1969

Proof-tests of two slender wed welded plate girders. May 1969 (patterson m.s. Thesis) (70-17)

P. J. Patterson

J. A. Corrado

J. S. Huang

B. T. Yen

Welded Plate Girders - Design Recommendations

PROOF-TESTS OF TWO SLENDER-WEB WELDED PLATE GIRDERS

P. J. Patterson
J. A. Corrado
J. S. Huang
B. T. Yen

Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

May 1969

Fritz Engineering Laboratory Report No. 327.7

TABLE OF CONTENTS

ABSTRACT

This report describes the testing of two full-size slender-web welded plate girders for the purpose of ascertaining the acceptability of proposed design recommendations for bridge girders under repeated loading. The girders were 32'-6" and 35'-3" long with one-quarter inch webs, resulting in nominal web slenderness ratios of 200 and 380. Panel aspect ratios ranged from 1.0 to 1.50. ASTM A36 material was used. These girders were subjected to repeated loading to 2.2 and 4.5 million cycles respectively, without development of fatigue cracks and were subsequently tested under static loads to determine the adequacy of the stiffeners. It is concluded that the proposed recommendations are sufficiently conservative for ordinary bridge plate girders.

1. INTRODUCTION

1.1 Background

The present effort is part of a continuing study of the behavior of slender-web welded plate girders. Many contributions have been made in this field, both in Europe and in this country, with studies having been in progress at Lehigh University since 1957.

For many years the attainment of the critical buckling stress in the web was considered to be the limit of structural usefulness for girders. As evidenced by many of the nearly three hundred entries in an early literature survey,(1) work progressed toward a high degree of development in the predicting of this critical stress.

The linear buckling theory having been finally demonstrated to give an overly conservative estimate of the static strength of slender-web plate girders,(1,2,3,4) design practice moved forward early in this decade to take advantage of inherent post-buckling strength.(5)

A question arose at this point as to whether or not these advances in the knowledge of girder behavior could be applied in the case of repeated loading, that is, for bridge girders.(6) To answer this question and to study the behavior of slender girders under repeated loading, especially the effect of out-of-plane web deflections, a testing program was begun at Lehigh University in 1960. Fatigue tests of nine large-size welded plate girders were carried out under loading conditions of bending, shear, and combined bending and shear.[7] From the results of these and other tests,[8,9] plate bending stresses due to repeated lateral web deflections were concluded to be a primary cause of fatigue cracks along web boundaries.[10] In addition, a method was developed for estimating these stresses from measured web deflections.

Based on these fatigue studies, design recommendations have been formulated for plate girders subjected to repeated loading.[11,12] The proposed limits on web slenderness ratio (β) are:

$$\beta \leqq 36,500/\sqrt{F_y} \tag{1}$$

for transversely stiffened girders, and

$$\beta \leqq 73,000/\sqrt{F_y} \tag{2}$$

for longitudinally stiffened girders.

For A36 steel, with a yield point of F_y = 36 ksi, these limits are 190 and 380 respectively. Panel aspect ratios are limited to 1.5 in either case. For transverse stiffeners, the area required to anchor the tension field is a function of yield stress, type of stiffener, the clear unsupported distance between flange components, the web thickness and the applied shear, and has been proposed to be:

$$A = \left[0.15\ \xi D \cdot t \left(1 - \frac{\tau_{cr}}{\tau_y}\right) \frac{V}{V_u} - 18t^2\right] \frac{F_{yw}}{F_{ys}} \quad (3)$$

where ξ = a coefficient of stiffener geometry

D = web depth

t = web thickness

τ_{cr}/τ_y = ratio of buckling stress to yield stress in shear

V/V_u = ratio of applied shear to shear strength

$F_{yw}\ F_{ys}$ = ratio of yield stress, web to stiffener

In the case of longitudinal stiffeners, the primary functions are to control lateral web deflections in bending by the formation of a nodal line, and to help anchor the tension field. In order to ensure the fulfillment of these functions, a proposed minimum radius of gyration is

$$r \geq \frac{d_o \sqrt{F_y}}{23{,}000} \quad (4)$$

where d_o = distance between transverse stiffeners.

1.2 Purpose and Scope

To insure the acceptability of the recommendations, a series of "proof-tests" were carried out at Fritz Engineering Laboratory in the summer and fall of 1968. Two large size welded plate girders were designed according to these recommendations with the exception that the AASHO minimum web thickness requirement (13) was intentionally violated, focusing attention on lateral web deflections and giving a more severe test of the proposed recommendations.

Two parameters that were of particular interest in this investigation are the web slenderness ratio β, and the panel aspect ratio α. In particular, what kind of performance under repeated loading could be expected from girder web panels with α and β at the allowable upper limits? These tests were run not only in fatigue, but were also carried out statically to determine the ultimate load of the individual panels.

Another item of interest was the size and arrangement of the stiffeners, both longitudinal and transverse. To be studied were the adequacy of one-sided stiffeners and the behavior of stiffener intersections having the longitudinal stiffener connected to or cut short from the transverse ones. Most of the stiffeners were instrumented to provide an indication of the stress development in the stiffeners and the distribution of stresses when an adjacent panel was loaded to its maximum capacity.

A further objective was the collection of additional web deflection data for the computation of web boundary stresses, so that an S-N curve could be established more precisely. It was anticipated that tests performed on these girders would provide some information for the modification, if necessary, of the proposed design recommendations. In this report are presented the essential data of the proof-testing and the discussion of the results. A more complete discussion of the behavior of slender web girders in general may be found in Ref. 7 and 10, the latter deals specifically with the development of the method for computing plate bending stresses from web deflections.

Whereas the proposed design recommendations set limits on the various parameters in terms of the yield point F_y, the two girders tested in this investigation were of A36 material. The validity of the limits for high strength material could be derived from results of earlier static tests(14,15) and through comparison of fatigue properties of materials. This will be discussed later in the report.

2. THE TEST GIRDERS AND SET-UP

2.1 Design Criteria and Description of Specimens

The two girders of ASTM A36 steel were intented for testing the upper, and most severe, limits of the design recommendations. In each case, the nominal web slenderness ratio was aimed at the highest value permitted. The panel aspect ratios should represent a range of values, including the maximum permitted ratio of 1.5. (11, 12)

With these limits set, the loading and stroke capacity of the pulsating equipment plus available space on the dynamic test bed dictated such details as web thickness and overall girder length.

To allow the comparison of the results of tests on the transversely stiffened girder (F10) with a previous series of fatigue tests,(7) its web depth was kept at 50 inches. By selecting a thickness of 1/4 in., the nominal web slenderness ratio was 200, the maximum value permissible. The 1/4 in. web would have more severe lateral deflection than that of a 5/16 in. web. In addition it was desired to have individual panels which would fail statically by shear, by pure bending, and by interaction of shear and bending. Thus, through the selection of flange size and panel length two of the five panels were controlled by shear, one by bending and two by interaction. Panel aspect ratios were 1.2 and 1.5. The geometry of this girder is depicted in Fig. 1.

The girder had transverse stiffeners only : loading and bearing stiffeners being two-sided, and intermediate stiffeners on one side only. Stiffeners were cut short of the tension flange. This detail has been shown to be adequate even at ultimate loads,[16] and it reduces the possibility of fatigue cracks initiating in the flange at this point.

Girder F11 included both longitudinal and transverse stiffeners. Again, a 1/4 in. web was selected resulting in a 95 in. depth for a nominal slenderness ratio of 380. With the addition of two 1-1/4 inch flanges (Fig. 2), the girder depth was then slightly more than eight feet. Panel aspect ratios were 1.0, 1.2 and 1.5. Again, loading and bearing stiffeners were two-sided with the intermediate stiffeners placed on one side of the web, cut short of the tension flange. The longitudinal stiffener was located on the opposite side of the web. Two different details were used where the longitudinal stiffener met the loading stiffeners. In one case, the longitudinal stiffener was made continuous by welding it to the loading stiffener. In the other, it was cut back so that it did not touch the vertical stiffener.

The sizes of transverse and longitudinal stiffeners conformed to those specified in the design recommendations,[11,12] and were smaller than those suggested in Refs. 3, 5 and 17. Extrapolations from results of previous experiments indicated that these stiffeners of girders F10 and F11 should be adequate . Whether or not they were, was one of the purposes of the testing.

Fabrication techniques were not specified except that the girders should be assembled according to common practice and be welded in accordance with the American Welding Society Specifications.[18] The sizes of the fillet welds were allowed in some instances to be smaller than that permitted by AWS. Nominal weld sizes used are indicated in Figs. 1 and 2.

2.2 Properties and Test Loads of Specimens

Up to this point, any discussion of dimensions of the girders and yield stress levels has been in terms of nominal design values. Upon delivery of the girders, average width and thickness were obtained for the webs and flanges by direct measurement at a number of locations, and the values shown in Table 1. Tensile specimens, some oriented parallel to and some perpendicular to the direction of rolling, were cut from extra lengths of flange and web components, and tested by the investigators to determine the static yield stress level and the tensile strength of the material. These values are presented in Table 2.

With the measured dimensions and material properties known, analyses were made for each panel to predict its static ultimate strength and mode of failure. These analyses were carried out according to recommended procedures which are outlined in Refs. 11 and 12. The ultimate strength as well as the web buckling loads, for comparison, are listed in Table 3 for all panels. The static load-carrying capacity of a girder is the ultimate strength of the weakest panel. The buckling loads shown correspond to the mode of failure for that panel.

The maximum loads to be applied in fatigue testing were determined through the incorporation of a factor of safety to the load-carrying capacity of the girders. With a factor of safety of 1.8 for bridges, the maximum fatigue loads are approximately 55% of the girder capacities. The range of load to be applied to a girder was arbitrarily chosen as approximately 27.5% of the capacity, or half of the maximum load, resulting in a minimum load of 27.5% of the capacity of the girder. Analyses of several two, three and four span highway bridges considering dead load, live load and impact indicated that this loading condition is practically the most severe case. For the transversely stiffened girder, F10, the loads were so determined, and are listed in Table 4. For the longitudinally stiffened girder, F11, limitations of the equipment dictated the magnitude of the maximum load be 51.0% of the predicted carrying capacity. The load range of approximately 27.5% was maintained, thus the minimum load was 23.6% of the capacity of the girders.

2.3 Test Set-up and Instrumentation

The specimens were tested at Fritz Engineering Laboratory, Lehigh University. The fatigue tests were conducted in the dynamic test bed using two Amsler pulsators and two hydraulic jacks to furnish the loads at about 250 cycles per minute. The static tests were carried out in a Baldwin hydraulic universal testing machine with a capacity of 5,000,000 pounds.

In each case, the girder was simply supported, and was braced laterally by means of 2-1/2 inch diameter pipes attached to the vertical stiffeners at one end and to a special supporting beam at the other. Figures 3 and 4 show F10 in the fatigue set-up and F11 in the static set-up, respectively.

In addition to the girders' ability to survive a specified lifetime of repeated loading, information was sought on four distinct aspects of girder behavior: (1) load versus vertical deflection of girder such as those shown in Figs. 5 and 6; (2) out-of-plane web deflection; (3) stresses on surfaces of the web adjacent to stiffeners and (4) stiffener stress distribution. The instrumentation by means of which this information was obtained was similar to that used in previous girder tests.[7]

Load-deflection behavior was monitored by the use of Ames one-thousandths inch dial gages located at mid-span and at both ends of each girder. Support deflections were in this way excluded from the girder deflection.

Electrical resistance strain gages were used to measure web and stiffener strains during the static tests before and after the fatigue loading, as well as to monitor web strains when fatigue testing was in progress. Since cracks might occur at panel boundaries when the web plate has relatively large lateral deflections,[10] strain gages were placed on the web as close to the flanges and stiffeners as possible. Linear and rosette gages (SR-4 A1 and AR1) were employed according to the gage location shown in Fig. 7. All gages, both on web and on stiffener, were placed back-to-back so that the surface stress, average stress and secondary bending stress could be differentiated. The majority of strain gage outputs were recorded by means of a B & F Strain Recorder with an IBM Keypunch attached; the remainder were handled with self and manual-balancing strain indicators.

Out-of-plane web deflections were measured extensively by means of Ames dial gages mounted on a rigid framework (Fig. 8). For girder F10, with the origin for cartesian co-ordinates being defined at girder mid-height, deflection readings were taken at $y = 0$, ± 5, ± 9, ± 15, ± 18 and ± 21 inches. These readings were taken at some fifty-five locations along the girder length. In the case of girder F11, for a better detection of the influence on the web behavior of the longitudinal stiffener, deflection readings were taken at more locations: $y = +6.5$, $+16.5$, $+21.5$, $+25.5$, $+28.5$, $+31.5$, $+40.5$ and $+44.5$ above the horizontal centerline, and at $y = -3.5$, -15.5, -25.5, -35.5, -40.5 and -44.5 below. Again readings were taken at nearly fifty positions along the length of the girder. These web deflections serve the dual purposes of affording qualitative insight into the web behavior under load, especially above the buckling load, and of providing a means of computing the plate bending stresses in the web.

2.4 Expected Fatigue Results and Static Failure Modes

Preliminary stress analyses and the S-N curve of previous fatigue tests[10] afforded an indication as to whether or not fatigue cracks of the web be expected to form.

The plate bending stresses along web boundaries of girder F10 were sufficiently low so that this girder was not expected to have any fatigue cracks before the attainment of two million cycles. In fact, cracks were not anticipated even at ten million cycles. In the case of F11, with a much higher web slenderness ratio and relatively large web deflections, stresses at some locations were comparable to those which caused fatigue cracks at around two million cycles in previous girders. It was felt before the test that cracking might be observed. Specifically, the area adjacent to the intermediate stiffener and above the mid-depth of the girder was regarded as the most likely place to develop a crack.

The static failure modes for girders under various loading conditions have been treated in detail in the literature.[1,14,15,17] The emphasis in the present tests was to investigate stiffener behavior and to collect additional web deflection data. As much as possible, panels adjacent to transverse stiffeners were loaded to failure in shear or combined shear and bending so as to develop tension-field action in the panel and compression in the stiffener. The failure mode of every panel was expected to confirm the prediction shown in Table 3.

3. TESTING OF SPECIMENS

3.1 General Procedure

The testing of each girder was carried out in three phases: initial static loading, fatigue testing, and static tests to failure. The first two took place in the dynamic test bed with loads applied by Amsler hydraulic jacks. The static tests to failure were conducted in the Baldwin five-million pound universal testing machine. The procedure followed was the same for both girders.

The initial static loading phase consisted of a preload to P_{max} for alignment, for minimizing the effects of residual stresses, and for settling the test set-up; then another loading to P_{max} for strain gage and web deflection readings. The quantitative determination of girder web deflections at P_{max} and P_{min} provided data for the prediction of plate bending stress, and hence the possibility of fatigue cracks, if any.(10)

3.2 Fatigue Tests

After the measurement of deflections and strains, the fatigue testing could begin with two million cycles as a minimum goal. At the rate of 250 cycles per minutes, that took about 130 hours, or more than five days of continuous cycling.

While under fatigue loading, the girders were closely examined at frequent intervals (initially two, later three or four hours, day and night) with floodlight and magnifying glass in an effort to detect macroscopic fatigue cracks. Monitoring of dynamic strains on the web of F 11 was performed occasionally.

When the tests had run to two million cycles without any cracks having been observed, loading was allowed to continue for a period of time dependent on the expected behavior of the girder and on the scheduling demand for the testing facilities. The test of girder F10 was terminated after 2.2 million cycles, whereas F11 was allowed to run to 4.5 million cycles, (slightly more than twelve days, continuously). After termination of the fatigue test, each girder was inspected at P_{max}. Neither showed any evidence of fatigue cracks.

3.3 Static Tests

With fatigue tests successfully completed, the two girders could be loaded statically to failure for the examination of stiffener behavior. Also observed were the web deflections beyond those at the maximum loads of the fatigue testing.

Since the function of transverse stiffeners to be examined was their ability to carry tension-field-induced compression, shear failure of girder panels was intended. Panels that would have incurred bending failure (Table 3) were reinforced with a cover plate so as to allow testing of the panels in shear. The reinforcement of girder panels is shown in Figs. 9 and 10 where the failure modes of each test are also sketched.

Girder F10

The complete testing history of this girder is depicted by Figs. 5 and 9. The first test (T1) resulted in failure of panel 4 at a load magnitude of 170 kips. At 130 kips yield lines commenced in the top flange over panels 2, 3, and 4 as well as in the web of panel 4. By the time 165 kips were applied, practically the entire tension diagonal of panel 4 had yielded. Loading was stopped at 170 kips when the web deflection was judged excessive, and flange yielding was becoming pronounced. The girder was reinforced by welding a cover plate over panels 2, 3, and 4 and adding a transverse stiffener in panel 4.

The cover plate and the additional stiffener increased the bending strengths of panels 2 and 3 and the static strength of panel 4, so that the failure for the second test (T2) would be confined to panel 1 alone. Indication of tension field by yielding was quite pronounced at 180 kips. At 184.5 kips failure occurred when girder deflection increased without further increase in load. A photograph of the panel after failure is included as Fig. 11. The girder was reinforced by welding a transverse stiffener in panel 1 (Fig. 9).

At the last load of test T2, yielding was seen to initiate in panel 5. After the reinforcing of panel 1 and resumption of loading, panel 5 continued to take load in test T3. At 190 kips the small panel beyond panel 5 yielded and the web beyond the bearing stiffener deformed. Panel 5 lost proper anchorage for its tension field and the load fell off. Figure 12 is a photograph of panel 5 after the test. No failure of any stiffener was observed in girder F10.

