EXPERIMENTAL INVESTIGATION OF RIGID FRAMES INCLUDING KNEE CONNECTION STUDIES
-FRAME ASSEMBLY TESTS-

Research Report

by

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Submitted to

MESCO Metal Buildings Corporation
Grapevine, Texas

Report No. FSEL/MESCO 85-01

June 1985

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CHAPTER 1

INTRODUCTION

1.1 Background

An extensive research program to study the behavior of single story, single bay, rigid frames is being conducted at the Fears Structural Engineering Laboratory, University of Oklahoma under the sponsorship of MESCO Metal Buildings Corporation, hereafter referred to as MESCO. The frames are fabricated from plates and consist of three or four major components: two columns and one or two rafter sections. Figure 1.1 is an elevation view of a typical frame. Both the columns and the rafter sections can be tapered members. Connection between the components is made using end-plates and high-strength bolts.

MESCO fabricates the frame components using plate material having a nominal yield strength of 50 ksi. Welding of the rafter and column flange-to-web connection is done only on one side of the web plate using the submerged arc process. A325 bolts are used in the rafter-to-column connections. The frames are designed using a computer program developed by MESCO. Applicable provisions of the AISC Specification are satisfied.

Details and results of frame assembly (knee area) studies are included in this volume. Supporting test data are found in the appendices. Full scale frame tests are described in subsequent reports.

1.2 Scope of Research

To study the structural characteristics of frames configured as shown in Figure 1.1, two series of tests were conducted:

1. Frame assembly or knee-area tests (FA-series),
2. Full frame tests (FR-series).

The purpose of each test series is as follows.

Frame Assembly Tests. The objective of the frame assembly tests was to study the behavior of the knee portion of typical frames. Eight specimens (FA-1 thru FA-8) were used for the tests. Each specimen consisted of a rafter section and a column section connected as shown in Detail A of Figure 1.1. The specimens were subjected to a single applied force, located so that the moment, shear and thrust in the knee area approximated actual design values for combined dead plus live loads. Special test setups were constructed to provide lateral support equivalent to that provided in an actual building and to facilitate loading.

Details of the FA- testing program, comparison of experimental and analytical deflections and failure loads are found in subsequent chapters.

Frame Tests. To verify analytical procedures used by

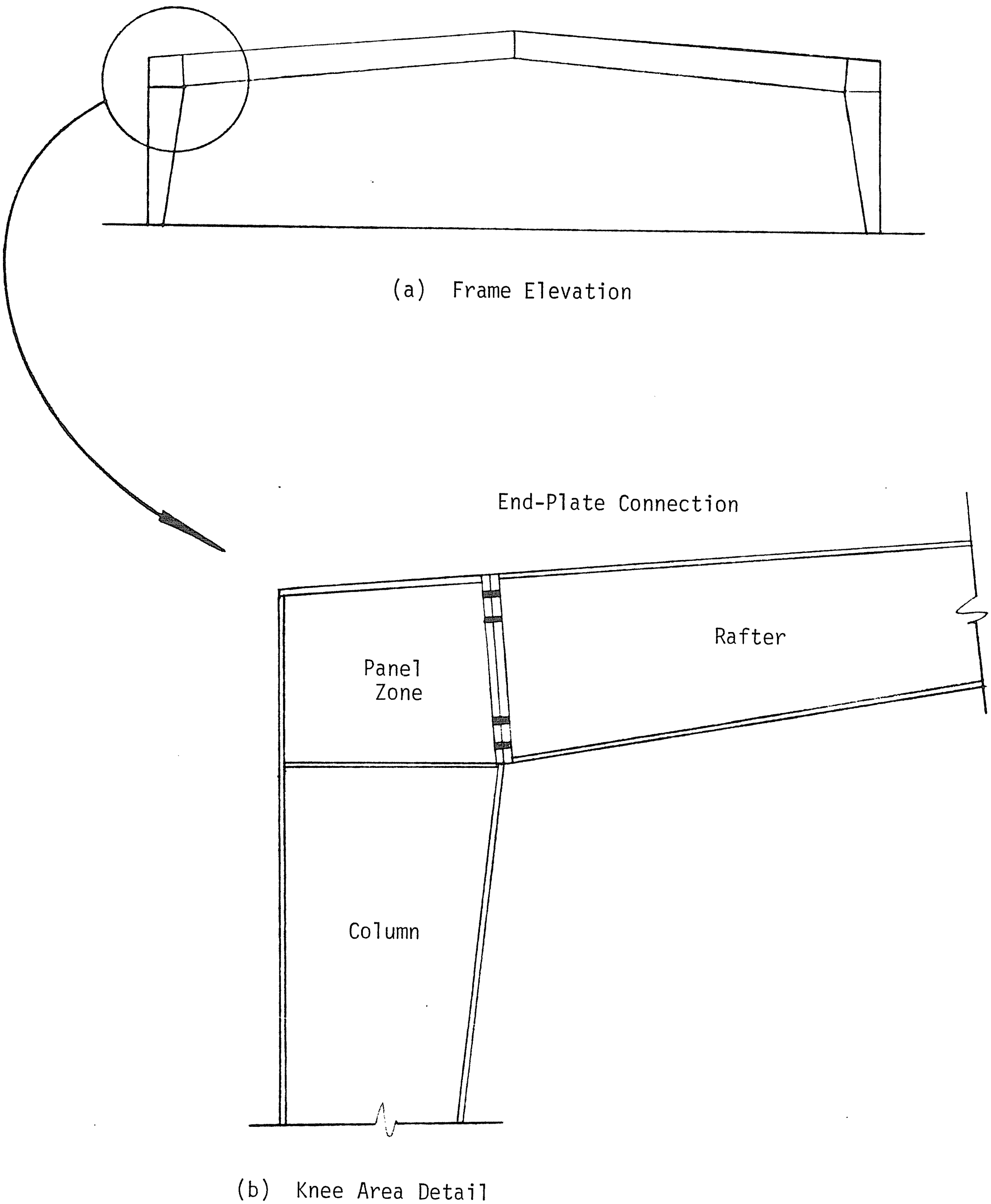


Figure 1.1 Typical Rigid Frame and Connection Detail

MESCO to predict frame strength and stiffness, single bays of typical buildings were constructed over the laboratory reaction floor. The setup consisted of two frames spaced 24 ft. on center, roof purlins and wall girts, flange brace angles, and side wall rod braces. Roof deck and side wall panels were installed. Simulated live and wind loads, alone and in combination, were applied to the frames using hydraulic cylinders. A total of six tests were conducted: (1) working live load on one frame (both slopes loaded), (2) unbalanced live load on both frames simultaneously (one slope on each frame loaded), (3) wind load on both frames simultaneously, (4) combined wind and unbalanced live load on both frames simultaneously, and (5) full live load on each frame to failure (2 tests).

Complete test results and comparisons with analytical predictions are found in separate reports.

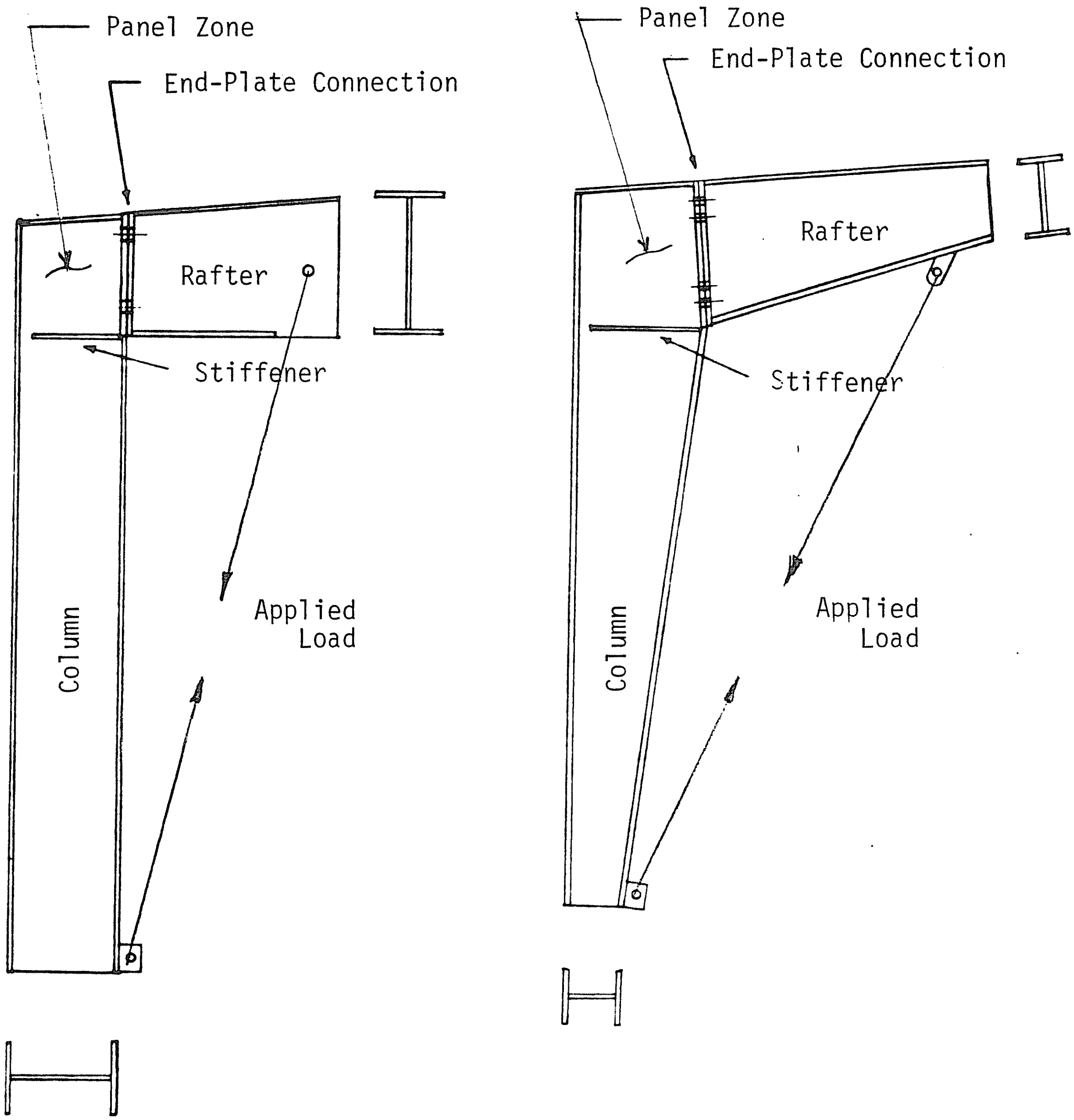
CHAPTER II

TESTING DETAILS

2.1 General

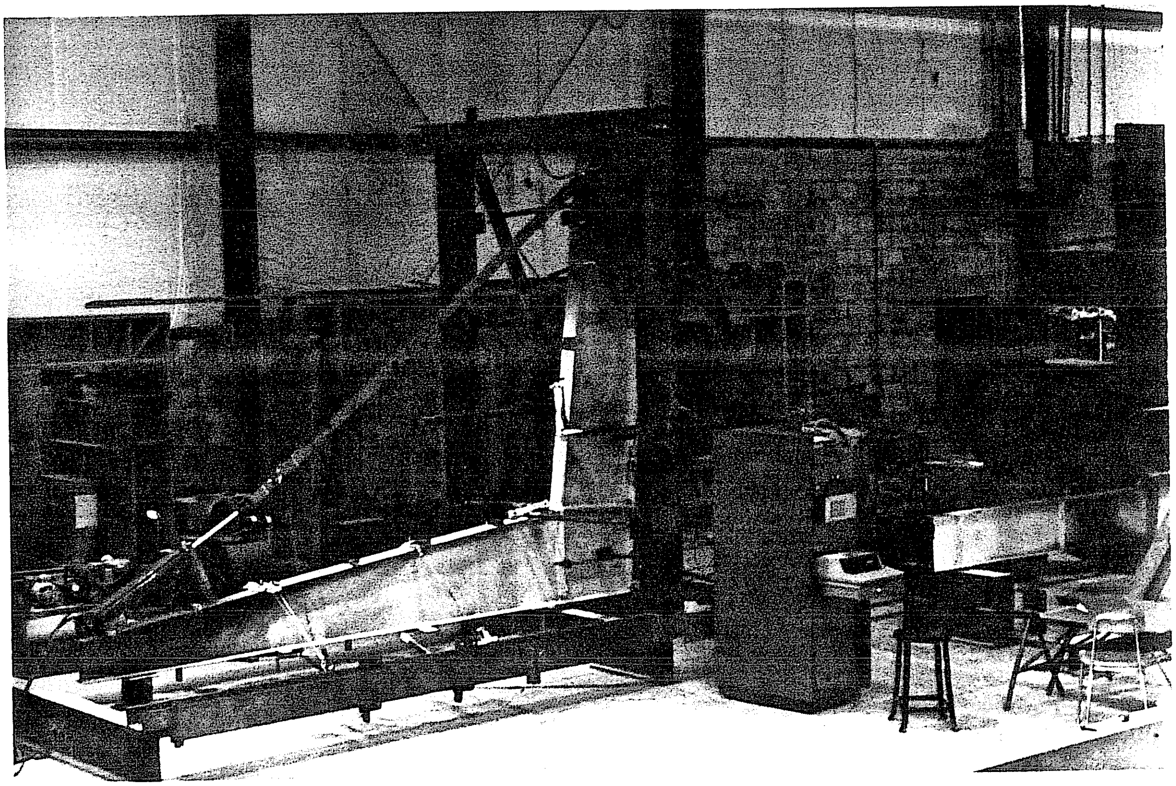
The objective of the frame assembly tests was to study the behavior of the knee portion of typical rigid frames when subjected to moment, shear and thrust caused by gravity loading. Each specimen consisted of a rafter section and a column section connected using a moment end-plate connection. The assemblies were subjected to a single force as shown in Figure 2.1. The configuration of the test specimen and location of the line of action of the applied force was determined for each specimen so that the moment, shear and thrust at the end-plate connection were identical to actual frame design values for combined dead plus full live loads. The end-plate connections for all specimens were designed to have a strength greater than required so that failure would occur in the members and not the connection.

To facilitate loading and support, the specimens were tested in a rotated position relative to use in a building. Figures 2.2 and 2.3 are photographs and schematic drawings of the test setups.

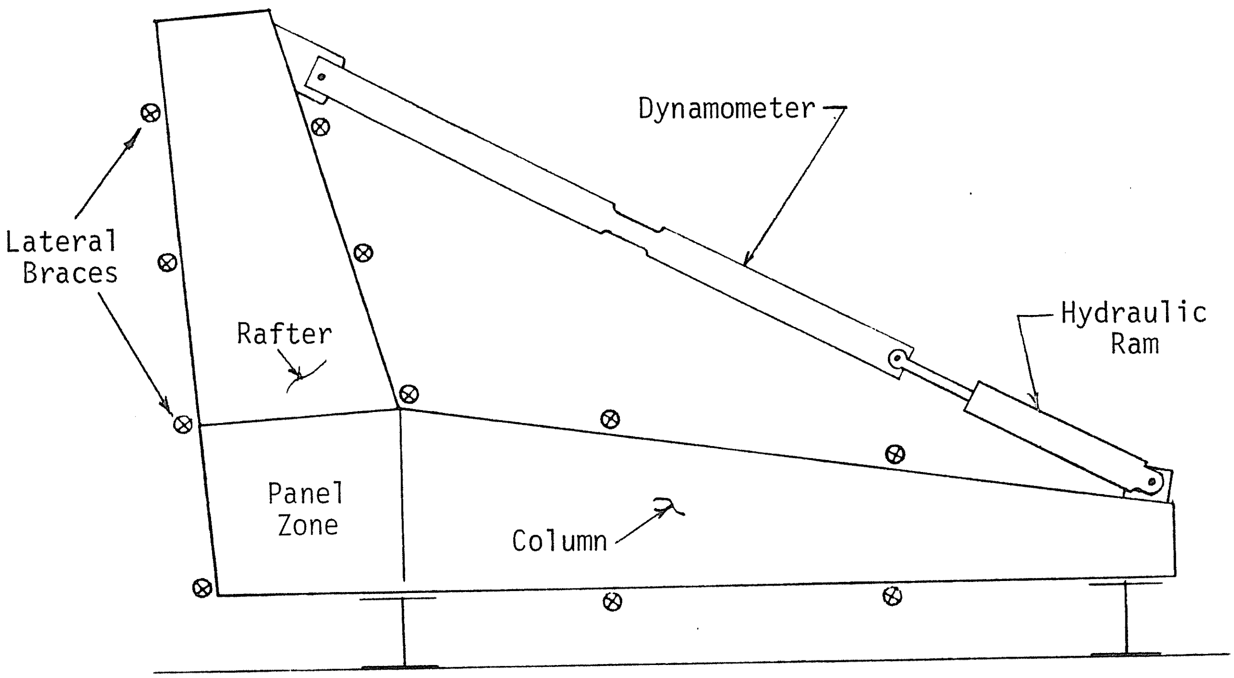


(a) Specimens FA-1 and FA-2 (b) Specimens FA-3 to FA-8

Figure 2.1 Test Specimens

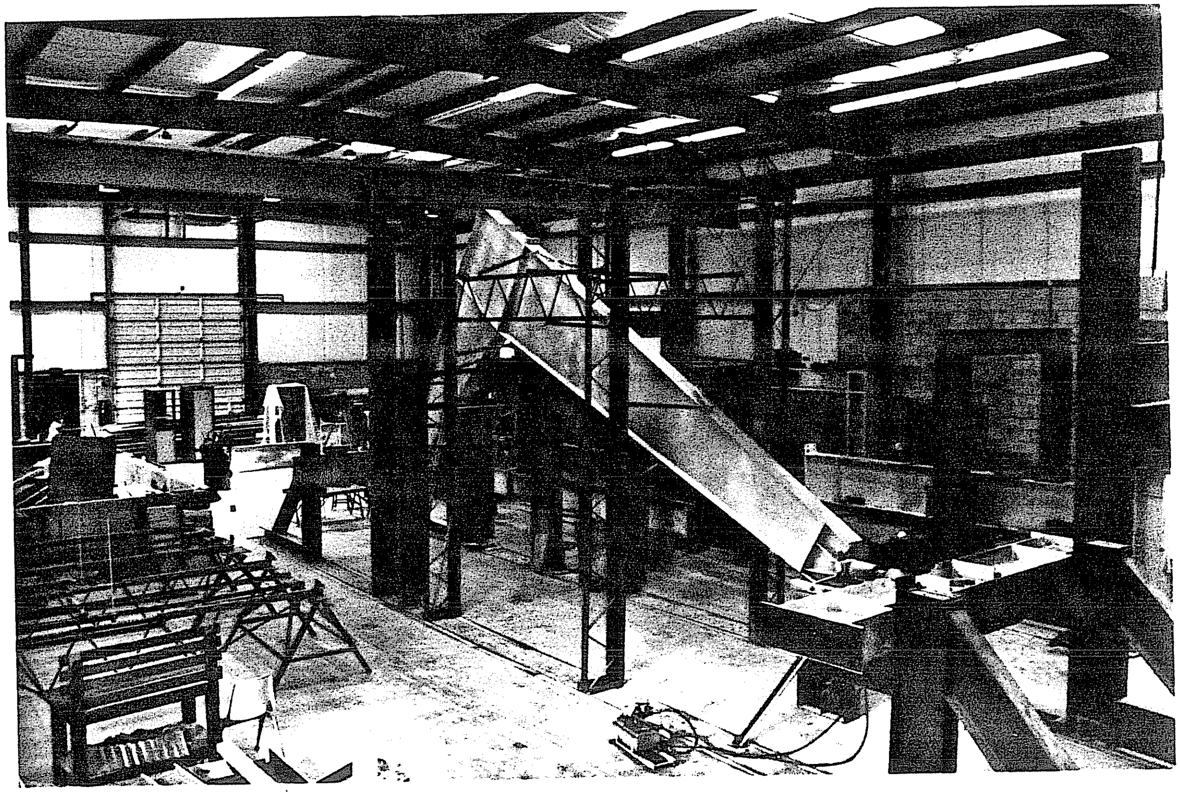


(a) Specimen FA-4 in Test Setup

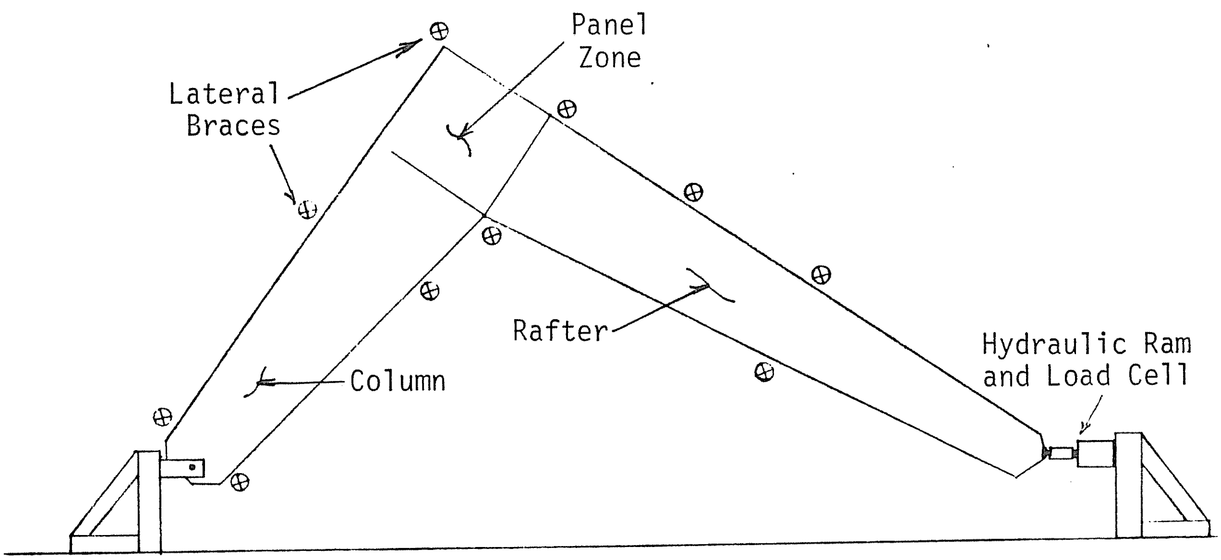


(b) Typical Test Setup

Figure 2.2 Test Setup, Tests FA-1 to FA-5, FA-7, FA-8



(a) Specimen FA-6 in Test Setup



(b) Test FA-6 Setup

Figure 2.3 Test Setup, Test FA-6

2.2 Description of Specimens

Each rafter and column section was fabricated by MESCO Metal Building Corporation as a "I" section made from A572 Grade 50 steel plate and bar, A570 grade E 50 ksi yield steel sheet. Specimens FA-1 and FA-2 consisted of a tapered rafter section and a constant cross section (prismatic) column section; all other specimens consisted of tapered rafters and columns sections. Stiffeners were welded to both sides of each column web at the reentrant corner, a cap plate was welded to the column, and end plates were welded to the members shown in Figure 2.1. The area bounded by the column cap plate, end-plate, web stiffener and outside flange is referred to here as the "panel zone." The column web plate within the panel zone will be referred to as the "panel zone web plate."

A325 bolts were used to connect the column and rafter sections. The instrumented bolts (see Section 2.4) were first tightened with the data acquisition system connected and then the remaining bolts were tightened by "feel" to the same torque.

2.3 Test Setup

All test specimens, except Specimen FA-6, were supported in and laterally braced to a frame as shown in Figure 2.2(a). Lateral brace locations for each specimen are shown in the appropriate appendix. The loading mechanism consisted of a tension hydraulic ram and dynamometer connected to the ends of the specimen as shown in Figure 2.2(b).

The specimen for Test FA-6 was considerably larger than the others and required a separate test setup. The specimen was simply supported and the flanges were laterally braced as shown in Figure 2.3(a). The loading mechanism consisted of a compression hydraulic ram and load cell as shown in Figure 2.3(b).

All specimens were whitewashed prior to testing so that local yielding, as evidenced by flaking of mill scale, could be detected during the test.

2.4 Instrumentation

Test data were obtained through the use of displacement transducers, a transit and scales, dynamometers, bolt strain gauges, strain gauges and a strain rosette. Typical instrument locations are shown in Figures 2.4 and 2.5.

A wire type displacement transducer was used to measure the shortening of the chord connecting the specimen ends. A probe type displacement transducer was used to measure out-of-plane web displacement at the center of the panel zone. This probe was supported by the column cap plate and column web stiffener so that all measurements were relative to the top plate and stiffener. Scales were mounted on the compression flanges of both the column and rafter sections (except for Test FA-6) so that lateral movement could be measured with a transit. A dynamometer was used to measure the force applied by the hydraulic ram.

For at least two tension bolts in each test, strain gauges were installed in holes drilled through the head and

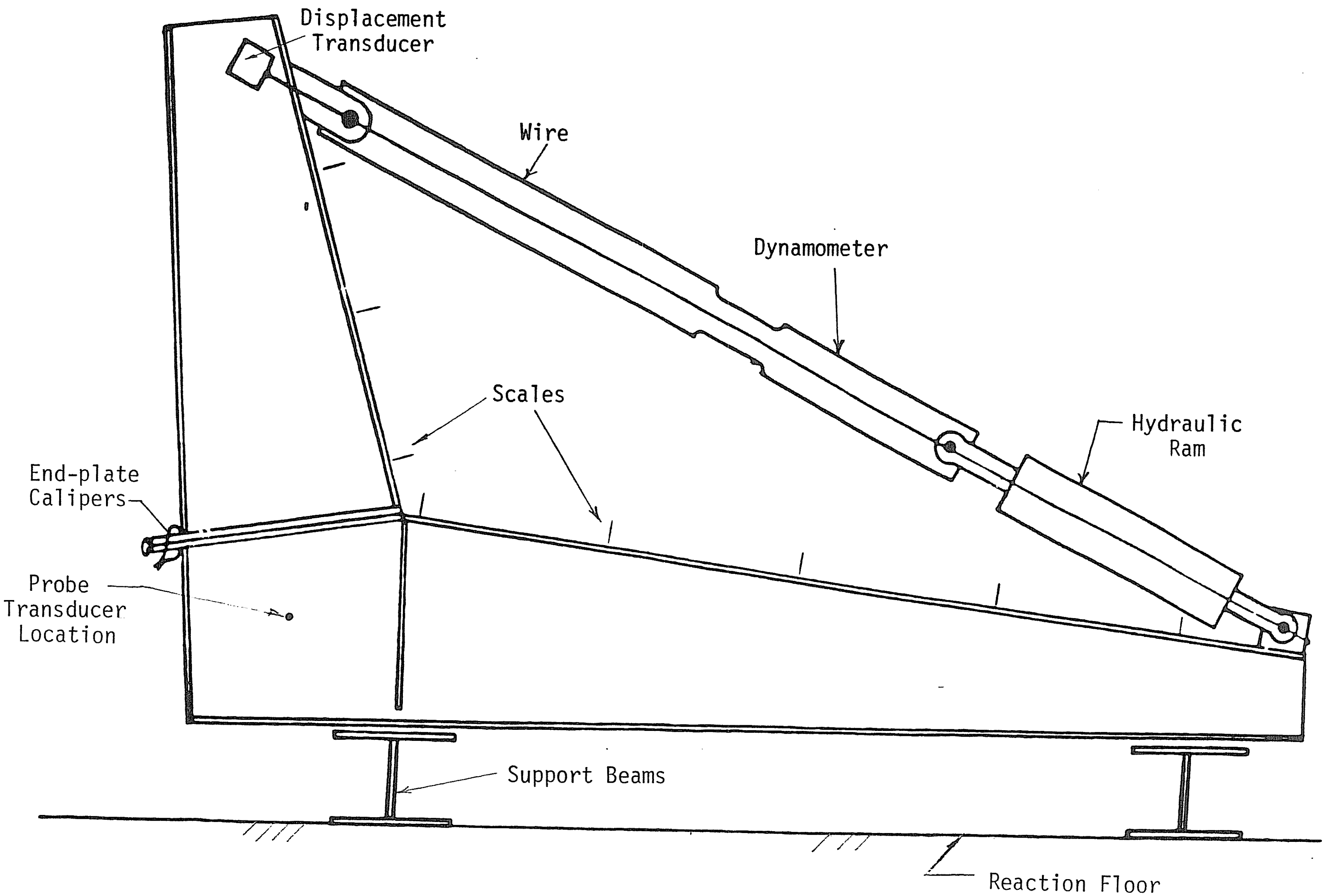


Figure 2.4 Typical Instrumentation

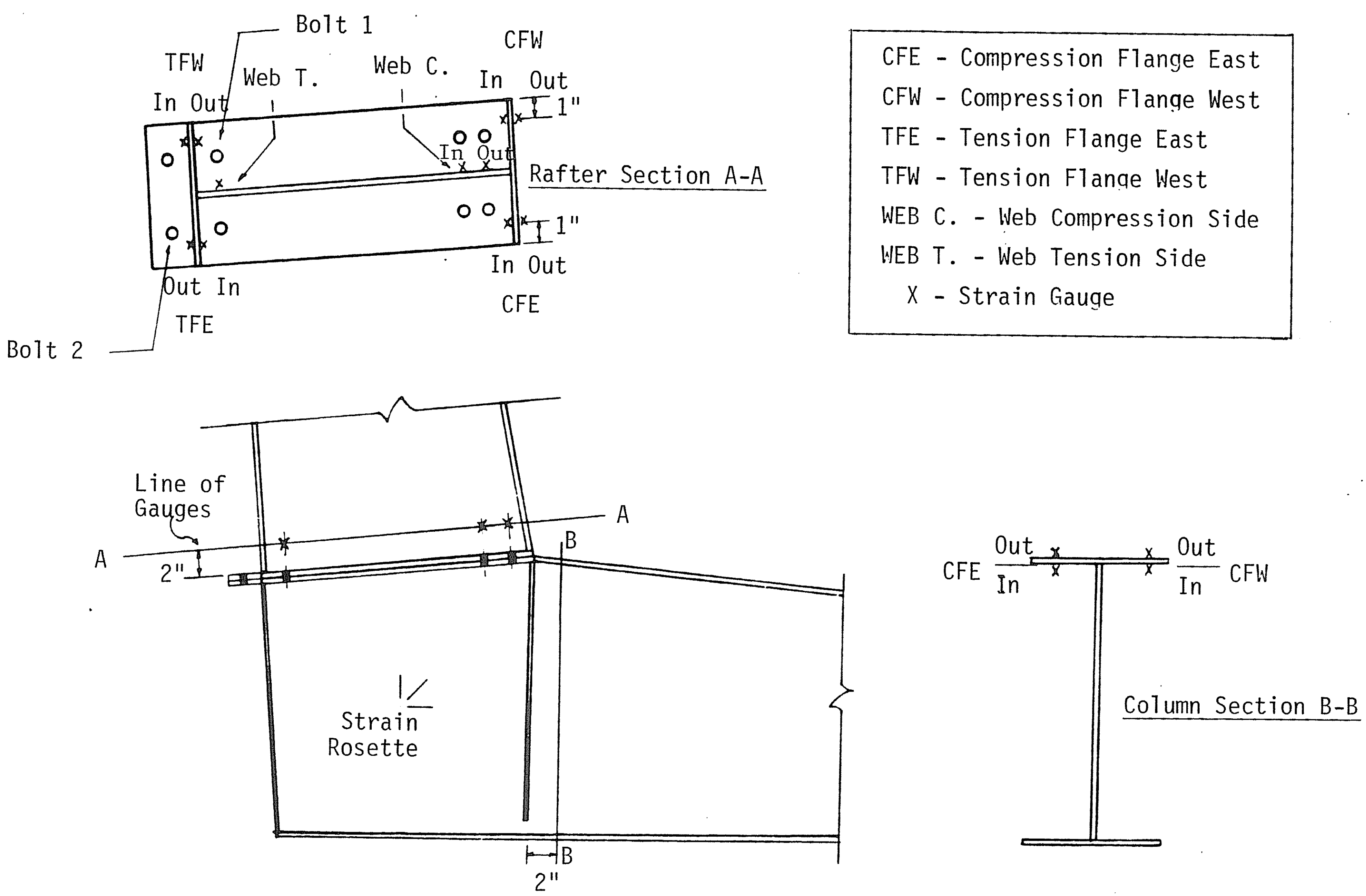


Figure 2.5 Typical Strain Gauge Locations

into the unthreaded shank so that bolt forces could be determined. The instrumented bolts were calibrated prior to installation in the specimens using a universal testing machine.

Strain gauges were mounted on the rafter web and flanges at a section two inches from the end-plate (Figure 2.5). Strain gauges were also mounted on the column compression flange two inches from the end-plate (Figure 2.5). Finally, strain gauges were mounted on the column compression flange, two inches from the reentrant corner. For Tests FA-3 through FA-6, a strain rosette was mounted at the center of the panel zone so that the principal strains and their orientation could be determined.

All instrumentation, except the transit and scales, was connected to a Hewlett-Packard Data Acquisition System which was used to monitor, record and plot test data in real time. Lateral displacement scale readings were taken using a surveyor's transit.

2.5 Testing Procedure

After the lateral braces and loading mechanism were attached to the specimen and the instrumentation was connected to the data acquisition system, testing was started. A load equal to approximately 30% of the predicted specimen failure load was first applied to the specimen. All instrumentation was checked and the specimen unloaded.

After unloading of the specimen, initial readings were taken and the specimen was incrementally loaded until failure. At each load increment, scale readings were recorded manually and all other data were recorded by the data acquisition system. Load versus chord displacement and load versus bolt force were plotted as the test progressed. Notes and photographs were taken as the specimen began to yield.

Failure of the specimen was considered to have occurred when further increase in load was not possible. After failure, the specimen was unloaded and final readings were taken.

CHAPTER III

TEST RESULTS

3.1 General

Eight tests, FA-1 to FA-8, were conducted. Tables 3.1 and 3.2 list the test parameters and Table 3.3 summarizes pertinent test results. The following sections describe each test with supporting data found in the Appendices A thru H for Tests FA-1 thru FA-8, respectively. Each appendix includes (1) a test summary sheet; (2) schematic drawings showing specimen details, the test setup, lateral brace locations, lateral displacement scale locations, and strain gage locations; (3) plots showing load versus predicted and experimental chord displacement, bolt force versus load, load versus lateral deflections, load versus rafter flange and web stresses; (4) plot of load versus lateral web deflection, and (5) plots of stress variation across the rafter.

The observed failure modes were associated with either a) local flange buckling, b) web plate yielding, c) panel zone yielding, d) panel zone buckling without tension field action, or e) panel zone buckling with tension field action. The failure load corresponding to each failure mode was calculated using the following criteria and an assumed yield stress of 55 ksi.

Table 3.1

Specimen Panel Zone Parameters

Test No.	Panel Zone Dimensions			Slenderness Ratios		Panel Zone Stiffener
	h(in.)	a(in.)	t(in.)	h/t	a/t	
FA-1	9.8	6.55	0.161	60.9	40.7	Partial Length
FA-2	12.8	9.90	0.156	82.1	63.5	Partial Length
FA-3	24.0	24.0	0.186	129.0	129.0	Partial Length
FA-4	31.9	32.0	0.218	146.3	146.8	Partial Length
FA-5	27.0	26.9	0.183	147.5	147.0	Partial Length
FA-6	48.0	48.0	0.328	146.3	146.3	Partial Length
FA-7	29.5	28.4	0.150	198.7	189.3	Full Length
FA-8	26.8	26.0	0.150	178.3	173.3	Full Length

Note: All dimensions are measured.

Table 3.2

Specimen End-Plate Connection Parameters

Test No.	End-Plate Configuration	Bolt Diameter (in.)	End-Plate Dimensions (in.)	Rafter Section	Column Section
FA-1	2 Bolt Flush	7/8	6 x 1/2	Tapered	Prismatic
FA-2	2 Bolt Flush	7/8	6 x 1/2	Tapered	Prismatic
FA-3	4 Bolt Flush	3/4	6 x 1/2	Tapered	Tapered
FA-4	4 Bolt Extended	3/4	6 x 1/2	Tapered	Tapered
FA-5	4 Bolt Flush	3/4	6 x 1/2	Tapered	Tapered
FA-6	6 Bolt Extended	3/4	8 x 1/2	Tapered	Tapered
FA-7	4 Bolt Flush	3/4	8 x 1/2	Tapered	Tapered
FA-8	4 Bolt Flush	3/4	8 x 1/2	Tapered	Tapered

Note: All dimensions are nominal.

Table 3.3

Summary of Test Results

Test No.	Panel Zone Plate Buckling (kips)	Panel Zone Tension Field Action (kips)	Exper. Failure Load (kips)	Predicted / Experimental x 100%		Failure Mode
				Plate Buckling	Tension Field Action	
FA-1	13.6	N.A.	13.0	105	N.A.	Panel Zone Yielding Local Flange Buckling
FA-2	22.0	N.A.	22.1	99	N.A.	Panel Zone Yielding
FA-3	20.8	35.7	19.8	106	180	Panel Zone Buckling
FA-4	23.8	50.0	32.1	74	156	Panel Zone Buckling
FA-5	17.0	36.3	29.4	58	123	Panel Zone Buckling
FA-6	66.8	141.3	63.8	104	221	Rafter Web Yielding
FA-7	7.6	26.5	26.4	21	100	Panel Zone Buckling
FA-8	10.0	29.0	29.3	34	99	Panel Zone Buckling

Note: All calculations based on an assumed yield stress of 55 ksi.

Local Flange Buckling Failure Criterion:

$$\frac{f_a}{Q_s F_y} + \frac{f_b}{Q_s f_y} = 1.0 \qquad (3.1)$$

where f_a, f_b = axial and flexural stresses, respectively, due to the applied load P, Q_s = the axial stress reduction factor from Formulas C2-3 and C2-4 of the 1978 AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, and F_y = yield stress of the material.

Web Plate Yielding Failure Criterion:

$$f_v = F_y/\ \ 3 \qquad (3.2)$$

where f_v = shear stress at cross-section due to the applied load P.

Panel Zone Yielding Failure Criterion:

$$f_v = F_y/\ \ 3 \qquad (3.3)$$

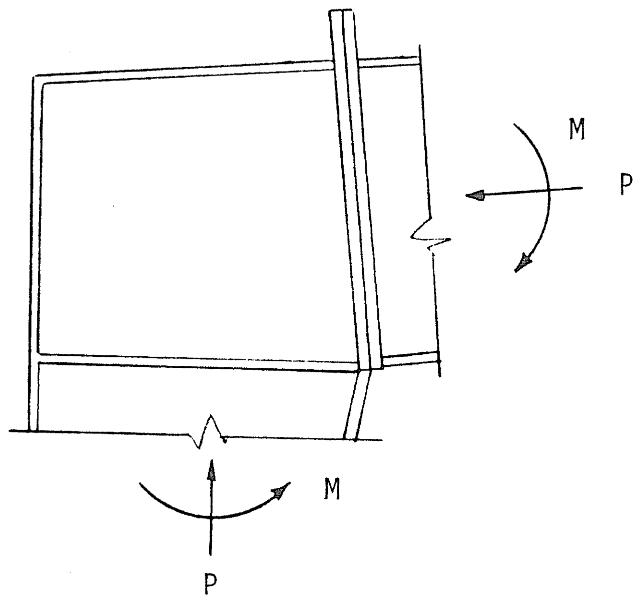
where

$$f_v = V/ht \qquad (3.4)$$

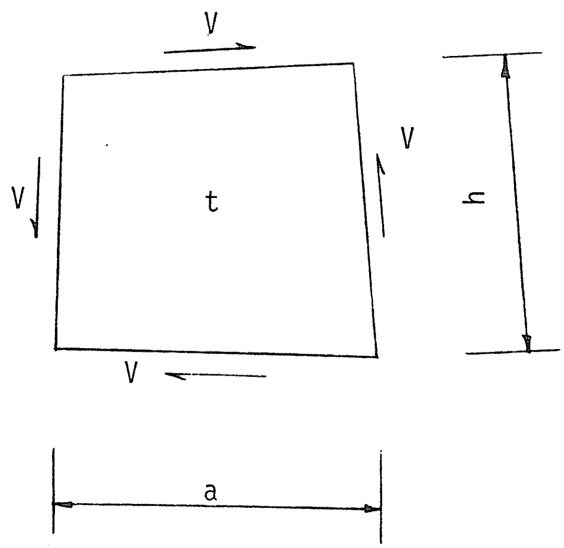
and

$$V = M/a - P/2$$

where M, P = moment and axial force in the rafter at the panel zone, a = width of the panel zone at the stiffener on the column and h = depth of panel zone at rafter (see Figure 3.1). Note that V is the shear force on the panel zone plate.



(a) Knee Area



(b) Panel Zone Plate

Figure 3.1 Panel Zone Geometry and Forces

Panel Zone Buckling without Tension Field Action Failure Criterion:

$$f_v = F_y C_v/\sqrt{3} \leq F_y/\sqrt{3} \qquad (3.5)$$

where

$C_v = \dfrac{45{,}000\ k}{F_y (h/t)^2}$ when C_v is less than 0.8

$C_v = \dfrac{190}{h/t}\sqrt{\dfrac{k}{F_y}}$ when C_v is more than 0.8

$k = 4.00 + 5.34/(a/h)^2$ when a/h is less than 1.0.

$k = 5.34 + 4.00/(a/h)^2$ when a/h is more than 1.0

and all other variables are as previously defined.

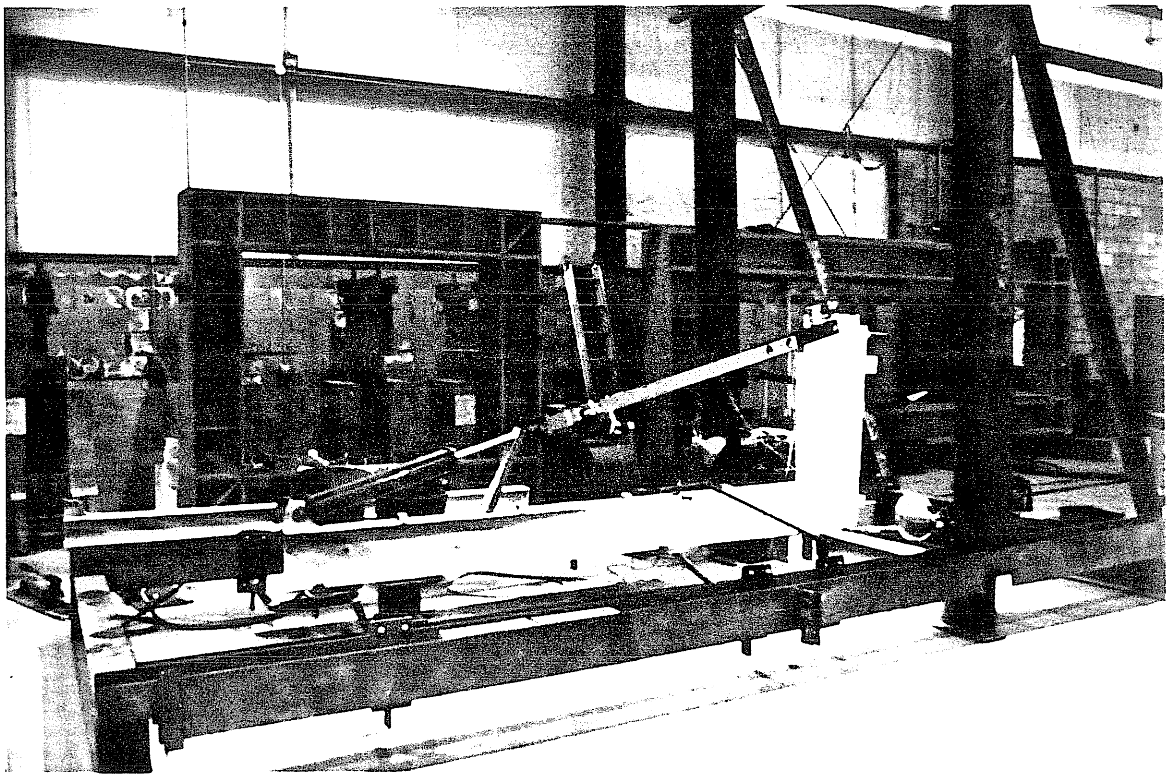
Panel Zone Buckling with Tension Field Action Failure Criterion:

$$f_v = \frac{F_y}{\sqrt{3}} \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \leq \frac{F_y}{\sqrt{3}} \qquad (3.6)$$

with all variables as previously defined.
A discussion of each test follows.

3.2 Test FA-1

Pertinent parameters for this test are: prismatic column section; tapered rafter section; two tension bolt (7/8 in. diameter), flush end-plate (6 in. by 1/2 in.) connection; and a nominal 10 in. by 6-1/2 in. by 0.190 panel zone (h/t = 60.9), stiffened with a partial length



a) Overview of Test Setup

b) Panel Zone Yielding

Figure 3.2 Photographs of Test FA-1

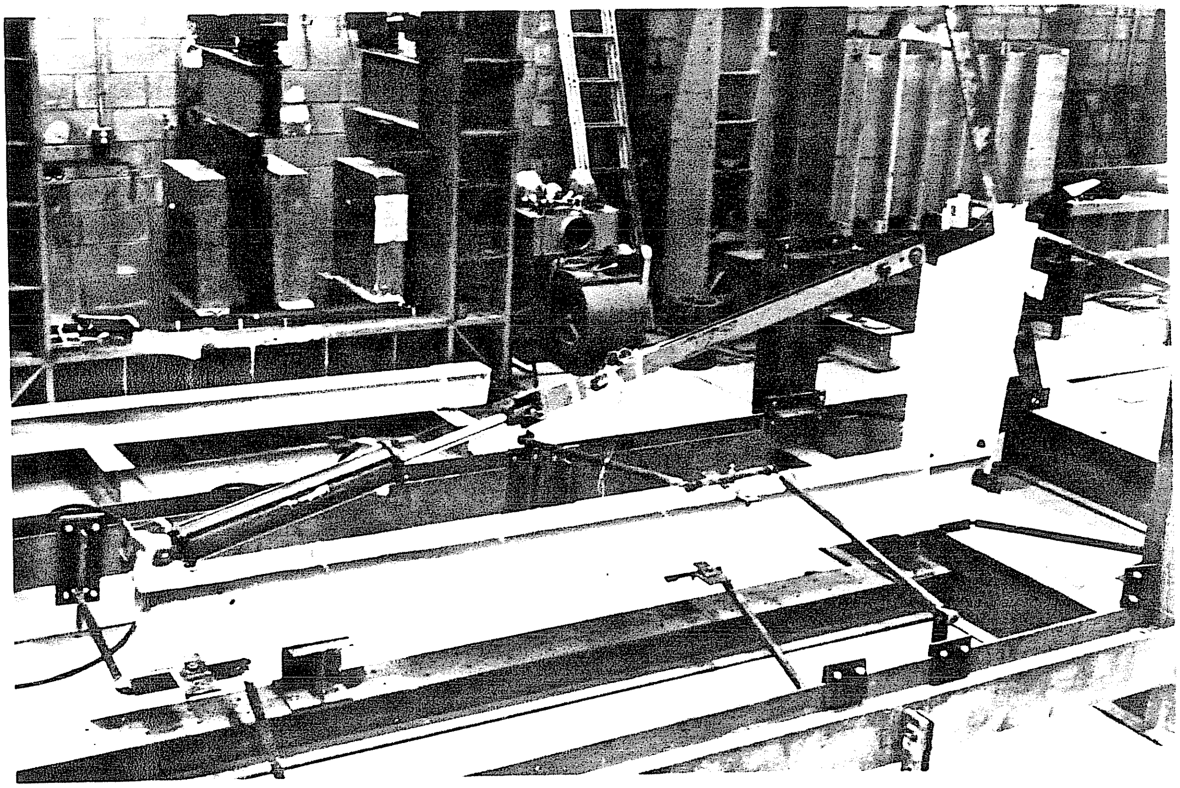
stiffener. The stiffener was welded to the inside (compression) column flange and web plate but did not extend to the column tension flange. Figure 3.2 is photographs of the test.

The failure of Specimen FA-1 was the result of local buckling in the column compression flange or panel zone yielding. Initial flange yielding was observed in the areas adjacent to the rafter and adjoining the column web splice at a test load of 5.5 kips. With additional load application, the entire length of flange between these areas yielded and buckling occurred at 13.0 kips. This failure load slightly exceeds the predicted local buckling failure load of 12.4 kips (Equation 3.1).

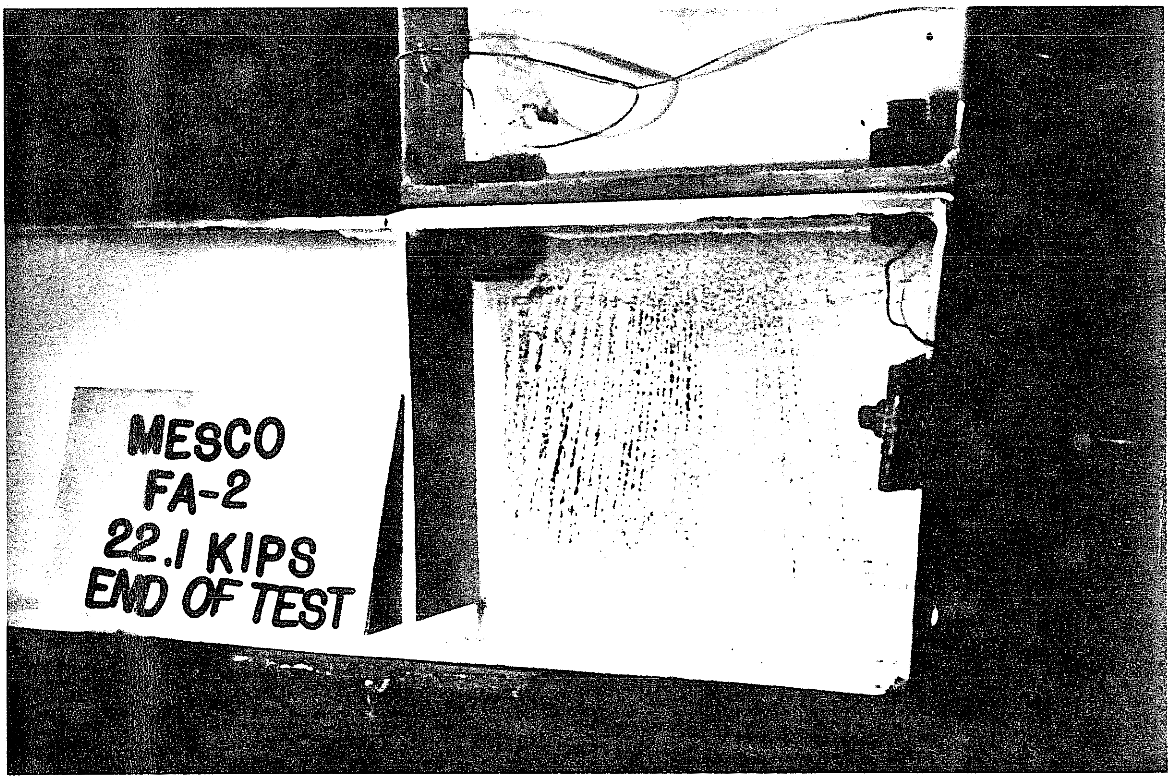
At the 13.0 kips failure load extensive yielding was present throughout the panel zone. Yielding in the panel zone was first observed at the test load of 7.6 kips and had extended to cover virtually the entire panel prior to the failure of the column flange. The predicted yield load for the panel zone (Equation 3.2) was 13.6 kips.

From Figure A.6, it is evident that the frame assembly was significantly more flexible than predicted assuming a rigid column-to-rafter connection. The flexibility is attributed to the two tension bolt, flush end-plate connection used to connect the rafter and column sections.

Bolt forces increased above the pretension level, Figure A.7, but did not exceed the tensile strength of the bolts. Lateral movement of the column compression flange was not significant, Figure A.8. Measured rafter flange



a) Overview of Test Setup



b) Panel Zone Yielding

Figure 3.3 Photographs of Test FA-2

and web stresses (as computed from strain gage data) varied approximately linearly with applied load until near the failure load level, Figures A.9 and A.10. Center of panel zone deflections did not exceed 0.005 in., Figure A.11. Stress variation across the rafter near the end-plate connection are shown in Figures A.12 and A.13.

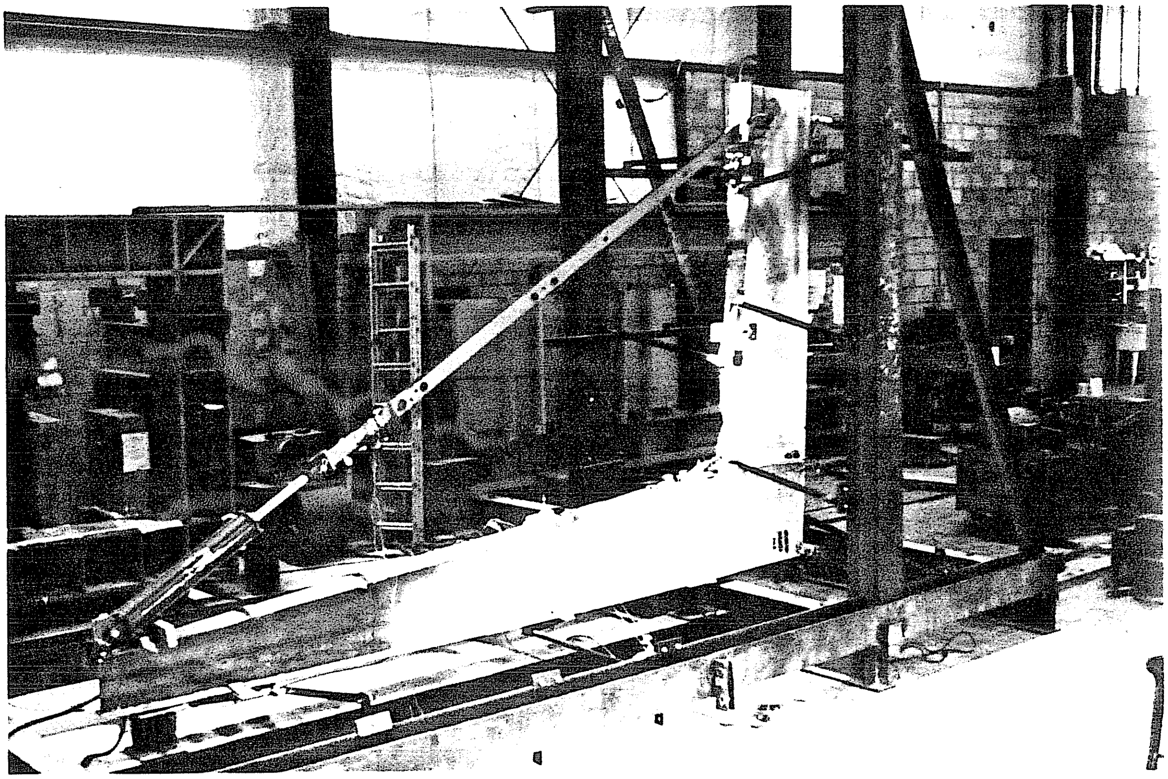
3.3 Test FA-2

Pertinent parameters for this test are: prismatic column section; tapered rafter section; two tension bolt (7/8 in. diameter), flush end-plate (6 in. by 1/2 in.) connection; and a nominal 13 in. by 10 in. by 0.156 in. panel zone (h/t = 82.1), stiffened with a partial stiffener detailed as described for specimen FA-1. Figure 3.3 is photographs of the test.

