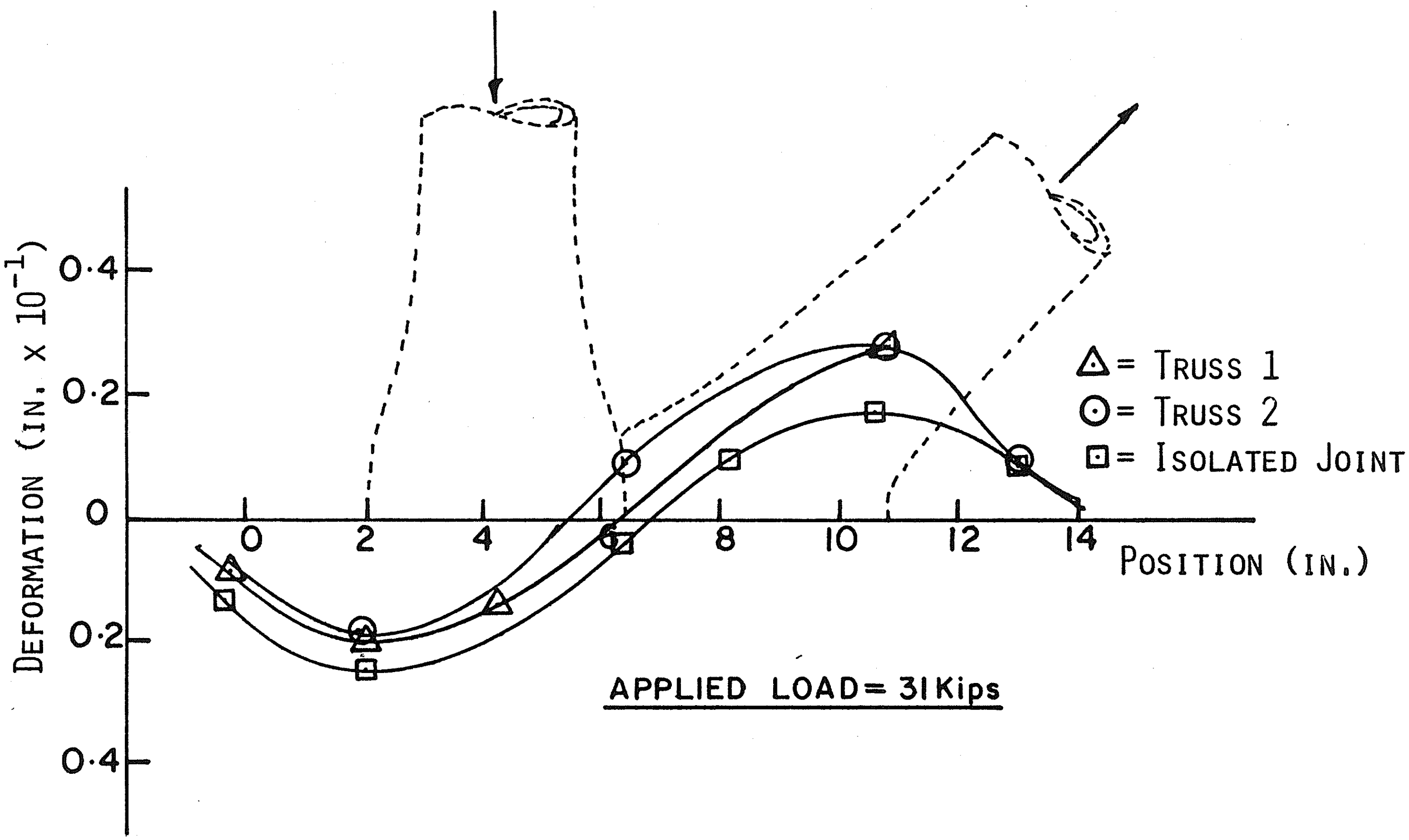


Fig. 4.5(ℓ) Comparison of Chord Face Deformation - Joint 4C50

yield load. It can be seen that the patterns of chord face deformation are very similar among the three joints in each case. However, in the case of joints 3C50 and 4C50 the chord face deflection under the tension web in the truss joints was greater than that in the isolated joints. On the other hand, that under the compression web was smaller.

Due to overall truss deflection, relative rotation occurred between the web members and the chord. This rotation was in the same direction as the chord face rotation. Thus it tended to reduce the displacement of the compression web into the chord face. Just the opposite phenomenon occurred in the case of the tension web. In the isolated joints, the relative rotation between the chord and the web members was not present.

4.3 Contribution of Joint Deformation to Overall Truss Deflection

As the deflection of a truss under load is an important criterion in design, it was decided to investigate the relationship between joint stiffness and truss deflection. Using energy methods and the member stiffnesses, the midspan truss deflection, due to member deformation only, was first calculated. Then, using the joint stiffnesses in Table 4.5 for the compression webs and the corresponding tension web displacements obtained from the chord face deformation plots, the increase in midspan deflection due to the test joint deformations was calculated. The results and percentage increase in deflection caused by joint deformation are given in Table 4.6. It can be seen that within the elastic range, the deformation of the test joints contributed less than 3 percent to truss deflection.