<u>Girder F11</u>

All three panels of this girder were taken to ultimate load. The load-deflection curve, Fig. 6, and the sketches of failure modes in Fig. 10 may be used to trace the testing history.

At about twice the maximum fatigue loads, yielding initiated at the bearing stiffeners. Since these and the loading stiffeners had been inadvertently underdesigned for static testing, doubler plates were attached by clamping and then welding in place at various stages of testing.

In test T1, the yielding pattern indicating a tension field was beginning to form in panel 1 at 200 kips. Yielding had also started in the upper flange near the loading points and in the loading stiffeners. At 215 kips, the longitudinal stiffener was noticably deflected out-of-plane in panel 1 at a place where the stiffener was not straight before loading. Doubler plates were clamped to this stiffener at 220 kips but in vain.

It failed at 230 kips by deflecting with the web and out-of-plane as tension field deflections increased. The test was stopped at this point. A vertical stiffener was fitted to the contour of the web and welded in place.

Test T2 was to bring panel 2 to failure for observing and monitoring the behavior of stiffeners. As loads increased, it gradually became clear that the deformation of panel 1 in test T1 had been too excessive and that the reinforcements were not sufficiently strong to curtail the additional web deflection. Also, the general area below the loading points incurred extensive yielding with lateral deflection of the web between the loading stiffeners. The test was interrupted after 275 kips for further reinforcement by adding stiffeners and a cover plate as shown in Fig. 10.

Panel 2 failed in interaction of high shear and bending moment in test T3 at 294 kips. Before that, and after the initiation of yielding along the tension diagonal of the panel, yield lines also developed along the tension diagonal of panel 3 and beyond the support. The failure of T3, however, was in panel 2 and is typical of panels under the same loading condition.(1)

The girder was once more reinforced with cover plate and stiffeners (Fig. 10), and then subjected to loads. At 330 kips, excessive deformation and yielding beyond the support near panel 3 were evident. This area failed as an attempt was made to increase the load further. An overall photograph of the girder after completion of the static tests is included as Fig. 13.

The experimental strength of each test panel is listed in Table 4, together with the predicted values.

4. BEHAVIOR AND STRESS OF WEB AND STIFFENERS

4.1 Lateral Deflection and Stresses in Web

Qualitatively the web deflections of girder F10 and F11 were the same as those obtained from other test girders [1, 7, 8, 14, 15, 17]. Initial deflections of web, or out-of-flatness, existed before any application of loads. As loads were applied to the girders, lateral deflections increased gradually corresponding to the loads. The cross-sectional shapes at several transverse sections of the girders are shown as examples in Fig. 14, with exaggerated scales. The deflection pattern of a web panel depended on the loading condition. The deflected cross-sectional shapes in Fig. 14 depict a diagonal pattern in many panels which is the result of tension field action [3]. This pattern is clearly seen in Figs. 15 and 16, contour plots of the lateral deflections of two web panels. In Fig. 15, the gradual change of contour lines from one load to the other indicates the gradual change of web deflection and the forming of the tension field along a diagonal. The contour lines of Fig. 16 show the effect of the longitudinal stiffener on the deflection pattern of the web panel. Obviously this longitudinal stiffener prevented the upper part of the web from developing large lateral deflections such as those recorded in the lower portion of the web which were quite high at a load of about 65% of static strength.

It is the magnitude of the web deflections which were of interest in the testing; for the measured magnitudes could be used in an analysis to predict the web boundary stresses. Some measured data are shown in Table 5. The maximum initial deflections were 0.16 in. and 0.66 in. or 0.63 and 2.52 times the web thickness for F10 and F11 respectively. Both were within the AWS limit of permissible out-of-flatness.[18] The maximum out-of-plane deflections at approximately 55% of the panel strength were 1.67 and 4.30 times the web thickness for F10 and F11. It is important to note that the maximum fatigue loads were decided according to the panel strength which in turn depends on the web thickness.

By using measured deflections, out-of-plane bending stresses of the web plate were predicted and the magnitudes at some points are shown at their approximate locations in Fig. 17. These were ranges of fatigue stresses in the web at its junction with the stiffener or flange, and were found to be responsible for any previous fatigue cracks at these junctions.[10] The accuracy of these predictions were checked against stresses from strain gages (Fig. 18) and were considered acceptable.

The highest predicted range of plate bending stress for girder F10 was about 16 ksi and that for girder F11 was approximately 23 ksi. In comparison with results from previous fatigue tests, Fig. 19, it is obvious that the possibility of incurring cracks was not high for girder F10 at three or four million cycles, whereas F11 might develop cracks at around two million cycles. The fatigue testing of F10 was, therefore, terminated shortly after two million cycles; and girder F11 carried to 4,500,000 cycles when equipment availability dictated stopping.

After fatigue testing, the behavior of the webs and the eventual failure of the panels under static load, as described in Section 3.3, showed no extraordinary phenomena. The failure modes were generally as predicted, conforming to those reported in previous static tests.[1, 17] For both girders, yielding at the general area of anchorage for tension field in the last test panel caused failure of the panel to occur below the predicted loads. This, too, agreed well with results of previous tests.

4.2 Stiffener Behavior and Stresses

There was no yielding or buckling of intermediate transverse stiffeners even when an adjacent panel failed in tension field action. (Only a contour-fitting reinforcement stiffener, used for repair, bent out of plane due to excessive web deflection of girder F11; test T2, Fig. 10). The loading and bearing stiffeners of this girder yielded near their contact zones because of insufficient size; however the yielded areas were not anchoring a tension field and reinforcements corrected the under-design (Sec. 3.3). The longitudinal stiffener of girder F11, on the other hand, deformed out of its plane for a short length near (x = - 122) where initial out of straightness of about 1/4 in. had been observed.

In addition, the intermediate transverse stiffeners and the longitudinal stiffener twisted slightly when adjacent web plates deflected laterally. The amount of twisting differed along the length of a stiffener as well as from one stiffener to another. Since it is the relative rotation between the web and the stiffener that is significant for web plate bending stresses, the absolute rotation of the stiffeners were not recorded. Past attempts to measure relative rotation had not been sufficiently accurate, therefore measurements were not made in these tests. During repeated loading, the longitudinal stiffener in panel 1 of girder F11 appeared to twist relatively more than other stiffener. Being the longest of all stiffeners, and in a panel of highest slenderness and aspect ratio and highest lateral deflection, a relatively higher twisting was expected.

Both the transverse and the longitudinal stiffeners deflected perpendicular to the plane of the web. Measurements indicated that maximum deflections of the transverse stiffeners at P_{max} were less than 1/10 of the web thickness. Such a magnitude was considered small enough to be ignored in the prediction of web plate bending stresses.[10] At the maximum fatigue load, the maximum deflection of the longitudinal stiffener was approximately 1/2 of the web thickness. This deflection with the web, together with the twisting significantly reduced the plate bending stresses of the web at the stiffener and thus reduced the possibility of early fatigue cracks along this longitudinal stiffener.

The directions of deflections of one-sided stiffeners generally followed the deflection patterns of the web plates. Consequently, the directions of the deflections differed from panel to panel and could be toward or away from the stiffener, as can be detected from Fig. 14. This phenomenon strongly influenced the stresses in the stiffener plates, and is discussed below.

Stresses in the transverse intermediate stiffeners were computed from measured strains. The average uni-axial stresses calculated from measurements of back-to-back strain gages are shown in Fig. 20 for a stiffener (x = - 165) of girder F10 when the neighboring panel failed in shear. Tension field action of the panel demanded anchorage at the upper part of the stiffener, resulting in high compressive stress in that region. (At the corner of the stiffener-web-flange junction, the vertical compressive strain in the web indicated that yielding had occurred.) At a point in the stiffener six inches below the flange, the compressive stress was about 13 ksi, much less than the yield point of the material. Further down along the stiffener-to-web junction, the stress magnitude decreased linearly to zero at the bottom, where no tension field anchorage took place. By comparison, the stresses at the free edge of the stiffener changed from tension in the upper portion to compression below. This was the obvious result of the edge loading applied to the one-sided stiffener, with deflection of the stiffener plate concave along the loading edge.

When a longitudinal stiffener was present, tension field action took place in the sub-panels and the stress distribution in a transverse stiffener adjacent to this panel reflected this subdivision. For the stiffener at x = - 154 at the ultimate load of panel 1, the axial stresses are shown in Fig. 21 with straight lines connecting data points. Below the longitudinal stiffener, where a tension field was prominent, the decreasing of compressive stress towards the bottom was evident along the stiffener-to-web junction, just as in the case described in the last paragraphs. Above the longitudinal stiffener, lower stresses were recorded, particularly at the location opposite the longitudinal stiffener. It is regrettable that there existed no strain gages just below the horizontal stiffener for a better picture of stress distribution in that vicinity. Nevertheless, the highest recorded axial stress of about 19 ksi was far below the yield point of the material. Stresses in the web along the transverse stiffener confirmed these magnitudes, as are indicated for two points in Fig. 21.

It is interesting to compare the stress distribution of the stiffeners of Figs. 20 and 21. Both were at the end of a girder next to a failed panel and a small end panel (Figs. 9 and 10). For the case without a longitudinal stiffener, the small end panel yielded and the tension field anchorage was primarily by the flange of the girder. The stresses in the stiffener were relatively low. Where the longitudinal stiffener existed, the small end panel also yielded, as well as the upper sub-panel. The longitudinal stiffener deformed, thus the transverse stiffener had to take higher stresses. Had the longitudinal stiffener been straight it would certainly have shared the duty of anchoring and would have reduced the stresses in the transverse stiffener.

For the longitudinal stiffener in panel 1 of girder F11, the magnitude of stresses at several points along its length are shown in Fig. 22. Near the transverse stiffener at x = - 154, the stresses were compressive because of tension field anchorage. At the other end where the stiffener was not continuous and there was no tension field in the next panel, the stresses were practically zero. In between, where local deformation of the stiffener plate occurred (around x = - 122), it was not capable of sustaining axial compression, thus resulting in negligible stiffener stresses in the vicinity. At mid-panel, the deflection of the stiffener towards its free edge caused in-plane bending of the stiffener plate and hence tension stresses along the free edge.

In panels 2 and 3 where the longitudinal stiffener was continuous, the stress magnitudes were uniformly low along the web (Fig. 23) even at the failure of panel 2 by combined shearing and bending of the girder. Again, as in panel 1 stresses at web points just below the stiffener agreed well with these magnitudes. What differed from panel 1 was that the web and stiffener deflections under load were towards the web; the free edge of the stiffener plate thus was under high bending stresses at the middle of each panel.

It is also of interest to know that stresses in stiffeners developed only at high loads near the static strength of the web panels (Figs. 24 and 25). For both transverse and longitudinal stiffeners, the web buckling load bore no significance in the development of stresses. At the maximum fatigue loads stiffener stresses were practically negligible. Even the surface stresses of the longitudinal stiffener, reflecting out-of-plane bending, could be considered insignificant at the maximum fatigue load.

5. DISCUSSION

5.1 Slenderness and Aspect Ratios for Webs

The results of the proof-testing demonstrated that no fatigue cracks occurred in the two welded plate girders of A36 steel with maximum recommended web slenderness and panel aspect ratios under the most severe loading conditions. It is the intent here to discuss the factors governing fatigue cracks of the web and to judge the adequacy of the recommended limits for girder webs.

As has been discussed earlier, webs of girder panels are not initially flat and deflect further under load. Plate-bending stresses are thus produced which are the primary cause of web cracks in repeated loading. Therefore, the factors influencing the formation of cracks are: 1) the initial deflection of web, 2) the increase of web deflection under load, 3) the magnitude and range of loading, 4) corresponding plate bending stresses, and 5) the properties of the web material in terms of a stress-fatigue life relationship.

The magnitude of initial deflection (w_i) or out-of-flatness, depends on the thickness of the web (t), its slenderness ratio (β), the panel aspect ratio (α), as well as the flatness of the web plate before fabrication and the technique of fabrication. For any girder, this magnitude of initial deflection can not be controlled by the designer.

Measurements on test girders have furnished a diagram relating w_i/t and β for girder panels with web thicknesses of 3/16 to 3/8 in. and aspect ratios of 0.8 to 1.5 (Fig. 25). From this diagram it may be stated that for a given web thickness, the magnitude of initial deflection increases with the web depth. Also from Table 5, it can be said that a given value of β, the amount of initial out-of-flatness is larger for longer panels. It follows that, generally, for a given set of α and β values, the initial deflection is inversely proportional to the web thickness. Because the two proof-test girders were built of 1/4 in. plates, their initial web deflections were consequently higher than would have been recorded for 5/16 in. or thicker webs. In fact, the magnitude of the initial web deflection of the test panels of girders F10 and F11 were the highest recorded for their respective slenderness ratios (Fig. 25).

The increase of lateral deflections under load is a function of the load magnitude, the panel geometry, the web boundary conditions, and the magnitude of the initial out-of-flatness. Efforts have been made to predict these deflections but only a very limited number of very simple cases were examined.[19] While studies are being carried out for more general conditions of panel geometry and web boundary conditions, experimental results indicate that the increase of web deflections in practical load range is generally proportional to the magnitude of the initial web deflections. (See, for example, Table 5 in this report and Fig. 93 of Ref. 8). It follows from this and the statement of the last paragraph that, for a given value of β, thinner webs deflect more than thicker webs, and the webs of girders F10 and F11 deflected more than those of girders with 5/16 in. webs when subjected to load.

As pointed out earlier (Sec. 2.2), the load magnitude and the load ranges applied to the test girders were the most severe condition anticipated for bridge girders. Consequently the web deflection conditions of the girders F10 and F11 represented cases much more severe than commonly encountered. What remains to be found is the relationship between thse web deflections and the plate bending stresses at the web boundaries, and to compare them with the fatigue properties of the material.

The web plate bending stresses can be estimated by using the method of Ref. 10. For ordinary web boundary conditions, the relationship appears to be linear between the range of stresses and the range of web deflections (corresponding to the range of applied loads), Fig. 26. Higher lateral deflections create higher plate bending stresses. Higher initial deflection thus cause higher plate bending stresses; and girders F10 and F11 should have relatively much higher stresses than would be expected of thicker web plates.

The highest web boundary plate-bending stresses were only about 16 ksi and 23 ksi for girders F10 and F11, respectively. The stress magnitudes would be lower for girders of 5/16 in. web plates and much lower for thicker webs, even with the highest permissible slenderness and aspect ratio. Consequently, no fatigue cracks would be expected to occur prior to two million cycles of load application, according to the S-N curve from previous experimental results as presented in Fig. 19. (It may even be predicted that no cracks would occur at all prior to ten million cycles for 5/16 in. or thicker webs if more data can be obtained for the S-N curve and a more accurate analytical estimate of stresses can be carried out).

However, the S-N curve is from ASTM A36 and A373 steel girders. While the magnitude of loads are higher on girders of higher strengths materials, the web deflections are also proportionally higher for these girders. If the relationship remains the same among the initial deflections, deflections under load, and plate bending stresses, girders built of higher strength steels would have proportionally higher plate bending stresses at web boundaries. Since the fatigue characteristics of fillet-welded structural elements of various steel differs little,(21) high plate bending stresses must be prevented. The web slenderness limits as described in Sec. 1.1 conservatively control the web thickness and keep web deflections of girders under load to the same order of magnitude as for structural carbon steel girders. Consequently, the testing of girders F10 and F11 represents a very severe case of deflection and stresses for girder panels of A36 steel as well as for panels of all steels. Because girders F10 and F11 sustained more than two million cycles of load without cracks, any girder panels conforming to this slenderness ratio limit with an aspect ratio less than 1.5 would thus not have web boundary cracks prior to two million cycles.

Therefore, it can be concluded that the recommended web slenderness and panel aspect ratios are sufficiently safe for bridge girders for two million cycles.

5.2 Stiffener Sizes and Rigidity

One of the purposes of the static testing to failure was to examine the behavior of stiffeners transverse and longitudinal, so as to study the strength and rigidity requirements. The functions of any stiffener in a plate girder are to maintain the web plate in place and to carry whatever forces which are transmitted onto the stiffener itself. The design requirements are derived from these functions.

For any stiffener, a minimum value of moment of inertia (I) is required to keep the web plate in its plane. The time-tested formulas for I in bridge design specifications[13] define this requirement up to the buckling of the web.[21] Since post-buckling strength of the web is utilized and stiffeners carry forces, more stringent rigidity requirements must be established in connection with the post-buckling behavior of the web.

While analytical studies are in progress, experimental results have indicated the adequacy of the existing limit of I both for one and two-sided transverse stiffeners in that there was never any buckling of such stiffeners at the failure of the neighboring panels.[1,14,17] The one-sided intermediate stiffeners of girder F10 and F11 further proved this point.

Because one-sided longitudinal stiffeners deflect perpendicular to the web and thus are subjected to bending in their own plane, a minimum value of radius of gyration (r) is specified (See Sec. 1.1) for the prevention of overall stiffener buckling under load. The longitudinal stiffener of girder F11 was for the examination of this requirement of r. Unfortunately, out-of-plane deflection of the stiffener in panel 1 prevented a clear cut determination of the adequacy of the requirement. It can only be considered that the local bending, or twisting, of the stiffener in this panel was the consequence of local out-of-straightness and the large web deflection, not the result of insufficient radius of gyration. In panel 2, around $x = + 116$, where the stiffener was continuous through the next panel, there was also an initial out-of-straightness but less pronounced than that in panel 1 (approximately 1/8 to 1/4 in.). With this smaller magnitude, a shorter panel length, and, more importantly, smaller web deflections, the longitudinal stiffener did not deform here. Further study should be (and is being) made. It is sufficient to indicate now that panels 1 and 2 had attained their maximum loads (Table 4) according to the recommended procedure of computation[11,12] without failure of the longitudinal stiffener.