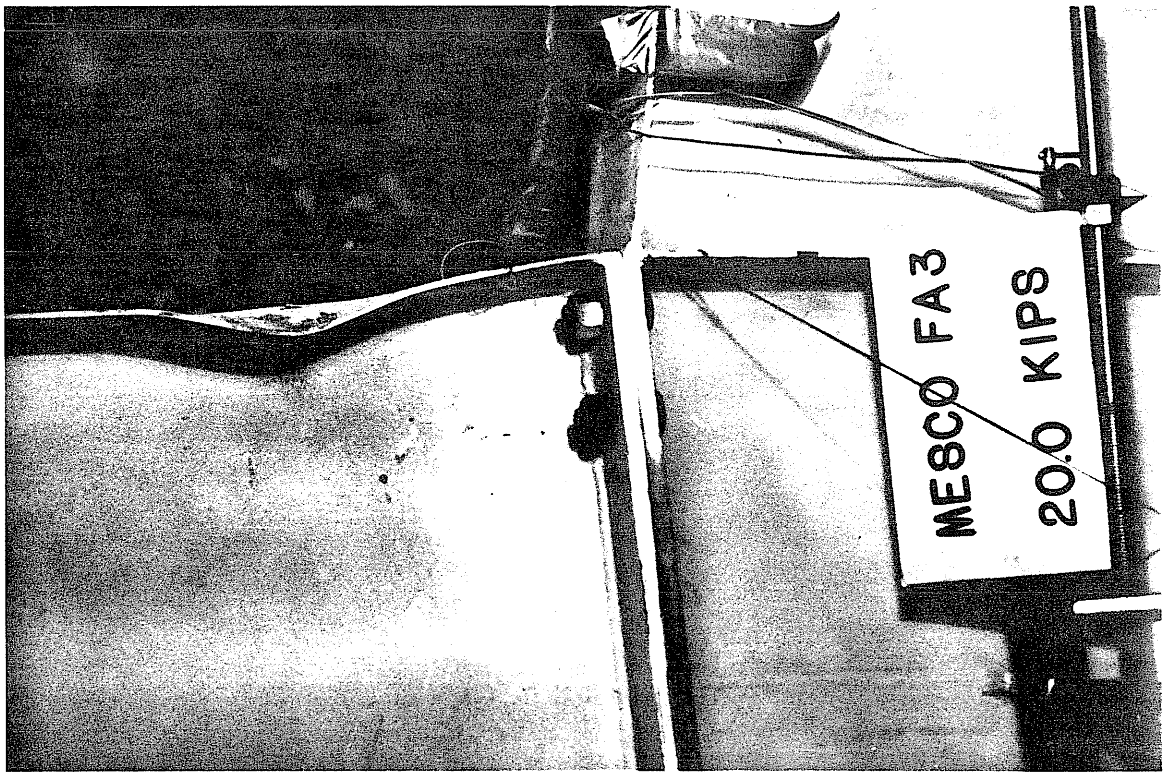
The maximum applied load was 22.1 kips and the failure mode for the specimen was yielding of the panel zone. The predicted failure load for this failure mode is 22.0 kips from Equation 3.2.

The assembly was considerably more flexible than predicted as shown in Figure B.6. The flexibility is attributed to the two tension bolt flush end-plate connection. Bolt forces increased above the pretension level in proportion to the applied load as shown in Figure B.7. Lateral movement of the rafter compression flange was insignificant, Figure B.8.

Figure B.9 shows the out-of-plane movement of the center of the panel zone. A reversal in the direction of panel zone movement occurred near the failure load,



a) Overview of Test Setup



b) Local Rafter Flange Buckling

Figure 3.4 Photographs of Test FA-3

however, deflections did not exceed 0.05 in.

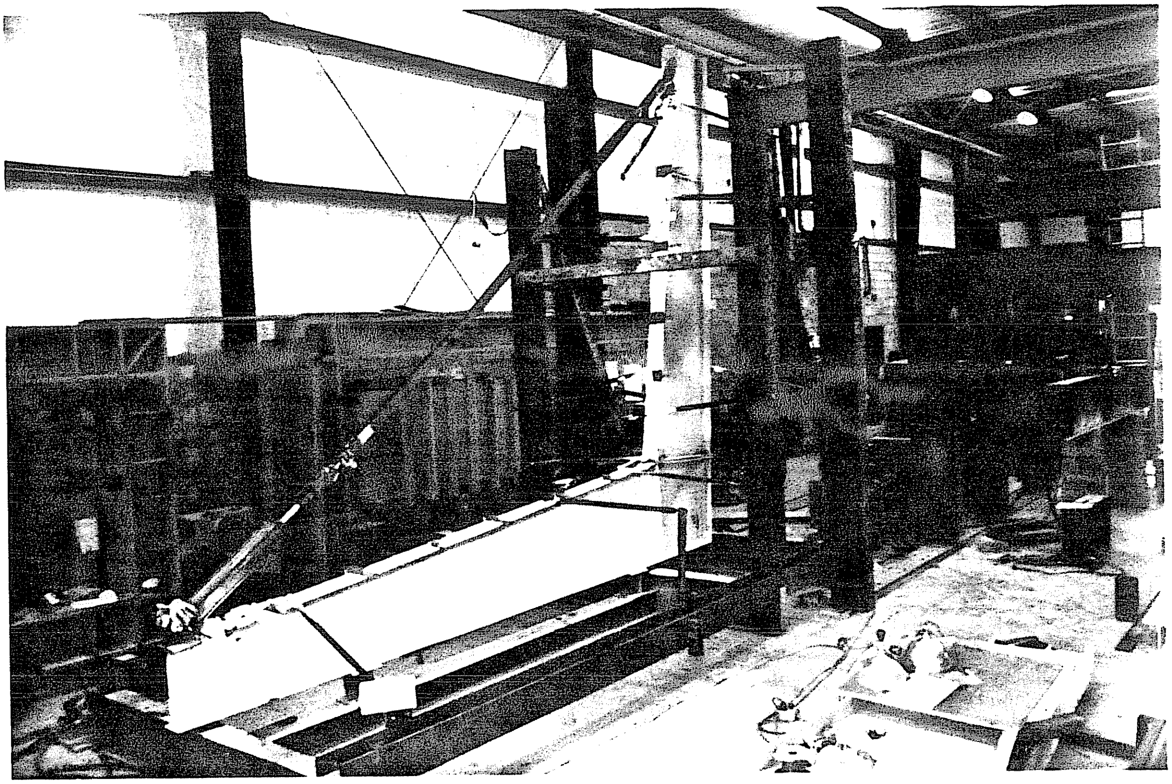
Rafter flange and web stresses varied linearly with applied load until near the failure load, Figures B.10 and B.11. Stress distributions across the rafter at a section near the end-plate are shown in Figures B.12 and B.13.

3.4 Test FA-3

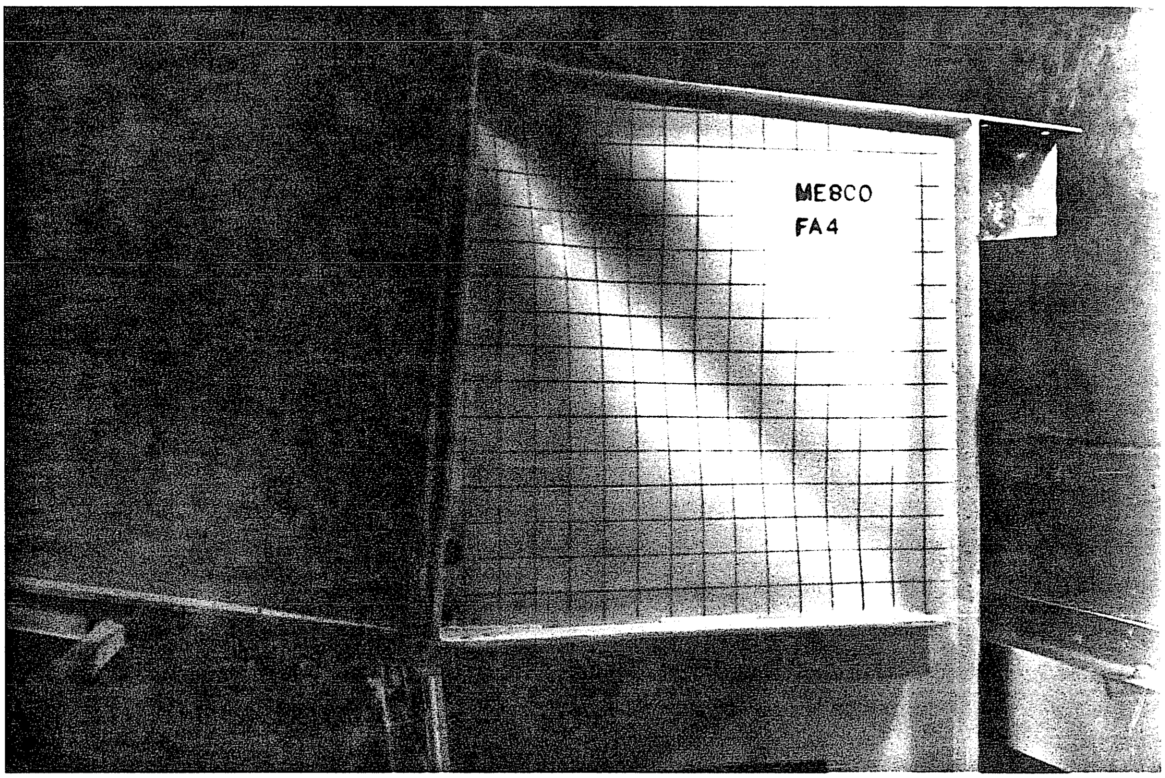
Pertinent parameters for this test are: tapered column and rafter sections; four tension bolt (3/4 in. diameter), flush end-plate (6 in. by 1/2 in.) connection; and a nominal 24 in. by 24 in. by 0.186 in. panel zone (h/t = 129.0) with a partial stiffener. Photographs of the test are shown in Figure 3.4.

The maximum applied load in this test was 19.8 kips. Failure occurred due to either local torsional plate buckling of the rafter compression flange or panel zone plate buckling. The local flange buckling is shown in Figure 3.3(b). However, Figure C.10 clearly shows significant out-of-plane movement of the center of the panel zone plate. The predicted failure load based on local flange buckling is 26.8 kips and on panel zone plate buckling is 20.8 kips. Thus, panel zone plate buckling is the recorded failure mode.

The experimental and predicted chord displacement curves, Figure C.6, are in good agreement reflecting the increased stiffness of the four tension bolt flush end-plate connection. The bolt forces remained at the pretension level as shown in Figure C.7.



a) Overview of Test Setup



b) Partial Panel Zone Tension Field

Figure 3.5 Photographs of Test FA-4

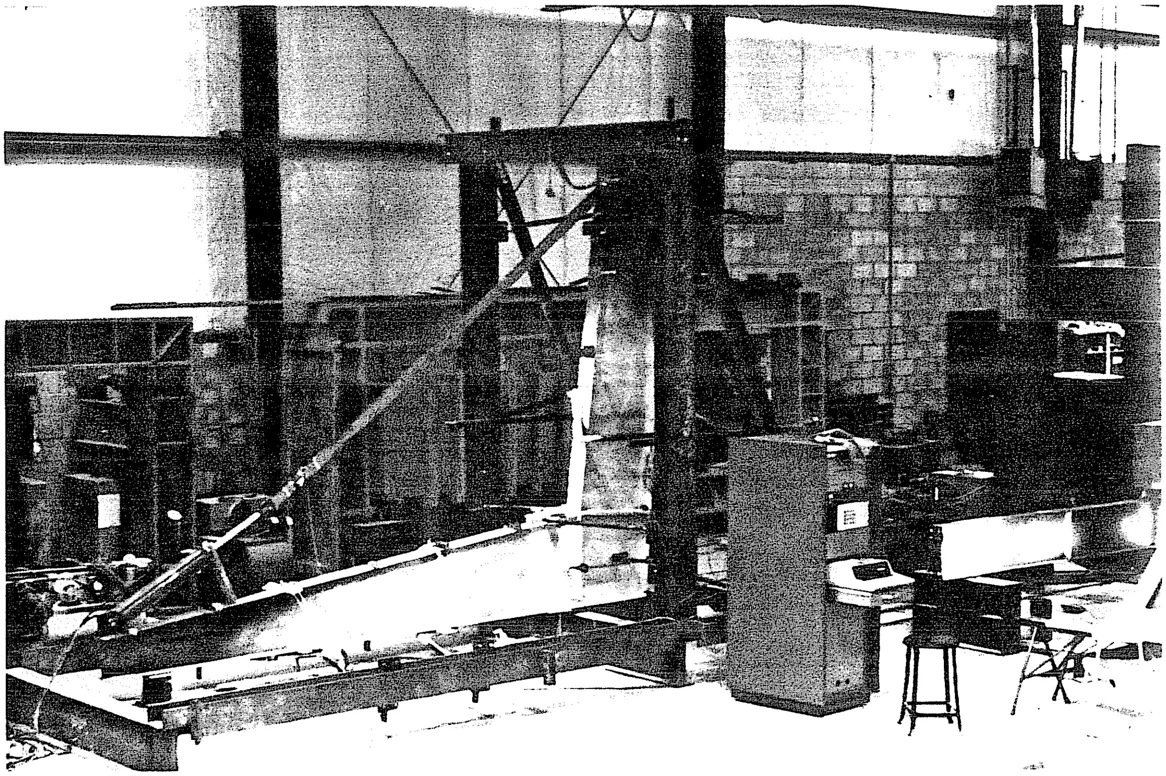
Lateral displacements of the column and rafter flanges were insignificant, Figure C.8. As previously noted, the out-of-plane movement of the center of the panel zone, Figure C.10, clearly indicates plate buckling. Deflections slightly exceeded 0.015 in.

Measured rafter flange, column flange and rafter web stresses varied in a somewhat linear manner with applied load, Figures C.11, C.12 and C.13. The variation of principal stresses with applied load is shown in Figure C.14 with supporting data found in Table C.1. Variation of stresses across the rafter at a section near the end-plate connection are shown in Figures C.15 and C.16.

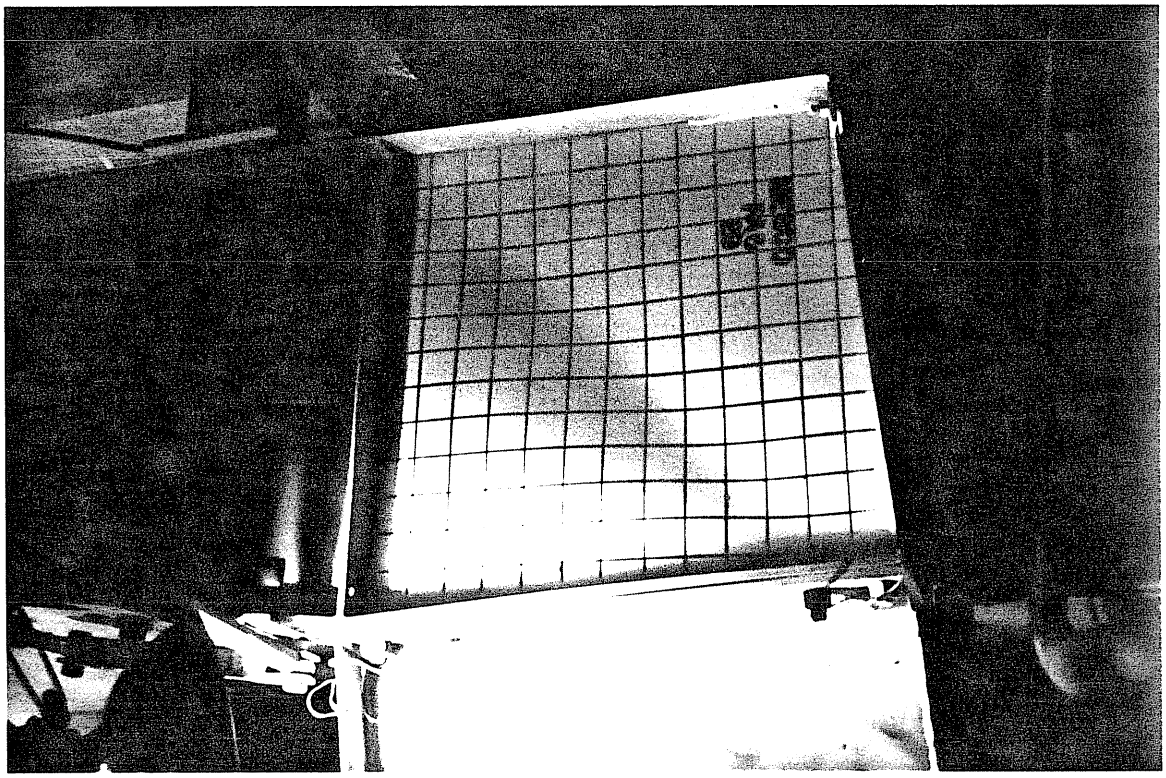
3.5 Test FA-4

Pertinent parameters for this test are: tapered rafter and column sections; four tension bolt (3/4 in. diameter), extended end-plate (6 in. x 1/2 in.) connection; and nominal 32 in. by 32 in. by 0.218 in. panel zone (h/t = 146.3), stiffened with a partial length stiffener. Photographs of the test are found in Figure 3.5.

Failure of this specimen occurred at 32.1 kips due to panel zone plate buckling. The calculated failure load using Equation 3.5 (plate buckling without tension field action) was 23.8 kips. If tension field action is considered, Equation 3.6, the predicted failure load is 50.0 kips. Examination of the out-of-plane movement of the center of the panel zone, Figure D.10, shows that plate buckling occurred at approximately 28 kips of applied load and that the panel zone continued to deflect out-of-plane with increasing applied load. Figure 3.4(b) shows that a



a) Overview of Test Setup



b) Partial Panel Zone Tension Field

Figure 3.6 Photographs of Test FA-5

tension field did not form diagonally across the panel zone due to the partial length stiffener, which is the reason for the failure load to be less than that predicted assuming full tension field action.

Excellent agreement exists between the measured and predicted chord displacements prior to the load at which the panel zone plate buckled, Figure D.6. Bolt forces remained at the pretension load level throughout the test as shown in Figure D.7.

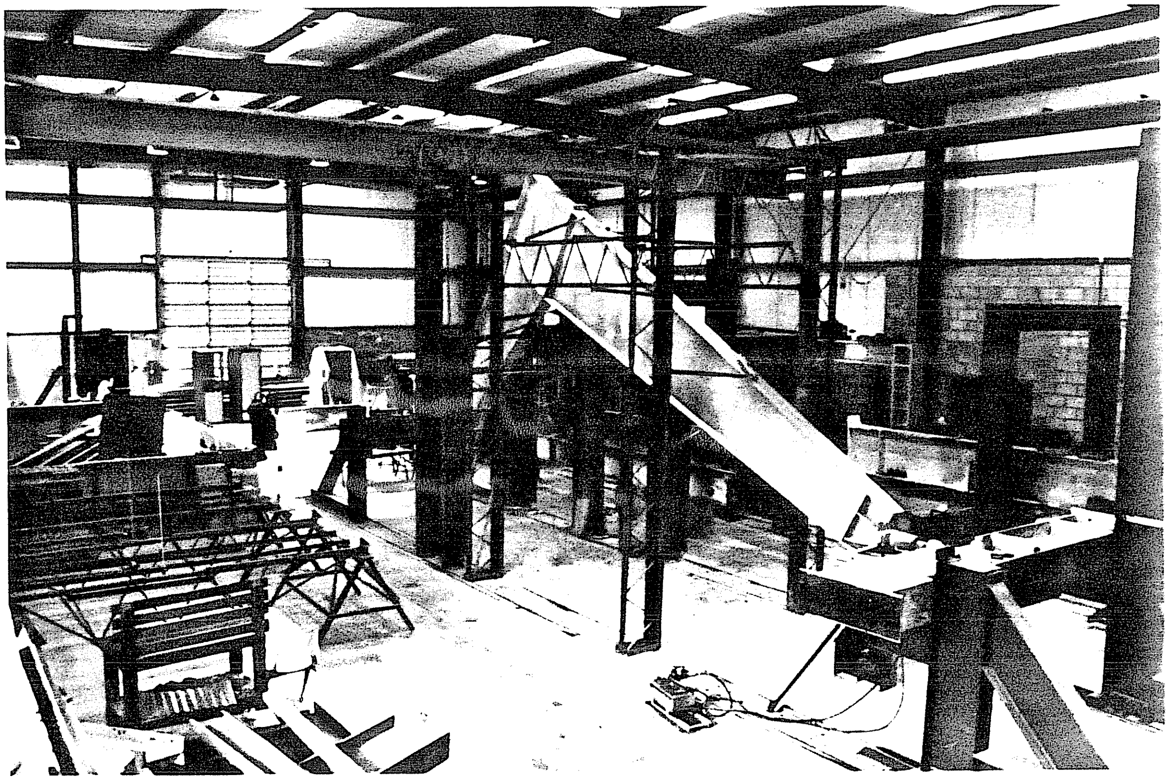
Lateral movement of the rafter and column compression flanges did not exceed 0.2 in. prior to failure of the specimen, Figures D.8 and D.9.

Results of the various rafter flange, column flange and column web strain measurements converted to stresses are shown in Figures D.11 thru D.18 and in Table D.1. In general, the measured stresses varied linearly with applied load until the panel zone plate buckled.

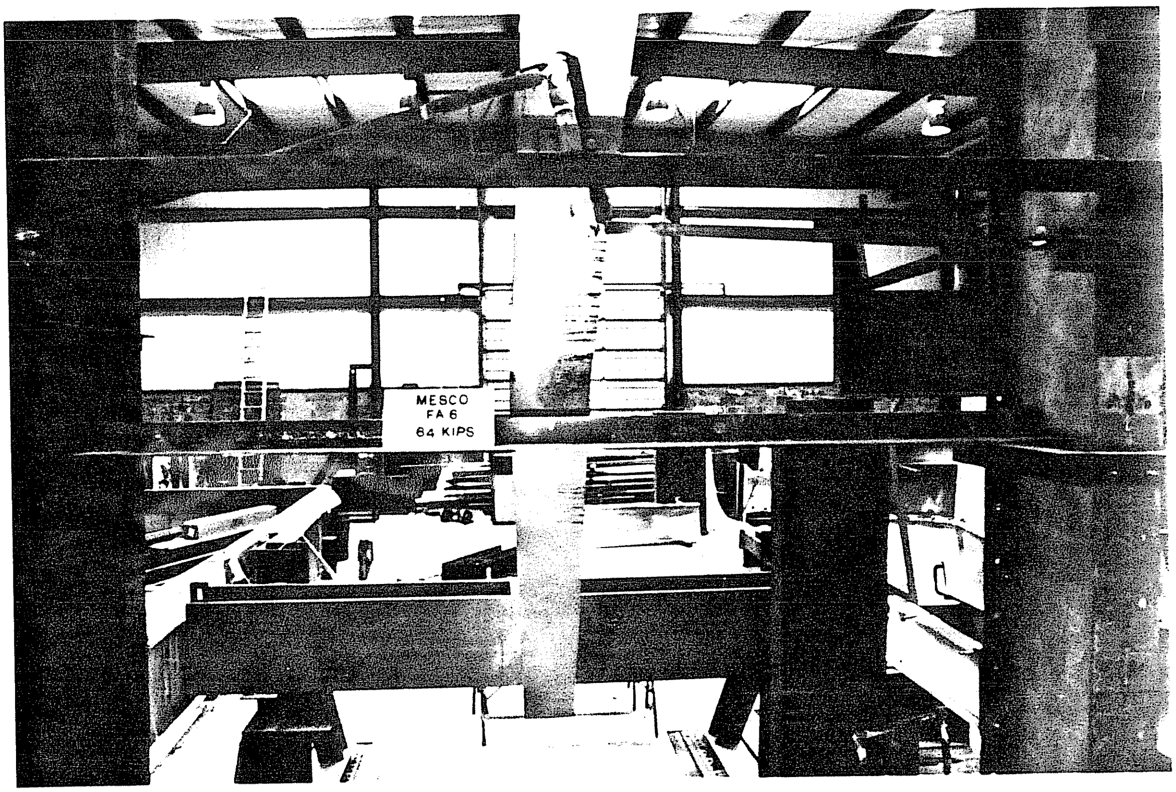
3.6 Test FA-5

Pertinent parameters for this test are: tapered column and rafter sections; four tension (3/4 in. diameter) bolt flush end plate (6 in. by 1/2 in.) connection; and a nominal 26 1/2 in. by 27 in. by 0.183 in. panel zone (h/t = 147.5). A partial length stiffener was used. Photographs of the test are found in Figure 3.6.

The failure load for the specimen was 29.4 kips and the failure mode was panel zone buckling. The predicted plate buckling load (no tension field action) from Equation



a) Overview of Test Setup



b) Yielding of Column Compression Flange

Figure 3.7 Photogtaphs of Test FA-6

3.5 is 17.0 kips. When tension field action is included, the predicted failure load increases to 36.3 kips. The incomplete development of the tension field as shown in Figure 3.5(b) due to the partial length panel zone stiffener accounts for the differences between the predicted and experimental failure loads.

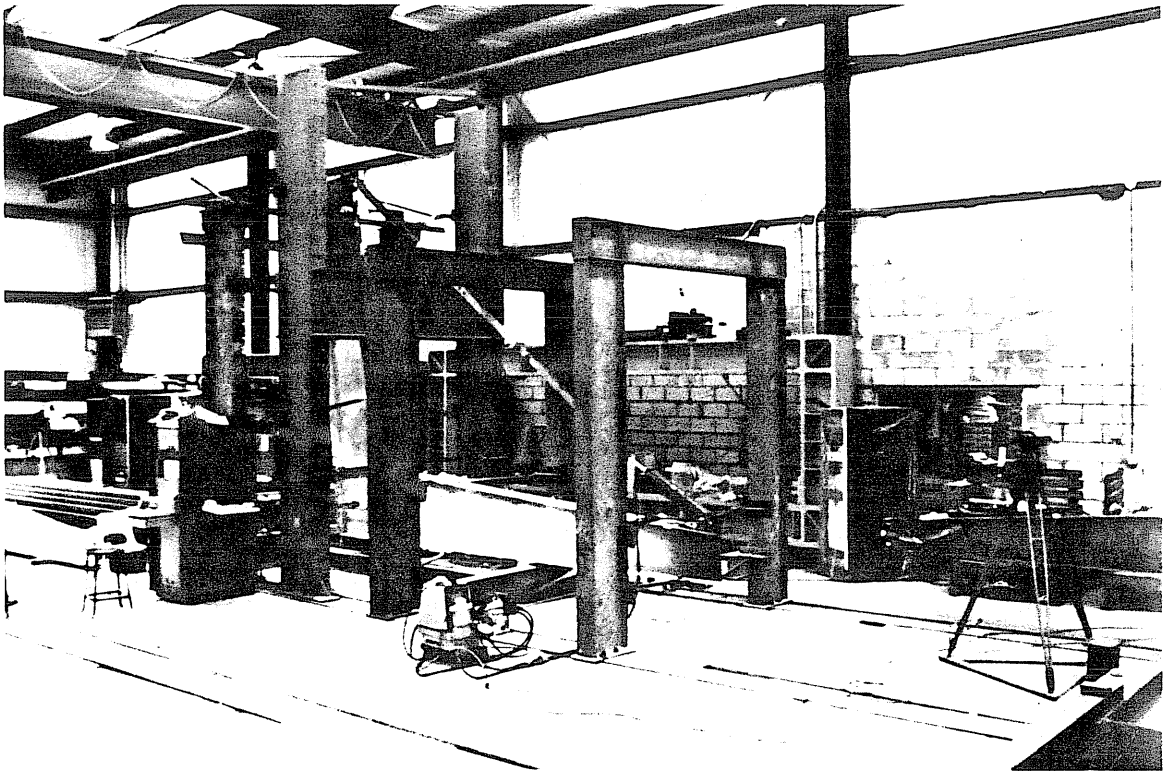
As shown in Figure E.6, the assembly was considerably more flexible than predicted assuming a perfectly rigid connection between the column and rafter sections. However, measured bolt forces did not increase above the pretension level, Figure E.7.

Lateral displacements of the column and rafter section compression flanges become significant only after the panel zone plate buckled, Figures E.8 and E.9.

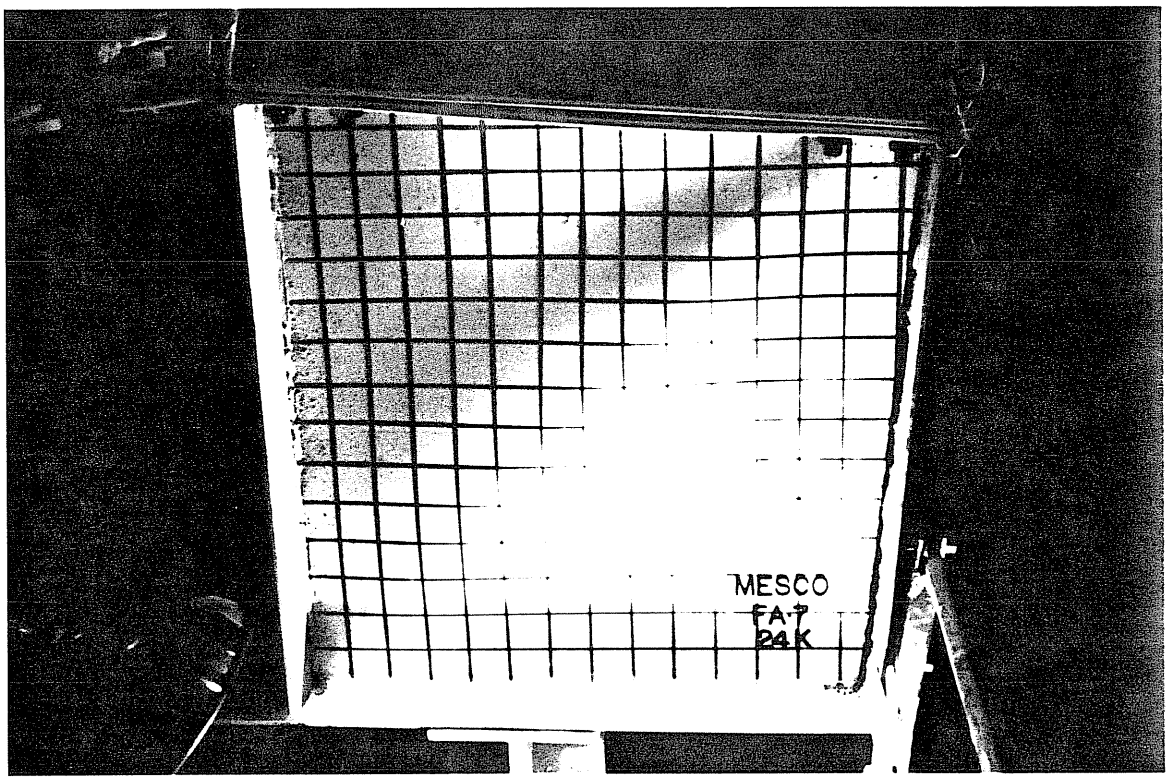
Figure E.10 shows the variation of out-of-plane panel zone plate movement with applied load. The movement was negligible prior to the 20 kips level. Above 20 kips, the lateral movement increased at a very high rate indicating plate buckling; the maximum displacement exceeded 0.06 in.

Load versus rafter flange, rafter web, column flange and panel zone principal stresses are shown in Figures E.12, E.13 and E.14. Data needed to compute the principal stresses is found in Table C.1. All relationships are nonlinear, possibly reflecting the out-of-plane movement of the compression flanges.

Stress distributions across the rafter near the end-plate are shown in Figures E.15 and E.16.



a) Overview of Test Setup



b) Panel Zone Tension Field

Figure 3.8 Photographs of Test FA-7

3.7 Test FA-6

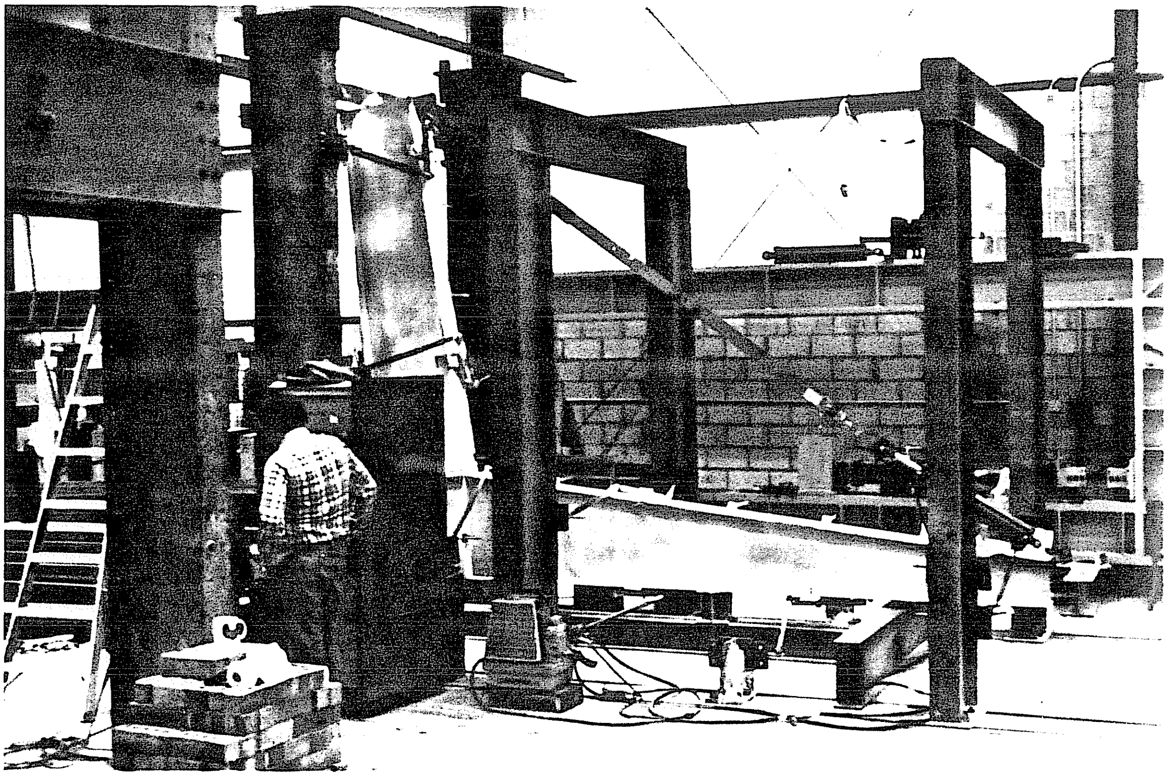
Pertinent parameters for this test are: tapered rafters and column sections; six bolt (3/4 in. diameter) extended end-plate (8 in. by 1/2 in.); and a nominal 48 in. by 48 in. by 5/16 in. panel zone (h/t = 146.3). A partial length stiffener was used. Photographs of the test are found in Figure 3.7.

The maximum applied load was 63.8 kips. Failure was by local buckling of the rafter compression flange or shear yielding of the rafter web plate adjacent to the end-plate connection. Local buckling of the rafter compression flange was evident, and some flaking of the whitewash did occur on the rafter web. The predicted failure load based on local buckling (Equation 3.1) was 102.8 kips and for shear yielding (Equation 3.2) 63.9 kips. It is assumed that shear yielding of the rafter occurred first causing local buckling of the rafter compression flange.

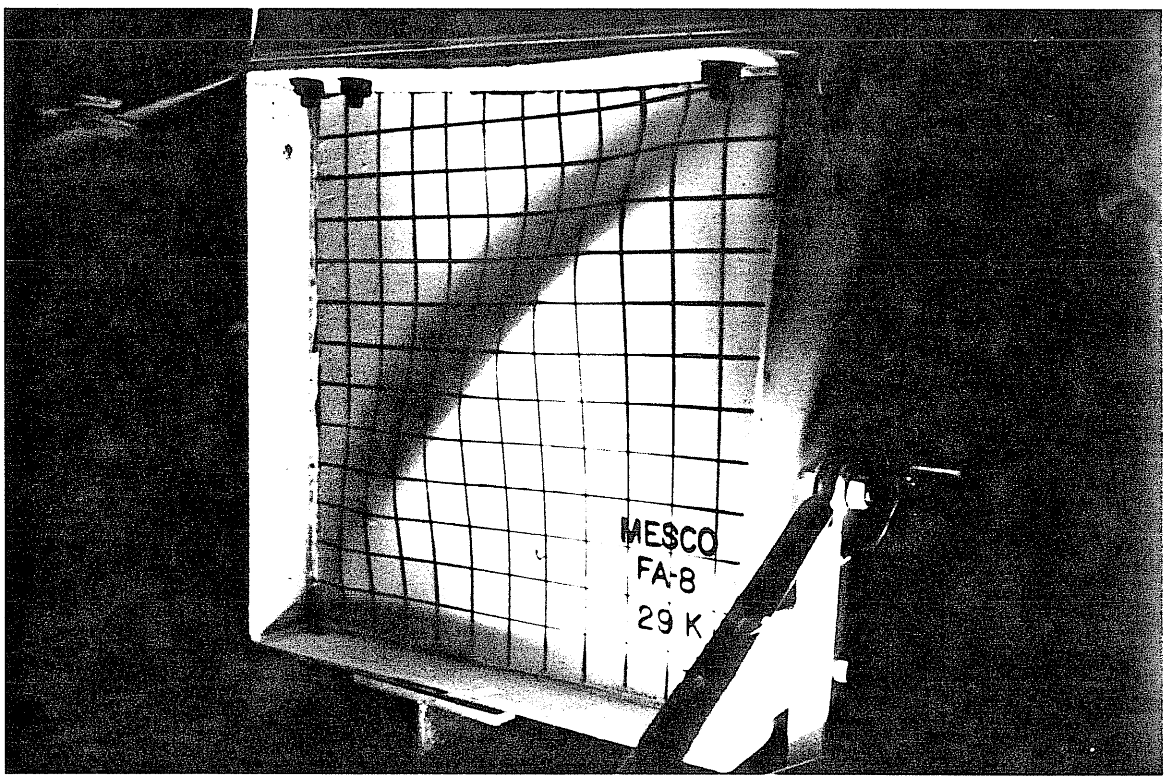
The applied load versus chord displacement curve shown in Figure F.5 is somewhat erratic probably due to the method of load application. Agreement with the predicted curve exists only below the 30 kips applied load level. Measured bolt forces increased above the pretension level when the applied load reached 30 kips.

Figure F.7 shows the panel zone web plate out-of-plane movement. Only very small displacements were recorded prior to failure of the specimen.

Measured stresses versus applied load are shown in Figures F.8 to F.13. Table F.1 contains principal stress



a) Overview of Test Setup



b) Panel Zone Tension Field

Figure 3.9 Photographs of Test FA-8

data. In general, the curves are erratic and nonlinear. Rafter stresses in some instances reached the assumed yield stress level (55 ksi). Distribution of stresses across the rafter section adjacent to the end-plate are shown in Figures F.14 and F.15.

3.8 Test FA-7

Pertinent parameters for this test were: tapered rafter and column sections; four tension bolt (3/4 in. diameter) flush end-plate (8 in. by 1/2 in.) connection; and a nominal 28 1/2 in. by 29 1/2 in. by 0.15 in. panel zone. Full depth stiffeners welded to both flanges and the web plate were used. Figure 3.8 is photographs of the test.

Failure occurred at 26.6 kips due to panel zone plate buckling. The corresponding predicted failure load considering only panel zone plate buckling is 7.7 kips, with tension field action considered, the predicted failure load is 26.4 kips.

Figure G.6 shows the predicted and experimental load versus chord displacement curves. The initial softening of the experimental curve is attributed to flexibility of the end-plate connection and the final portion to panel zone web buckling. Bolt forces remained near the pretension level until the applied load reached approximately 20 kips at which point a rapid increase in bolt force occurred.

Rafter flange and web stress versus applied load are shown in Figures G.8 and G.9. The curves are generally linear to the 20 kips applied load level and then become erratic.

Load versus out-of-plane panel zone plate displacements are shown in Figure G.10. The curve clearly reflects buckling of the panel zone. Deflections exceeded 0.12 in. at the failure load.

Principal stresses at the panel zone center versus load are shown in G.11 and G.12 with supporting data in Table G.1.

3.9 Test FA-8

Pertinent parameters for this test are: tapered rafter and column sections; four tension bolt (3/4 in. diameter), flush end-plate (8 in. by 1/2 in.) connection and nominal 27 in. by 30 in. by 0.150 in. panel zone (h/t = 178.3). Full depth, fully welded stiffeners were used. Photographs of the test are found in Figure 3.9.

Failure occurred at 29.3 kips due to buckling of the panel zone plate. The predicted plate buckling load without considering tension field action is 10.0 kips. With tension field action, the predicted failure load is 29.0 kips.

The experimental applied load versus chord displacement curve is shown in Figure H.6. Initial softening in the curve (applied load less than 20 kips) is attributed to connection flexibility and the final softening to panel zone buckling.

Bolt forces remained at the pretension level to the approximately 20 kips applied load level at which point the force in one bolt increased at a rapid rate, Figure H.7.

Rafter flange and web stresses generally varied linearly with applied load to the 20 kips load level and then varied erratically, Figures H.8 and H.9.

Panel zone plate buckling is clearly shown in the load versus lateral plate deflection plot in Figure H.10. Deflections exceeded 0.10 in.

Principal center of panel zone web plate stresses are shown as a function of load in Figures H.11 and H.12. Supporting data is found in Table H.1.

CHAPTER IV

SUMMARY, OBSERVATIONS AND CONCLUSIONS

4.1 Summary

Tests of eight single story, single bay frame assemblies, consisting of portions of rafter and column sections and the connecting elements are reported here. Configuration of the test specimens was such that upon application of a single test load, forces and moments in the knee area of the test assembly closely approximated the forces and moments due to gravity loads in the corresponding rigid frames.

End-plate moment connections were used to connect the rafter and column sections. Both two and four tension bolt flush and four and six bolt extended end-plate configurations were used. Bolts were either 3/4 in. or 7/8 in. diameter, ASTM A325.

Nominal panel zone dimensions varied from 6 1/2 in. by 10 in. by 0.161 in. to 48 in. by 48 in. by 3/8 in. Maximum panel zone plate slenderness ratios varied from 60.9 to 198.7. Both partial depth and full depth stiffeners were used in the column section opposite the rafter compression flange. All plate material had a nominal yield stress of 50 ksi.

The failure mode for all specimens, except FA-6, was directly related to panel zone plate yielding or buckling. The failure of specimen FA-6 was due to yielding of the rafter web plate.

Predicted failure loads for panel zone yielding were calculated using the plate girder web provisions of the AISC Specification with the factor of safety (1.67) removed and an assumed yield stress of 55 ksi.

4.2 Observations

The following observations resulted from the testing:

1. Use of standard elastic stiffness analyses to predict frame assembly in-plane deflections assuming full connection rigidity, significantly over-estimates the stiffness of the assembly if two tension bolt, flush end-plate connections are used (see Figures A.6 and B.6). Use of four tension bolt, flush end-plate connections significantly improves the frame assembly stiffness (see Figures C.6, E.6, G.6 and H.6). Results from both tests conducted using extended end-plates show good agreement between predicted and measured frame assembly chord displacements (see Figures D.6 and F.6).

2. Bolt forces were found to increase a substantial amount above the pretension load in two tension bolt, flush end-plate connections (see Figures A.7 and B.7). Bolt forces remained near the pretension level in four tension bolt flush connections if tension field action did not develop in the panel zone (see Figures C.7 and E.7), however, a rapid increase in bolt forces accompanied the

development of the tension field in Tests FA-7 and FA-8 (see Figures G.7 and H.7). Bolt forces remained at the pretension level in the four tension, extended end-plate connection used in Test FA-4 (see Figure D.7). Bolt forces increased above the pretension level in two of the instrumented bolts in the six tension bolt, extended end-plate connection used in Test FA-6 (see Figure F.6).

3. AISC plate girder web yielding, plate buckling and tension field strength equations can be used to accurately predict the strength of panel zone web plates if the implied factor of safety of 1.67 is removed and h is the depth of the web plate at the rafter. The resulting relationships are Equations 3.2, 3.3, 3.5 and 3.6 of this report.

The failure mode for Tests FA-1 and FA-2 was panel zone plate yielding (although the possibility exists the local rafter flange buckling also occurred in Test FA-1). The corresponding predicted failure loads from Equation 3.3 were 105% and 99% of the experimental failure loads (see Table 3.3).

The failure mode for Tests FA-7 and FA-8 was ultimate strength of the tension field. The predicted failure loads from Equation 3.6 for these tests were 100% and 99% of the experimental loads (see Table 3.3). Full depth column web stiffeners opposite the inside rafter flange were used for both specimens.

In Tests FA-3, FA-4, and FA-5, incomplete tension fields developed in the panel zone web plate. Partial depth column web stiffeners were used for each specimen.

The predicted failure loads from Equation 3.5 varied from 58% to 106% of the experimental failure loads when only plate buckling was considered and from 123% to 180% when tension field action was used, Equation 3.6 (see Table 3.3). Thus, it is evident that full tension field action cannot be developed with partial depth stiffeners.

4.3 Conclusions

The major conclusions from this study are:

1. Inadequate stiffness may result if flush end-plate connections are used to connect rafter and column sections for frames designed utilizing AISC Type 1 construction.

2. Bolt force magnitudes in end-plate connections between rafter and column section depend on the end-plate configuration and on the state of the panel zone tension field if it exists.

3. AISC plate girder web yielding, plate buckling and tension field action provisions can be used to design panel zone web plates if h is defined as the depth of the plate at the rafter.

4. Panel zone tension field action strength should only be considered if a full depth column web stiffener opposite the inside rafter flange is used. This stiffener must be welded to both column flanges and the column web.

APPENDIX A

FA-1 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: MESCO Knee Test

Test No.: FA-1

Test Date: April 9, 1984

Purpose: Test of Knee Area

Number of Tension Bolts: 2 Bolt Gage (g): 3" Pitch: 1.5"

Bolt Diameter: 0.875 (7/8") End Plate Thickness (t): 0.510"

End Plate Width (w): 6.0" End Plate Length (de): 10.3"

Panel Zone Web Plate Thickness: .161"

Initial Out-of-Straightness at the Center of Panel Zone: 0.015" (1/64")

Pretension Force per Bolt: 39 kips

Failure Load, (Total Load): 13 kips

Failure Mode: Local buckling of column flange and/or panel zone yielding

Predicted Failure Loads:

Method: Local buckling Total Load: 12.36 kips

Method: Shear in panel zone Total Load: 13.58 kips

Discussion:

-At load equal to 5.5^k some yielding occurred in column flange at the corner of knee and at the cross section where column web plate is spliced.

-At load equal to 7.55^k shear yield lines appeared on white wash in panel zone.

-Loading beyond 11^k caused severe yield lines to appear on column inner flange as well as in panel zone.

-At load level 12.5^k significant local buckling of column flange could be observed. Beyond 12.5^k local buckling of column flange became more severe.

-The maximum capacity of this specimen was 13^k.

