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TUBULAR JOINTS UNDER STATIC AND ALTERNATING LOADS

BY

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Department of Civil Engineering
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TUBULAR JOINTS UNDER STATIC AND ALTERNATING LOADS

(Phase I)

A Report of an Investigation

by

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to

Standard Oil Company of California
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I. INTRODUCTION

I-1. General

The design of welded truss-joints built from circular tubes has been complicated by the radial flexibility of the tube walls. For most joints this flexibility is a source of severe stress concentrations. In connections subjected to alternating loads, as in off-shore well-drilling structures, these stress concentrations are particularly dangerous since they can initiate an early failure in these joints.

In an attempt to reduce the influence of the wall flexibility of the column tube certain joints are presently designed by interwelding the incoming branch members. In those designs the force acting normal to the column tube can be reduced considerably. A second group of joints incorporates gusset plates or stiffening rings, which are intended either to contain the load transfer primarily within the gusset plate or to provide a number of elements - the stiffening rings - to stiffen the column wall and to distribute the incoming branch member forces over a larger part of this wall. A third approach to improve the stress distribution in a tubular joint is to increase the wall thickness of the column section. This can be achieved by simply applying a thicker-wall section in the vicinity of the joint. A fourth possibility to restrain the radial flexibility is to incorporate a concrete filled column member. Also a single cast-steel seat welded to the chord tube can be used to improve the stress distribution in the column section. Although all these designs improve in general the state of stress in the column wall, the altered stiffness often causes the development of critically stressed areas in the web members.

The actual design in most instances is decisively influenced by the site conditions as well as shipping, erection and foundation requirements of the offshore tower. Present design practice neglects the presence of stress concentrations and permits an increase of the basic allowable stress by one third in case lateral loads due to wave action, seismic excitations or ice flow are considered.

The complexity of the structural configuration of tubular joints permitted for long only a very crude and approximate analysis of the stresses in these connections. Recent analytical developments however, seem to indicate that it will be possible in the future to evaluate the stresses in critical areas more accurately. Although these limited analytical results, as based upon an assumed linear response of the joints, will undoubtedly aid in the effective development of tubular joints, the true response can only be predicted after selective experimental studies. Particularly the study of the behavior of joints under alternating loads, as in off-shore structures, will require extensive experimental investigations in order to improve the design of tubular joints. These studies should include an evaluation of the scale effects for both the pure structural response as well as for the influence of environmental conditions.

The studies reported herein deal with the first phase of a program to evaluate the relative behavior, under both static and alternating loads, of a number of specific joints of the type used in off-shore well-drilling structures.

I-2. Objectives and Scope

The objective of this program is to evaluate the response, under alternating loads, of several types of truss connections built from circular tubes. The joints are scaled-down versions of full scale connections commonly used in off-shore well-drilling structures. Based upon the results of the initial study on two-dimensional-truss joints, reported herein, the joint types showing a superior response will be selected for further detailed studies. These studies will cover the feasibility of geometric changes in the design of these specific connections in order to investigate the influence of several structural variables and to improve the response of these joints under alternating loads. In order to determine the life-expectancy of the selected joints the fatigue studies will be carried out at different levels of nominal alternating stress.

Also the influence of a corrosive environment as sea water will be studied. In general it can be noted that under extremely high stress levels and consequently large stress concentrations, the phenomenon of corrosion fatigue will probably be insignificant. To what degree corrosion fatigue plays a decisive roll in the behavior of the joints under small alternating loads will be investigated.

Although the initial studies will be carried out on joints with members in one plane only, future work will also include the study of three-dimensional truss joints. Because it will be advantageous from an experimental point of view to perform this part of the research on smaller scale connections a study of the possible scale effects will be included in the overall program. These results will also be significant to predict, based on the test results, the response of actual, full scale, joints.

The entire program is to be carried out in three phases. The first phase, the results of which are reported herein, covered the study of five different types of joints under static and alternating loads. The alternating loadings were applied with the specimen submerged in salt-water. The primary objective of this initial survey study was to evaluate the relative performance of these two-dimensional truss connections. Joints showing the most promising results were then selected for further study in the next phase of the program.

The second phase is to cover an extended feasibility study of the selected joint types in order to improve their design and subsequently their performance under alternating loads. These studies will include an evaluation of the endurance limit for different alternating stress levels. Also, the influence of the corrosive environment as compared to in-air conditions will be studied. In case the environmental conditions, even at low stress levels, should not influence the endurance limit the studies could be carried out in-air rather than in salt water. To evaluate possible scale effects a number of reduced-scale connections will be investigated under conditions identical to those used for the larger scale joints.

The third and presumably final phase of the program will cover the study of a limited number of two-dimensional truss joints basically similar to those tested in phase two, but with a different orientation of the diagonal branch member. (Type B joint in Fig. 2). Also the behavior of three dimensional truss joints will be studied. The results concerning the scale effects and the influence of the environmental conditions, to be obtained in phase II, are of significance for the design of this third phase. The studies in this phase will permit the evaluation of the behavior of more complex structural systems under alterning loads. Studies to investigate the response of these joints to random loadings might be included in this phase as well.

I-3. Acknowledgment

The work reported herein was conducted in the Structural Engineering Materials Laboratory, Division of Structural Engineering and Structural Mechanics, Department of Civil Engineering, University of California, Berkeley. The program was co-sponsored by several companies interested in off-shore structures. The United States Steel Corporation, Pittsburgh, Pennsylvania, donated the joint-specimens. The Standard Oil Company of California, San Francisco, acted as industry representative. The research grant was administered by the Office of Research Services, College of Engineering, University of California.

A number of persons made significant contributions to the work described in this report and the author wishes to thank them all. Particularly the contributions by Mr. O. Greste, Graduate Student, and Messrs. R. P. Englehard and P. F. Phaelzer, Undergraduate Students, are gratefully acknowledged.

II. PROGRAM OBJECTIVES - PHASE I

The principal objective of the first phase of this program was to determine, for five basic types of joints, the response of these connections under identical alternating loads until failure occurred. These studies, which were performed under maximum design stress levels in the diagonal web member, were designed to evaluate the relative performance of these joints as based upon the endurance limit, or number of load cycles prior to failure. It is anticipated that the results of this first phase together with the results of subsequent future studies will allow the development of recommendations concerning the design of tubular connections subjected to alternating loads.

In order to evaluate the relative structural value of a number of tubular connections, commonly used in off-shore tower design, the following basic joints, as shown in Figure 1, were selected and were included in Phase I:

Type 1: A joint without inerwelded branch members.

Type 2: A joint with interwelded branch members.

Type 3: A joint with an in-plane gusset plate; slotted branch members welded to a gusset plate which passes entirely through the chord member.

Type 4: A joint with four circular ring stiffeners around the chord member.

Type 5: A joint with the cast-steel seat; the branch members connected to a cast-steel seat welded to the chord tube.

Because the primary program objective was a comparative study of the endurance limit for the several joints, the connections were designed with identical member sections and tested under the same load conditions. Because of the two different web member arrangements encountered in off-shore tubular structures, two basically different joint-loading conditions were considered. Figure 2 shows a number of off-shore trusses. The joint types A and B, although similar in arrangement, will be loaded quite differently when the structure will be subjected to lateral loads. In case of a type A joint the vertical component of the diagonal tensile force will be reacted by a compressive force in the column member. For a type B joint however, the vertical component of the diagonal tensile force will be balanced by a tensile force in the column member. In the first case the general stress flow will be more concentrated than in the latter case. Also the effect of radial contraction of the loaded members seems to be more critical for joint type A with opposing member loads (tension in diagonal, compression in column) than for joint type B, where the same two members are subjected to the same forces. Therefore also the secondary stresses in a type A joint are probably comparatively more critical. Consequently, the type A loading has been selected for the joints tested in the first phase. It is anticipated to study a limited number of joints under a type B loading in phase III.

It should be noted that the alternating branch and chord-member forces are a result of the lateral loads acting on a true structure. The joints are normally designed such that the alternating forces produce maximum nominal stresses in the diagonal equal to the allowable design stresses. In addition to these alternating forces the column member is under sustained compression due to the dead load. Both these loadings were incorporated in all studies. Figure 3 shows the general loading agreement.

A second objective of this first-phase program was to evaluate the influence of the column wall thickness and the ring stiffeners on the endurance limit. Therefore joints Type 1 and 2 were studied with both thin and thick-walled column sections. For the same purpose joint Type 4 was investigated with a variation in the size and spacing of the ring stiffeners. These secondary objectives were only limited in scope and increased the total number of different joints studied under identical maximum alternating stress levels to eight.

The third and final objective was related to the test dependability and the influence of the alternating stress level on the number of cycles prior to failure. To evaluate the test dependability one additional joint Type 2, thin-walled - was duplicated and tested at the maximum stress level. Two other joints - Types 3 and 4 - identical to those studied under maximum alternating stress levels, were investigated at a nominal stress level of only 80% of maximum. The latter results served as an initial indication of the influence of the magnitude of the alternating stress-level on the endurance limit.

III. JOINT-SPECIMENS

III-1. General

The two-dimensional truss joint selected for these studies was formed by a column member and two identical web members. The angles between the web members and the column member were 35° and 90° respectively. A testing of full size joints with actual column members having an O.D. of 36 in. to 48 in. was considered impractical. Therefore the joints were designed with a 12 in. nominal column member. The actual 18 in. to 24 in. web members were reduced accordingly by the same scale factor. Since it could be expected that failure would develop due to stress concentrations resulting from structural arrangements it was felt that the reduction in size would not cause a scale effect on the test results. Therefore the following basic member sizes were selected for all specimens:

Column member: O.D. = 12-3/4 in., t = 0.330 in.

Web member: O.D. = 5-9/16 in., t = 0.1875 in.

Five basically different types of joint specimens were designed with these member sizes. Two specimens, Types 1 and 2 were also designed with a "thick-walled" column section in the immediate vicinity of the joint. The wall thickness was t = 0.562 in. A third joint, Type 4, was designed with a different ring stiffener arrangement and was designated as Type 4A. The detailed joint designs are presented in the following sections of this chapter.

The pipe material selected was seamless pipe API Spec. 5L - Grade B with a minimum yield point of 35 ksi. All identical pipe sections were taken from the same heat. All other steel was specified as ASTM A-36. The weld-size requirements between intersecting tubular sections are shown in Figure 4.

III-2. Joint Design

III-2-1. Joint Type 1 (Figure 5)

The Type 1 joint is a commonly used connection in which the incoming branch members are arranged without introducing an eccentric force on the column member. The separation of the two web members will undoubtedly result in a severe load transfer through the wall of the column section. This is particularly critical because of the radial flexibility of column member wall. To investigate the influence of the wall flexibility of the column section under these alternating load conditions, two joints with different wall thicknesses have been designed. The "thin-walled" joint specimen had a column member with a 12-3/4 in. O.D. and a wall thickness of 0.330 in. The "thick-walled" joint had a column member identical to the "thin-walled" joint; however, in the immediate joint area the thin-walled section was replaced by a 12-3/4 in. O.D. tube stub with a 0.562 in. wall thickness. See Figure 5. Due to this thick-walled element the local deformations and subsequent stresses of the column wall will undoubtedly be reduced as compared to those occurring in a thin-walled section under a similar loading.

III-2-2. Joint Type 2 (Figure 6)

In order to improve the load transfer between the web members joint Type 2 was designed with a negative eccentricity of 1/4 D. or 3-3/16 in. The resulting intersection between the two web members provides a direct load transfer between these two members and reduces the direct force of the horizontal web member on the wall of the column tube. Consequently the local stress concentrations in the column wall will be less severe than in case of a joint with non-intersecting web members. On the other hand the stress in the walls of the two branch members near the connecting weld can become quite critical. To reduce these stress concentrations to a limited degree the diagonal member is normally completely welded to the wall of the column members, while the horizontal web member is cut to fit over the diagonal tube. This arrangement was adopted for this joint specimen.

Joint Type 2 has been designed with the same two different wall thicknesses for the column member as selected for the design of the Type 1 joints. The "thin-walled" joint had a column-wall thickness of 0.330 in. while the "thick-walled" joint had locally a thickness of 0.562 in. The increased wall thickness in this case provides an increased radial stiffness of the column member and will draw more load towards the weld between the diagonal and column tubes. Consequently the weld between the two branch members will be less severely loaded.

III-2-3. Joint Type 3 (Figure 7)

The Type 3 joint was designed with an in-plane gusset plate in order to reduce the direct force effect on the tube wall of the column member. The web members were slotted and profiled, and welded to both the gusset plate and column member wall. Although the welds between web and column members were specified as non-strength welds their size was identical to those applied in other specimens. The 1/4-in. thick gusset plate penetrated the column wall along two diametrically opposite lines. A butt-weld was used in all connections between the tubular members and the gusset plate as shown in detail in Figure 7. This type of joint had the advantage of reducing the local deformations of the column member. Namely, firstly, the load transfer between the two web members takes place predominantly in the gusset plate rather than through the wall of the column member. Secondly, the completely penetrating gusset plate stiffens the column member and reduces the dangers of a low radial flexibility.

The major problem in this type of joint is the radial restraint the gusset plate imposes on the web members. Such a restraint causes severe bi-axial stress concentrations in the web member walls near the edge of the gusset plate. To reduce these concentrations of stress it is conceivable to provide tapered cutouts in the gusset plate to lower the stiffness of the plate in its own plane. Figure 8 shows a possible arrangement for such cutouts. In that case the gradually increasing stiffness of the gusset plate along the welds to the branch members will permit a more uniform transfer of the member forces into the gusset plate. Such a transfer will lower the high local longitudinal stresses in the wall of tube immediately in front of the edge of the gusset plate. Furthermore the reduced in-plane stiffness of the gusset plate will allow the web members to deform radially more freely. As a result the concentrations of radial stresses in the tube walls near the edge of the gusset plate will also be reduced.

III-2-4. Joint Type 4 (Figures 9 and 10)

The design of the Type 4 joint is often dictated by the need to maintain for one or more reasons a free interior column member. In order to stiffen the column member wall and to prevent severe radial stress concentrations, rings are used to distribute the web member forces over a larger portion of the column section. Areas of possible critical stress concentrations can be expected in the web members where the rings penetrate the tube wall. Also the column wall between the two branch members can be stressed severely.

Normally two sets of double rings are incorporated in these designs. In this program two joints with different ring-spacings and ring sizes were investigated. These specimens, identified as Type 4 and 4A, are shown in Figure 9. Joint details of the weldments between rings and tubes are given in Figure 10.

III-2-5. Joint Type 5 (Figure 11)

Joint Type 5 was a new type of joint with a cast-steel seat welded to the column member. The material was cast-steel ASTM 1 Spec. A-27, Class 70-36.

The branch members were welded directly to the seat stubs along a circular section. This type of joint has the advantages of simple welding and simple cuts for the branch members. The load transfer between the web members seem to be quite adequate without introducing any serious stress concentrations in the wall of the column members.

III-3. Specimen Design

The specimens were all designed to be mounted in a specially developed loading frame. To prevent the influence of the end connections on the response of the actual joint under alternating loads the distance from the joint to the end of each member was at least three times its diameter. Figure 12, representing joint Type 3, shows the typical overall dimensions of a specimen in which all member system lines intersect at one point (so-called zero joint-eccentricity). In Figure 13 the typical dimensions for a specimen with a so-called negative eccentricity joint (Type 2) are shown.

The tapered end plates of the branch members were selected in order to prevent the development of serious stress concentrations in the tube walls near the end connections. These connections had been used successfully before to develop under static loading the full strength of the diagonal member.

IV. TESTING PROCEDURE

IV-1. General

The tests were carried out in two steps. Each of the eight different specimens was first studied under static loads to determine the general stress distribution. Therefore a large number of electric-resistant strain gages with a 1/4 in. gage length were applied and recorded at step-wise increasing loads. At the same time the stress trajectories were determined by means of stress coat, a brittle coating. The second step involved an alternating loading of the same specimens. After removing both the stress coat and the strain gages (except a couple located at critical locations) each specimen was placed in a tank with a 3.5% NaCl salt-water solution. Intermittently during the alternating load tests a few gages were recorded continuously for a couple of minutes. The cycling speed during these tests was approximately 15 cycles per minute. The alternating loads were applied until failure of the joint was observed. The three duplicated specimens were subjected to alternating loads only.

IV-2. Loading

IV-2-1. General

In order to load the specimens according to the general force arrangement shown in Figure 3 a special loading frame was designed. This frame is shown schematically in Fig. 14. The actual frame with a mounted specimen and the tank are shown in Figure 15. Both zero and negative eccentricity joints were tested in this frame. For alternating load studies the specimens were placed in the specially designed tank and submerged in a 3.5% NaCl salt water solution. Although it was realized that the salinity of sea water varies as far as chemical composition, depth and geographic location are concerned, the solution used in these tests was considered acceptable as a mean condition.

IV-2-2. Dead load

The dead load stress of half the allowable normal stress, or 11 ksi, was to be introduced by a compressive axial load in the column members. This load should not be affected by the load application resulting from the lateral forces acting on the structure. Using four 1-in. diameter bolts to pre-compress the column member directly would indeed provide the pre-compression load. However, this force would change as soon as a load should be applied to one of the web members. It was thus necessary to introduce a flexible element between the end of the column tube and the compression plates of the bolt-arrangement. By using four double coil springs with an average spring constant of 11.7 k/in. each it was possible to introduce a pre-load of 140 k. in the column member while deforming the springs about 3 in. The spring coils were 16 in. high. The outside diameter was 6-7/8 in. and the coils 1-5/8 in. and 7/8 in. round. Figure 16 clearly shows the springs and prestressing bolts. Comparing the large spring deflection with the small values for the elongation of the rods (0.144 in.) and the shortening of the column members (0.0284 in.) under the same load it becomes obvious that a load subsequently applied to this joint will hardly change these initially introduced forces.

The pre-load of 140 k. was obtained by loading the column-tube and spring assembly in the 4,000,000 lb. universal testing machine up to a load of 148 k. The nuts of the 1 in. diameter bolts were subsequently tightened and the external load released. This procedure provided the require pre-load. The actual amount of the pre-load was checked by means of strain gages which had been placed on the four 94 in. long pre-stressing bolts, to calibrate the bolts prior to testing. A deviation of $\pm$ 5% from the intended pre-tension loads in each of these bolts was considered acceptable.

IV-2-3. Lateral load

The influence of the lateral loads was introduced by a double acting hydraulic cylinder with a maximum capacity of $\pm$ 80 k. This load cylinder acted against the horizontal branch member as shown in Figs. 14 and 16. For the static load tests the load was increased in steps of 5 k. to a maximum of 50 k. (compressions). For the alternating load tests the horizontal branch member was subjected to maximum loads of + 50 k. and - 50 k. The load in the diagonal under this condition was 87.2 k. The corresponding nominal stress in the same member was 27.8 ksi. Assuming a basic allowable stress of 22 ksi the permissible design stress under lateral load conditions would be 1.33 x 22 ksi = 29.3 ksi. The stress value of 27.8 ksi used in these tests was considered adequate for all practical considerations.

The alternating loading system was force controlled, by means of a combined signal from the eight strain gages located on the moving rod of the load cylinder and spaced at 90^{o} intervals. These gages were used to calibrate the hydraulic cylinder load in both compression and tension.

IV-3. Instrumentation

The instrumentation was designed to obtain information about the general stress distribution in the eight different types of joints when subjected to a step-wise increased static load.

Each of these joints was therefore instrumented with a considerable number of electric resistant strain gages. With the exception of joints 4 and 4A the maximum numbers of gage circuits for each joint was 96. Joint Types 4 and 4A were instrumented with approximately 125 gages. Figures 17 through 23 show the strain-gage instrumentation for all eight joints. The gages were concentrated on one side of the specimen. A limited number of gages were placed on the opposite side of the joint to check any out-of-plane eccentricity.

In addition to the strain-gages, stress coat was applied on the other side of the joint than where the gages were located. The objective of using this brittle coating was to obtain information about the stress trajectories only.

Photo-stress was used in an attempt to obtain a photoelastic record of the state of stress in the gusset plate between the branch members. A photo elastic plastic sheet was glued to the gusset plate on the strain-gaged side of the joint. The opposite side had been sprayed with stress coat.

For the alternating load studies, with the specimen submerged, all instrumentation was removed except two or three strain gages located in critically stressed areas. These water sealed gages were to provide an early indication of a progressive failure under alternating loads.

IV-4. Recording

The strain gages were recorded on a Gilmore multi-channel strain gage recorder at 5 k. intervals. The strain values due to the applied static compressive load were simultaneously printed and plotted. For each load step the 96 channel recorder scanned the strains of all gages and plotted the strain value for each gage on a separate chart. These charts are part of a continuous graph-paper loop. The strain values in each diagram are identified by a channel number and are plotted against the appropriate load.

The stress-coat cracks developed during loading or if necessary activated by cooling while under maximum load, were observed, retraced and photographed to provide a general pattern of cracks to indicate the compression stress trajectories. The stress-coat was not used with the intent to provide quantitative information about the strain values. The latter information together with the stresses were obtained from the strain-gage data.

The photo-stress was observed under step-wise increased static loads. The resulting frings were viewed through a so-called photostress large field meter, and subsequently photographed on colored film. The isoclinics were directly plotted on the surface of the plastic sheet.

During the alternating tests two or three strain gages were monitored intermittently on a Sanborn multi-channel recorder. This instrument provides a continuous record while activated. The strains were recorded for about 30 cycles every couple of hundred cycles. Observation of this record would normally show a change in the recorded strains when in the vicinity of these gages a failure would develop. In some instances this phenomenon could indeed clearly be observed.

V. RESULTS OF STATIC TESTS

V-1. General

The static tests were carried out until the compressive load applied to the horizontal branch member attained at least a value of 50 k. For some joints the load was increased beyond this load until the recorded maximum strain reached a value of approximately 2 times the elastic strain value. The nominal stress in the diagonal member for a load of 50 k was 27.8 ksi or approximately 1.33 times the standard allowable stress of 22 ksi. The applied force and the subsequent reactions were superimposed on the pre-compression load of the column member.

The strains caused by the pre-load of 140 k were recorded for the thin-walled Type 1 joint only. This connection was tested first and indicated that the stresses in the column wall near the branch members were slightly smaller than the nominal stress of P/A. The recorded strains in the branch members were very small as compared to those introduced by the subsequently applied cylinder load of 50 k. Hence the strains under this pre-load condition were not recorded for the remaining joints. Hence the strains and stresses presented in the following sections are a result of the lateral load only.

The stress distribution for the applied load of 50 k will be presented for each type of joint. These results are based on the elastic-strain - load ratios and determined from the recorded load-strain plots. Whenever necessary for an accurate evaluation of the elastic strains at 50 k., the load-strain plots were recorded for several load repetitions.

In addition to the stress information also the load-strain curves for a number of gages placed at critical locations will be presented for each joint.

The stress-coat pattern as traced and photographed will complete the information on the behavior of the different joints in the nominally elastic range.

The results of the photo-stress study for the gusset-plate joint Type 3 will also be presented.

V-2. Detailed Test Results - (Static Loads)

V-2-1. Joint Type 1 (Figures 5 and 17)

The stress distribution in the thin and thick-walled joints are presented in Figures 24 and 25 respectively. It is of interest to note that the stresses in the diagonal along the weld are markedly influenced by the thickness of the column wall. For the thin-walled joint the stresses in the wall of the diagonal near the 35^{o} and 145^{o} corners are smaller than in the side portions of the diagonal. (13.8 ksi and 19.3 ksi against 30.7 ksi and 26.7 ksi). This behavior is completely contrary to the one observed in the thick walled joint. In that case the stresses near the crotch are found to be considerably larger than those observed on the side of the diagonal; namely 27.1 ksi and 31.0 ksi against 19.4 ksi and 15.2 ksi. The maximum elastic stress concentration factor noted in either joint was approximately $31.0/f_{av} = 1.1$.

The recorded longitudinal strains in the diagonal associated with the above discussed stress values for the thin and thick-walled joints are presented in Figures 26 and 27 respectively. The strains in Figure 26a show a low strain rate for gages 10 and 23 located near the crotch as compared to the higher values for the other gages located on the diagonal member. This result indicates that the stress flow in this joint must have been diverted towards the side welds between diagonal and column member. Figure 26b supports this trend very strongly whereas gages 21, 26 but particularly gage 13 show very large strains. Also the pertinent stress values presented in Figure 24 give the same indication. The specific behavior can be explained because the stiffness of the column member wall parallel to the plane of the truss is considerably larger at the side than at the crown.

Considering the strains in the diagonal of the thick walled joint as presented in Figure 27 it becomes evident that, contrary to the results observed for the thin walled section, the maximum strains occur in the corners rather than in the side portions of the wall of the diagonal member. This seems to indicate that the effective stiffness of the thick-walled column section parallel to the plane of the truss is different from that observed for the thin-walled section. As a result the stresses are concentrated near the corner areas rather than near the side welds.

In general it can be noted that the recorded longitudinal strain values at 50 k, as shown in Figures 26 and 27 agree very well with the nominal longitudinal strain of approximately 960 μ in/in. (Assuming E = 29,000,000 psi).

This nominal strain is based on the assumption that the diagonal is radially unrestrained. In reality the weld between diagonal and column sections restrains the branch tube and changes the nominal strain values. Also the bending stiffness of the weld and the adjacent column wall will effect the stresses in the wall of the diagonal tube when subjected to axial forces. This additional restraint will undoubtedly be larger in a joint with a thick-walled column member than in a joint with a thin-walled section. This effect might have caused the basically different stress distributions in the diagonals of the thin and thick-walled joints.

While for both joints the stress concentration in the diagonal member was approximately 1.1, the strain-concentration factor at 50 k. was close to 3.0. For the thin-walled joint this factor resulted from the recorded side-wall strains while for the thick-walled joint the severe strain concentration occurred in the crotch areas of the wall of the diagonal.

Figure 28 shows for both joints the load-strain curves for the longitudinal gages on the horizontal member and the strains in the most highly stressed gages on the wall of the column tube. The strains in the horizontal member appear not to be very severe. The stresses in these members show the same trend as noted before for the diagonal, namely, higher stresses on the side wall of the member for the thin-walled joint than for the thick-walled connection. However, in both joints the maximum strains occur in the wall closest to the diagonal (gage 36). These high strains and associated stresses in that location are due to the concentrated load transfer between the branch members in that vicinity.

A comparison between both the stresses and strains in the column wall near the weld between the horizontal and column members shows the extremely favorable influence of the thicker wall. The critical column-wall area near the toe of the side wall of the horizontal member (gage 42) was most effectively improved by the thicker wall. (from -29.1 ksi to +4.4 ksi). The strains show a similar improvement (from -740 μ in/in to +20 μ in/in).

For both the thin and thick-walled specimens stress coat was used to evaluate the general stress directions. The location of the isotropic stress point on the wall of the column member 3 to 4 in. from the weld between column and horizontal branch member clearly indicated for the thin-walled joint the weakness of this column section. This was contrary to the results obtained for the thick-walled joint as shown in Figure 29 where the isotropic point on the wall was found to be located close to the center line of the column member. This result is similar to the pattern which was found in earlier studies of thin-walled joints with intersecting branch members.[1] It can be expected that under static loads a joint with a thicker wall and non-intersecting branch members will behave similar to a joint with a thin-walled column but intersecting branch members.

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1. Bouwkamp, Jack G., "Research on Tubular Connections in Structural Work," Bulletin No. 71, Welding Research Council, August 1961.

V-2-2. Joint Type 2 (Figures 6 and 18-19)

Figures 30 and 31 show the principal stresses and directions under applied compressive load of 50 k. for the thin and thick-walled specimens respectively. The stresses in the diagonal near the welds indicate that for the thick-walled joint the load transfer through the side weld is more concentrated than for the thin-walled connection. (stresses of 31.9, 31.3 and 33.1 ksi against 33.0, 22.2, 22.9 and 10.9 ksi). At the same time the high stresses in the corner areas of the diagonal member of the thin-walled connection were reduced considerably by the influence of a thicker column wall. (stresses of 35.8 and 21.6 ksi against 27.8 and 15.6 ksi respectively.) Although this behavior is contrary to the results observed in the Type 1 joints it seems quite reasonable that a thicker column wall, resulting in a radially stiffer column section, will attract a larger portion of the load transfer and thus reduces the concentrated transfer between the intersecting branch members.

Another important difference between the stress results for the Type 2 joints as compared with those for the Type 1 joints can be noted for the horizontal branch members. According to Figures 31 and 32 the stresses in the horizontal tube near the corners are relatively close, namely 17.5 and 12.9 ksi and 17.8 and 11.2 ksi for the thin and thick-walled specimens respectively. However, for the Type 1 joints the comparative stresses were 39.0 and 6.3 ksi and 27.8 and 5.4 ksi respectively.

Another significant improvement of the state of stress in the Type 2 joint as compared to the Type 1 joint was noted in the wall of the column member. For the thin-walled joints the stresses in the column wall near the horizontal member were very much lower for the Type 2 joint than for the Type 1 joint, namely 8.5 and 10.5 ksi against 29.1 and 37.4 ksi respectively. A similar result, although not so pronounced, was observed in the column wall along the weld between the diagonal and column members. For the thick-walled joints the stresses for Type 2 are in general also basically smaller than observed in Type 1.

The above discussed improvements are a result of the interwelding between the branch members. This arrangement allows a direct transfer between the web members and consequently reduces the transfer through the wall of the column member. This effect will of course be more pronounced for thin than for thick-walled joints. In the latter case a thicker wall will draw, despite the interwelded web members, an appreciable amount of the total branch member forces to the wall of the column member.

The load-strain plots for a number of selected gages located on the wall of the diagonal are presented in Figures 32 and 33. In general it can be noted that the strains are relatively uniform. For the thin-walled joint the maximum strain seems to occur in the diagonal near the corner between the two branch members (gage 31 in Figure 32). For the thick-walled joint the maximum strain was observed in the wall of the diagonal near the crotch between the column and the diagonal (gage 1 Figure 33). It is in agreement with the earlier observations of force flow towards a thicker column that the strain-rate of gage 31 on the thin-walled joint is larger than the strain-rate of gage 12 on the thick-walled joint. In the latter case gage 12 remained even fairly elastic up to the maximum applied load of 75 k. This is contrary to the behavior of gage 31, which started to yield significantly after the applied load reached 50 k. Contrary to expectations however, the elastic strain-rate for gage 1 located on the diagonal near the corner between the diagonal and column member showed a higher value for the thin-walled joint than for the thick-walled joint. The recorded strains for gage 1 on the thin-walled joint remained practically elastic over about 70 k. However the same gage for the thick-walled joint showed a progressive yielding after reaching a load of 50 k. (Figure 33), which might indicate a large force draw towards this thick-walled section after all.

In general it should be noted that based on the available strain data the maximum stress concentration factor in the Type 2 joints ranged between 1.2 and 1.3, which is slightly larger than observed for the Type 1 joints. The maximum strain concentration factor at 50 k. was about 1.2 (gage 1, thin-walled joint) and (gage 38, thick-walled joint). The latter result indicates that under the design load of 50 k. the maximum strains in the Type 2 joints are only 40% or less of those recorded in the Type 1 joints. (maximum strain concentration factor of 3.0.)

A study of the load strain curves for gages located on the horizontal branch member and the column tube of the thin-walled Type 2 joint indicated that no significant yielding did occur during the static test. The highest recorded strains in the horizontal web tube at 60 k. were -1100 and -1000 μ in/in. The highest strains recorded on the thin-walled column section were noted in gages 74 (-900 μ in/in), and 71 (+650 μ in/in). The maximum recorded strain in gage 75 was +250 μ in/in. All other gages recorded strains at 60 k. of less than 300 μ in/in.

The strain data for gages located in the horizontal branch tube and column wall of the thick-walled Type 2 joint indicated high strains directed towards the weld between the two branch tubes. The strains which were recorded up to a load of 75 k. all indicate a substantially linear load-strain relation. The highest strains at 75 k. were recorded for gages 83 and 85 with strains of -1200 and -900 μ in/in. The strains in gages 77, 78 and 82 reached a maximum value of approximately -500 μ in/in. The strains for all other gages, even along the weld between the horizontal web member and the column, were less than 350 μ in/in. The maximum strains in the thick-walled column wall were observed in the corner between column and diagonal. Gages 42 and 43 recorded strains of -580 and +480 μ in/in respectively. All other gages recorded strains of less than 300 μ in/in. The general trend observed in both specimens was a gradual change of magnitude in the radial strains of the column wall from a positive strain near the corner between diagonal and column to a negative strain near the corner between the horizontal member and column tube. For the thick-walled joint for instance, gages 43 and 59 ranged from +480 μ in/in to -280 μ in/in.

The results of the stress-coat study on both joints are presented in Figures 34 and 35. The practically identical crack patterns clearly show the effectiveness of the interwelding between the branch members and, contrary to the results of the Type 1 joints, the seemingly negligible influence of the thickness of the column member.

V-2-3. Joint Type 3 (Figures 7 and 20)

The stress distribution in the gusset plate joint for an applied load of 50 k. is shown in Figure 36. Since the primary load transfer in this type of joint occurs between the web members and the gusset plate the instrumentation was concentrated in a number of sections on the walls of the web members. From the calculated stresses shown in this figure it is obvious that the severely stressed area in the diagonal occurred near the tip of the gusset plate (area A). In the horizontal member the critical stresses were observed in the wall of this tube on the side of the diagonal near the intersection with the gusset plate (area C). The high stresses in area A are due to the concentrated load transfer directed towards the extending gusset plate. As a result, the transfer along the other weld between diagonal and gusset plate will be less severe. The noted stress-concentration in area C is clearly a result of the concentrated force flow between the two branch members. Since the gusset plate penetrates the horizontal member at two diametrically opposite points, there is no attraction of force due to a gusset-plate extension as in the case of the diagonal. However, on the other hand, the force flow between the horizontal member and the gusset plate will be directed towards the diagonal. Hence area C rather than area D will be severely stressed.