To carry forces, minimum areas are specified for stiffeners, considering an effective width of web plate adjacent to the stiffener as part of the stiffener.[11,12] It has been shown that transverse stiffeners designed according to tension field theory and without an effective width of web are sufficiently strong[2,14,17] but maybe overly conservative.(11,12) In the test girders F10 and F11, no yielding was observed on the one-sided transverse stiffeners which were designed assuming participation of web. The stress distribution diagrams such as those shown in Figs. 20 and 21 indicate that, in fact, the stresses were below the yield strength, particularly in the case of panels without longitudinal stiffeners. This implies that further reduction in area requirement may be possible. However, it is not judged advisable at the present when knowledge is not sufficient on the post-buckling behavior of webs and on the corresponding stress distribution along the stiffeners. Furthermore, for transverse stiffeners the I and width to thickness ratio requirement usually demand an area larger than that specified by the strength consideration.

The area requirement for longitudinal stiffeners follows the same development as that for transverse stiffeners. However, with the radius of gyration provision in addition to that of the moment of inertia, for stability there usually exists sufficient area of stiffener to carry forces. No area requirement was specified(11,12) and the longitudinal stiffener of F11 was so designed. The low stress magnitudes along the web at failure of the panels (Figs. 22 and 23) testified to the acceptability of this procedure and the adequacy of the stiffener area.

In reviewing the overall behavior of the stiffeners, it is considered that the rigidity and the area requirements are adequate because no yielding or buckling of stiffener occurred. It is also considered that the requirements are not overly conservative since the one-sided stiffeners deflected and twisted with the web. Therefore, the acceptance of the requirements is recommended.

A question often arises whether the longitudinal stiffeners should be connected to the intersecting transverse stiffeners for strength and rigidity. Judged by the behavior of transverse stiffeners which are cut short of the tension flange and are subjected to compressive stresses from tension field action, it was believed that no connection was necessary for the intersecting stiffeners. In the only case where the longitudinal stiffener was cut short (F11, Panel 1, Fig. 22), the failure of the panel was indeed by tension field action, and the compressive stresses at the free end of the stiffener were low. However, the stresses at the continuous ends of the stiffener in panels 2 and 3 were also low (Fig. 23) indicating the insignificance of the stiffener connection for strength. For rigidity, the intersection of the longitudinal and transverse stiffeners always provide high stiffness regardless of being connected to or cut short of one another. This is proven by the fact that the web deflection were practically zero at these intersections. Consequently, it may be concluded that longitudinal stiffeners may be cut short at the intersection with transverse stiffeners, for easy fabrication and maintenance.

6. SUMMARY AND CONCLUSIONS

In this study, two large-size welded plate girders of ASTM A36 steel were tested under severe fatigue loading and then under static loads to failure. The specimens were designed according to recommendations[11,12] with the exception that the minimum web thickness criterion was violated on the non-conservative side. The following results were obtained:

1. Under a fatigue load range of approximately half-maximum to maximum with the latter equivalent to the working load, one girder sustained 2,200,000 cycles and the other underwent 4,500,000 cycles, both without any cracks.

2. The web slenderness and panel aspect ratios of these girders were at or about the maximum permissible value; the initial out-of-straightness of the webs in these panels was practically the highest for each of the panel geometries.

3. The longitudinal stiffener effectively controlled web deflections in the practical load range, and the deflection of webs in the load range was generally proportional to the magnitude of the initial web deflections.

4. Plate bending stresses of the webs were low along their boundaries. These stress magnitudes and the fatigue results agreed well with the S-N curve from previous tests, so that normally no fatigue cracks would occur in these girders.

5. The stiffeners were on one side of the web only with minimum required rigidity and area. All of these stiffeners deflection and twisted slightly with the web deflection when under load.

6. No failure or yielding of stiffeners occurred. Only local deformation was observed at a point of initial out-of-straightness in a longitudinal stiffener.

7. Recorded stresses in the stiffeners and adjacent to the webs were below the yield strength of the material, particularly for transverse stiffeners without longitudinal stiffener.

8. The longitudinal stiffener behaved approximately the same whether it was cut short of or connected to the intersecting transverse loading stiffeners.

9. Girders failed in static testing in predicted modes of failure by tension field action. All except the last panel of each girder attained their predicted load carrying capacity.

In considering the above results and the fact that the girders a) had maximum permissible conditions of geometry, with web thickness less than specified, b) were subjected to the most severe loading conditions which are expected of bridge girders, and consequently c) sustained larger web deflections and higher stresses in the web and the stiffeners than those in girders with permissible thickness of web, it is concluded that the design recommendations are acceptable for ASTM A36 steel bridge plate girders for two million cycles, and may be conservatively used for all steels with the recommended web slenderness ratios.

Table 1 - PLATE DIMENSIONS (IN.)

ELEMENT \ GIRDER		F-10 Nominal	F-10 Measured	F-11 Nominal	F-11 Measured
TOP FLANGE	Width	16	16.05	14	14.16
TOP FLANGE	Thickness	1	0.997	1¼	1.260
WEB Thickness		¼	0.257	¼	0.262
BOT'M. FLANGE	Width	16	16.00	14	14.12
BOT'M. FLANGE	Thickness	1	0.998	1¼	1.271

Table 2 - MECHANICAL PROPERTIES

ELEMENT \ GIRDER	F-10 σ_y ksi	F-10 σ_u ksi	F-10 Elongation %	F-11 σ_y ksi	F-11 σ_u ksi	F-11 Elongation %
TOP FLANGE	28.8	59.9	35.6	27.2	59.0	32.2
WEB	38.7	65.6	30.7	34.2	61.0	29.4
BOT'M. FLG.	31.6	62.7	36.9	26.3	59.8	36.3

Table 3 - PANEL STATIC STRENGTH *

Girder	Panel No.	Panel α	Panel β	Web Buckling (Kips)	Shear (Kips)	Bending (Kips)	Interaction (Kips)	Mode of Failure
F-10	1	1.5	195	79	*181*	276	—	Shear
	2	1.2		63	202	172	*154*	Inter.
	3	1.2		100	—	*172*	—	Bend.
	4	1.5		57	181	172	*148*	Inter.
	5	1.2		92	*202*	324	—	Shear
F-11	1	1.5	364	12	*212*	297	216	Shear
	2	1.2		40	350	293	*261*	Inter.
	3	1.0		49	*373*	589	404	Shear

* In Terms of Jack Load, P

Table 4 - TEST LOADS

GIRDER	PANEL \ LOAD	STATIC P_{TH} (kips)	STATIC P_{EX} (kips)	STATIC P_{EX}/P_{TH}	FATIGUE P_{MAX} (kips)	FATIGUE P_{MAX}/P_{TH} (%)	FATIGUE P_{MIN} (kips)	FATIGUE P_{MIN}/P_{TH} (%)
F-10	1	181	184	1.07	82	45.3	41	22.6
	2	154	—	—		53.2		26.6
	3	172	—	—		47.6		23.8
	4	148	170	1.09		55.4		27.7
	5	202	(190)	(0.99)		40.6		20.3
F-11	1	212	230	1.09	110	51.8	50	23.6
	2	261	294	1.13		42.2		19.2
	3	373	(332)	(0.89)		29.5		13.4

Table 5 - WEB DEFLECTIONS

Girder	Panel No.	Panel α	Panel β	w_i ‡	$\frac{w_i}{t}$	w_p *	$\frac{w_p}{t}$
F-10	1	1.5	195 (t = 0.257 in.)	0.162	0.63	0.429	1.67
	2	1.2		0.122	0.48	0.186	0.725
	3	1.2		0.046	0.18	0.127	0.495
	4	1.5		0.111	0.43	0.172	0.670
	5	1.2		0.054	0.21	0.229	0.875
F-11	1	1.5	364 (t = 0.262 in.)	0.661	2.52	1.128	4.30
	2	1.2		0.457	1.74	0.706	2.70
	3	1.0		0.395	1.51	0.635	2.42

* w_p = Deflection at Approx. 55 % of Panel Strength

‡ w_i = Laterial Deflection Before Application of Loads

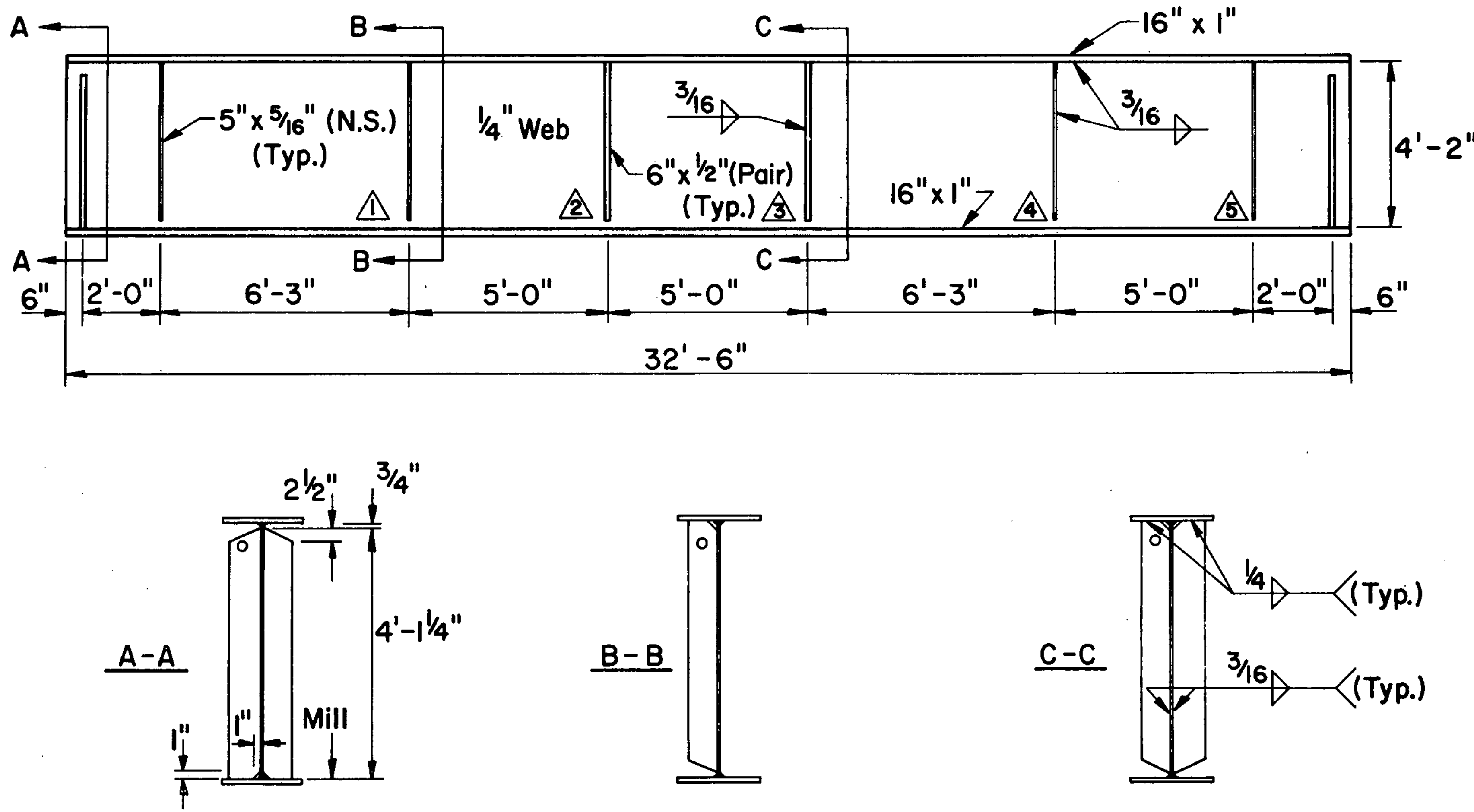

Fig. 1 Girder F10

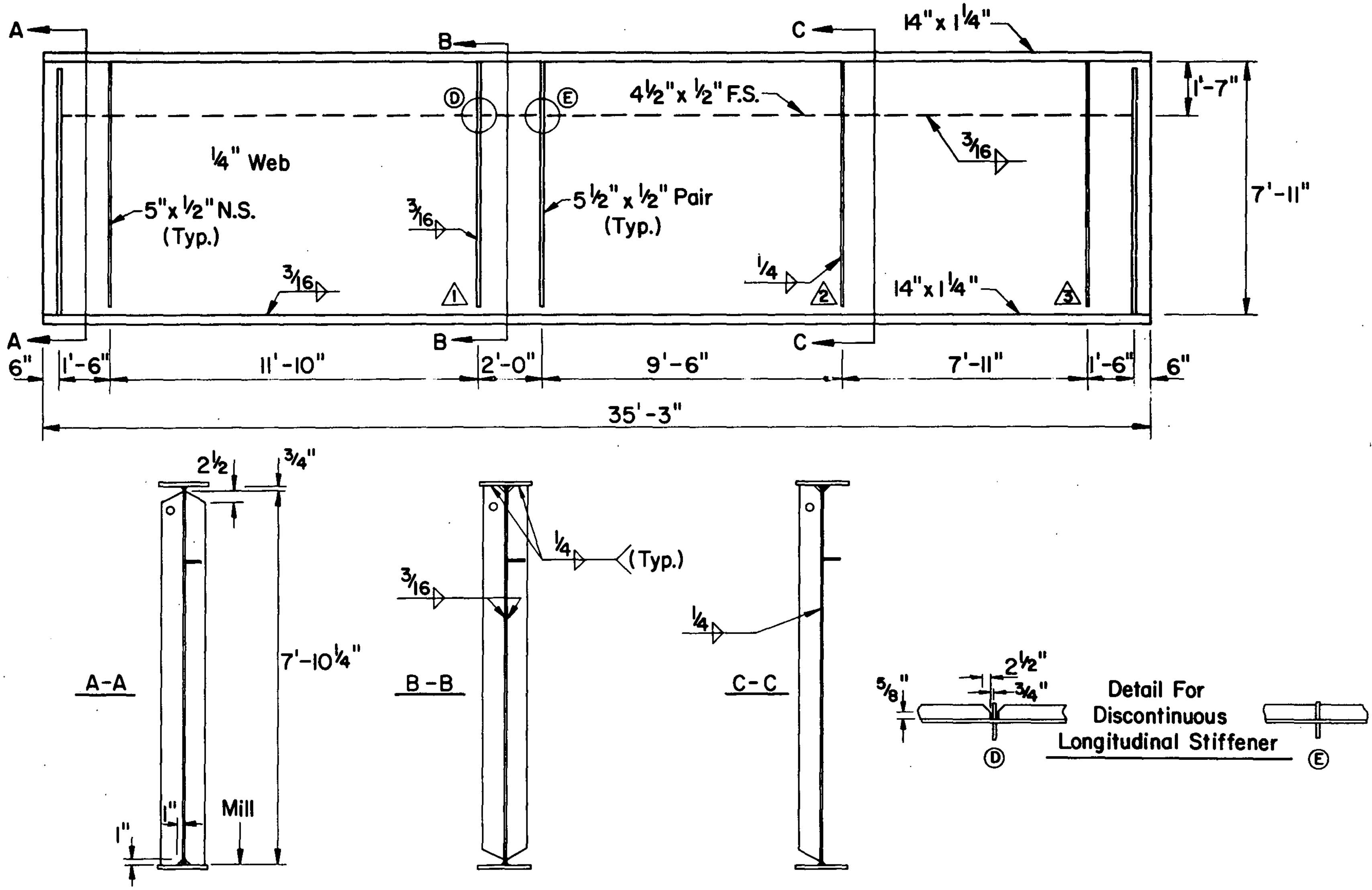

Fig. 2 Girder F11

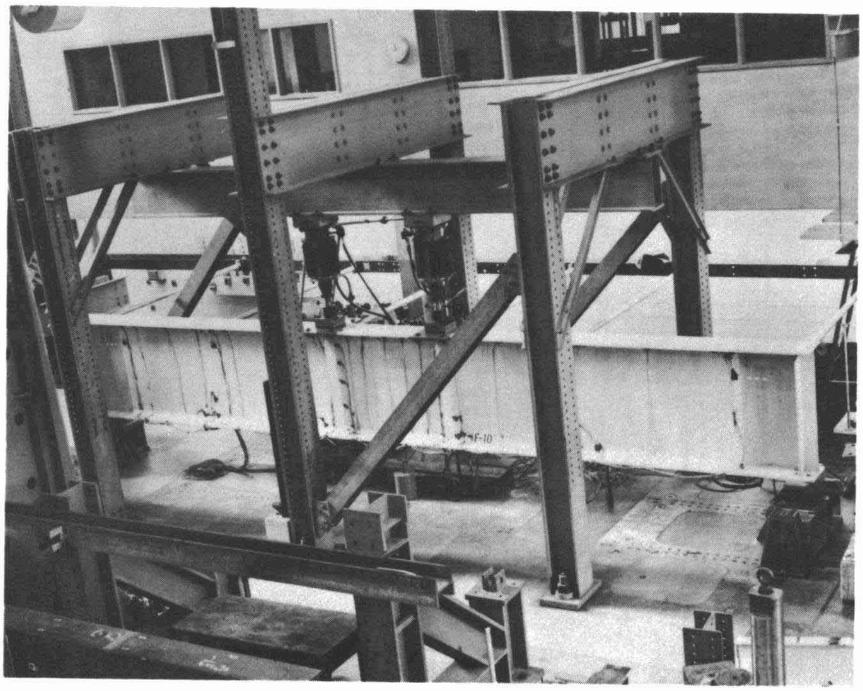

Fig. 3 Fatigue Test Set-Up (F10)