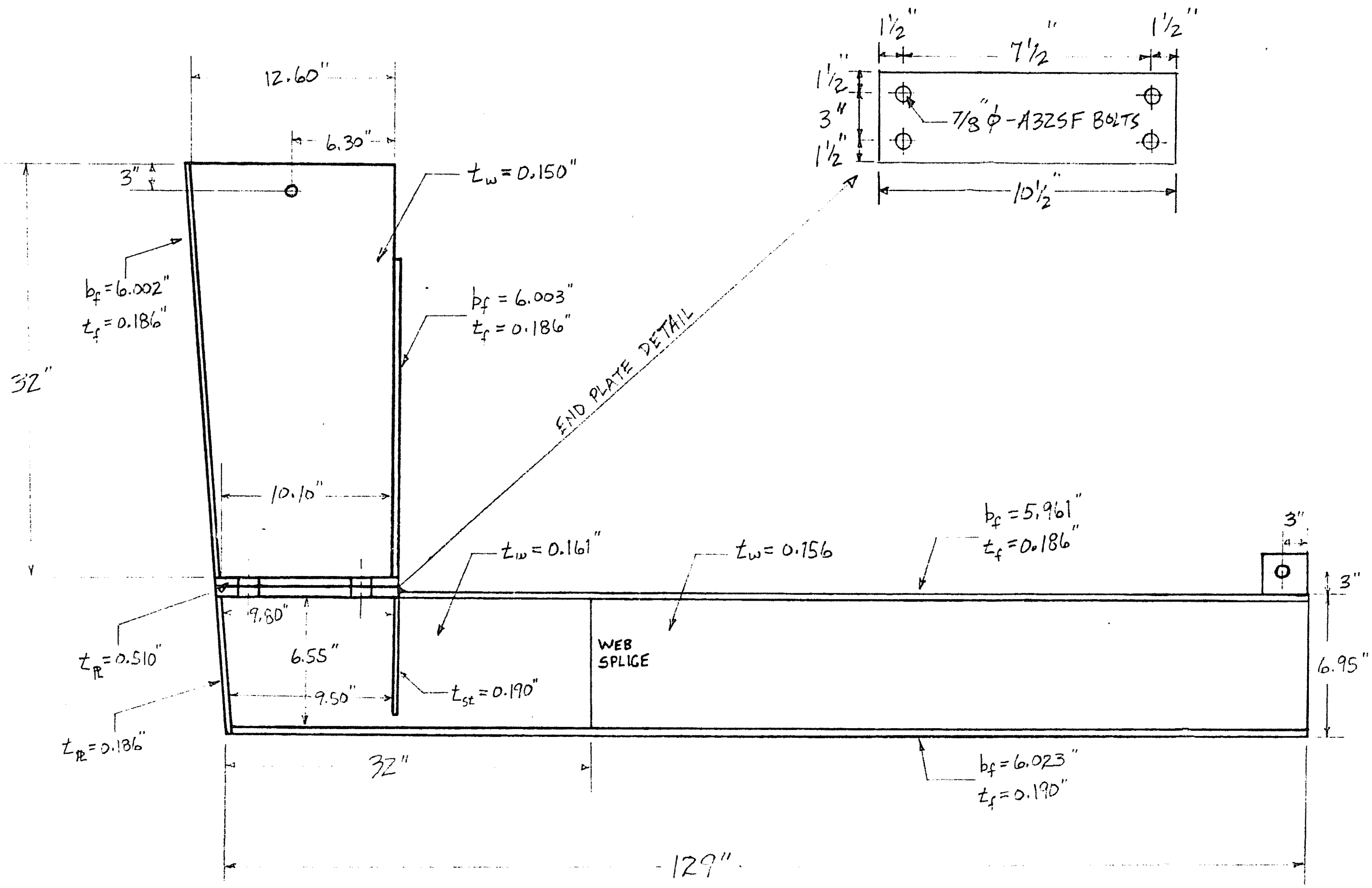


FIGURE A.1 SPECIMEN DETAILS, TEST FA-1

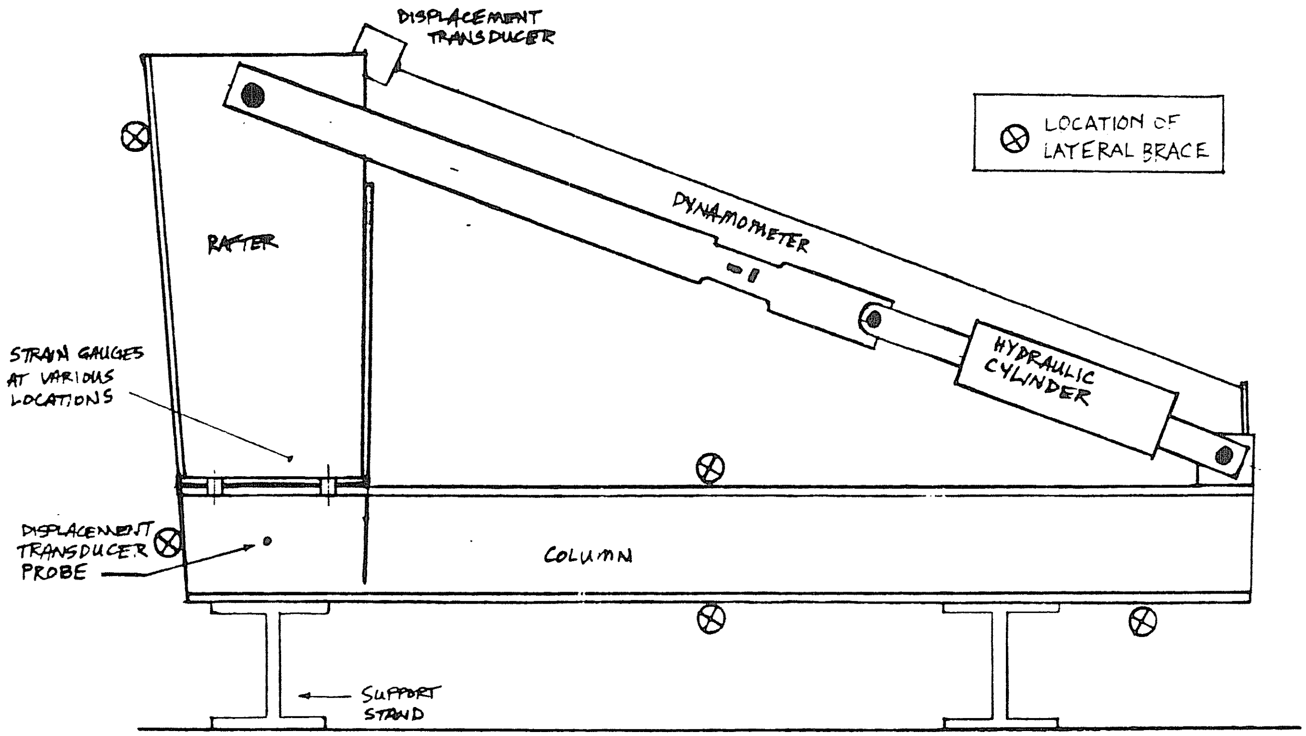


FIGURE A.2 TEST SET-UP, TEST FA-1

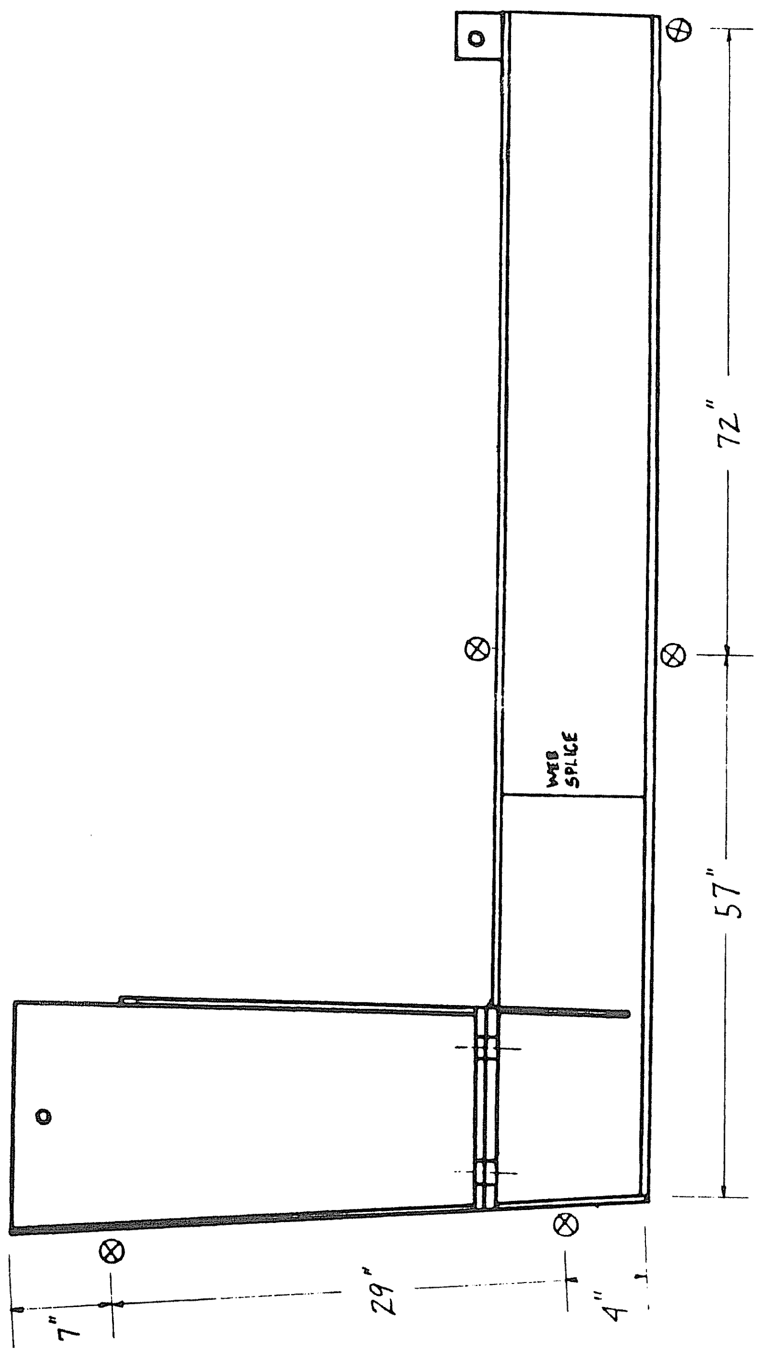


FIGURE A.3 LATERAL BRACE LOCATIONS, TEST FA-1

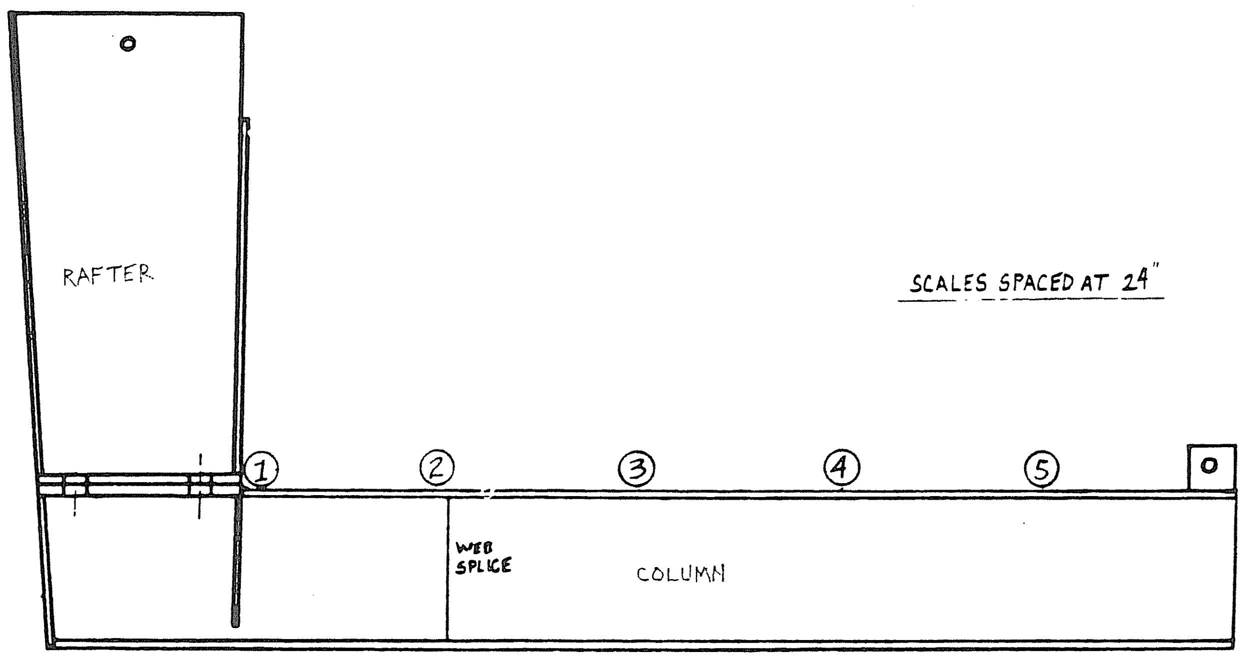


FIGURE A.4 LATERAL DISPLACEMENT SCALE LOCATIONS, TEST FA-1

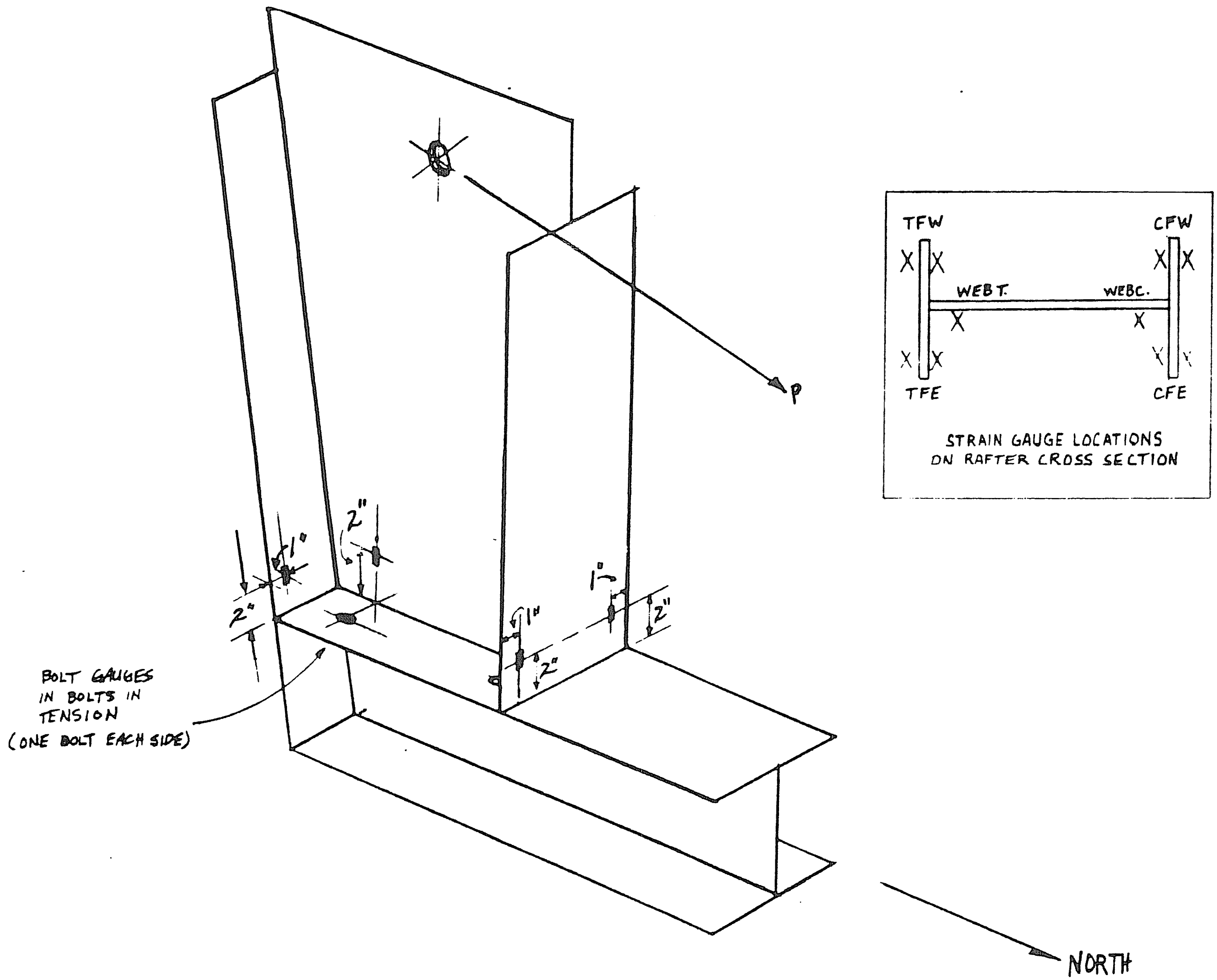


FIGURE A.5 STRAIN GAGE LOCATIONS, TEST FA-1

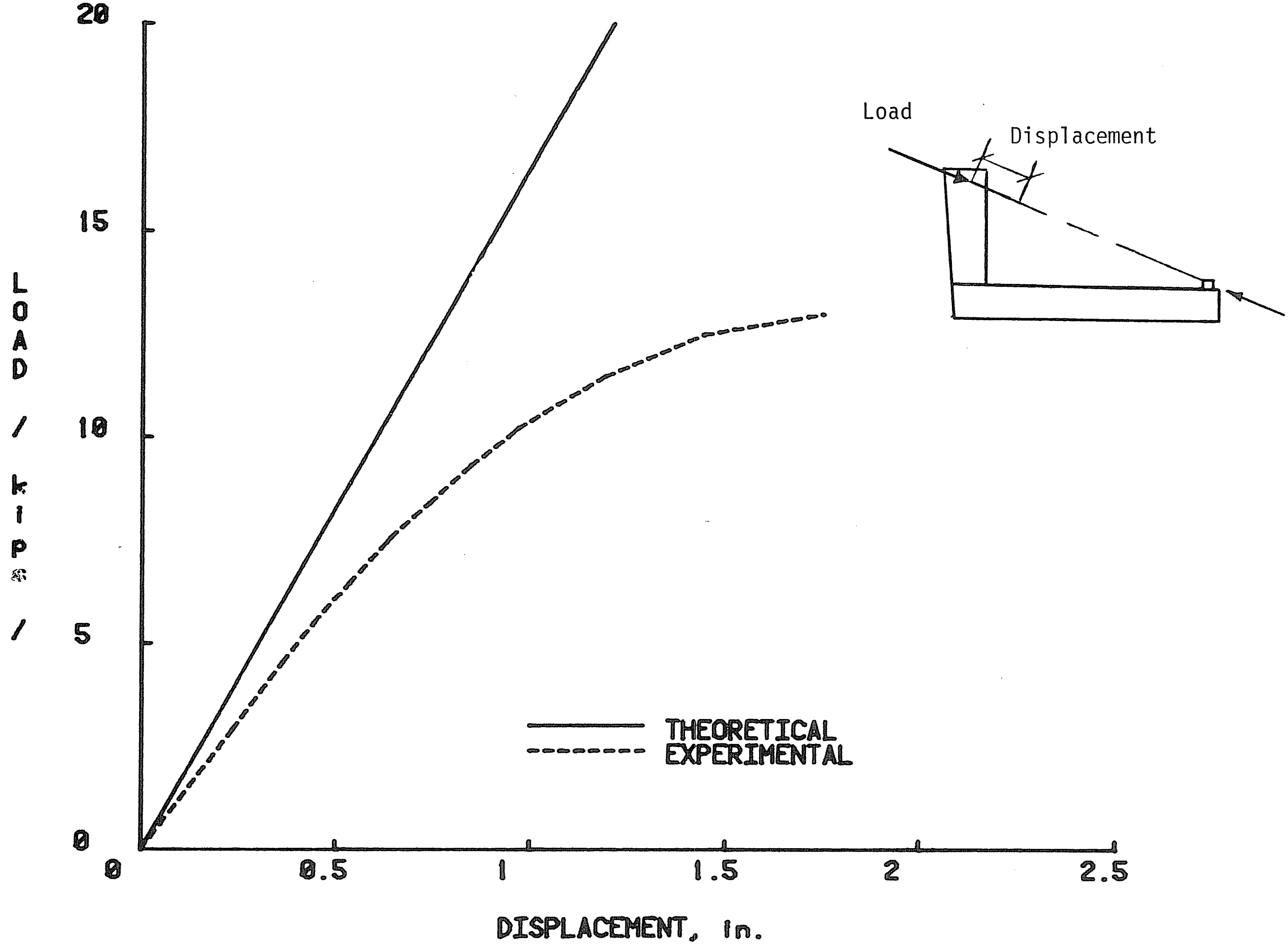


FIGURE A.6 **LOAD VS CHORD DISPLACEMENT, TEST FA-1**

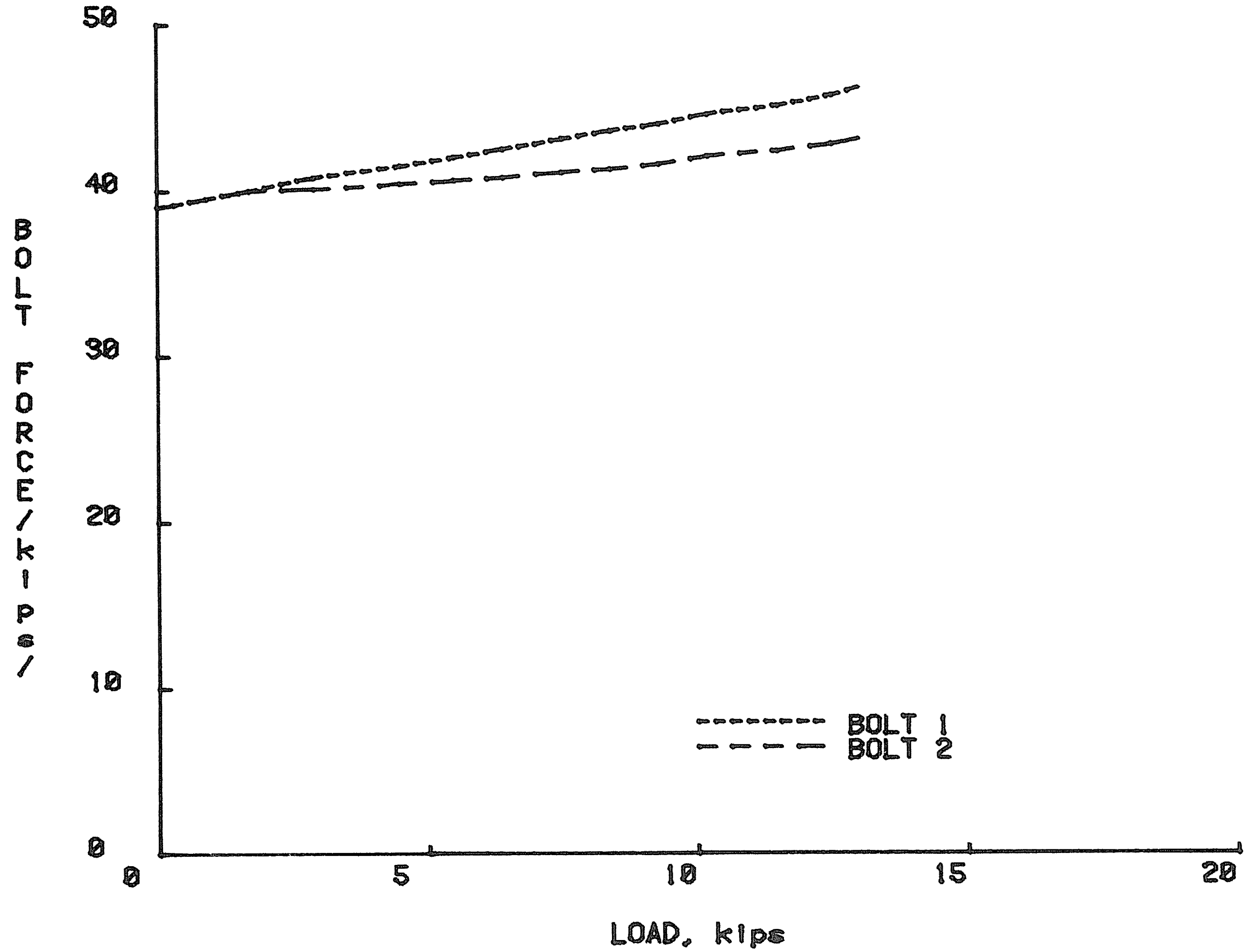


FIGURE A.7 BOLT FORCE VS LOAD, TEST FA-1

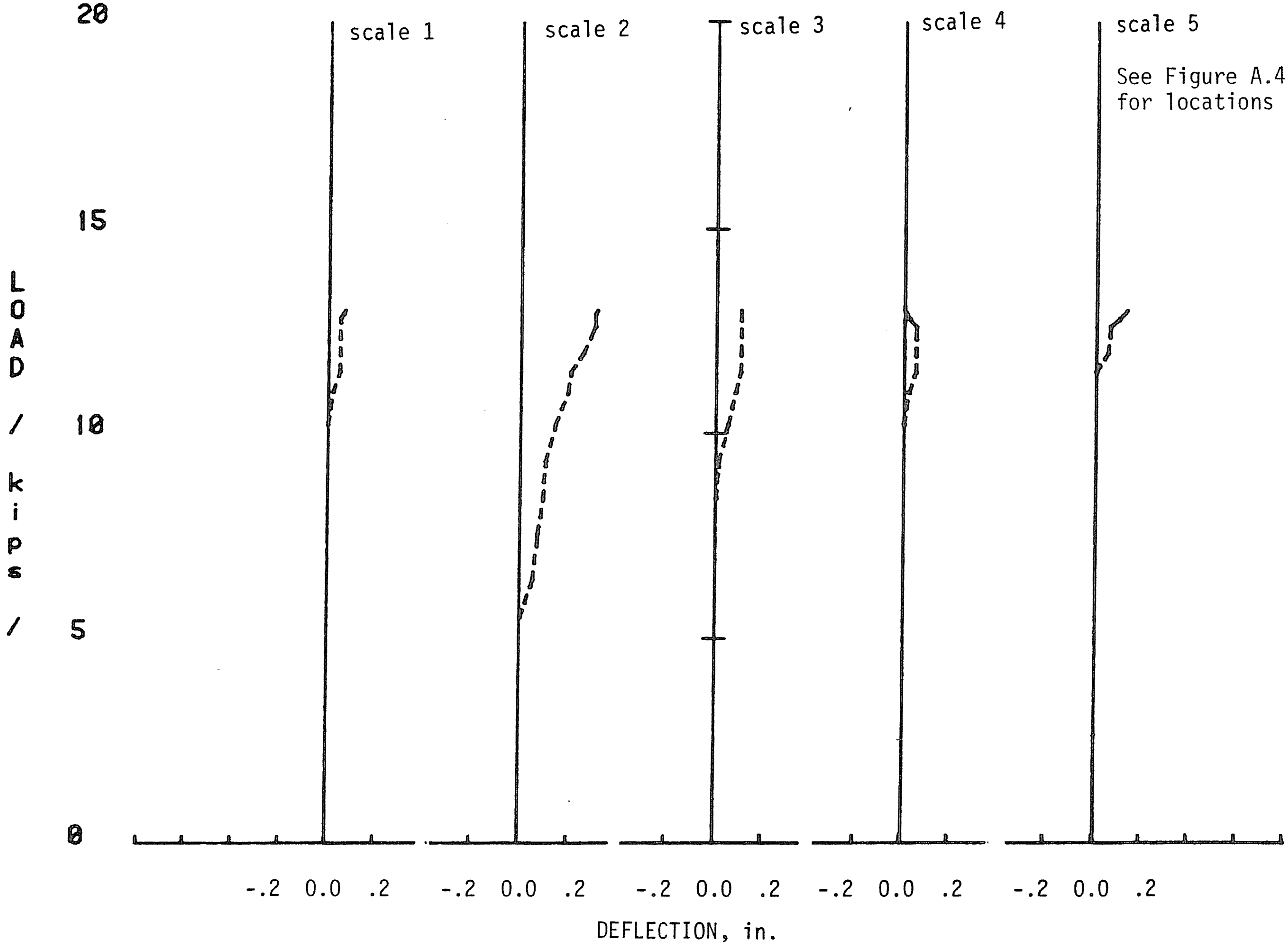


FIGURE A.8 **LOAD VS LATERAL DEFLECTIONS, TEST FA-1**

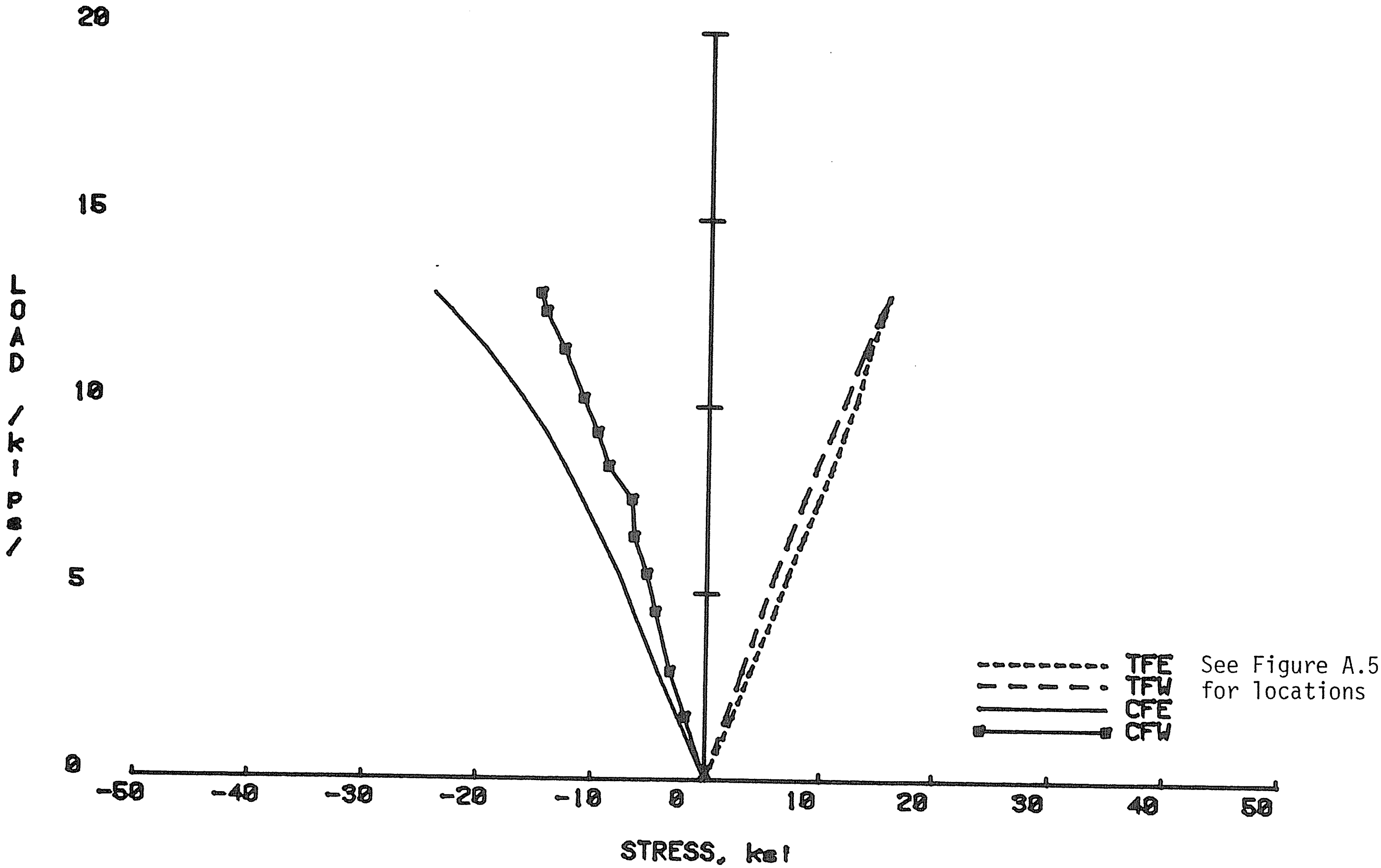


FIGURE A.9 LOAD VS RAFTER FLANGE STRESSES, TEST FA-1

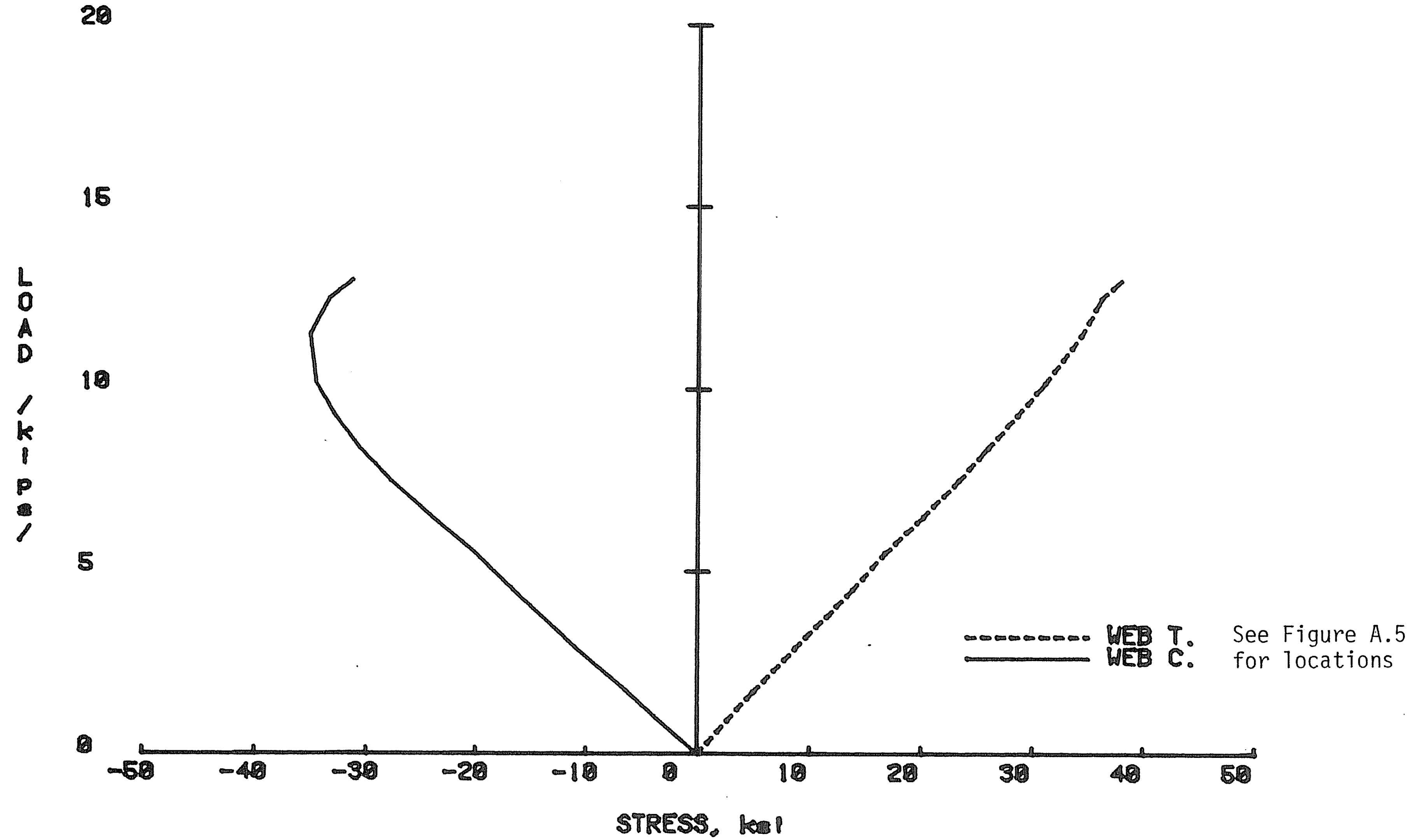


FIGURE A.10 LOAD VS RAFTER WEB STRESSES, TEST FA-1

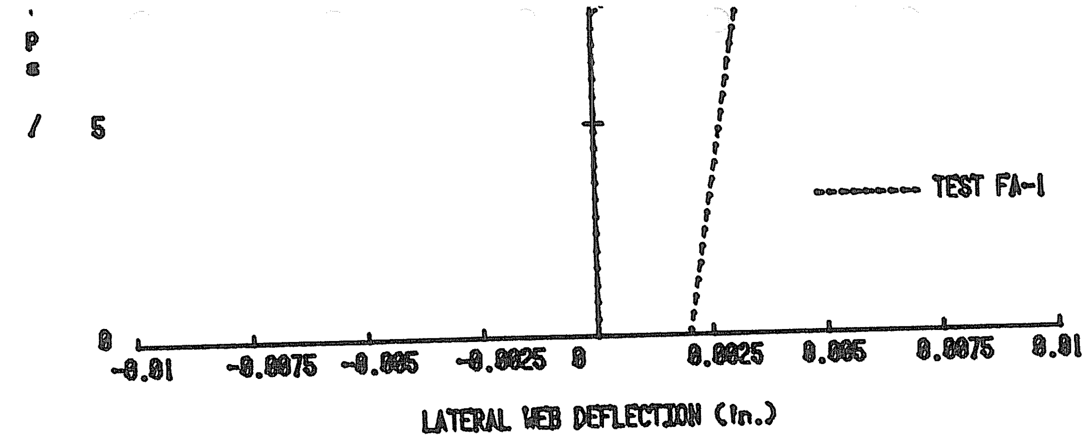


FIGURE A.11 LOAD VS. CENTER OF PANEL ZONE DEFLECTION, TEST FA-1

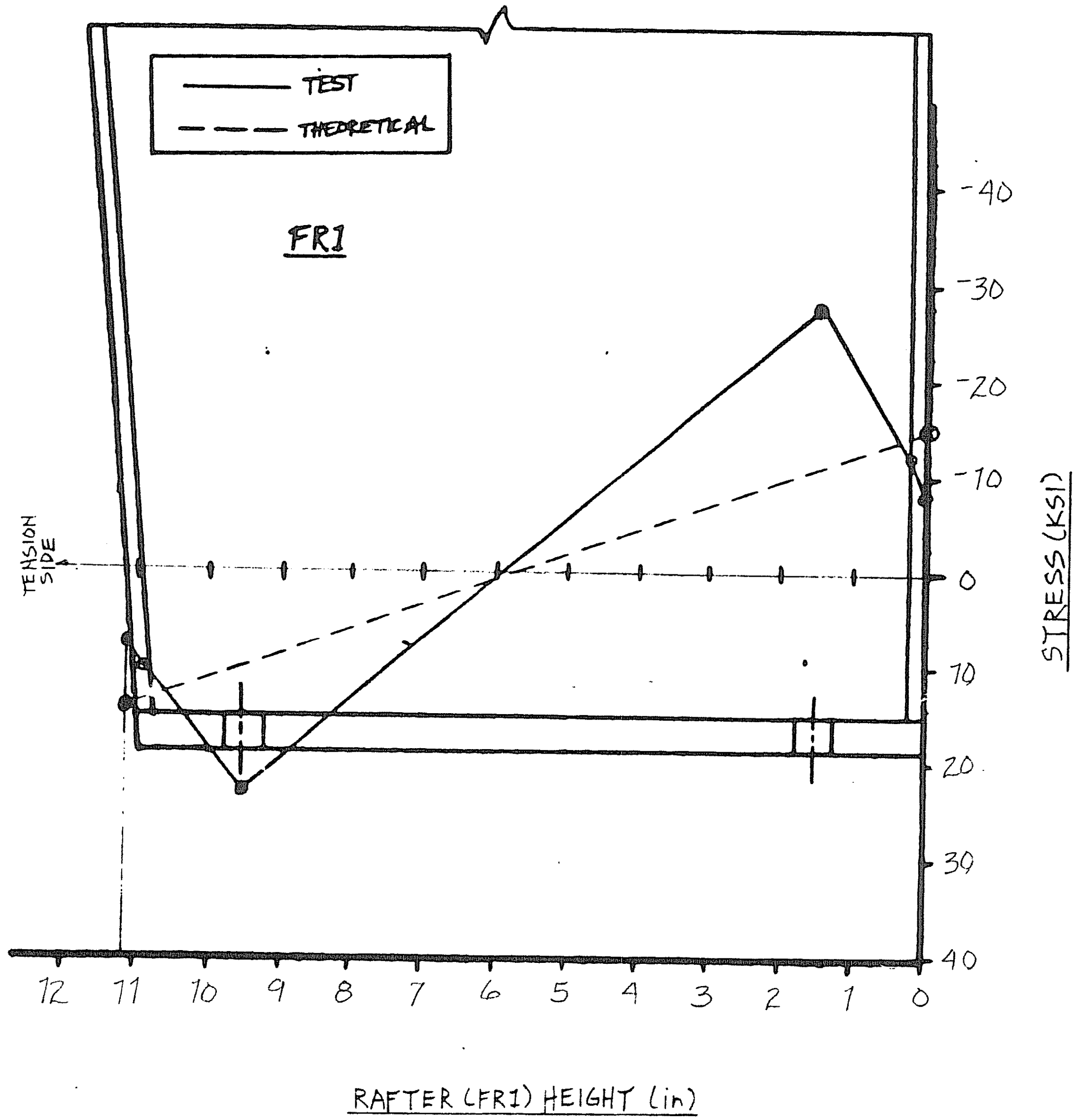


FIGURE A.12 STRESS VARIATION ACROSS RAFTER AT WORKING LOAD, TEST FA-1

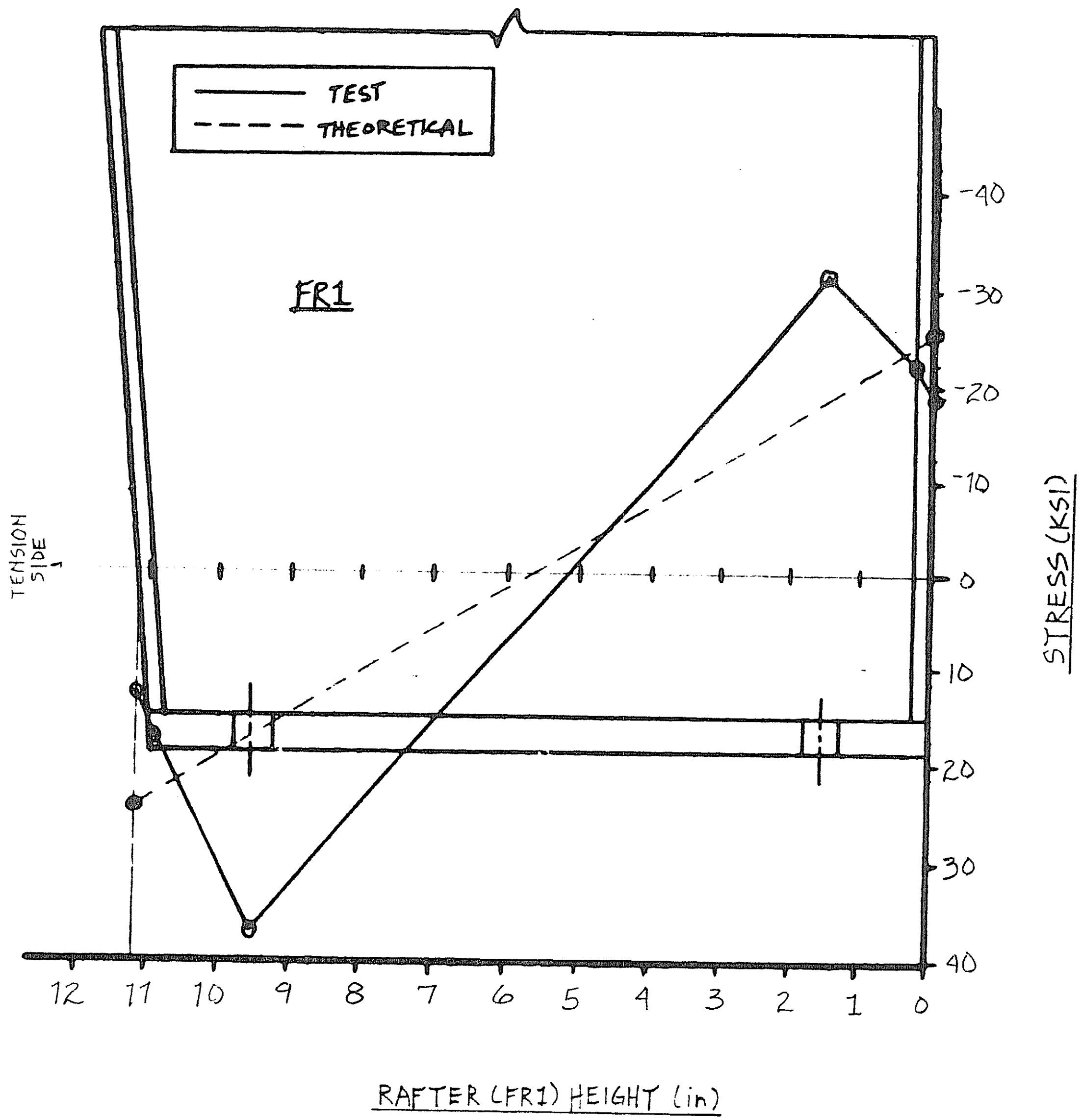


FIGURE A.13 STRESS VARIATION ACROSS RAFTER AT FAILURE LOAD, TEST FA-1

APPENDIX B

FA-2 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: MESCO Knee Test

Test No.: FA-2

Test Date: April 4, 1984

Purpose: Test of knee area

Number of Tension Bolts: 2 **Bolt Gage (g):** 3" **Pitch:** 1.5"

Bolt Diameter: 0.85" (7/8") **End Plate Thickness (t):** 0.510"

End Plate Width (w): 6.0 **End Plate Length (de):** 13.2"

Panel Zone Web Plate Thickness: .165"

Initial Out-of-Straightness at the Center of Panel Zone: 0.0"

Pretension Force per Bolt: 39 kips

Failure Load, (Total Load): 22.1 kips

Failure Mode: Panel zone yielding

Predicted Failure Loads:

Method: Panel zone yielding **Total Load:** 21.98 kips

Method: ____ **Total Load:** ____

Discussion:

-At load equal to 4^k, some yielding occurred at web in panel zone and some yielding at column flange at the cross section where web of column is spliced. At load equal to 18.8 diagonal line appeared on whitewash in panel zone. At this load level the load-deflection curve started to become nonlinear and load-lateral deflection of web indicated large lateral deflections. This could be interpreted as sign of buckling of web in panel zone.

-As load increased, yielding of web and straightening (see curves) in panel zone indicated development of tension field. Continued loading of specimen caused yielding of tension field material. The maximum capacity was 22.1^k.