It is particularly interesting to study the load transfer between the branch members and the gusset plate in more detail. The strain gage instrumentation was concentrated along three sections on the diagonal and two sections on the horizontal. Figure 37 shows the exact position of these gage lines. The calculated longitudinal stresses in each of these sections are presented in Figures 38 and 39. The strains from which these stresses are calculated were of course recorded on the outside of the tubes only, and might have been influenced by local bending of the tube walls. Nevertheless, the average stress of 26.8 ksi in Section 1 agrees very well with the nominal average stress of 27.8 ksi. For Sections 2 and 3 the average stresses are approximately 21 and 18 ksi. Assuming that these stress values represent the membrane stresses at these sections one could conclude that the member force had been reduced rapidly to only about 80 and 65% respectively. Hence the weld of 3.2 in., between Sections 1 and 2 transferred approximately 20% of the member load or 17 k. into the gusset plate. Through the 3.8 in long weld, between Sections 2 and 3, an additional load of 13 k. seems to have been transferred. This would indicate that 35% of the total member load, or 30 k., has been transferred by about the first 7 in. of the lower welds between the diagonal and the gusset plate. The remaining 65% seemed thus to be transferred through the rest of that weld together with the upper welds between gusset and diagonal and the welds between diagonal and column tube.

The stress distributions in Sections 4 and 5 are presented in Figure 39. The average calculated stress of 15.6 ksi in Section 4 was in excellent agreement with nominal stress of 15.9 ksi. The average stress in Section 5 was found to be about 13 ksi., indicating a reduction in force of about 15%. This result indicates that 15% of the member force must have been transferred from the tube wall to the gusset plate through a weld length of only 0.8 in. Considering the stress intensity in the areas C and D as shown in Figure 36 one could conclude that about 80% of the transferred load, or 12% of the member force, (or 6 k) must have been transferred by the lower welds near area C. The balance of about 3%, or 1.5 k. was transferred through the welds near area D.

Comparing the load transfer in the two branch members it is obvious that the average shear force of 6 k. over 0.8 in. in the lower welds of the horizontal member on either side of the gusset plate is more severe than the average shear force of 17 k. over 3.0 in. in the two lower welds of the diagonal. Assuming for both branch members a uniform shear transfer along the weld lengths considered, the shear concentration factors for the 3.0 in. long welds of the diagonal and the two 0.8 in. long welds of the horizontal member can be evaluated as 2.8 and 3.7 respectively.

The recorded longitudinal strains in the diagonal member are presented in Figures 40 and 41. The longitudinal strains in Section 1, Figure 40a, clearly shows the severe yield strains in the wall area A near the tip of the gusset plate. (gages 79 and 80). The other gages in that same section indicated practically a completely elastic behavior. The results for Section 2, as shown in Figure 40b, indicate a considerable reduction in strain along the lower weld (gage 69 versus gage 79). The other strains in that section were quite similar to those observed in Section 1 away from the critical area A. The longitudinal strain in Section 3 near the lower weld was further reduced (gage 66), while the strain in gage 54 located diametrically opposite that point reached a maximum for this section (see Figure 40c). After a load of 30 k. was applied the behavior of gage 54 became inelastic. However, the maximum strains recorded in area B for a 50 k. load applied to the horizontal member were far less than those observed in area A, namely 1600 versus 5800 μ in/in.

Figure 41 shows the strains for a number of gages in line with and along the upper weld between diagonal and gusset plate. It is of interest to note the increase of the strains while approaching the point where the gusset plate penetrates the tube wall. (gages 90, 76 and 54 successively). Also the sudden reduction in strain along the weld near the edge of the gusset plate (gage 54 versus 52), followed by the gradually successive decrease of the strains along the weld (gages 52, 49 and 43) is an important aspect of gusset plate joints. The same was found for the strains along the lower weld. This behavior indicates a high concentration of shear through the first part of the welds between branch tubes and gusset plates.

The longitudinal strains in Section 4 and 5 of the horizontal branch tube are presented in Figure 42. The concentrated force flow in the wall area

C is clearly illustrated by the high strains recorded in Section 4 for gages 18 and 17 (Figure 42a). Remarkably enough the behavior of these gages was elastic but non-linear. This would indicate a second order behavior of combined axial load and local bi-axial bending. The same can be noted for the gages on the opposite side (area D). The strain record for the gages in Section 5 showed a similar but less pronounced elastic non-linear behavior. In comparing the strains recorded in Section 4 and 5 one can note the considerable reduction of strain along the lower weld and the almost identical values along the upper weld.

Photo-stress sheets were applied on one side of the two exposed portions of the gusset plate. The isoclinics and isochromatics were recorded under several loads. The results of these observations are presented in Figure 43. The isochromatics indicated a maximum strain of about 1000 μ in/in., or a stress of about 30 ksi. The critically stressed areas of course were located near the points of penetration along the free edge of the gusset plate. The experience gained during this particular study indicated that the photo-stress technique is not well suited for this type of investigation. The information density is rather limited. Hence the results of the strain data are not very reliable.

The stress-coat study showed excellent agreement with the generally observed behavior of this joint. The traced crack-pattern as shown in Figure 44 agreed very well with the stress trajectories constructed from the photo-stress information. The limited crack pattern in the column section seems to indicate that the load transfer between the two branch members has been confined predominantly to the gusset plate.

V-2-4. Joint Types 4 and 4A (Figures 9 and 21-22)

The principal stresses for the joints 4 and 4A due to applied load of 50 k. are presented in Figures 45 and 46. Because both the ring-stiffness and the ring spacing had an effect on the load transfer in these joints it is difficult to determine the influence of these parameters accurately. However, the stresses derived from the measured strains in the different ring stiffeners, as shown in Figure 47, seem to indicate a definite trend. Considering the fact that the section modulus of each ring stiffener in joint 4 is about 80% larger than the section modulus of a joint 4A stiffener, one can expect that for the same ring load the stresses in a ring of the 4A type joint would be about 80% larger than in the ring of a Type 4. joint. However, the results presented in Figure 47 indicate that the stresses in the ring stiffeners of the joint Type 4A were only 20-30% larger than those evaluated for the stiffeners of the joint Type 4. This indicates that the ring loads in the Type 4 joint must have been about 40% larger than those in the rings of the joint Type 4A. This seems to be logical since stiffeners spaced further apart (Type 4) will carry more of the load in the outside wall of the branch members than in case of closer spaced stiffeners (Type 4A). Considering the average stresses derived from the strains in the tube walls near the tip of each stiffener it seems that these stress values for both the diagonal and horizontal web members in the Type 4 joint are larger than for the Type 4A joint. Also the highest strains were measured in Type 4 rather than in Type 4A. (gages 112 and 104 versus 30 and 93.)

It is realized that the above discussion neglects the tangential ring stresses due to the axial-force in the rings. However, considering the low section modulus for points along the outside perimeter of these rings and the continuous shear transfer between rings and column tube wall, one can conclude that the stresses due to axial load effects are small compared to the bending stresses.

An observation of the stresses in the wall of the column tube clearly shows the shear action developed in these joints between the two sets of ring stiffeners.

A study of the strains recorded for all gages indicated that nowhere in this joint any significant yielding did occur. The strain-load curves were practically all completely linear. The Tables I through IV give the strain values for a number of important gages located on the web members of both specimens at subsequent loads of 40 k., zero and 50 k. These strains show the recorded linear response of these joints.

The stress-coat data for joint 4 is shown in Figure 48. The 45° orientation of the traced cracks on the center part of the column wall illustrate the earlier discussed shear behavior of this part of the tube wall. The results for joint Type 4A were similar. It is of interest to note on the ring stiffeners the stress trajectories and the locations of the isotropic points near the free edge.

Table I. Longitudinal Strains In Diagonal: Joint Type 4

Gage No.	Pos. Strain Micro in/in		
	40 k	0 k	50 k
88	770	70	930
86	685	60	860
83	740	60	945
76	650	30	830
72	730	10	880
70	960	200	1430
74	950	105	1240
81	825	60	1040
91	620	65	770
108	810	70	980
104	1200	20	1500
102	880	130	110
97	760	90	980

Table II. Longitudinal Strains in Horizontal Tube: Joint Type 4

Gage No.	Neg. Strain Micro in/in		
	40 k	0 k	50 k
123	80	0	105
122	475	40	610
117	190	30	260
116	1070	200	1460
114	520	120	635
125	85	0	110
119	190	10	230
112	1100	150	1340

Table III. Major Strains in Diagonal: Joint Type 4A

Gage No.	Pos. Strain Micro in/in		
	40 k	0 k	50 k
96	710	0	865
98	580	0	760
101	580	15	780
111	405	10	570
112	640	20	800
108	560	10	790
106	400	20	520
103	580	20	800
85	475	10	635
93	885	20	1100
90	660	20	900
80	545	0	735

Table IV. Longitudinal Strains in Horizontal Tube: Joint Type 4A

Gage No.	Neg. Strain Micro in/in		
	40 k	0 k	50 k
15	90	20	100
17	400	20	530
24	250	5	350
27	500	5	700
28	440	5	555
12	80	5	85
123	120	15	170
30	915	20	1145

V-2-5. Joint Type 5 (Figures 11 and 23)

The longitudinal and radial stresses for joint Type 5 as calculated from the recorded strains are presented in Figure 49. The longitudinal stresses in the branch members differ considerably from the nominal stress values of 27.8 ksi and 15.9 ksi for the diagonal and horizontal members respectively. The larger actual stresses are a result of a secondary bending of the tube walls near the welds between seat and web members. This behavior is due to the combined influence of an insufficient weld penetration and a misalignment (over the wall thickness) of tube and seat walls. Even small discrepancies of this nature cause considerable bending strains. The force flow in the horizontal member is normally concentrated towards the side of the diagonal. However, the observed strains seem to imply a severe misalignment as well. The calculated stresses indicate a stress concentration factor in the horizontal member of about 3.0.

The stress condition in the diagonal member was quite uniform and indicated a stress concentration factor of only about 1.5.

The load-strain curves for the gages near the welds between web members and seat are presented in Figure 50. The largest strains occurred in the diagonal member, where, after reaching a load of 20 k. the behavior became non-elastic. The maximum strain concentration factor in the diagonal at 50 k. was about 4.0. A similar result was observed from the strains recorded by gage 28 located on the horizontal web member.

Figure 51 shows the traced pattern obtained from the stress coat study. The results show exceptionally clearly the stress-trajectory pattern for the entire joint.

VI. RESULTS OF DYNAMIC LOADING

VI-1. General

The influence of the alternating lateral loads on the offshore structure was represented by the double acting hydraulic cylinder with a maximum capacity of $\pm$ 80 k. For the alternating load tests the horizontal branch member was subjected to maximum loads of + 50 k. and - 50 k. The load in the diagonal under this condition was 87.2 k. The corresponding nominal stress in the same member was 27.8 ksi.

The alternating loading system was force controlled, by means of a combined signal from the eight strain gages located on the moving rod of the load cylinder and spaced at 90° ingervals. These gages had been used to calibrate the hydraulic cylinder load in both compression and tension.

For the alternating load studies the specimens were placed in a specially designed tank and submerged in a 3.5% NaCl salt water solution.

A couple of gages located in critically stressed areas were observed during these alternating load studies in order to provide an early indication of a progressive failure. In case failure progressed very slowly the specimens were subjected to a subsequent number of cycles while maintaining the same alternating load. This subsequent behavior provided an indication of the life-expectancy of these joints after initial failure had developed.

VI-2. Detailed Test Results - Dynamic Loads

VI-2-1. Joint Type 1

The thin-walled Type 1 joint, which was the first specimen to be studied was pre-tested in air before it was to be submerged. However, already after 60 cycles, under an alternating load of 50 k., a 2-in. long crack was noted in one side of the column wall near the deepest point of the intersection between the horizontal branch member and the column section. This would indicate that failure must have been initiated within the first 10 to 30 cycles. Figure 52 shows the failed specimen after 450 cycles. During this process the alternating load was maintained at 50 k. The initial crack in the column wall along the weld between the horizontal branch member and the column had gradually propagated along about 60% of this weld and concentrated on one side of the joint as can be noted in Figure 52. The same figure also shows a tearing-away crack towards the right. Following the 450 load reversals an additional crack (hair-line) was noticed in the column wall near the toe of the weld between the diagonal and column members immediately adjacent to the other branch member. The extent of this crack is indicated by the arrow in the left corner of Figure 52. This crack was symmetrical with respect to the plane of the truss.

The thick-walled joint (Type 1) was submerged in the salt-water solution and subjected to the standard 50 k. load reversal. Without any prior indication of a local failure the joint failed suddenly after 9900 cycles. Collapse resulted from a complete instantaneous fracture of the wall of the horizontal branch member immediately above the weld between this member and the thick-walled column section. Inspection of the fractured tube wall seemed to indicate that the overall failure was not introduced by a local fatigue failure. Figure 53 shows the fractured connection. The weld deposit, noticeable on the inside of the tube, must have resulted from too much exterior weld penetration.

VI-2-2. Joint Type 2

The thin-walled joint developed after 4000 load cycles a rapidly increasing fatigue crack in the diagonal tube near the weld between the two branch members. Figure 54 shows the extent of this crack. A subsequently tested duplicate specimen developed after 3000 cycles failure in the same location, but on both the near and far sides of the joint.

The thick-walled joint performed considerably better than the thin-walled. After 7000 cylces a crack was noticed in the wall of the diagonal tube along the lower portion of the weld between the two branch members. Figure 55 shows the resulting crack length after 7500 cycles. The center arrow indicates the location of the initial crack while the two outer arrows indicate the extent of the crack propagation after 7500 cycles. Although half the length of the weld between these two web members had failed the identical area on the far side of the joint did not undergo any failure.

The maximum recorded strains in both specimens were about 1600 μ in/in, and occurred in the wall of the diagonal.

VI-2-3. Joint Type 3

The gusset plate joint developed a fatigue crack in the wall of the horizontal branch tube after 4000 cycles. This crack originated above the weld between the gusset plate and the horizontal member and was located on the side of the diagonal member. Figure 56 shows the extent of the crack propagation after 7000 cycles (length of crack 3 in.).

It is of interest to note that despite an observed maximum longitudinal strain of about 5800 μ in/in. in the diagonal, failure occurred in the horizontal member. The recorded maximum strains in the vicinity of the cracked zone was only about 1900 μ in/in. (under a 50 k. static load). Reason for the observed failure seems to have been a large radial strain in this location in addition to the noted longitudinal strain. The radial strains recorded in the diagonal were relatively small compared to those in the horizontal member.

A duplicate Type 3 specimen was subsequently subject to an alternating load of 40 k. In that instance initial failure occurred only after 6500 cycles. The type of failure was identical.

VI-2-4. Joint Types 4 and 4A

The Type 4 joint, with the widely spaced rings, seemed to have experienced a crack initiation after about 1000 cycles. This crack occurred in the wall of the column tube at the toe of the weld between the horizontal and column member on the side of the diagonal. At subsequent inspections of the specimen, after additional cycles of load were applied, changes in the crack could not be observed. There was no evidence of a crack over the full wall thickness as could have been concluded from water coming through the crack after the specimen had been lifted out of the salt-solution. At 4500 cycles however, a very distinct crack was noticed and water came through this crack from the inside of the tube after lifting the specimen out of the tank. Figure 57 shows the location and extent of this failure after about 5500 cycles.

This failure indicated that the shear deformation of the column wall between the two relatively rigid sets of double rings caused a local bending in the column tube wall between the two web members.

The observed strains during the static load tests did not surpass 800 μ in/in.

The Type 4A specimen with the more closely spaced rings experienced after 7720 cycles a crack in the horizontal web member above the toe of the weld between that member and the column tube. Figure 58 shows the extent of this crack after 7720 cycles. This crack was partly covered with a water proofing epoxy for a strain gage located quite close to the crack. The early development of this failure could therefore not have been noticed by visual inspection. The strain-gage data which were obtained at intervals during the test indicated a distinct change in strain distribution after about 1500 cycles. It seems therefore logical to conclude that the early development of the crack started already at about 1500 cycles. After 7720 cylces the crack had propagated over a length of 2.5 in.

The development of the crack in the wall of the branch member rather than in the wall of the column section can be explained by considering the influence of the ring spacing and ring stiffness. The closer spaced rings in joint 4A permit a more concentrated load transfer towards the ultimately cracked area than would be the case for joint 4 where the wider spaced rings take up more of the load. The latter trend is further increased by the larger ring stiffness in joint 4 as compared to joint 4A. Finally the stiffer rings of joint 4 versus those in joint 4A limit the development of deflections over a larger range of the column tube wall. This restriction in flexibility makes the column wall relatively weaker and susceptible to failure at a lower number of cycles (1000 versus 1500).

The strains observed in the 4A joint were also nowhere larger than 800 μ in/in.

Following the standard tests a duplicate Type 4 joint was studied under alternating loads of 40 k. Failure at two locations in the wall of the horizontal branch member developed after 10200 cycles. One crack occurred in the wall immediately above the weld between this section and the column member on the side facing the diagonal. (Similarly as observed for joint Type 4A). The other crack was noted in the wall of the horizontal member above the point where the ring stiffener nearest to the diagonal penetrated this tube wall.

VI-2-5. Joint Type 5

The joint Type 5 failed after 1350 cycles of 50 k. alternating loads. Failure occurred almost instantaneously through a fracture of the weld between the diagonal and the cast-steel seat. The failure indicated a slight misalignment of the diagonal member and the seat as well as an insufficient penetration of the weld. These factors must have contributed significantly to the observed early collapse.

VII. CONCLUSIONS

VII-1. Static Load Studies

Static load studies on a number of typical joints as used in offshore structures and shown in Figure 1 provided information regarding the efficiency of tubular joint designs.

1. Joints in which the web members are not interconnected exhibit a severe stress distribution in the column member wall if this wall thickness is less than 2.5% of the column outer diameter. Early failure under alternating loads can be expected in that case. Increasing the column wall thickness to 4.5% D improves the general stress pattern and removes the critically stressed areas from the column wall to the web members.

2. For joints with intersecting web member the critically stressed area is transferred from the column section to the crotch area between the two branch members. This behavior is independent of the column wall thickness, but is more severe for joints with thin-walled column sections than for thick-walled sections. In case of a thicker column wall the above crotch area is namely released because more force is drawn to the weld between diagonal and column section. This will be particularly effective for joints under alternating loads.

3. For the gusset plated joints the critically stressed areas occur in the web member sections near the points where the gusset plate penetrates the walls of the web tubes. Particularly the radial restraint

imposed by the gusset plate is critical when the restraining occurs at diametrically opposite points as, for instance, for the horizontal branch member. Failure can be expected in that wall on the side of the diagonal. The only possibility to prevent such a failure is to develop a gradual load transfer between web members and gusset plate. This can be achieved by gradually scallopping the gusset plate. In that instance both the axial and in-plane stiffness of the gusset plate towards the center of the joint will increase gradually along the welds between the branch members and the gusset plate. This configuration, with a taper along each weld of 1 in. over 2 in. length will reduce effectively the biaxial concentration of both the longitudinal and radial stresses in the web-tube walls.

4. Ring-stiffened joints improve the radial stiffness of the column wall most effectively. The critical area however seems to be located either near the intersection of the ring stiffeners and web members or in the column wall directly between the two incoming branch members. Although it does not seem to be possible to reduce the stress concentration near the ring stiffeners, the concentration in the column wall can be reduced by increasing the wall thickness of the column member, or by applying an in-plane gusset plate section between incoming branch members.

5. Joints with joint-seats will exhibit critical stresses near the welds between web members and joint-seat. Proper weld penetration and alignment of connecting parts is essential for a relatively effective response of this type of joint.

VII-2. Alternating Load Studies

A study of the endurance limit of tubular connections as encountered in offshore well-drilling structures subjected to alternating loads indicated the severe nature of such loads for the type of connections shown in Figure 1. Under an alternating load of +50k. and -50k., resulting in nominal stresses of about 28 ksi and 16 ksi in the web members, the minimum number of cycles prior to failure was not more than 10,000.

1a. Joints with a column-wall thickness of less than 2.5% of the outer diameter of the column member and without an intersection between the branch members will fail after less than 100 cycles when subjected to an alternating load causing a nominal stress level in the diagonal equal to the permissible design stress.

1b. Intersecting the branch members or using an in-plane gusset plate to transfer the member forces will increase the endurance limit for the same loading to not more than 5000 cycles.

1c. Joints in which ring-stiffeners or joint seats are used will probably fail before reaching 2000 cycles of load reversal.

2. Increasing the radial stiffness of the column wall seem to be most effective, provided that stress concentrations can be limited. Increasing the wall thickness from 2.5% O.D. to 4.5% O.D. raised the endurance limit from less than 100 cycles to 10,000 cycles for a joint without intersecting branch members. With intersecting branch members and an increased column-wall thickness, the joint failed at about 7000 cycles.

3. In general it can be noted that large bi-axial strains are more critical in developing failure under alternating loads than very large uni-axial strains. Also the endurance limit does not seem to increase substantially under alternating loads equivalent to 80% of the design stress. This is probably due to the severe stress concentrations in these joints even under these reduced loads.

4. Because of the high stress and strain concentrations in these joints the influence of corrosion fatigue on the results can be considered negligible or even non-existent.

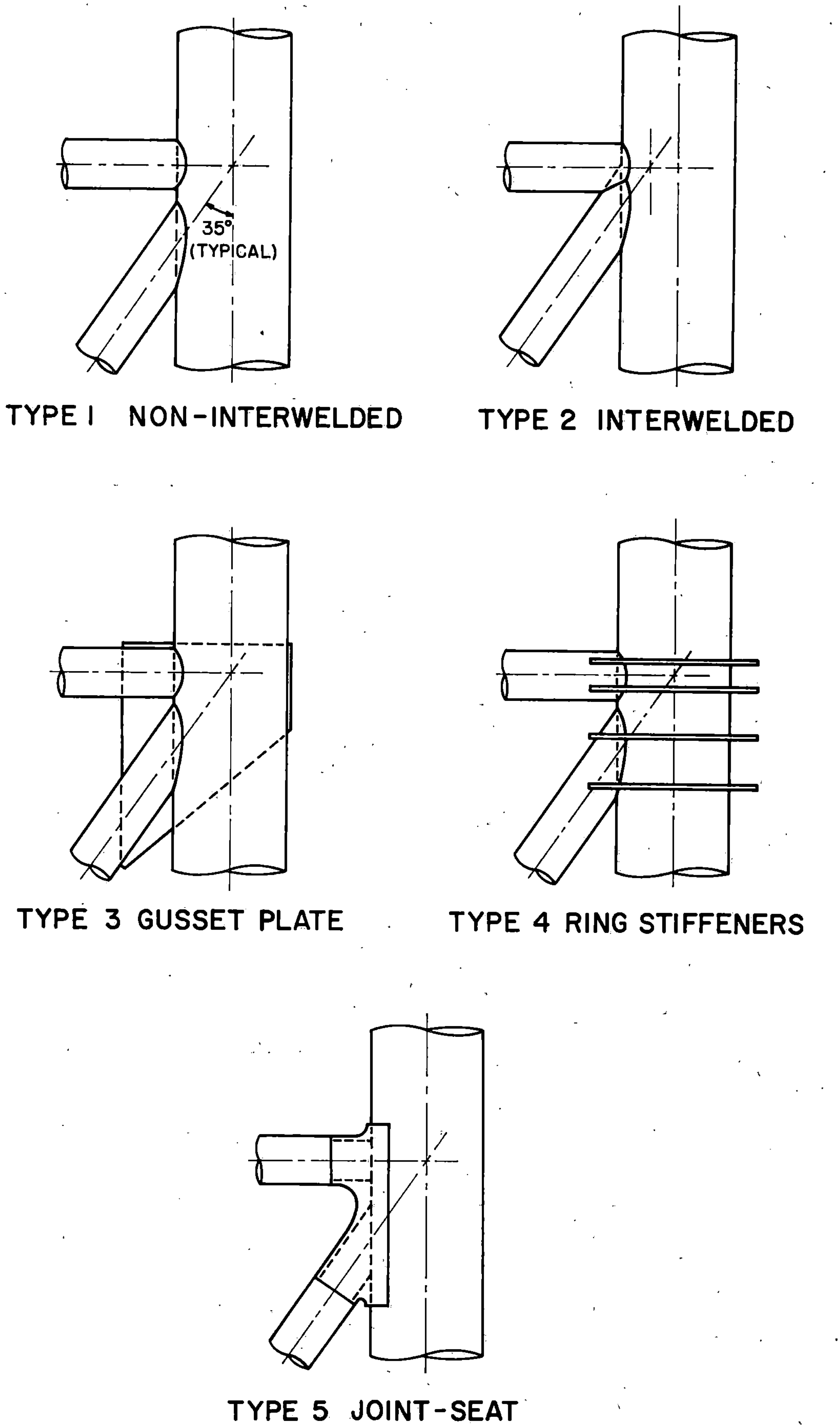
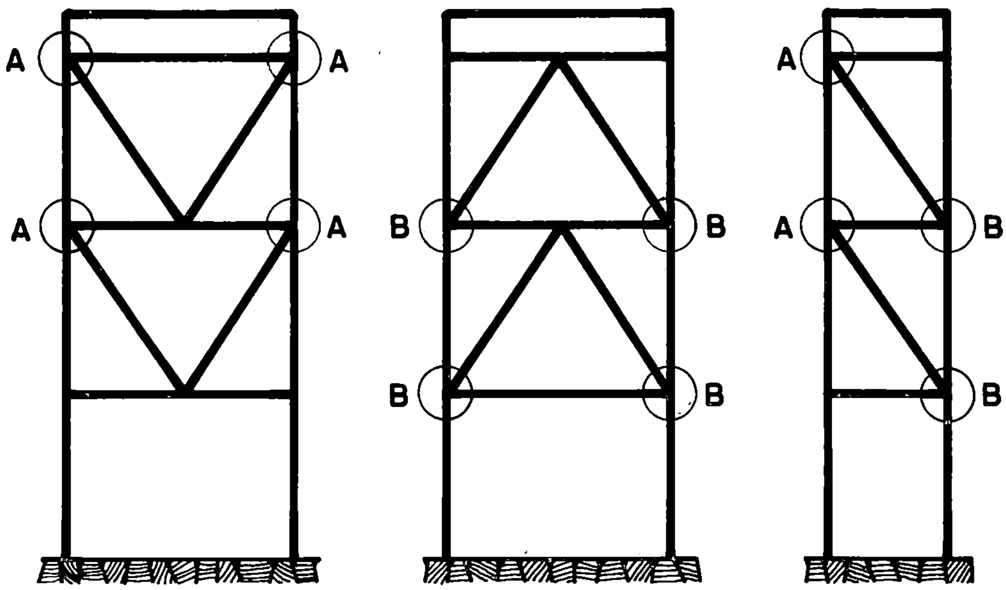
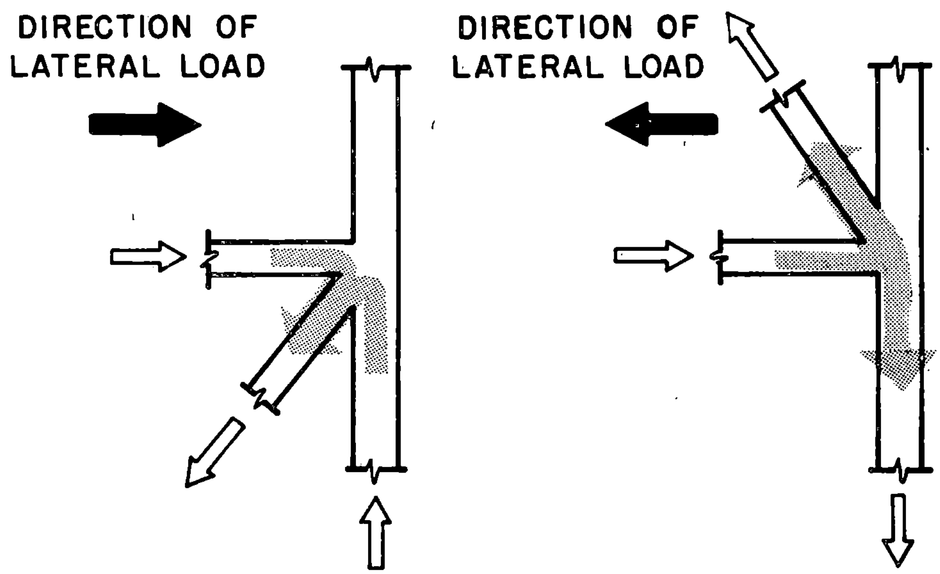


FIG. 1 BASIC TYPES OF TUBULAR JOINTS



OFF-SHORE TRUSSES



FORCE FLOW
IN TYPE A

FORCE FLOW
IN TYPE B

FIG. 2 FORCE FLOW IN TRUSS-JOINTS
DUE TO LATERAL LOADS

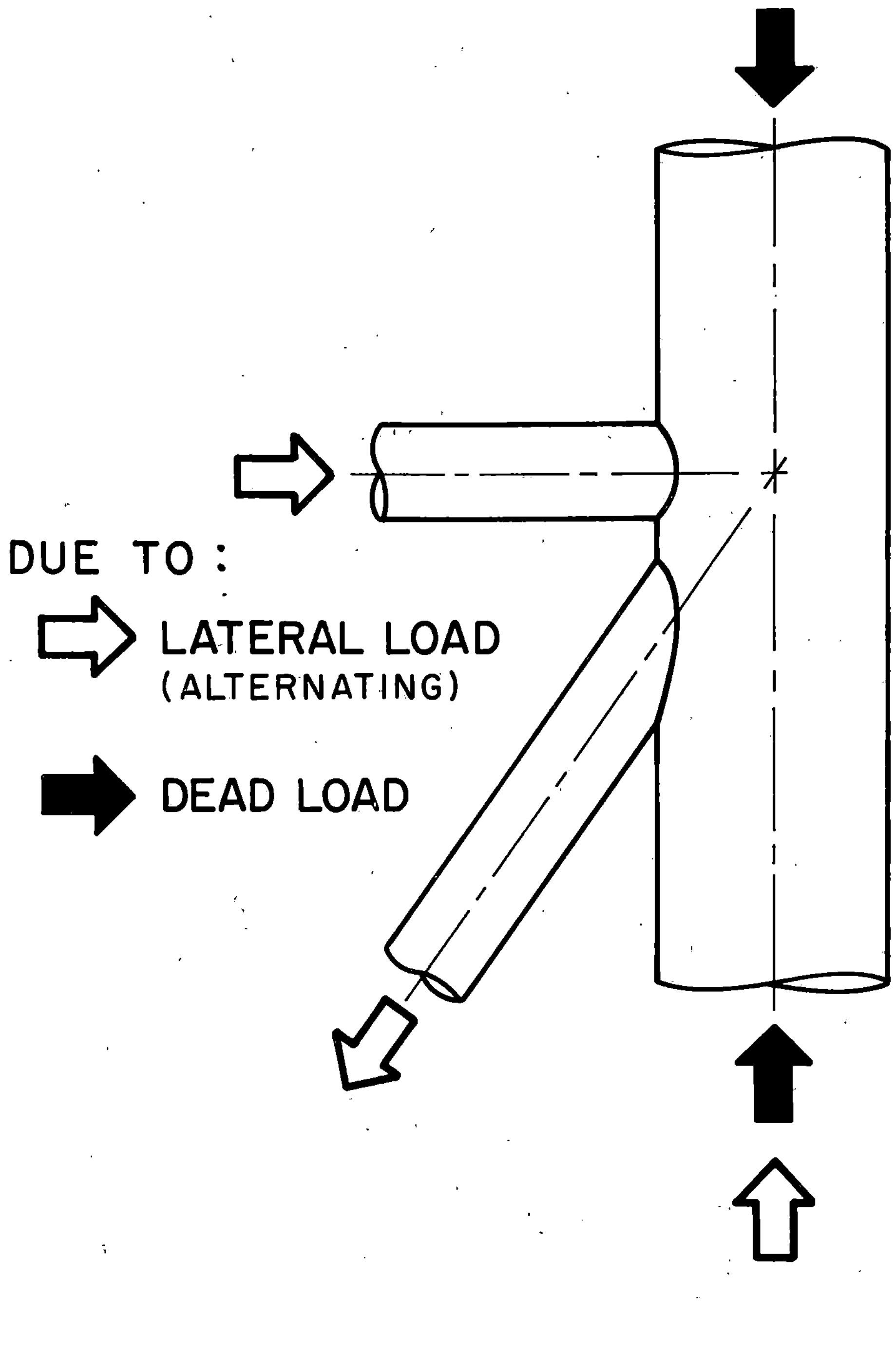
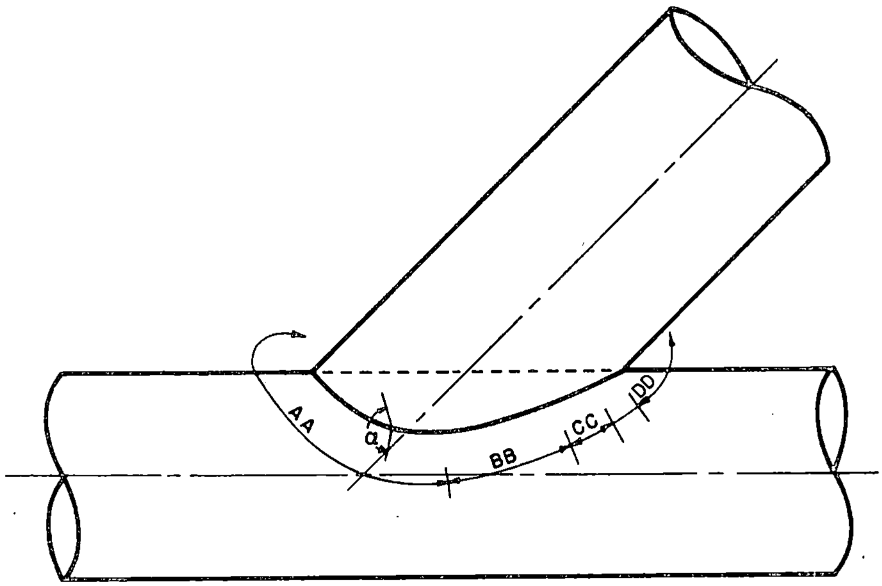
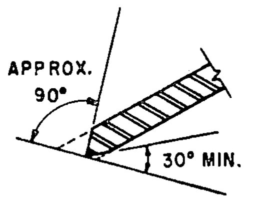