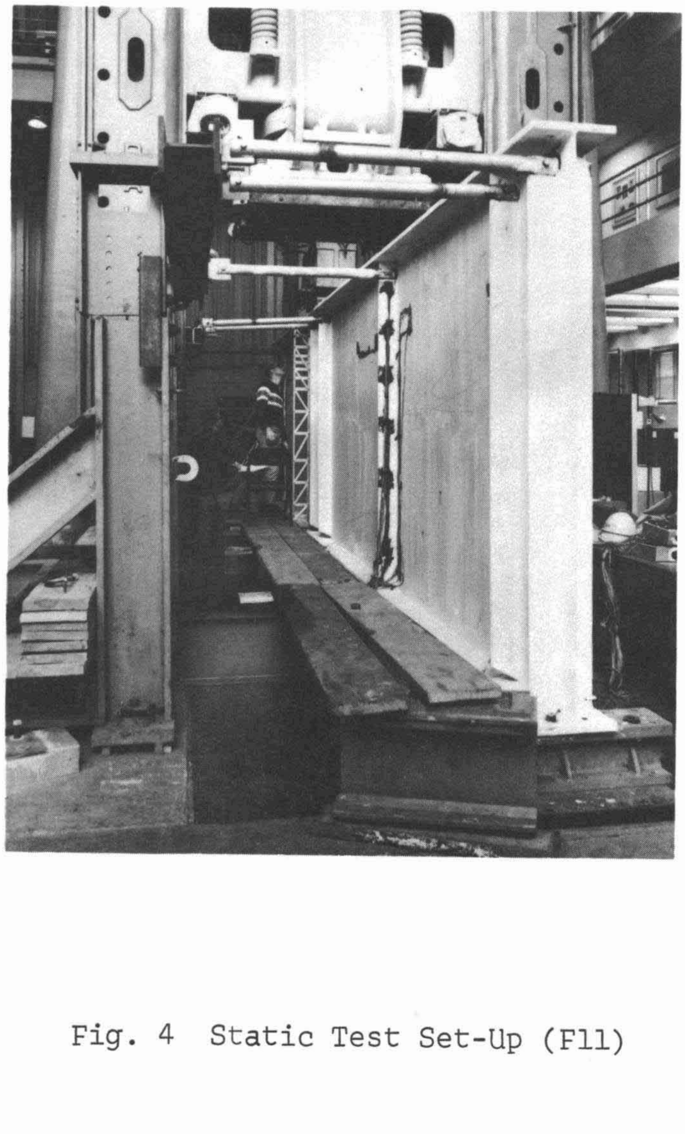

Fig. 4 Static Test Set-Up (F11)