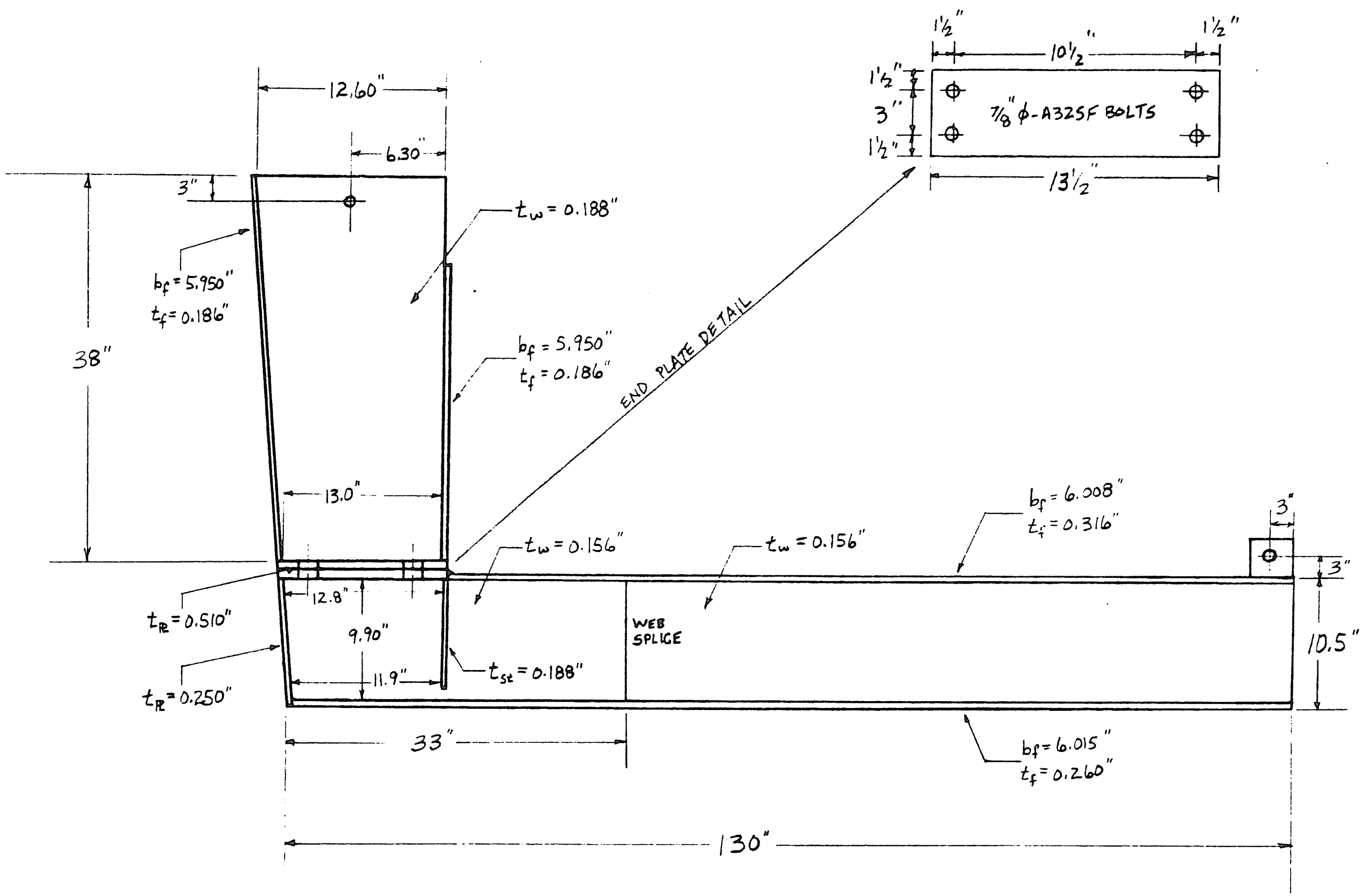


FIGURE B.1 SPECIMEN DETAILS, TEST FA-2

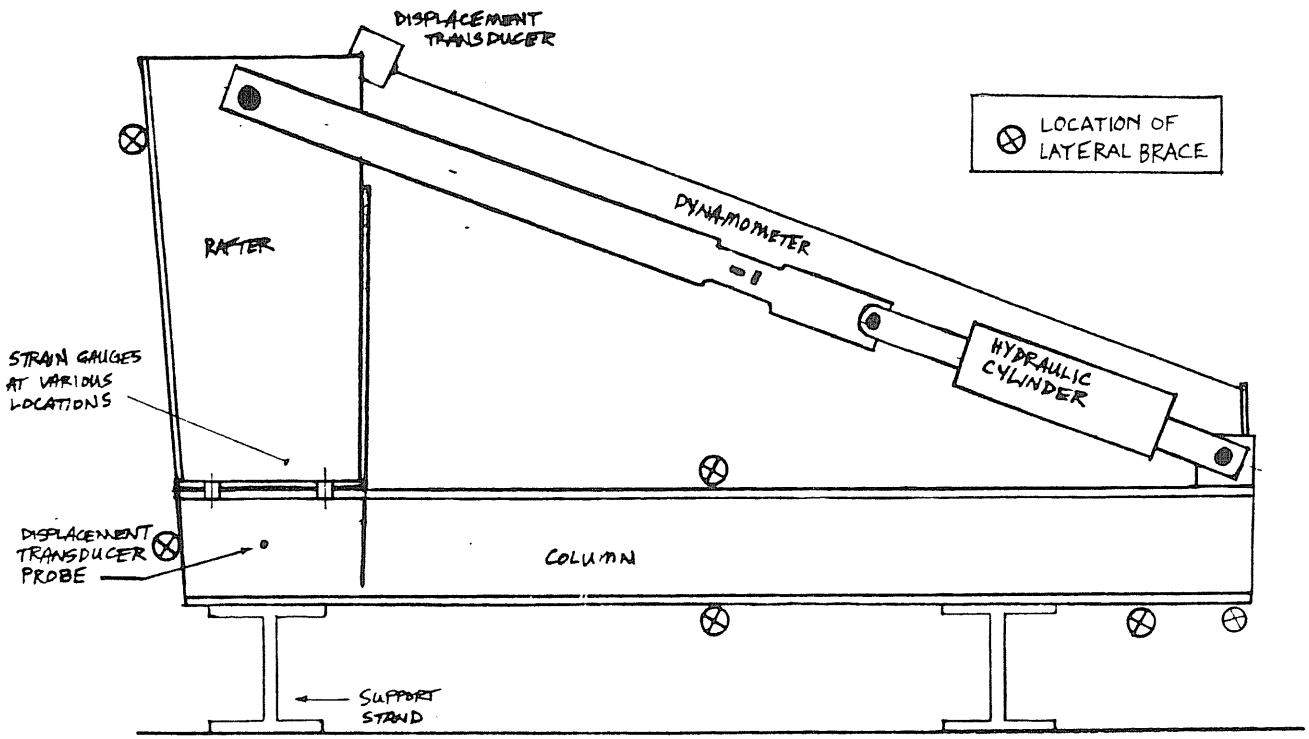


FIGURE B.2 TEST SET-UP, TEST FA-2

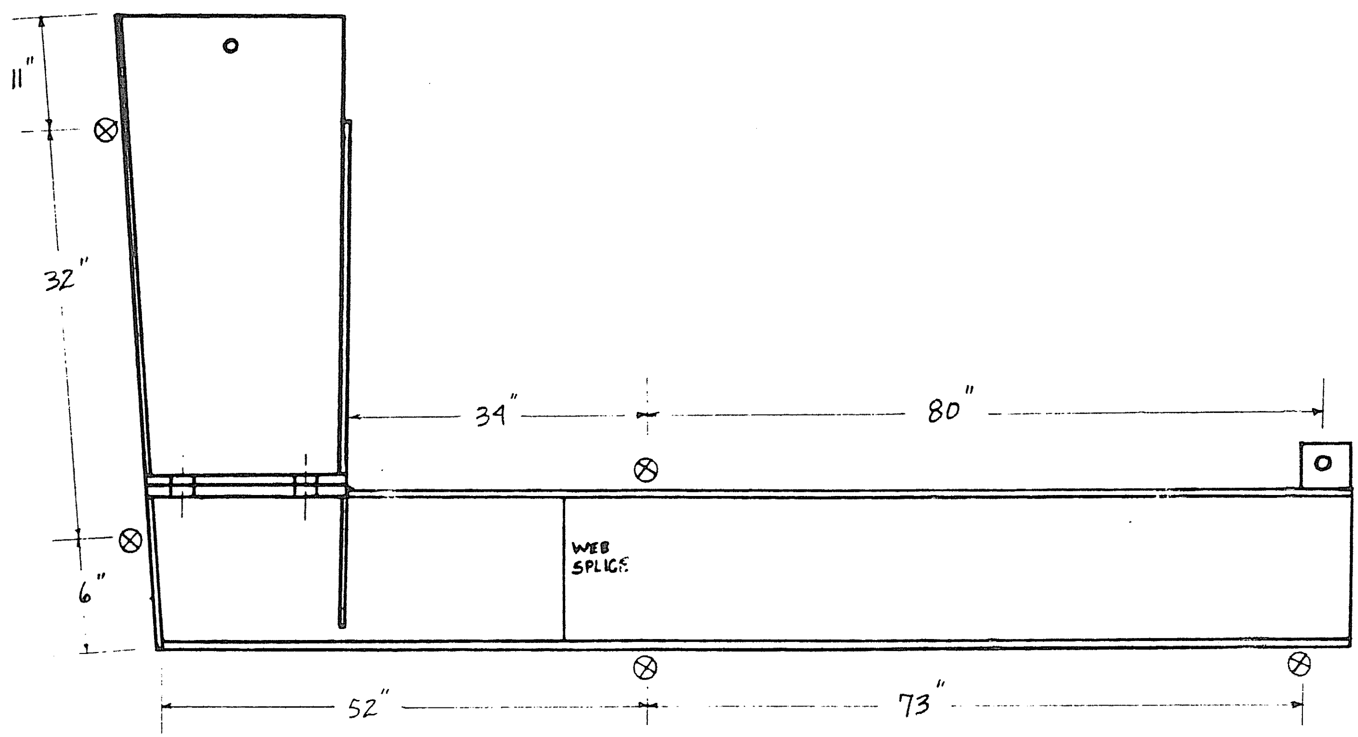


FIGURE B.3 LATERAL BRACE LOCATIONS, TEST FA-2

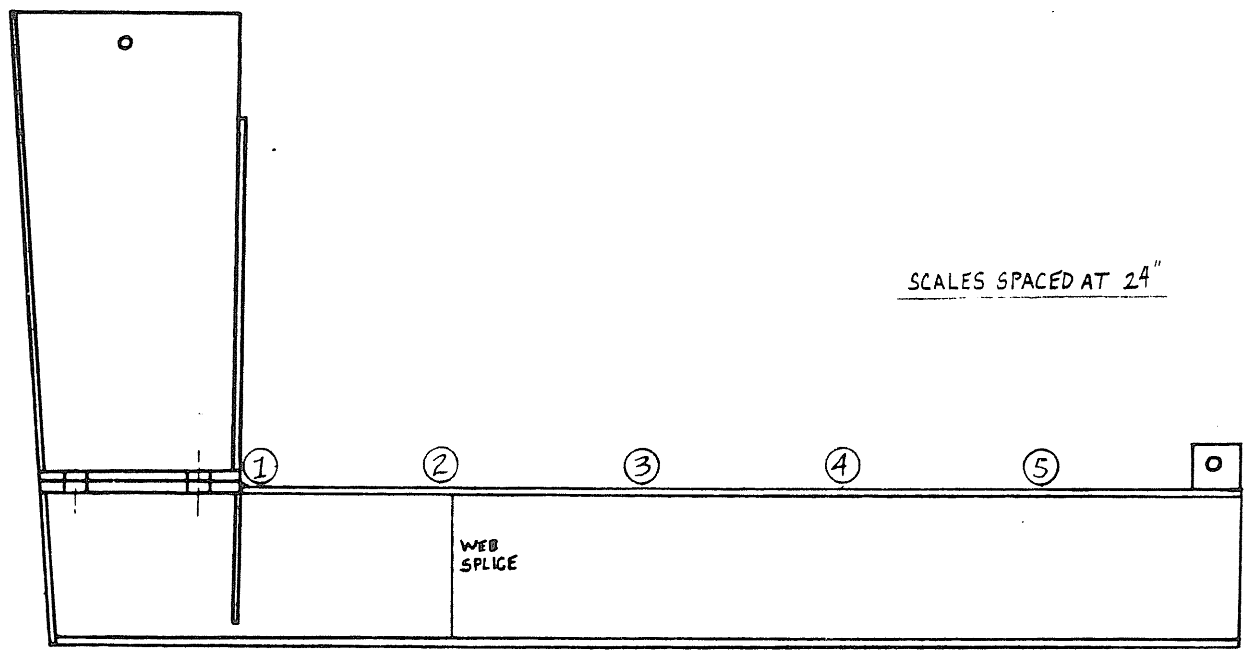


FIGURE B.4 LATERAL DISPLACEMENT SCALE LOCATIONS, TEST FA-2

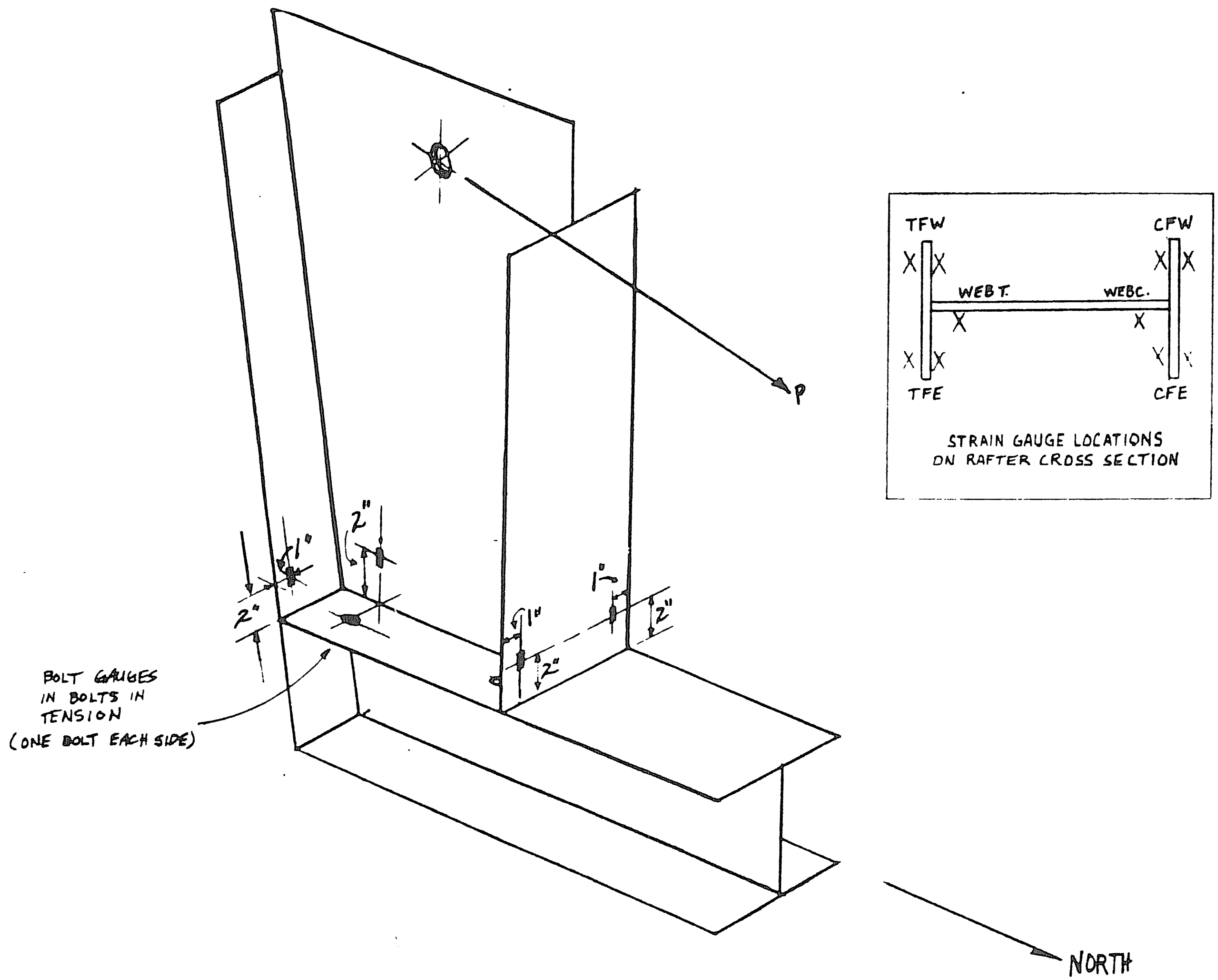


FIGURE B.5 STRAIN GAGE LOCATIONS, TEST FA-2

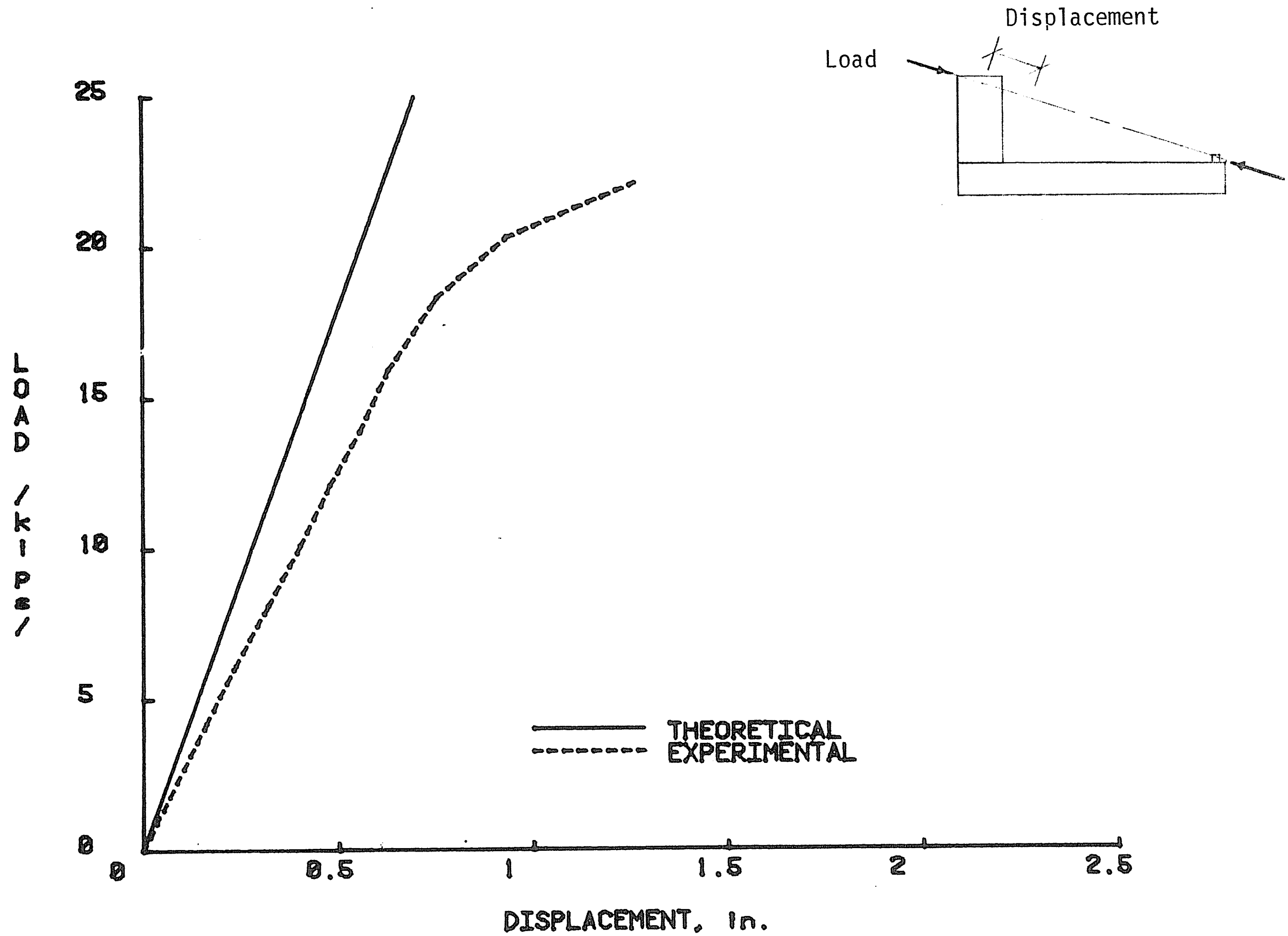


FIGURE B.6 LOAD VS CHORD DISPLACEMENT, TEST FA-2

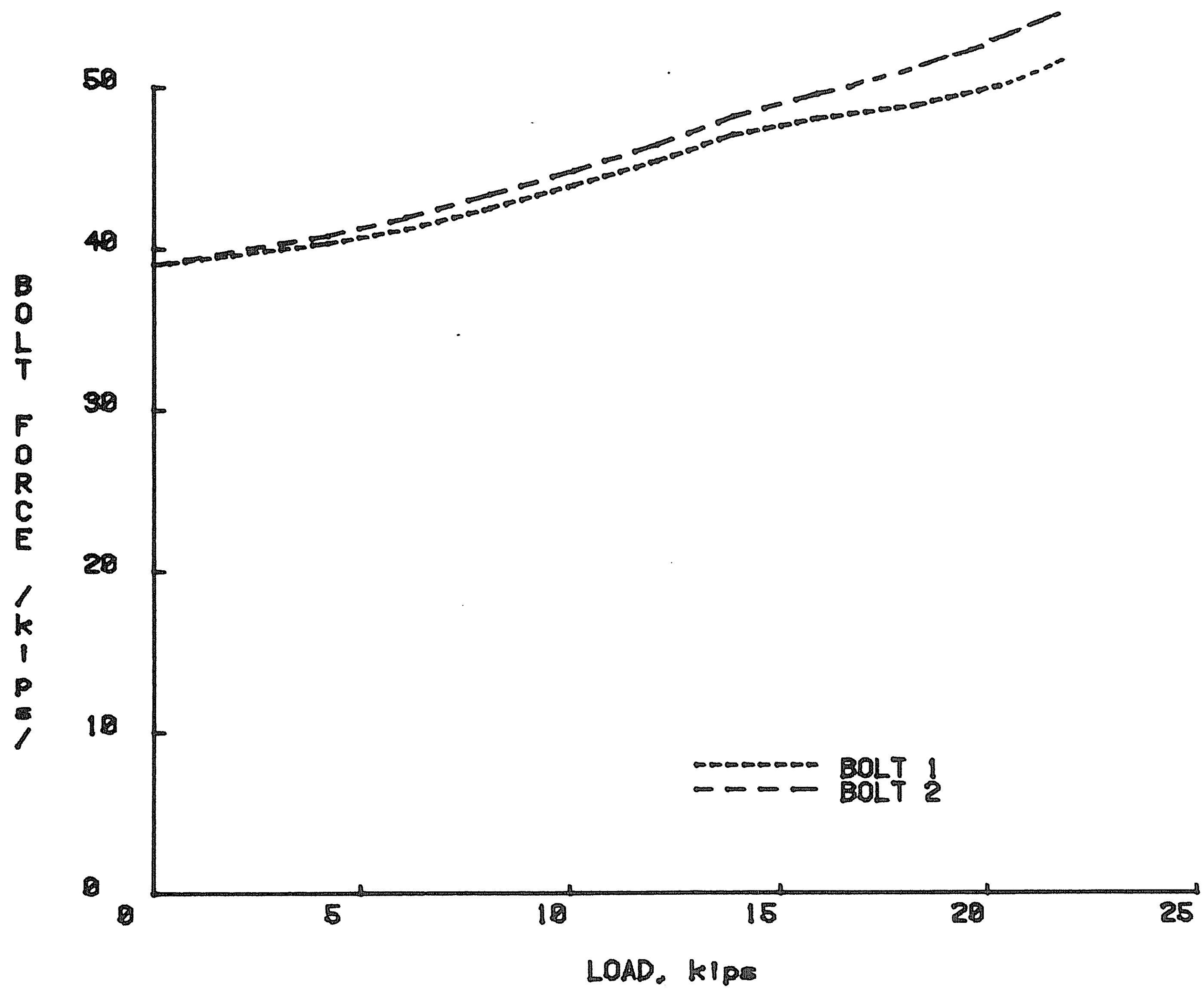


FIGURE B.7 BOLT FORCE VS LOAD, TEST FA-2

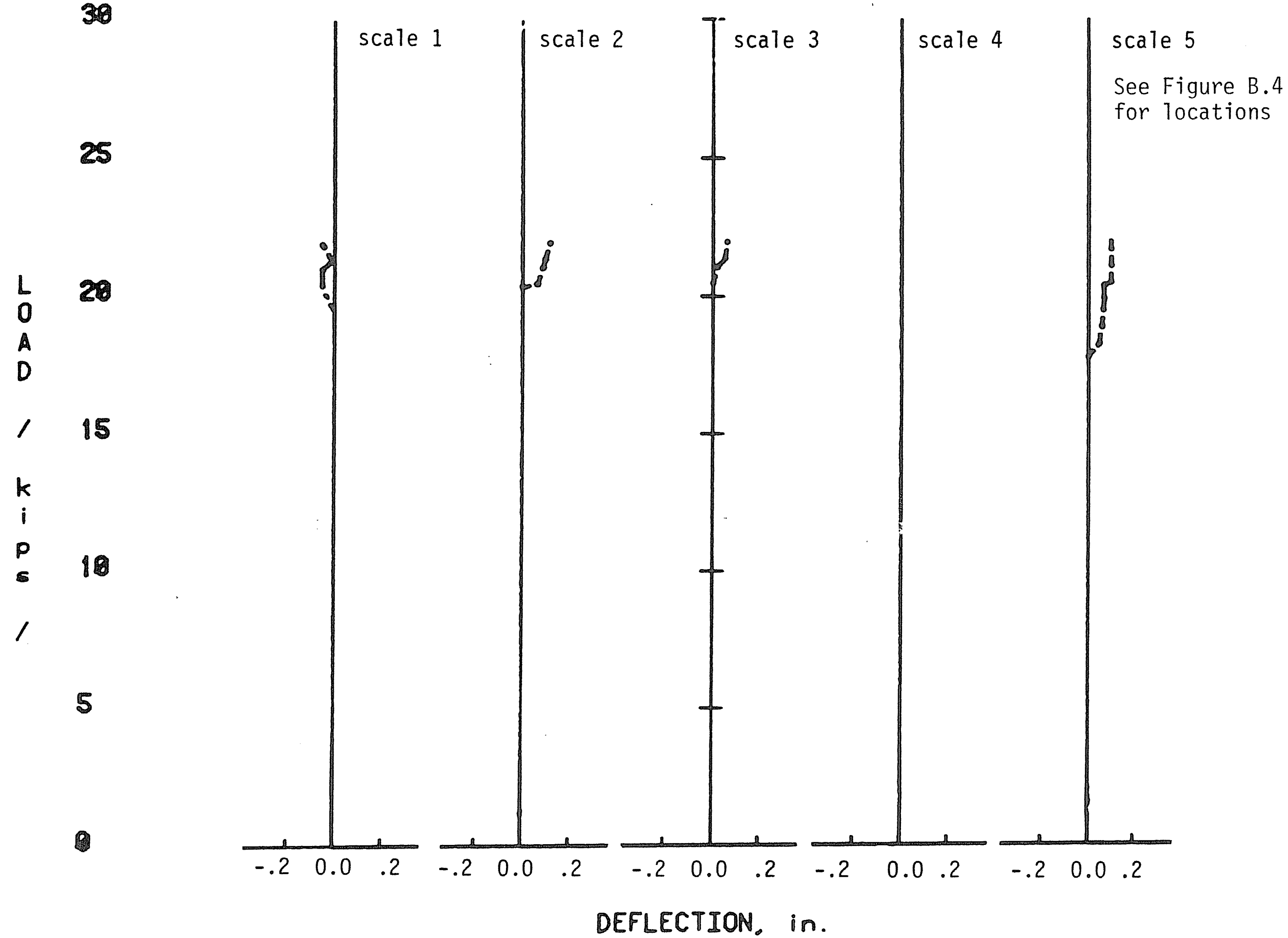


FIGURE B.8 **LOAD VS LATERAL DEFLECTIONS, TEST FA-2**

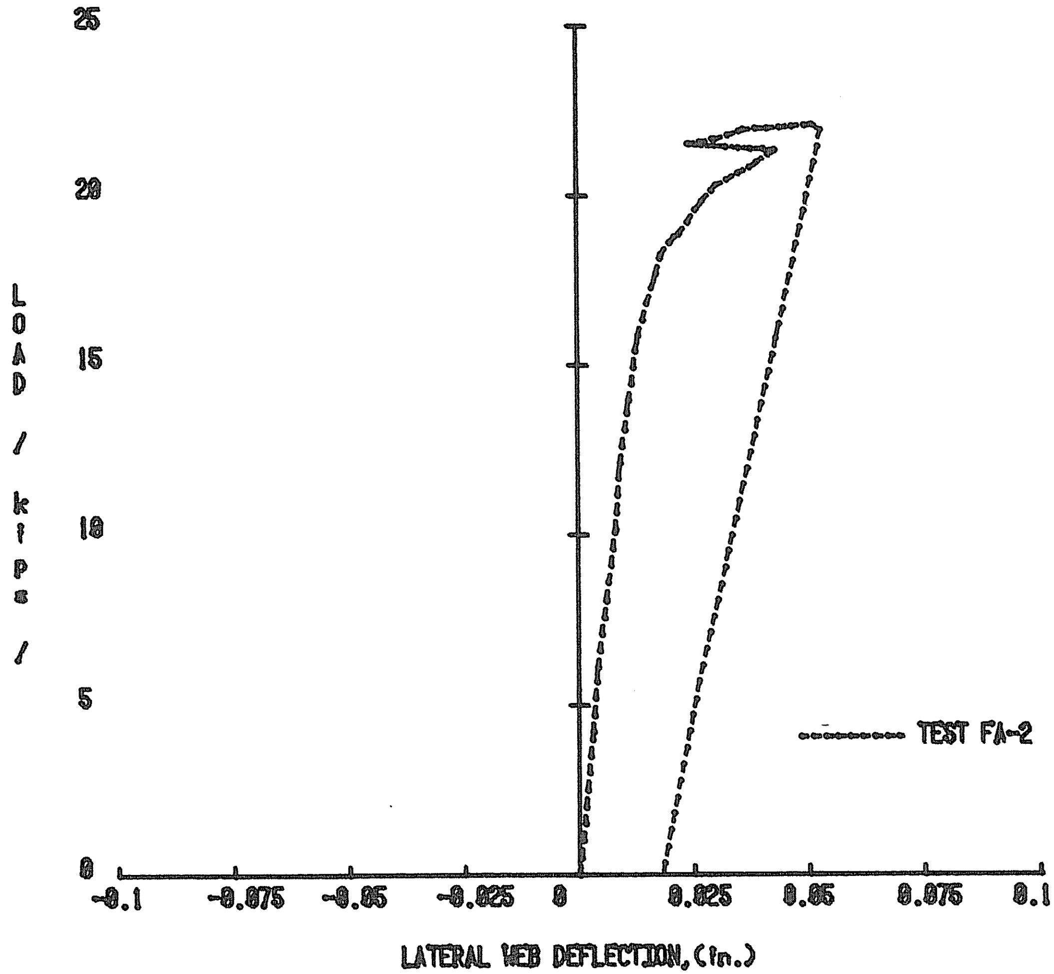


FIGURE B.9 LOAD VS. CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-2

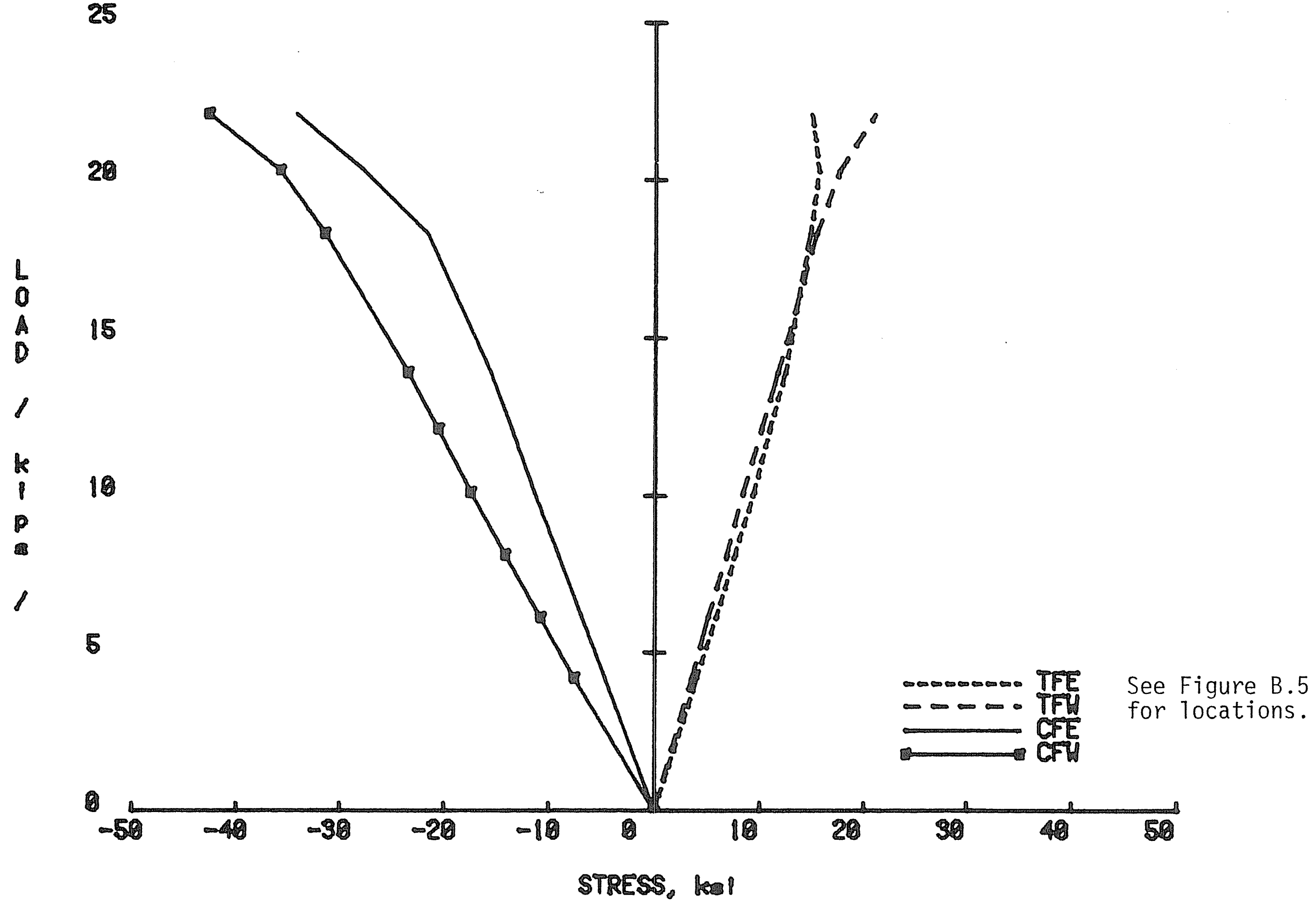


FIGURE B.10 LOAD VS RAFTER FLANGE STRESSES, TEST FA-2

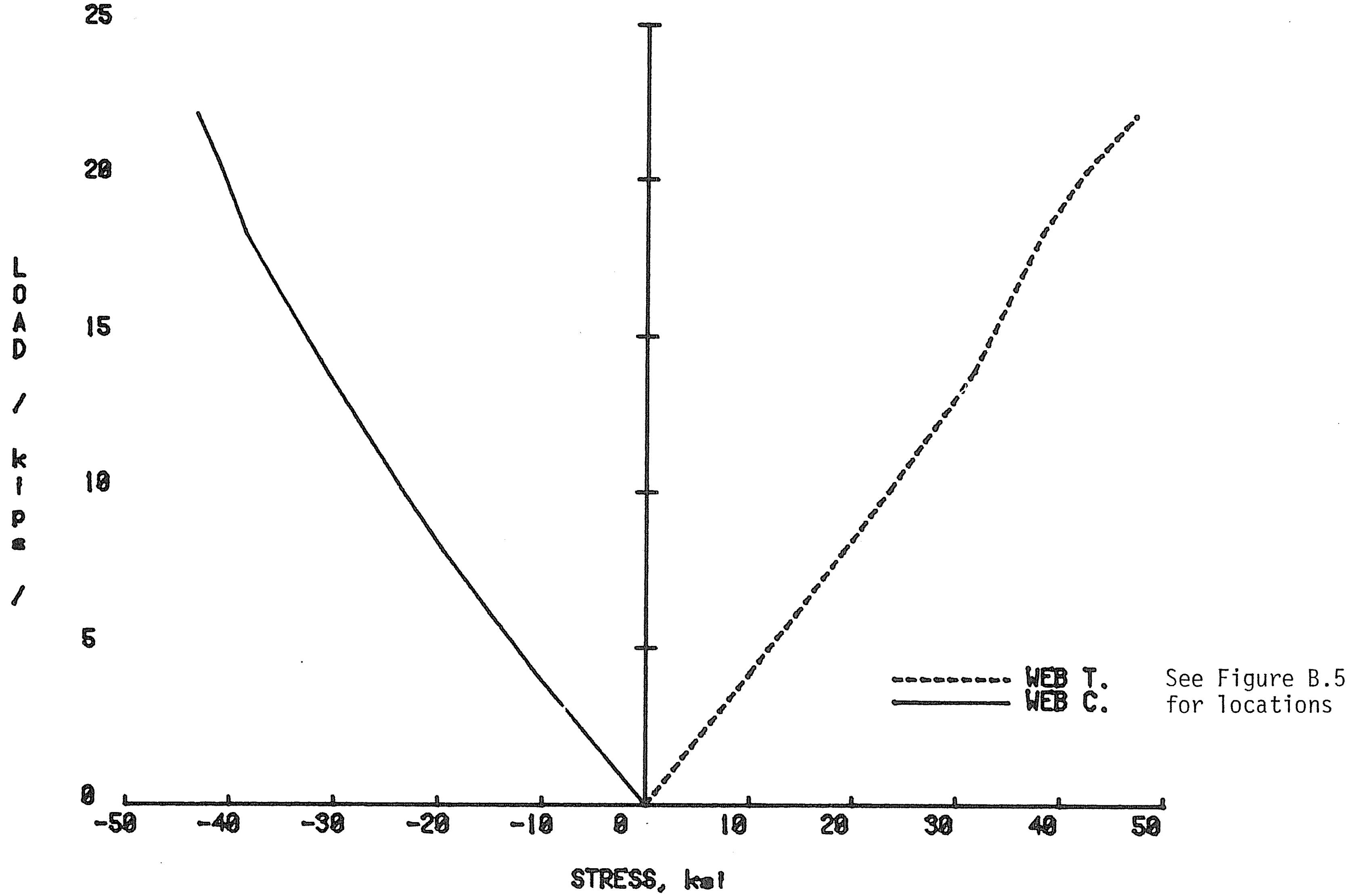


FIGURE B.11 LOAD VS RAFTER WEB STRESSES, TEST FA-2

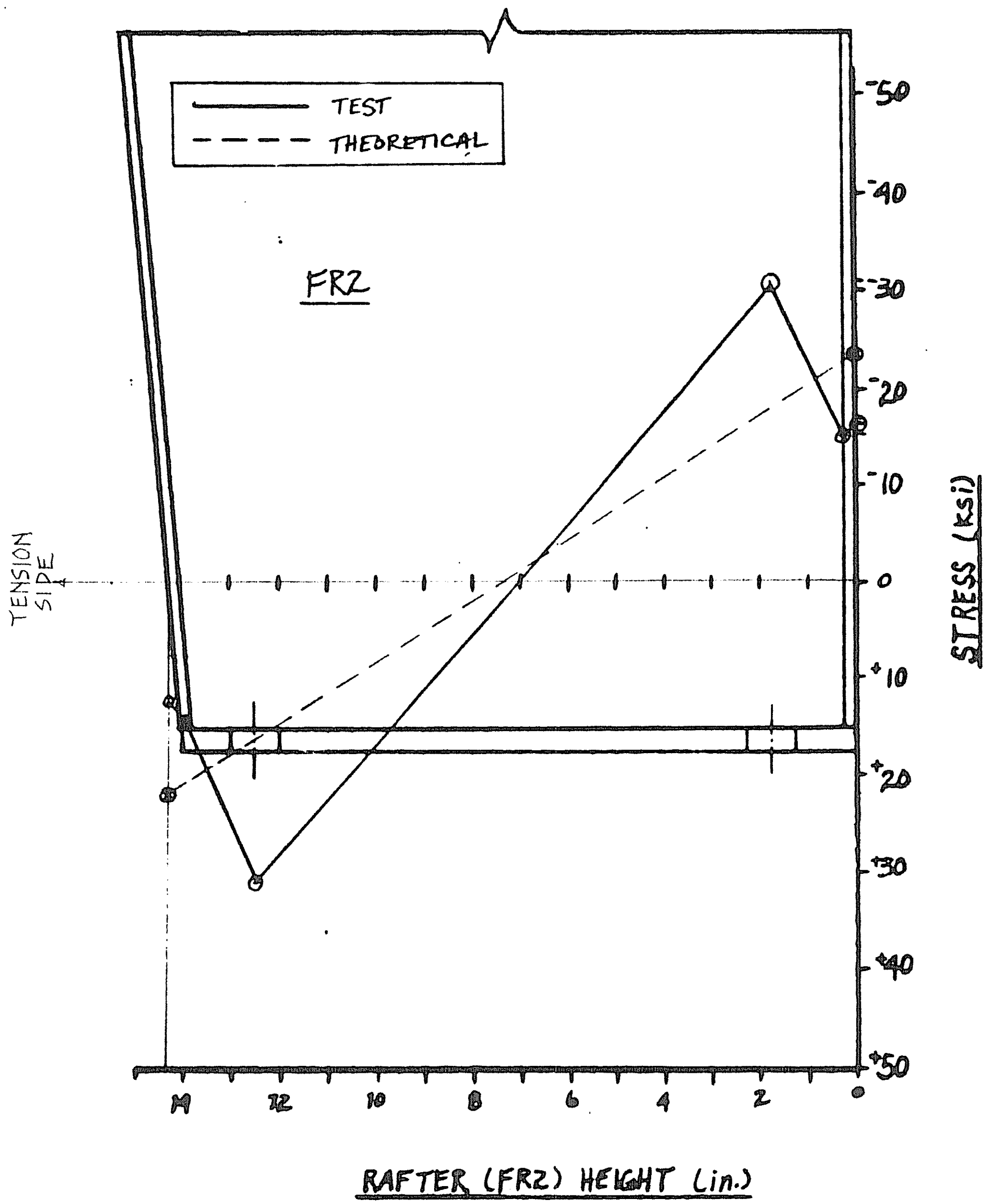


FIGURE B.12 STRESS VARIATION ACROSS RAFTER AT WORKING LOAD, TEST FA-2

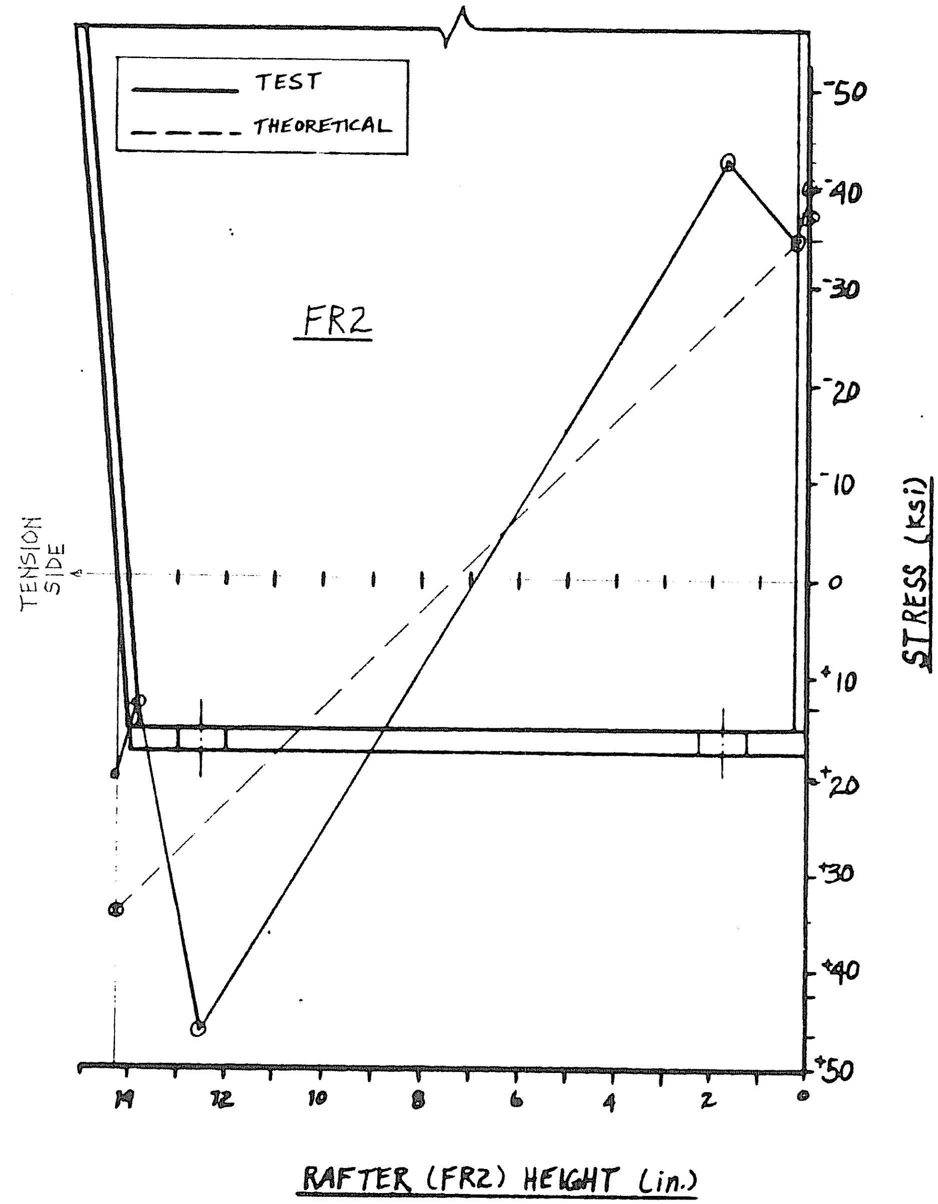


FIGURE B.13 STRESS VARIATION ACROSS RAFTER AT FAILURE LOAD, TEST FA-2

APPENDIX C

FA-3 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: MESCO KNEE TEST

Test No.: FA-3

Test Date: May 22, 1984

Purpose: Test of knee area

Number of Tension Bolts: 4 Bolt Gage (g): 3" Pitch: 3"

Bolt Diameter: 3/4" End Plate Thickness (t): 0.510"

End Plate Width (w): 6" End Plate Length (de): 24.37"

Panel Zone Web Plate Thickness: 0.186"

Initial Out-of-Straightness at the Center of Panel Zone: 0.291"

Pretension Force per Bolt: 28 kips

Failure Load, (Total Load): 19.8 kips

Failure Mode: Panel zone plate buckling

Predicted Failure Loads:

Method: Comp. failure of Col. Total Load: 26.84 kips

Method: ~~Panel Zone plate~~ Flange buckling Total Load: 20.84 kips

Discussion:

-At 15 kips load, some yield lines were observed on compression flange of rafter near end plate connection.

-At 17.5 kips load, severe yield lines occurred at rafter as well as column compression flange. The compression flange of the rafter started to twist.

-At 17.9 kips load, buckling of compression flange of column and rafter was quite visible. Buckling of rafter was more severe than column.

-At 19.8 kips load, very severe buckling of compression flange of rafter could be observed. The specimen did not take load larger than 19.8 kips.