FIG. 3 GENERAL LOAD ARRANGEMENT

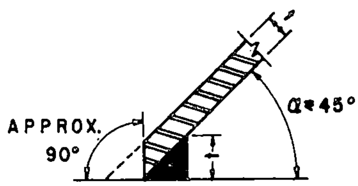


TYPICAL CONNECTION ELEVATION

ANGLE α IS MEASURED PERPENDICULAR
TO JOINT LINE AND ON THE OUTSIDE OF PIPE

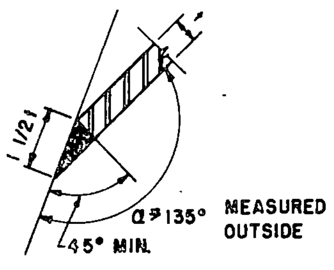


TRANSITION FROM
DETAIL CC TO DD
BY GRADUAL UNIFORM
BEVELING

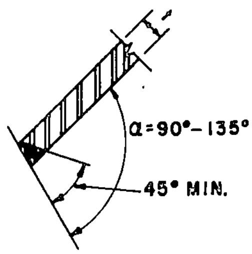


MAKE PENETRATION AS
COMPLETE AS POSSIBLE

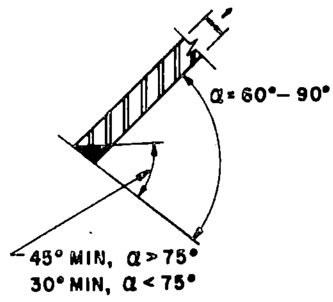
DD



AA



BB



CC

DETAILS OF WELD FROM ONE SIDE ONLY

FIG. 4 WELDING DETAILS

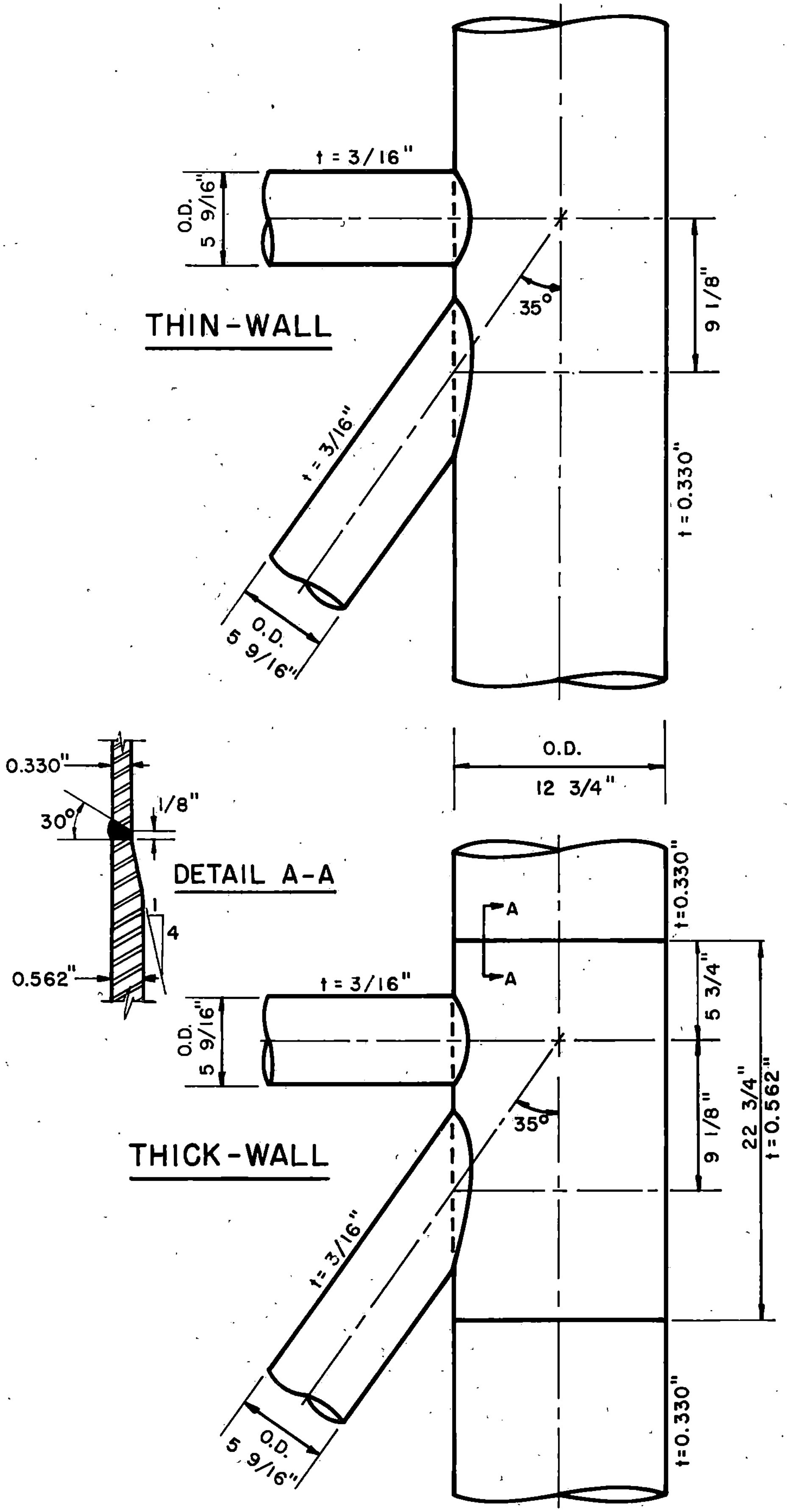


FIG. 5 JOINT TYPE I (ZERO-ECCENTRICITY)

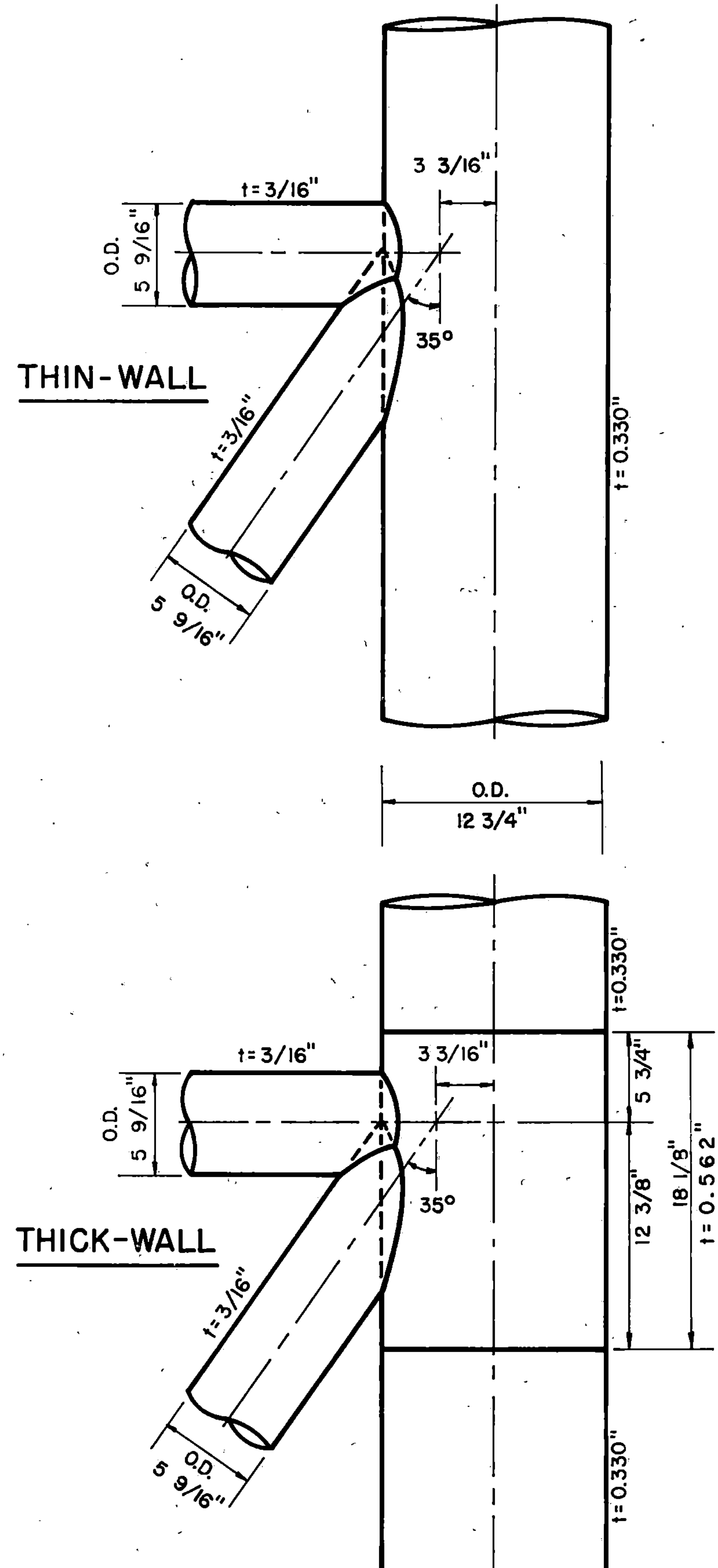


FIG. 6 JOINT TYPE 2 (NEGATIVE-ECCENTRICITY)

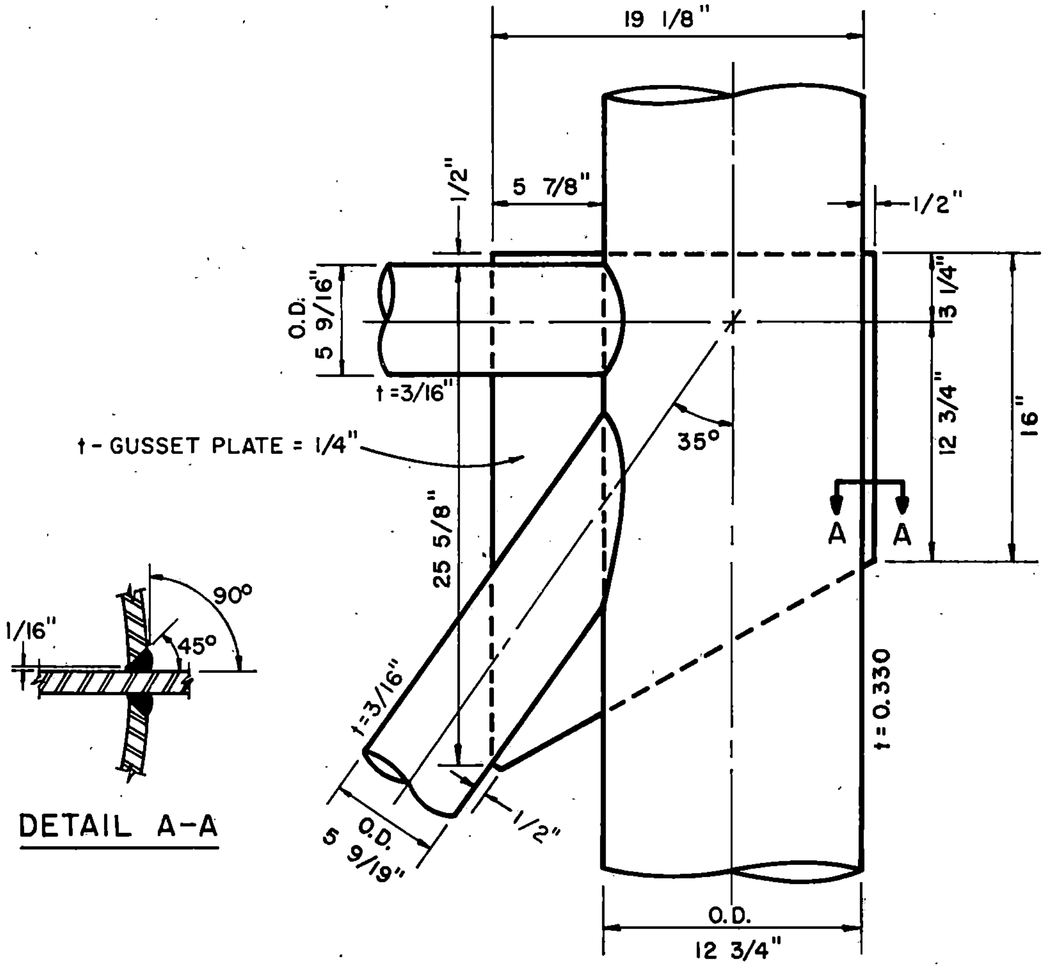


FIG. 7 JOINT TYPE 3

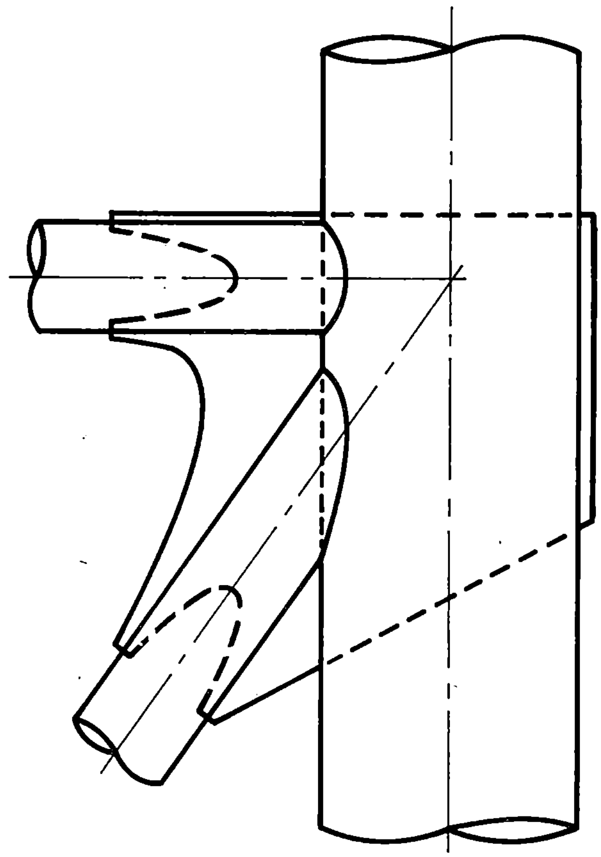


FIG. 8 GUSSET PLATE WITH CUT-OUTS

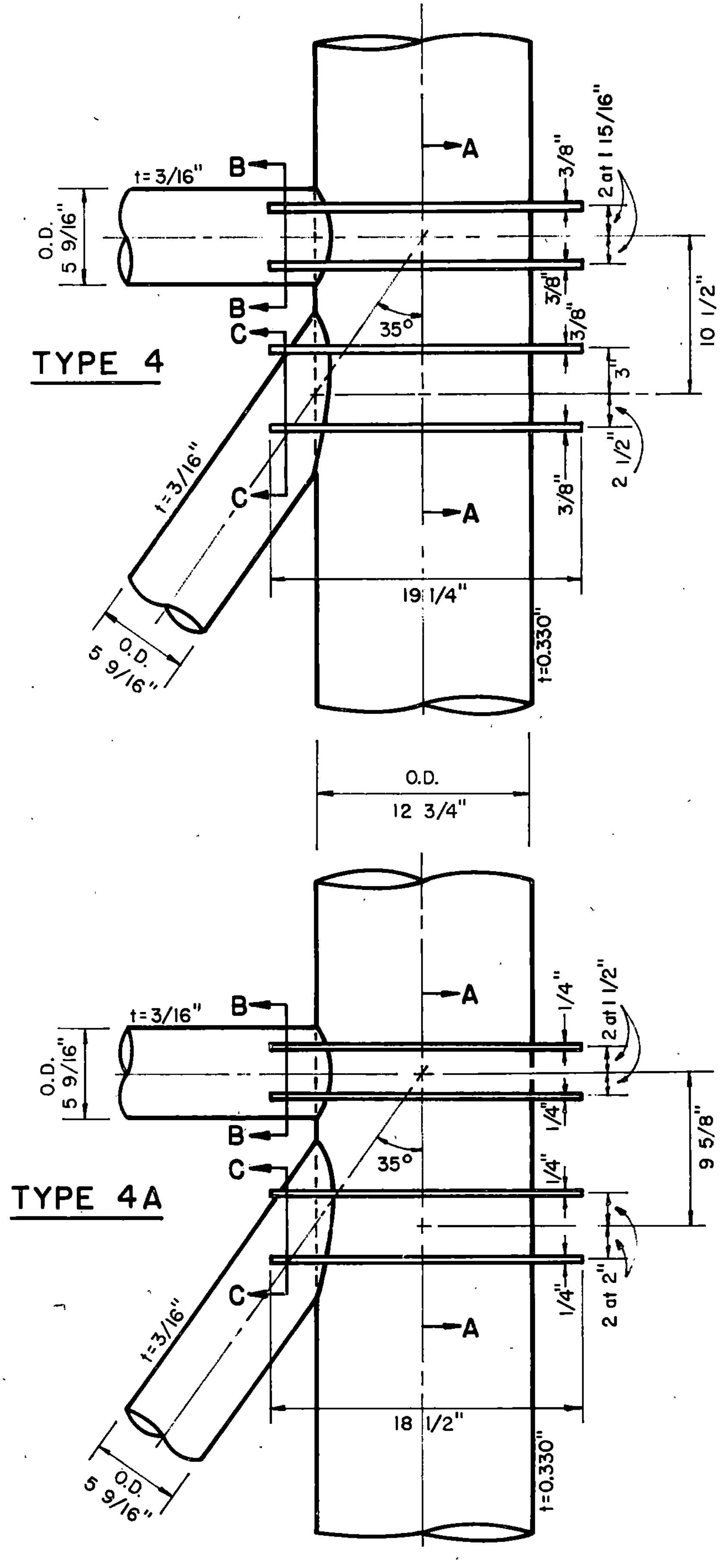


FIG. 9 JOINT TYPE 4

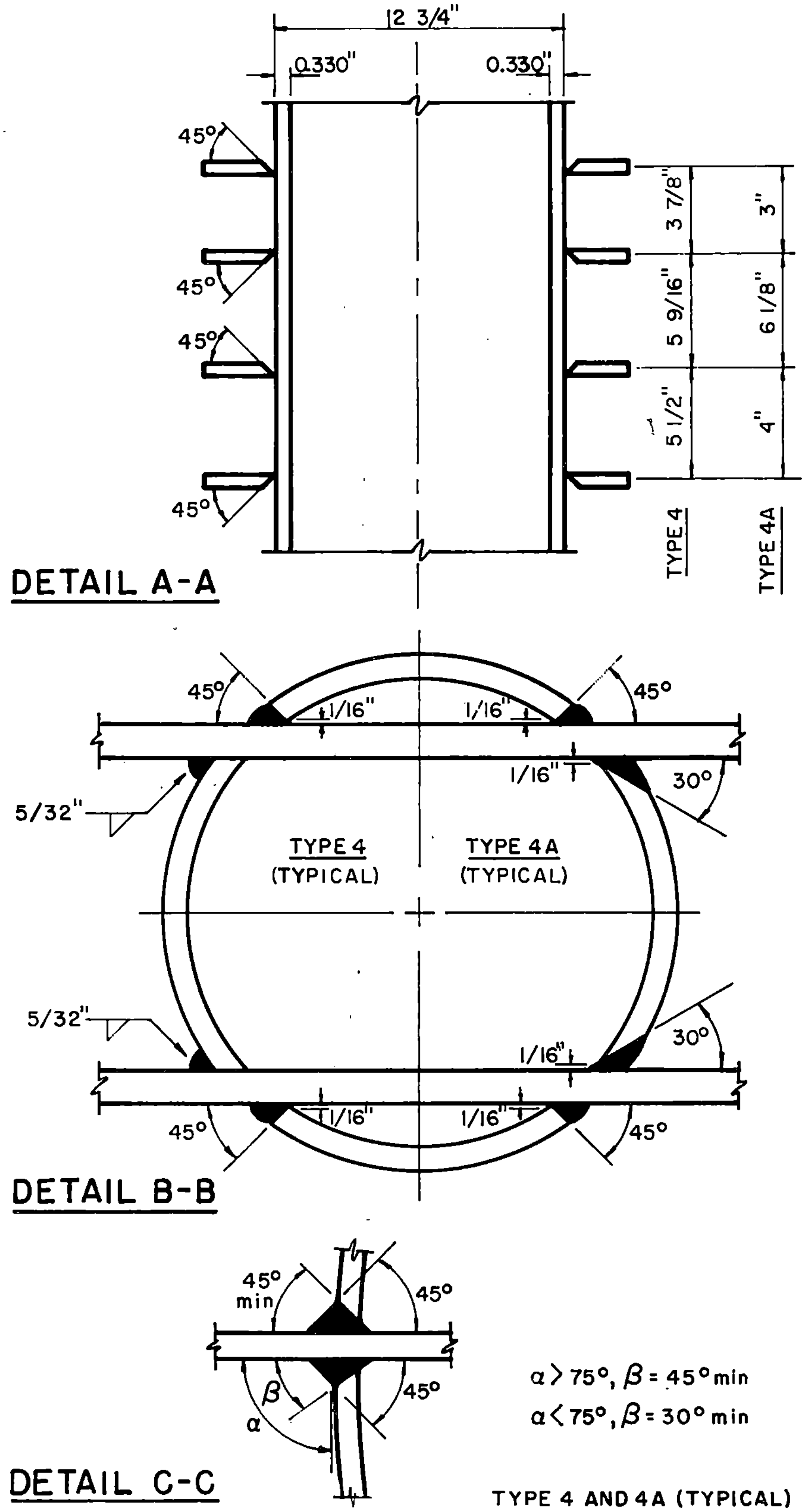


FIG. 10 DETAILS OF RING TO TUBE WELDS FOR JOINT 4

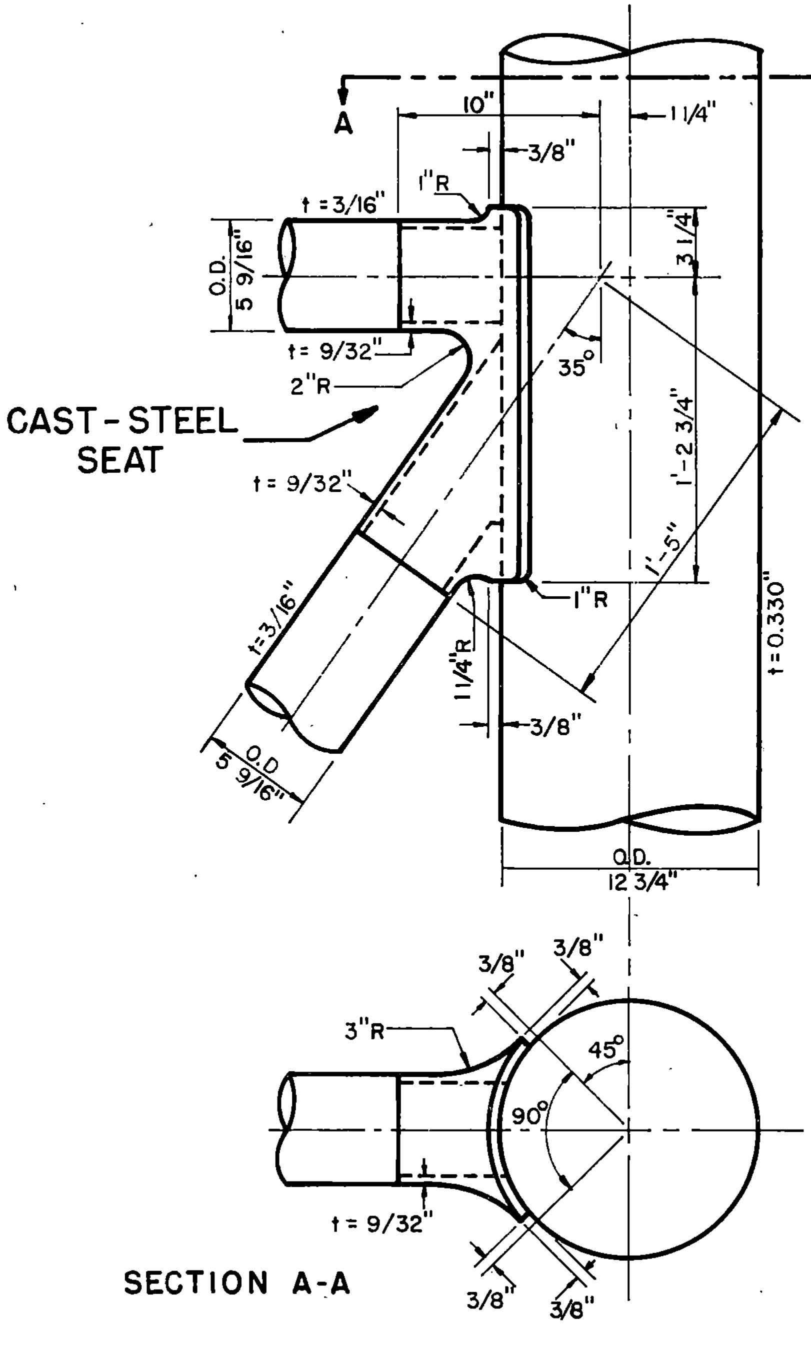


FIG. 11 JOINT TYPE 5

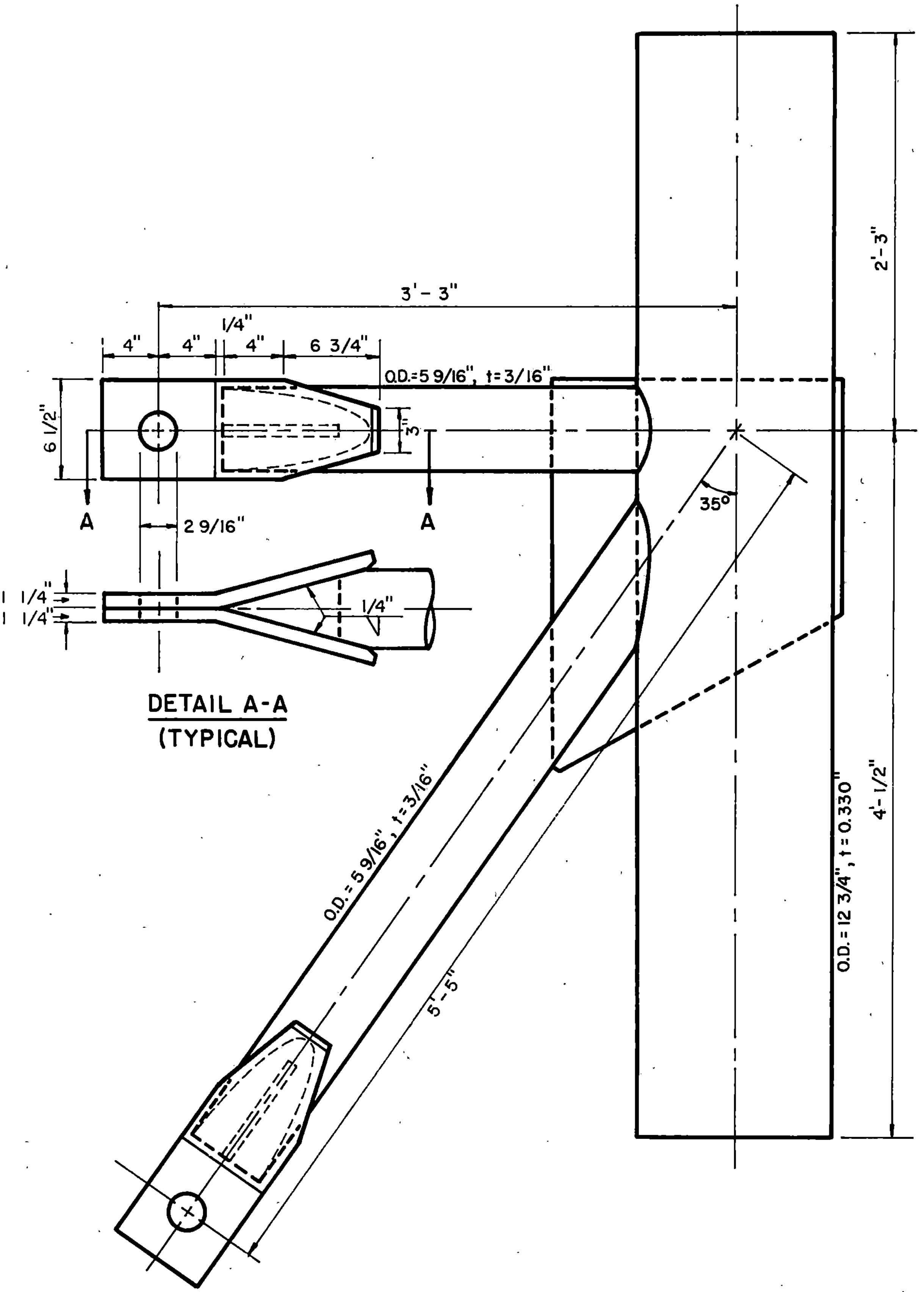


FIG. 12 ZERO-ECCENTRICITY SPECIMEN

(TYPE 3 TYPICAL DIMENSIONS)

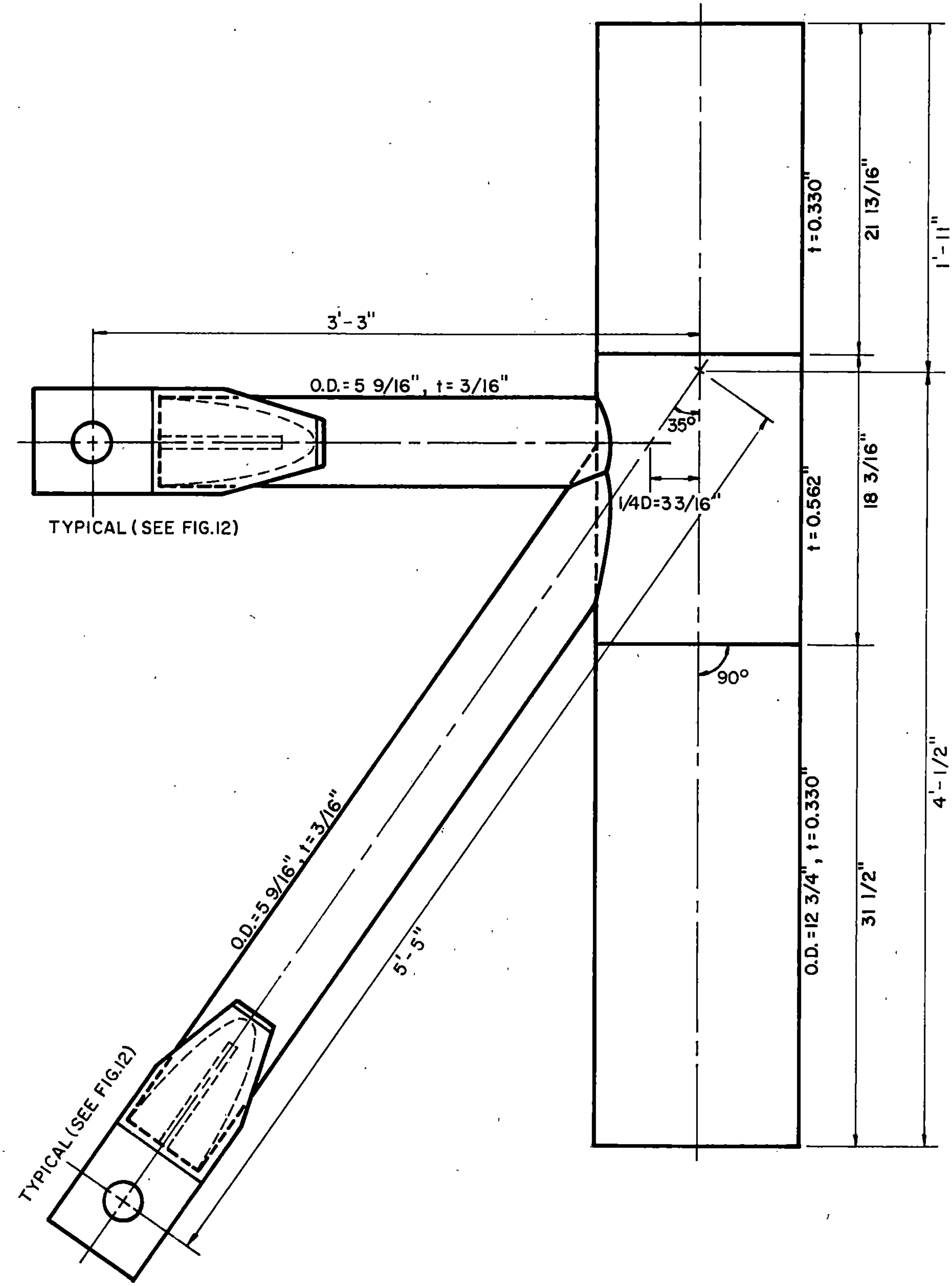


FIG. 13 NEGATIVE-ECCENTRICITY SPECIMEN

(TYPICAL DIMENSIONS)