TABLE 4.6 DEFLECTION OF THE TRUSSES 1 AND 2 DUE TO JOINT AND MEMBER DEFORMATION

Truss	Deflection Due to Member Deformation δm (in.)	Deflection Due to Joint Deformation δj (in.)	Percentage Increase Due to Test Joint Deformation
1	0.0640	0.0014	2.17
2	0.0640	0.0018	2.76

4.4 Strain Readings

Strain gauge readings were used to determine in-plane bending moments at the ends of the web members at the test joints. Figures 4.6(a) to (c) show typical strain gauge locations and typical load-strain plots. The moments at the ends of the compression web were then used to compute the location of its point of inflection at various load levels.

Plots of load applied to the truss vs location of point of inflection in the web, are shown in Figure 4.7. It was found from these plots that the point of inflection lay very near the mid length of the compression web until the chord face began to yield. As the chord face deflection at the joint at one end of the member exceeded approximately 1 percent of the chord thickness, the bending moment of the web at that end decreased and the point of inflection moved toward that joint. The bending moment at that end of the web decreased because, as the chord face yielded, the webs moved into it without rotating. In all cases, points of inflection moved upward because the top chord was thinner than the bottom one and it began to yield first.

The strain gauge readings at locations 1,2,3,4,9,10,11 and 12, near the joints at the two ends of the compression web, were used to examine the bending behaviour of the web. The in-plane deflection of the compression web was consistent with the chord face deformation previously described.

The strains at locations 1,2,9 and 10 illustrated that the compression web bent into an S-shape with a point of inflection about the mid-length of the web. This was confirmed from the readings of strain gauges 5,67 and 8 which were located at the mid-length of the compression web.