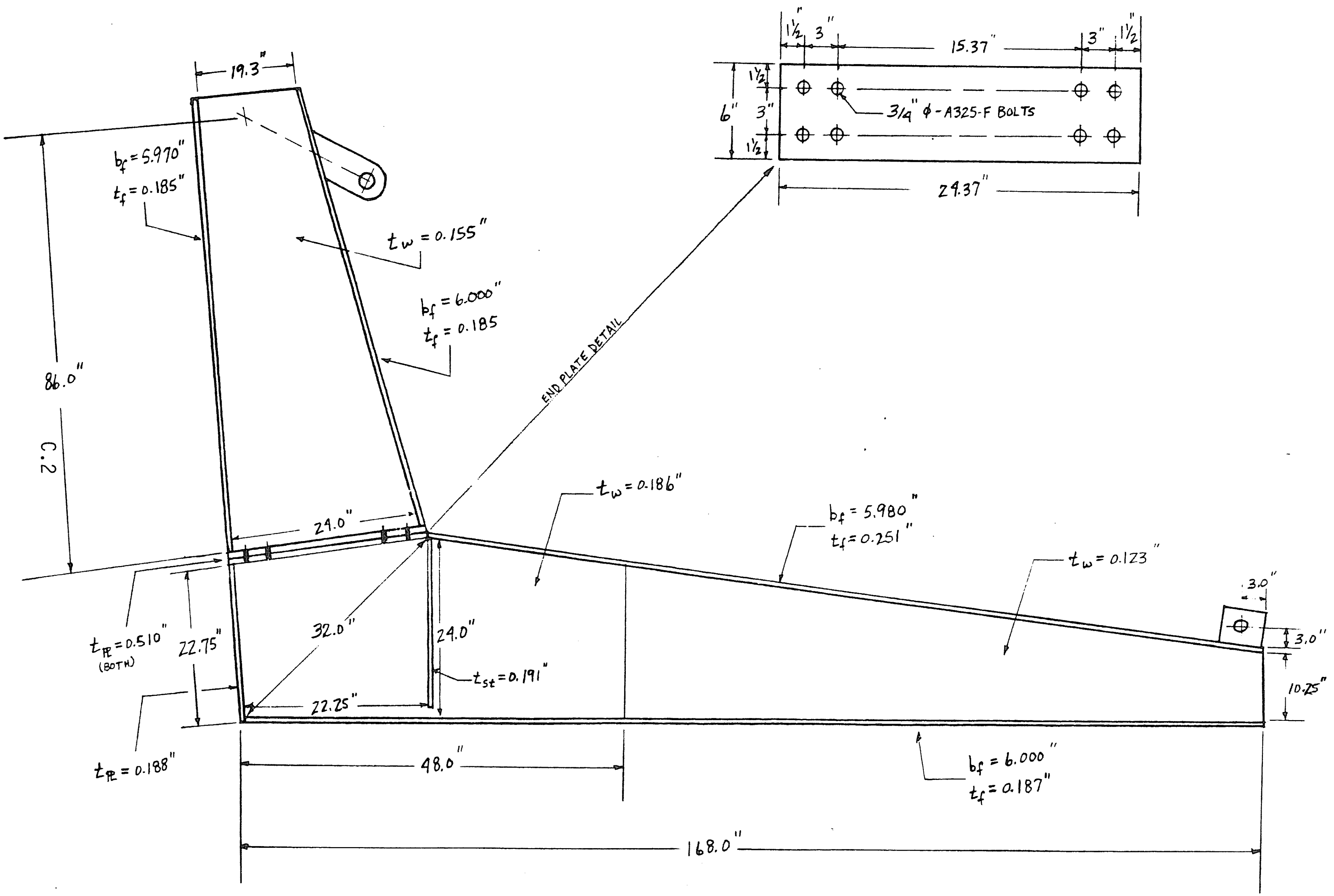


FIGURE C.1 SPECIMEN DETAILS, TEST FA-3

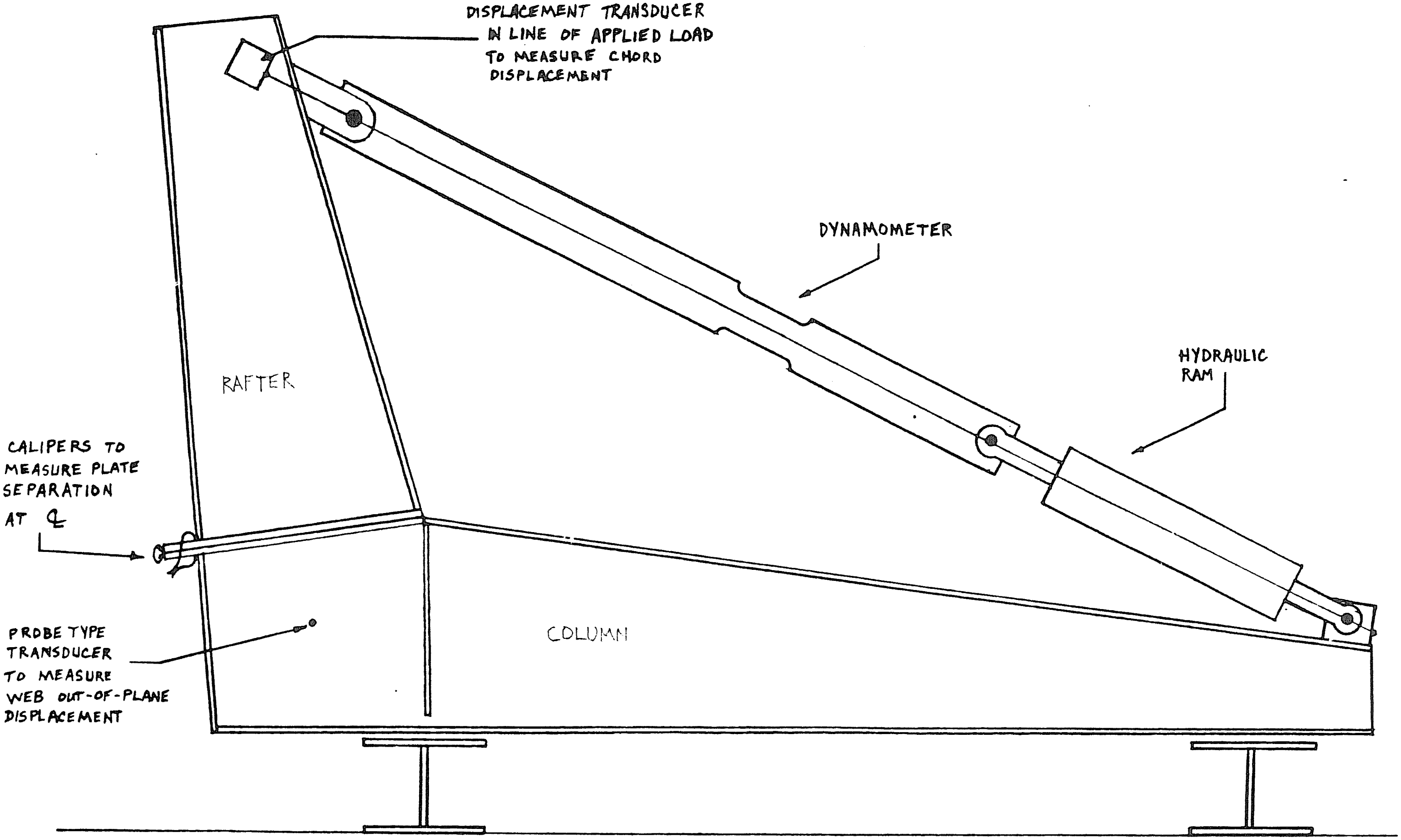


FIGURE C.2 TEST SET-UP, TEST FA-3

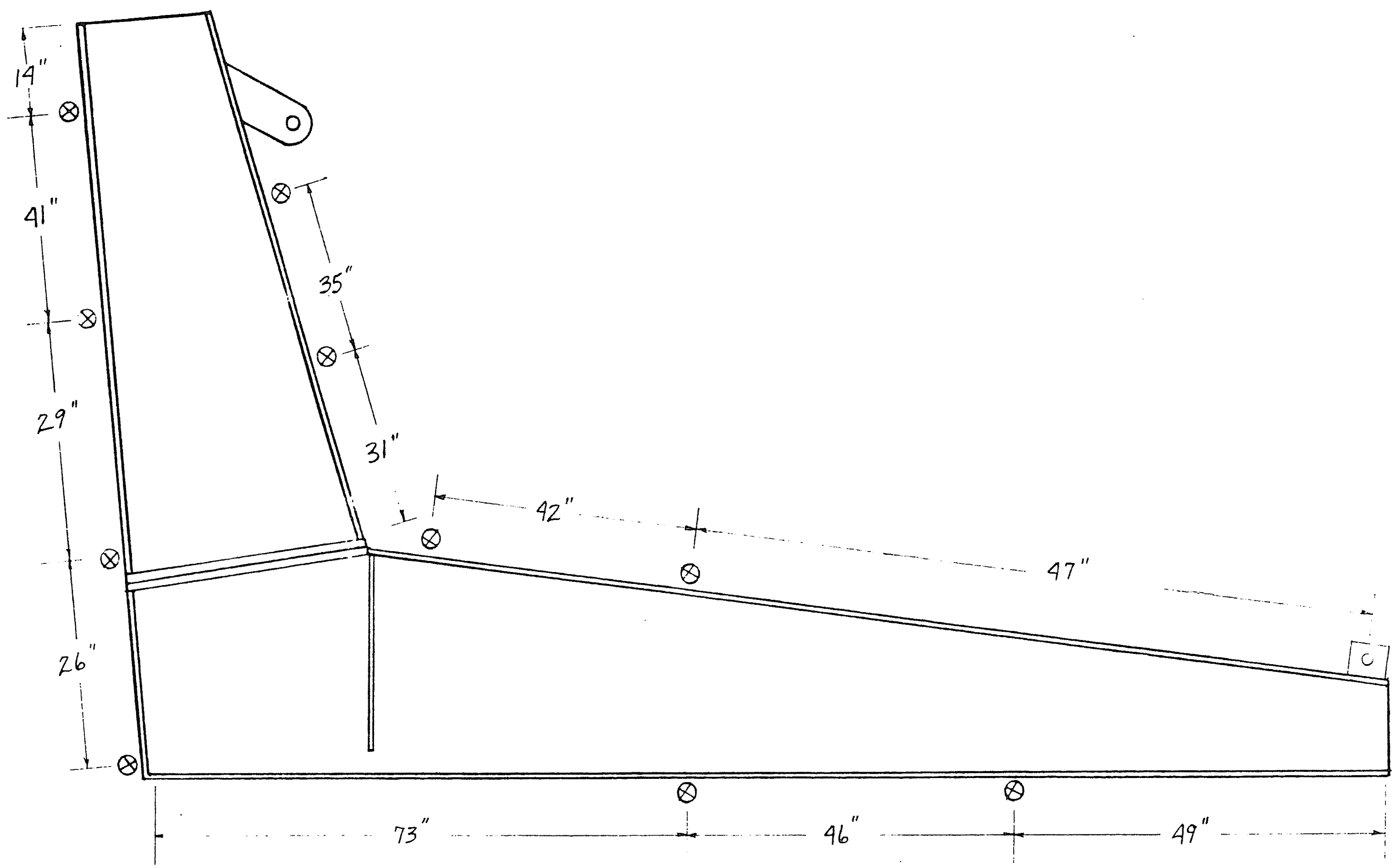


FIGURE C.3 LATERAL BRACE LOCATIONS, TEST FA-3

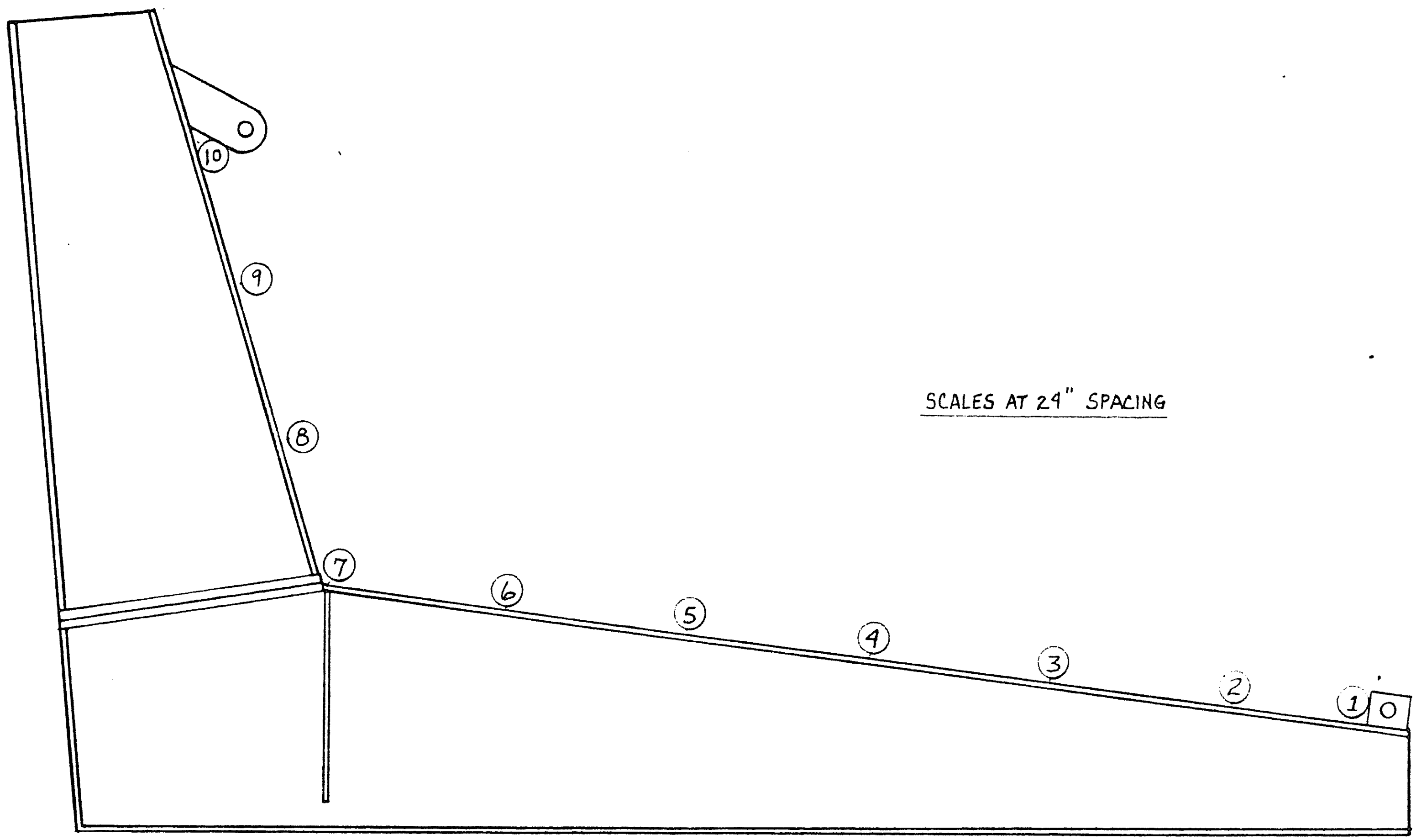


FIGURE C.4 LATERAL DISPLACEMENT SCALE LOCATIONS, TEST FA-3

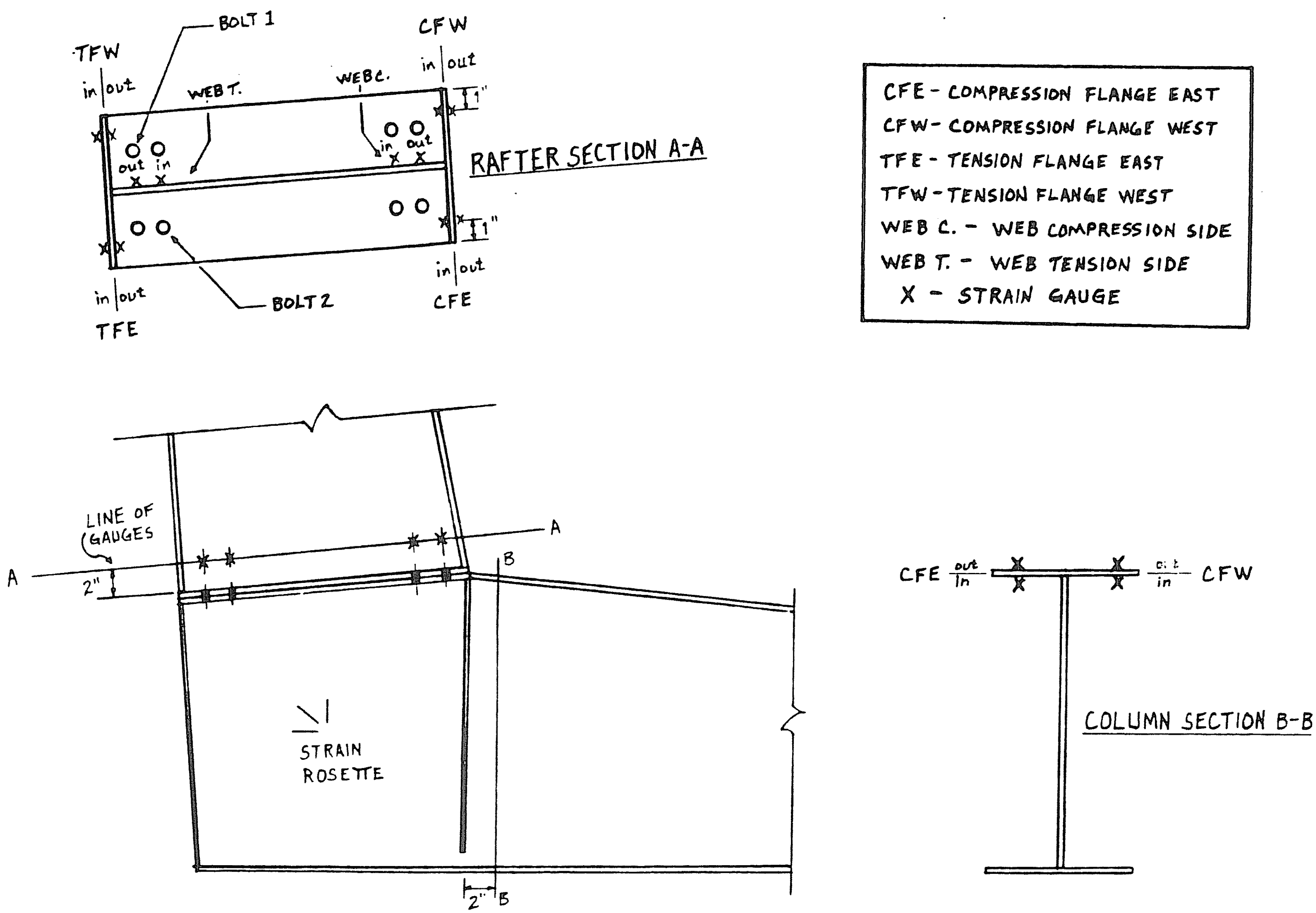


FIGURE C.5 STRAIN GAGE LOCATIONS, TEST FA-3

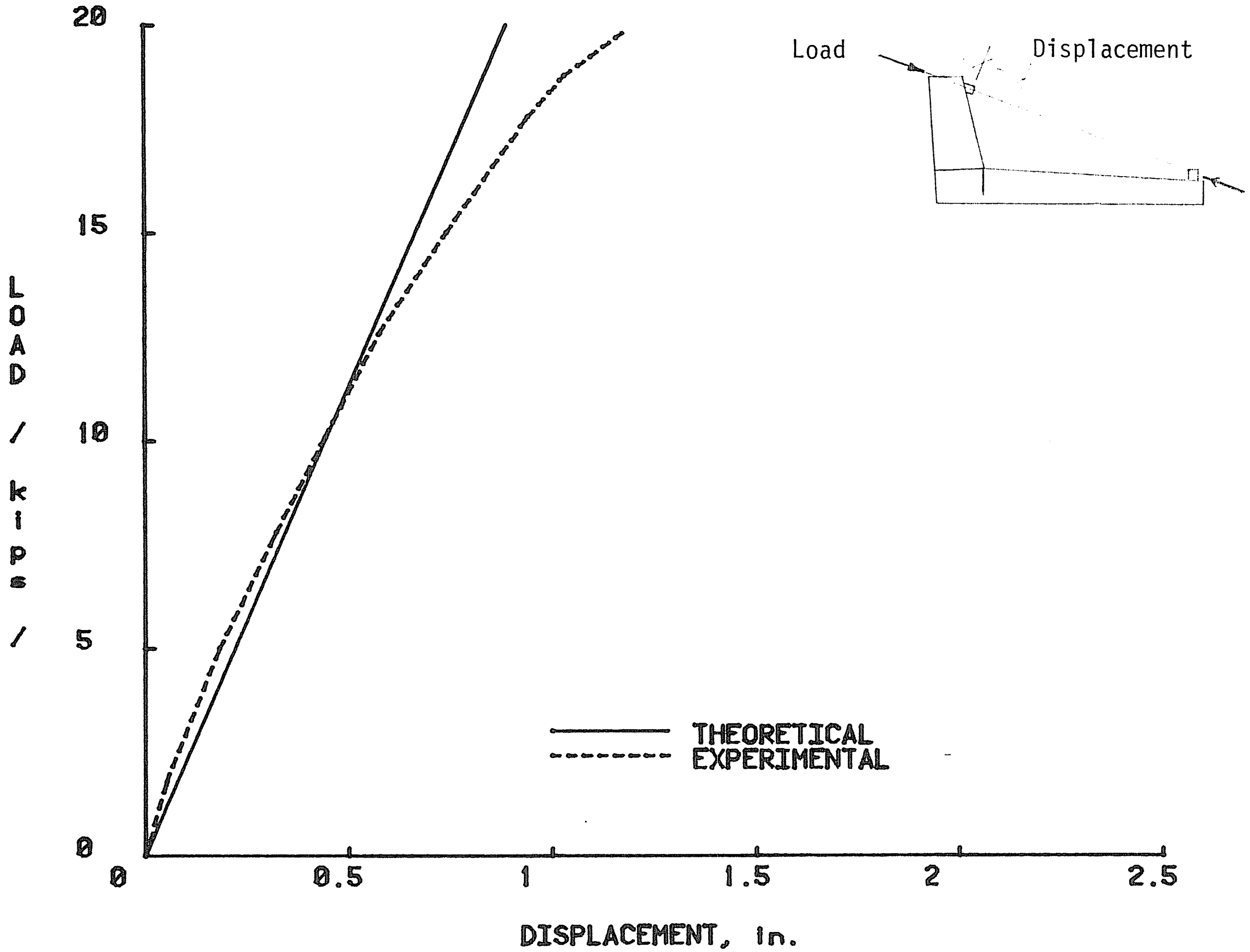


FIGURE C.6 **LOAD VS CHORD DISPLACEMENT, TEST FA-3**

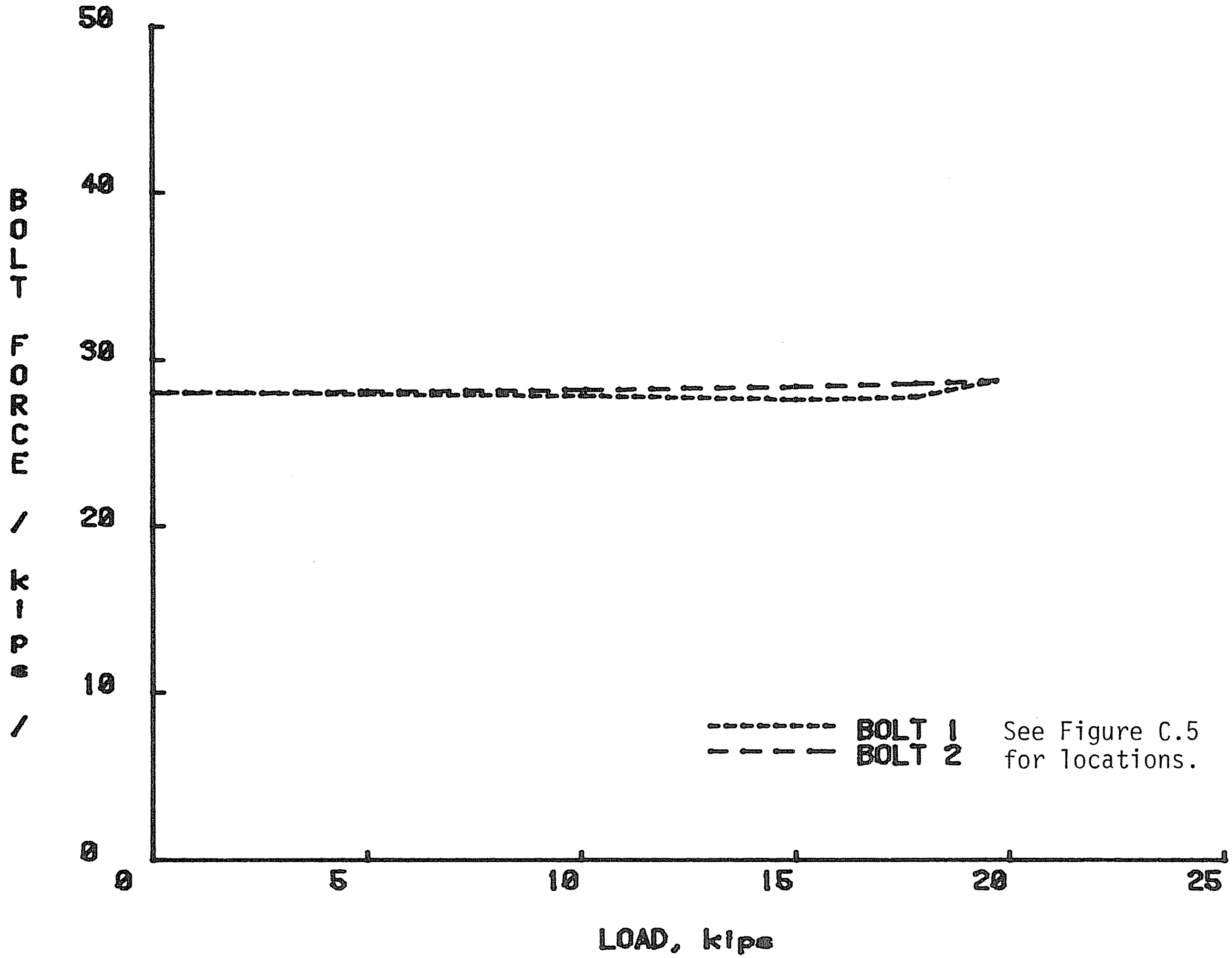


FIGURE C.7 BOLT FORCE VS LOAD, TEST FA-3

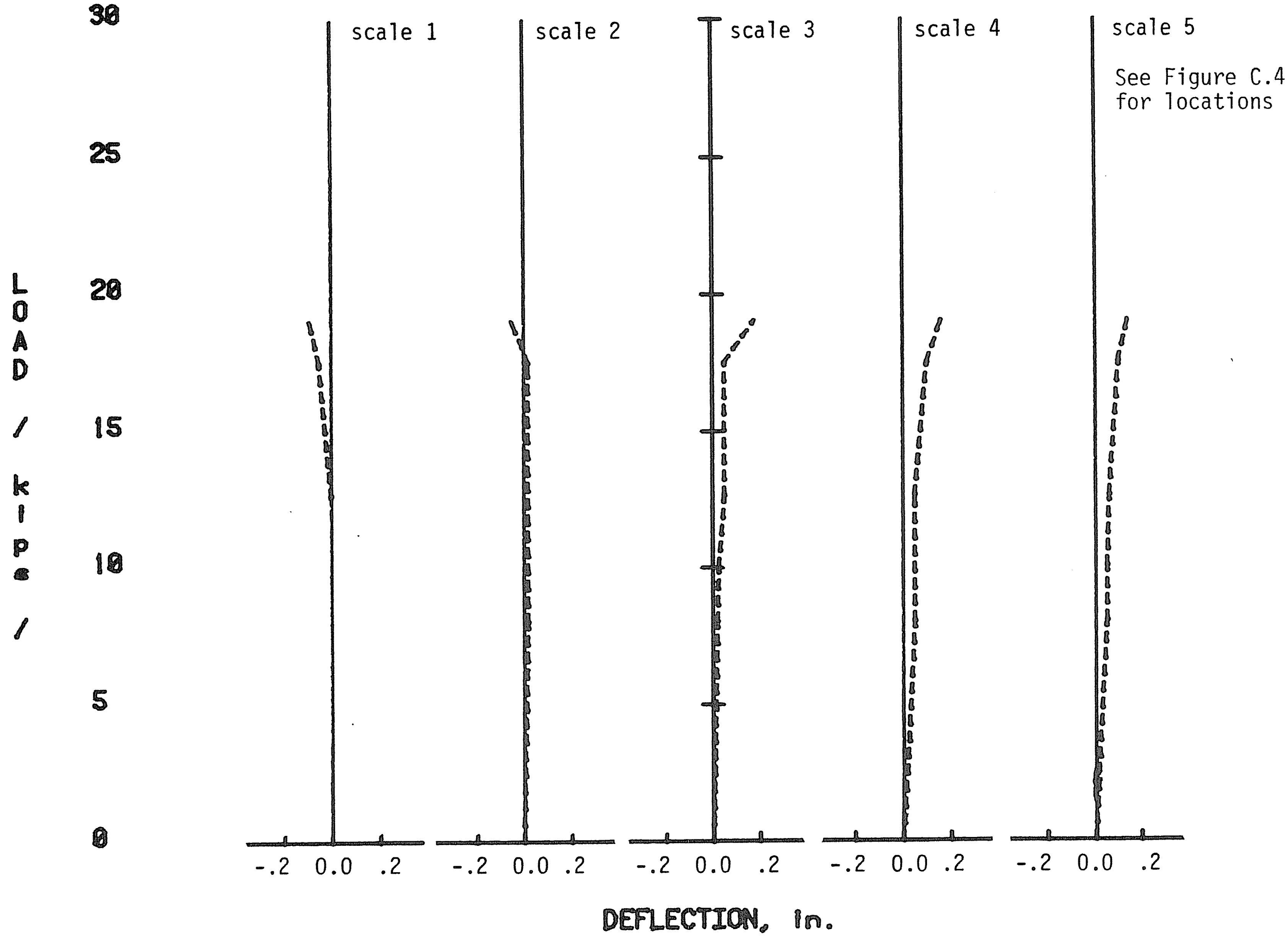


FIGURE C.8 **LOAD VS LATERAL DEFLECTIONS, TEST FA-3**

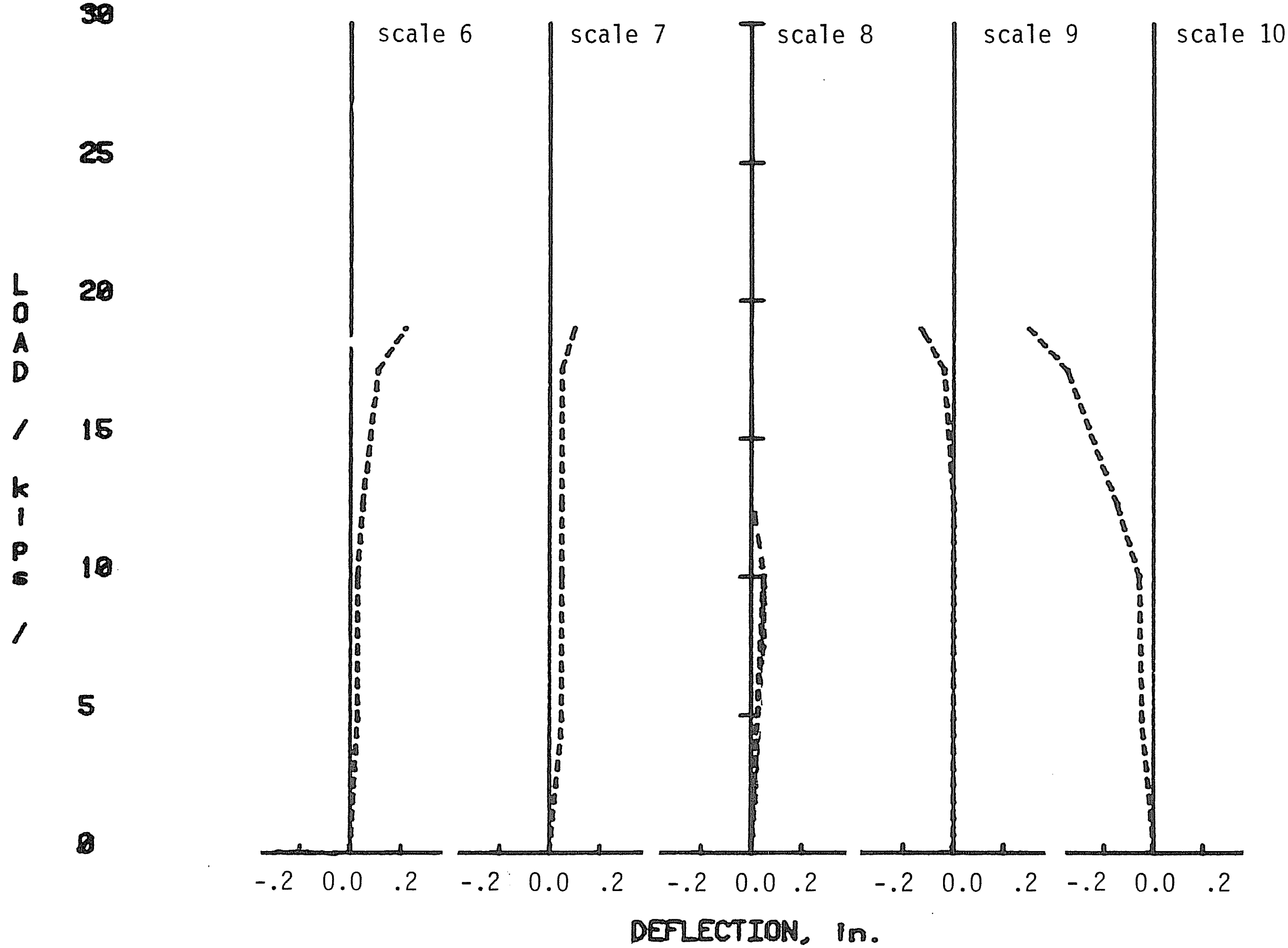


FIGURE C.9 **LOAD VS LATERAL DEFLECTIONS, TEST FA-3**

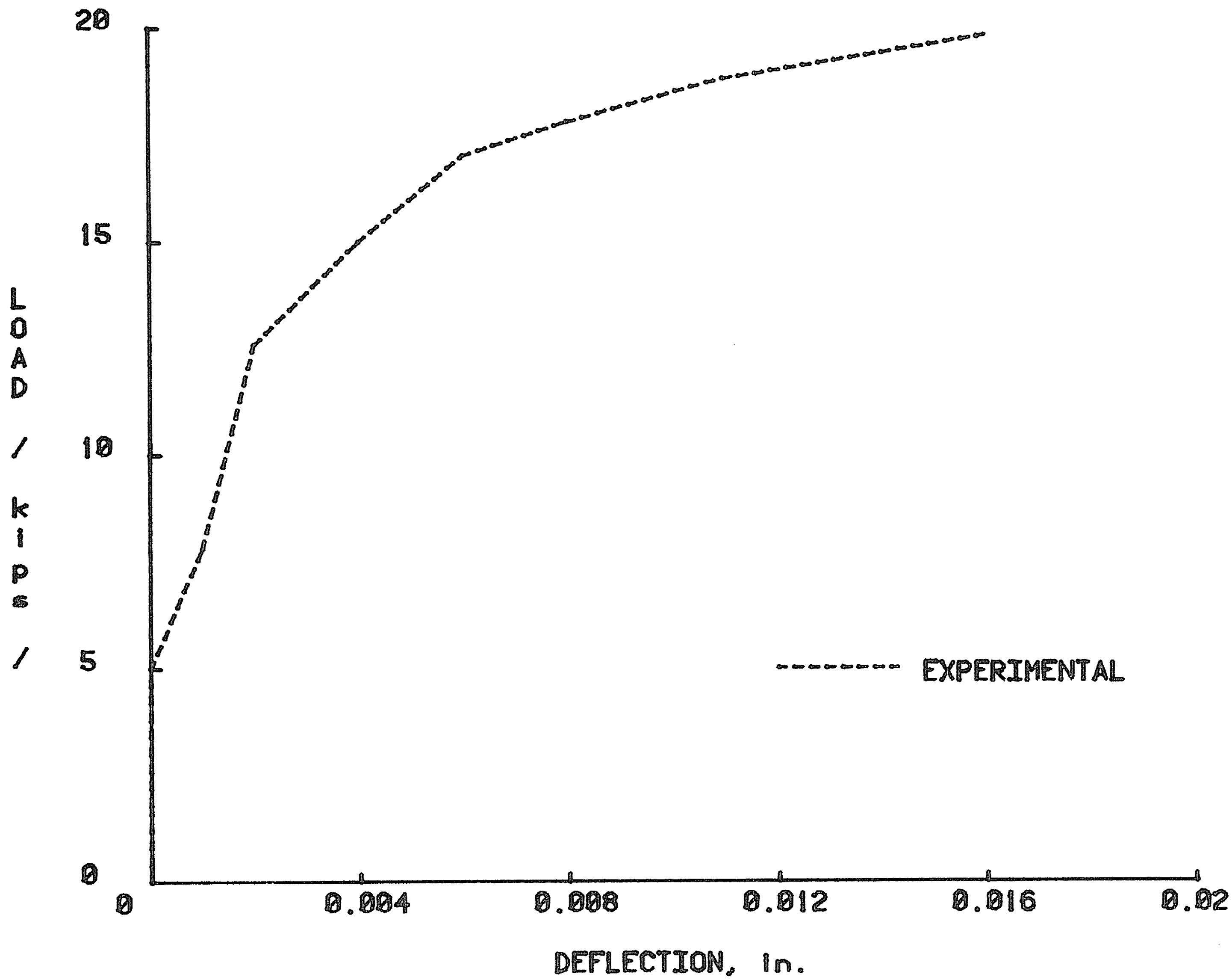


FIGURE C.10 LOAD VS. CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-3

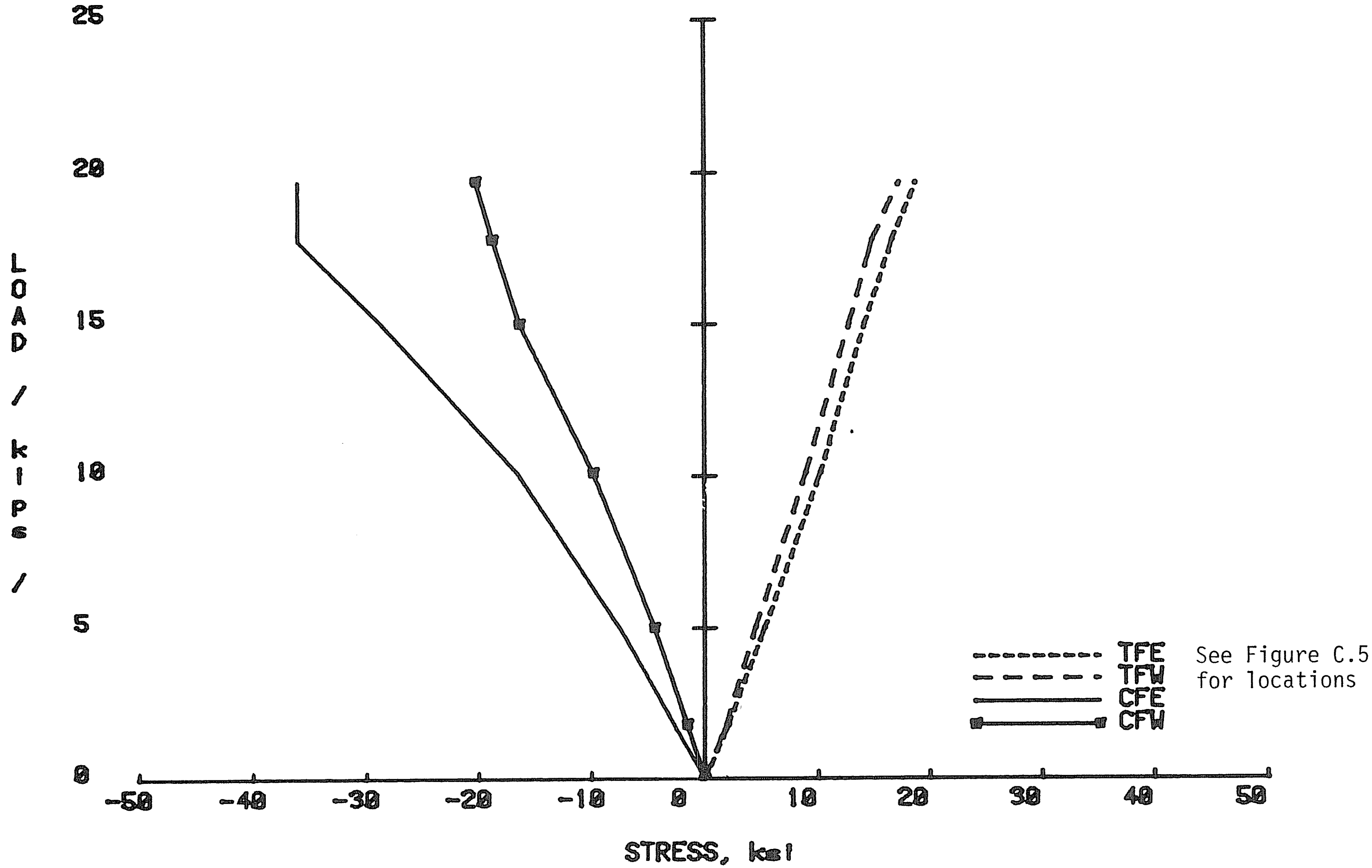


FIGURE C.11 LOAD VS RAFTER FLANGE STRESSES, TEST FA-3

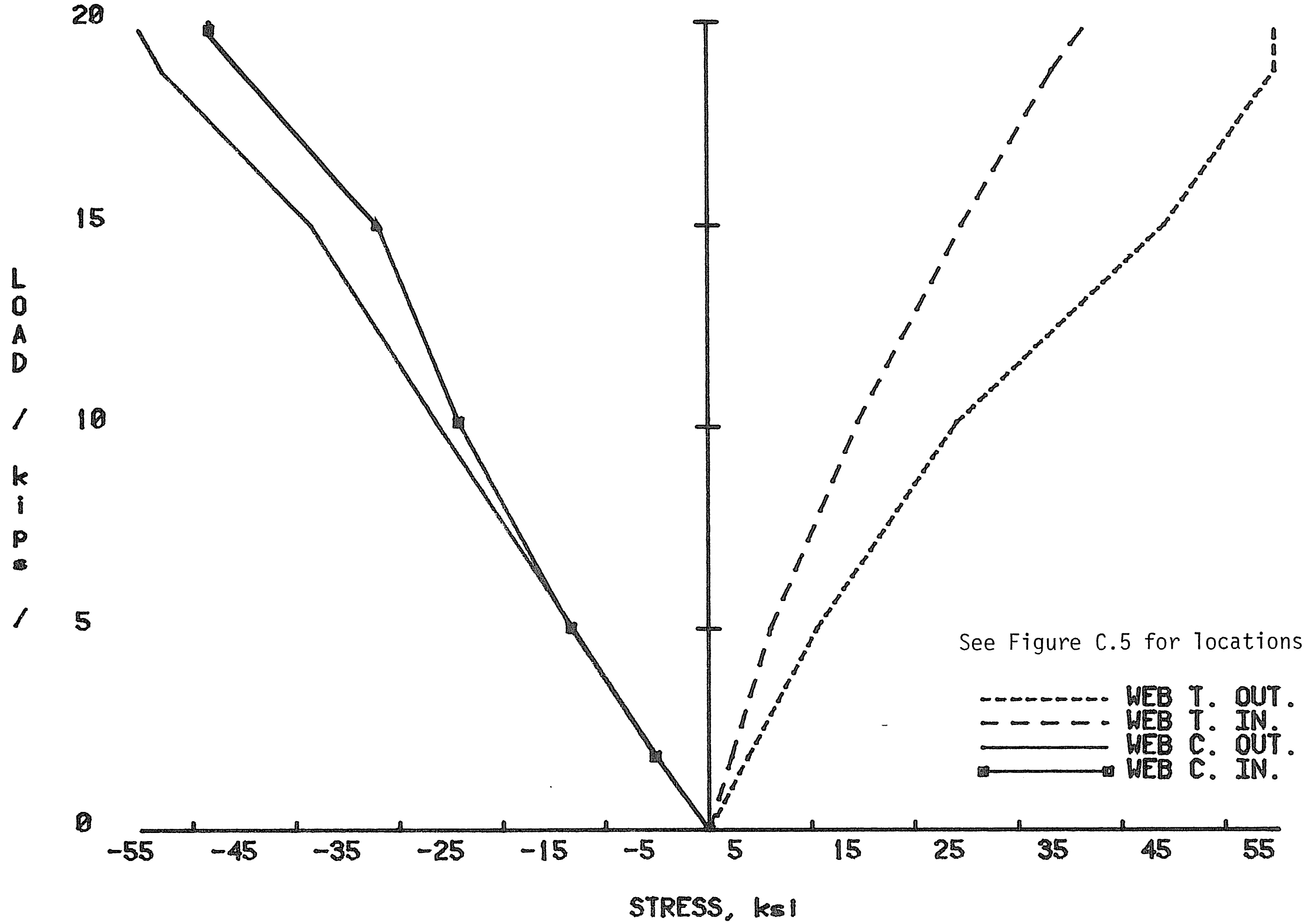


FIGURE C.12 LOAD VS RAFTER WEB STRESSES, TEST FA-3

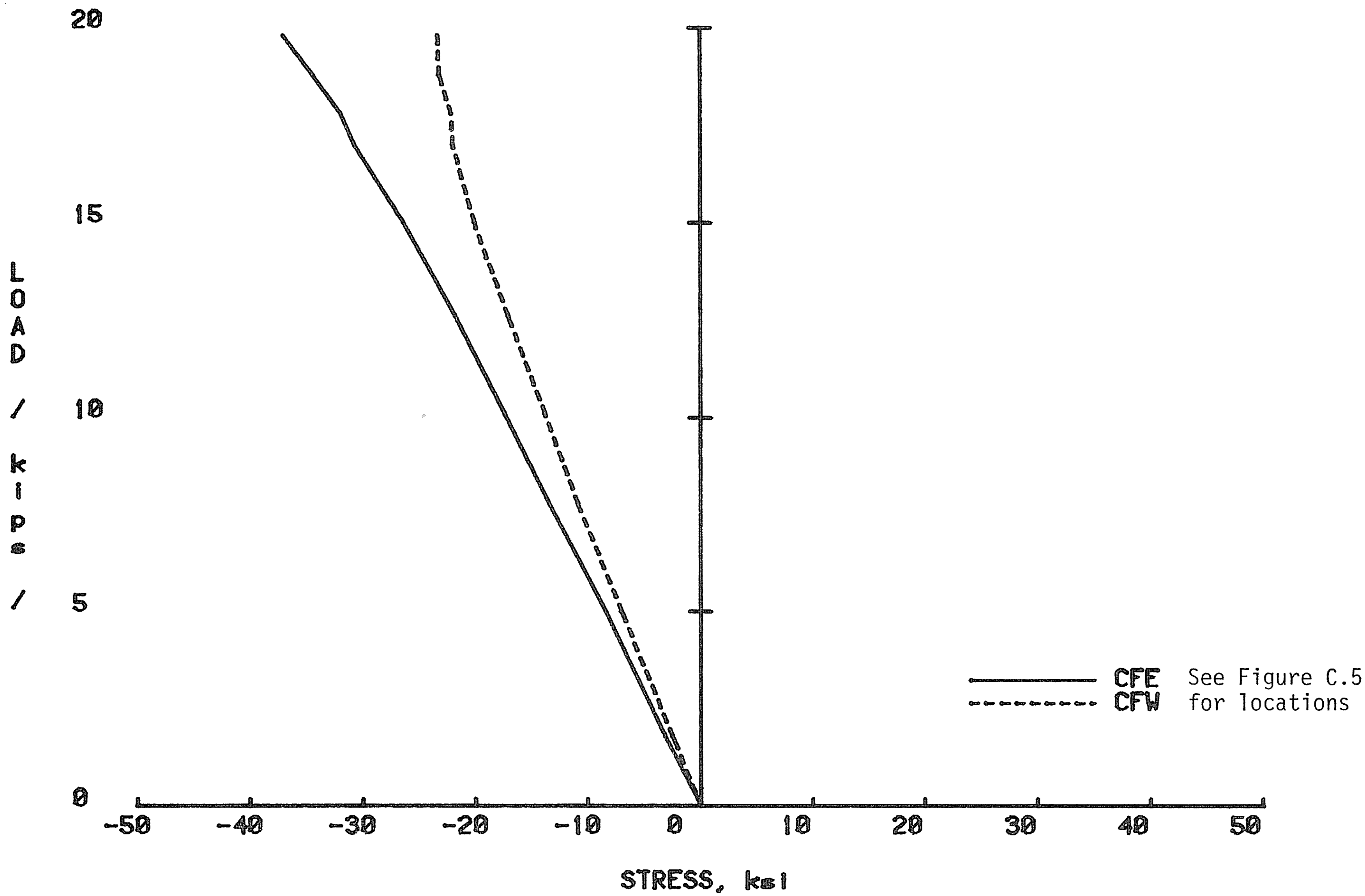


FIGURE C.13 **LOAD VS COLUMN STRESS, TEST FA-3**

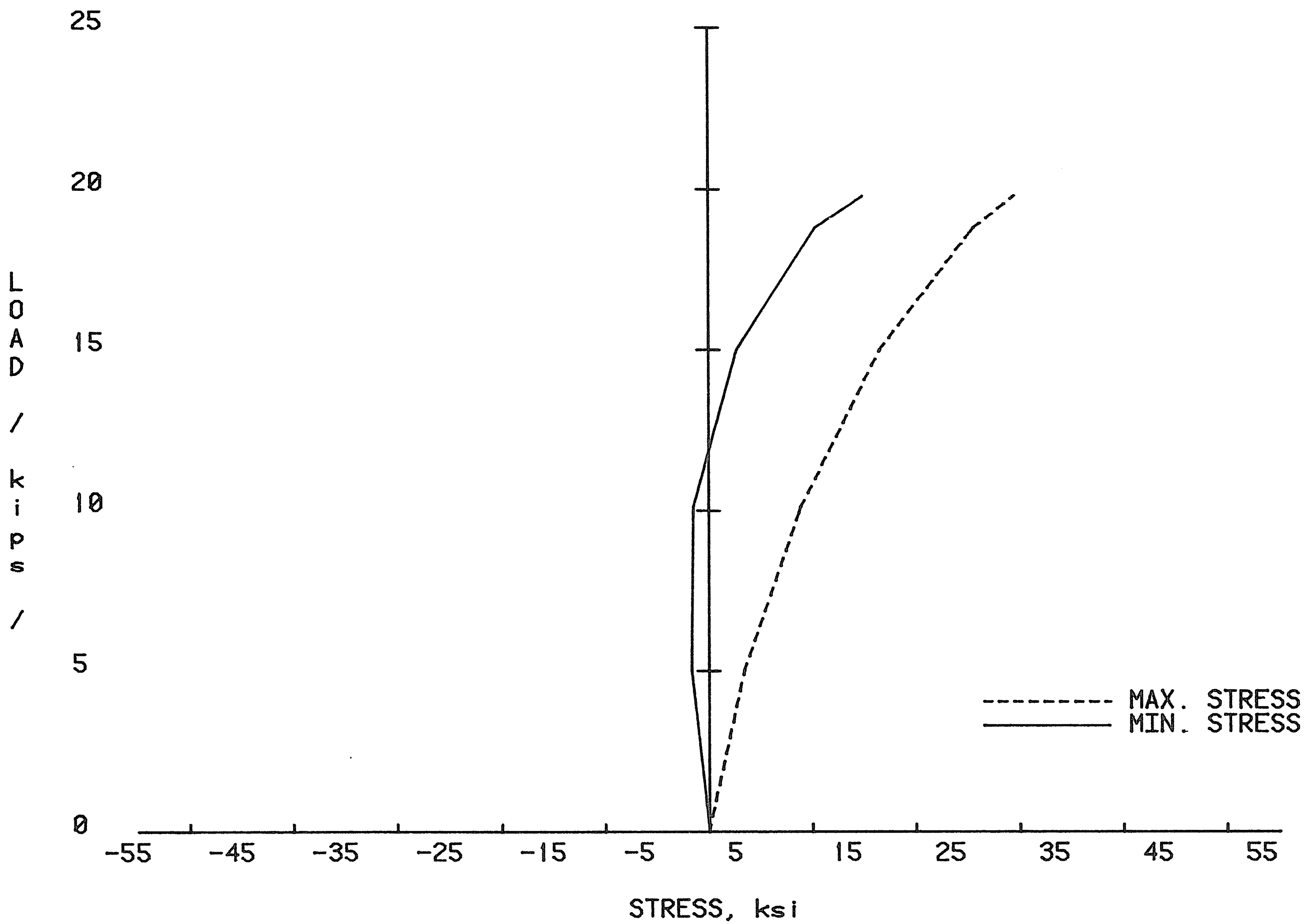


FIGURE C.14 **LOAD VS PRINCIPAL STRESSES, TEST FA-3**

(Measured at Center of Panel Zone)

TEST FA-3

Load, k	Horizontal Stress, ksi	Vertical Stress, ksi	Effective Stress, ksi	Shear Stress, ksi
5.00	0.86	0.84	4.43	2.51
10.10	3.67	3.74	9.79	5.23
15.00	9.21	9.93	15.31	6.89
18.80	17.15	18.77	22.31	7.60
19.80	21.28	23.09	25.51	7.22

Load, k	Principle Stress, ksi		Principle Strain Micro Strain		Theta, degrees
5.00	3.4	-1.7	133.	-92.	44.9
10.10	8.9	-1.5	324.	-145.	-44.8
15.00	16.5	2.7	540.	-78.	-43.5
18.80	25.6	10.3	776.	91.	-41.9
19.80	29.5	14.9	862.	209.	-41.4

Table C.1 Stress and Strain at Center of Panel Zone

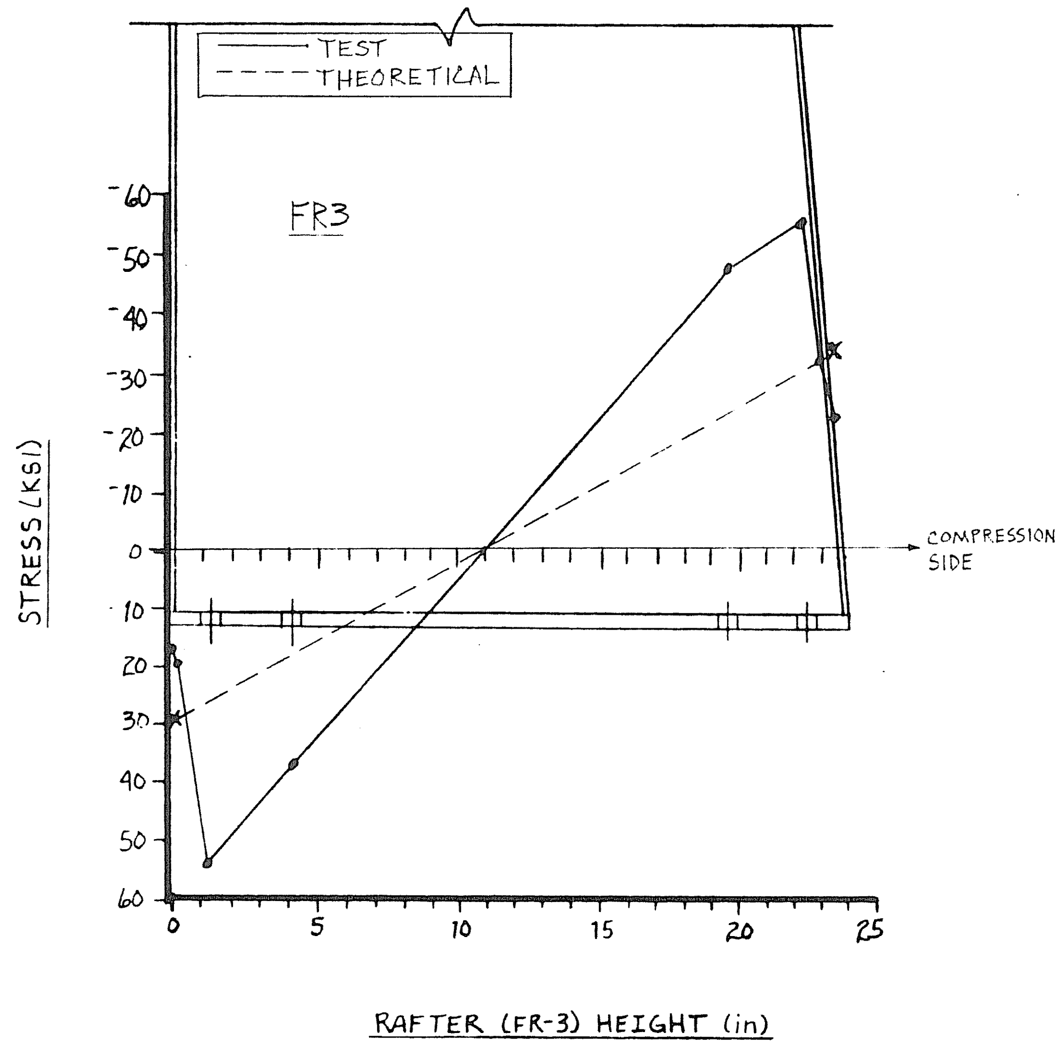


FIGURE C.15 STRESS VARIATION ACROSS RAFTER AT WORKING LOAD, TEST FA-3

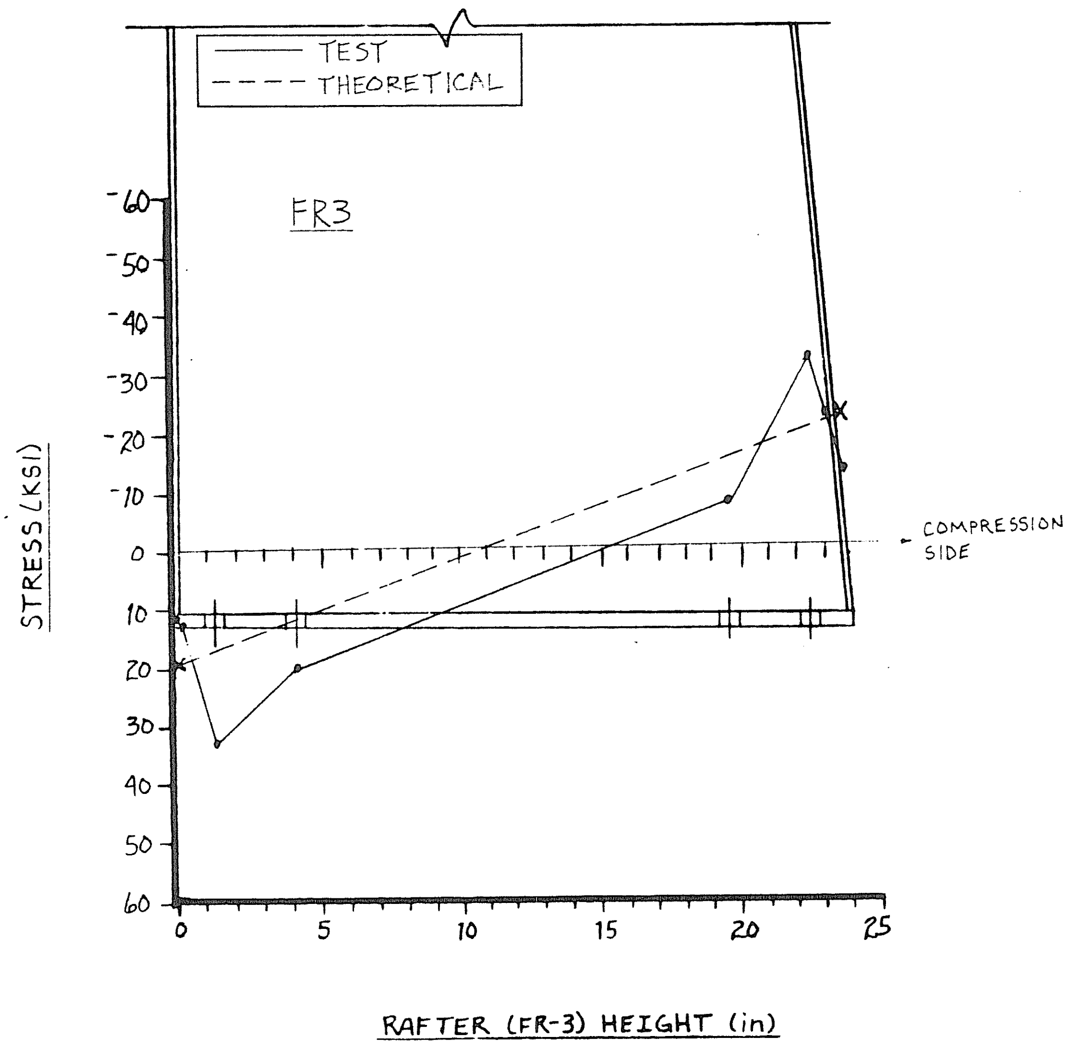


FIGURE C.16 STRESS VARIATION ACROSS RAFTER AT FAILURE LOAD, TEST FA-3

APPENDIX D

FA-4 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: MESCO Knee Test

Test No.: FA-4

Test Date: 6/11/84

Purpose: Test of Knee Area

Number of Tension Bolts: 4 **Bolt Gage (g):** 3" **Pitch:** 3"

Bolt Diameter: 3/4" **End Plate Thickness (t):** .510"

End Plate Width (w): 6" **End Plate Length (de):** 35.5"

Panel Zone Web Plate Thickness: .218"

Initial Out-of-Straightness at the Center of Panel Zone: .312"

Pretension Force per Bolt: 38.0 kips

Failure Load, (Total Load): 32.1

Failure Mode: Buckling of panel zone and severe yielding of rafter flanges.

Predicted Failure Loads:

Method	Total Load
Bending + axial in rafter	44.46 kips
Panel Buckling (no TFA)	23.75 kips
Panel Buckling (TFA)	50.04 kips

Discussion:

-At load equal to 25 kips, some minor yielding was observed on compression flange of rafter ar re-entrant corner.

-At 30 kips load, yielding of compression flange of rafter increased and new yield lines could be seen on tension flange of rafter.

-At 31.4 kips, severe yield lines occurred on panel zone in 45^{o} angles indicating buckling of panel zone.

-At 31.8 kips, widespread yielding could be seen on compression flange of rafter as well as the column. Maximum load that could be applied to specimen was 32.1 kips.