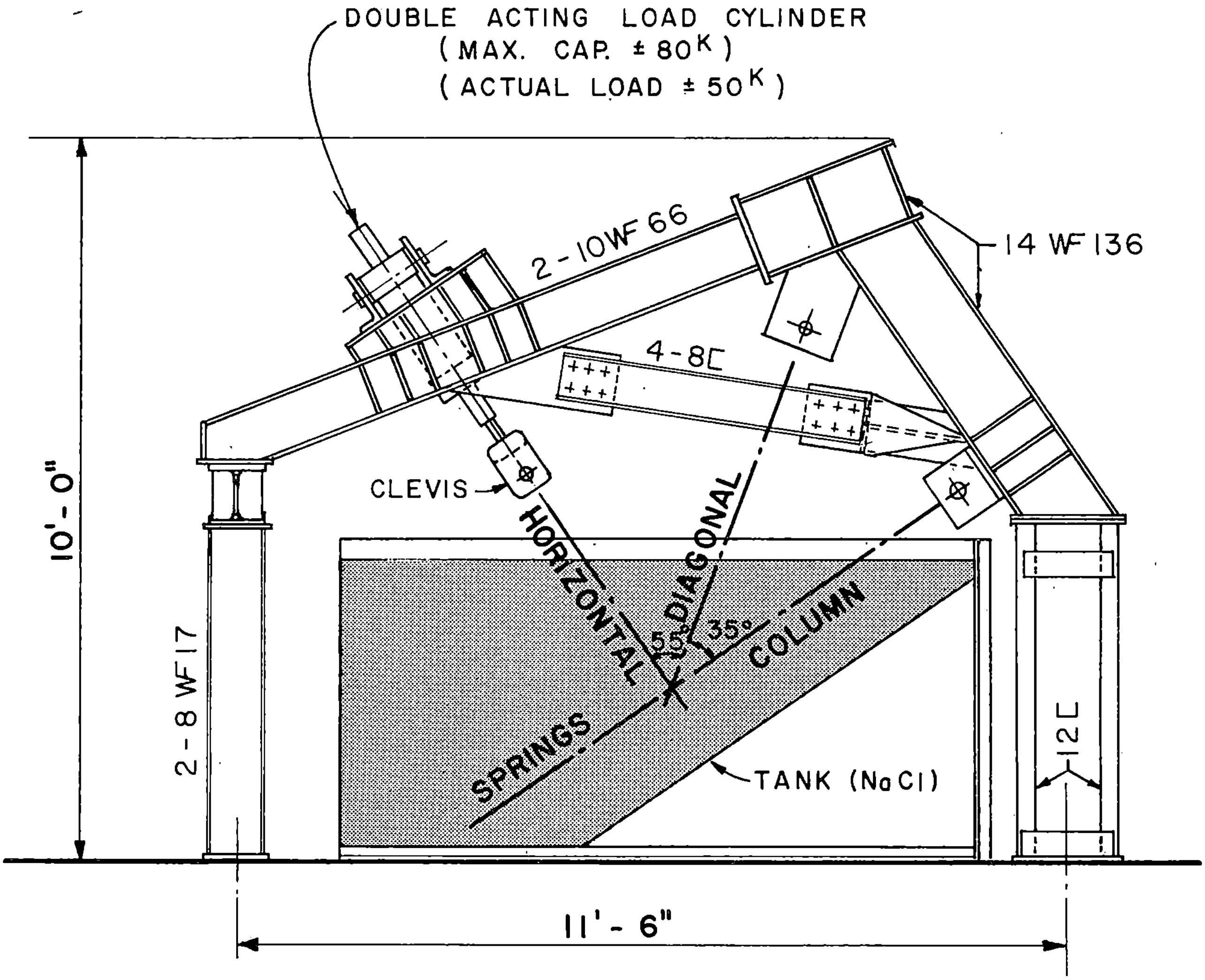


FIG. 14 LOADING FRAME WITH TANK

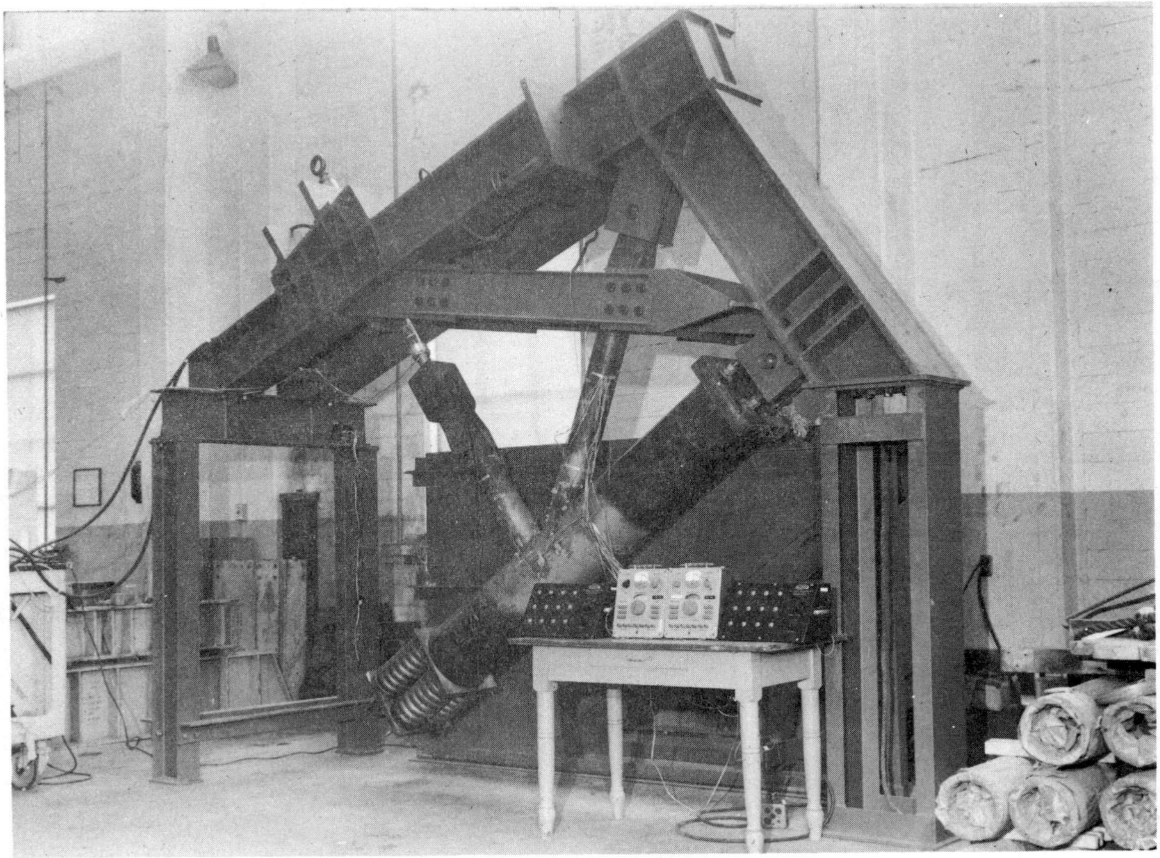


FIG. 15 TEST ARRANGEMENT

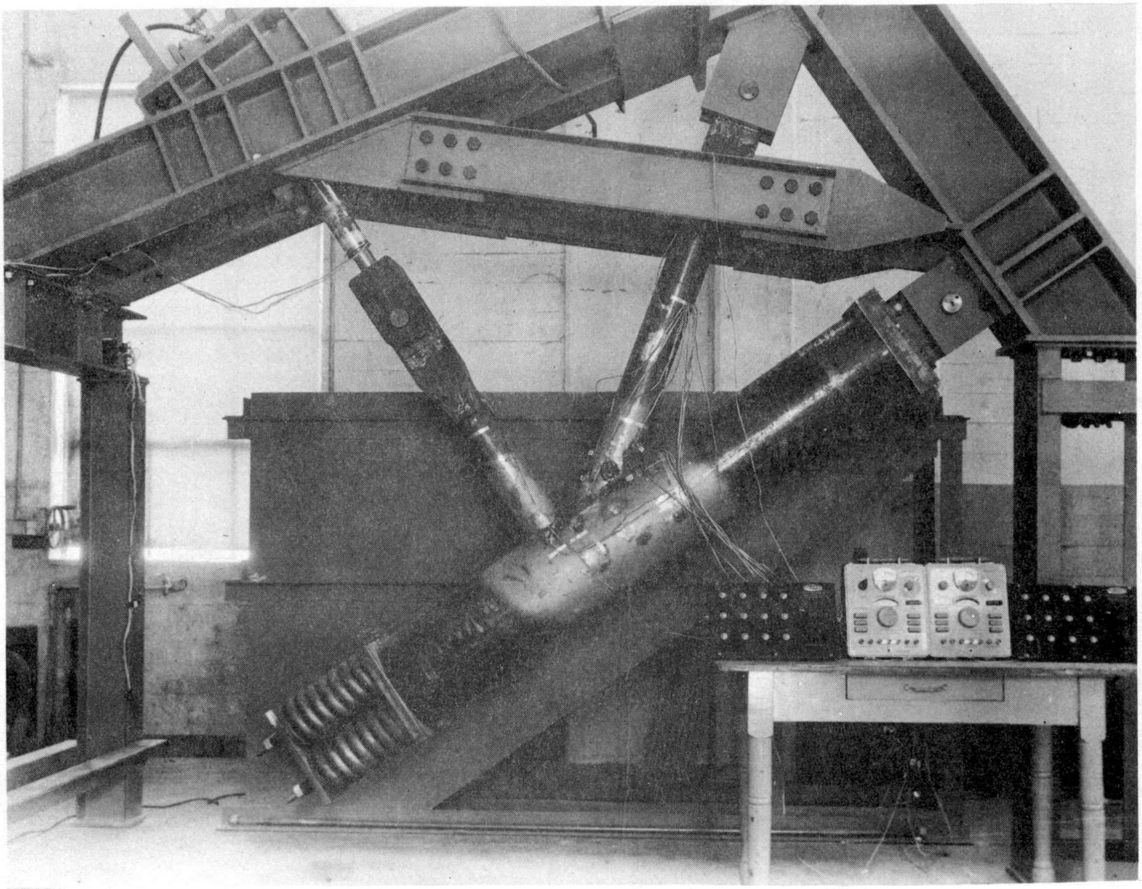
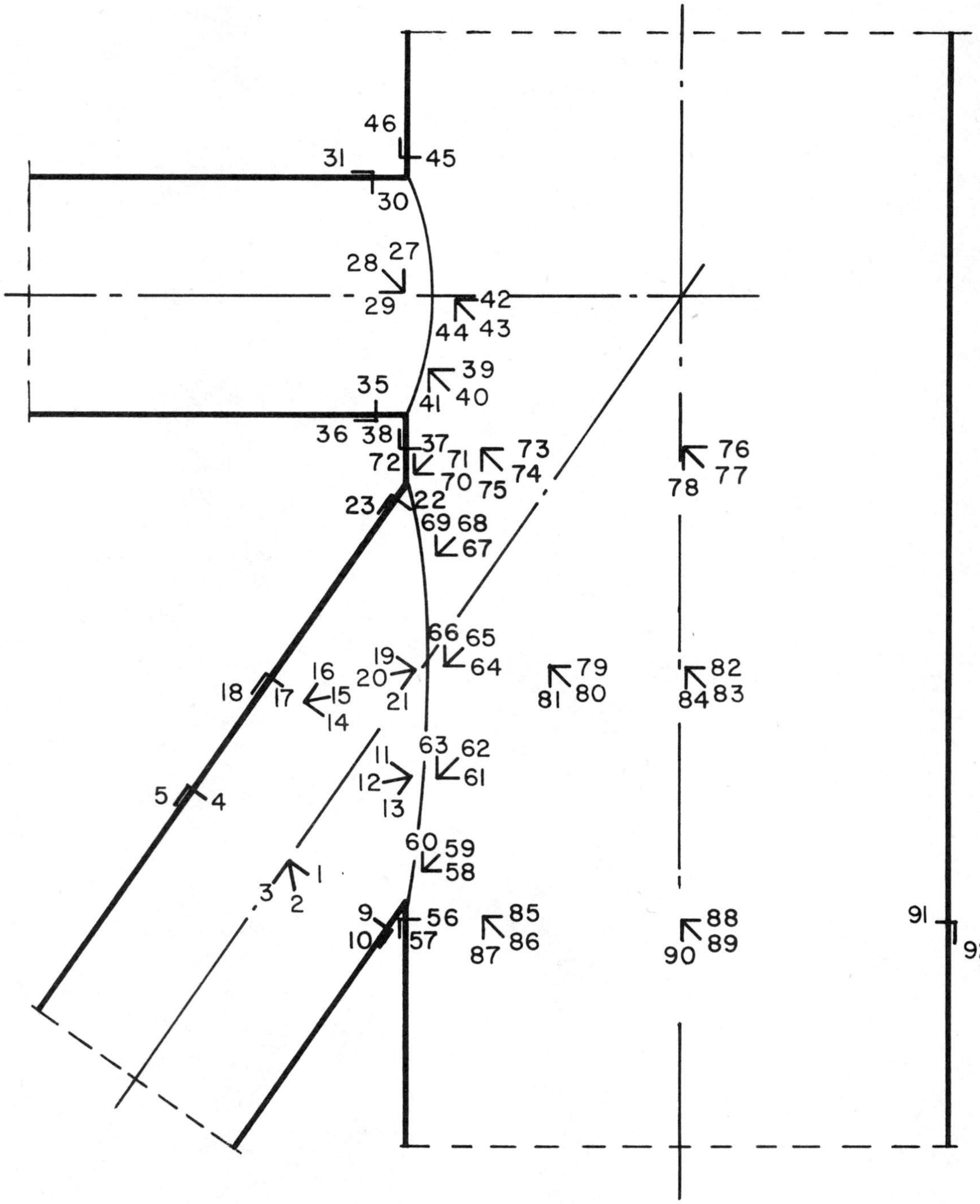


FIG. 16 MOUNTED SPECIMEN



GAGES 1, 2, 3; 19, 20, 21; 39, 40, 41; 64, 65, 66; 70, 71, 72;

AND 88, 89, 90 REPEATED ON FAR SIDE

FIG. 17 STRAIN GAGE INSTRUMENTATION
(JOINT TYPE I - THICK WALL & THIN WALL)

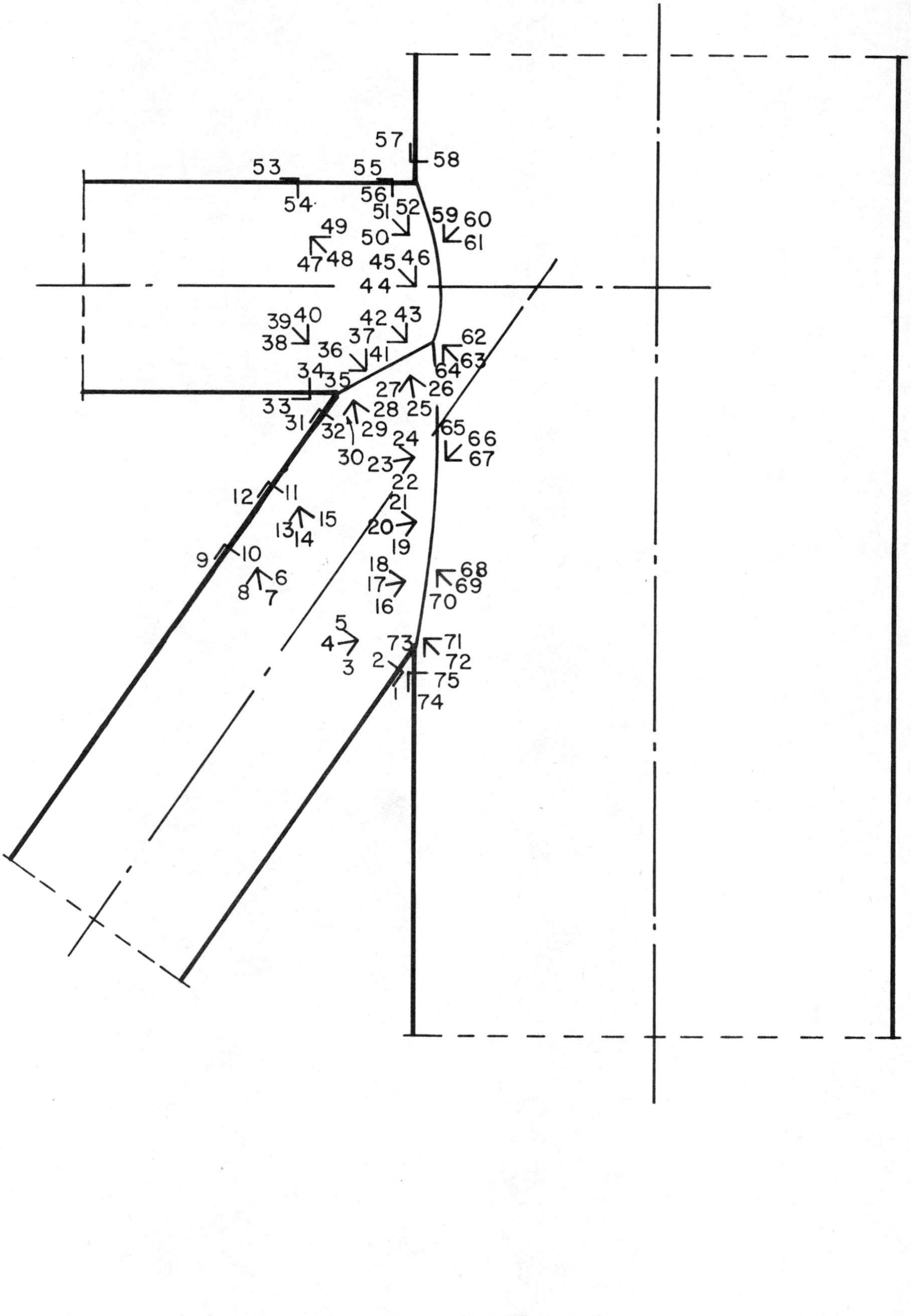
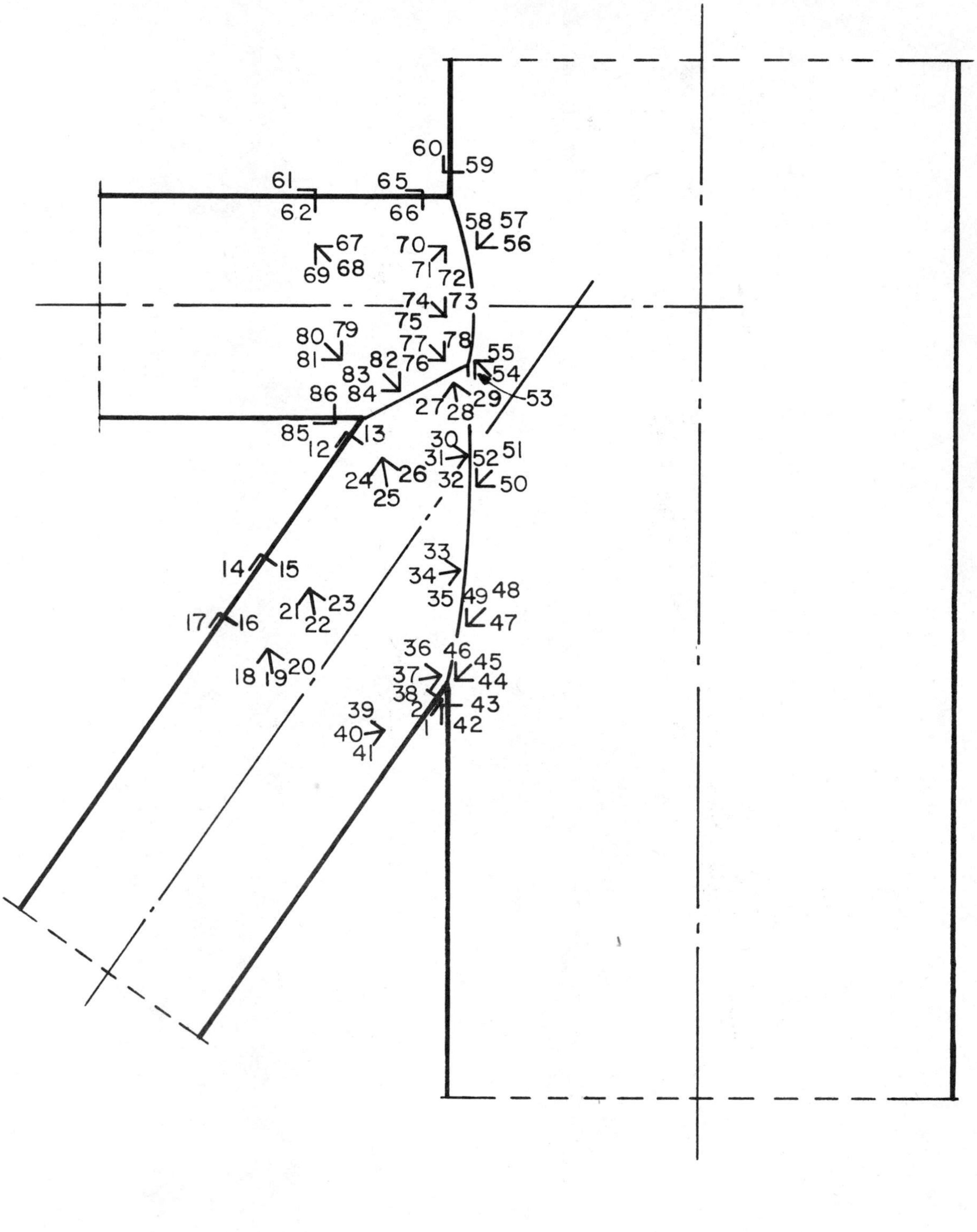


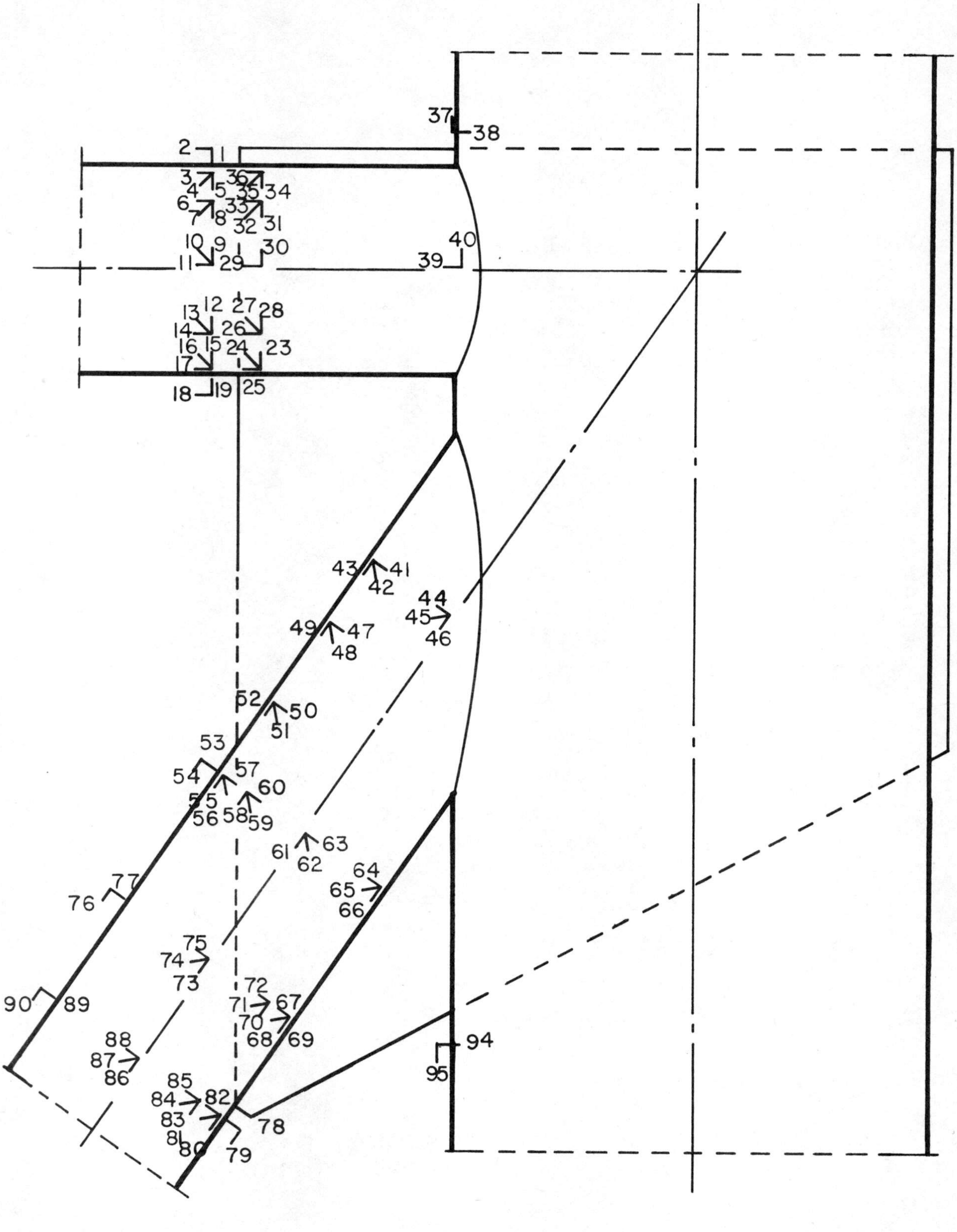
FIG. 18 STRAIN GAGE INSTRUMENTATION

(JOINT TYPE 2 - THIN WALL)



GAGES 27 TO 35; 70, 71, 72; & 76, 77, 78 REPEATED ON FAR SIDE

FIG. 19 STRAIN GAGE INSTRUMENTATION
(JOINT TYPE 2 - THICK WALL)



GAGES 9, 10, 11 AND 86, 87, 88 REPEATED ON FAR SIDE

FIG. 20 STRAIN GAGE INSTRUMENTATION

(JOINT TYPE 3)

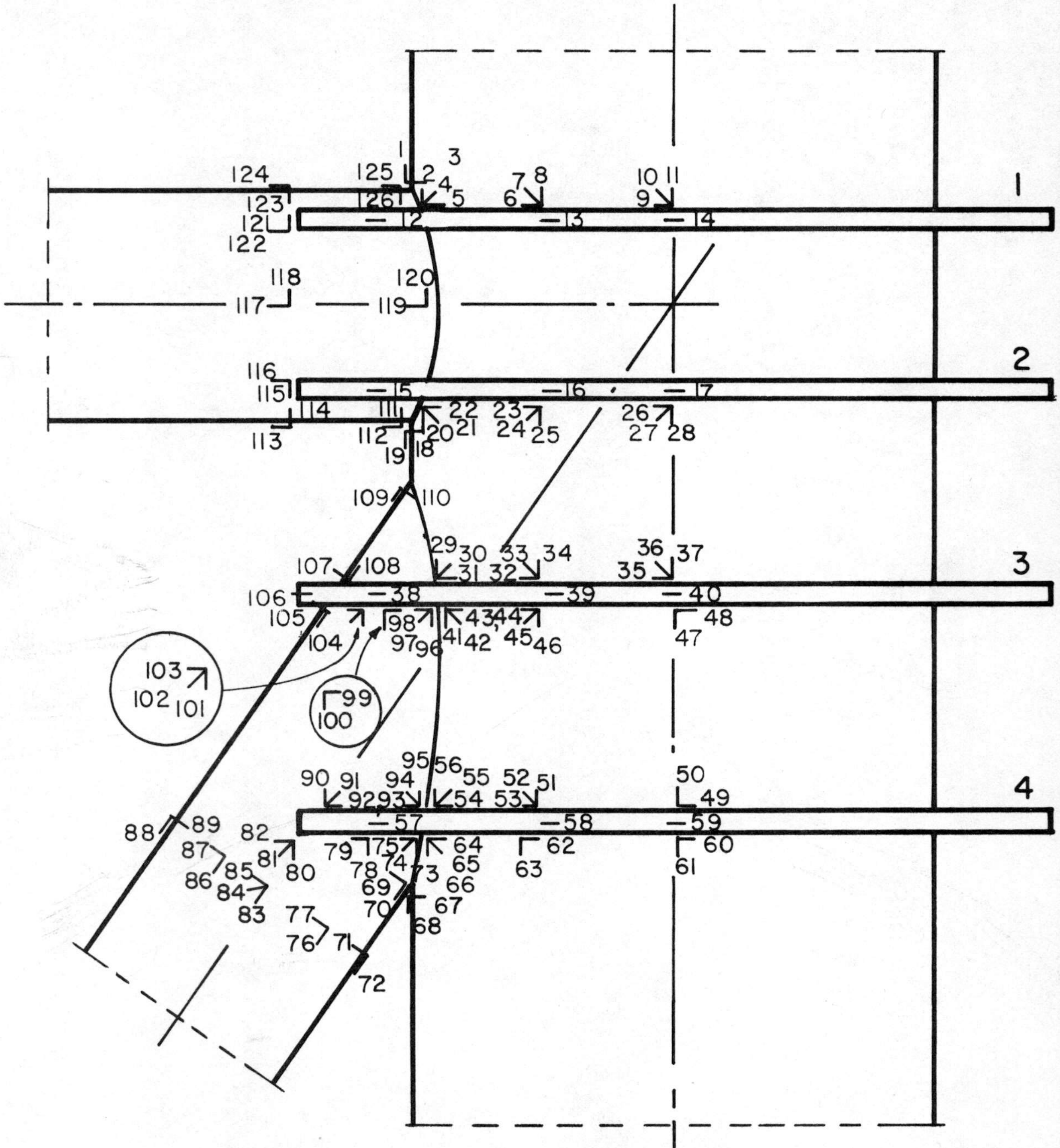


FIG. 21 STRAIN GAGE INSTRUMENTATION

(JOINT TYPE 4)

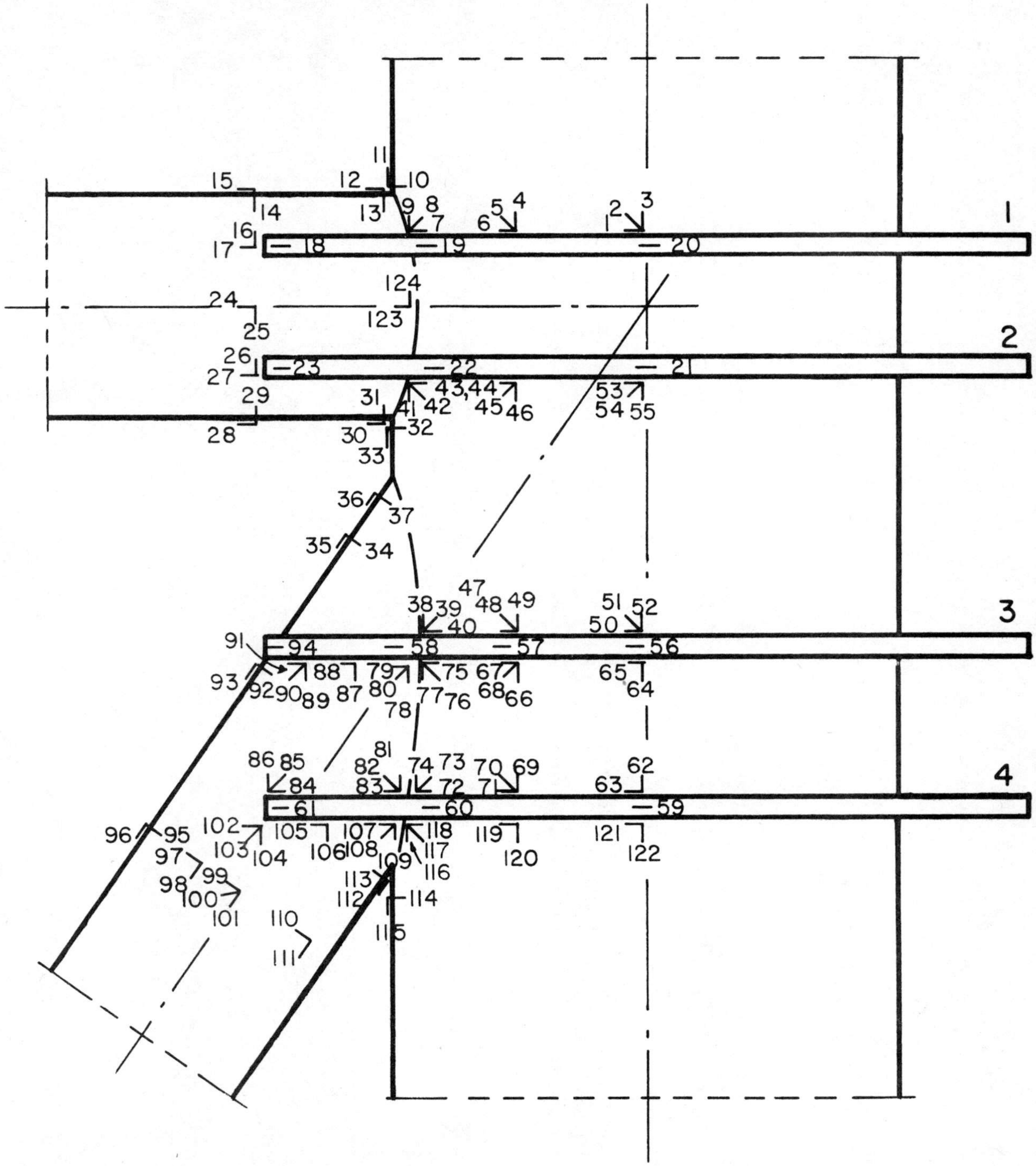
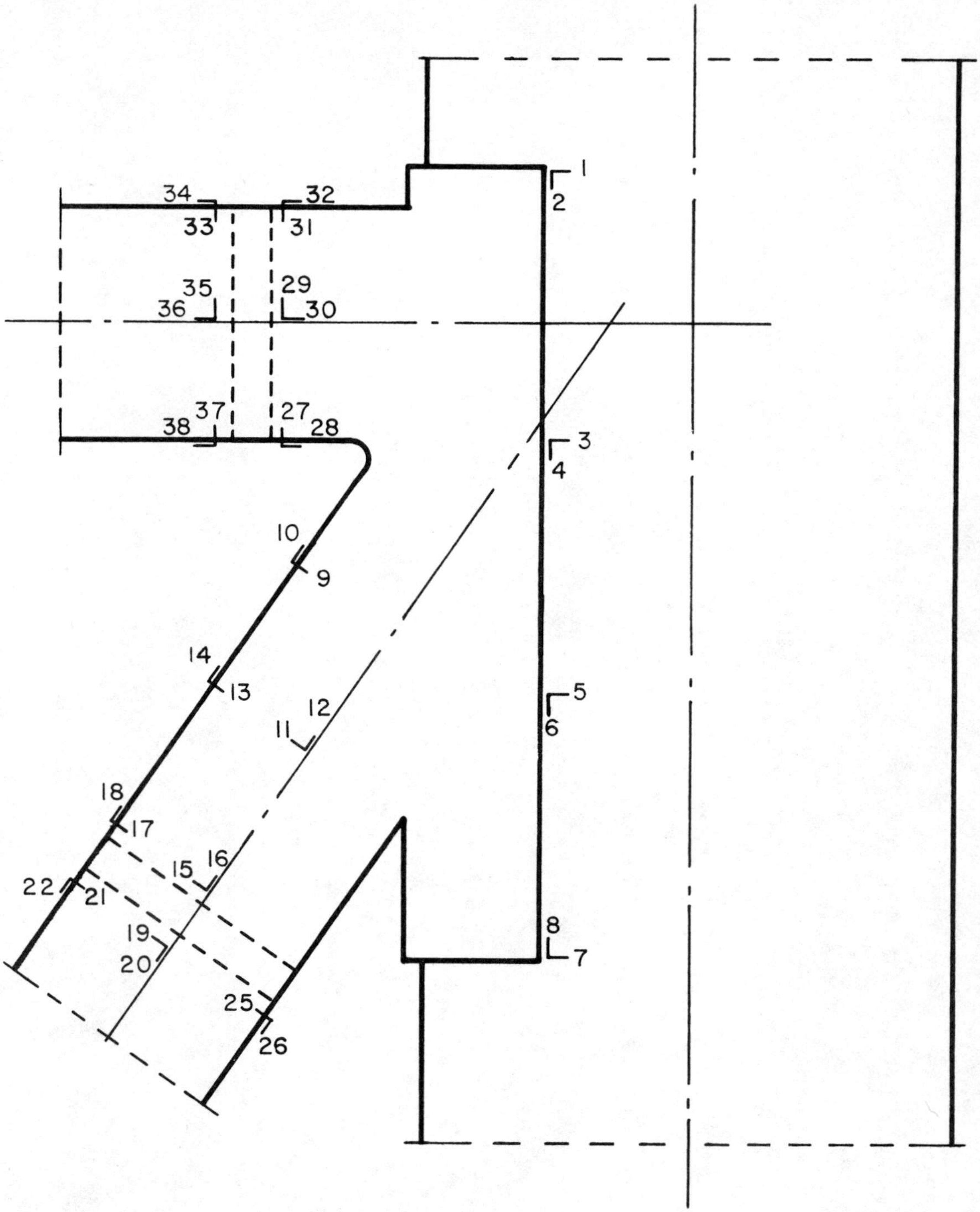