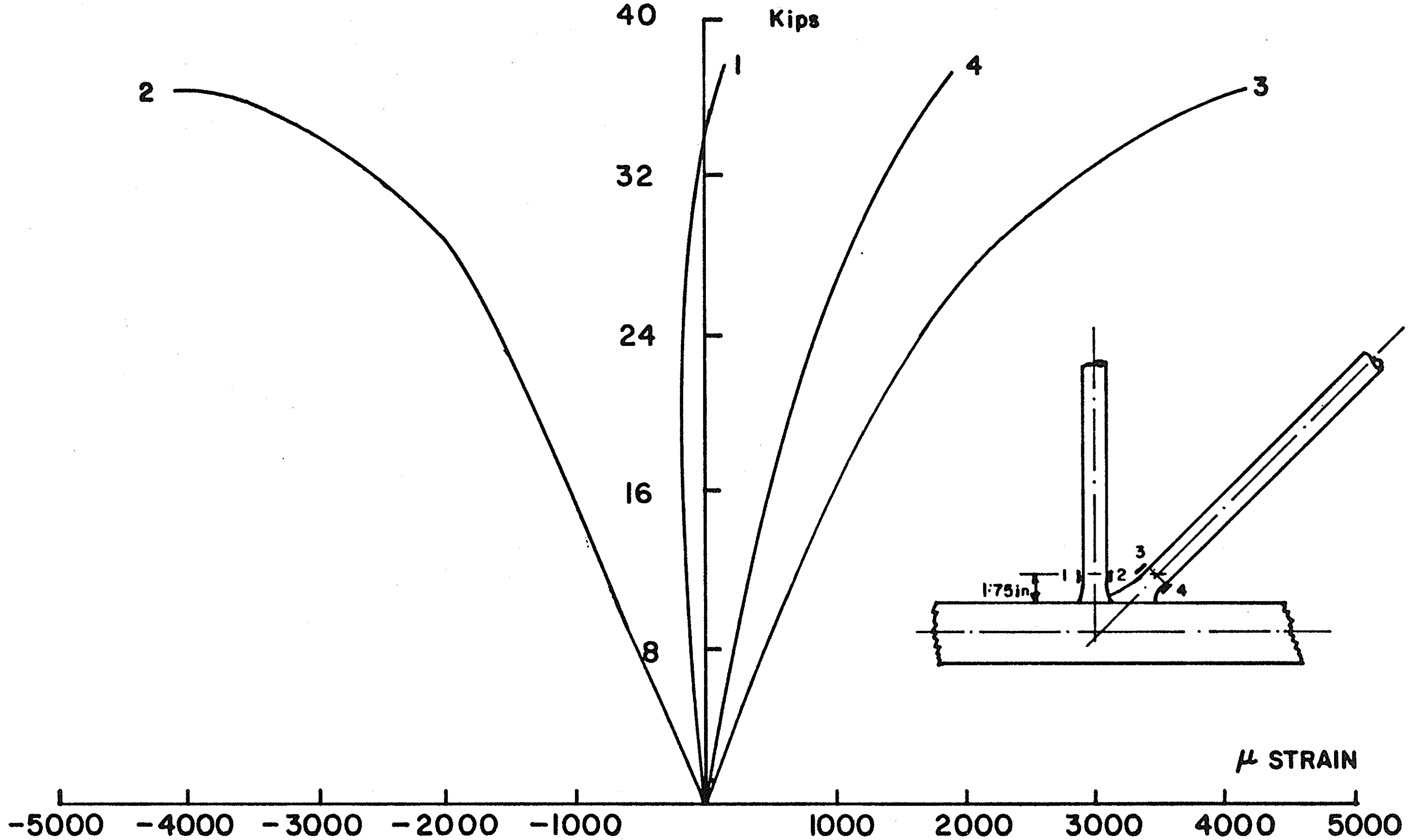


FIG. 4.6(a) STRAIN PLOTS, JOINT 4A50, TRUSS-1

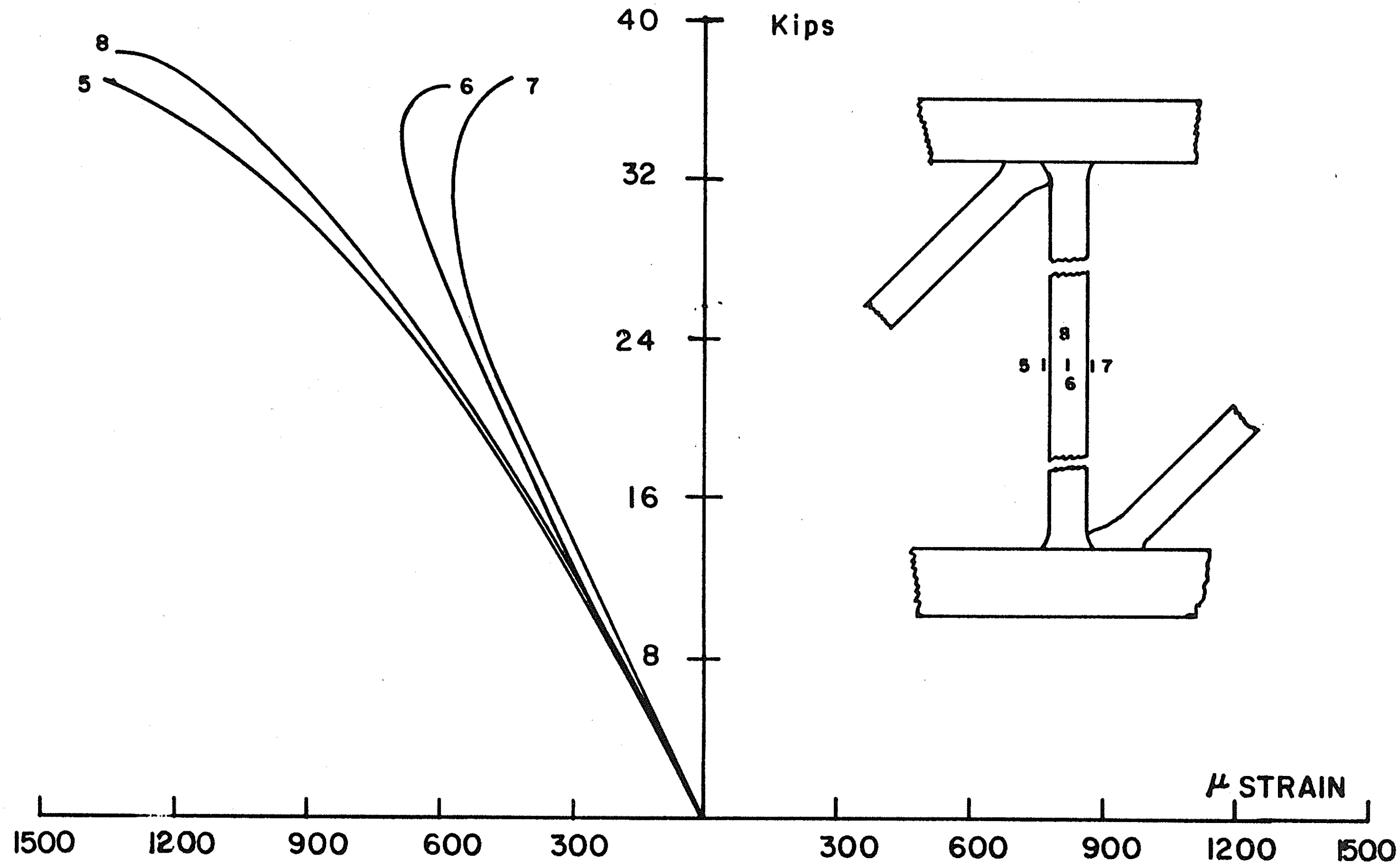


FIG. 4.6(b) STRAIN PLOTS, GAUGES 5,6,7,8, WEB 1.66 OD x 0.188

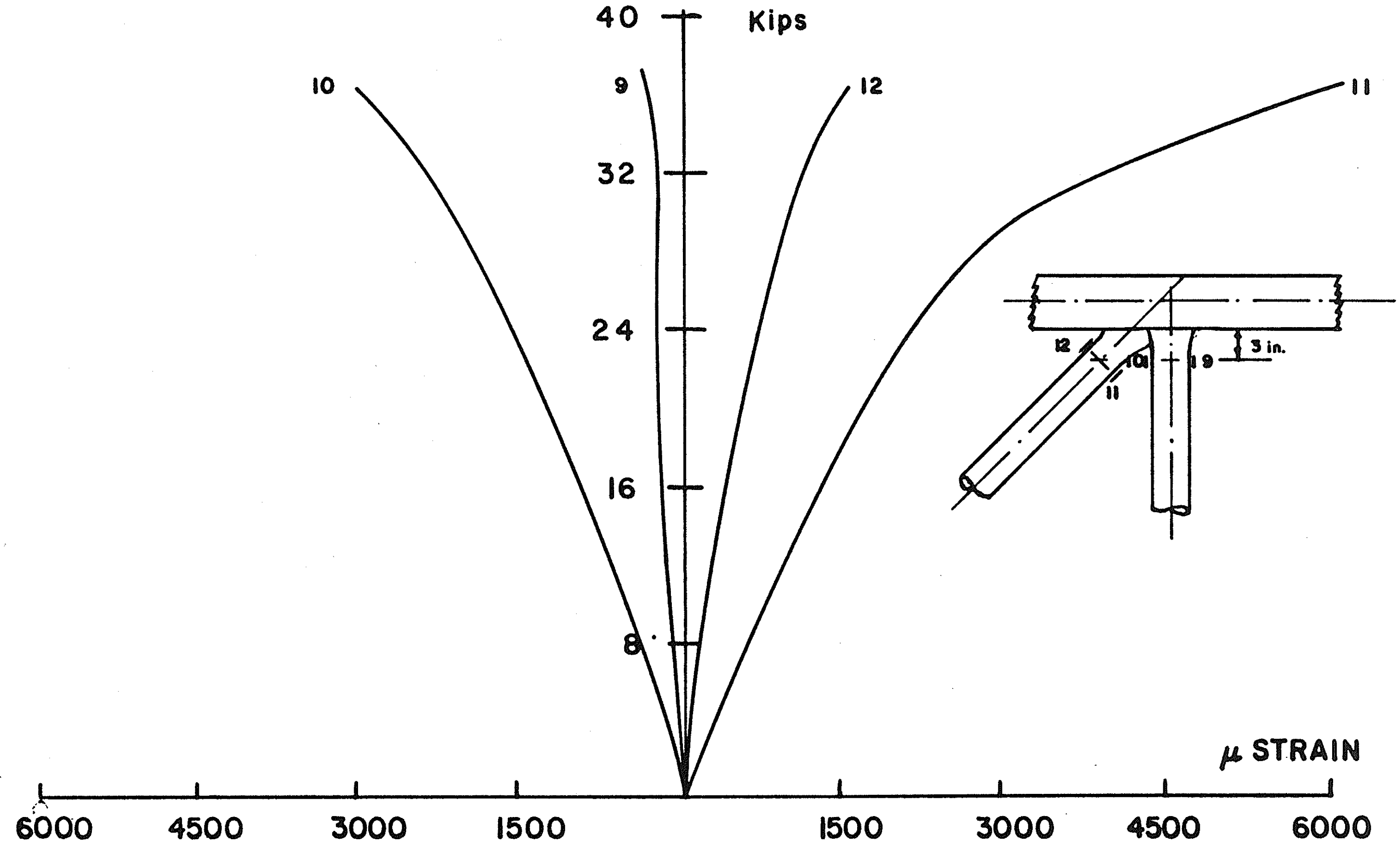


FIG. 4.6(c) STRAIN PLOTS, JOINT 3A50, TRUSS-1

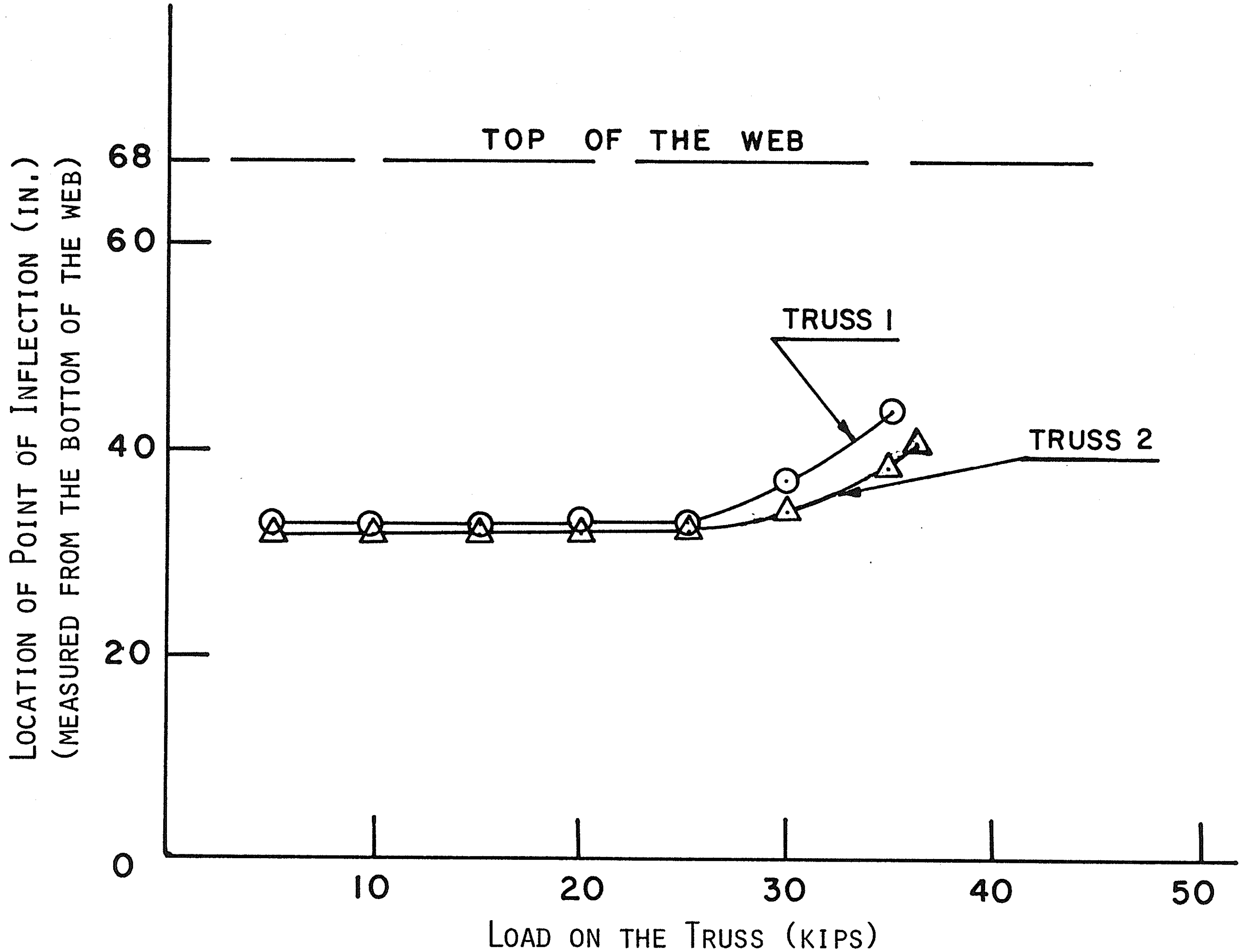


FIG. 4.7(a) PLOT OF POINT OF INFLECTION VS LOAD

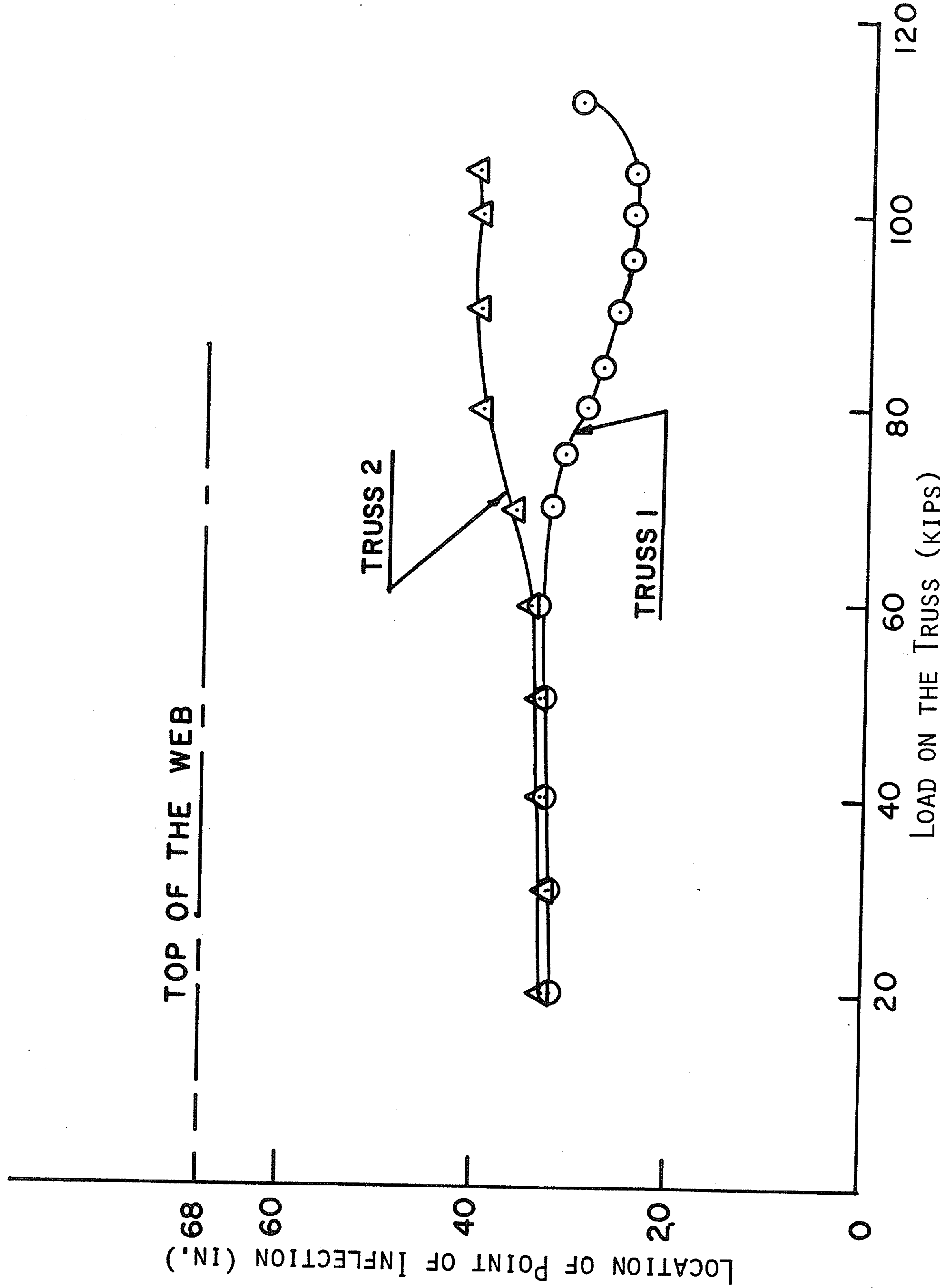


FIG. 4.7(b) PLOT OF POINT OF INFLECTION VS LOAD

The readings for the latter four strain gauges confirmed that very little bending strain was present at that location. For joint 4A50, which had a slender web but a moderately thick chord face, a reversal of the local moment acting on the chord face was noticed. The reason may be that as joint 3A50, which was at the far end of the compression web of joint 4A50, began to yield, the chord face at joint 4A50 resisted the in-plane rotation of the web and eventually the web buckled out-of-plane. This reversal of moment was verified from the strain readings for gauge 1, as shown in Figure 4.6(a).