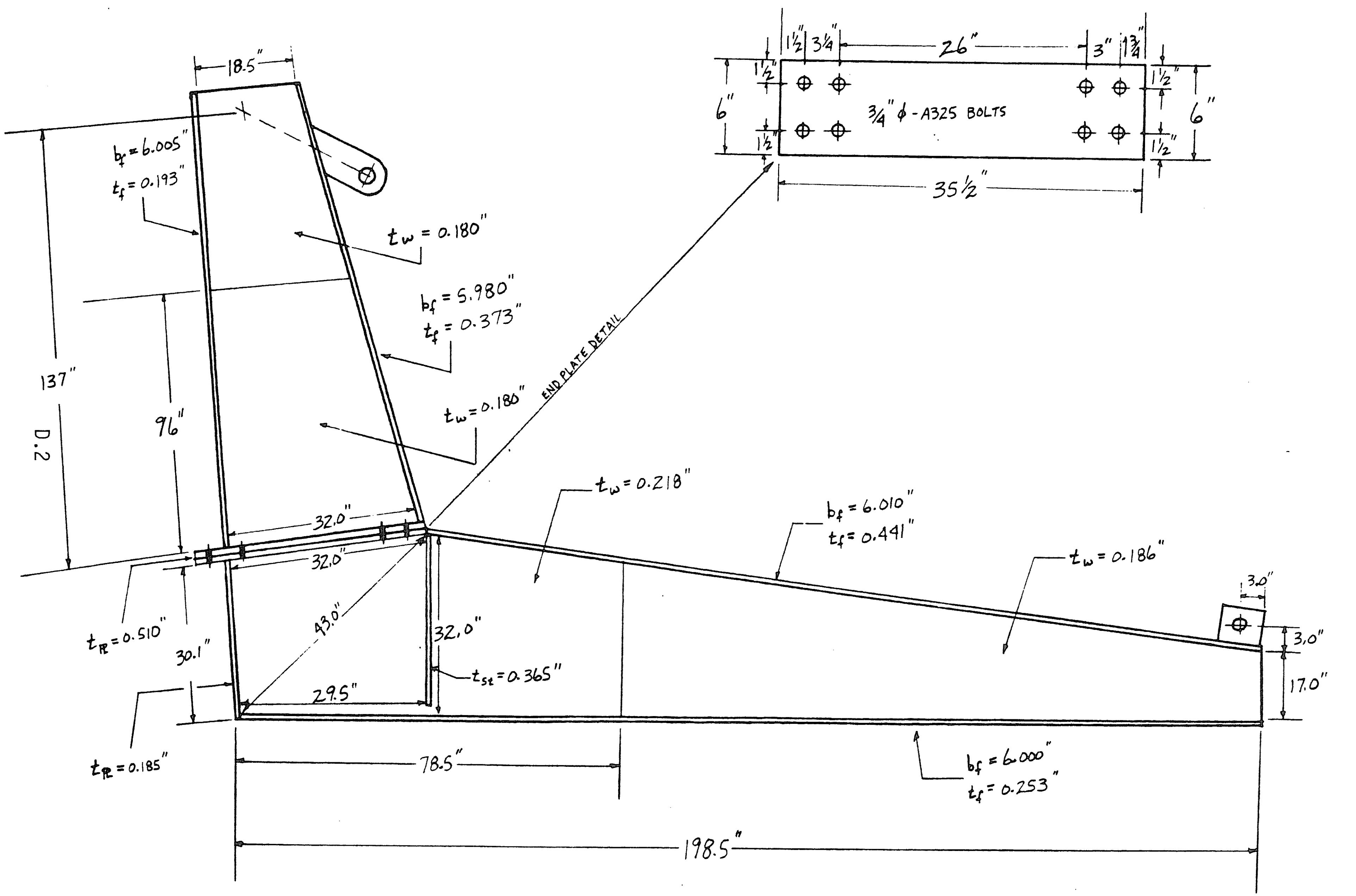


FIGURE D.1 SPECIMEN DETAILS, TEST FA-4

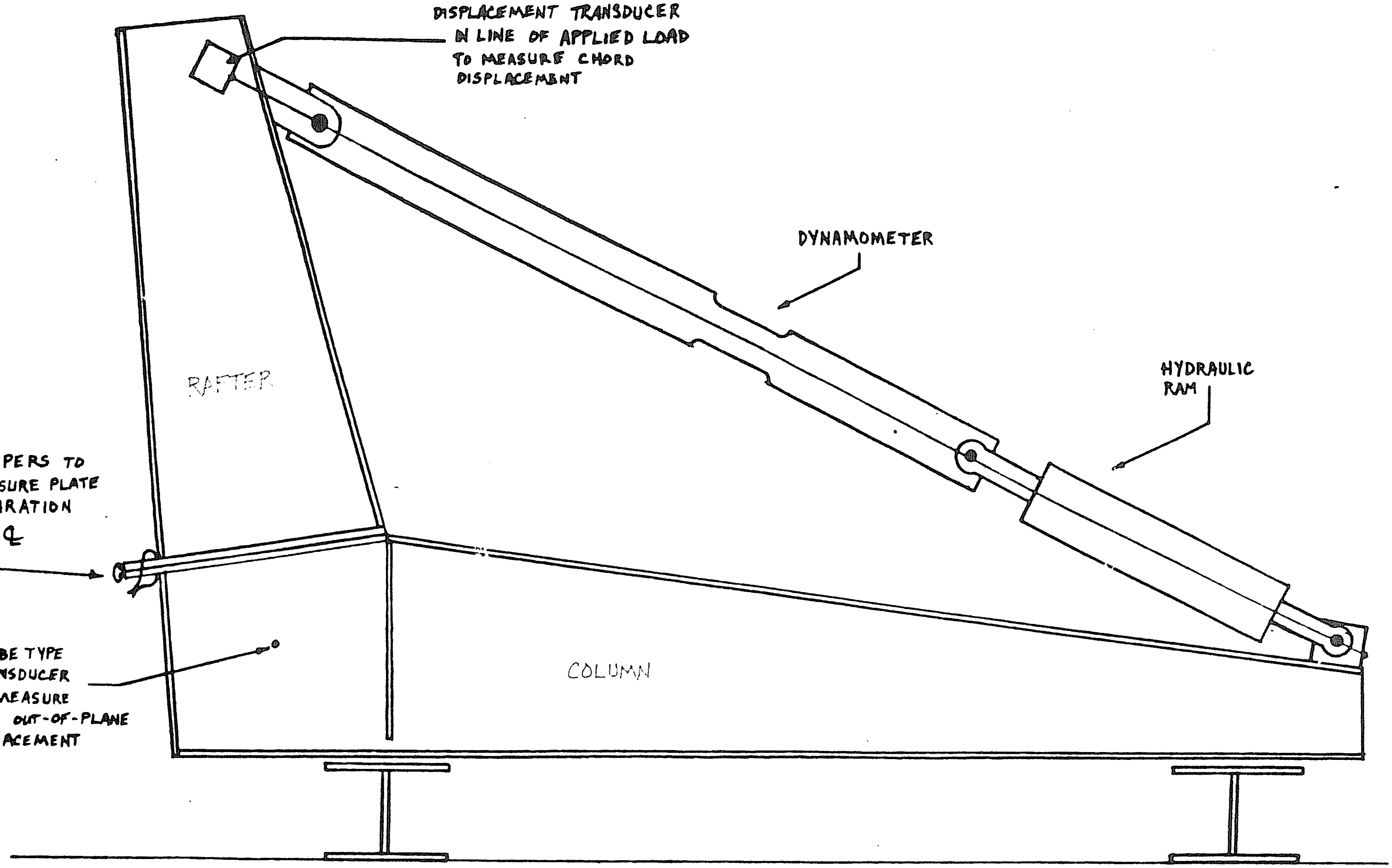


FIGURE D.2 TEST SETUP, TEST FA-4

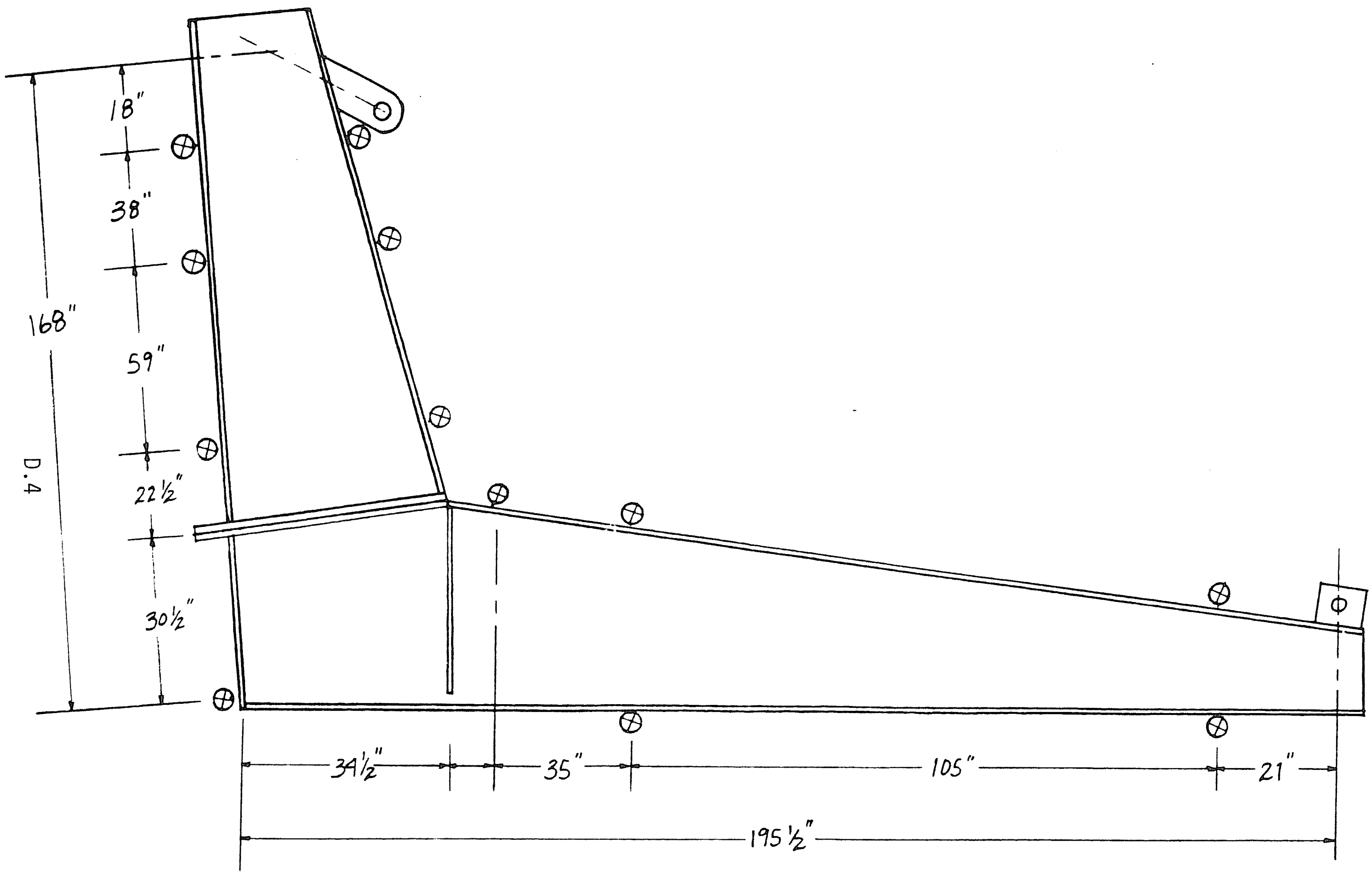


FIGURE D.3 LATERAL BRACE LOCATIONS, TEST FA-4

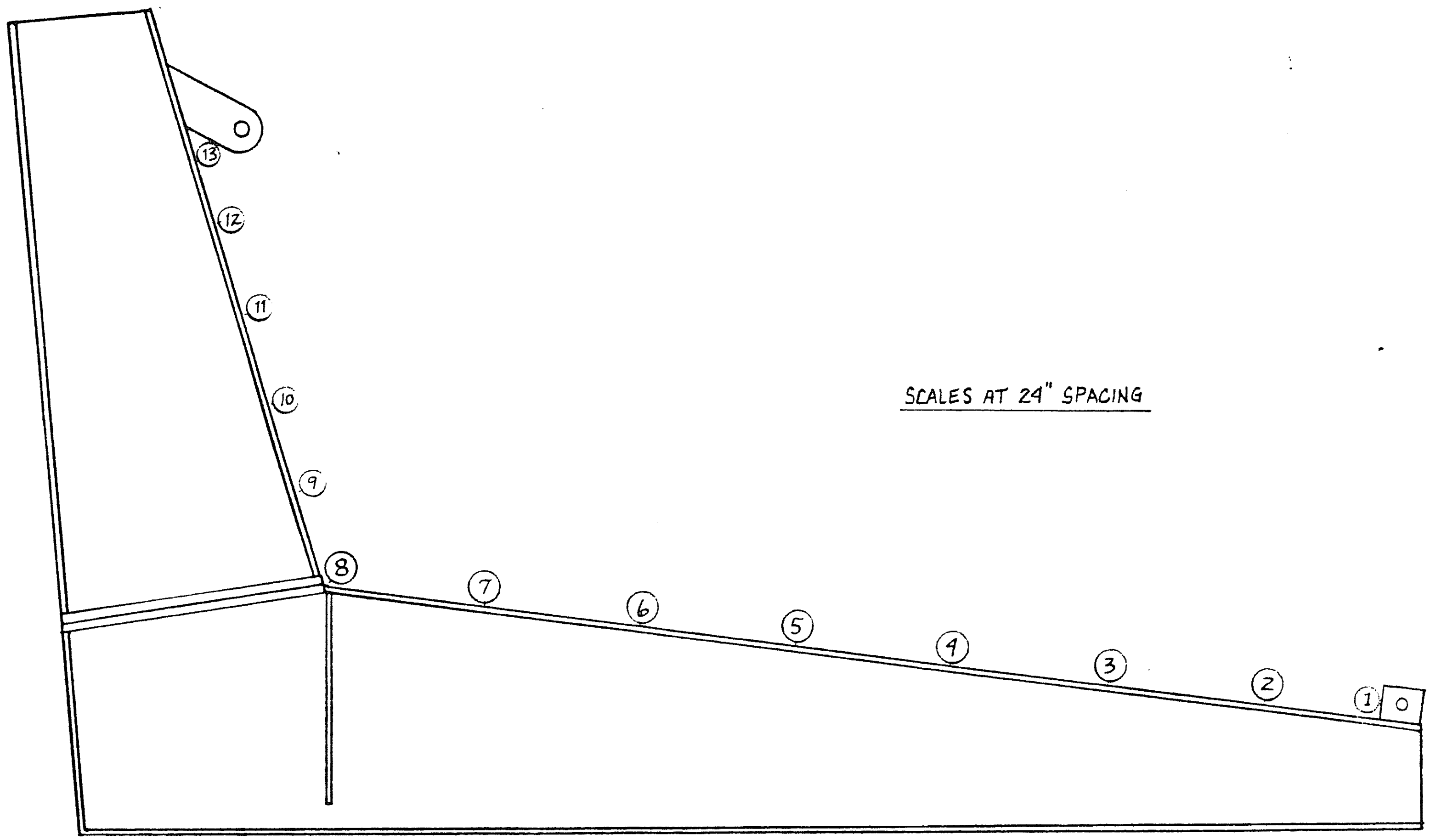


FIGURE D.4 LATERAL DISPLACEMENT SCALE LOCATIONS, TEST FA-4

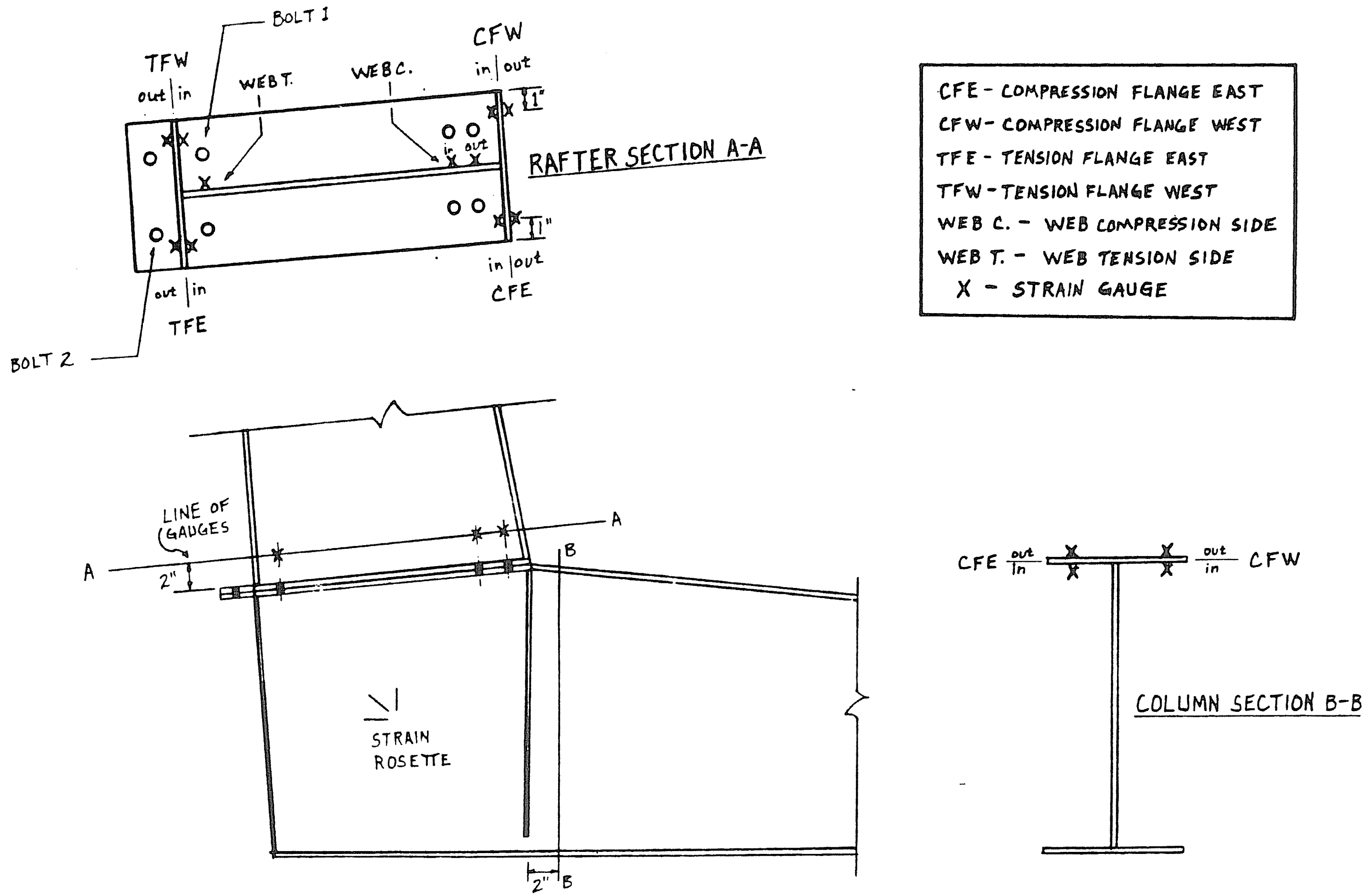


FIGURE D.5 STRAIN GAGE LOCATIONS, TEST FA-4

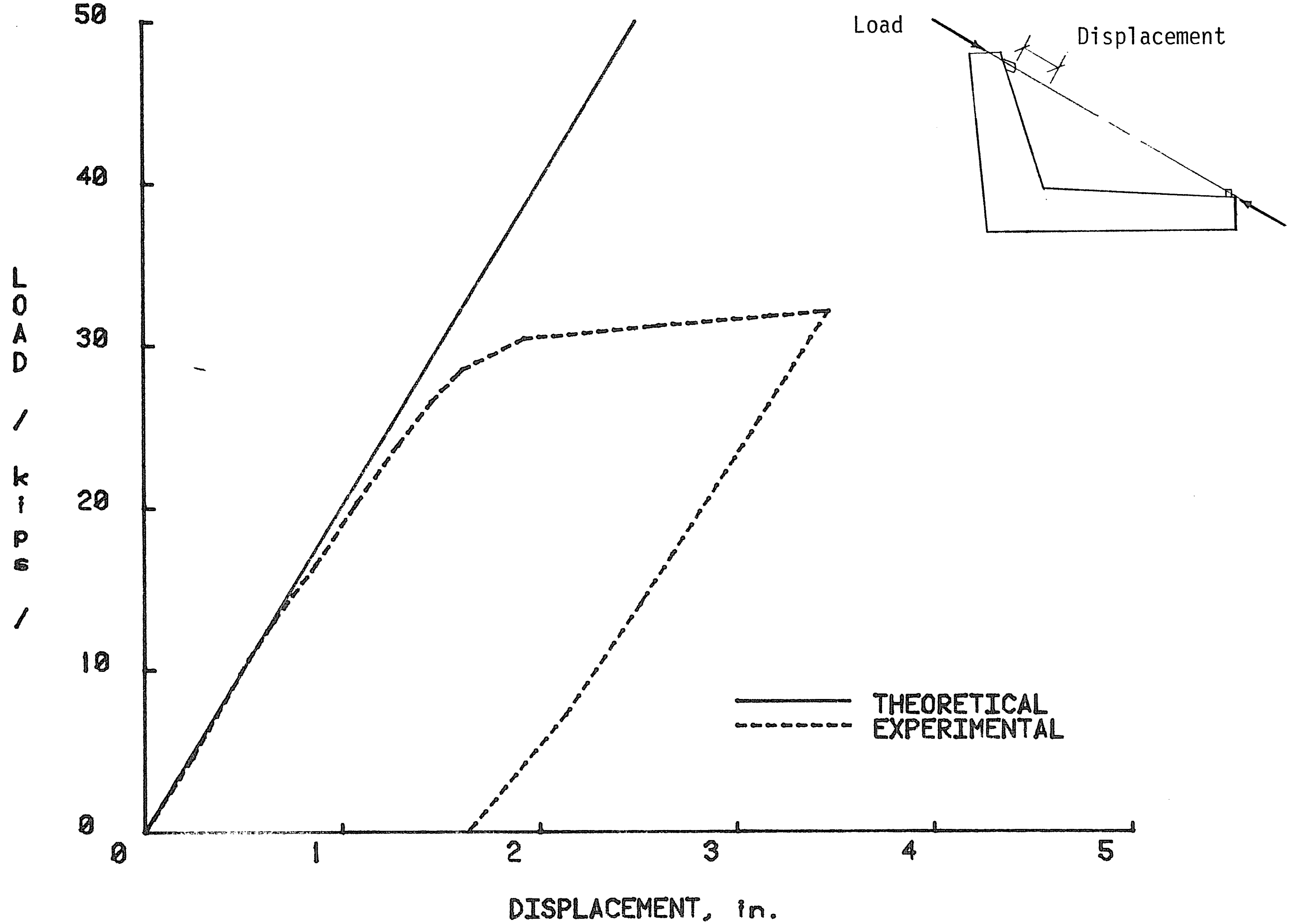


FIGURE D.6 **LOAD VS CHORD DISPLACEMENT, TEST FA-4**

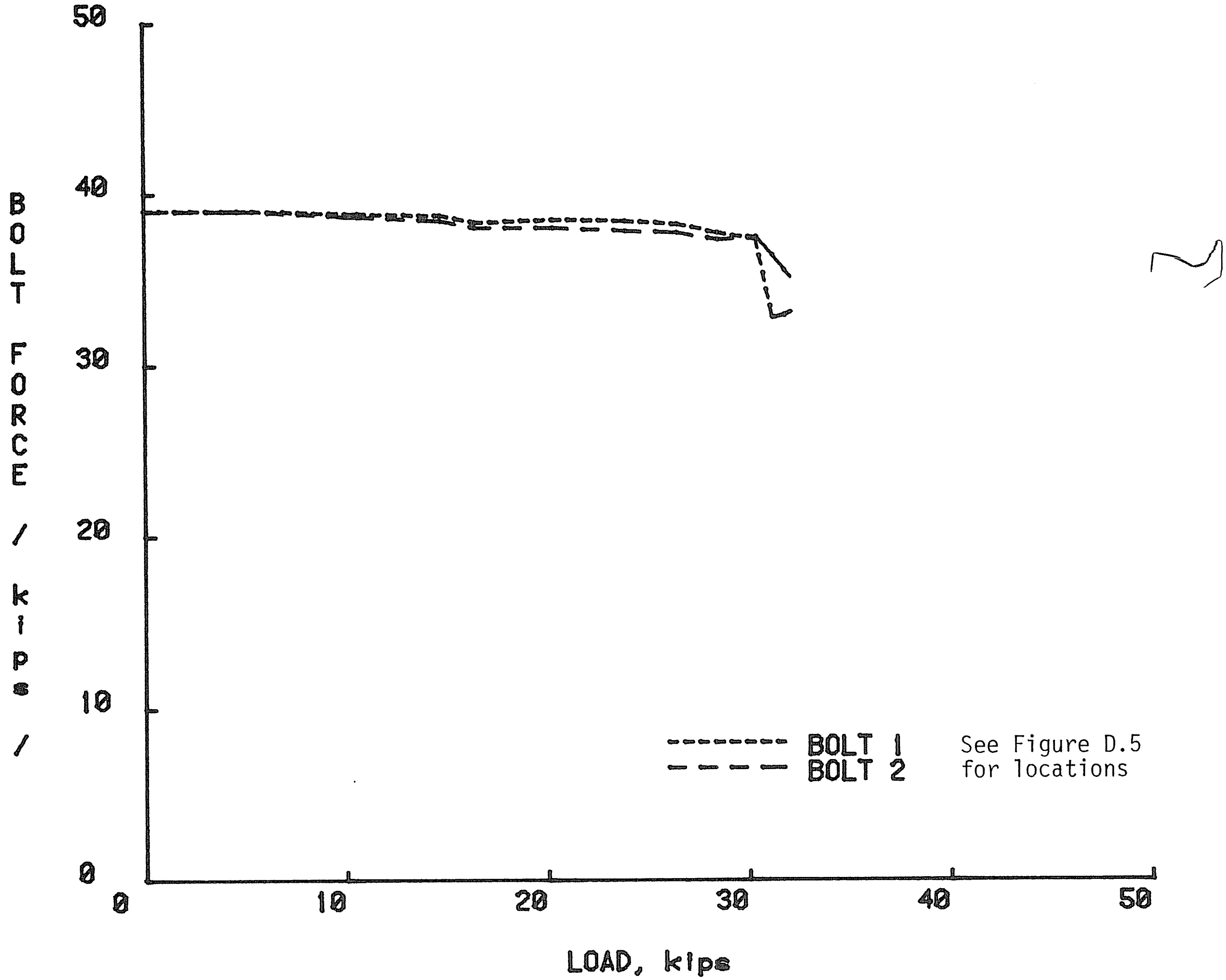


FIGURE D.7 **BOLT FORCE VS LOAD, TEST FA-4**

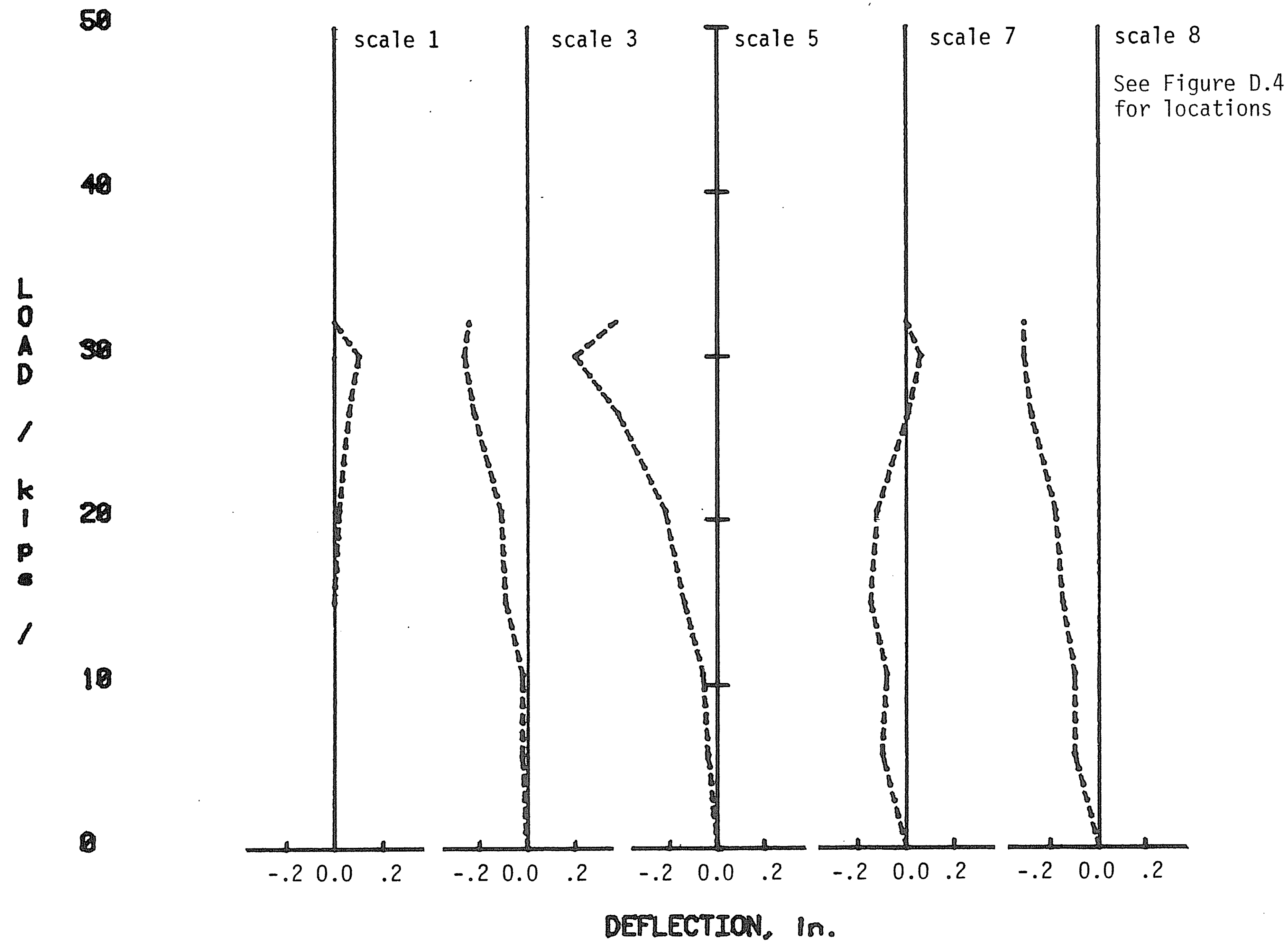


FIGURE D.8 **LOAD VS LATERAL DEFLECTIONS, TEST FA-4**

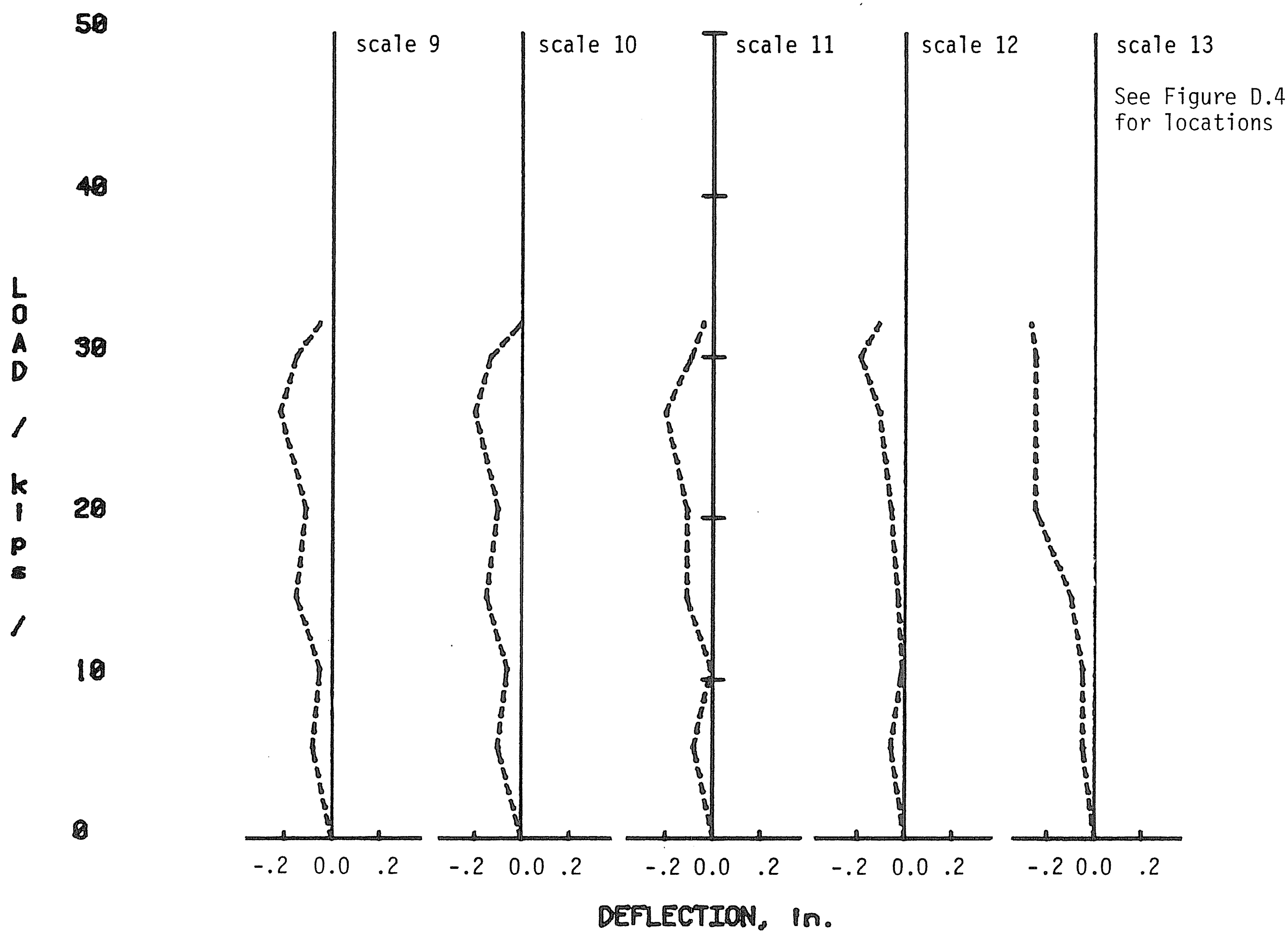


FIGURE D.9 **LOAD VS LATERAL DEFLECTIONS, TEST FA-4**

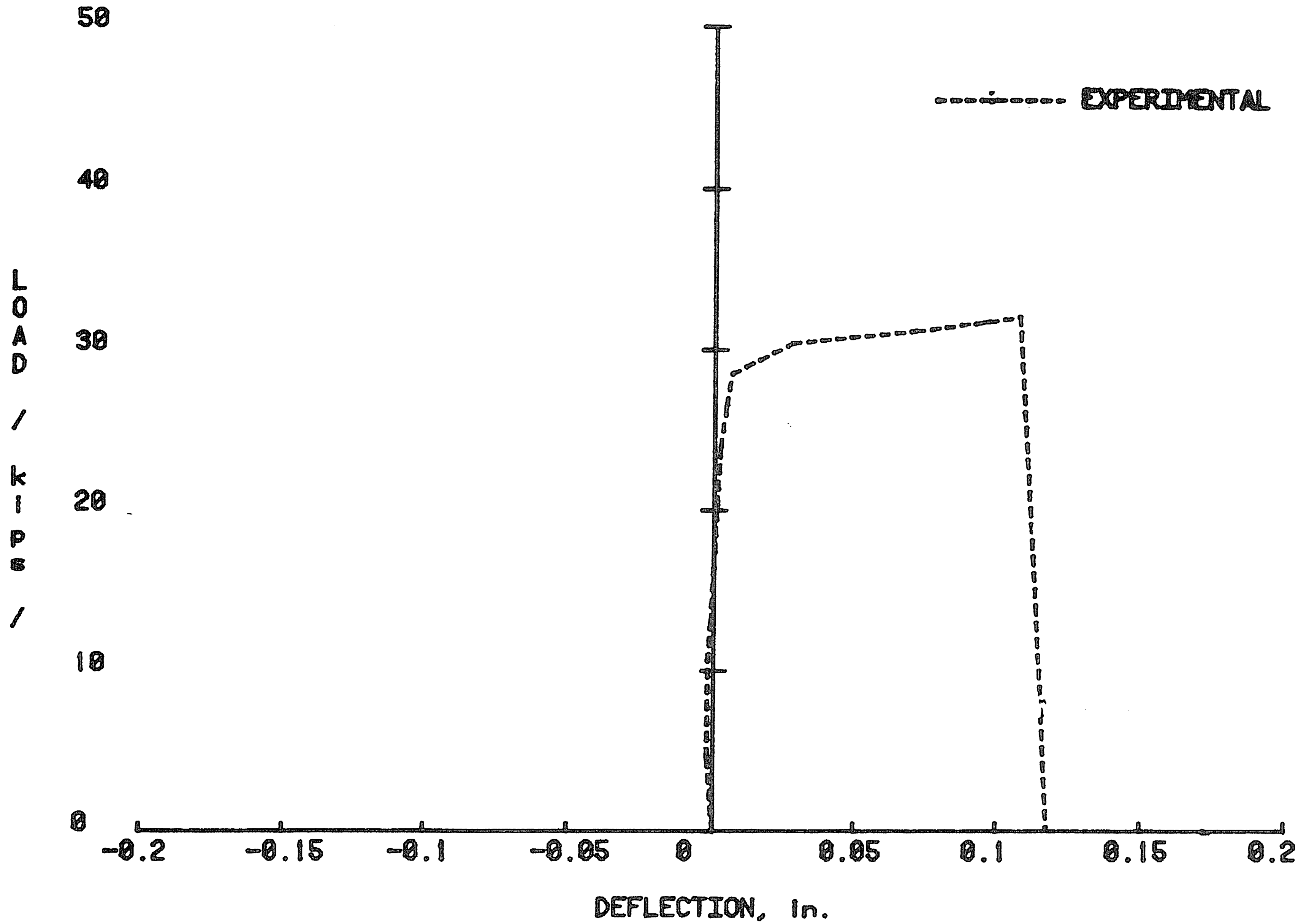


FIGURE D.10 LOAD VS. CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-4

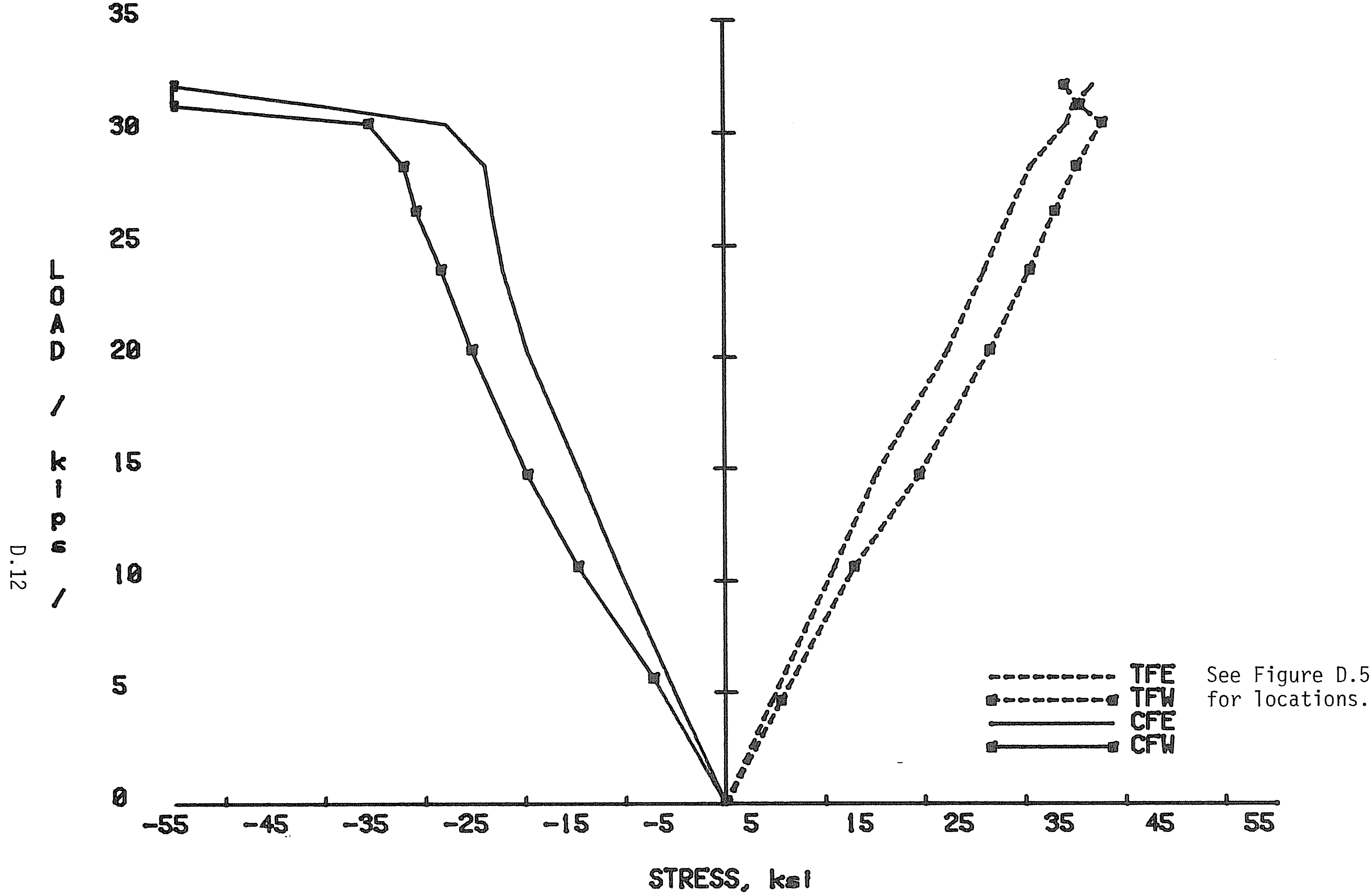


FIGURE D.11 **LOAD VS RAFTER STRESS, TEST FA-4**

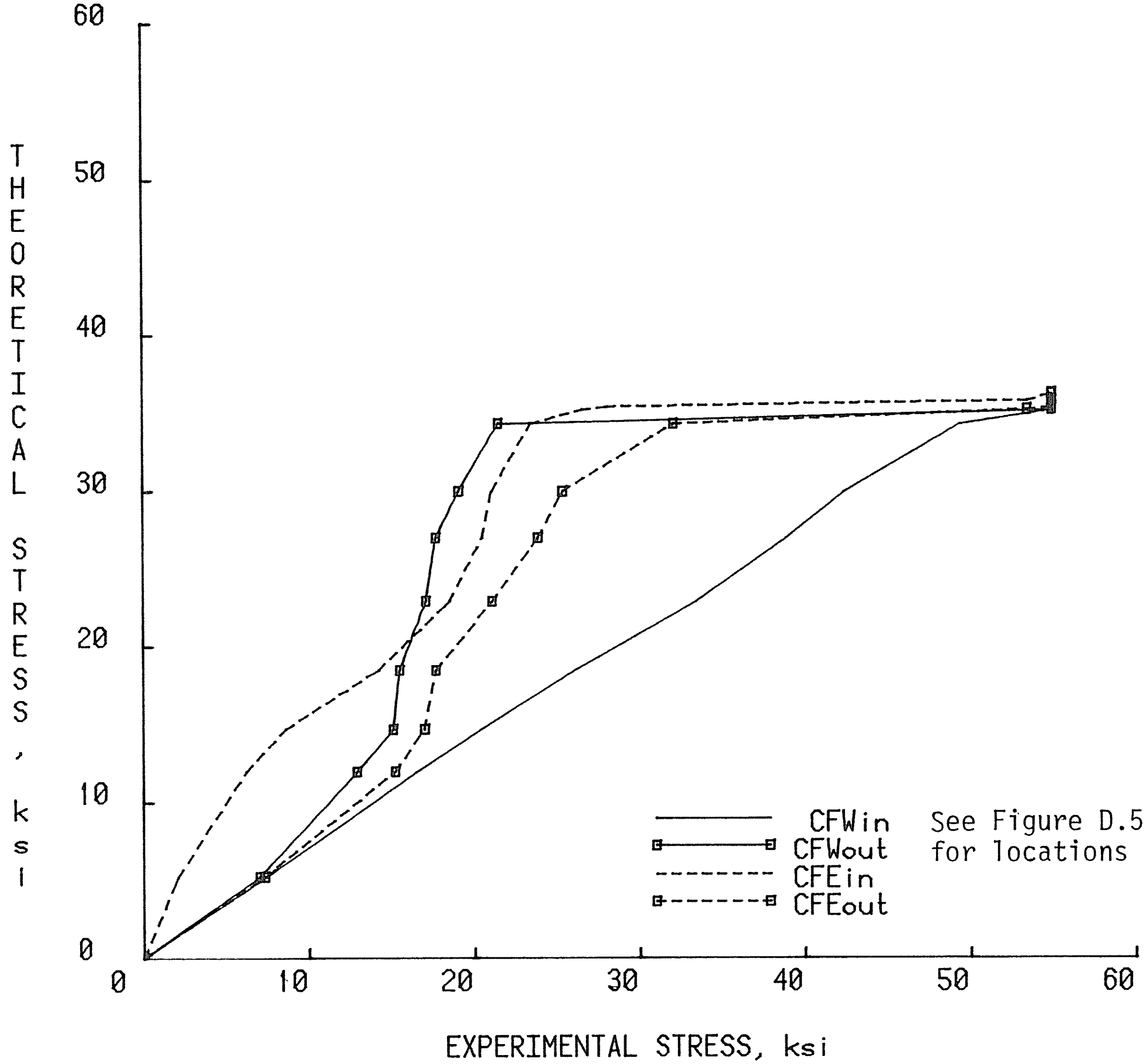


FIGURE D.12 THEORETICAL VS EXPERIMENTAL RAFTER STRESS, TEST FA-4

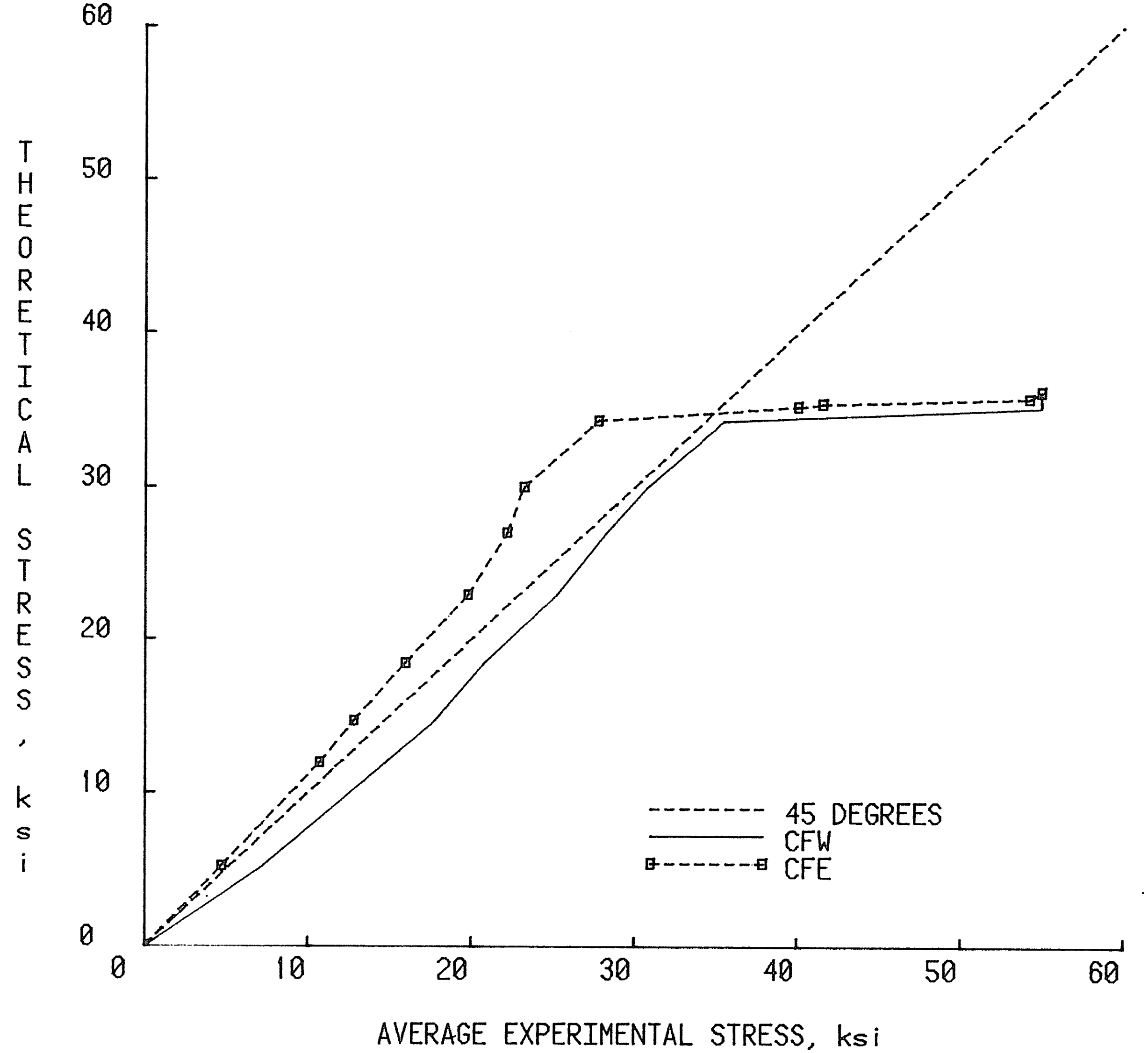


FIGURE D.13 THEORETICAL VS AVERAGE EXPERIMENTAL RAFTER FLANGE STRESS, FA-4

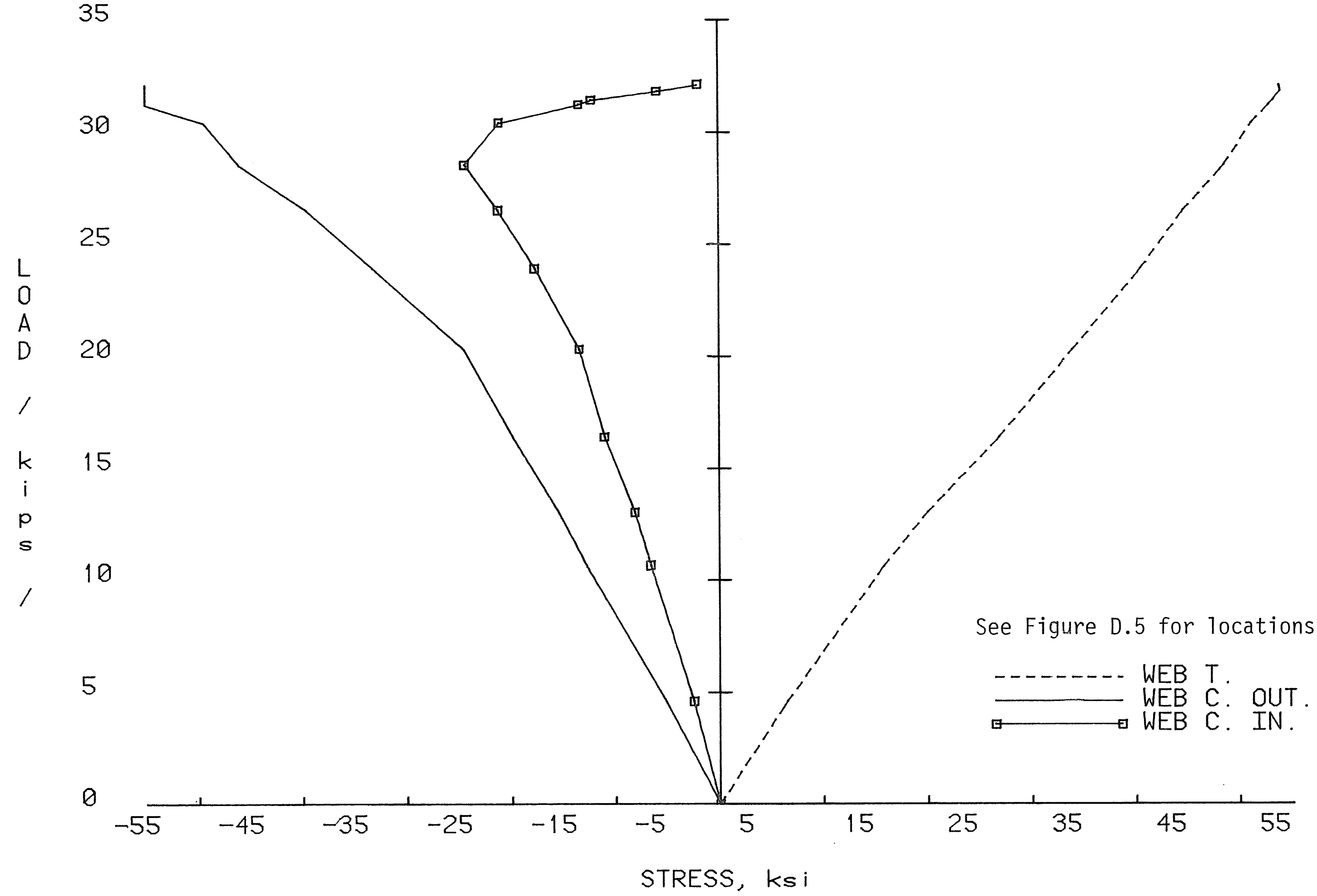


FIGURE D.14 LOAD VS RAFTER WEB STRESS, TEST FA-4

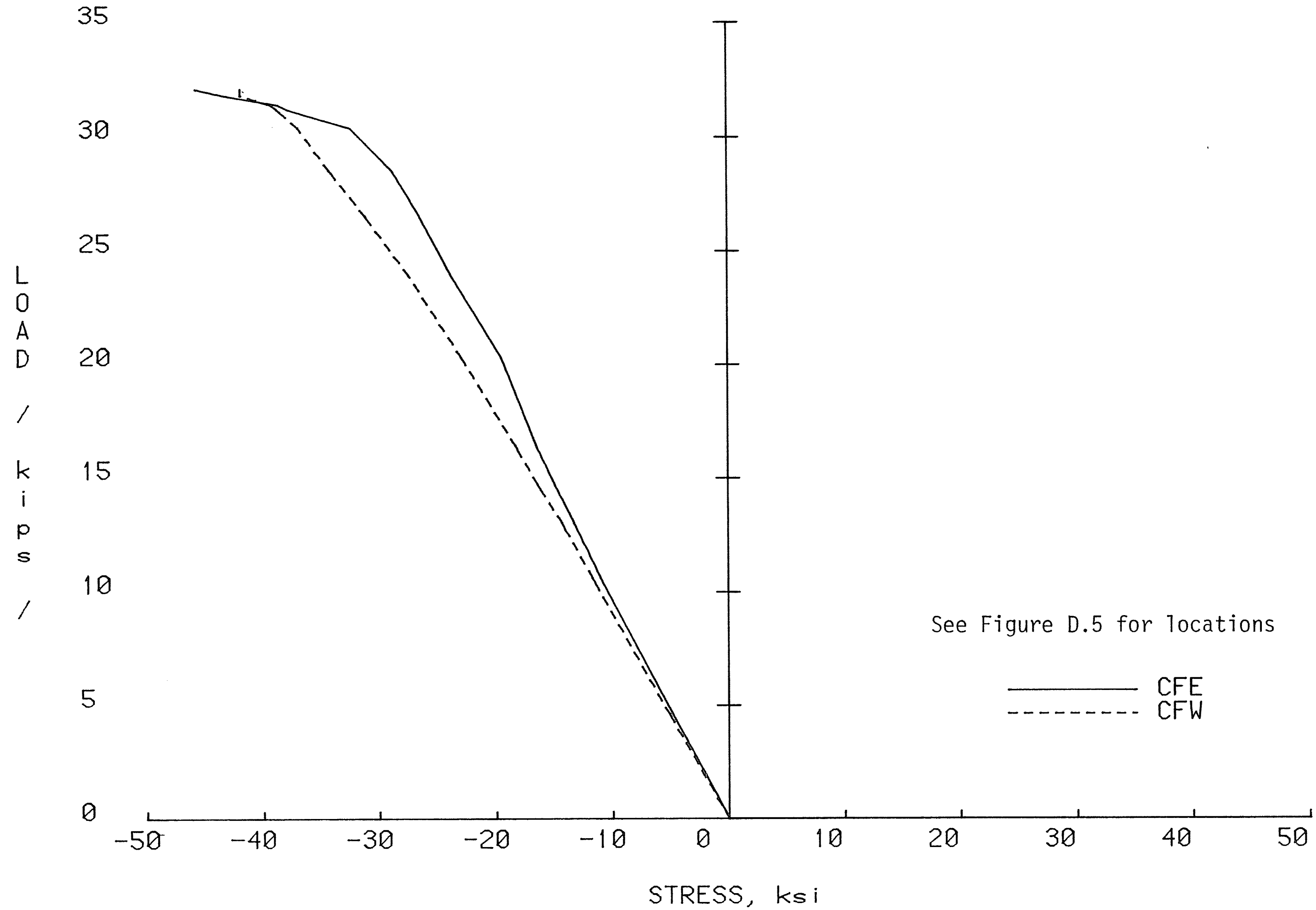


FIGURE D.15 LOAD VS COLUMN FLANGE STRESS, TEST FA-4

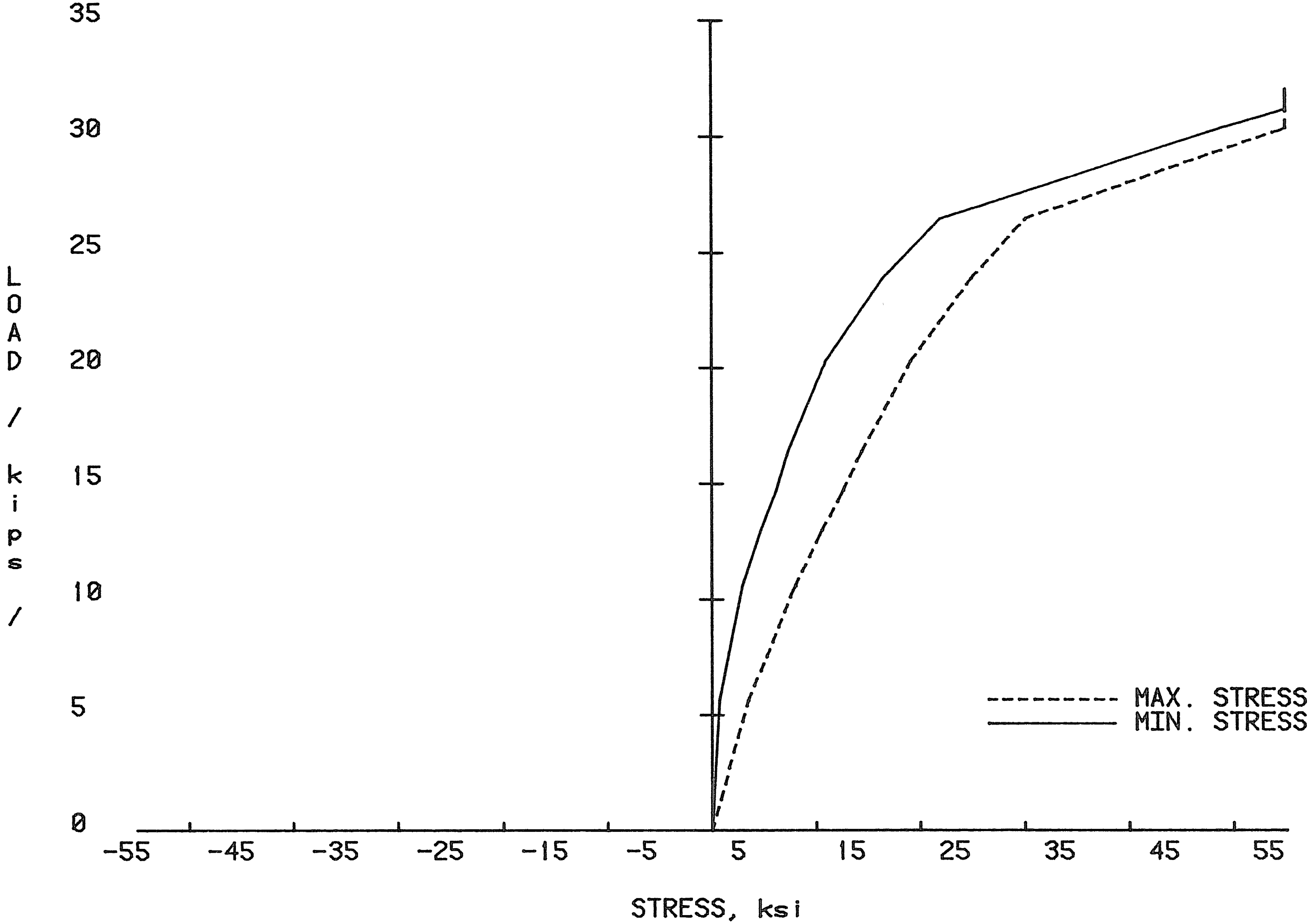


FIGURE D.16 **LOAD VS PRINCIPAL STRESSES, TEST FA-4**

(Measured at Center of Panel Zone)

TEST FA-4

Load, k	Horizontal Stress, ksi	Vertical Stress, ksi	Effective Stress, ksi	Shear Stress, ksi
5.60	2.38	1.85	3.17	1.34
10.60	5.93	4.97	6.97	2.47
13.00	8.20	7.13	9.21	2.90
14.70	9.97	8.79	10.93	3.18
16.40	11.55	10.16	12.47	3.48
20.30	15.72	14.32	16.64	4.07
23.90	21.34	20.00	21.93	4.17
26.50	26.50	25.49	26.93	4.03
30.40	54.32	52.60	54.06	4.54
31.20	105.63	112.57	111.98	14.15
31.80	143.58	156.90	157.60	26.67
32.10	162.97	178.98	181.44	34.13

Load, k	Principle Stress, ksi		Principle Strain, Micro Strain		Theta, degrees
5.60	3.5	0.7	112.	-10.	39.3
10.60	8.0	2.9	244.	19.	39.5
13.00	10.6	4.7	317.	53.	39.8
14.70	12.6	6.2	371.	82.	39.7
16.40	14.4	7.3	421.	103.	39.4
20.30	19.1	10.9	548.	177.	40.1
23.90	24.9	16.4	688.	310.	40.4
26.50	30.1	21.9	809.	446.	41.4
30.40	58.1	48.8	1498.	1083.	39.6
31.20	123.7	94.5	3287.	1980.	-38.1
31.80	177.7	122.8	4859.	2394.	-38.0
32.10	206.0	135.9	5699.	2555.	-38.4

Table D.1 Stress and Strain at Center of Panel Zone

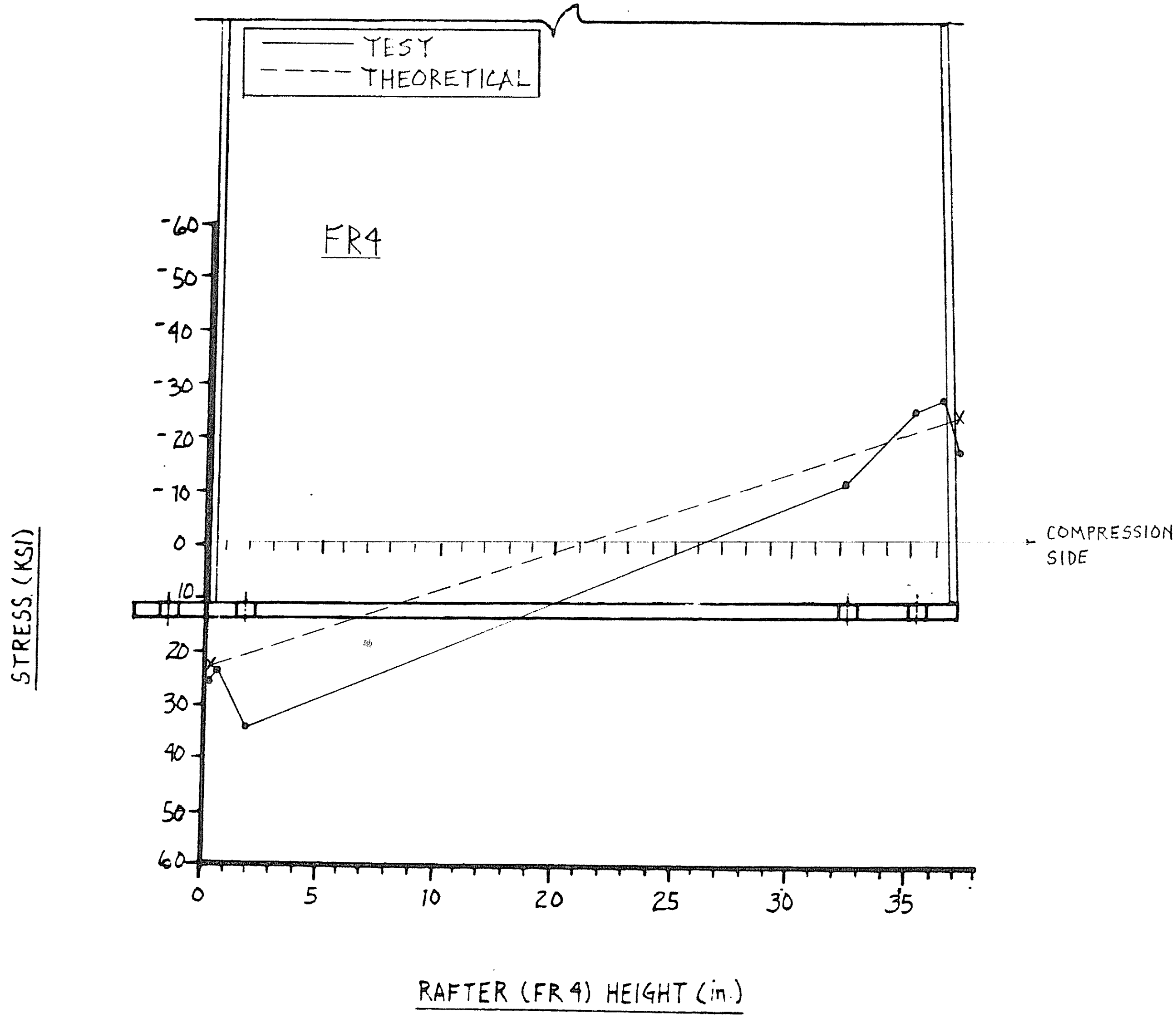


FIGURE D.17 STRESS VARIATION ACROSS RAFTER AT WORKING LOAD, TEST FA-4

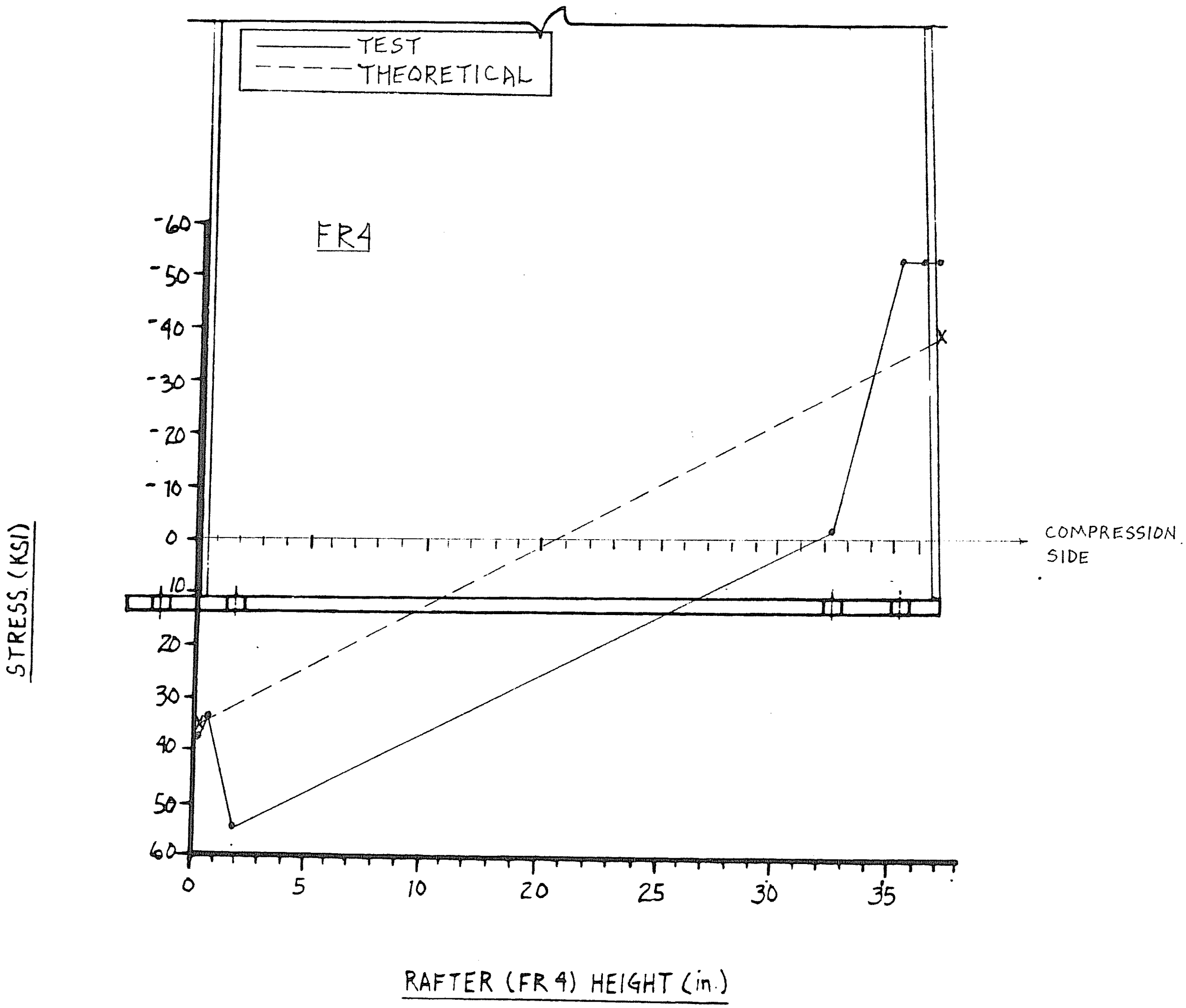


FIGURE D.18 STRESS VARIATION ACROSS RAFTER AT FAILURE LOAD, TEST FA-4

APPENDIX E

FA-5 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: MESCO Knee Test

Test No.: FA-5

Test Date: May 25, 1984

Purpose: Test of Knee Area

Number of Tension Bolts: 4   Bolt Gage (g): 3"   Pitch: 3"

Bolt Diameter: 3/4"   End Plate Thickness (t): .510"

End Plate Width (w): 5 15/16"   End Plate Length (de): 27 1/4"

Panel Zone Web Plate Thickness: 3/16 (nominal)   .183 (measured)

Initial Out-of-Straightness at the Center of Panel Zone: .214"

Pretension Force per Bolt: 28 kips

Failure Load, (Total Load): 29.4 kips

Failure Mode: Panel zone buckling

Predicted Failure Loads:

Method: Flexural and compr. stress intlg. Total Load: 34.1 kips

Method: Panel Buckling (No TFA) Total Load: 17.0 kips

Panel Buckling (TFA) 36.3 kips

Discussion:

-Up to 20 kips load, behavior of test specimen was almost elastic with no yield lines on the whitewash coating.

-At 25 kips load, panel zone buckled. Also significant plate separation could be observed at tension side of end plate connection. Bolt force in tension bolts was increasing quite rapidly.

-At 28 kips load, yield lines could be seen on compression flange of the column.

-At 29.2 kips load, column compression flange failed.