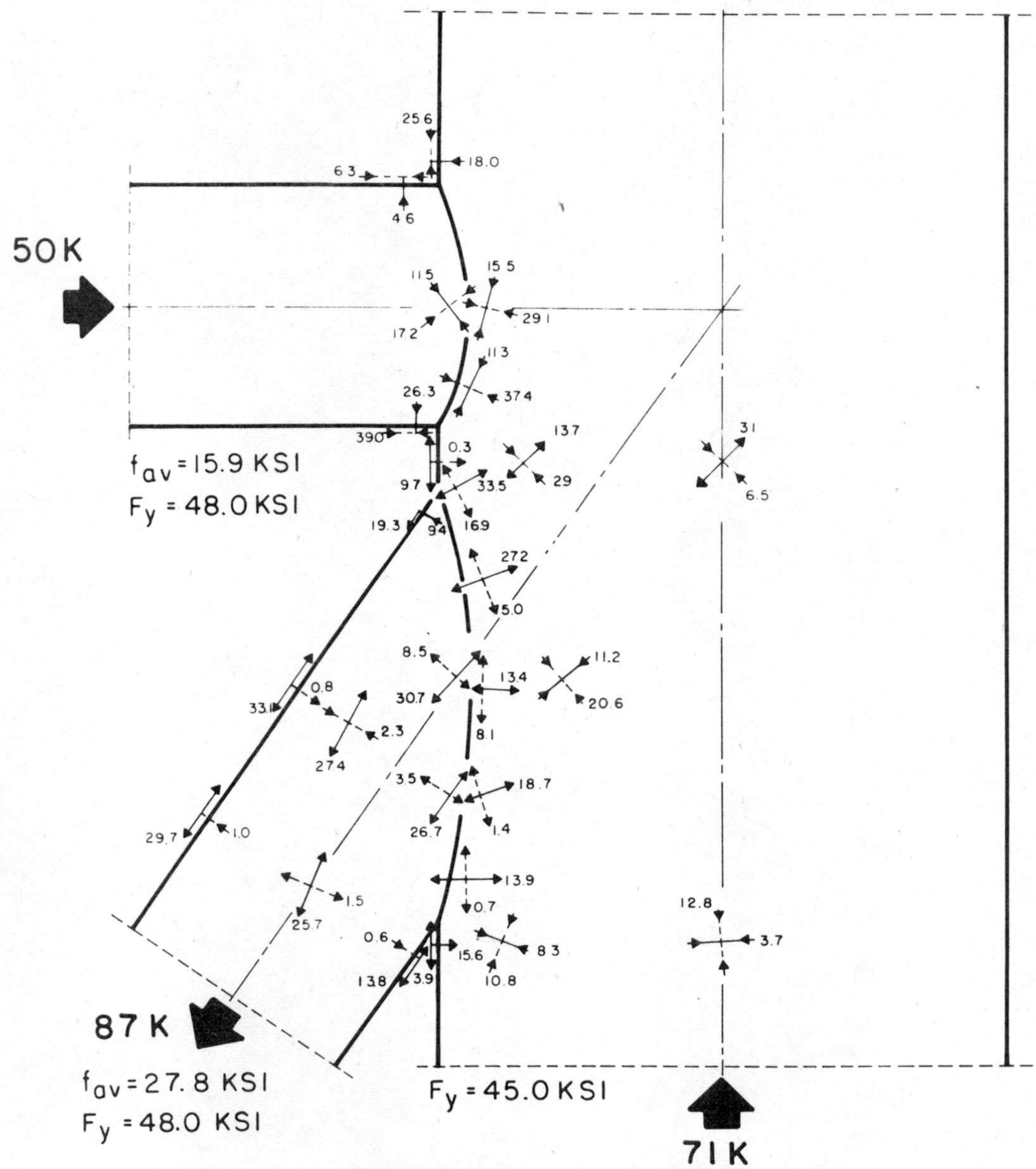
FIG. 22 STRAIN GAGE INSTRUMENTATION

(JOINT TYPE 4A)



GAGES 19, 20 AND 35, 36 REPEATED ON FAR SIDE

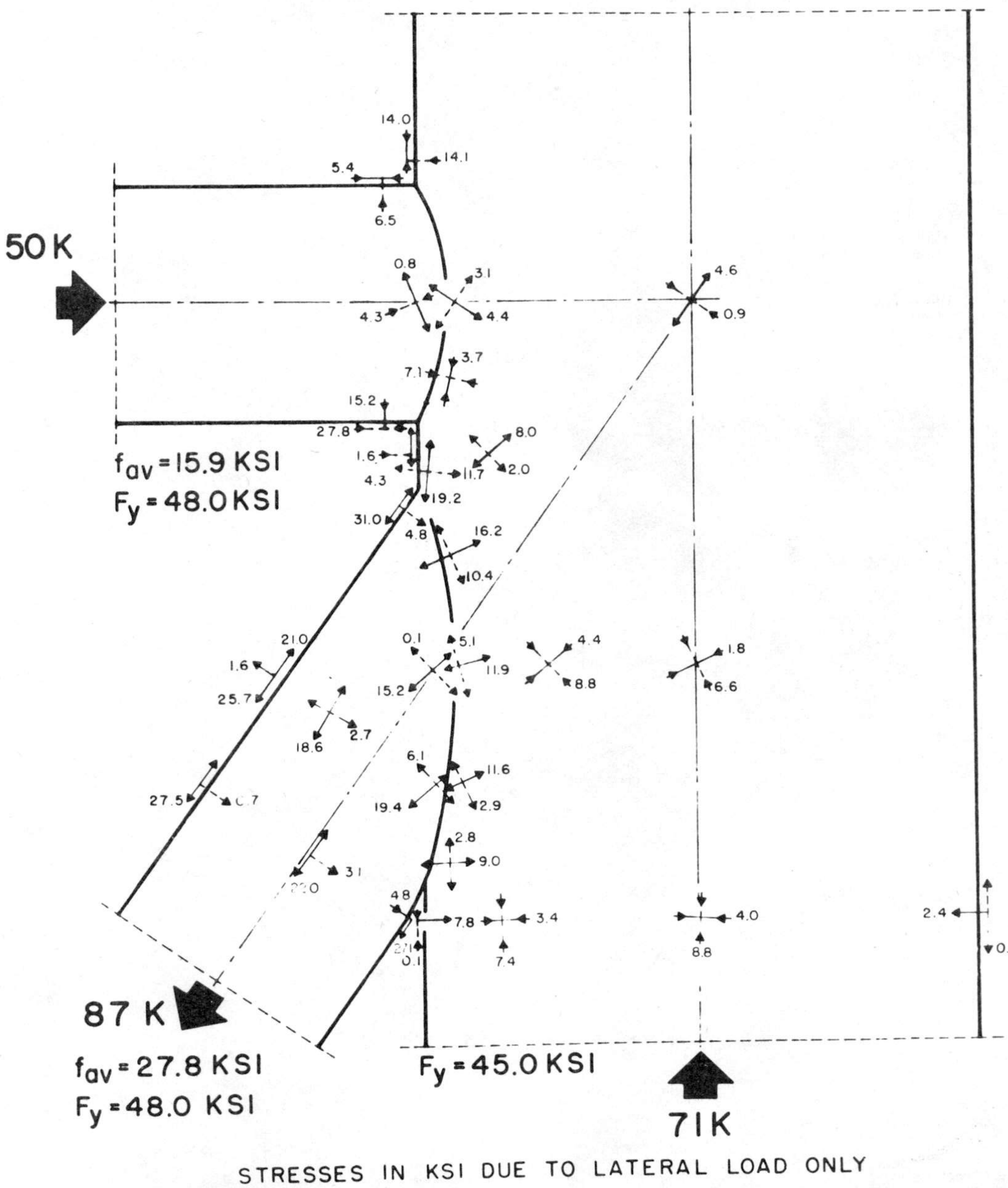
FIG. 23 STRAIN GAGE INSTRUMENTATION
(JOINT TYPE 5)



STRESSES IN KSI DUE TO LATERAL LOAD ONLY
COLUMN PRELOAD = 140 K

⟵⟶ TENSION ⟶⟵ COMPRESSION

FIG. 24 PRINCIPAL STRESSES — JOINT TYPE I - THIN WALL



STRESSES IN KSI DUE TO LATERAL LOAD ONLY

COLUMN PRELOAD = 140K

⟷ TENSION   ⟶⟵ COMPRESSION

FIG. 25 PRINCIPAL STRESSES – JOINT TYPE I – THICK WALL

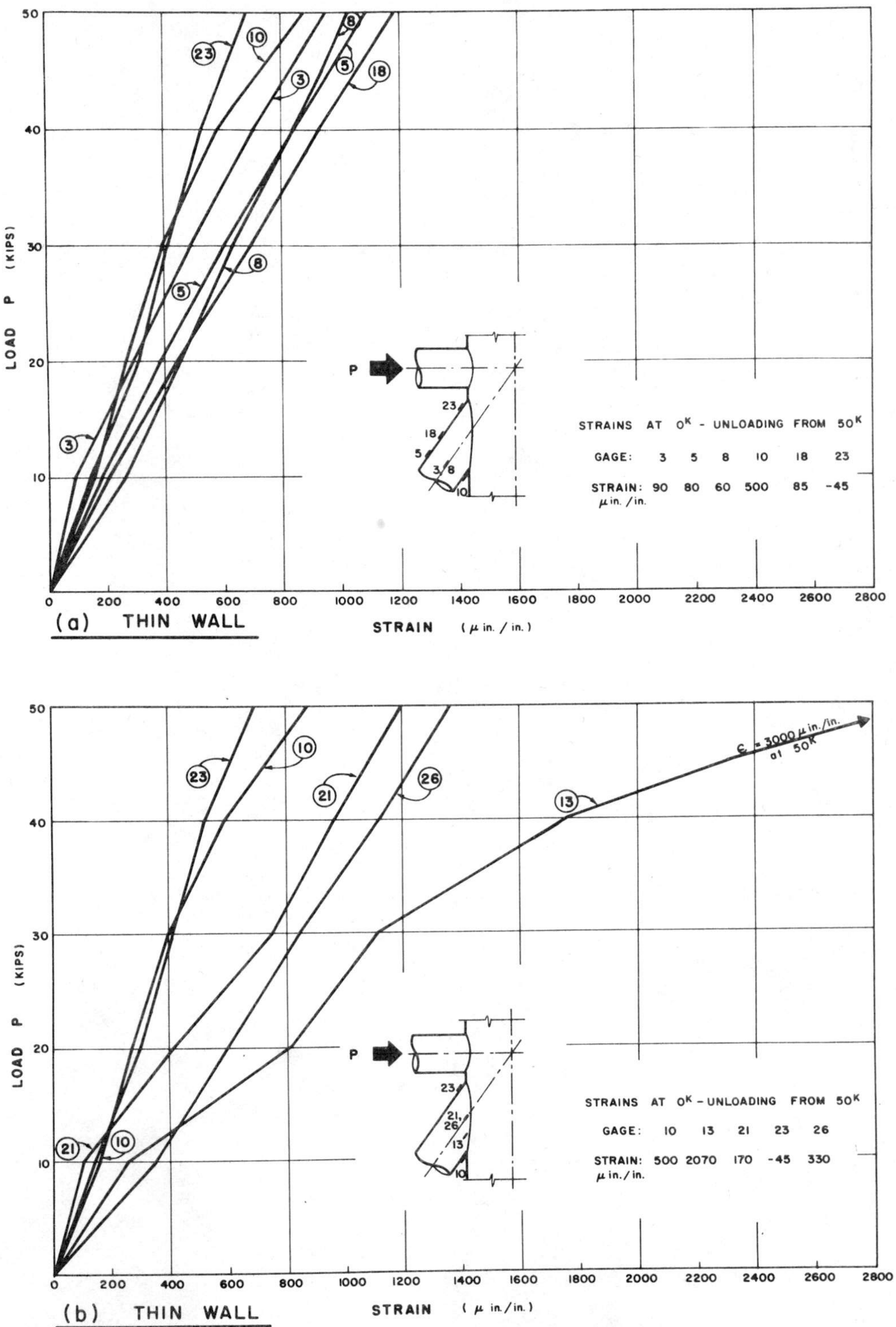
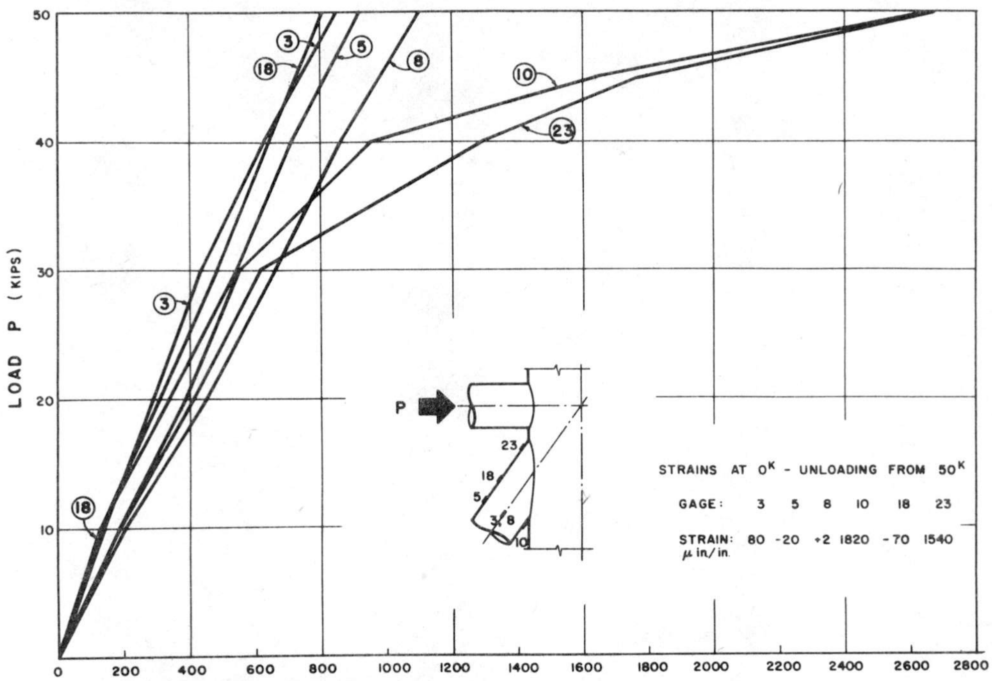
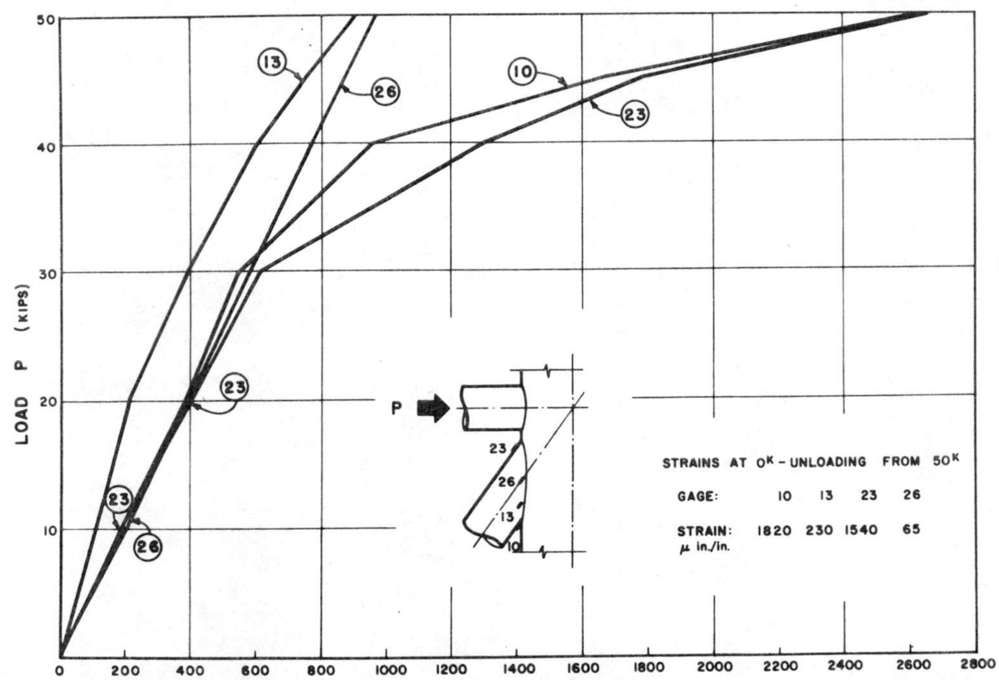


FIG. 26 STRAINS IN DIAGONAL ; JOINT TYPE I



(a) THICK WALL



(b) THICK WALL

FIG. 27 STRAINS IN DIAGONAL ; JOINT TYPE I

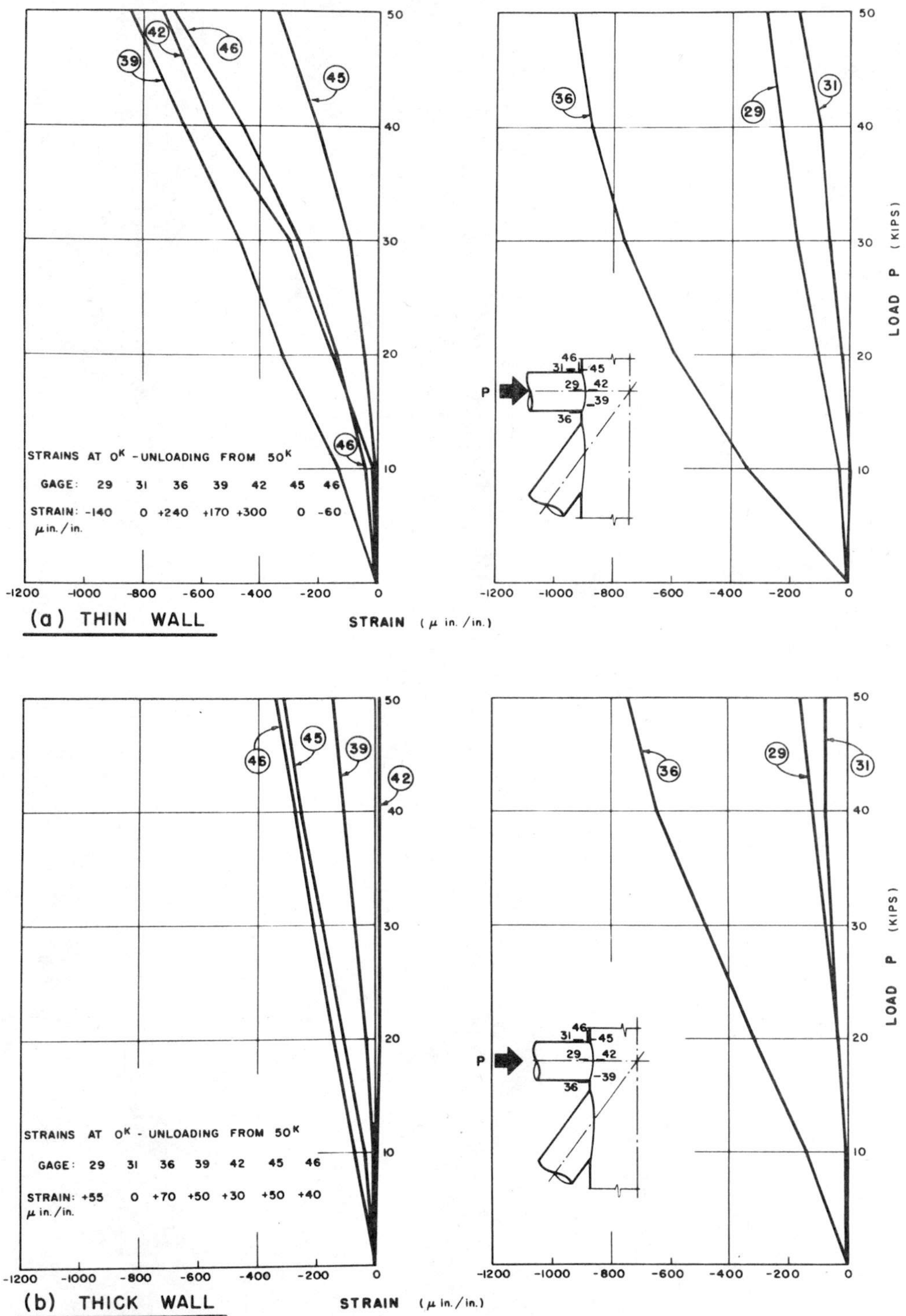


FIG. 28 STRAINS IN HORIZONTAL & COLUMN MEMBERS
JOINT TYPE I

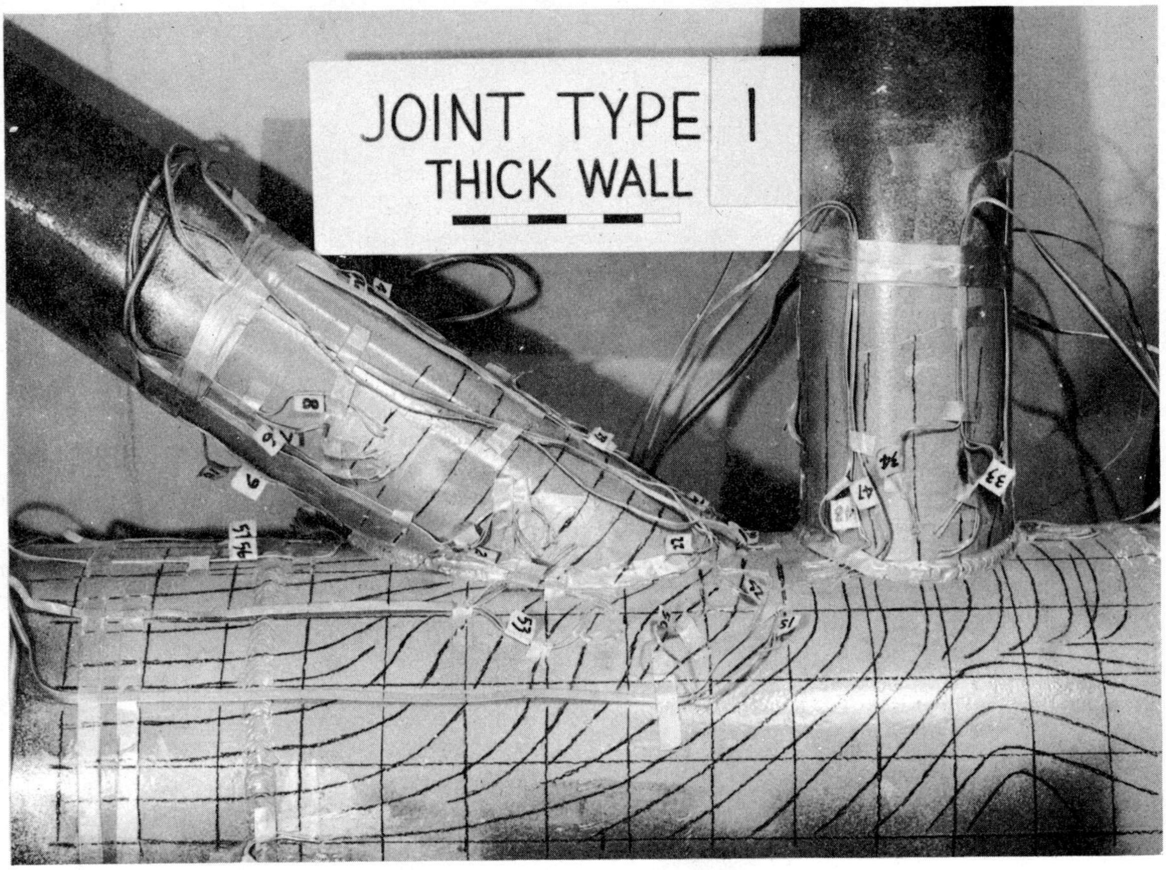
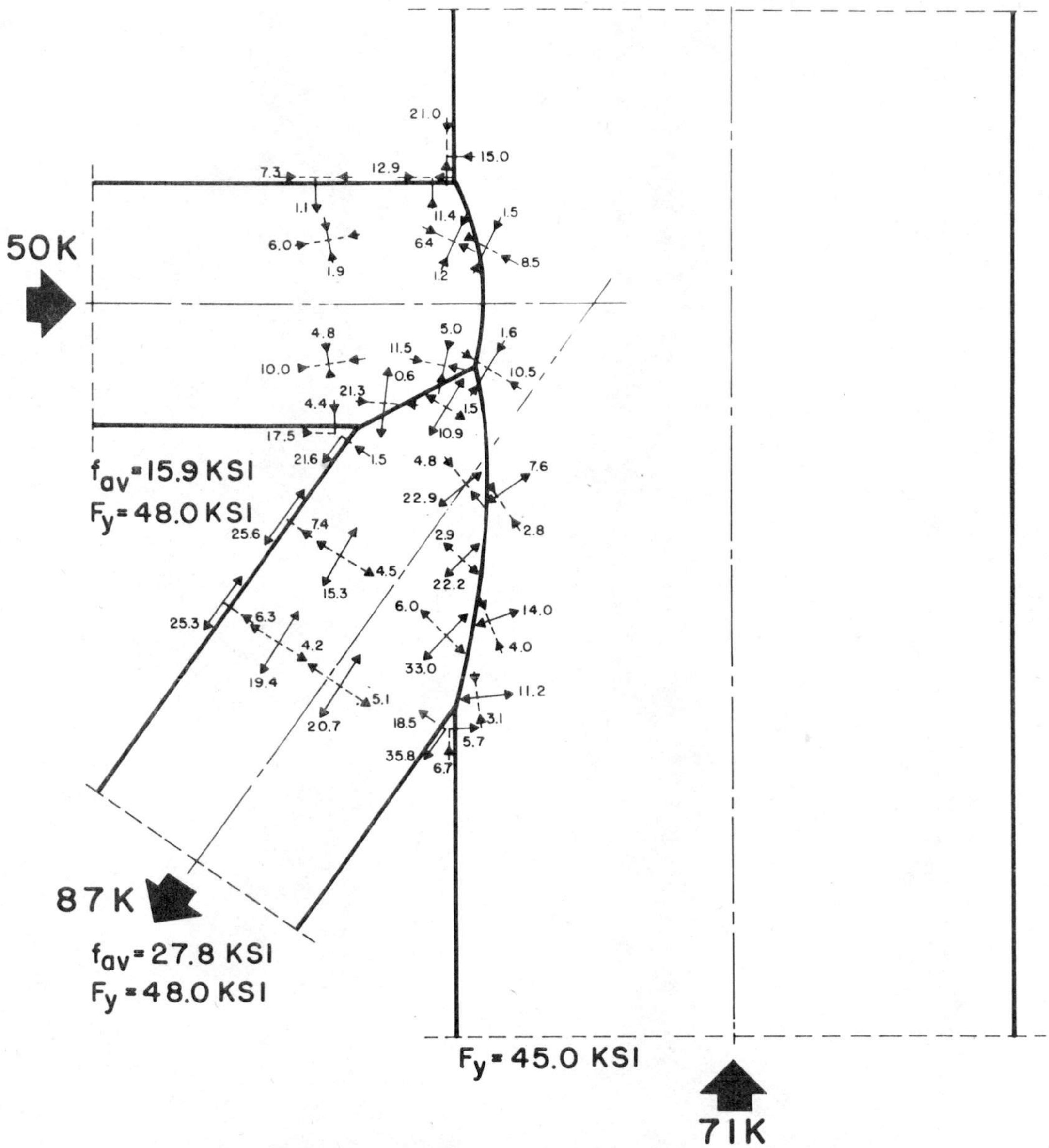