The readings at the mid-lengths of the slender compression webs (those connecting joints 3A50 and 4A50) were used to construct Southwell plots as illustrated in Appendix I, in order to compute elastic buckling loads and effective length factors for out-of-plane buckling of the webs.

The buckling loads obtained from the Southwell plots were 18.2 kips and 17.8 kips for the compression webs in trusses 1 and 2 respectively. The corresponding buckling loads obtained from direct observation were 17.4 kips and 18.4 kips.

The values of effective length factors obtained from the Southwell plots were 0.78 for the slender webs in both trusses. This value is approximately 10% larger than the value of 0.7 recommended for a compression web in conventional tubular trusses.[24] In a study of effective length factors for cropped tubular steel struts, carried out at the University of Manitoba[29], end-cropped 1.66 O.D. H.S.S. strut specimens with various degrees of end-fixity were found to have effective length factors ranging from 0.64 to 1.0.

The calculated buckling loads and the effective length factors for slender webs are presented in Table 4.7.

TABLE 4.7 COMPARISON BETWEEN BUCKLING LOADS BY SOUTHWELL PLOT AND TEST RESULTS

Truss	BUCKLING LOAD (KIPS)		Percentage Difference	Effective Length Using Southwell Plot
	Southwell Plot	Direct Observation		
1	18.2	17.4	4.1	0.78
2	17.8	18.4	3	0.78

CHAPTER V

CONCLUSION

Statical load tests were performed on two similar twenty-four foot span, six foot deep welded Pratt trusses composed of square H.S.S. chords and round end-cropped H.S.S. webs. The trusses each incorporated four test joints which were similar to truss joint specimens tested as isolated specimens, in an earlier investigation.

The test results indicated the following:

1. The yield loads for three of four test joints, for both trusses, were virtually identical. The fourth test joint showed little sign of yielding, as its compression web buckled before the joint capacity was reached.
2. Load factors for corresponding joints in both trusses were within six percent of each other.
3. The ultimate loads of the joints increased with an increase in the ratio of web diameter to chord width.
4. The elastic stiffnesses of the joints (defined as the initial slope of the load deformation plot), increased with a decrease in the ratio of chord width to chord thickness.
5. The elastic stiffnesses of the joints increased with an increase in the ratio of web diameter to chord width.
6. The elastic stiffnesses of corresponding joints in the two trusses were within seven percent of each other.
7. The deformed shape of the loaded chord face at the joint remained

approximately constant throughout the elastic range and even up to the ultimate load. The deformation profile in each case, was approximately linear over the length of the web-chord welded connection.