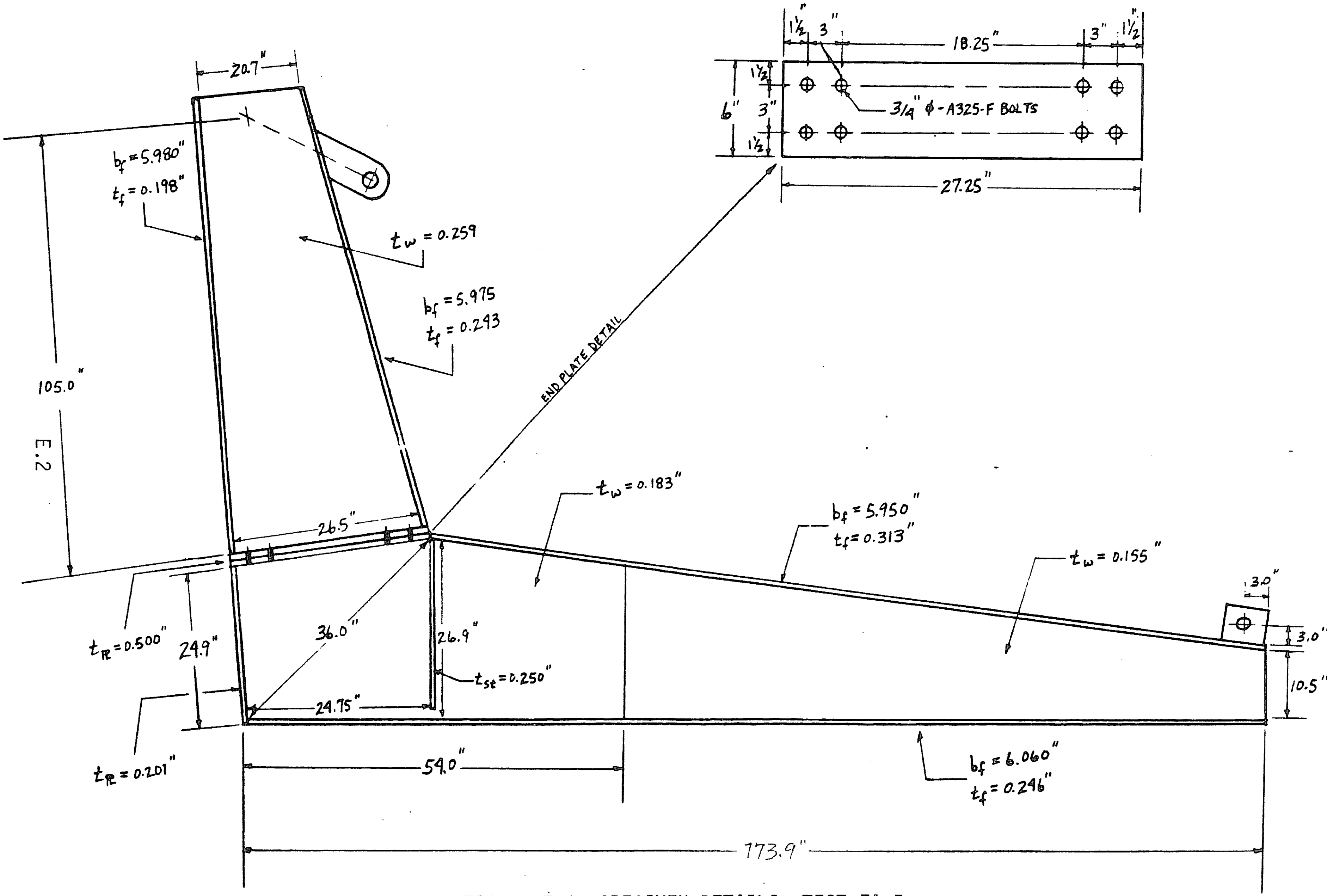


FIGURE E.1 SPECIMEN DETAILS, TEST FA-5

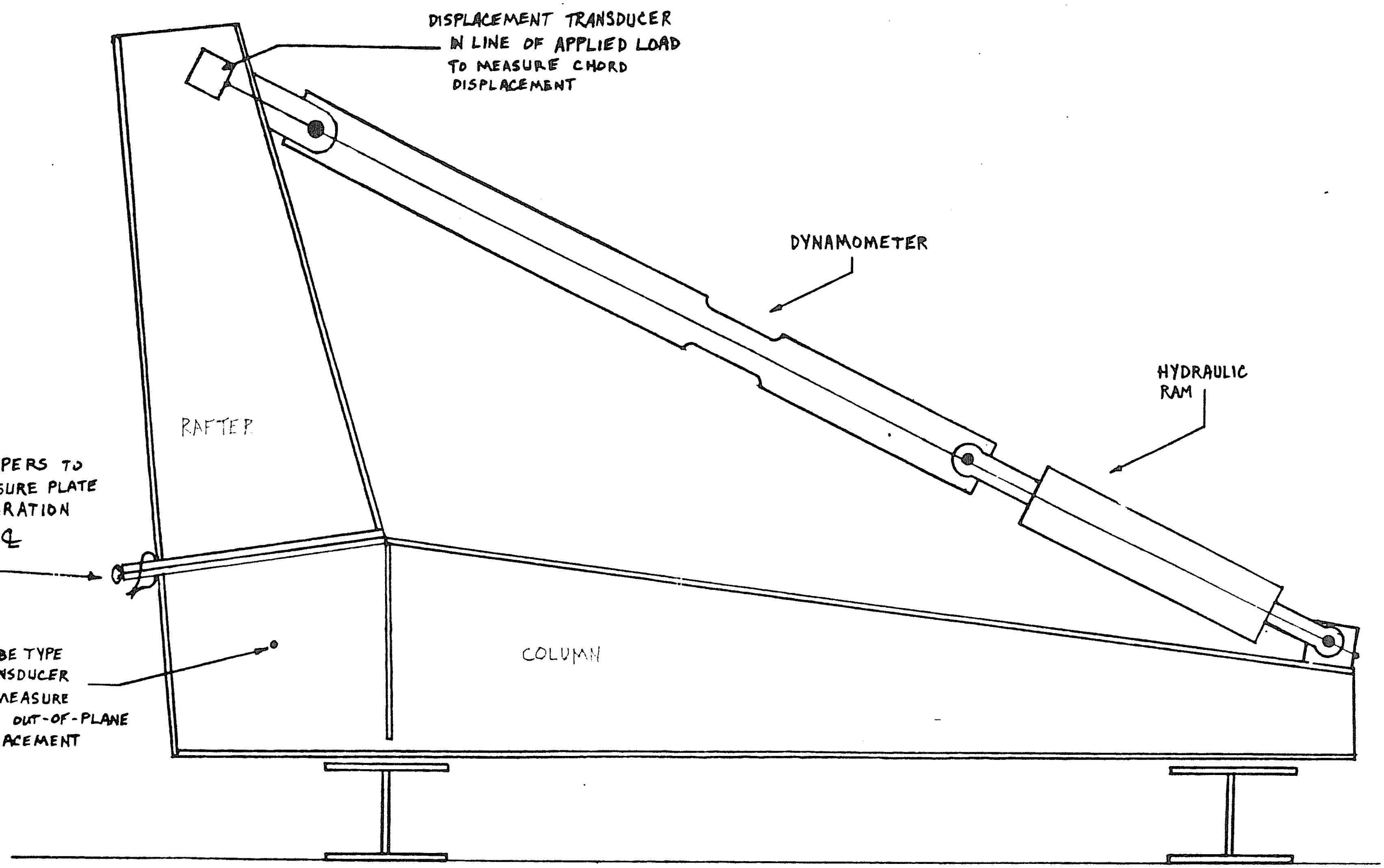


FIGURE E.2 TEST SET-UP, TEST FA-5

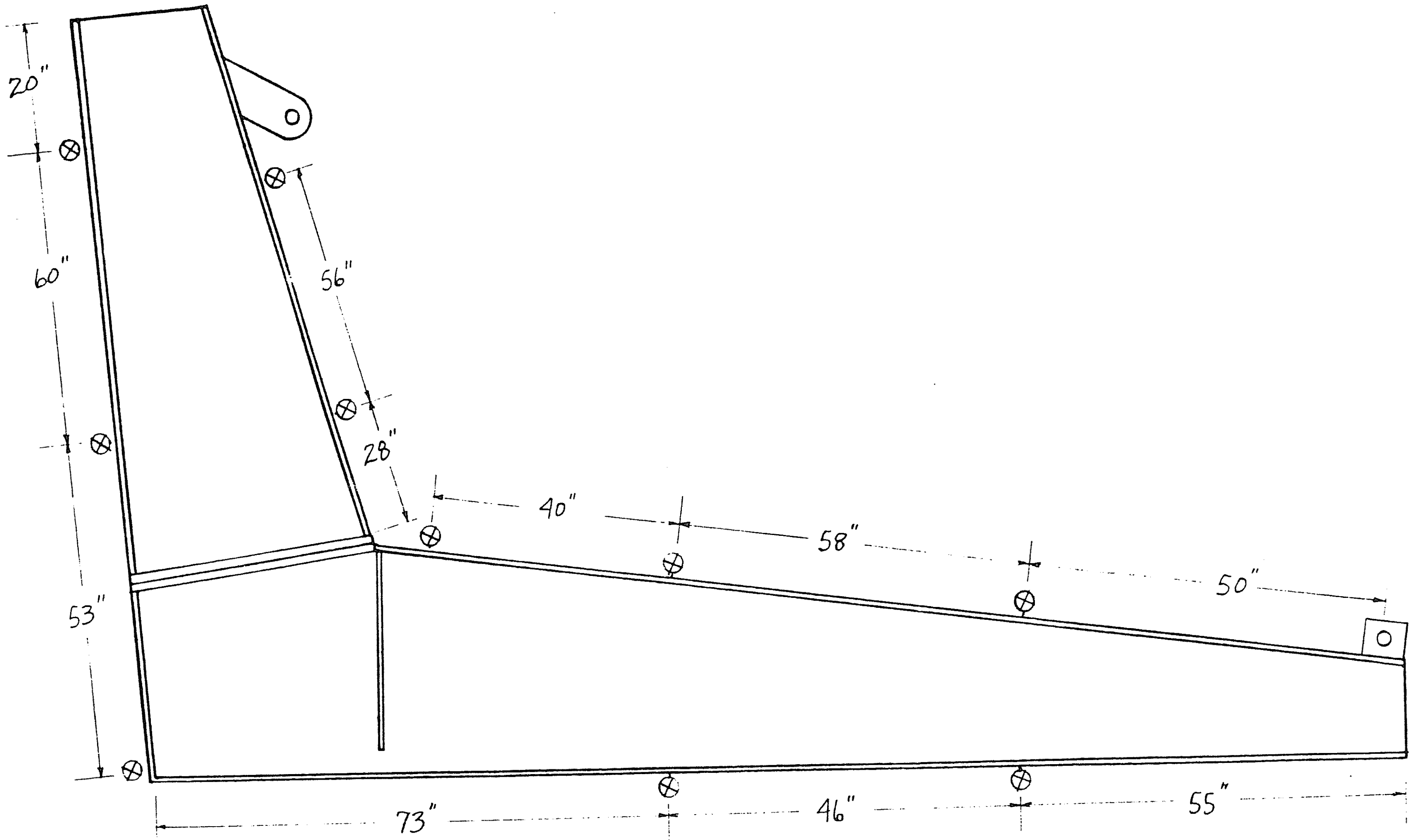


FIGURE E.3 LATERAL BRACE LOCATIONS, TEST FA-5

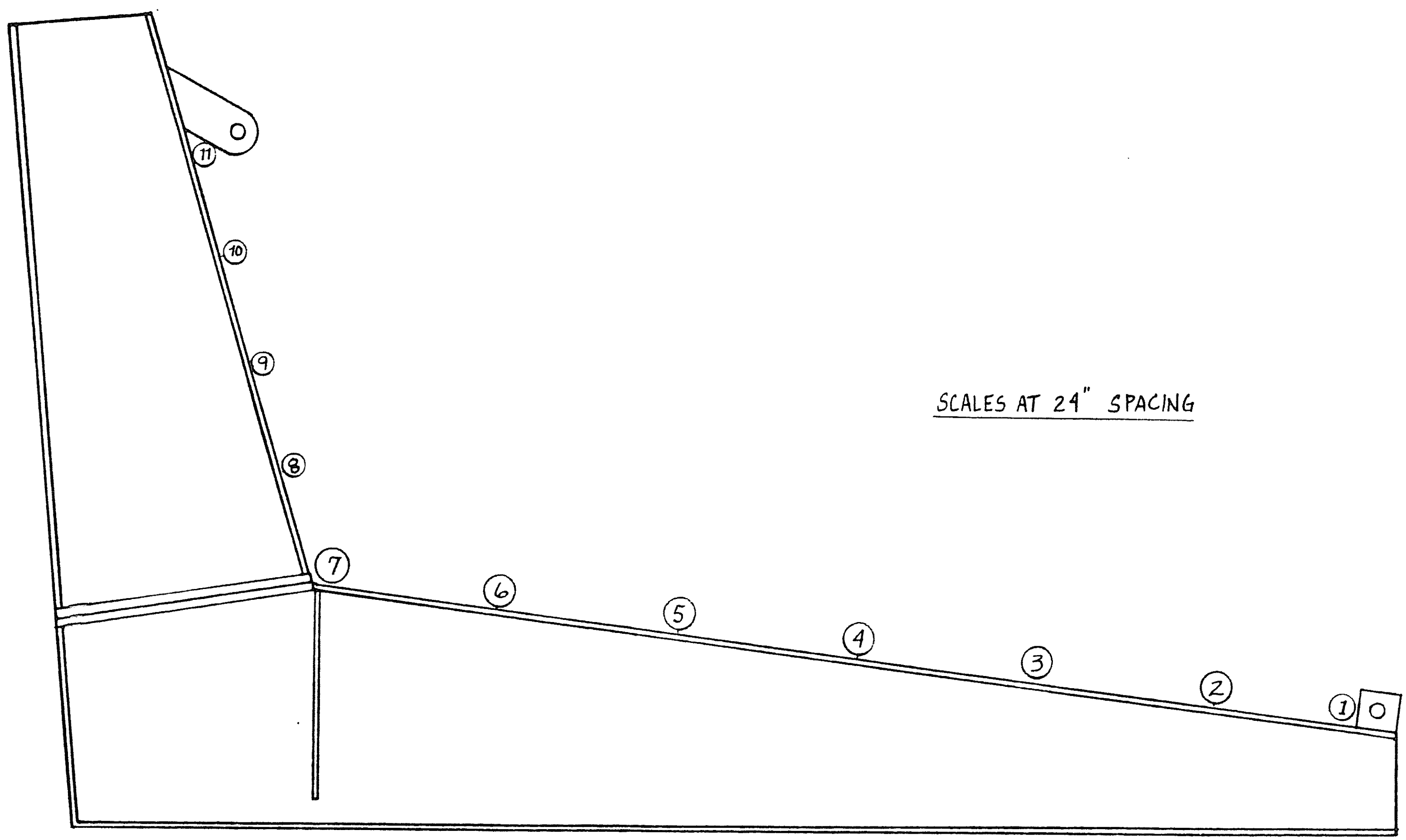


FIGURE E.4 LATERAL DISPLACEMENT SCALE LOCATIONS, TEST FA-5

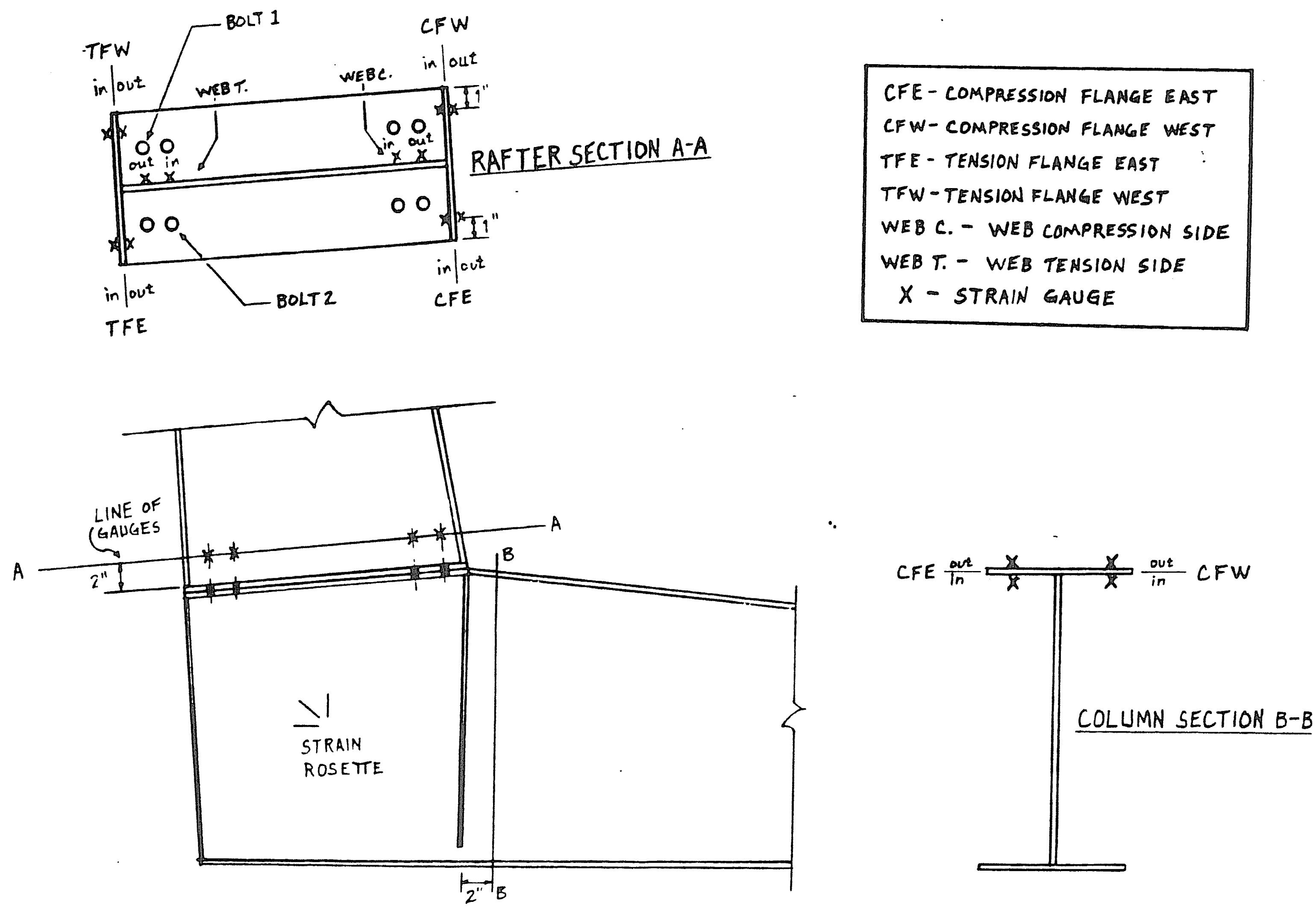


FIGURE E.5 STRAIN GAGE LOCATIONS, TEST FA-5

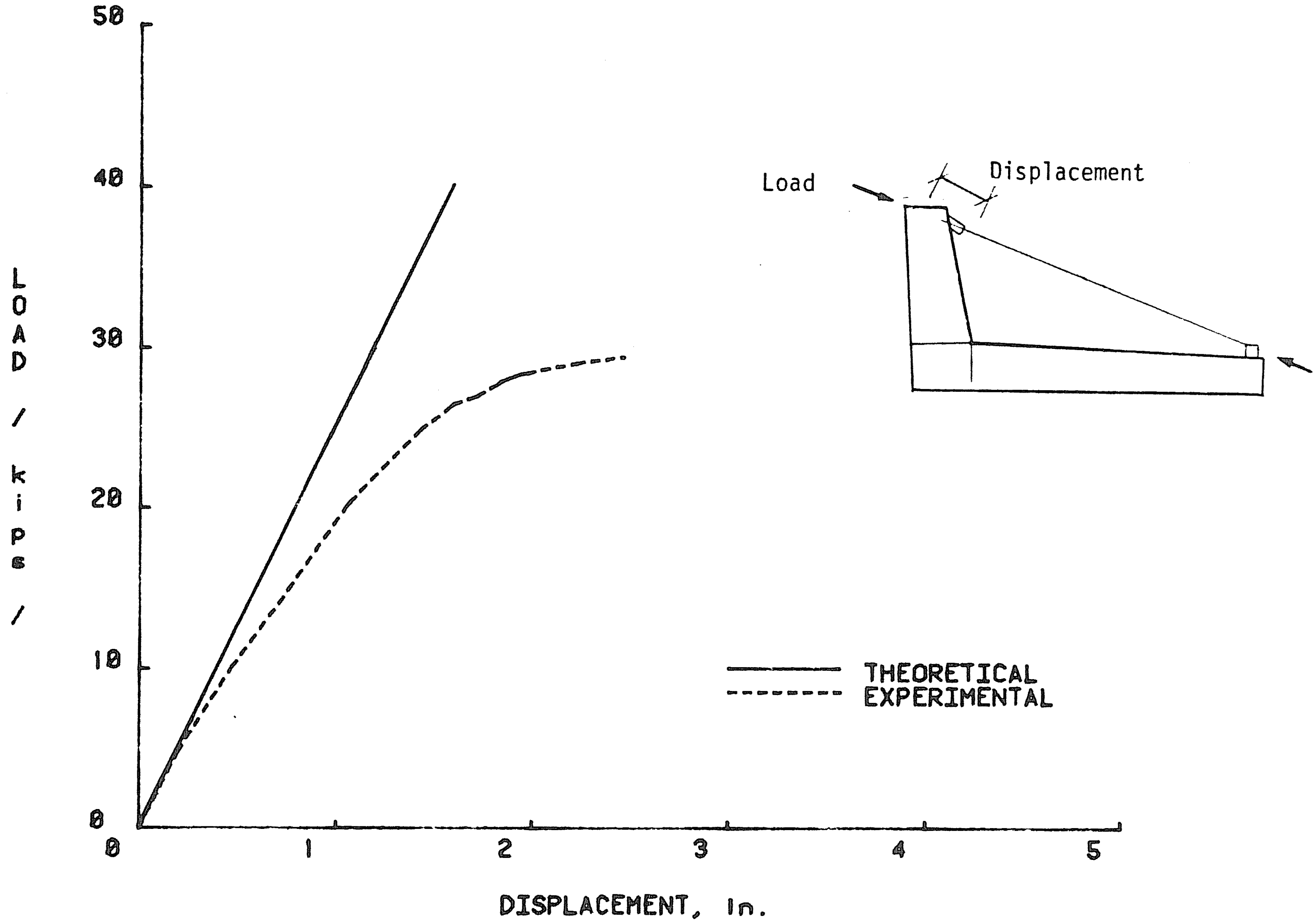


FIGURE E.6 LOAD VS CHORD DISPLACEMENT, TEST FA-5

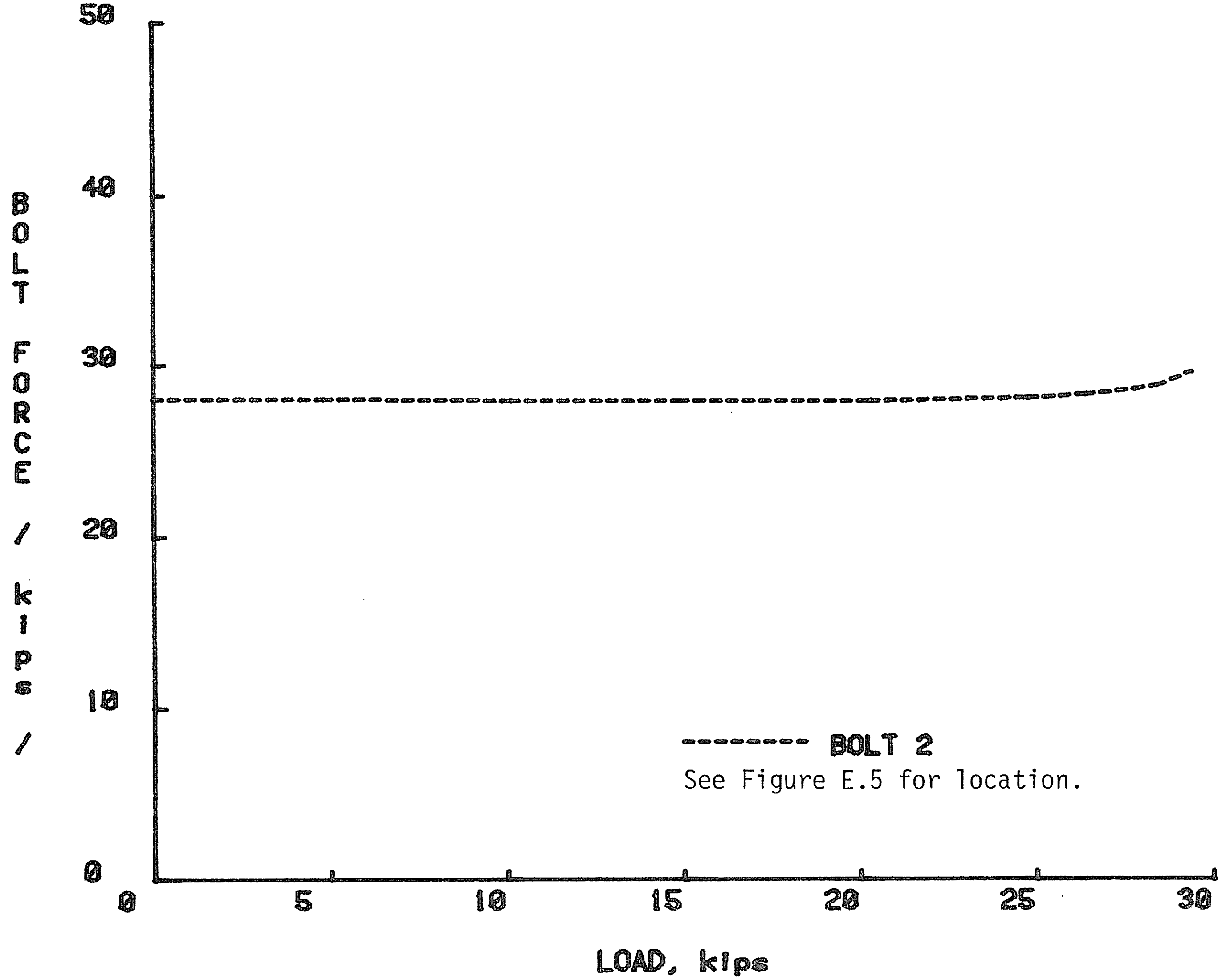


FIGURE E.7 BOLT FORCE VS LOAD, TEST FA-5

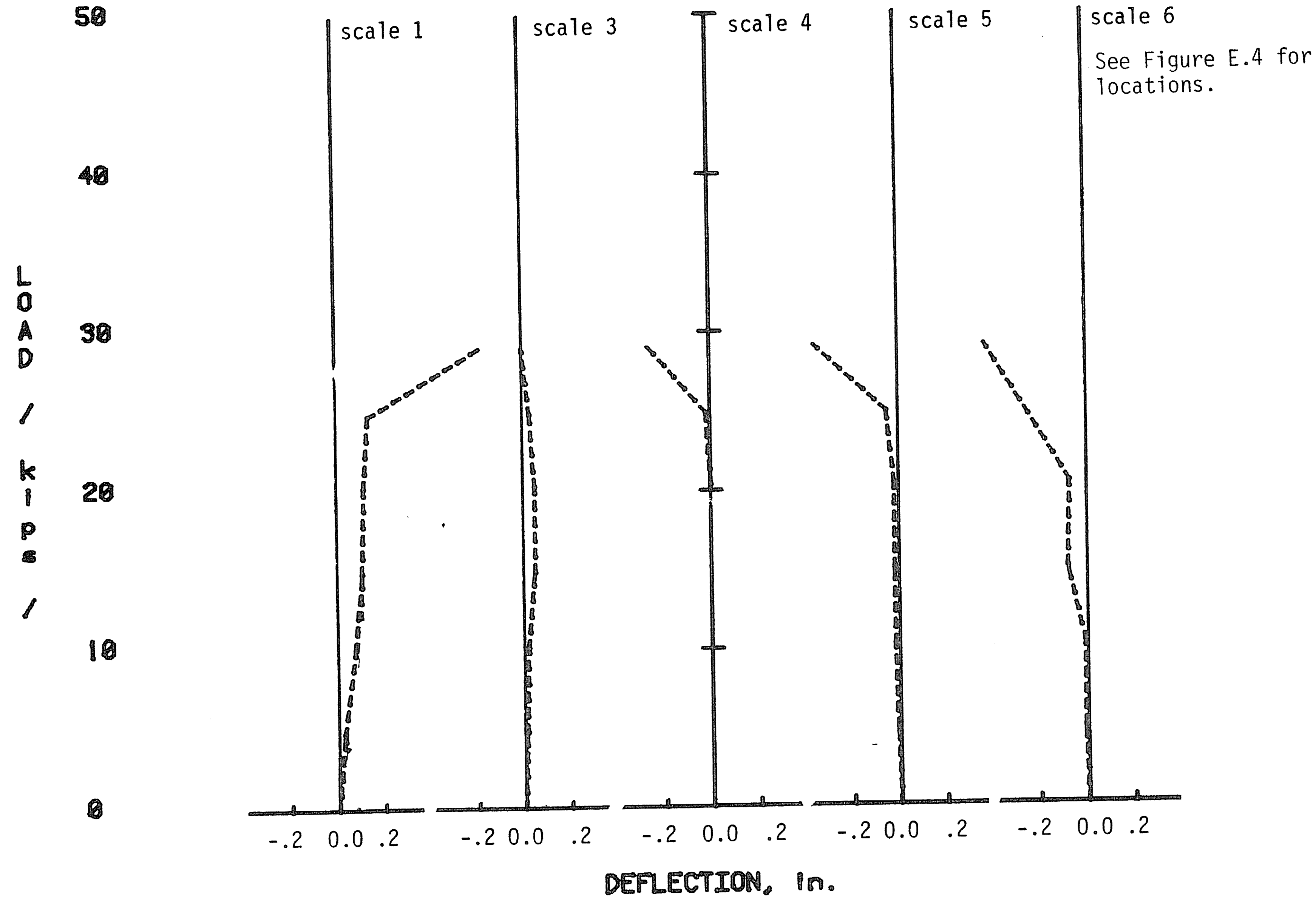


FIGURE E.8 LOAD VS LATERAL DEFLECTIONS, TEST FA-5

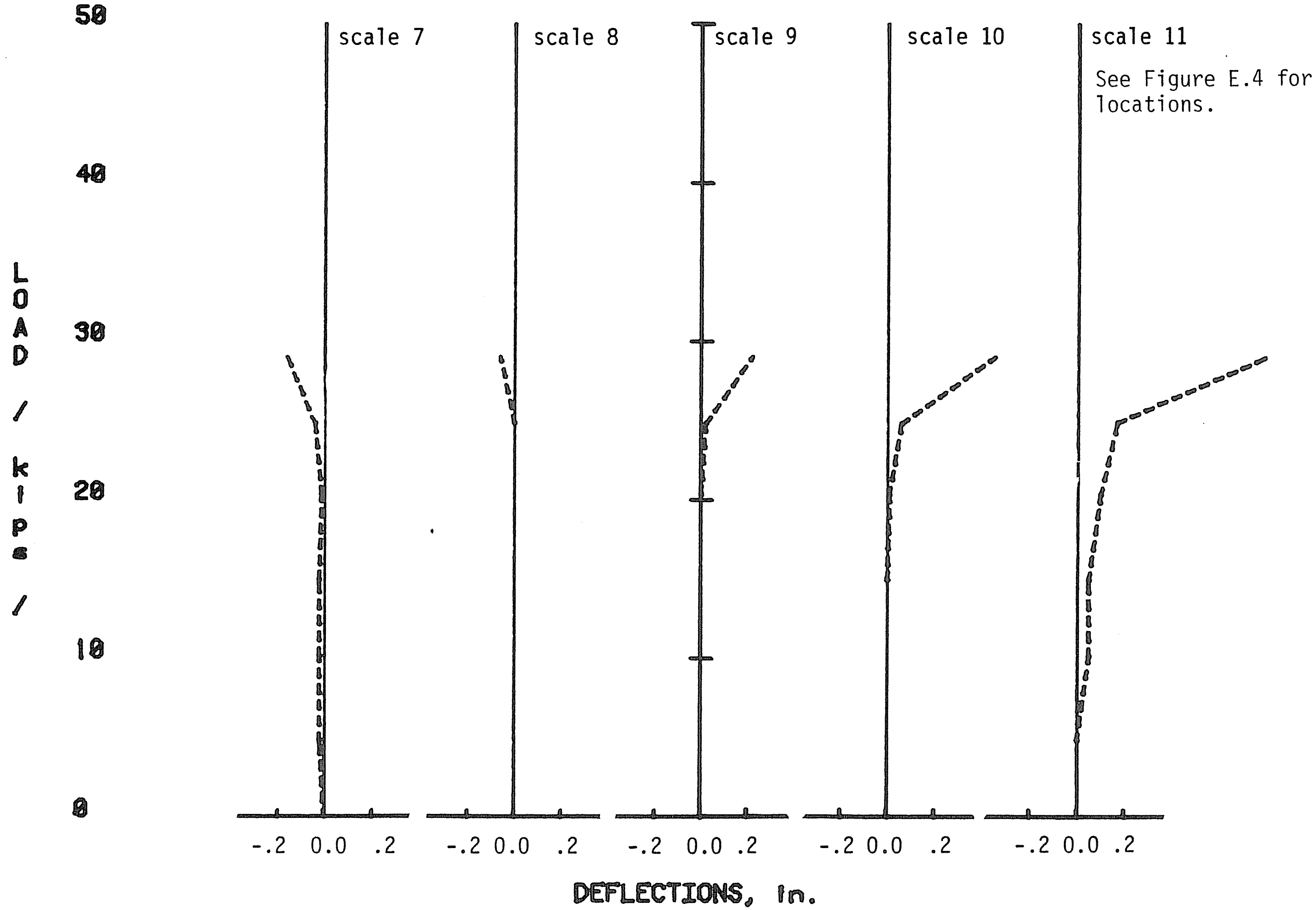


FIGURE E.9 LOAD VS LATERAL DEFLECTIONS, TEST FA-5

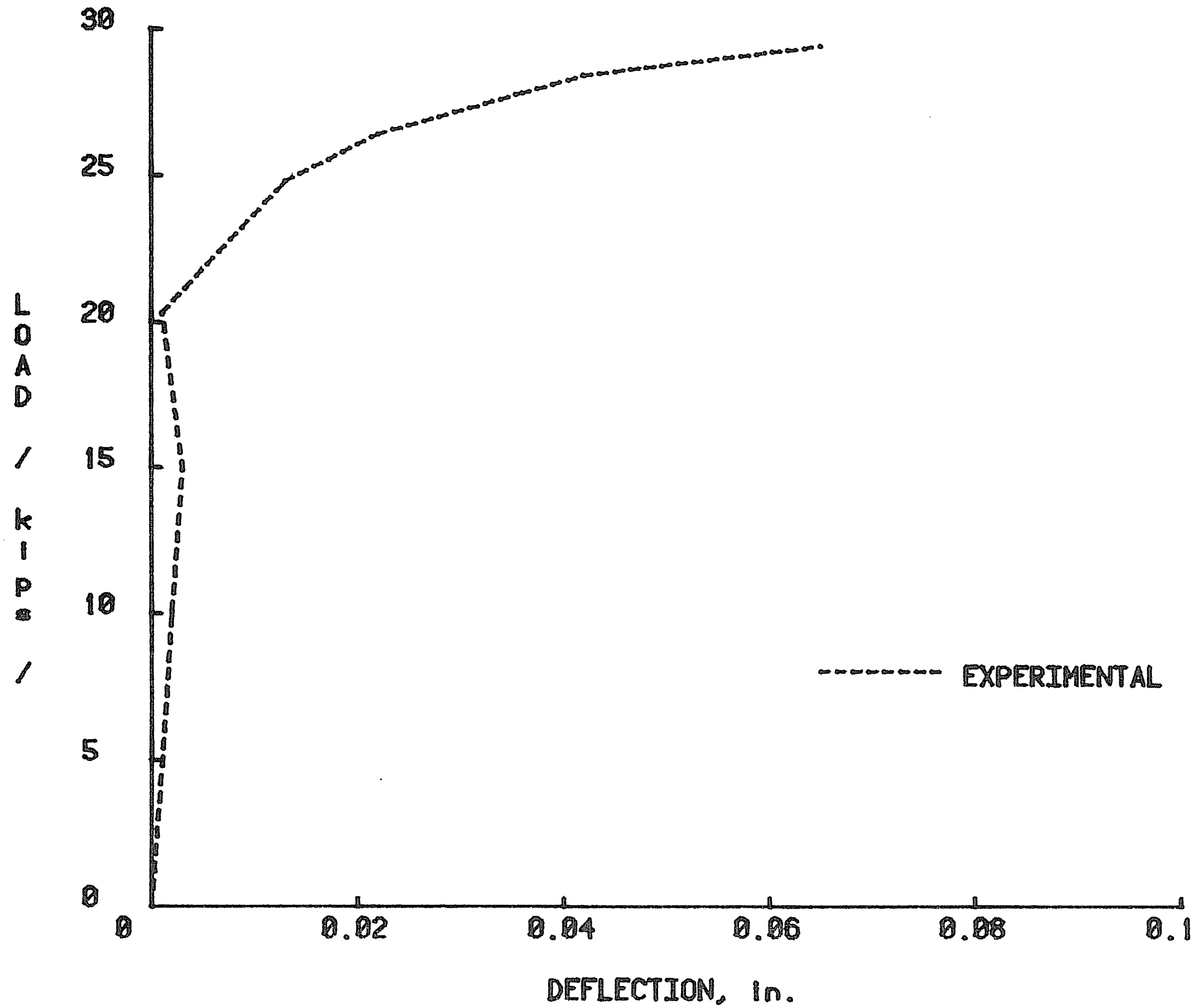


FIGURE E.10 LOAD VS. CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-5

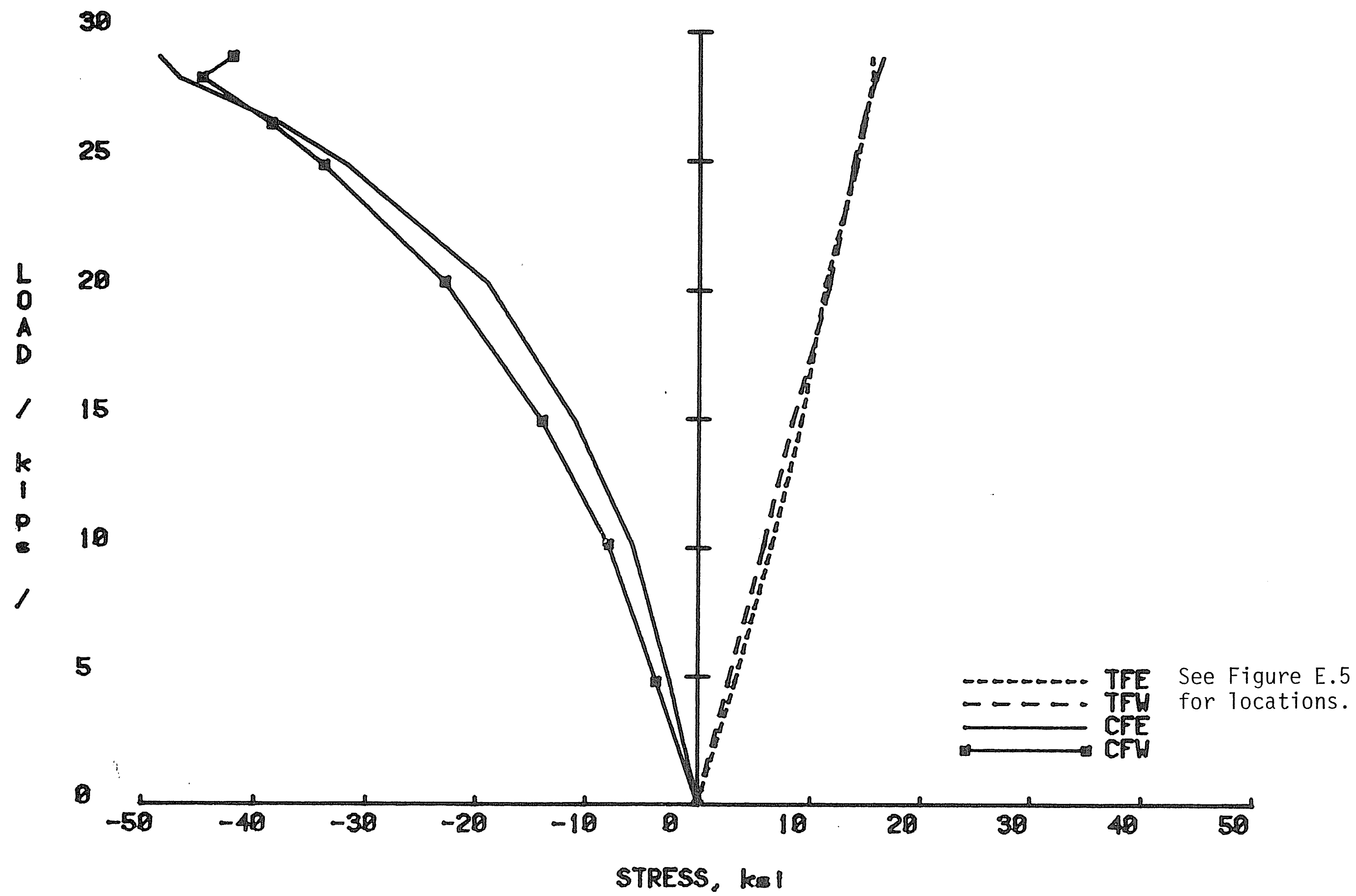


FIGURE E.11 LOAD VS RAFTER FLANGE STRESSES, TEST FA-5

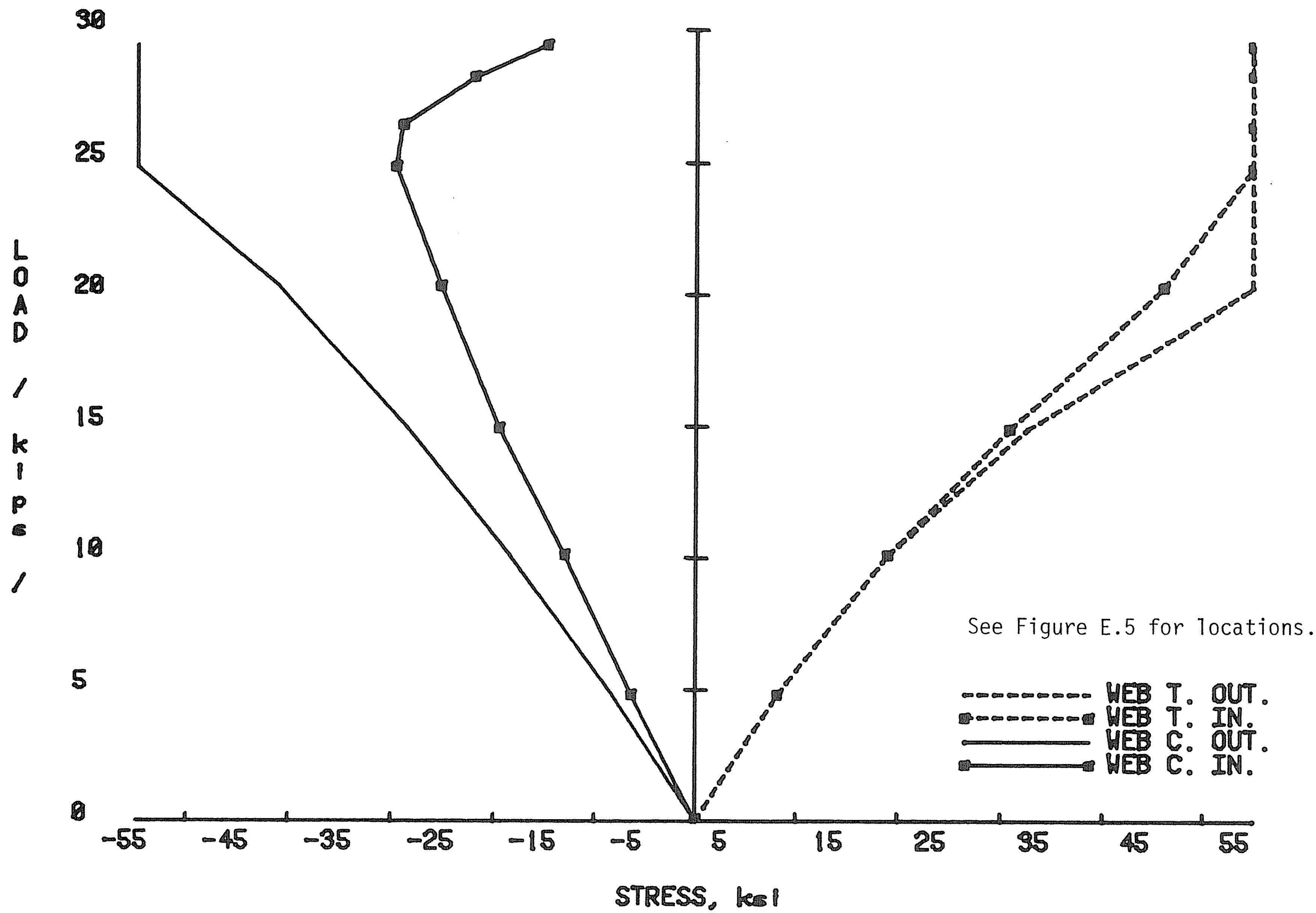


FIGURE E.12 LOAD VS RAFTER WEB STRESSES, TEST FA-5

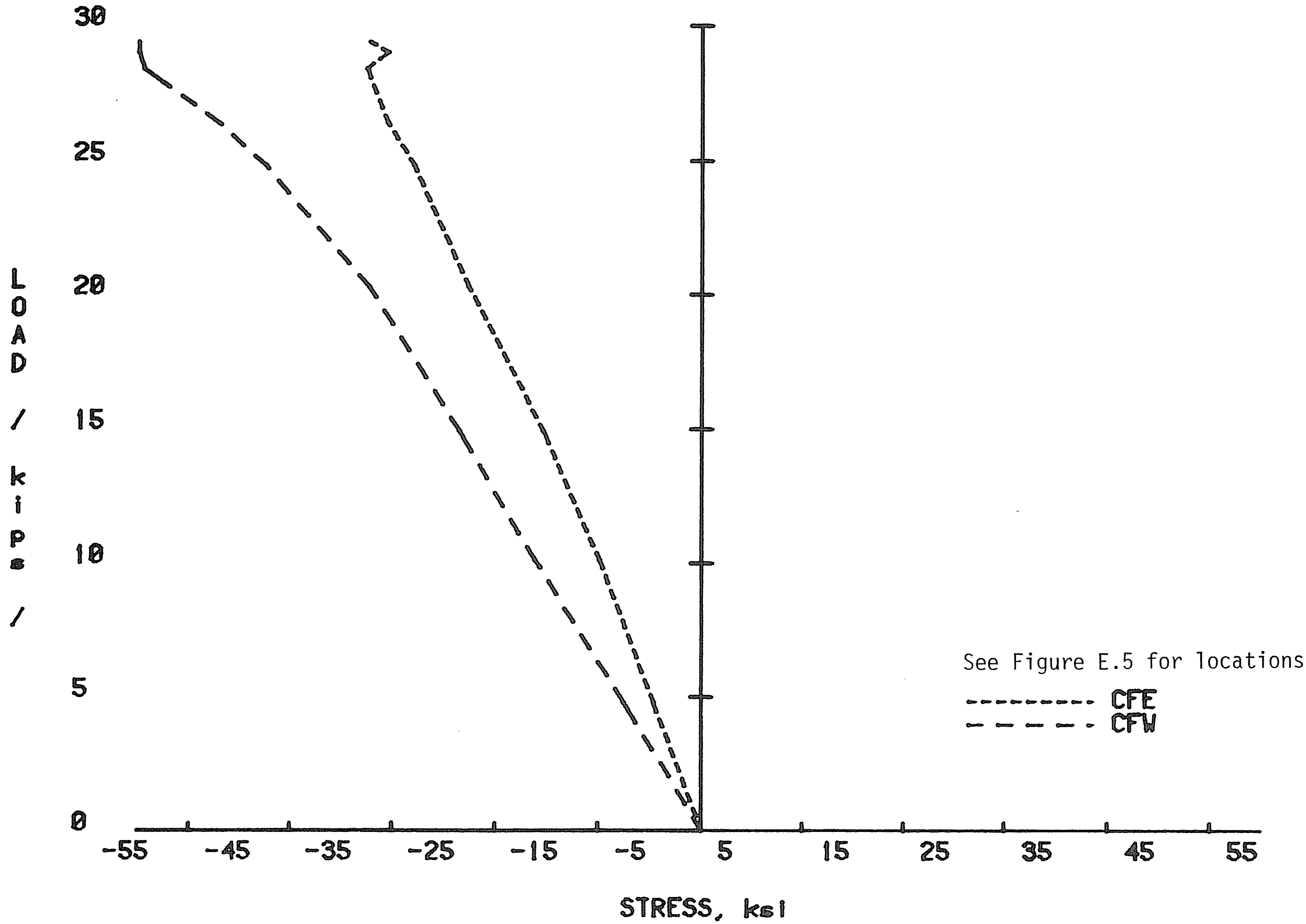


FIGURE E.13 **LOAD VS COLUMN STRESS, TEST FA-5**

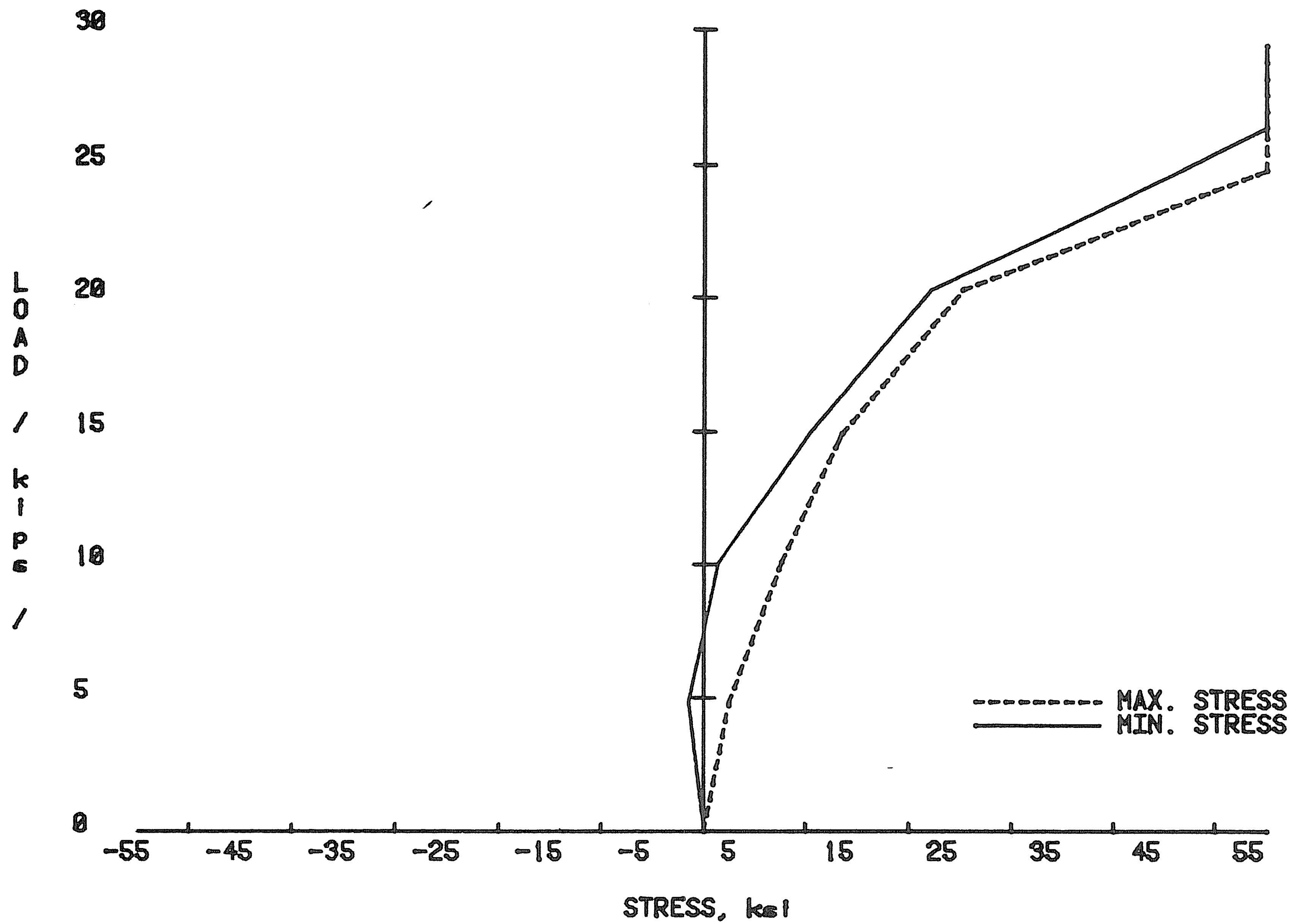


FIGURE E.14 **LOAD VS PRINCIPAL STRESSES, TEST FA-5**
(Measured at Center of Panel Zone)

TEST FA-5

Load, k	Horizontal Stress, ksi	Vertical Stress, ksi	Effective Stress, ksi	Shear Stress ksi,
4.80	0.56	0.43	3.48	1.99
10.10	4.27	4.76	7.03	3.10
14.90	11.08	12.78	12.24	1.34
20.30	22.46	24.89	23.83	0.97
24.80	56.90	58.77	60.96	11.09
26.40	86.76	85.13	92.30	19.42
28.20	153.84	140.63	159.68	35.07
29.40	246.59	223.41	250.86	49.33

Load, k	Principle Stress, ksi		Principle Strain Micro Strain		Theta, detrees
4.80	2.5	-1.5	101.	-77.	44.0
10.10	7.6	1.4	248.	-30.	-42.7
14.90	13.5	10.3	359.	217.	-28.8
20.30	25.2	22.1	641.	502.	19.3
24.80	69.0	46.7	1895.	897.	42.6
26.40	105.4	66.5	2946.	1203.	-43.8
28.20	182.9	111.6	5154.	1954.	-39.7
29.40	285.7	184.3	7944.	3401.	-38.4

TABLE E.1 Stress and Strain at Center of Panel Zone

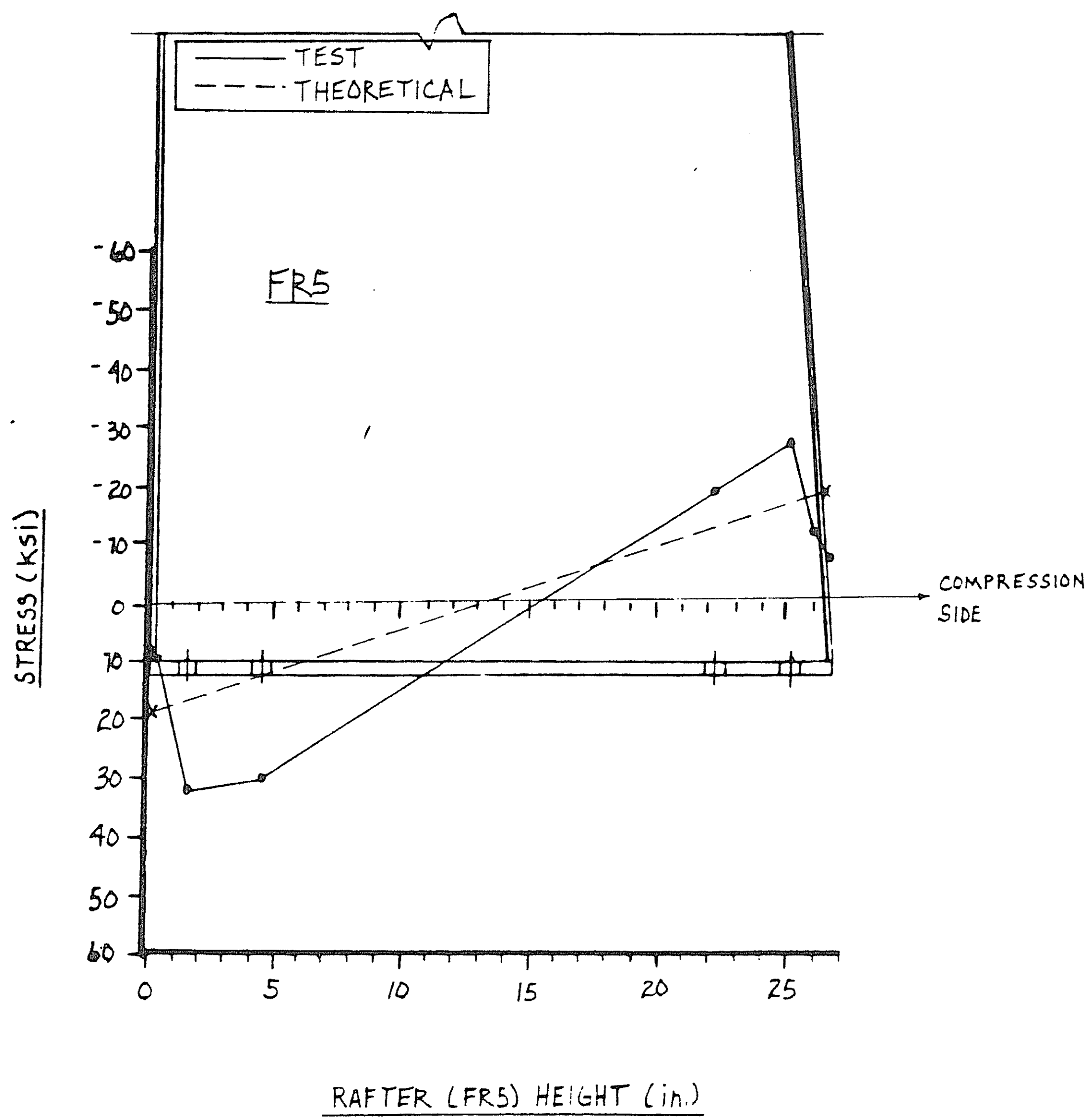


FIGURE E.15 STRESS VARIATION ACROSS RAFTER AT WORKING LOAD, TEST FA-5

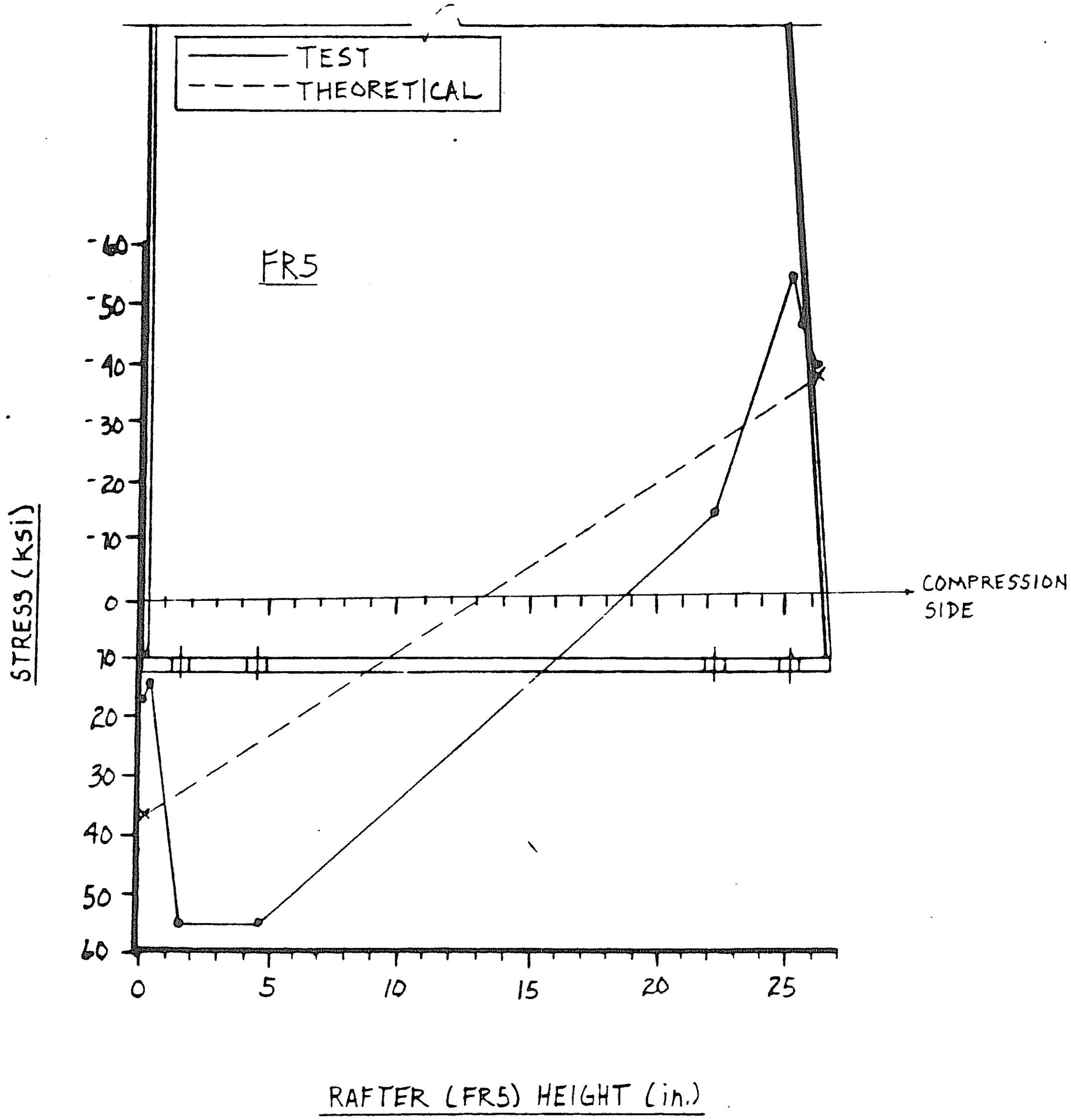


FIGURE E.16 STRESS VARIATION ACROSS RAFTER AT FAILURE LOAD, TEST FA-5

APPENDIX F

FA-6 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: MESCO Knee Test

Test No.: FA-6

Test Date: May 29, 1984

Purpose: Test of Knee Area

Number of Tension Bolts: 6 **Bolt Gage (g):** 3" **Pitch:** 3"

Bolt Diameter: 3/4" **End Plate Thickness (t):** .510"

End Plate Width (w): 8.0" **End Plate Length (de):** 51.75"

Panel Zone Web Plate Thickness: .32A"

Initial Out-of-Straightness at the Center of Panel Zone:

Pretension Force per Bolt: 28 kips

Failure Load, (Total Load): 63.8 kips

Failure Mode: Shear in rafter and lateral torsional buckling of column

Predicted Failure Loads:

Method: Flexural and compr. stress in rafter **Total Load:** 102.8 kips

Method: Shear in rafter web **Total Load:** 63.93 kips

Discussion:

-At 30 kips load, minor yielding was observed on compression area of rafter web adjacent to end plate.