FIG. 29 STRESS-COAT PATTERN
JOINT TYPE I - THICK WALL



STRESSES IN KSI DUE TO LATERAL LOAD ONLY

COLUMN PRELOAD = 140 K

⟷ TENSION ⟶⟵ COMPRESSION

FIG. 30 PRINCIPAL STRESSES - JOINT TYPE 2 - THIN WALL

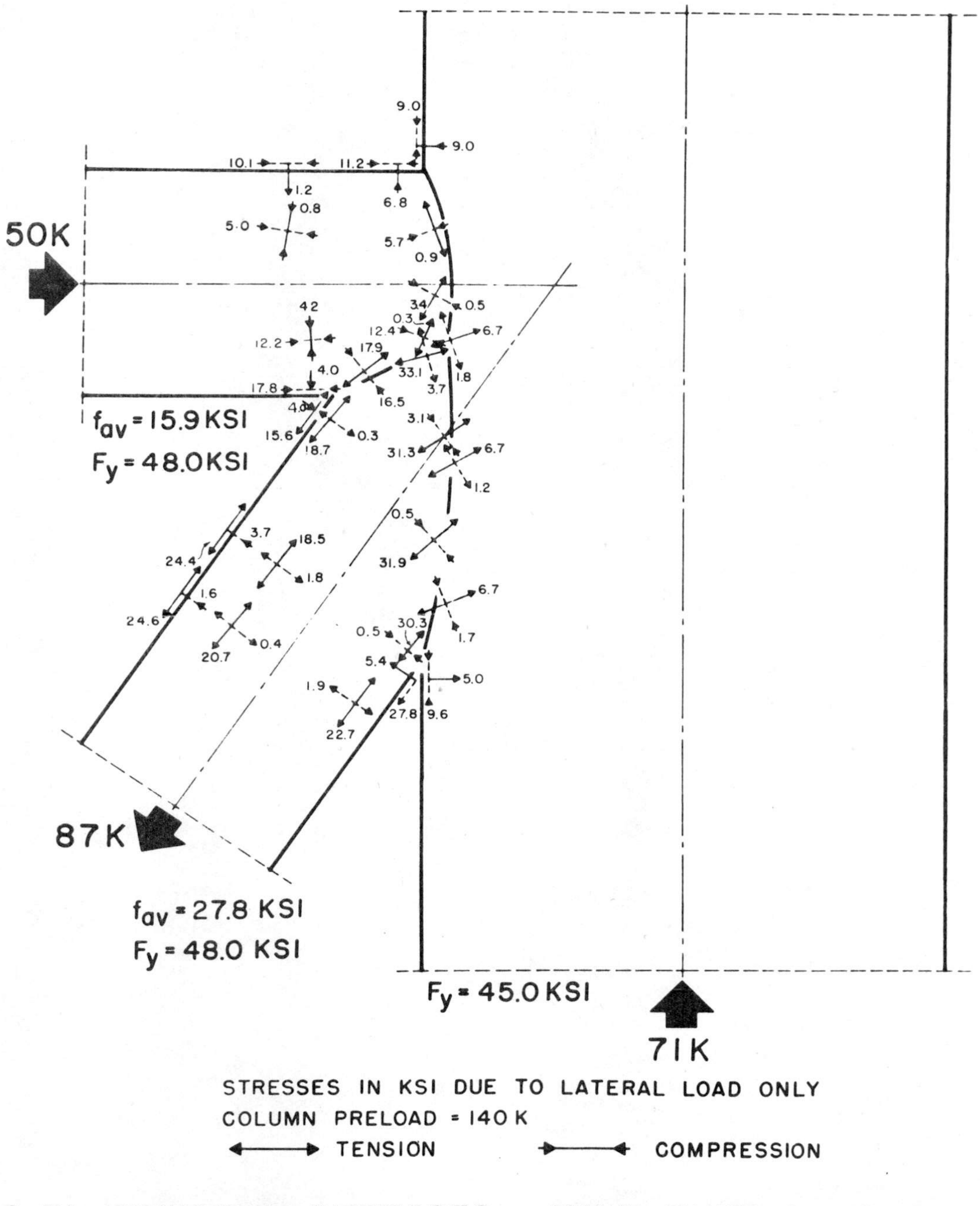
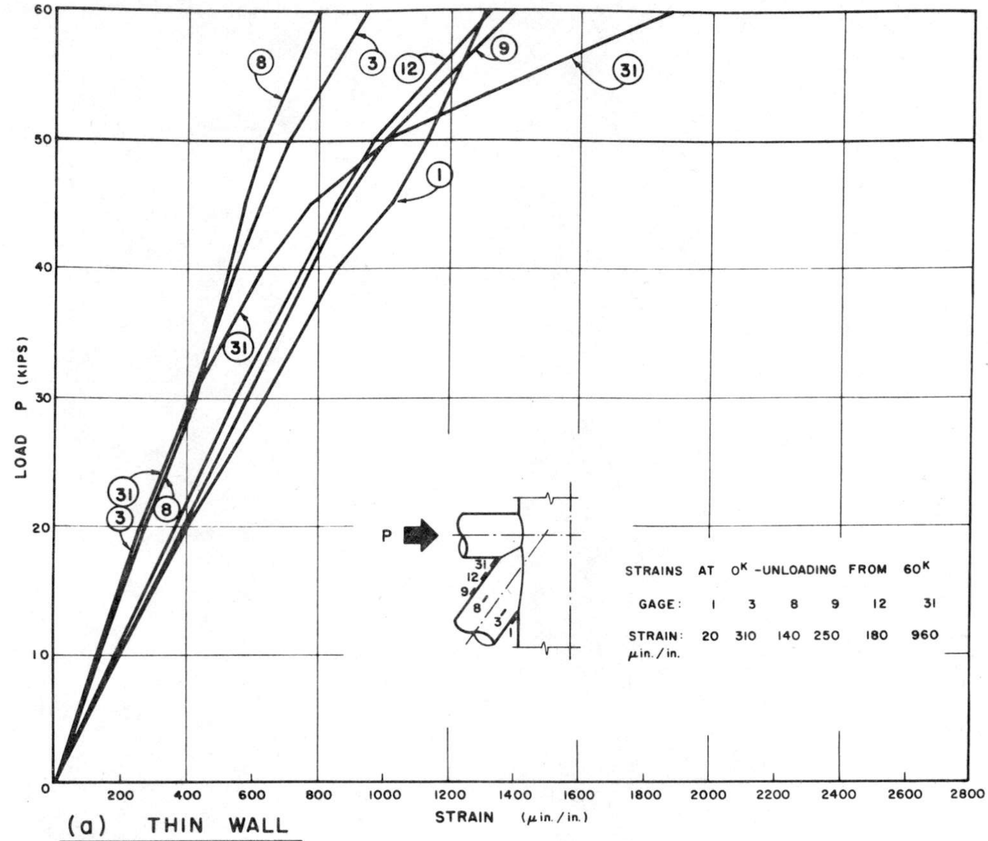
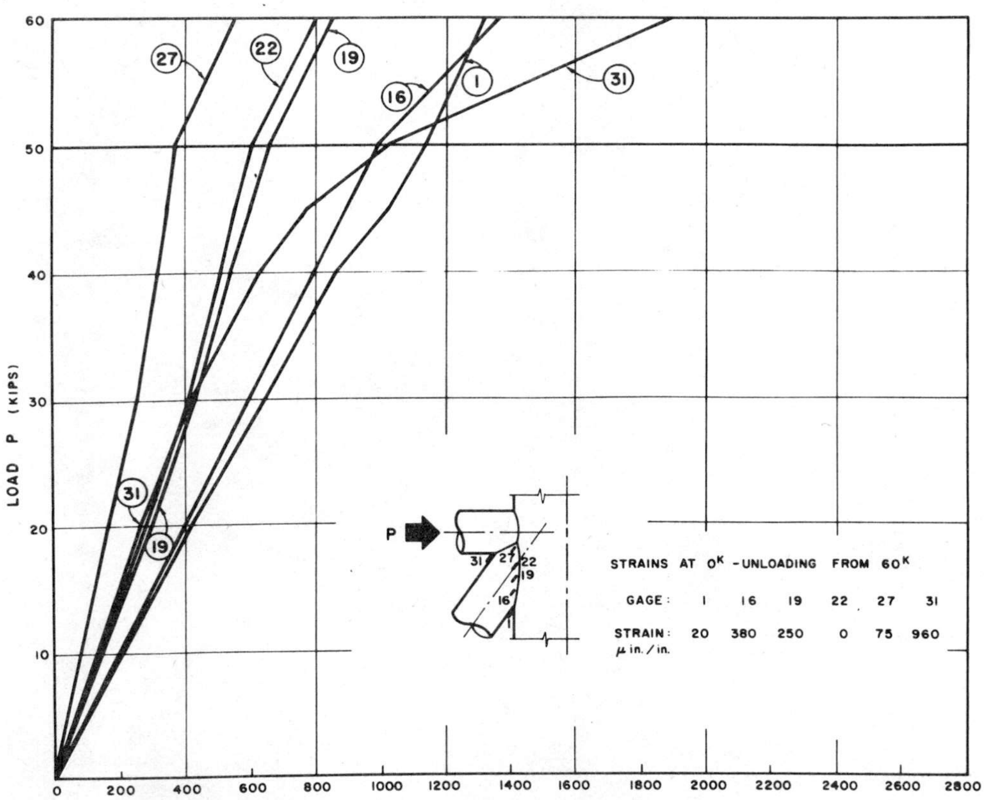


FIG. 31 PRINCIPAL STRESSES - JOINT TYPE 2 - THICK WALL

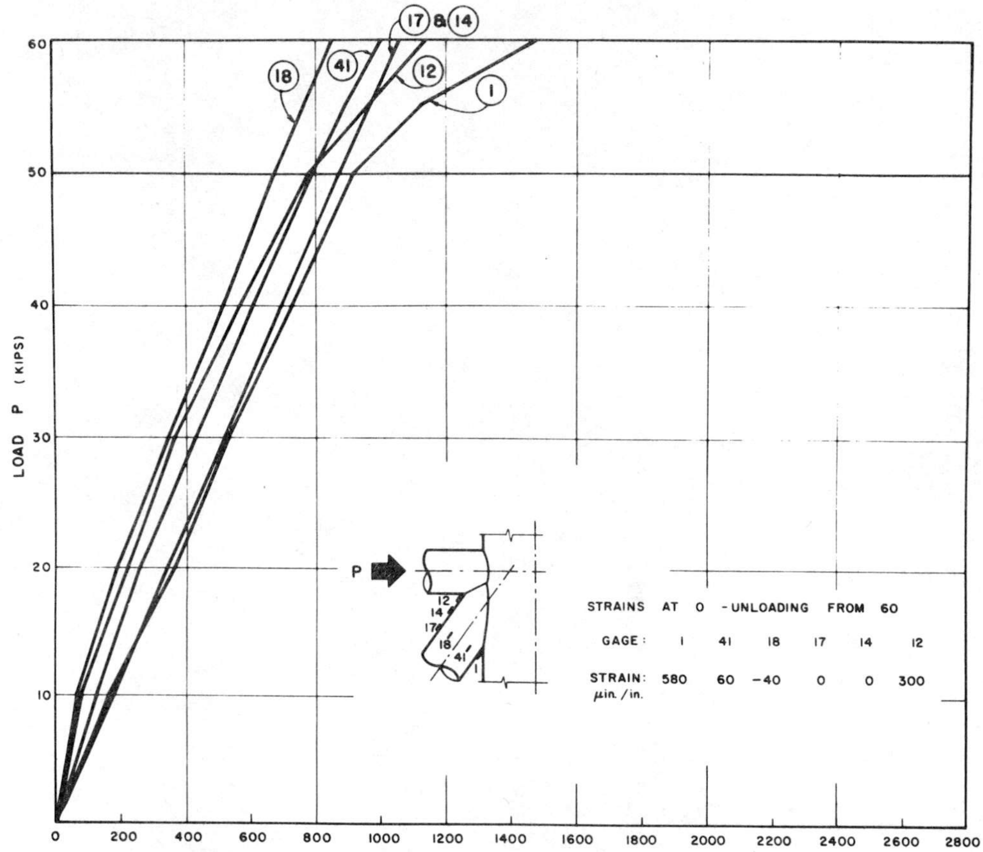


(a) THIN WALL

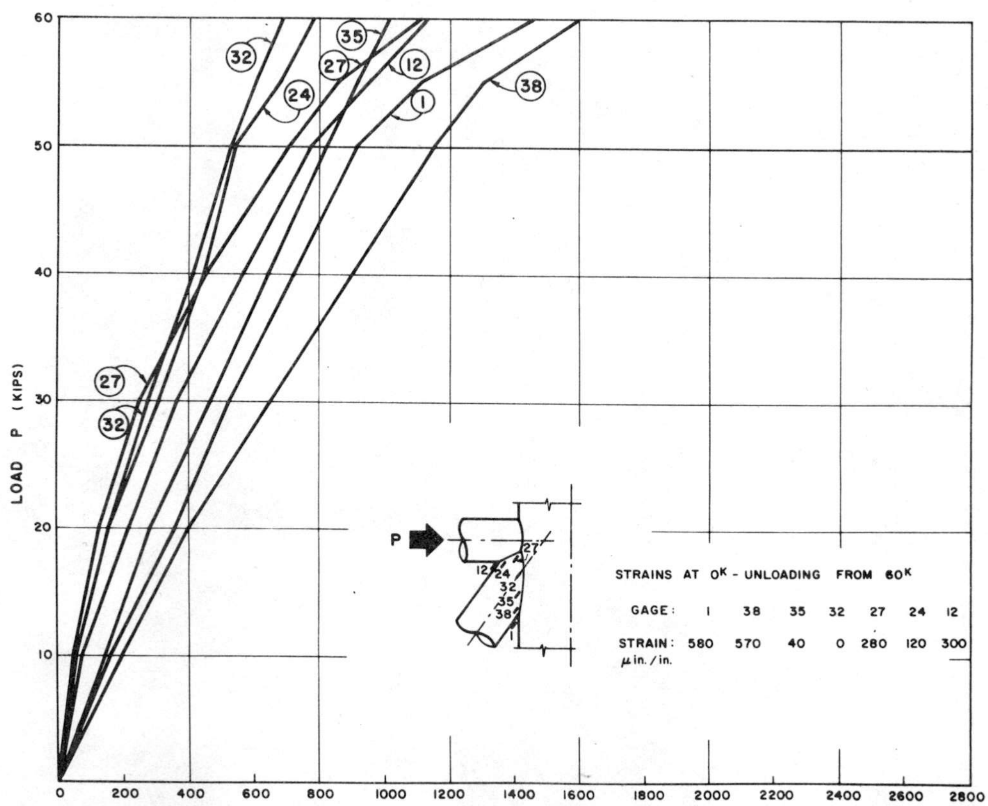


(b) THIN WALL

FIG. 32 STRAINS IN DIAGONAL : JOINT TYPE 2



(a) THICK WALL



(b) THICK WALL

FIG. 33 STRAINS IN DIAGONAL ; JOINT TYPE 2

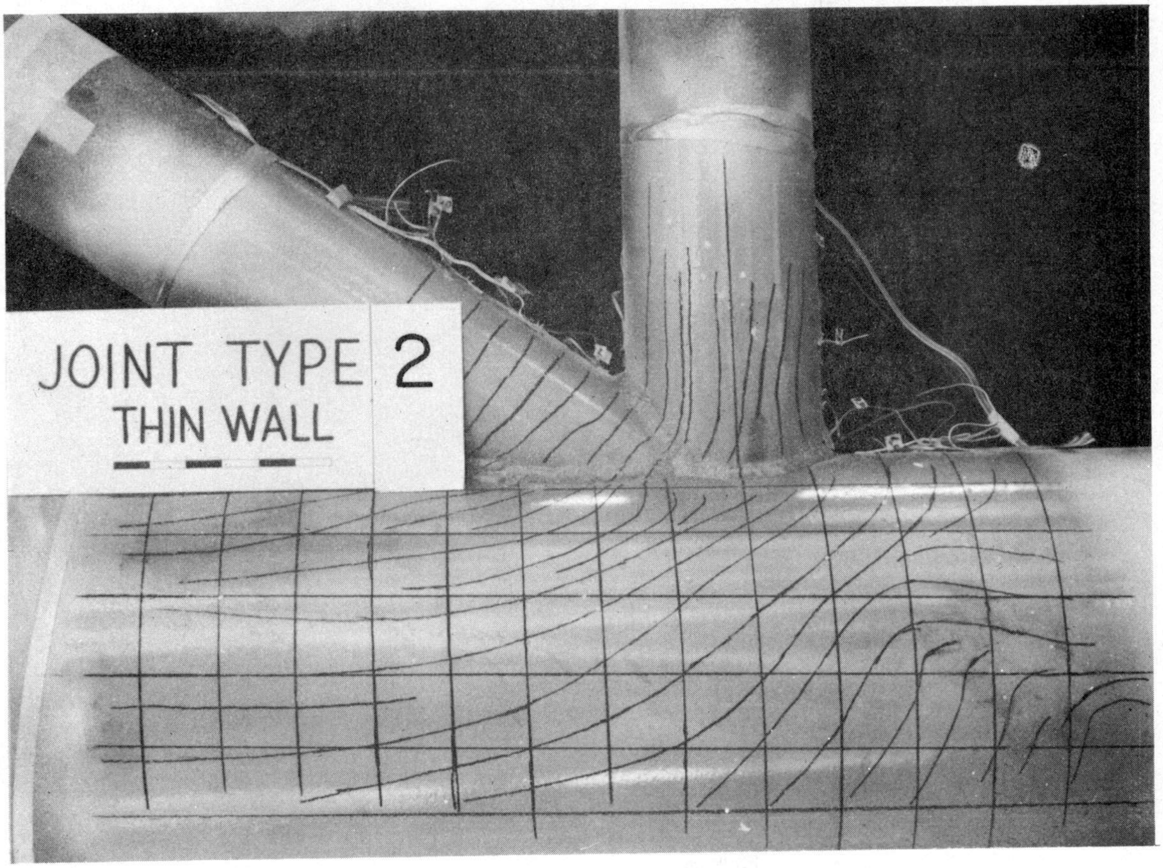


FIG. 34 STRESS-COAT PATTERN
JOINT TYPE 2 - THIN WALL

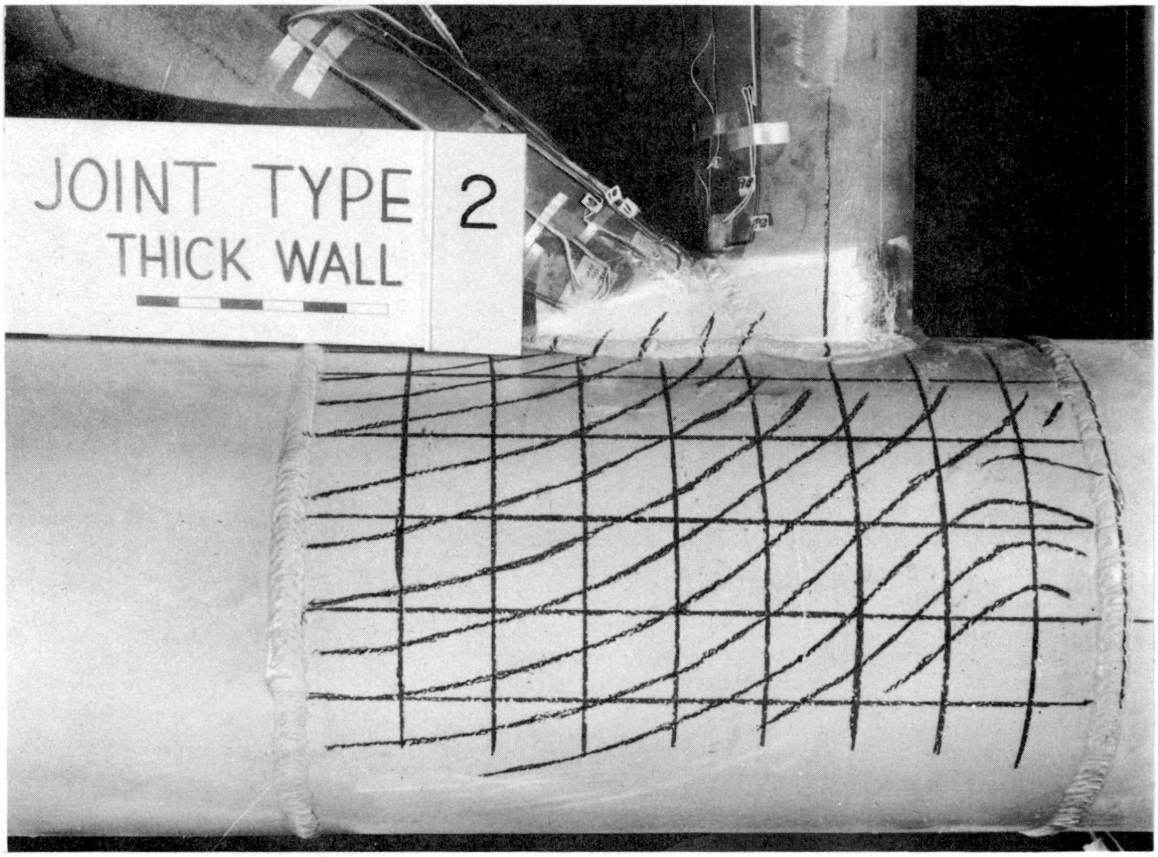


FIG. 35 STRESS-COAT PATTERN
JOINT TYPE 2 - THICK WALL

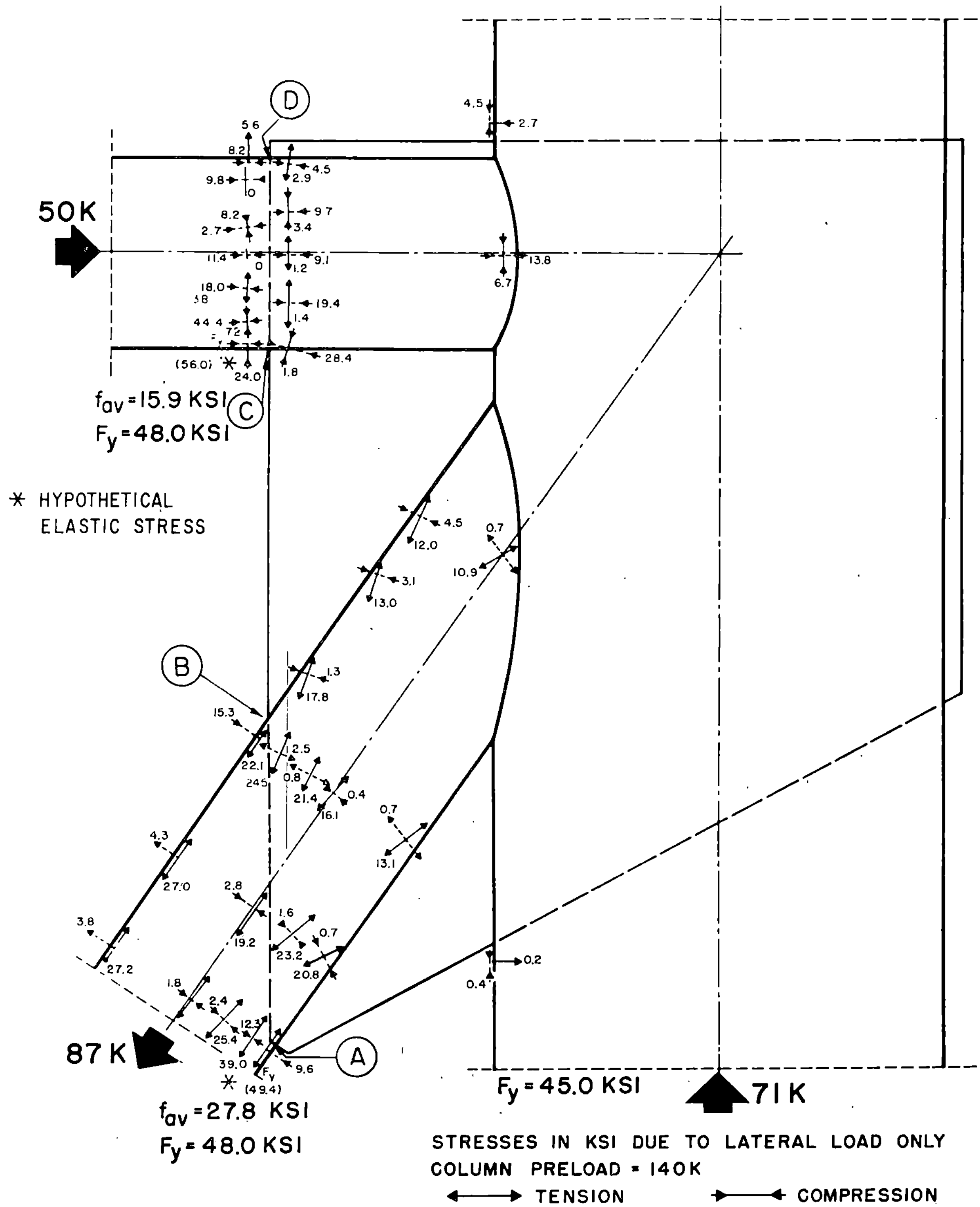


FIG. 36 PRINCIPAL STRESSES - JOINT TYPE 3 - GUSSET PLATE

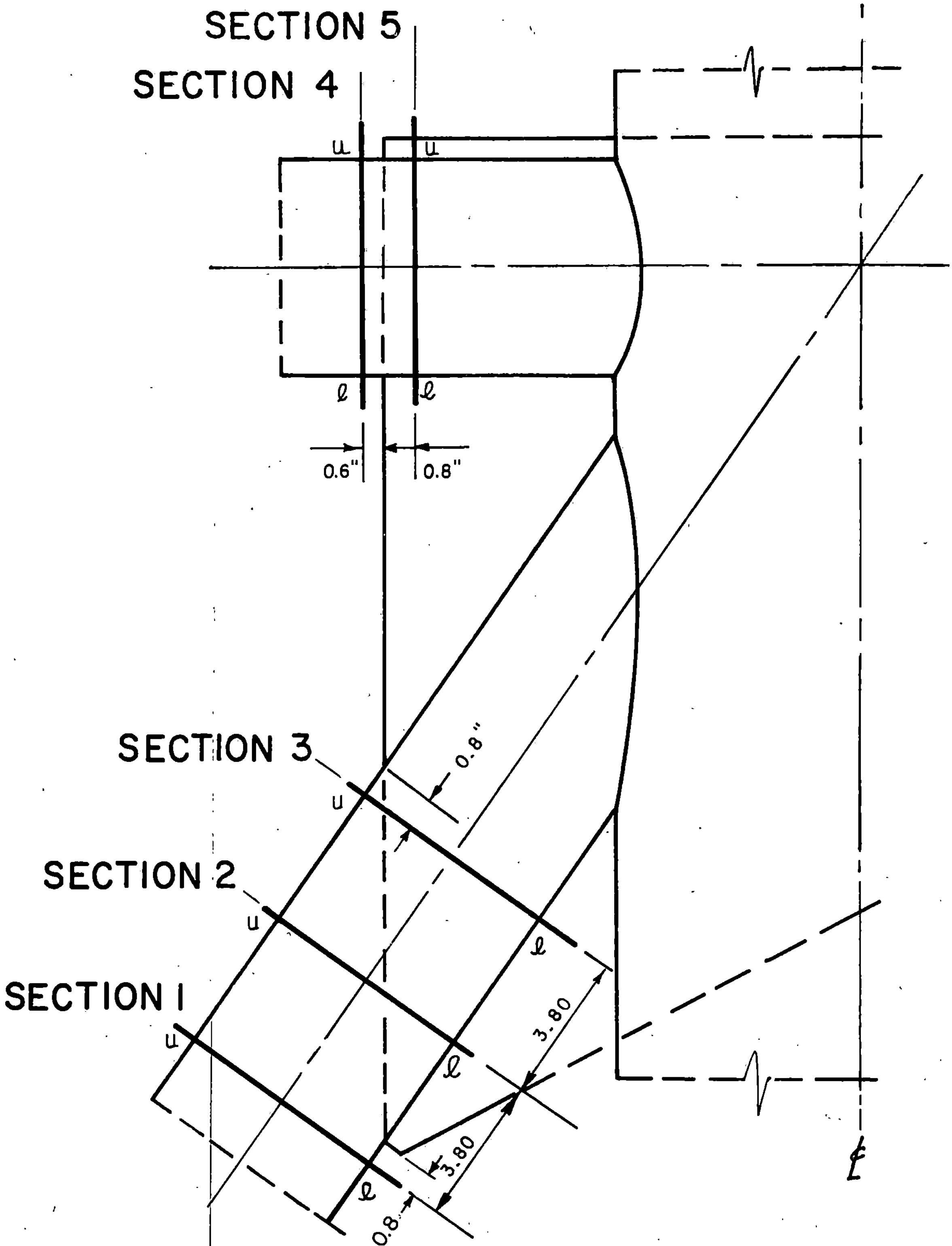
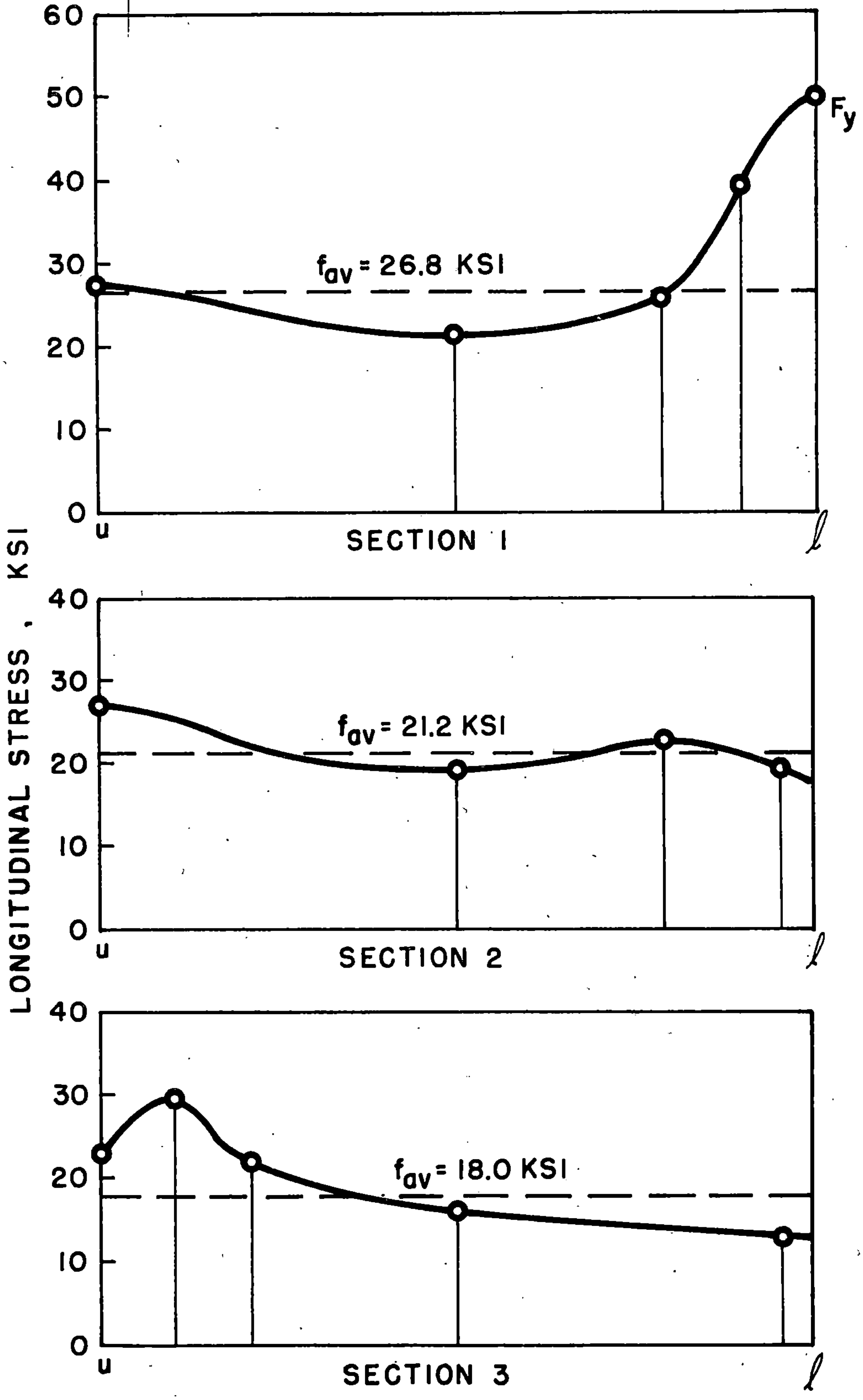
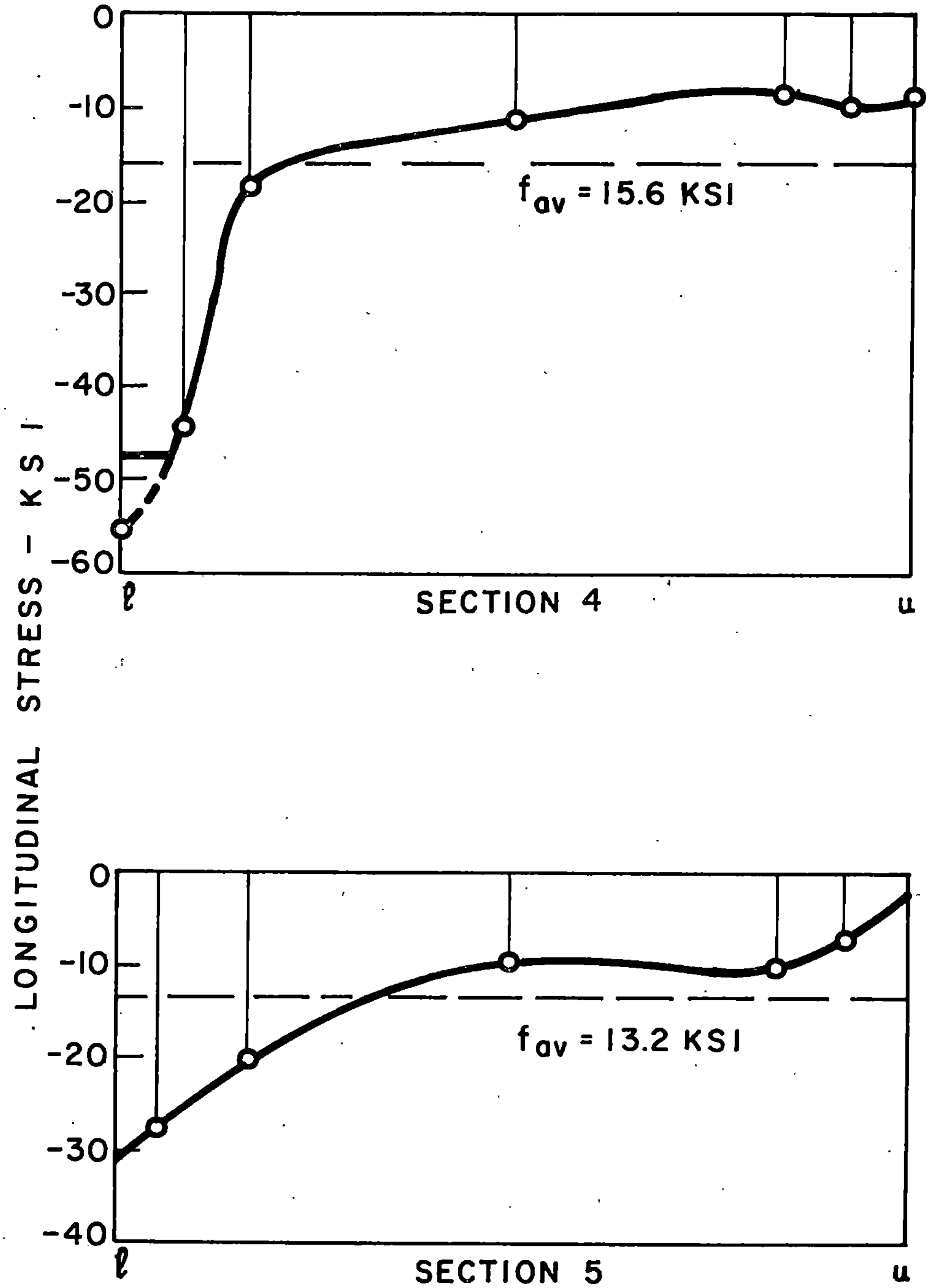