8. The local deformation of the chord wall at the joints contributed an increase of approximately three percent to the mid-span deflection of each truss.

9. The point of inflection in the compression web connecting the test joints was located at the mid-length of the web throughout the elastic range.

10. The joints with a slender compression web failed by buckling of the web out of the plane of the truss (perpendicular to the direction of cropping), with an effective length factor of 0.78. The joints which involved stocky web members failed primarily due to large deformation of chord face at the joint.

11. The isolated joint tests performed in an earlier investigation simulated the conditions in the truss fairly well. The elastic stiffnesses of corresponding joints in the isolated tests and the truss tests were within eight percent of each other.

The yield load of one of the joints involving a slender compression web was the same as that of the joint tested in isolation. However, the truss joints with stocky web members yielded at lower loads than did the corresponding isolated joints. Due to overall truss deflection, there was relative rotation between the web members and the chord in the truss joint. This reduced the yield loads and the ultimate loads of the stronger truss joints by approximately twenty percent. The relative rotation, and hence the reduction in yield load, did not occur in the isolated joint tests.

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A P P E N D I X

THE SOUTHWELL PLOT METHOD FOR CALCULATING ELASTIC BUCKLING LOADS

The Southwell Plot provides a means for determining the theoretical elastic buckling load for a real column. The inverse of the slope of the line obtained by plotting the ratio of bending strain to axial load along the abscissa, can be shown to equal the elastic buckling load for the column. Thus

$$P_{cr} = \frac{\varepsilon - P/EA}{\varepsilon - \frac{P/EA}{P}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where

P_{cr} = elastic buckling load

P = axial load

$\varepsilon - P/EA$ = bending strain.

An example of the Southwell Plot, that for the compression web connecting joints   and   in truss 1 tested in this study, is presented in Figure A1. When the elastic buckling load for a column has been determined, the effective length factor can be determined from the elastic buckling formula.

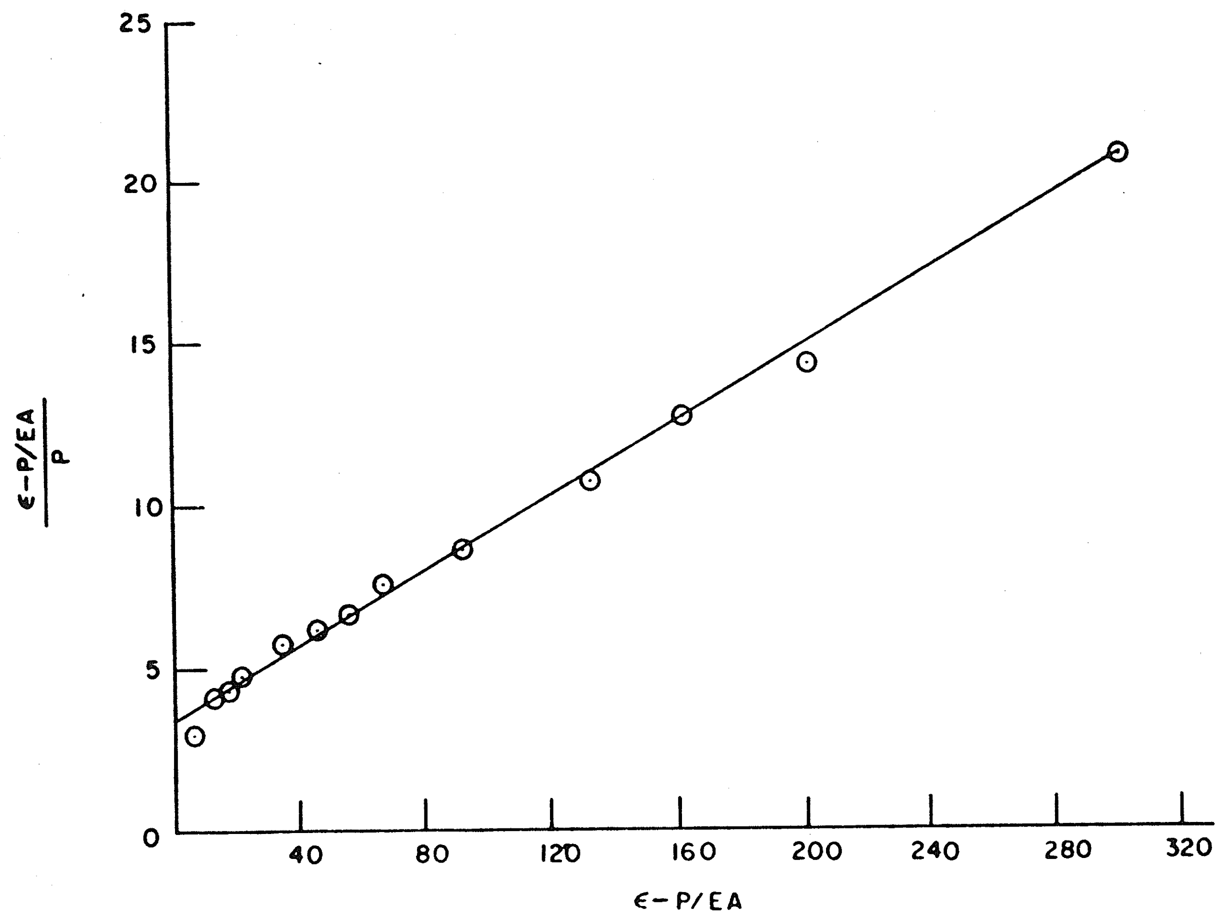
$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where