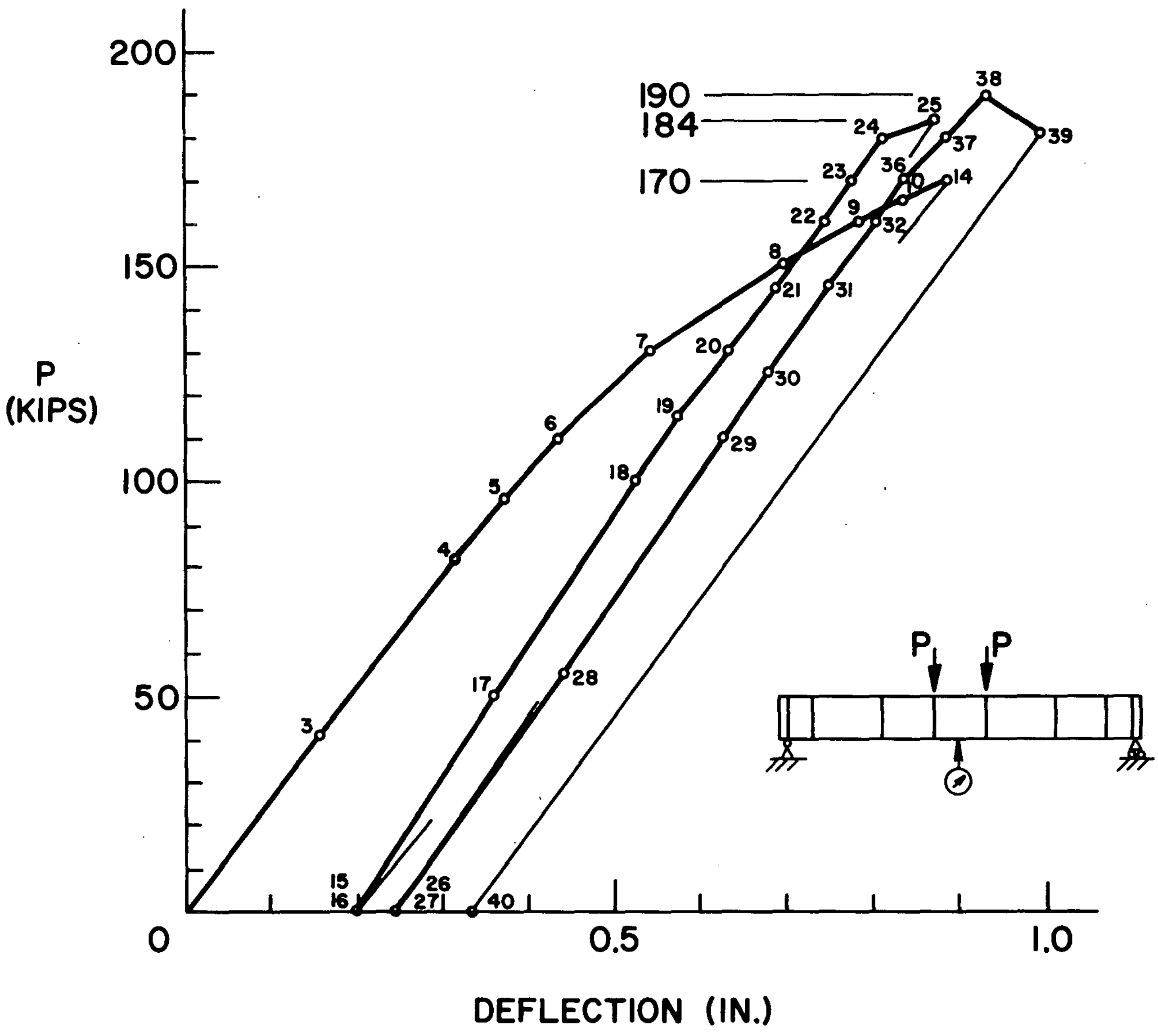

Fig. 5 Load Versus Deflection, F10

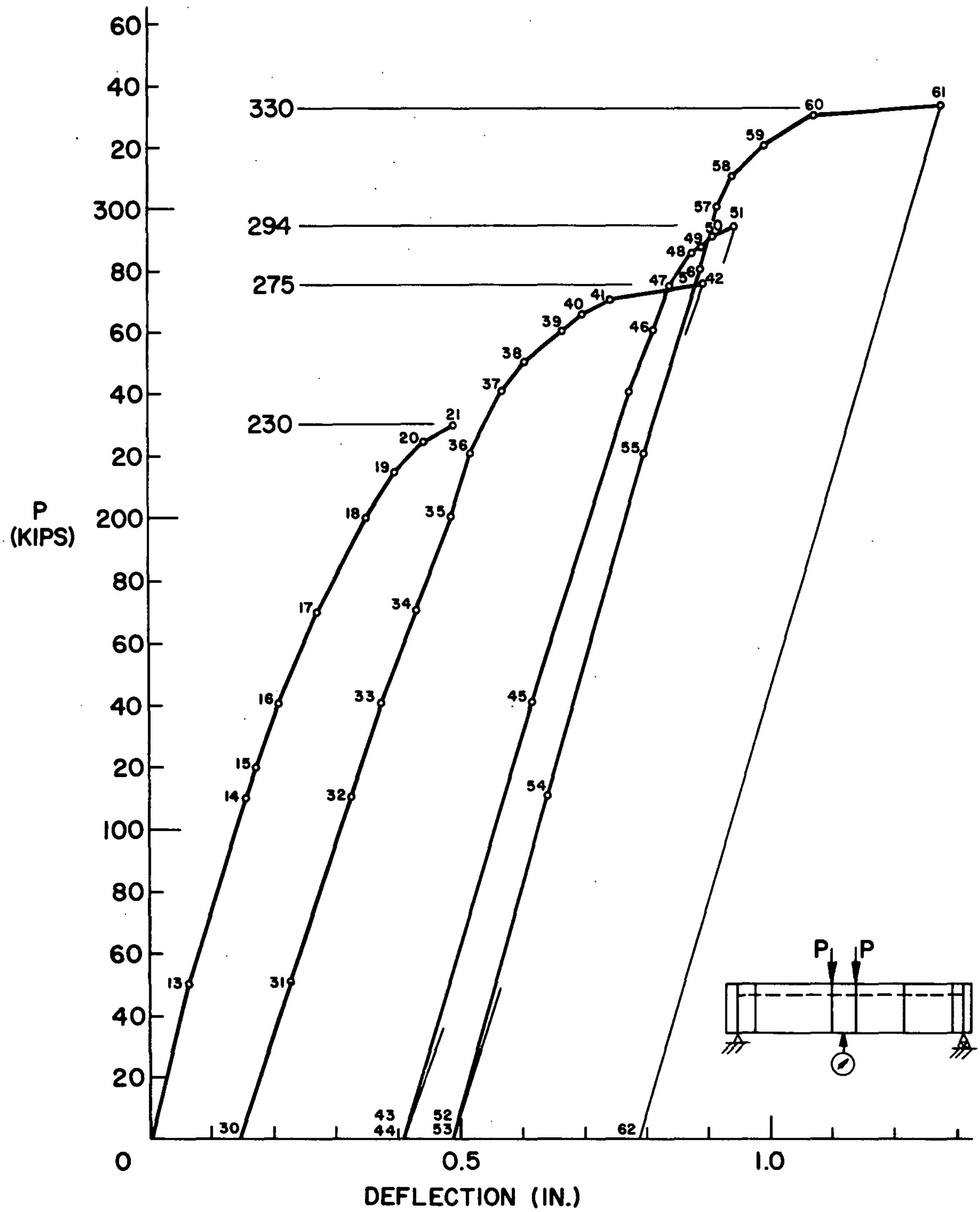

Fig. 6 Load Versus Deflection, F11

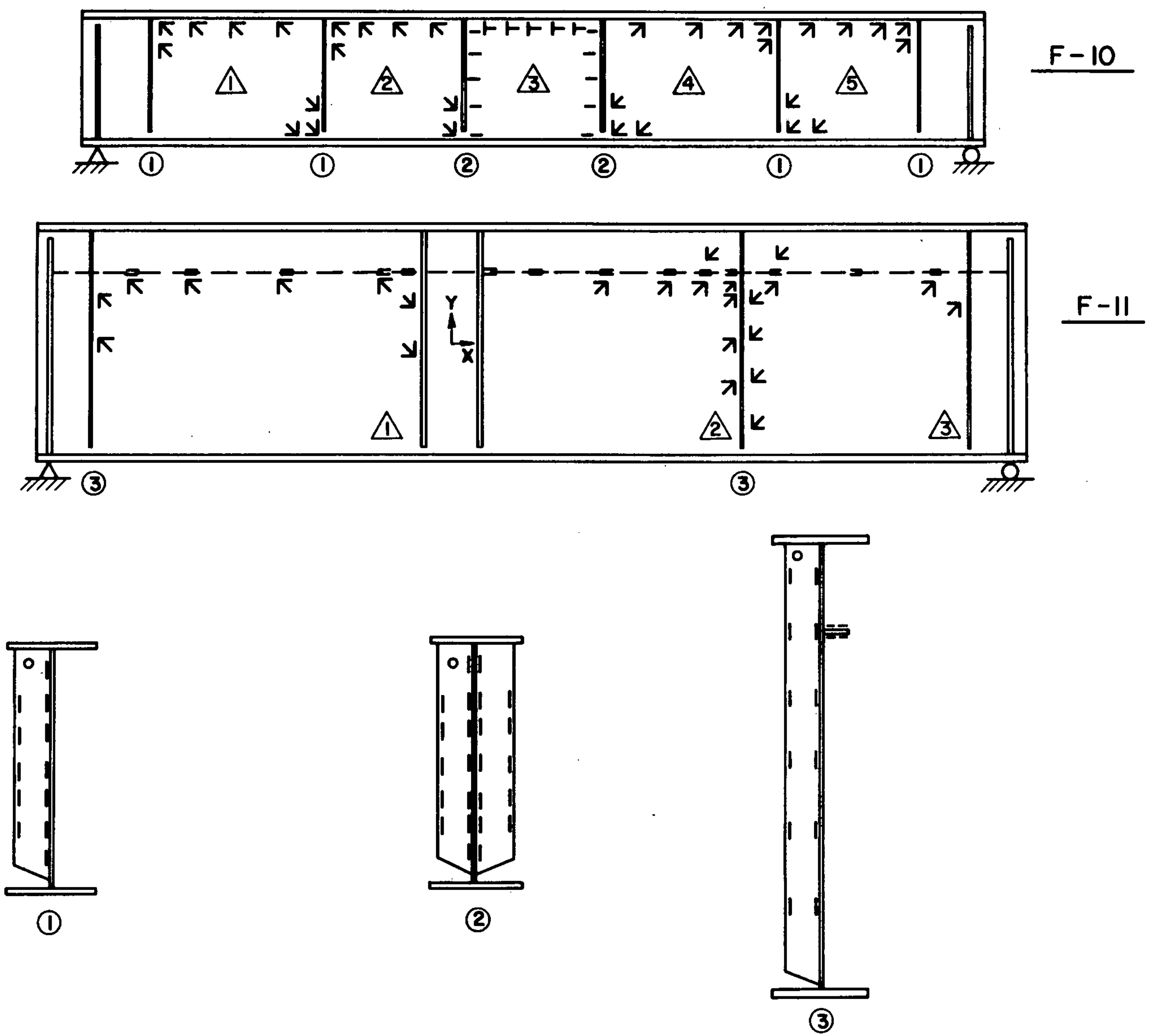

Fig 7 Strain Gage Layout, F10 and F11

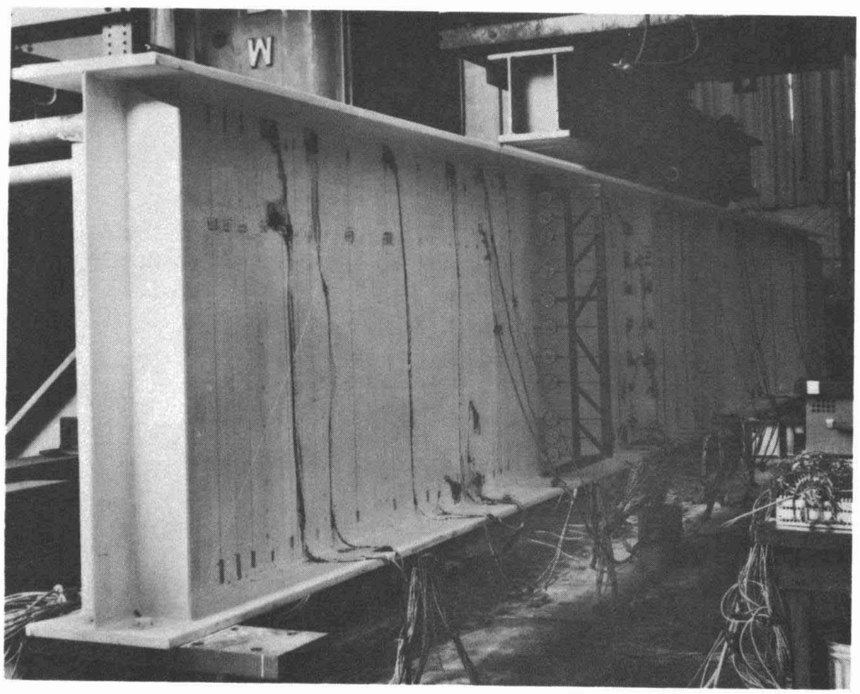

Fig. 8 Dial Rig for Web Deflection Measurement on F10

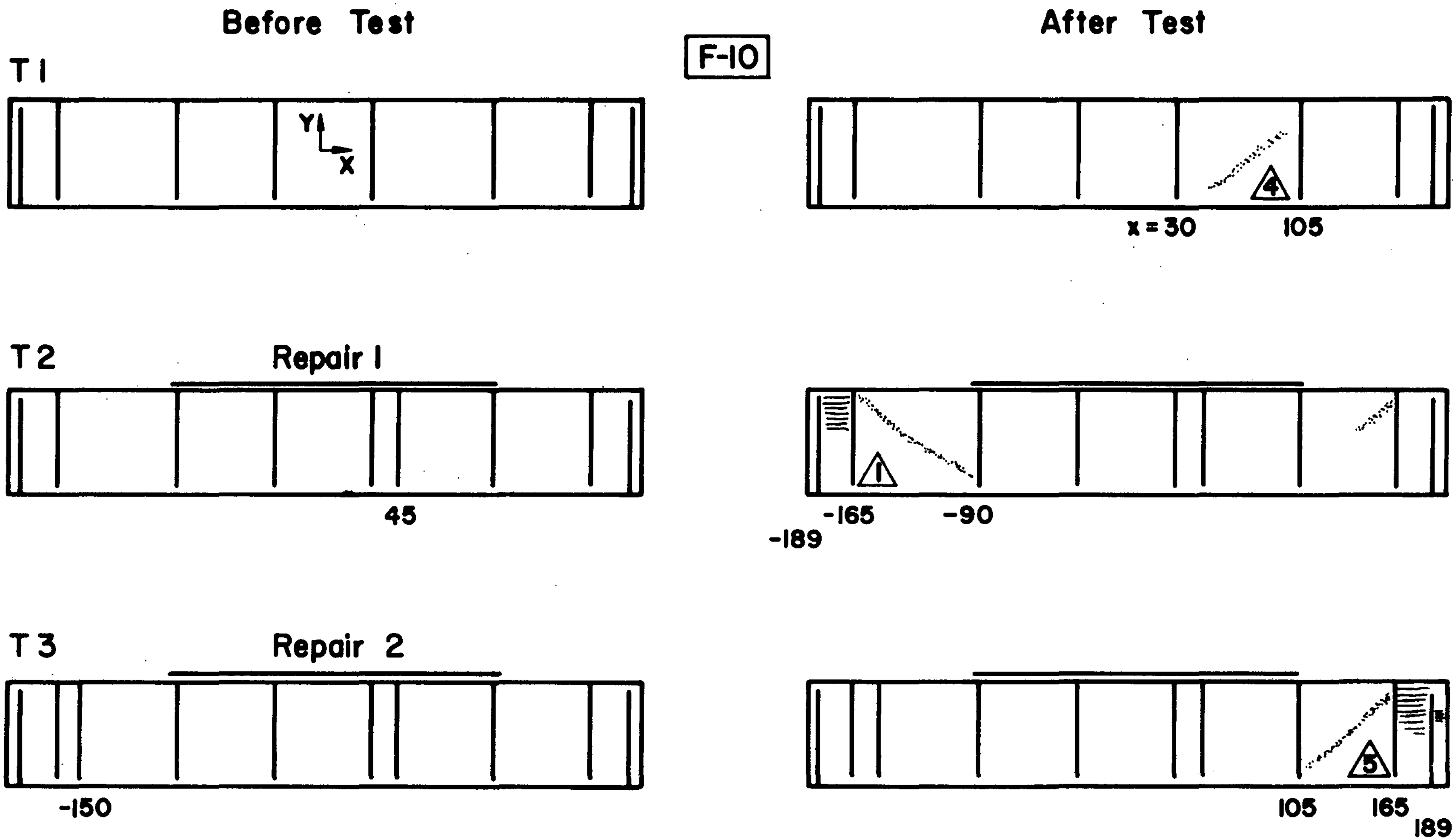

Fig. 9 Static Failure Modes and Repairs F10

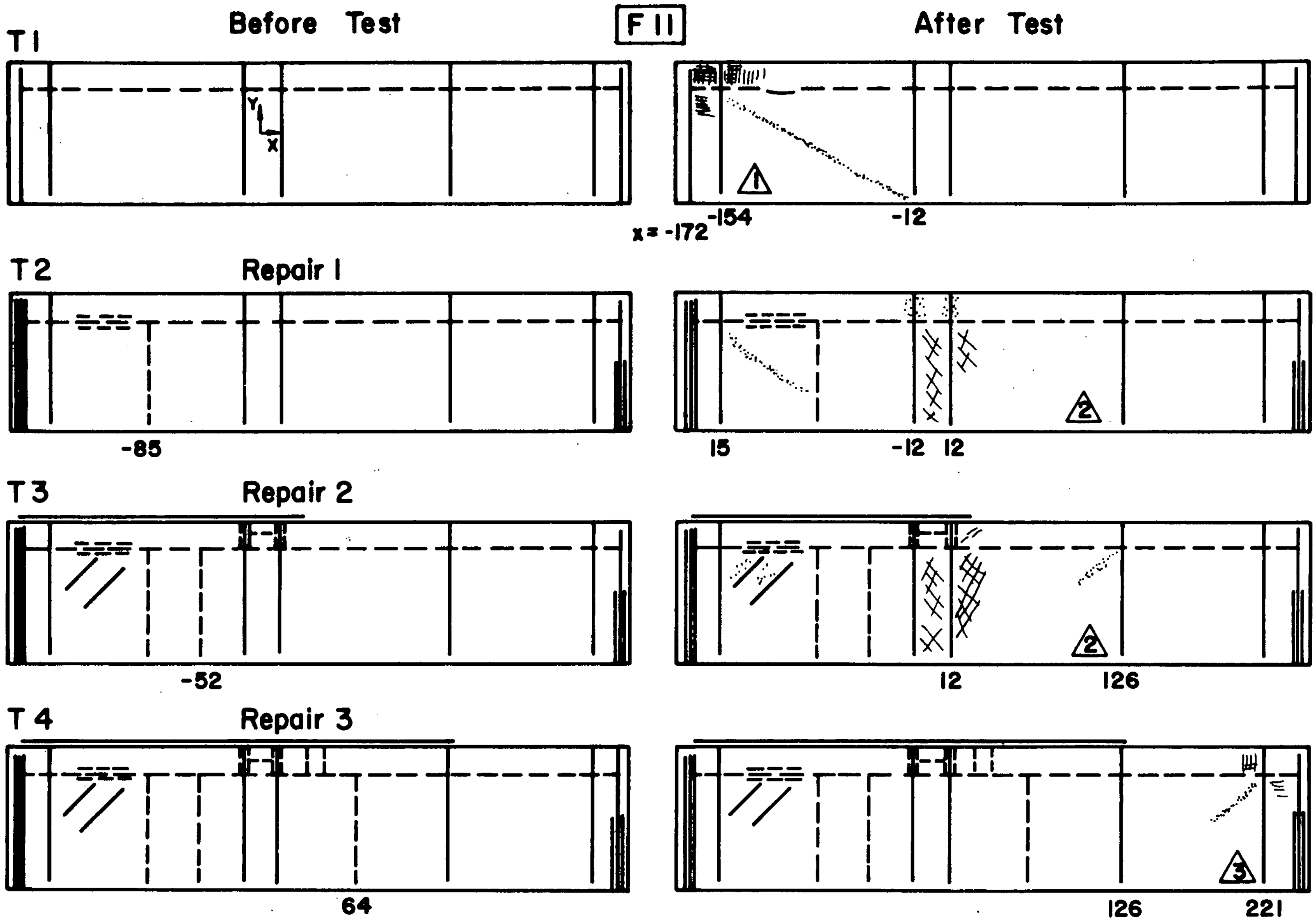

Fig. 10 Static Failure Modes and Repairs F11

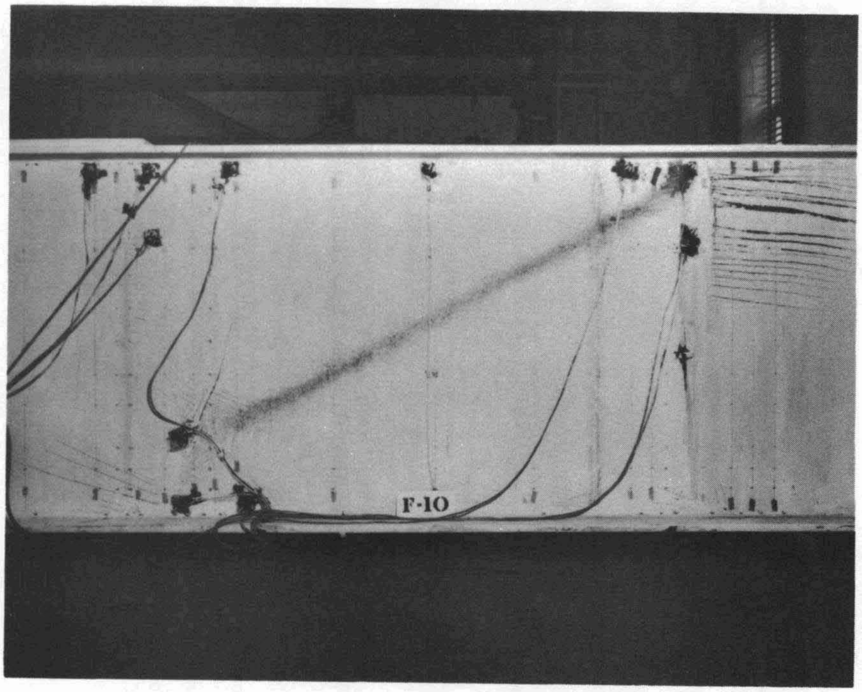

Fig. 11 F10, Panel 1 After Test

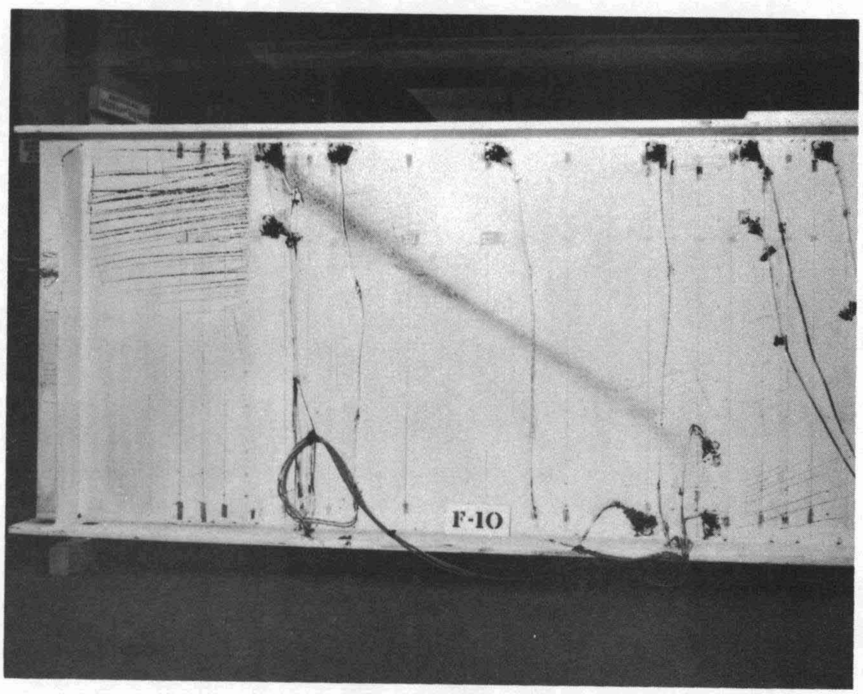

Fig. 12 F10, Panel 5 After Test

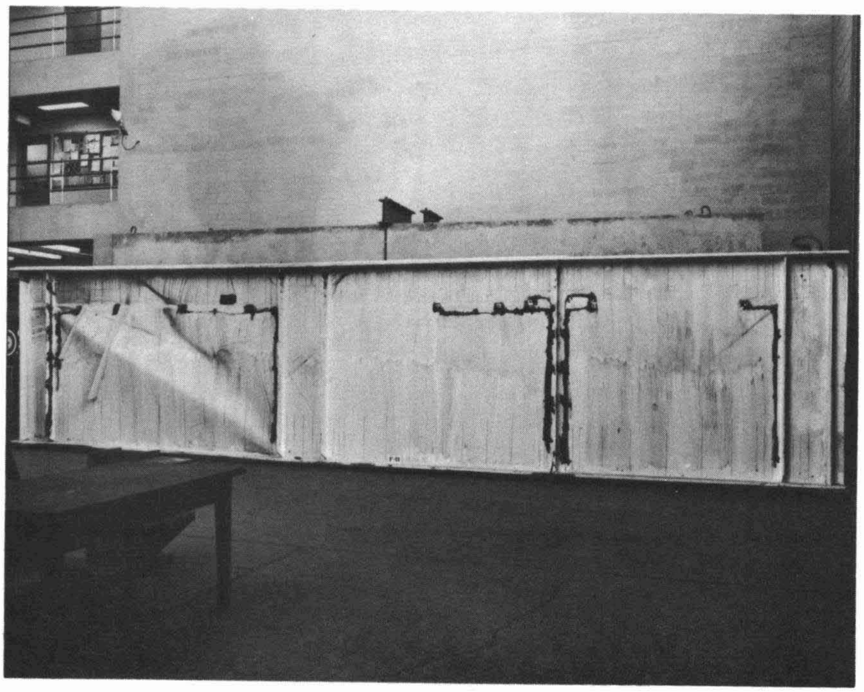

Fig. 13 Girder F11 After Testing

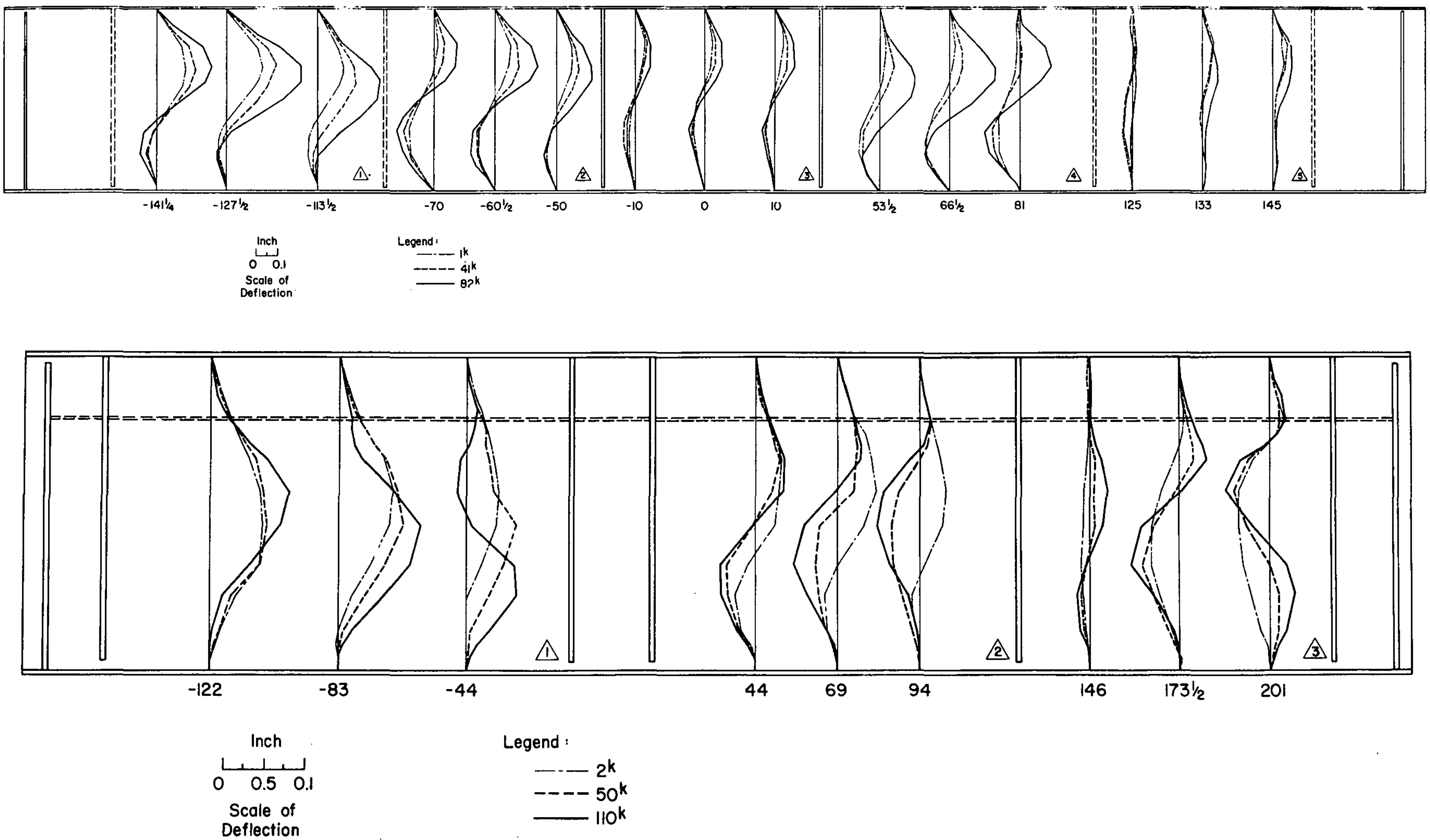

Fig. 14 Web Deflections, F10 and F11

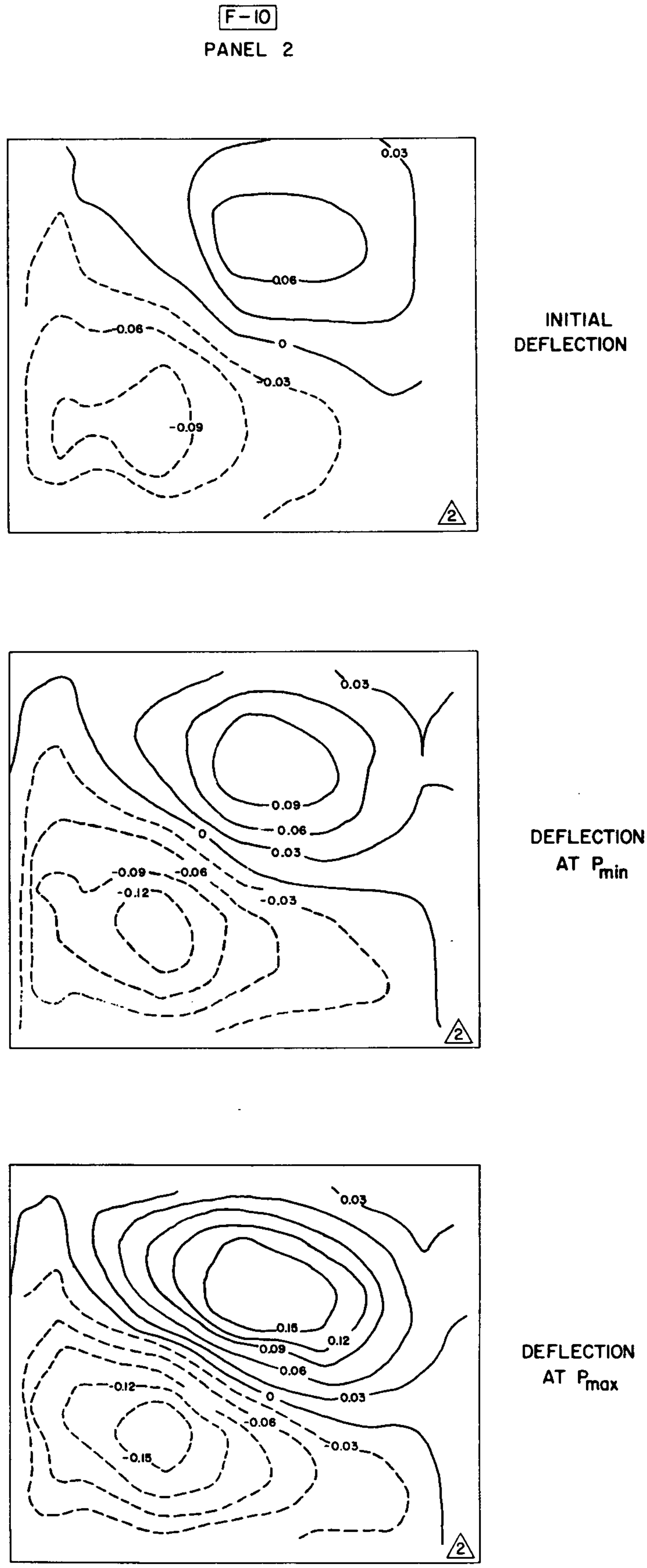

Fig. 15 Web Deflection Contours, F10 Panel 2

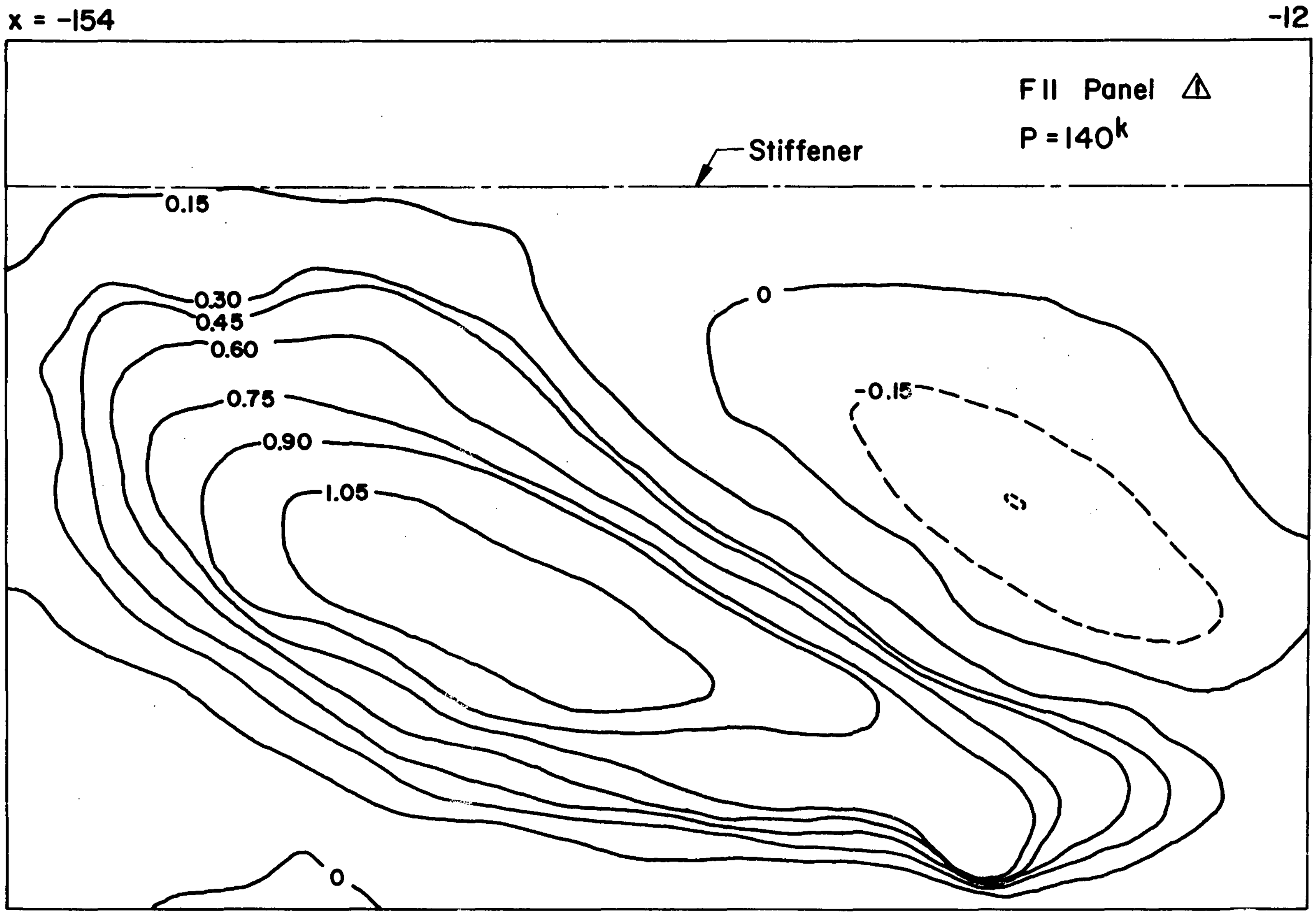

Fig. 16 Web Deflection Contours, F11 Panel 1

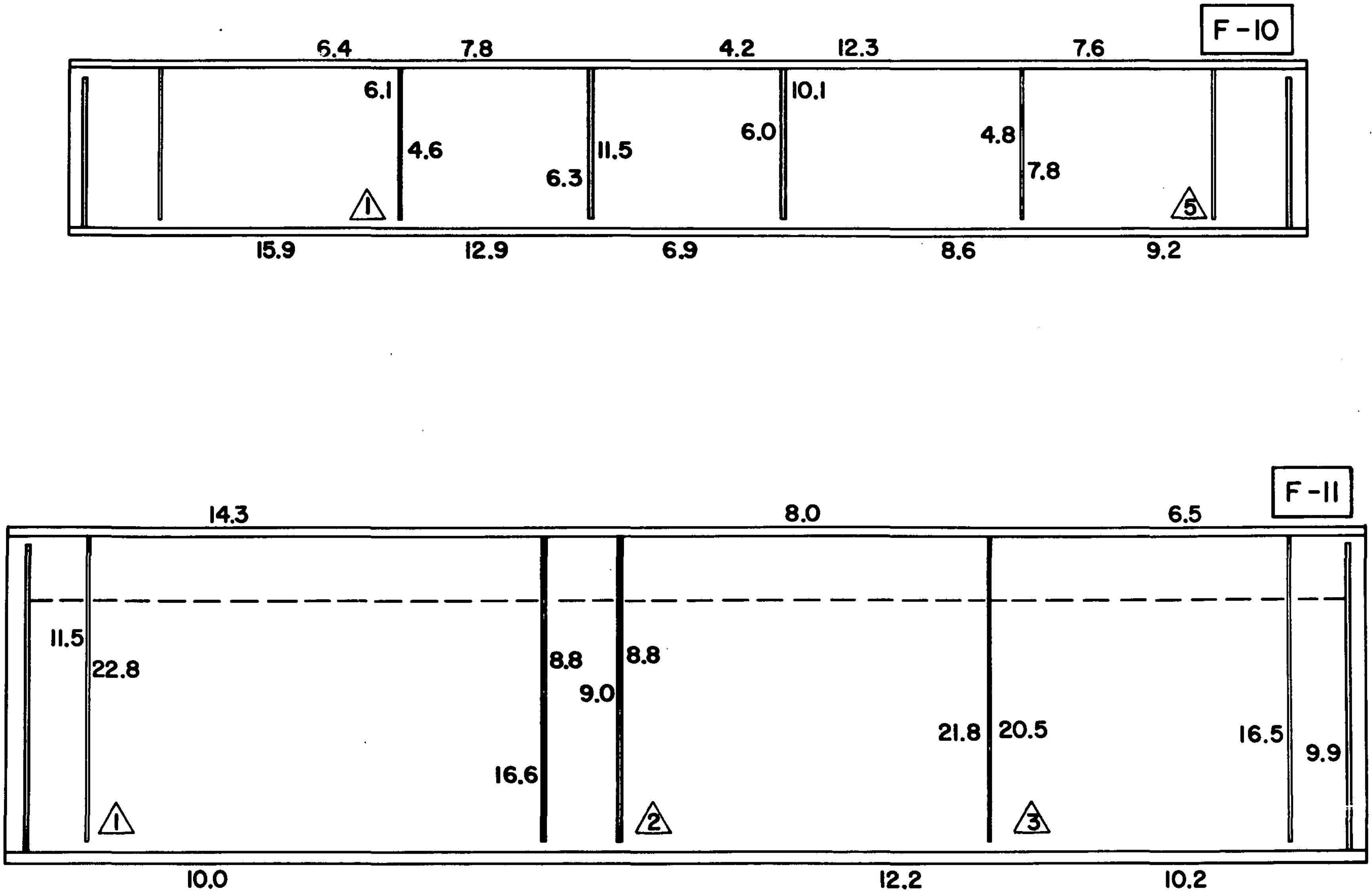

Fig. 17 Predicted Plate Bending Stresses at Boundary

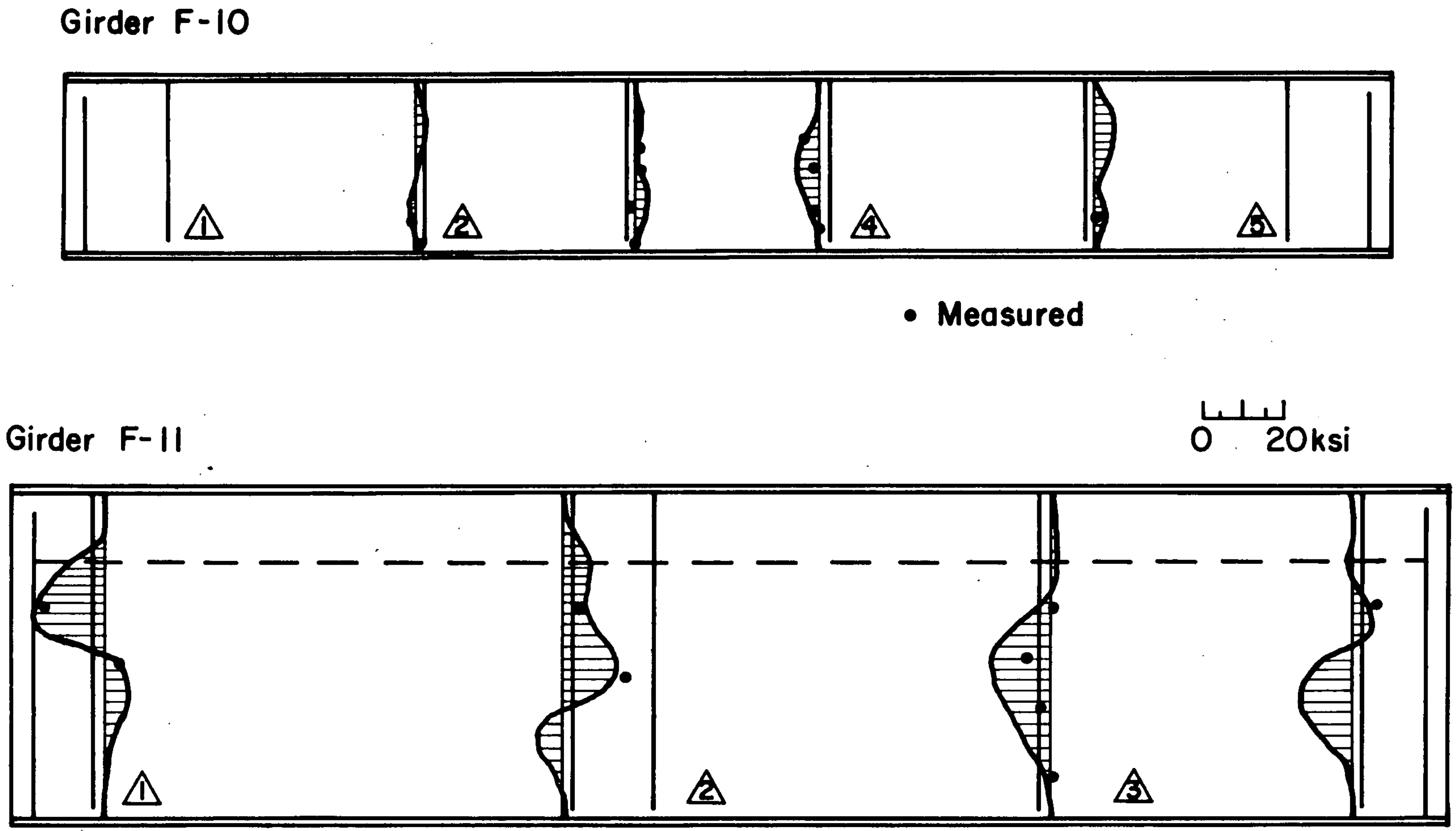

Fig. 18 Measured and Predicted Plate Bending Stresses

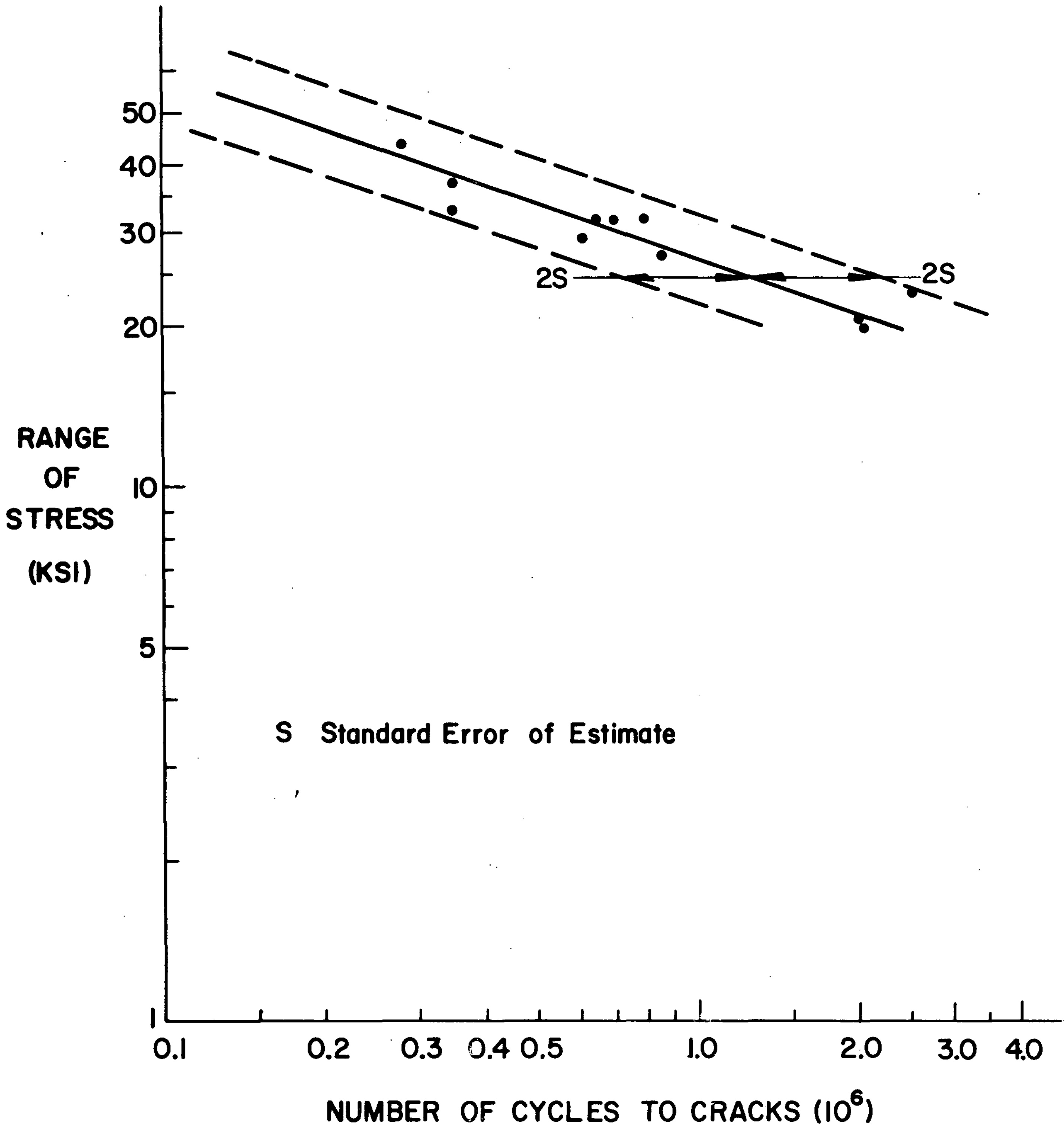

Fig. 19 Stress Range Versus Fatigue Life

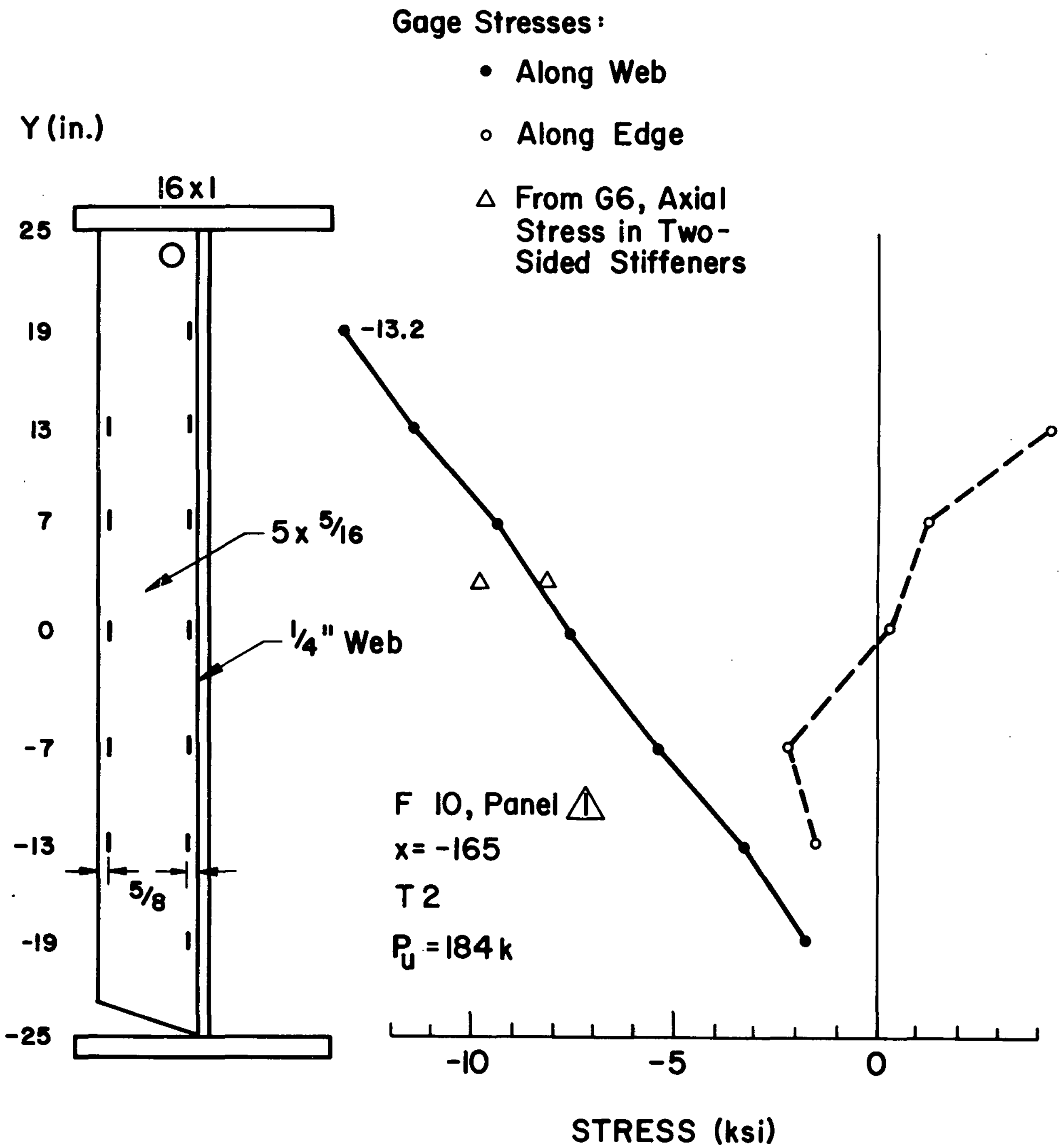

Fig. 20 Axial Stress Distribution on Transverse Stiffener F10

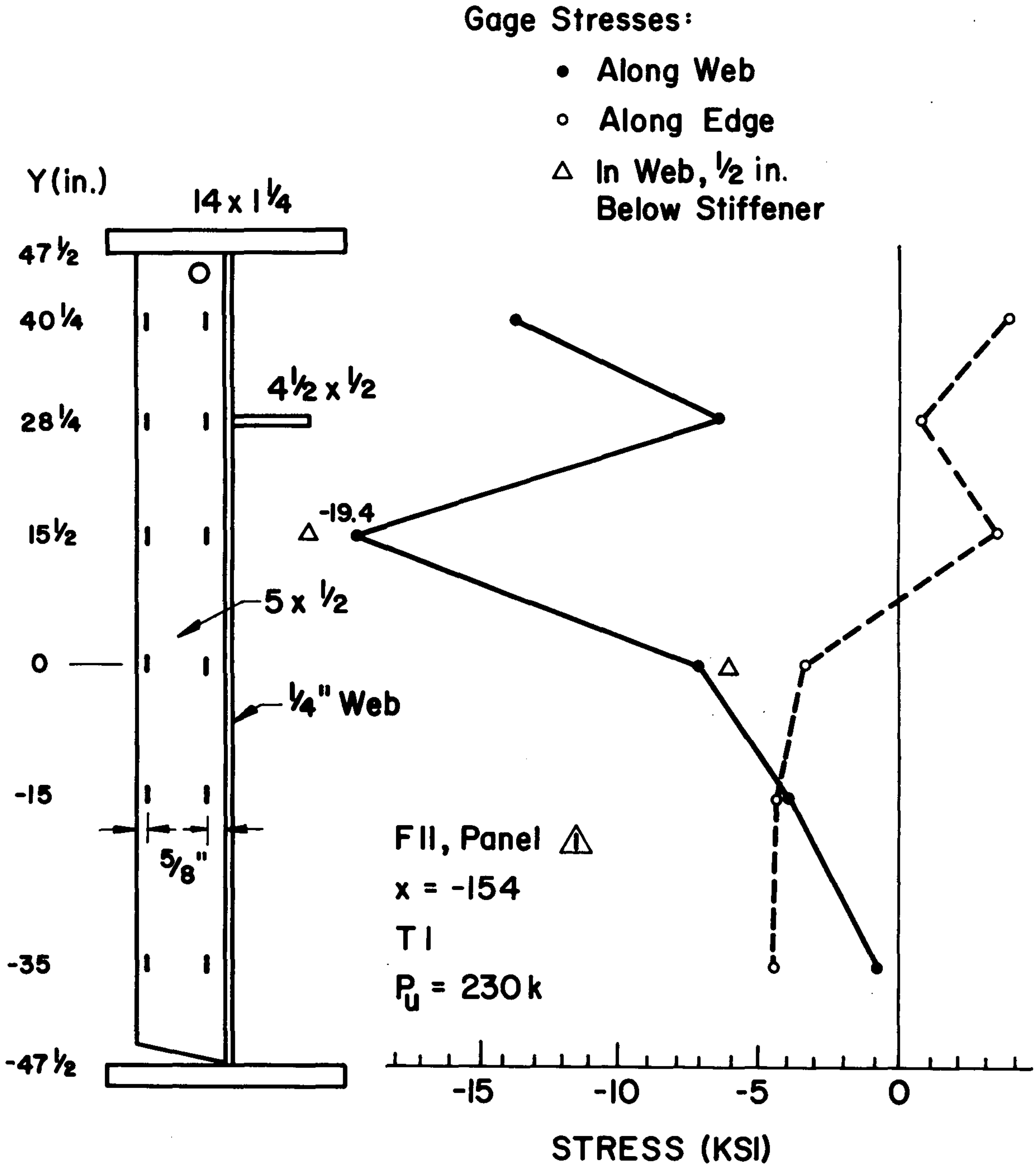

Fig. 21 Axial Stress Distribution on Transverse Stiffener F11

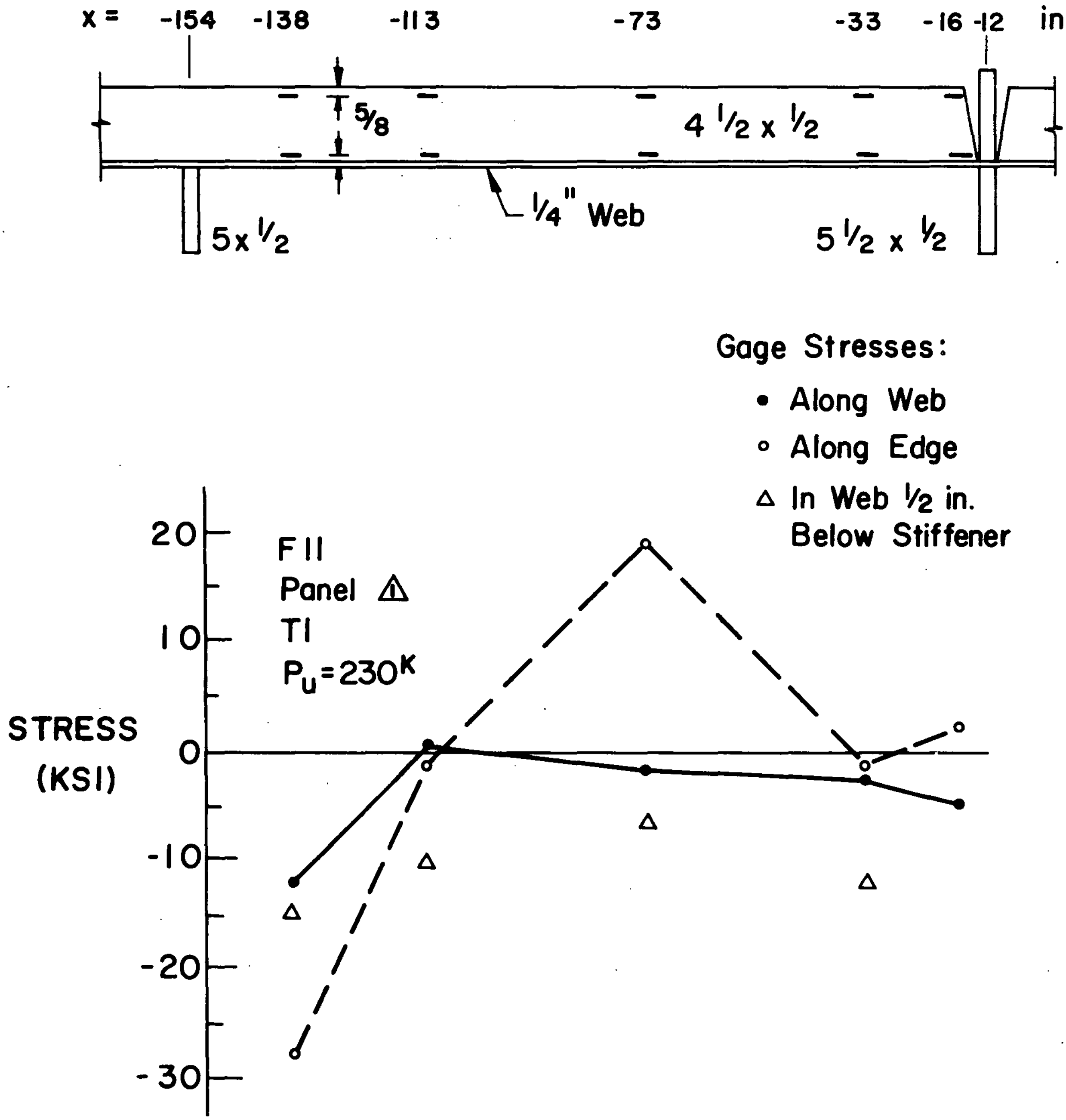