FIG. 37 LOCATIONS OF INSTRUMENTED SECTIONS
JOINT TYPE 3



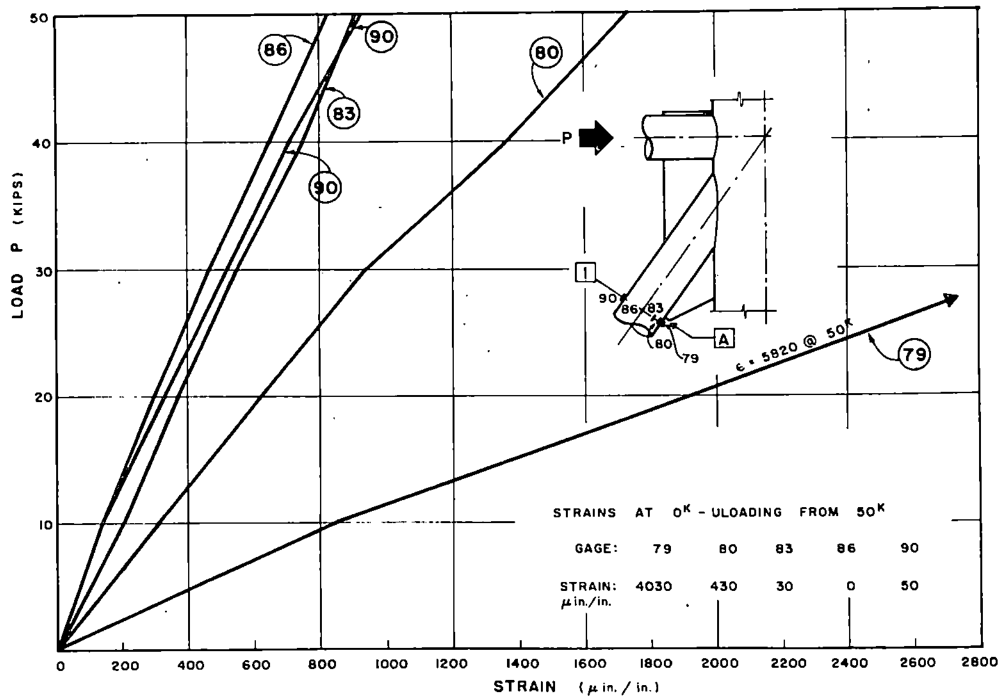
Points u, l correspond to those indicated in FIG. 37

FIG. 38 LONGITUDINAL STRESSES ALONG SECTIONS 1, 2 AND 3 (DIAGONAL TUBE)

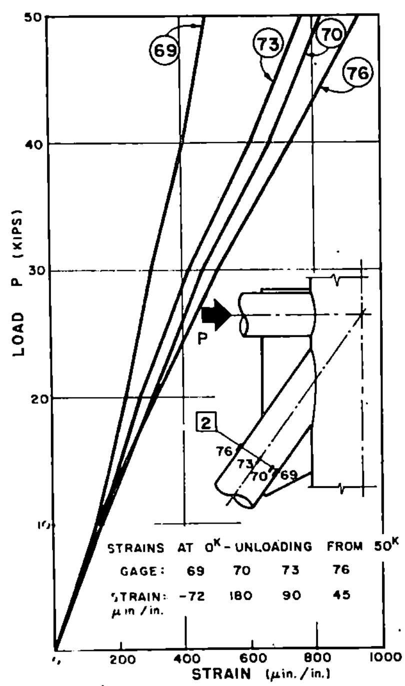


Points u, l correspond to those indicated in FIG. 37

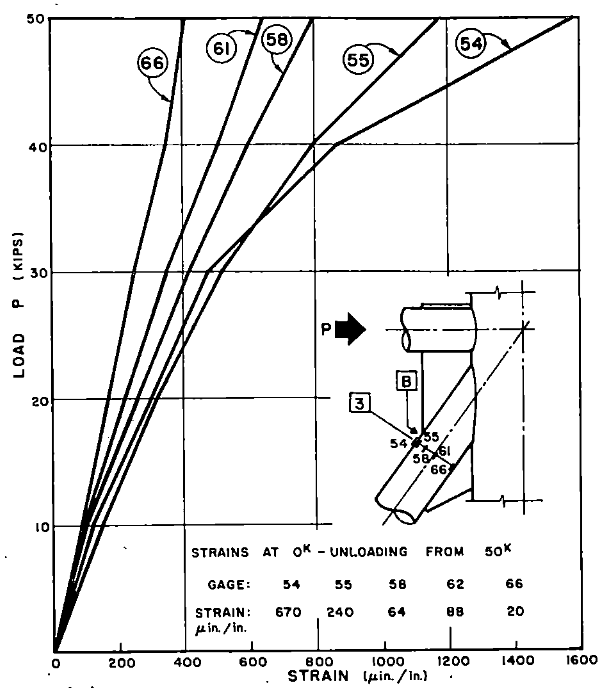
FIG. 39 LONGITUDINAL STRESSES ALONG SECTIONS 4 AND 5 (HORIZONTAL TUBE)



(a) SECTION 1



(b) SECTION 2



(c) SECTION 3

FIG. 40 STRAINS IN DIAGONAL; SECTIONS 1, 2 AND 3
JOINT TYPE 3

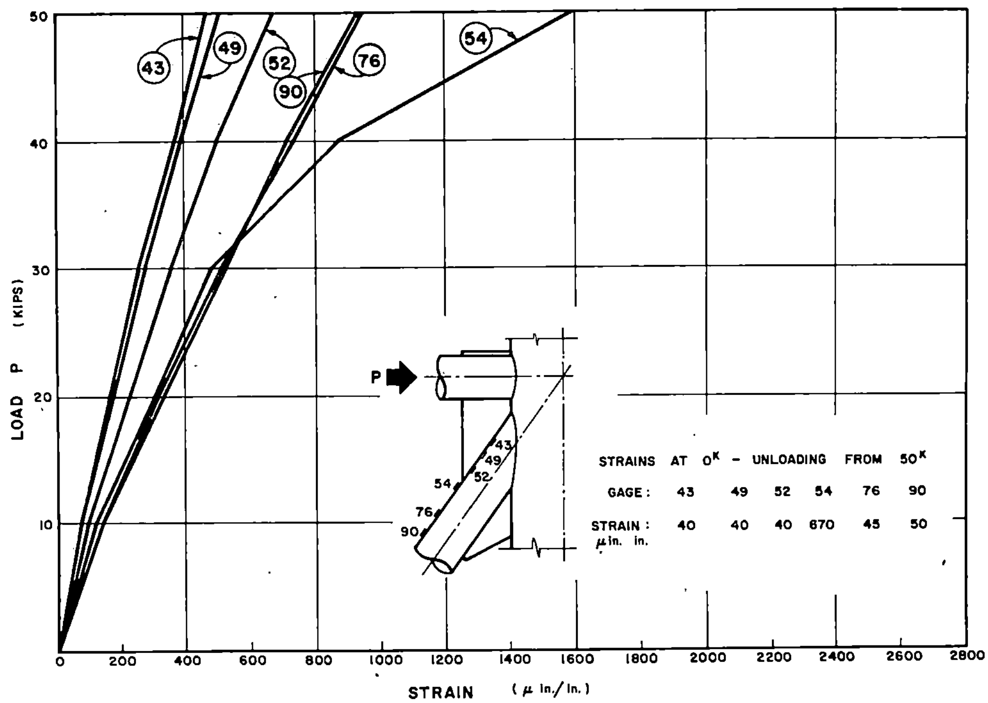


FIG. 41 STRAINS IN DIAGONAL; JOINT TYPE 3

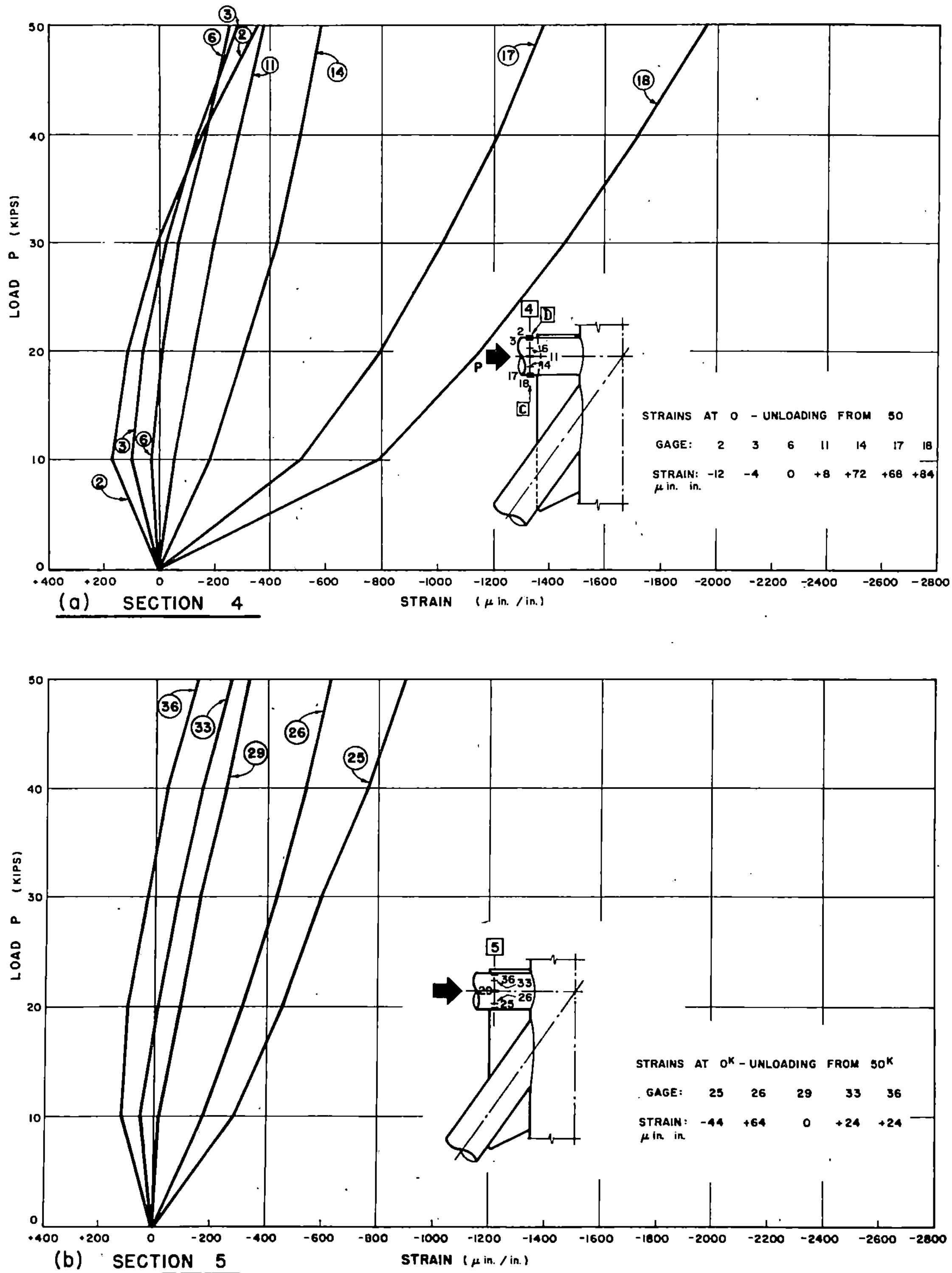


FIG. 42 STRAINS IN HORIZONTAL — SECTIONS 4 AND 5
JOINT TYPE 3

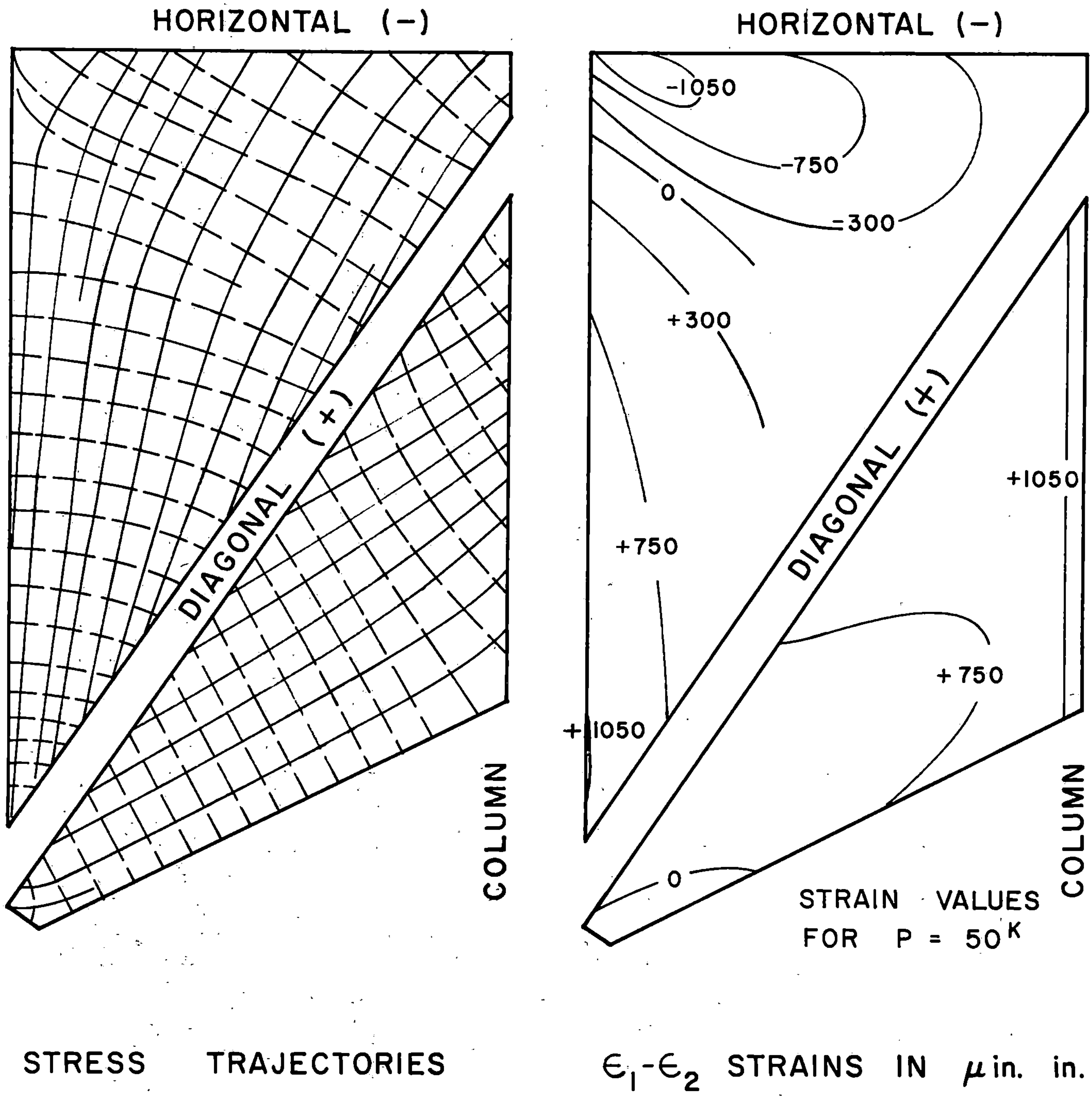


FIG. 43 RESULTS PHOTO-STRESS STUDY FOR GUSSET PLATE

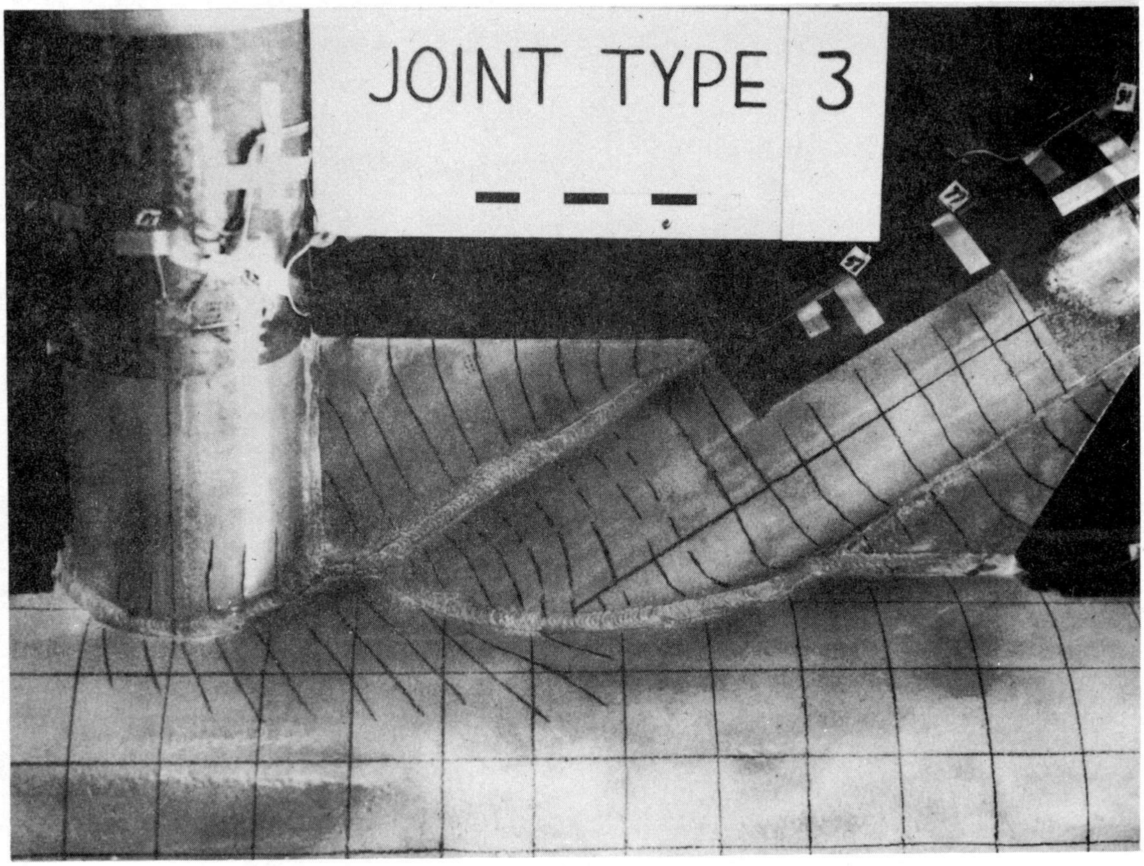
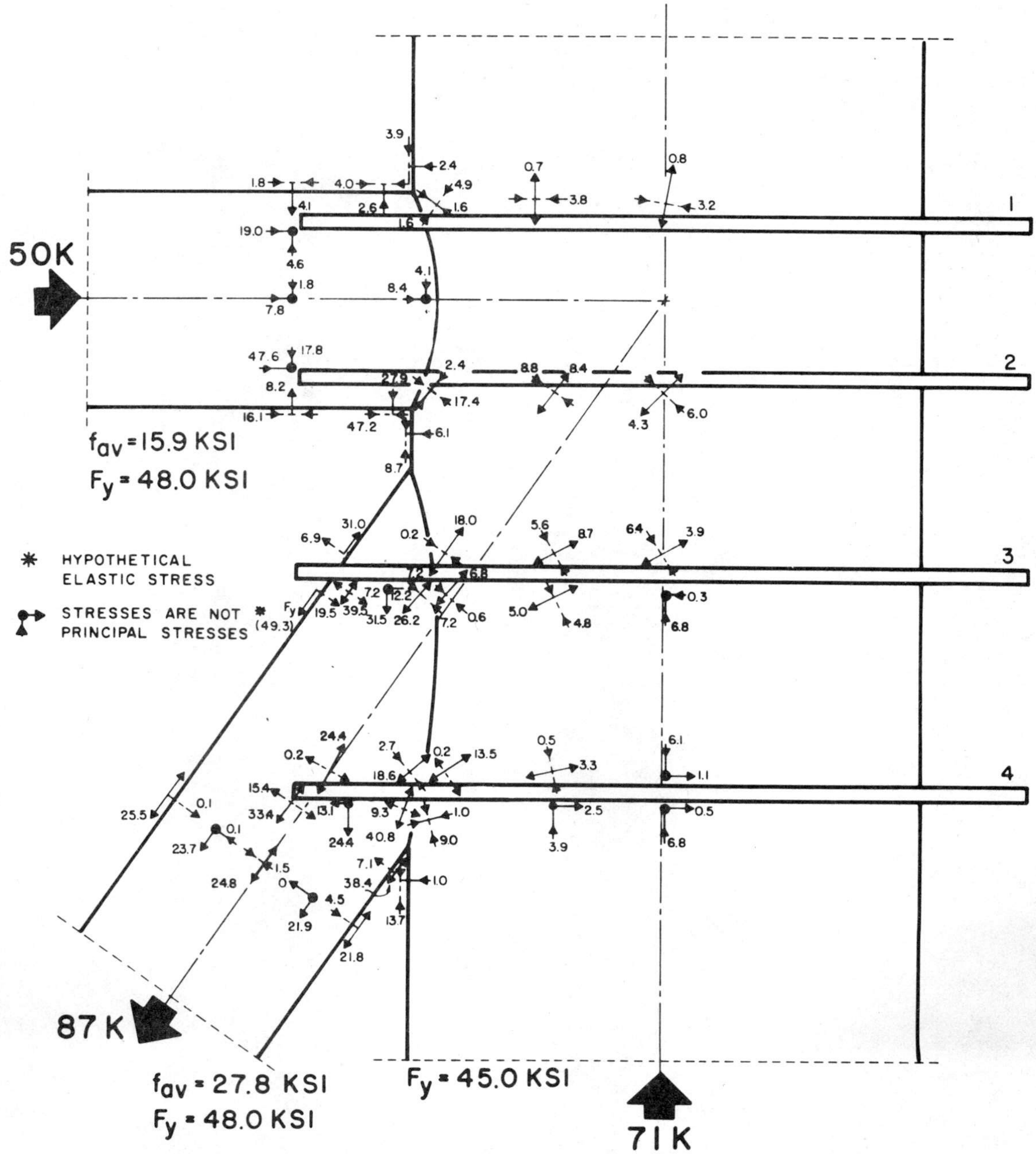