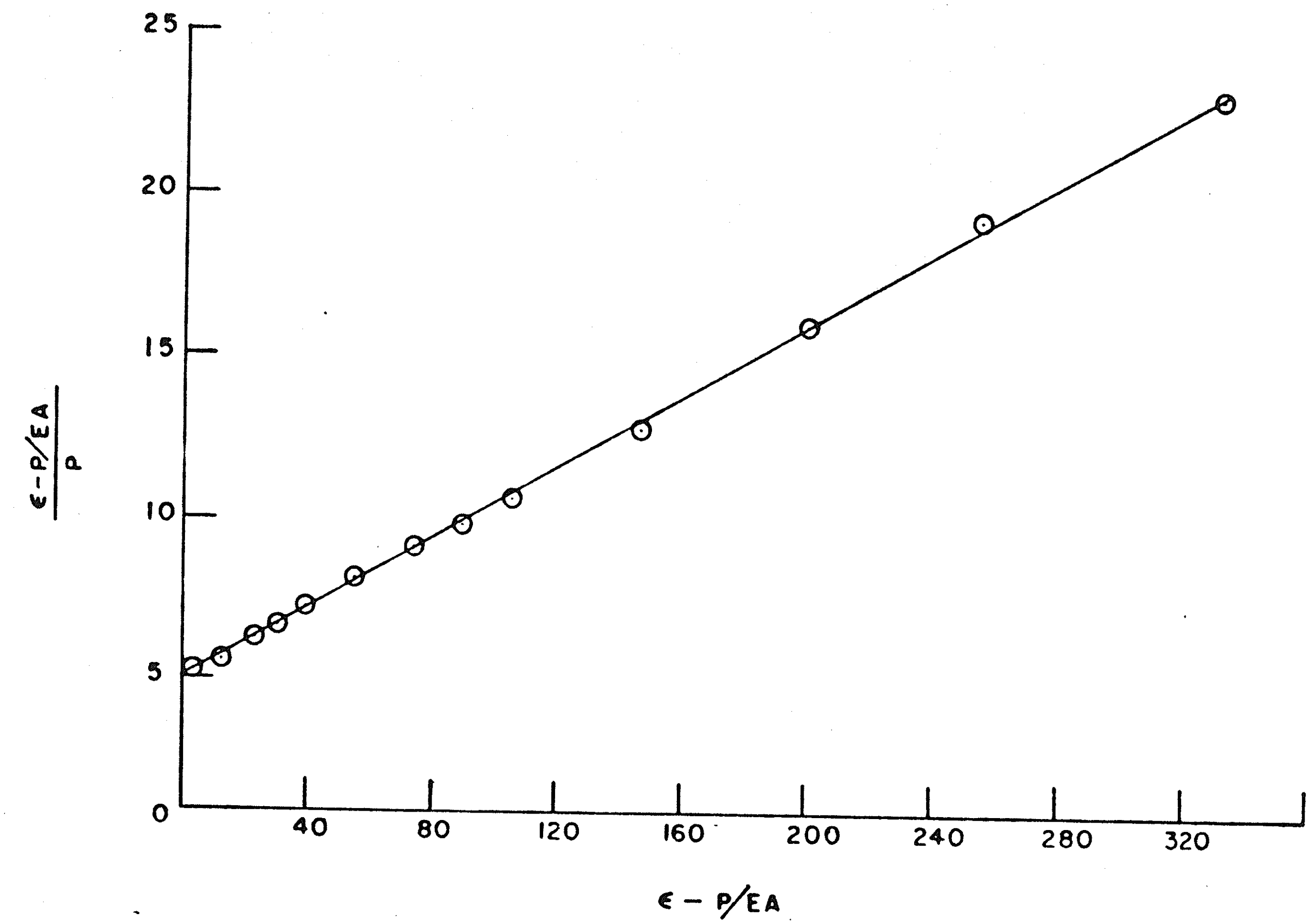
K = effective length factor

I = second moment of column cross-sectional area

L = column length.



SOUTHWELL PLOT WEB CONNECTING THE JOINTS
3A50 AND 4A50 (TRUSS-2)



SOUTHWELL PLOT WEB CONNECTING THE JOINTS
3A50 AND 4A50 (TRUSS-1)