Fig. 22 Axial Stress Distribution on Longitudinal Stiffener, F11 Panel 1

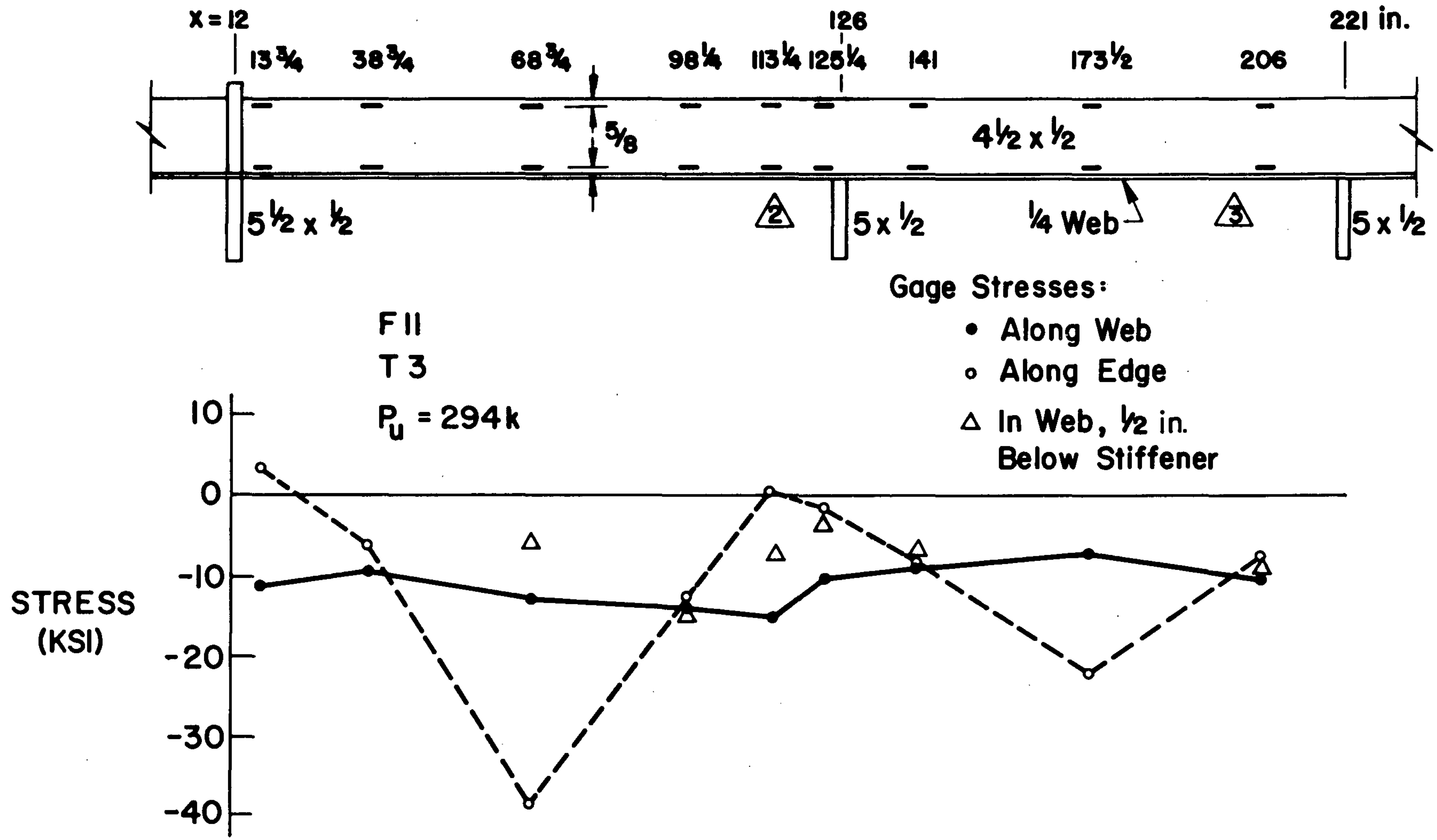

Fig. 23 Axial Stress Distribution on Longitudinal Stiffener, F11
Panel 2 and 3

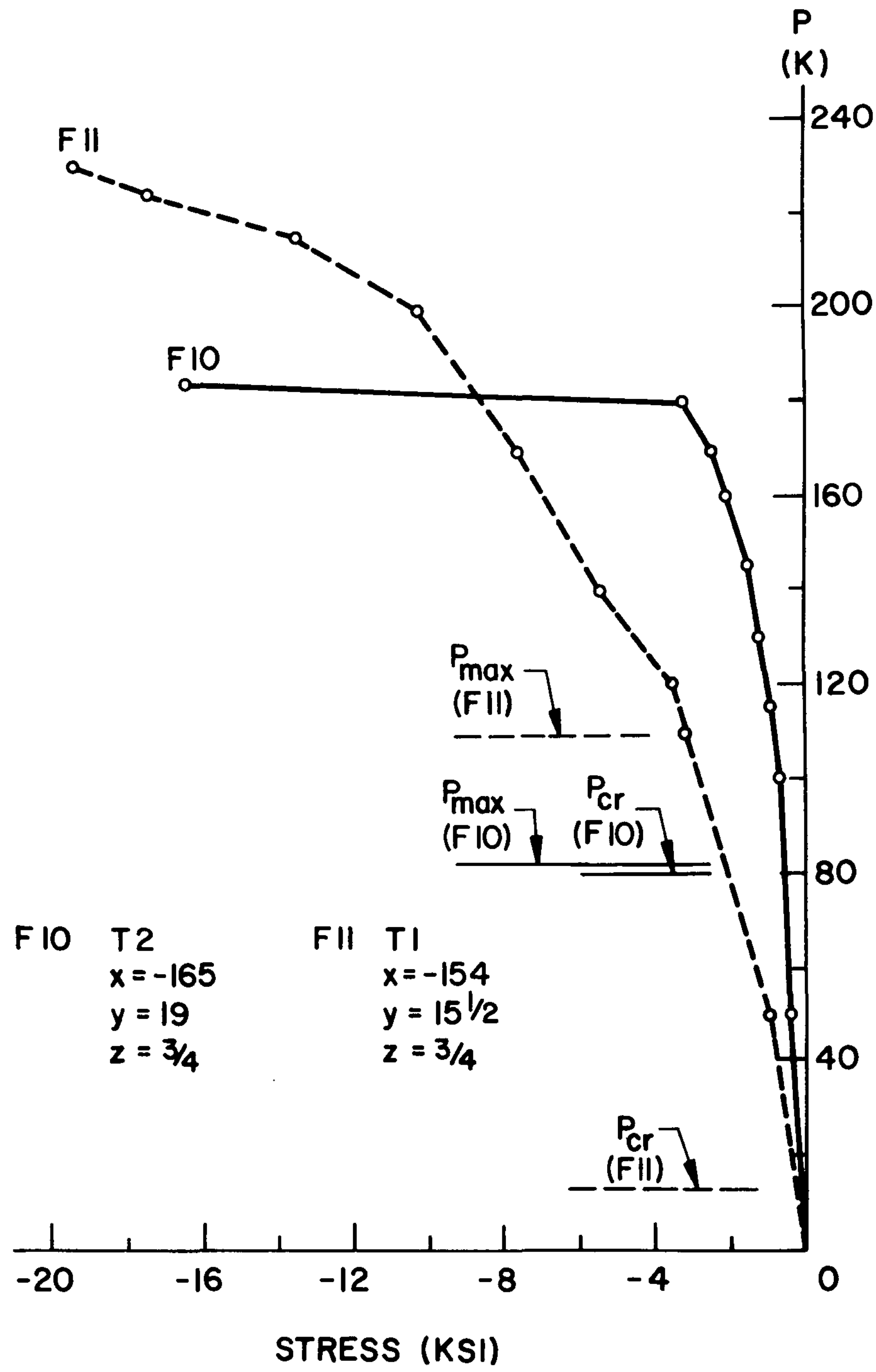

Fig. 24 Load Versus Axial Stress on Transverse Stiffeners F10 and F11

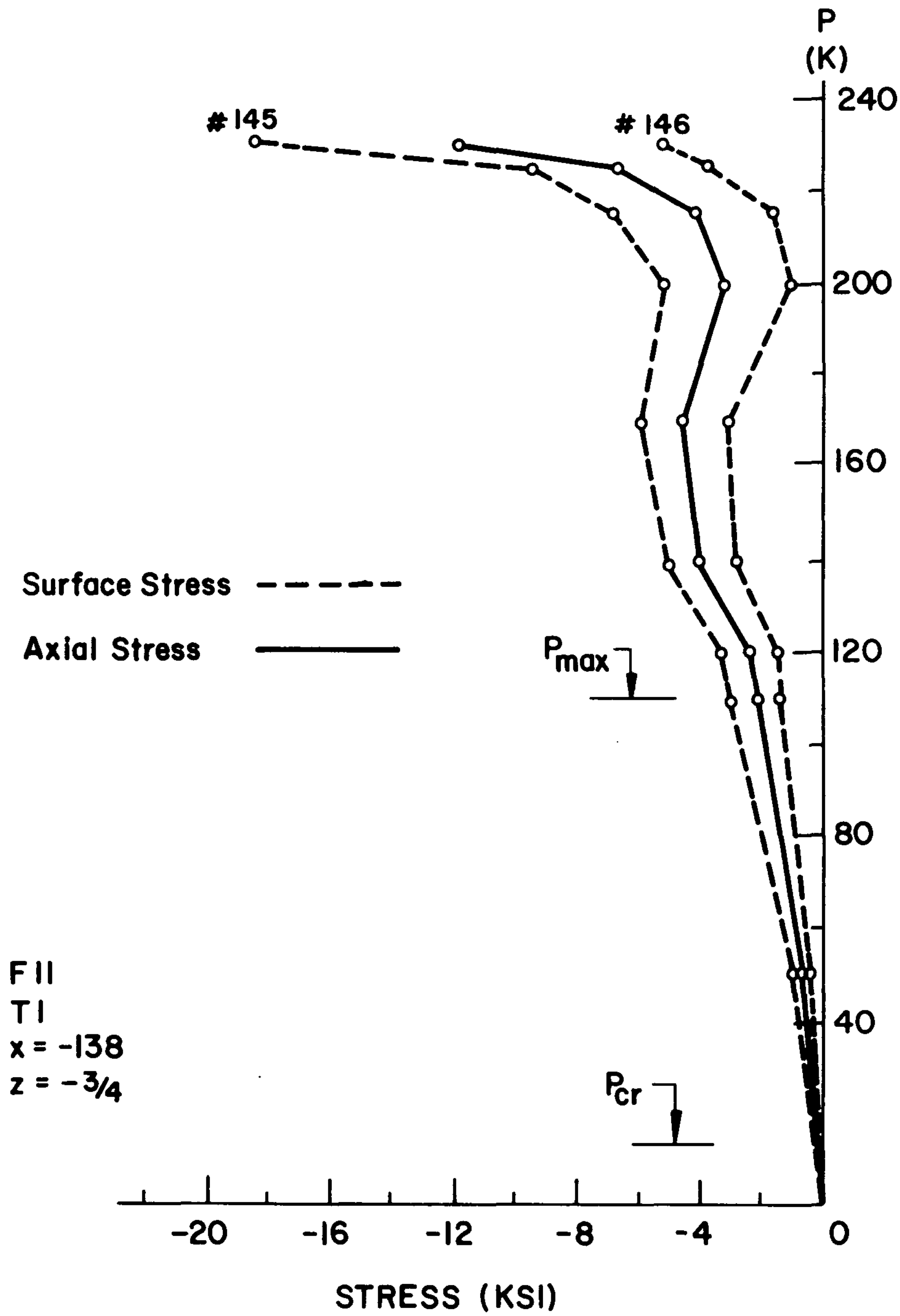

Fig. 25 Development of Longitudinal Stiffener, F11

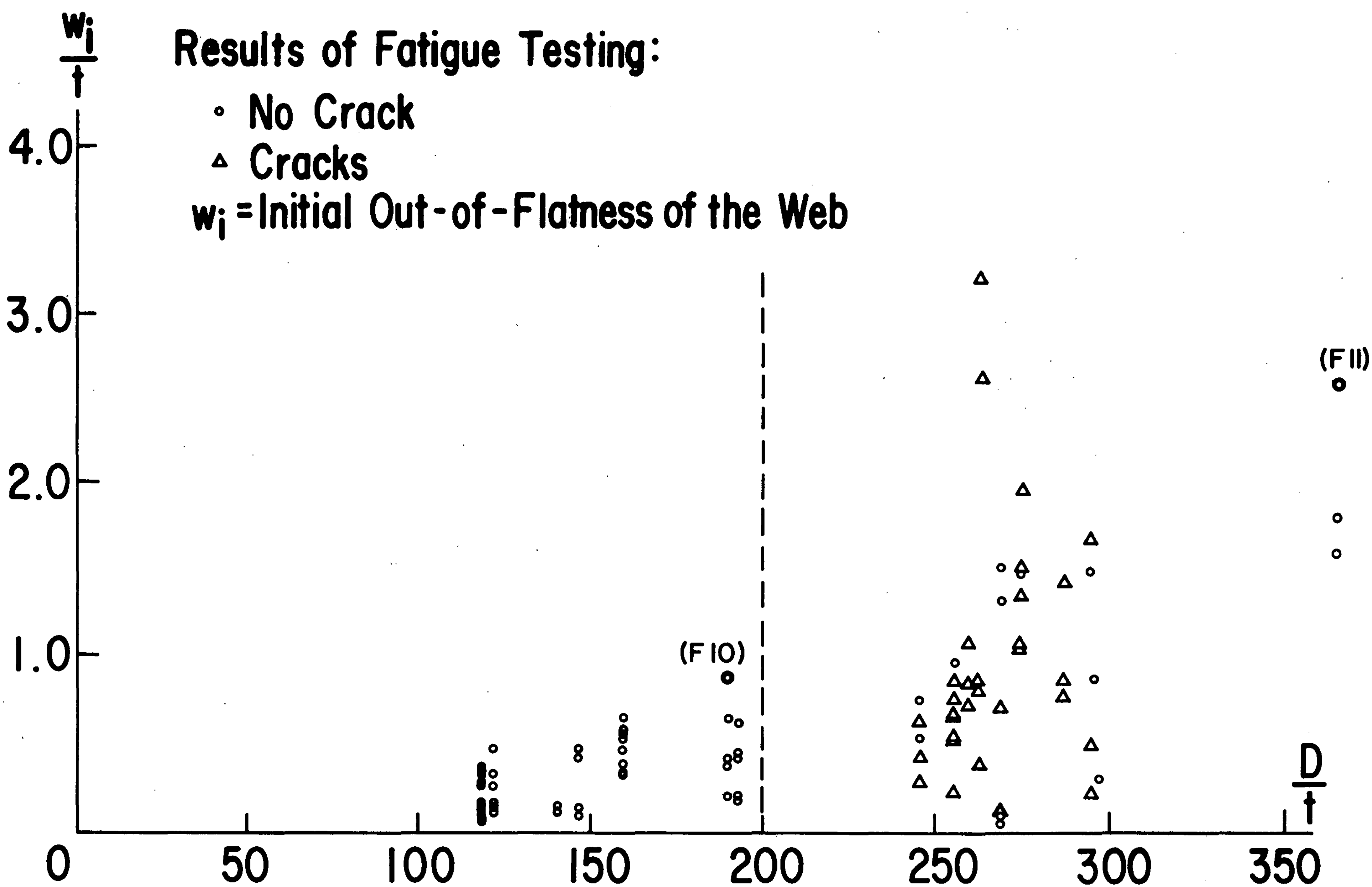

Fig. 26 Initial Deflection versus Slenderness Ratio

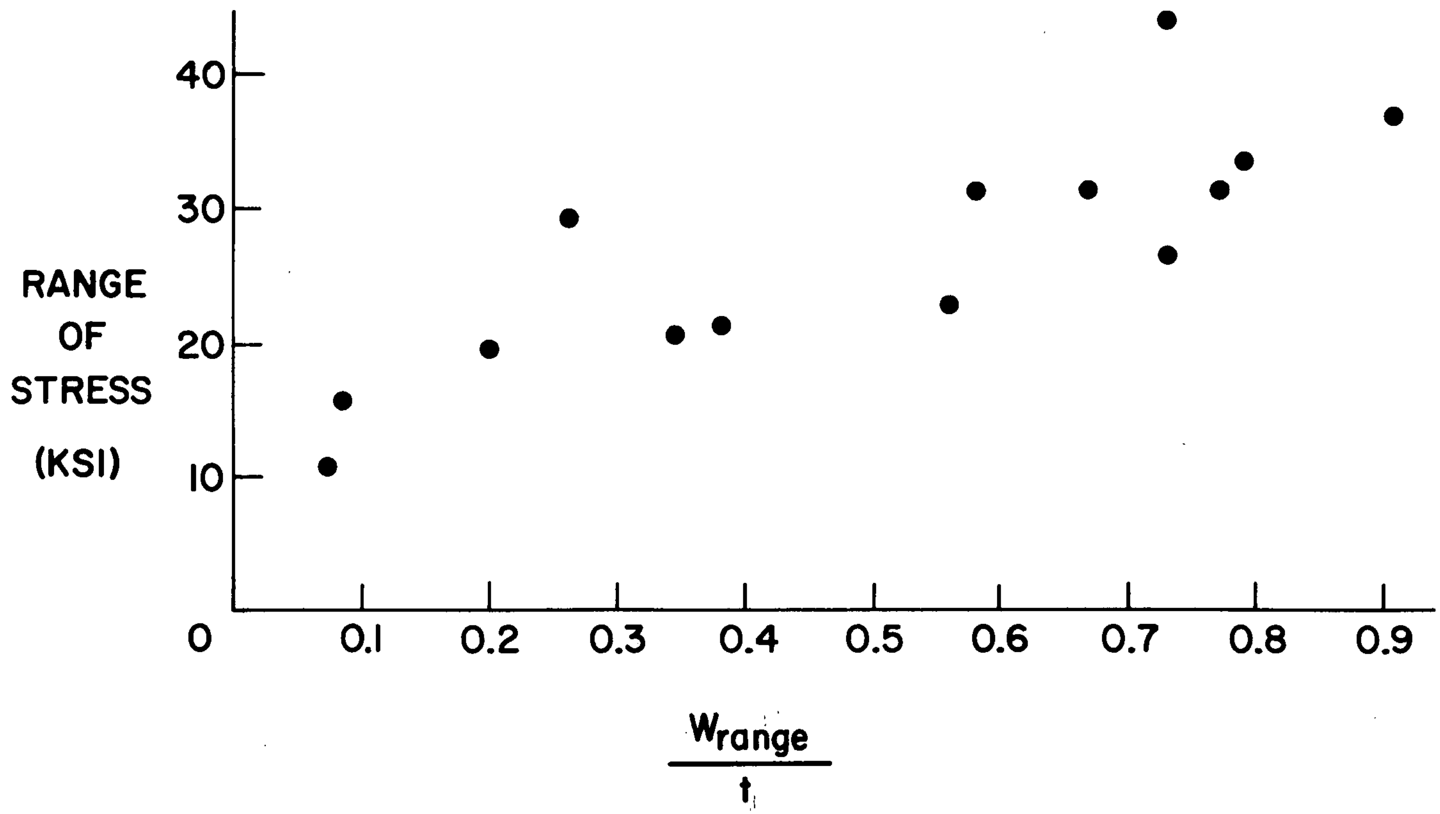

Fig. 27 Range of Stress versus Range of Web Deflection

REFERENCES

1. Basler, K., Yen, B.T., Mueller, J.A. and Thurlimann, B.
   WEB BUCKLING TESTS ON WELDED PLATE GIRDERS
   Bulletin No. 64, Welding Research Council, New York, September 1960.

2. Basler, K. and Thurlimann, B.
   STRENGTH OF PLATE GIRDERS IN BENDING, Proceedings, ASCE, Vol. 87, No. ST6, August 1961.

3. Basler, K.
   STRENGTH OF PLATE GIRDERS IN SHEAR, Proceedings, ASCE, Vol. 87, No. ST7, October 1961.

4. Basler, K.
   STRENGTH OF PLATE GIRDERS UNDER COMBINED BENDING AND SHEAR, Proceedings, ASCE, Vol. 87, No. ST7, October 1961.

5. AISC
   SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS, American Institute of Steel Construction, New York, April 1963.

6. Yen, B.T.
   ON THE FATIGUE STRENGTH OF WELDED PLATE GIRDERS, Fritz Engineering Laboratory Report No. 303.1, Lehigh University, Bethlehem, Pa., November 1963.

7. Yen, B. T. and Mueller, J. A.