FIG. 44 STRESS-COAT PATTERN
JOINT TYPE 3



STRESSES IN KSI DUE TO LATERAL LOAD ONLY

COLUMN PRELOAD = 140 K

TENSION COMPRESSION

FIG. 45 PRINCIPAL STRESSES - JOINT TYPE 4

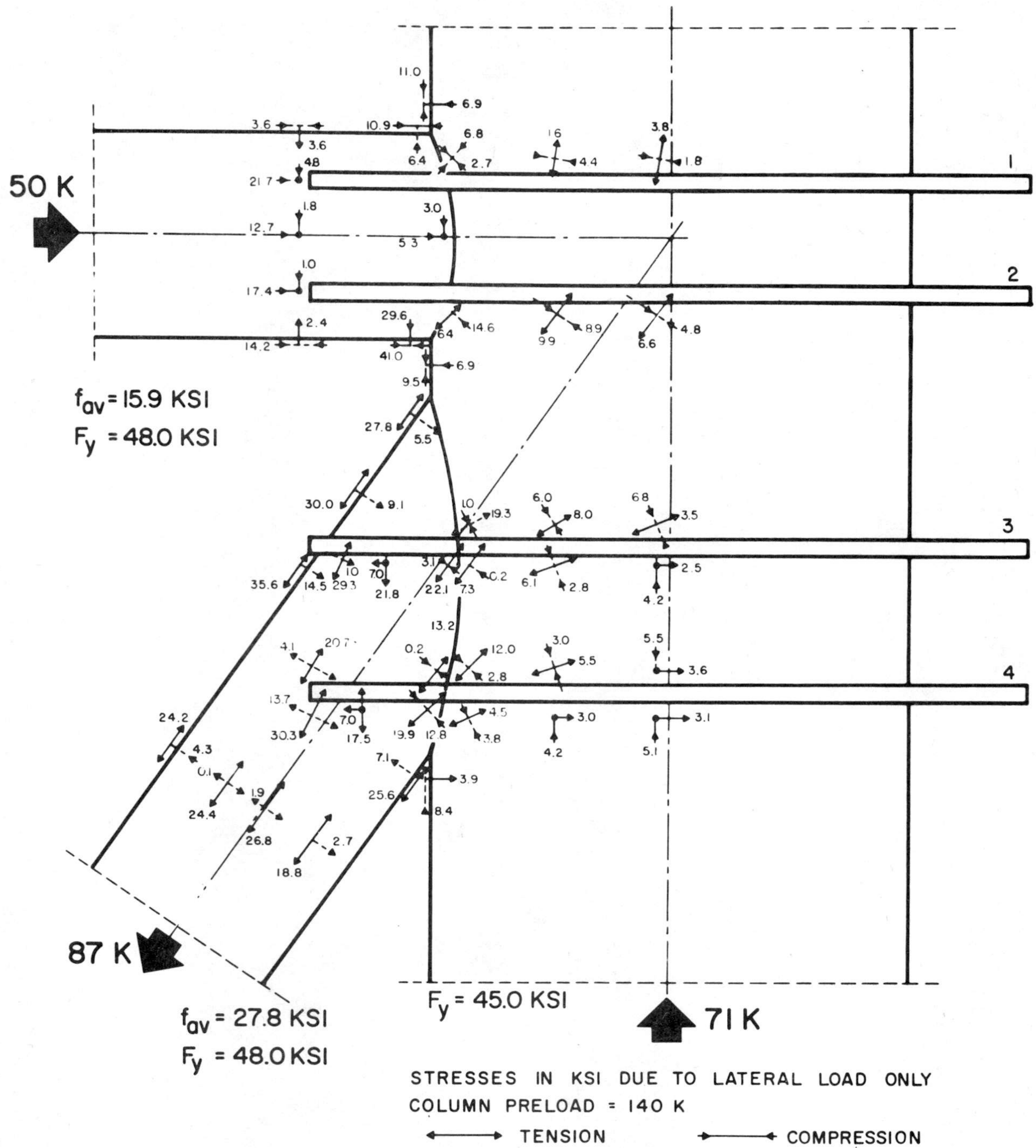


FIG. 46 PRINCIPAL STRESSES - JOINT TYPE 4A

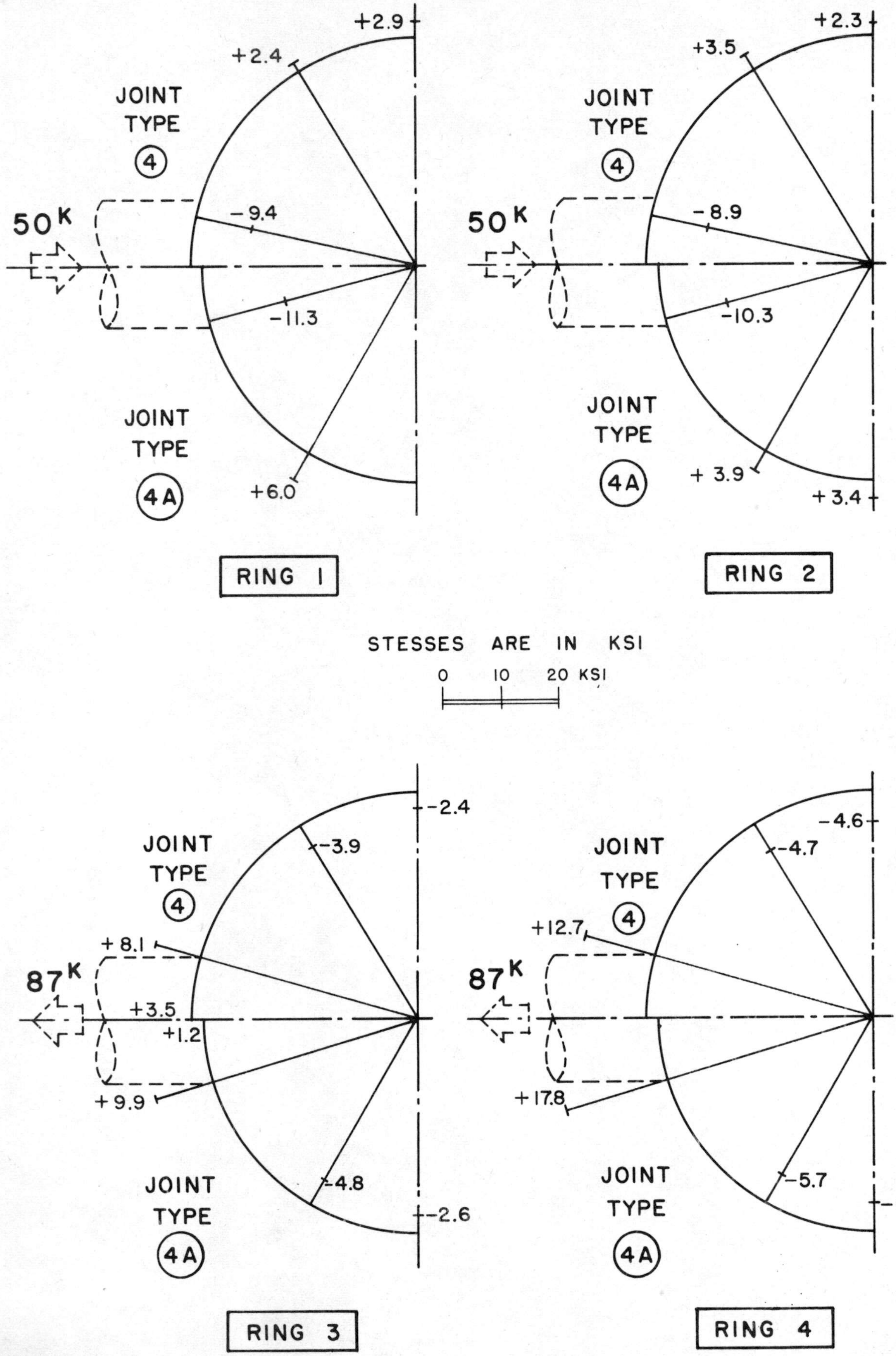


FIG. 47 TANGENTIAL STRESSES IN STIFFENING RING JOINT TYPE 4 & 4A

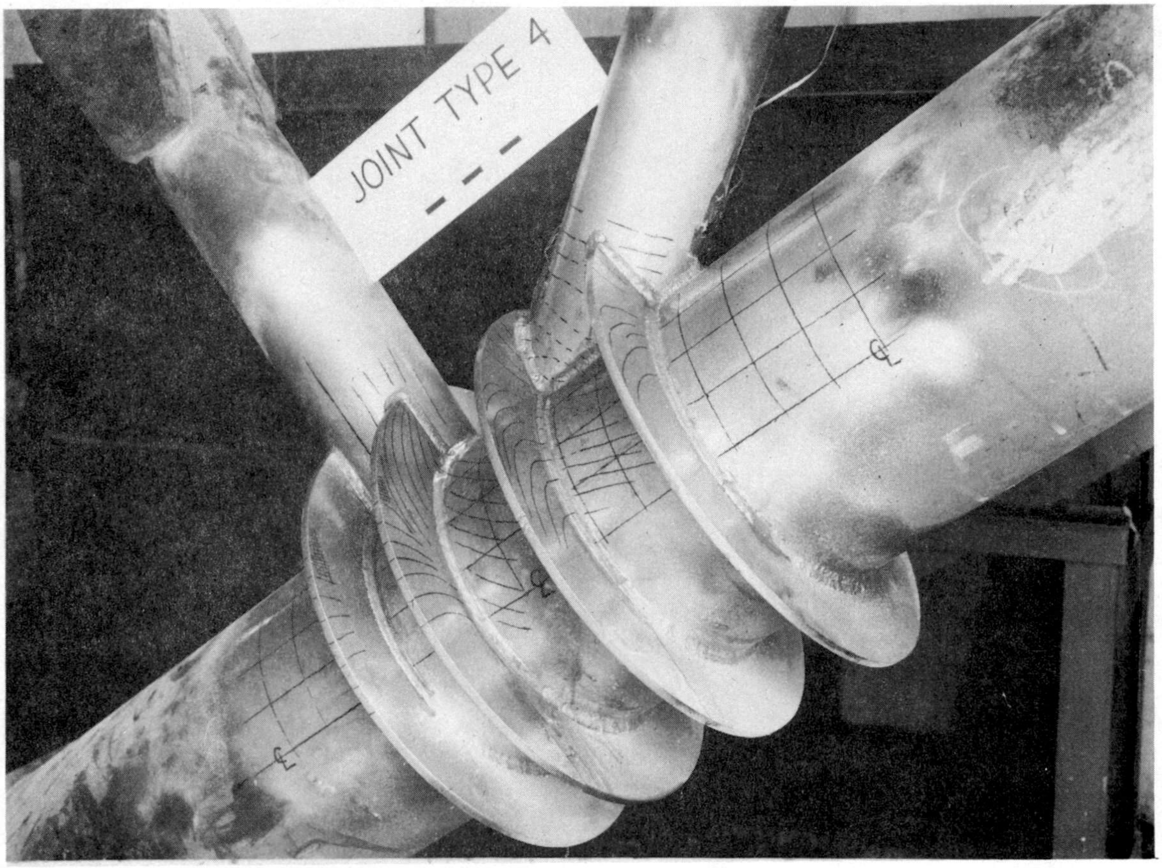


FIG. 48 STRESS-COAT PATTERN
JOINT TYPE 4

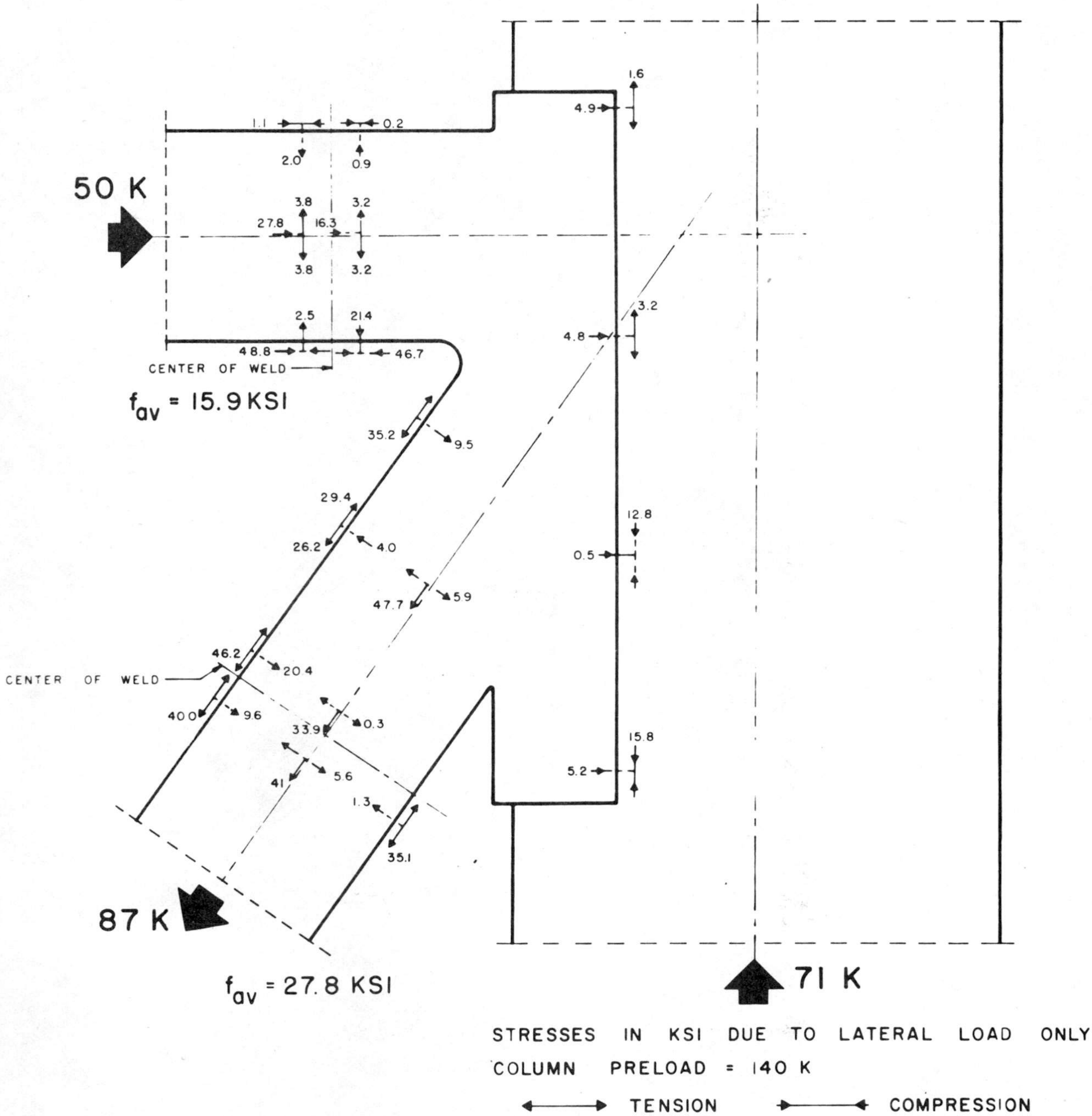
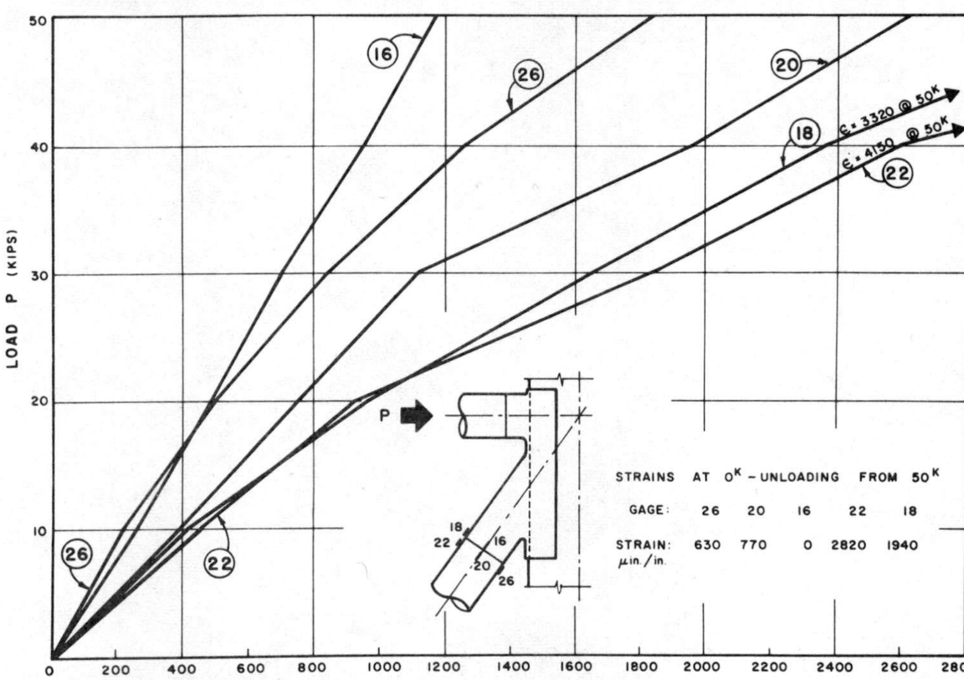
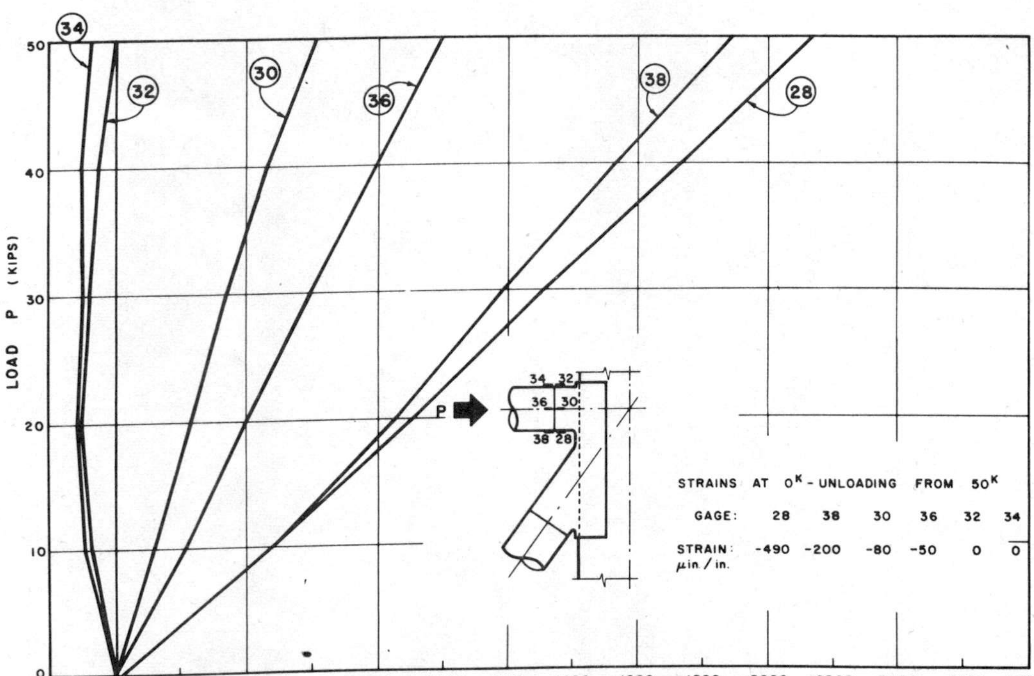


FIG. 49 LONGITUDINAL AND RADIAL STRESSES
JOINT TYPE 5



(a) STRAINS IN DIAGONAL



(b) STRAINS IN HORIZONTAL

FIG. 50 STRAINS IN WEB MEMBERS ; JOINT TYPE 5

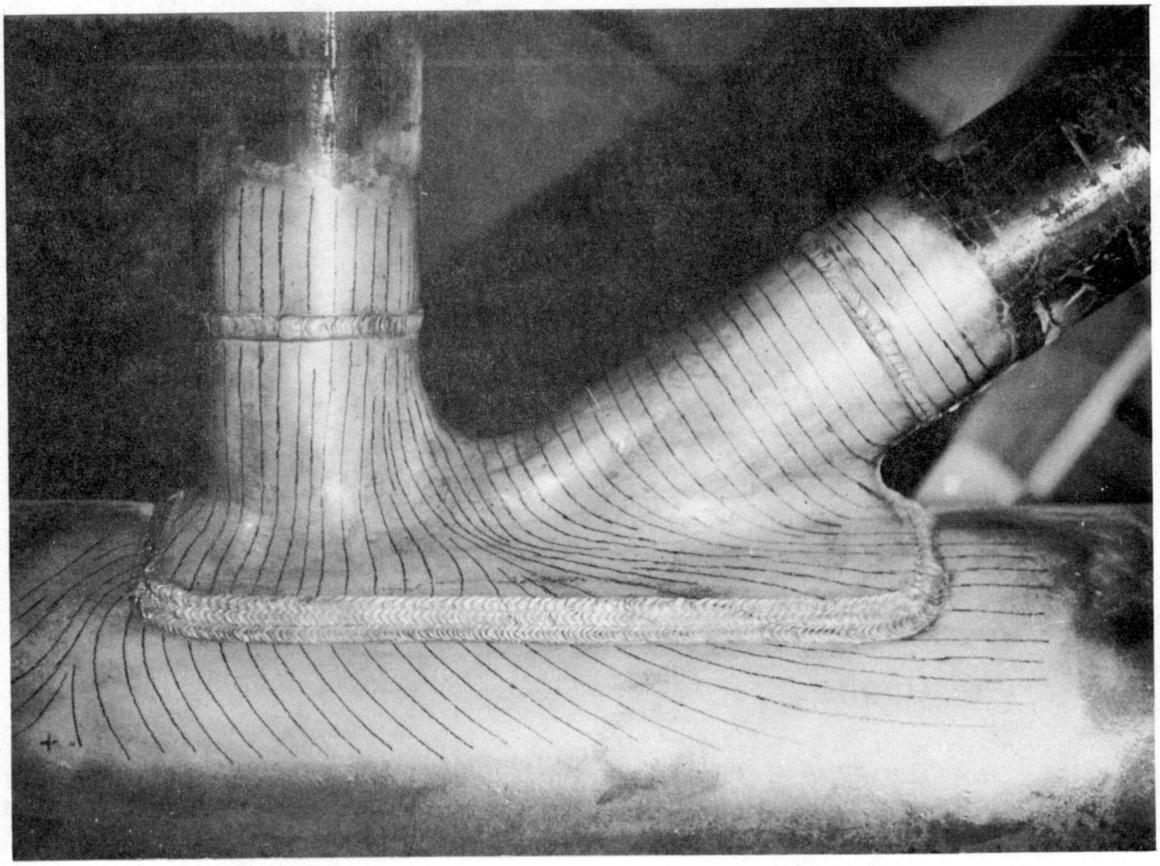


FIG. 51 STRESS-COAT PATTERN
JOINT TYPE 5

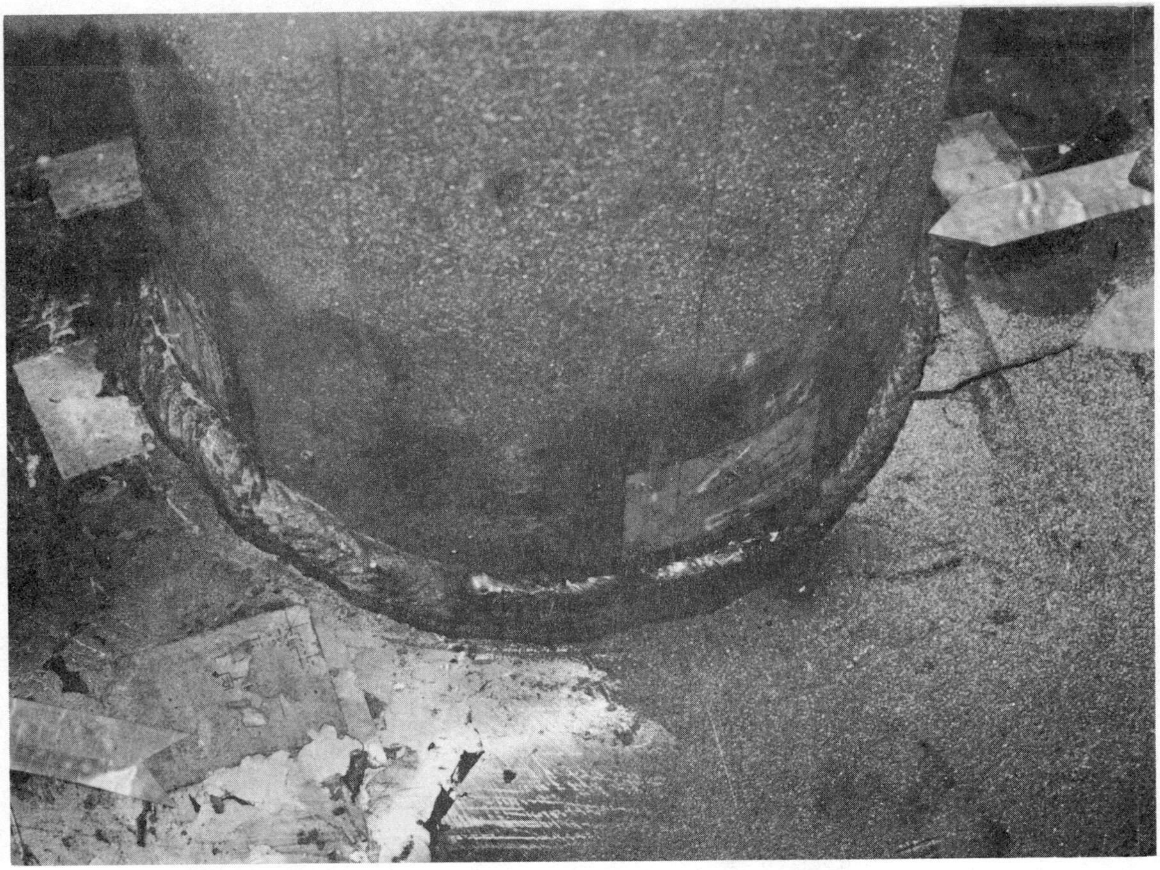


FIG. 52 JOINT TYPE I - THIN WALL
AFTER FAILURE (n = 450 CYCLES)

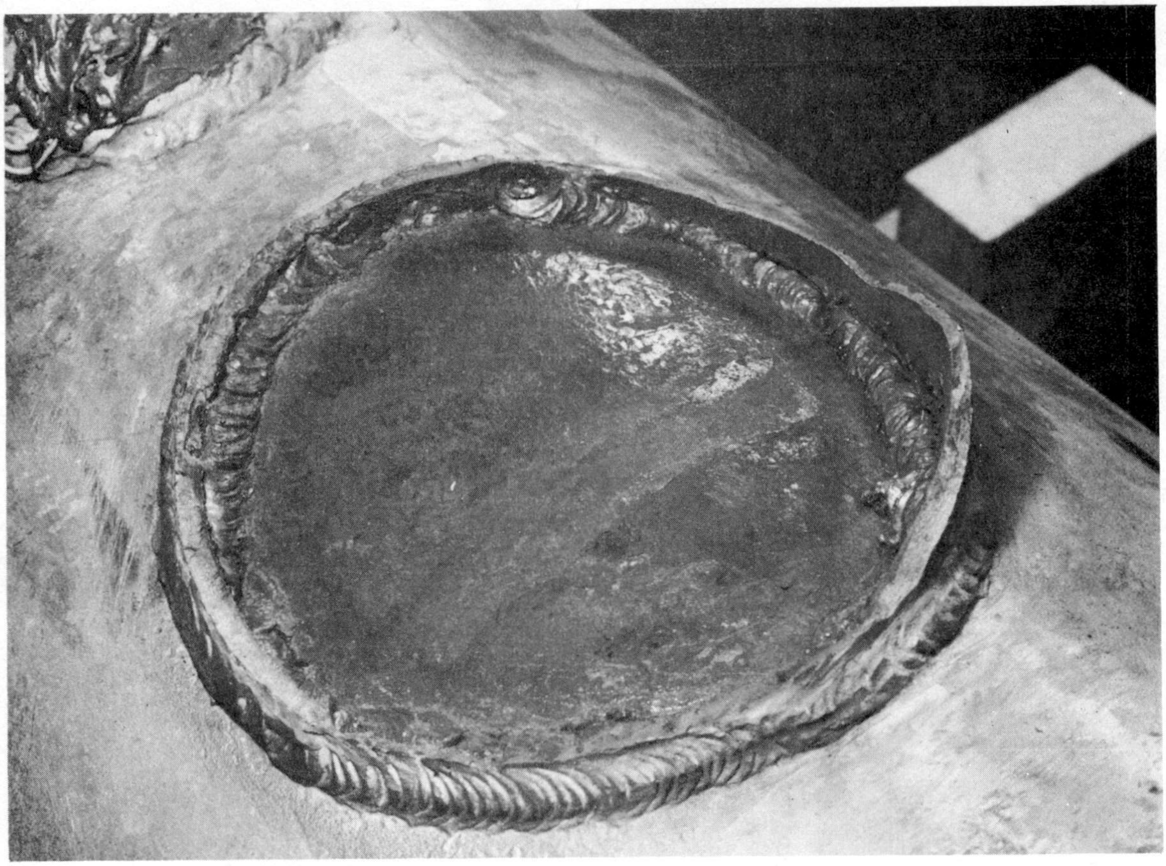


FIG. 53 JOINT TYPE I - THICK WALL
AFTER FAILURE (n = 9900 CYCLES)

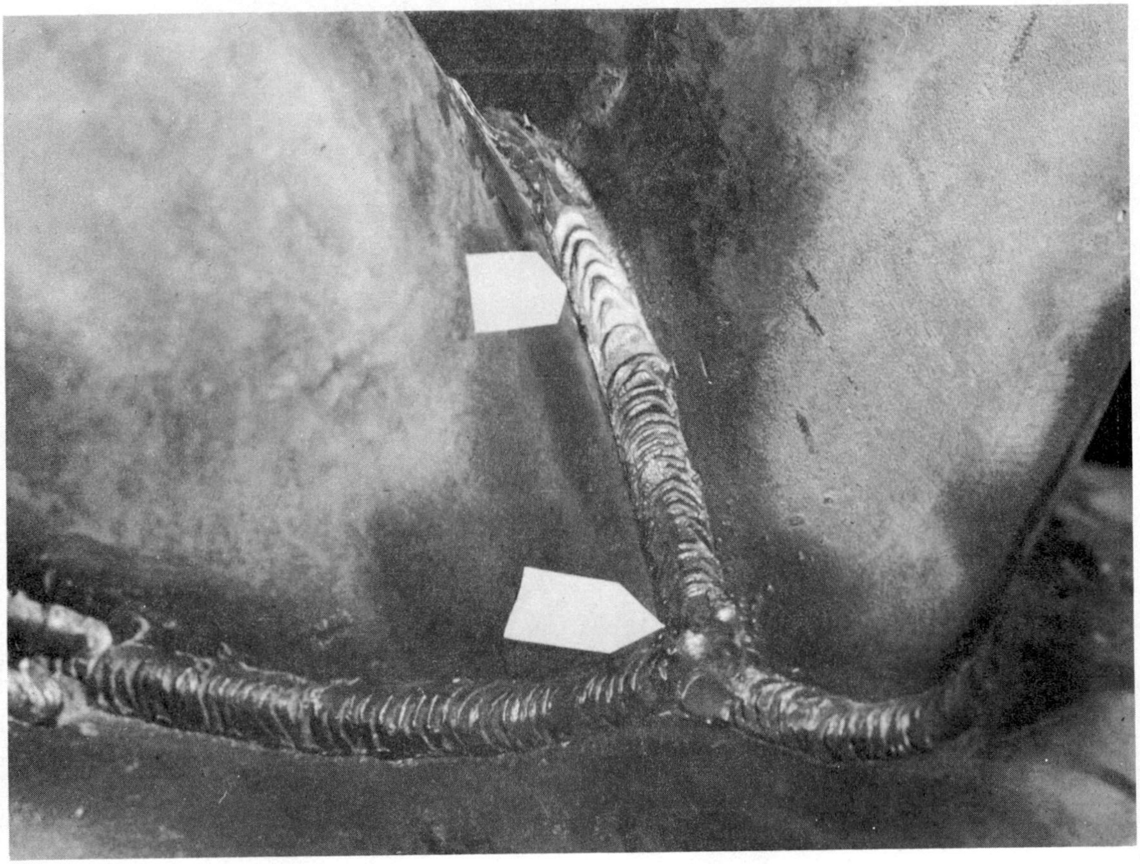


FIG. 54 JOINT TYPE 2 - THIN WALL

AFTER FAILURE (n = 4000 CYCLES)

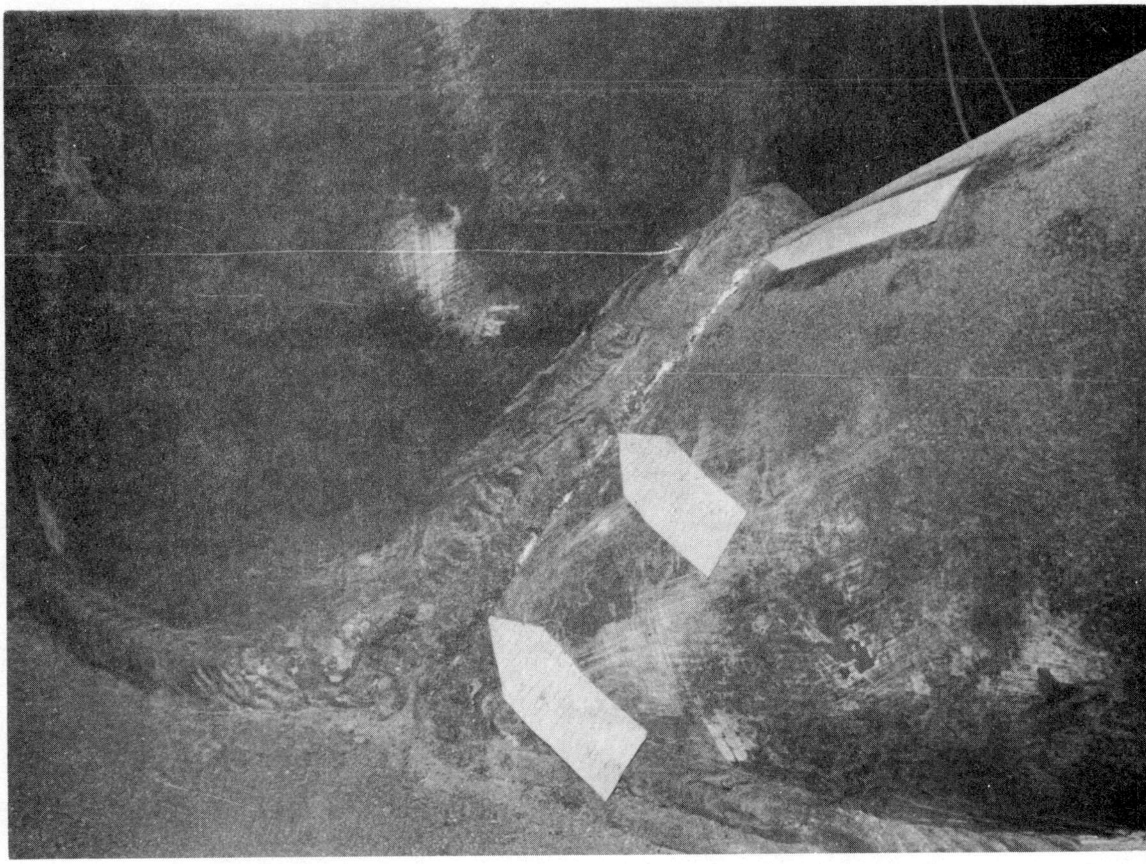


FIG. 55 JOINT TYPE 2 - THICK WALL

AFTER FAILURE (n = 7500 CYCLES)

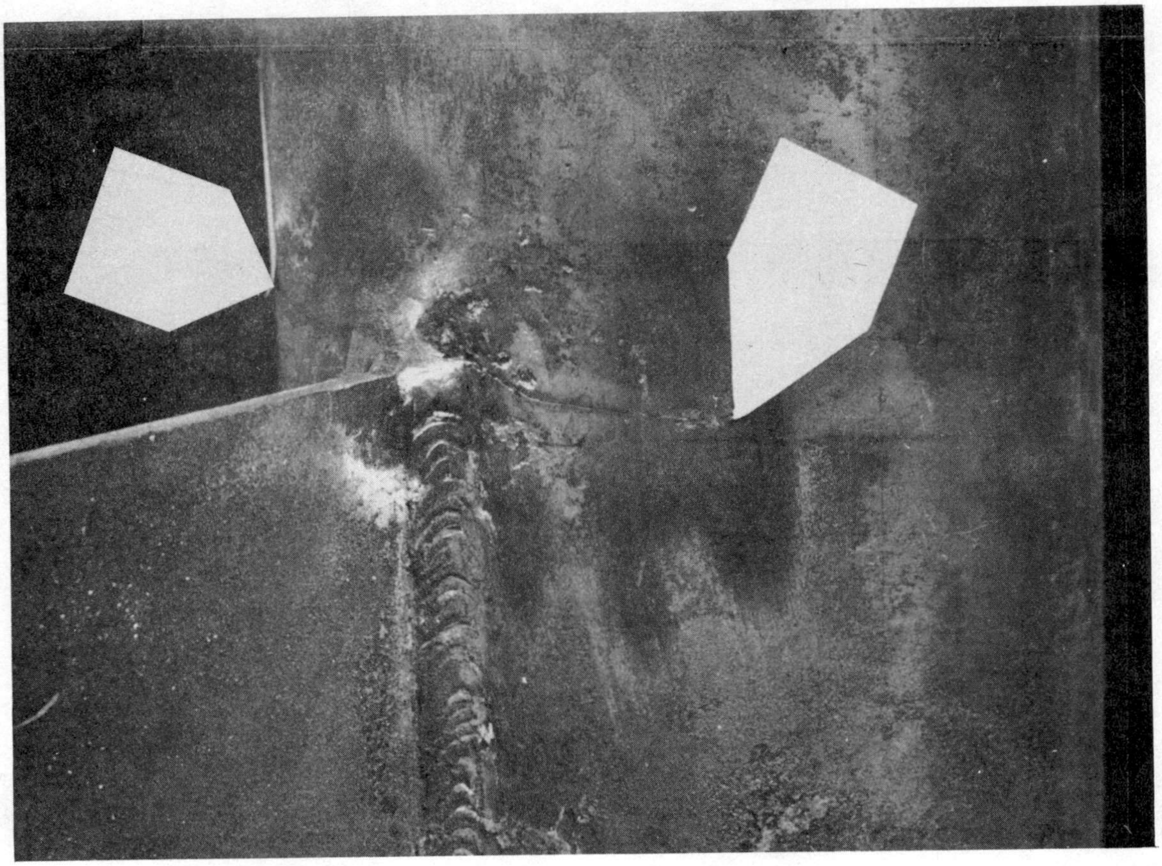


FIG. 56 JOINT TYPE 3 AFTER FAILURE

(n = 7000 CYCLES)

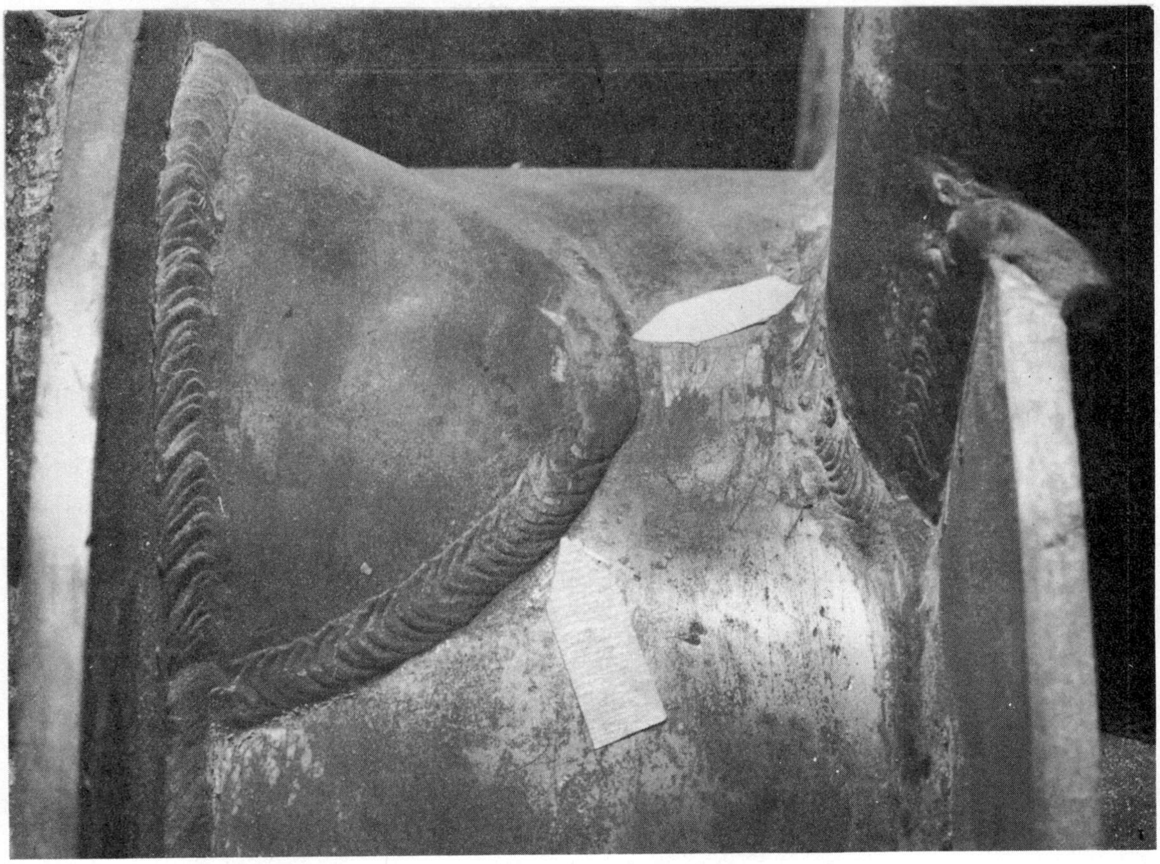


FIG. 57 JOINT TYPE 4 AFTER FAILURE

(n = 5500 CYCLES)

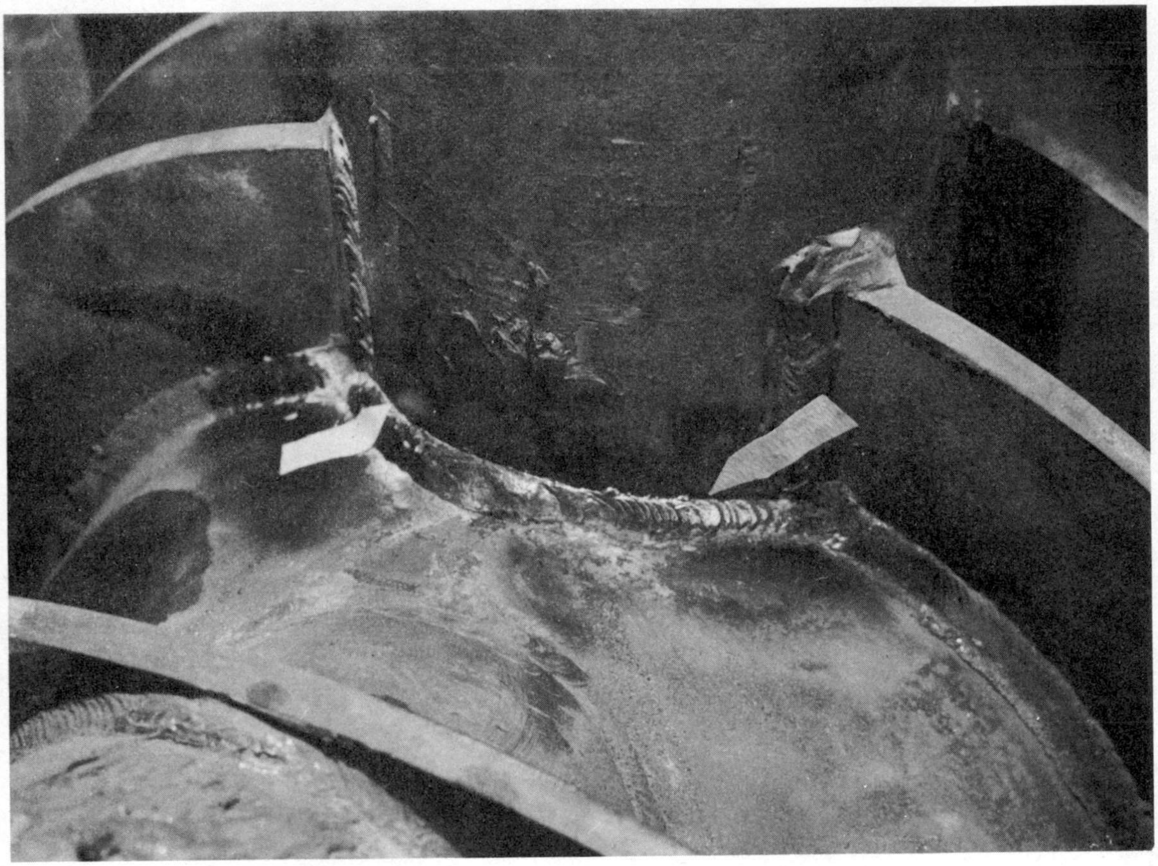


FIG. 58 JOINT TYPE 4A AFTER FAILURE

(n = 7720 CYCLES)