   FATIGUE TESTS OF LARGE-SIZE WELDED PLATE GIRDERS, Welding Research Council Bulletin No. 118, November 1966.

8. Goodpasture, D.W. and Stallmeyer, J.E.
   FATIGUE BEHAVIOR OF WELDED THIN WEB GIRDERS AS INFLUENCED BY WEB DISTORTION AND BOUNDARY RIGIDITY, Structural Research Series No. 328, Civil Engineering Studies, University of Illinois, Urbana, Illinois, August, 1967.

9. Lew, H. S., and Toprac, A. A.
   FATIGUE STRENGTH OF HYBRID PLATE GIRDERS UNDER CONSTANT MOMENT, Highway Research Record, No. 167, 1967.

10. Mueller, J. A. and Yen, B. T.
    GIRDER WEB BOUNDARY STRESSES AND FATIGUE, Bulletin No. 127, Welding Research Council, New York, January 1968.

11. Yen, B. T.
DESIGN RECOMMENDATIONS FOR BRIDGE PLATE GIRDERS, Lehigh University, Fritz Engineering Laboratory Report No. 327.6 June, 1969.

12. Vincent, G.S.
TENTATIVE CRITERIA FOR LOAD FACTOR DESIGN OF STEEL HIGHWAY BRIDGES, American Iron and Steel Institute, New York, February 1968.

13. AASHO
STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, American Association of State Highway Officials, Ninth Edition 1965.

14. Cooper, P.B., Lew, H. S. and Yen, B. T.
WELDED CONSTRUCTIONAL ALLOY STEEL PLATE GIRDERS, Proceedings, ASCE, Vol. 90, No. ST1, February 1964.

15. Carskaddan, P.S
BENDING OF DEEP GIRDERS WITH A514 STEEL FLANGES To be published in Proceedings, ASCE, ST,(1969).

16. Basler, K. and Thurlimann, B.
STRENGTH OF PLATE GIRDERS AND PLATE GIRDER RESEARCH Proceedings, National Engineering Conference, AISC 1958 and 1959.

17. Cooper, P.B.
STRENGTH OF LONGITUDINALLY STIFFENED PLATE GIRDERS, Proceedings, ASCE Vol. 93, St. 2, April 1967.

18. AWS
SPECIFICATIONS FOR WELDED HIGHWAY AND RAILWAY BRIDGES, American Welding Society, New York, 1966.

19. Timoshenko, S. and Gere, J. M.
THEORY OF ELASTIC STABILITY, Second Edition, McGraw-Hill Book Company, New York, 1961.

20. Munse, W. H.
FATIGUE OF WELDED STEEL STRUCTURES, Welding Research Council, New York, 1964.

21. Bleich, F
BUCKLING STRENGTH OF METAL STRUCTURES, McGraw-Hill Book Company Inc., New York, 1952.

ACKNOWLEDGMENTS

The research work on which this report is based was carried out at Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University. Dr. Lynn S. Beedle is the Director of the Laboratory and Dr. David A. VanHorn is the Chairman of the Civil Engineering Department.

The work was jointly sponsored by the Pennsylvania Department of Highways, the United States Bureau of Public Roads, the American Iron and Steel Institute and the Welding Research Council. The project was under the guidance of the Welded Plate Girder Subcommittee of WRC and the direct supervision of the WRC Task Group on Design of Girders. The financial support of these sponsors and the guiding suggestions extended by the individual members of the Task Group and Subcommittee are gratefully acknowledged.

The authors wish to thank Mr. John M. Gera for preparing the drawings; Mrs. Jean Leddon in typing the report; and Messrs. John C. Nothelfer and Kerry A. Drake for helping with the reduction of data.