-At 45 kips, the yielding was more severe on rafter web and some minor yield lines could be seen on compression flange of column between corner point and the point where web is spliced.

-At 55 kips, severe yielding of column compression flange could be observed. The compression flange has deformed laterally westward.

-At 60 kips, lateral deformation of column flange was considerable.

-At load equal to 64 kips, lateral torsional buckling occurred in the column.

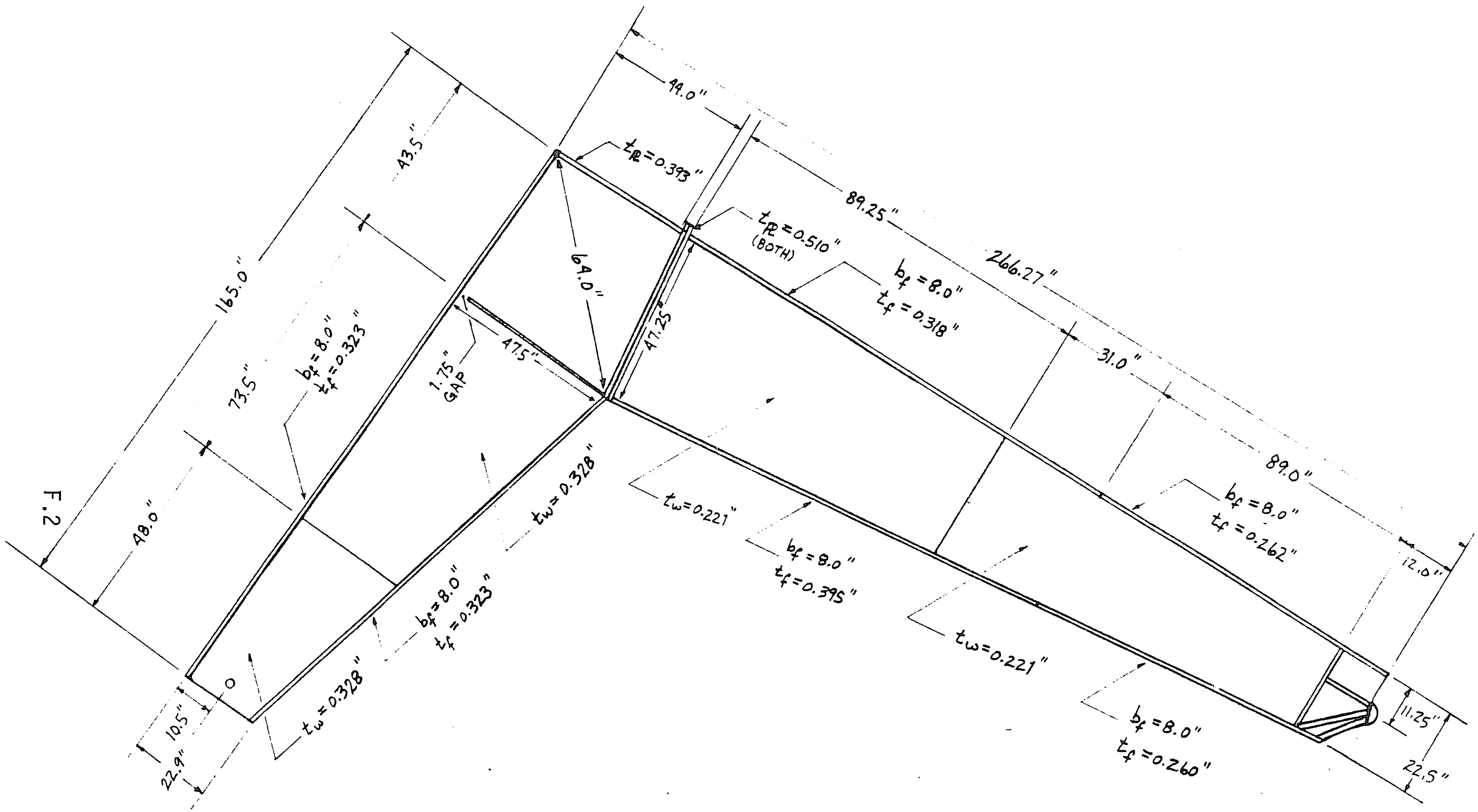


FIGURE F.1 SPECIMEN DETAILS, TEST FA-6

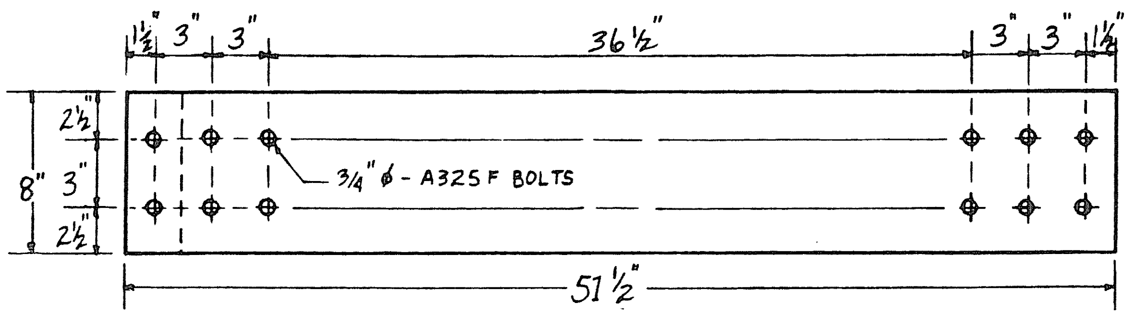


FIGURE F.2 RAFTER-to-COLUMN CONNECTION END-PLATE, TEST FA-6

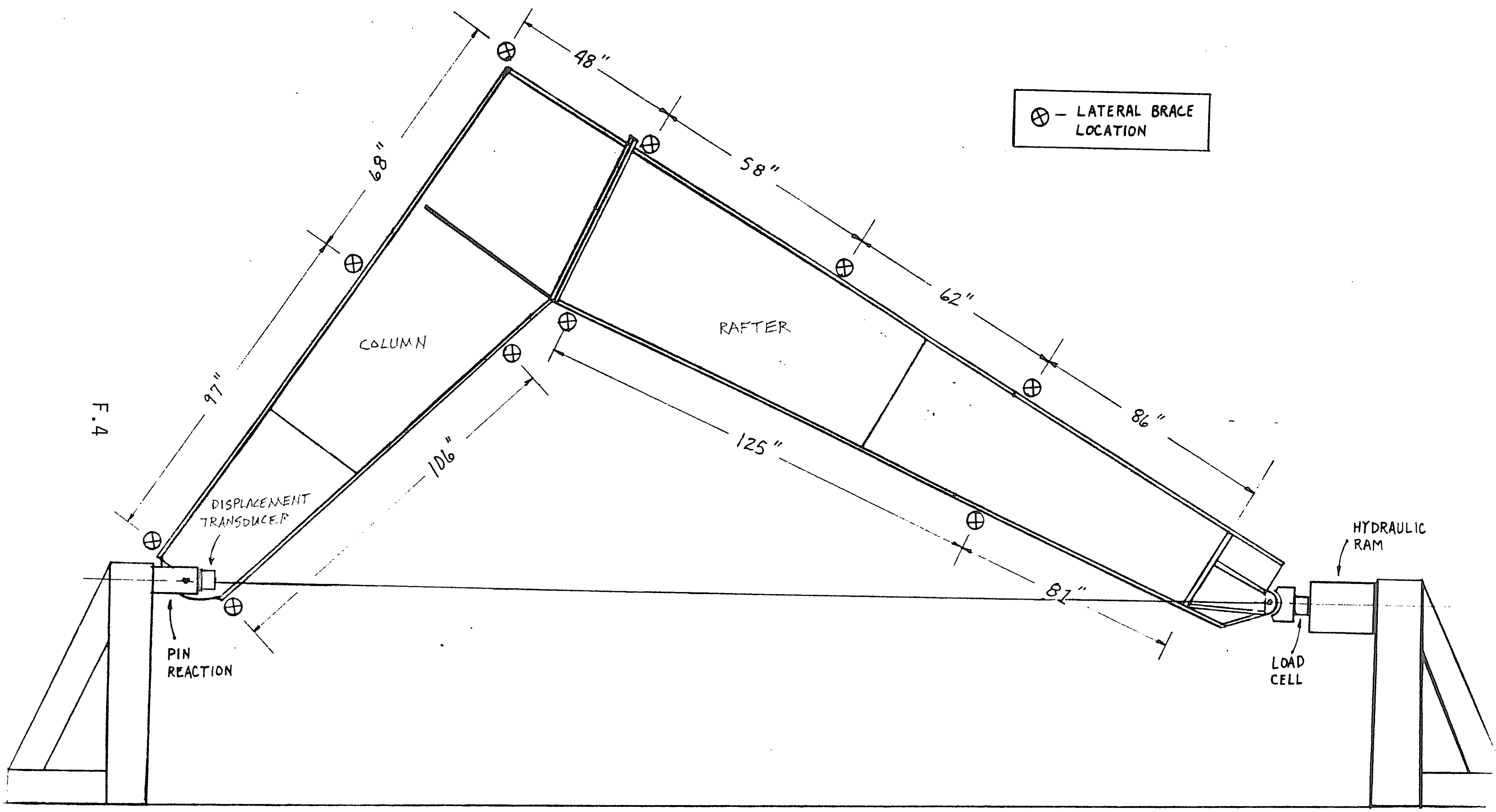


FIGURE F.3 LATERAL BRACE LOCATIONS, TEST FA-6

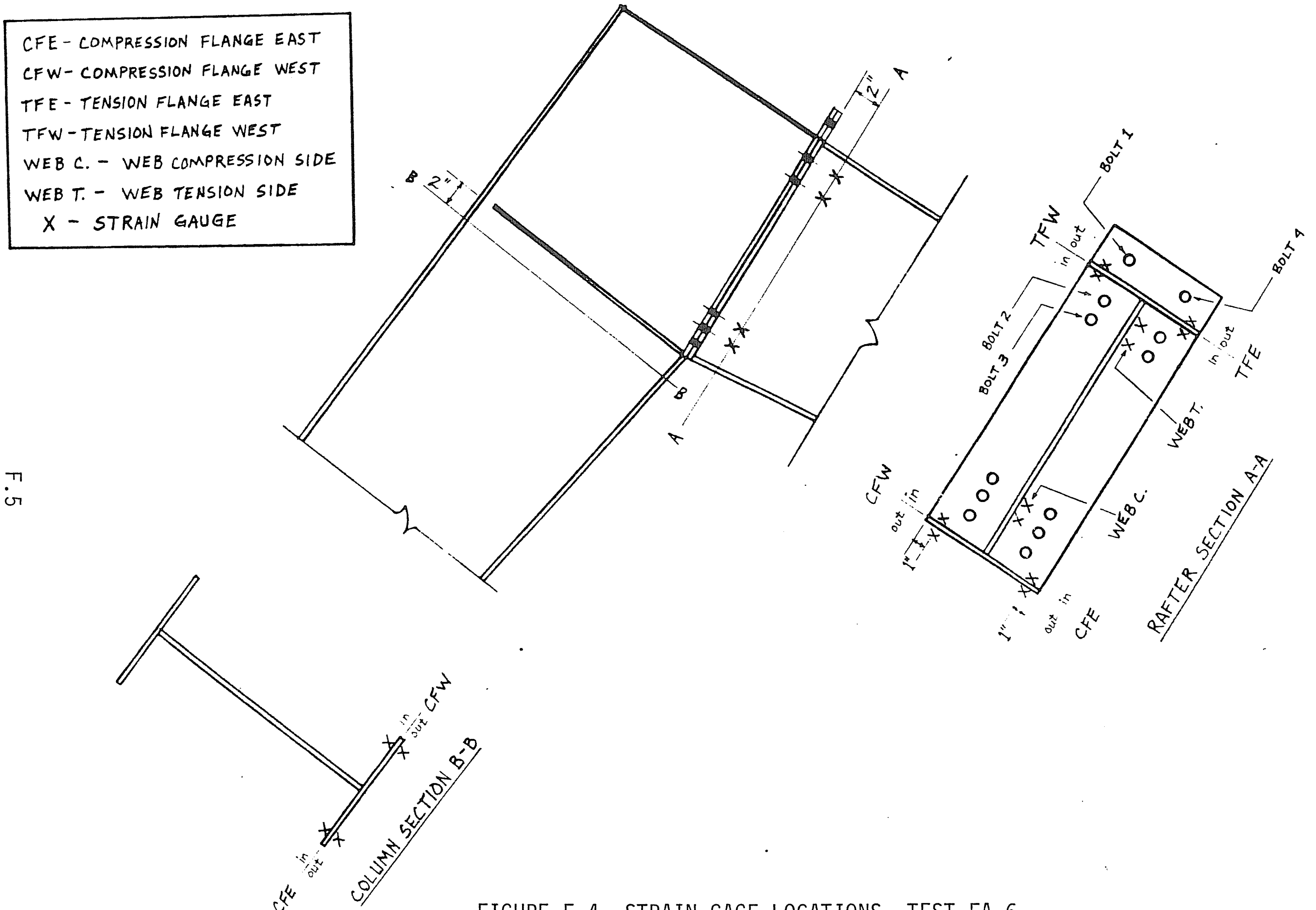


FIGURE F.4 STRAIN GAGE LOCATIONS, TEST FA-6

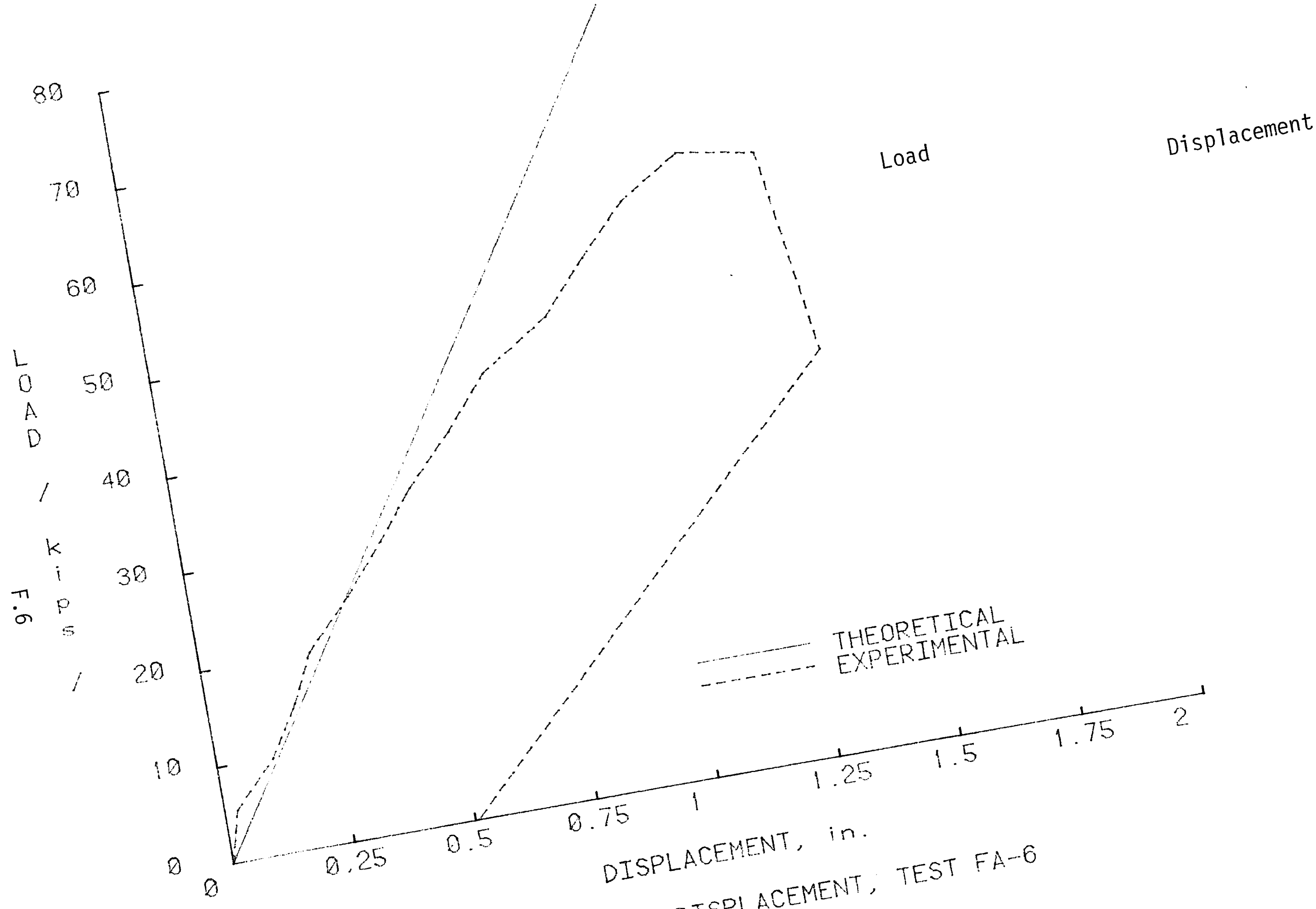


FIGURE F.5 LOAD VS CHORD DISPLACEMENT, TEST FA-6

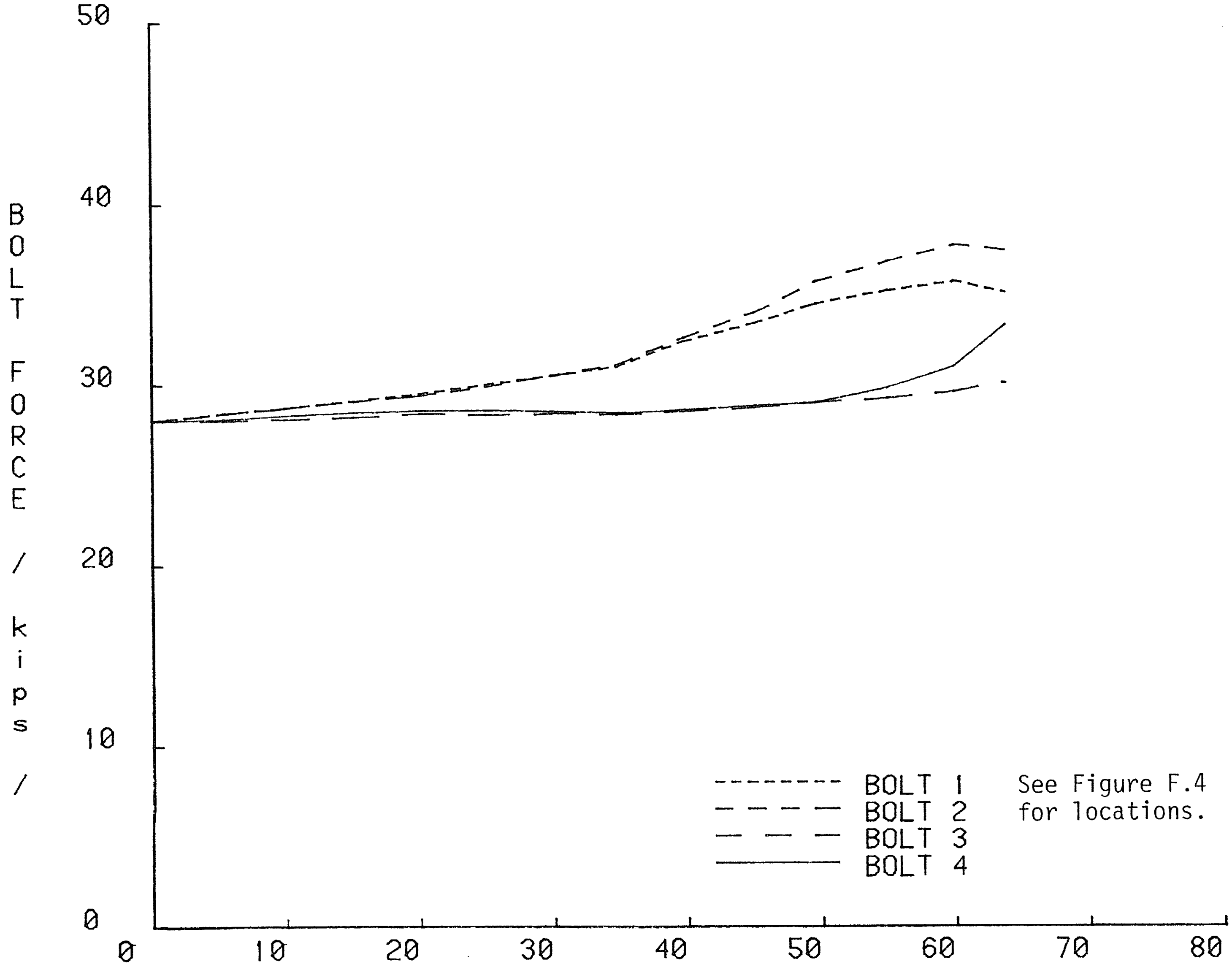


FIGURE F.6 BOLT FORCE VS LOAD, TEST FA-6

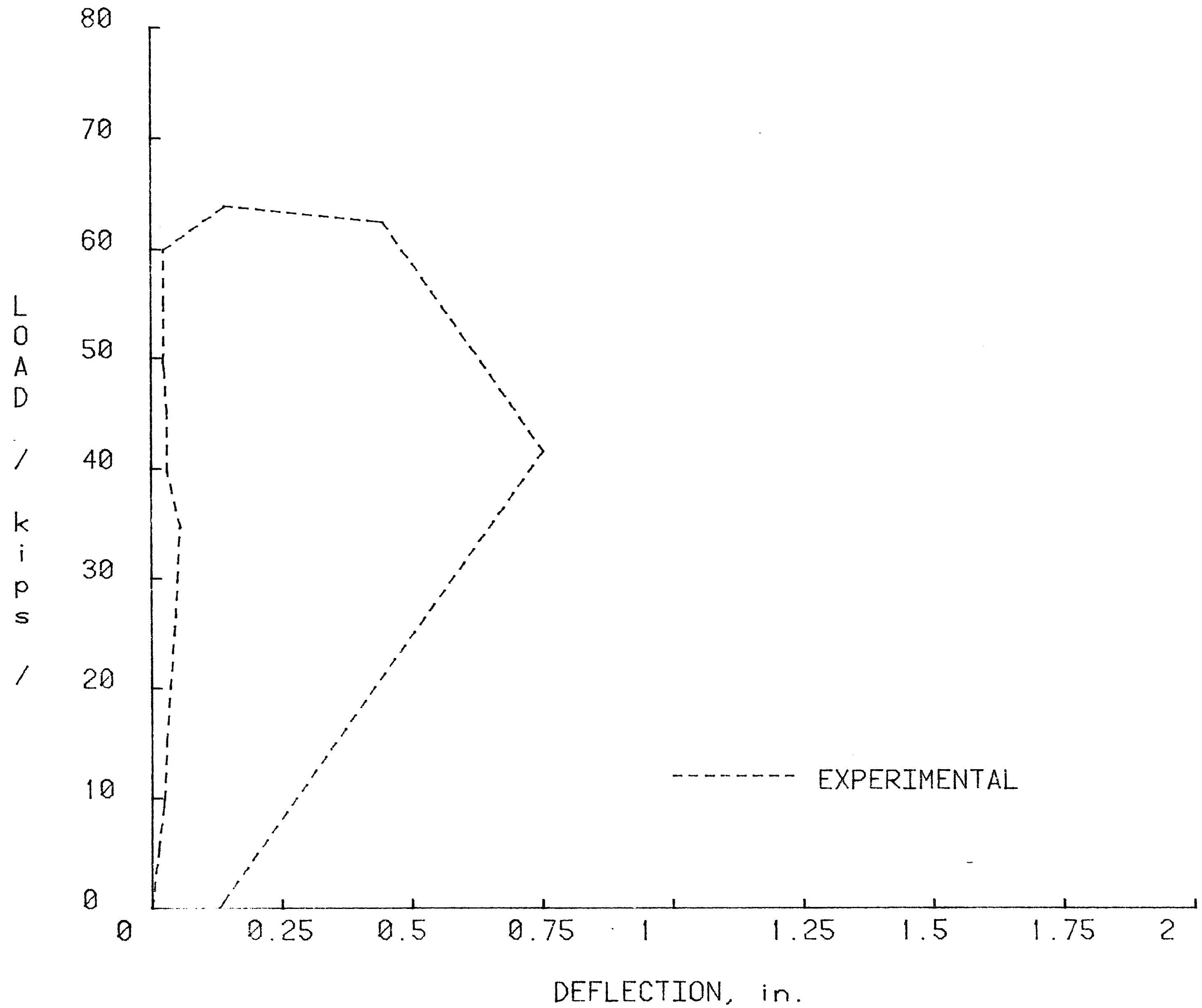


FIGURE F.7 LOAD VS CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-6

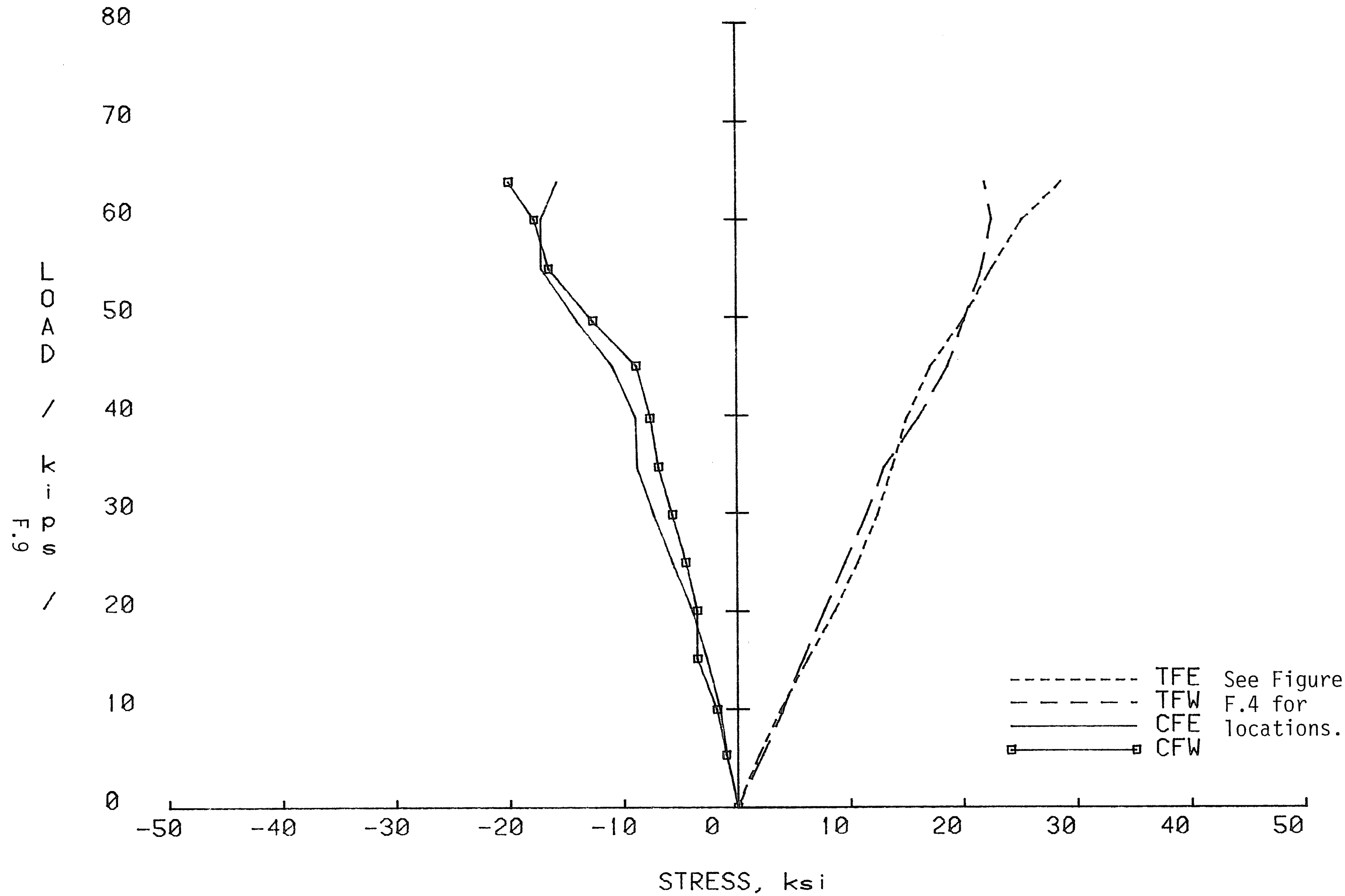


FIGURE F.8 LOAD VS RAFTER STRESS, TEST FA-6

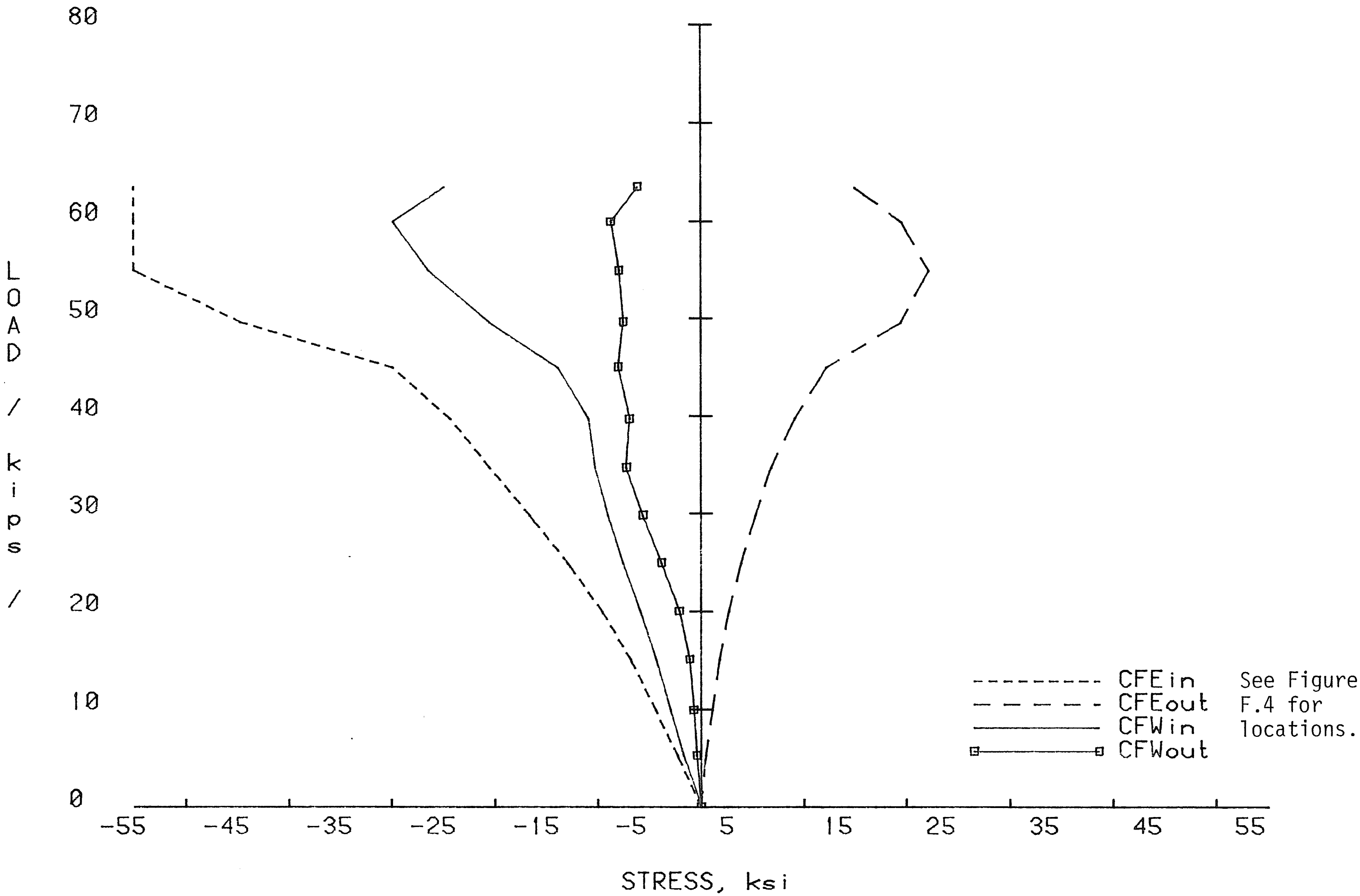


FIGURE F.9 LOAD VS COMPR. RAFTER STRESSES, TEST FA-6

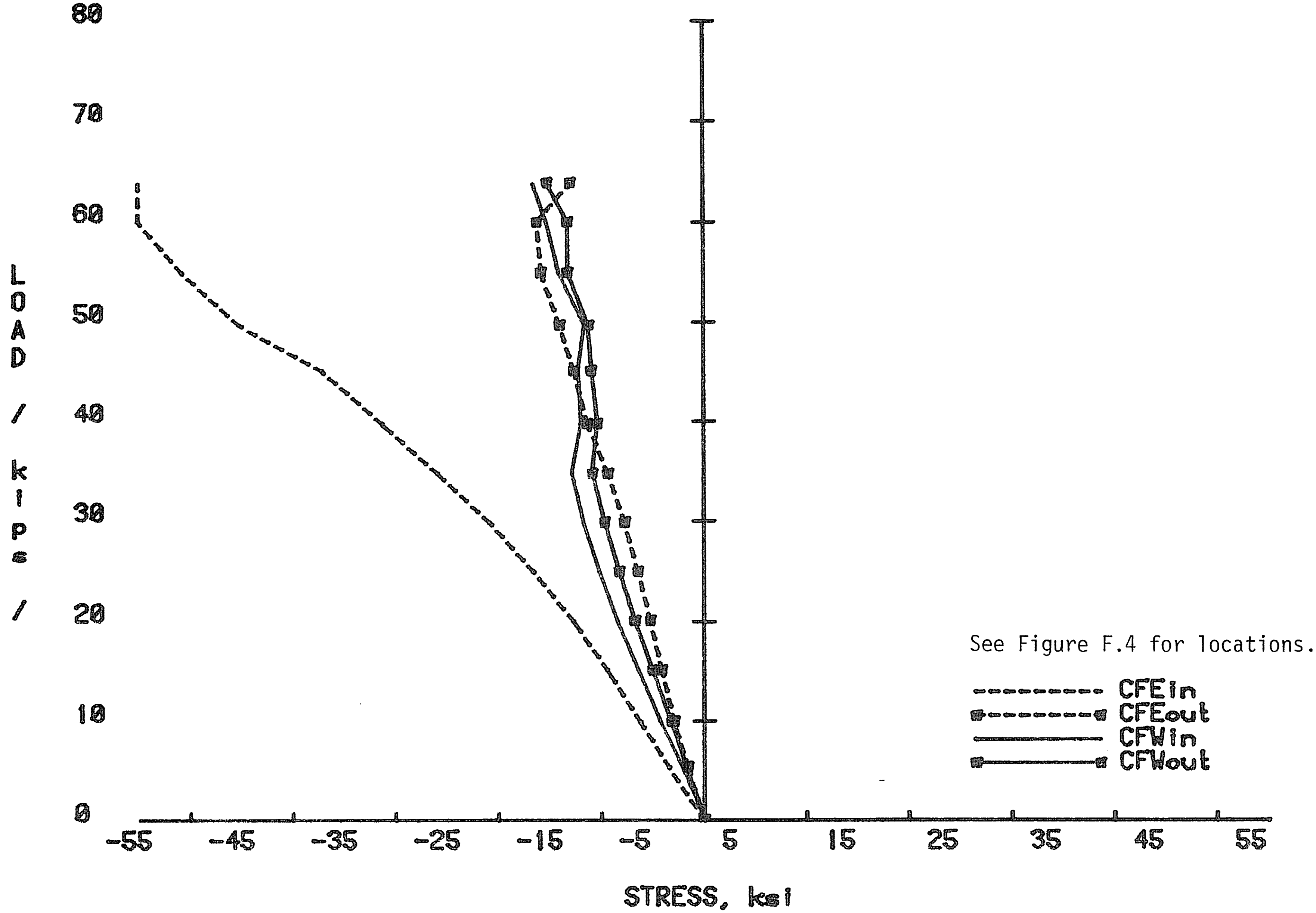


FIGURE F.10 **LOAD VS COLUMN STRESS, TEST FA-6**

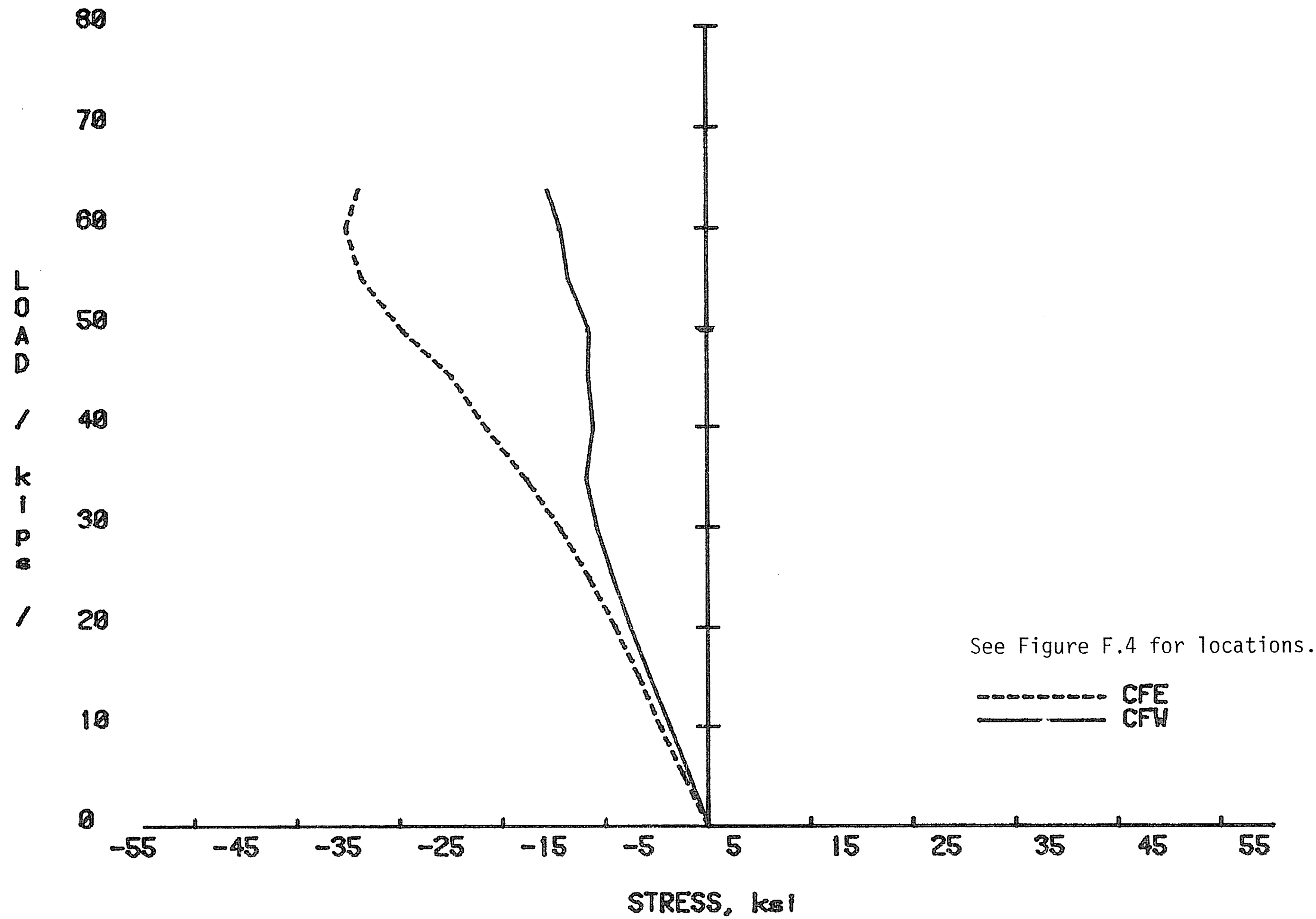


FIGURE F.11 **LOAD VS COLUMN STRESS, TEST FA-6**

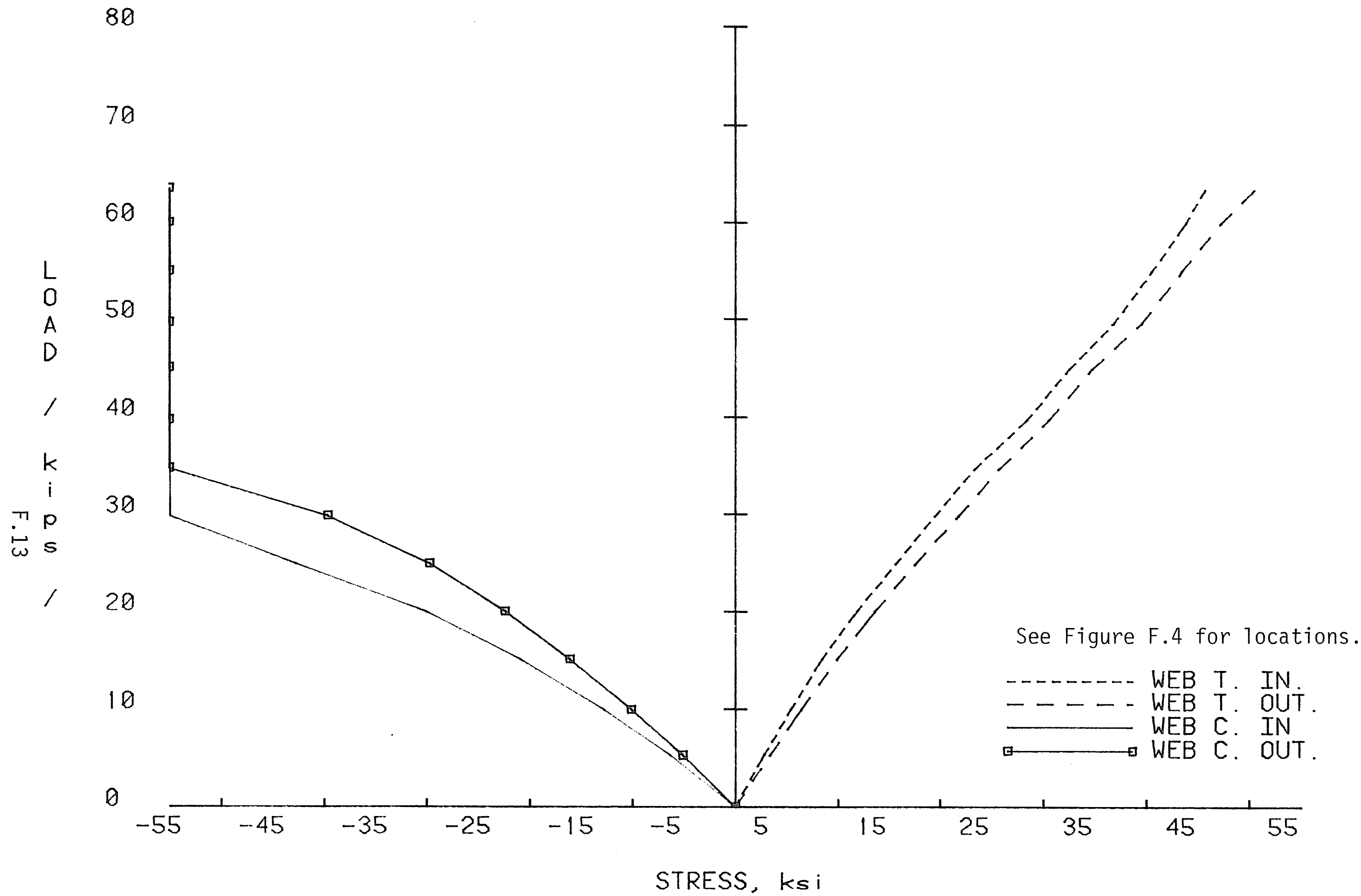


FIGURE F.12 LOAD VS WEB STRESS, TEST FA-6

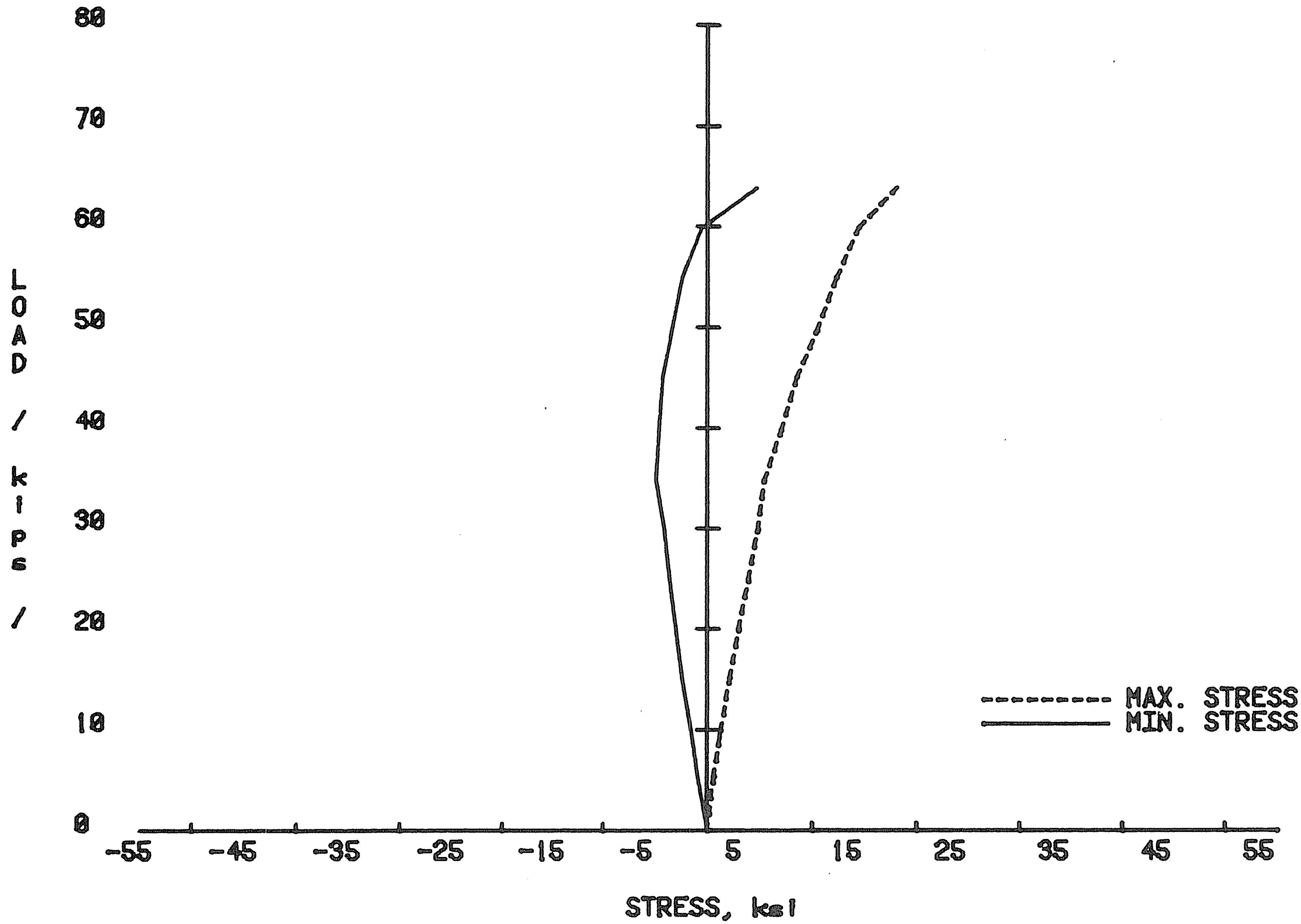


FIGURE F.13 **LOAD VS PRINCIPAL STRESSES, TEST FA-6**

(Measured at Center of Panel Zone)

TEST FA-6

Load, k	Horizontal Stress, ksi	Vertical Stress, ksi	Effective Stress, ksi	Shear Stress, ksi
5.30	-0.29	0.	1.33	0.75
9.90	-0.43	0.10	2.56	1.45
15.10	-0.56	0.27	3.91	2.22
20.00	-0.57	0.53	5.21	2.96
24.90	-0.49	0.78	6.49	3.69
29.80	-0.40	1.10	7.75	4.41
34.70	-0.60	1.10	8.90	5.06
39.70	0.22	2.18	10.15	5.73
45.00	1.08	3.11	11.29	6.32
49.60	2.52	4.53	12.50	6.85
54.90	3.88	5.98	13.61	7.25
59.90	5.91	7.89	14.72	7.44
63.80	10.68	12.02	16.18	6.63

Load, k	Principle Stress, ksi		Principle Strain, Micro Strain		Theta, degrees
	1	2	1	2	
5.30	0.6	-0.9	31.	-38.	-39.5
9.90	1.3	-1.6	62.	-70.	-39.8
15.10	2.1	-2.4	98.	-105.	-39.7
20.00	3.0	-3.0	134.	-135.	-39.8
24.90	3.9	-3.6	171.	-164.	-40.1
29.80	4.8	-4.1	209.	-192.	-40.2
34.70	5.4	-4.9	236.	-224.	-40.3
39.70	7.0	-4.6	290.	-232.	-40.1
45.00	8.5	-4.3	338.	-237.	-40.4
49.60	10.4	-3.4	395.	-225.	-40.8
54.90	12.3	-2.4	447.	-209.	-40.9
59.90	14.4	-0.6	503.	-170.	-41.2
63.80	18.0	4.7	573.	-25.	-42.1

Table F.1 Stress and Strain at Center of Panel Zone

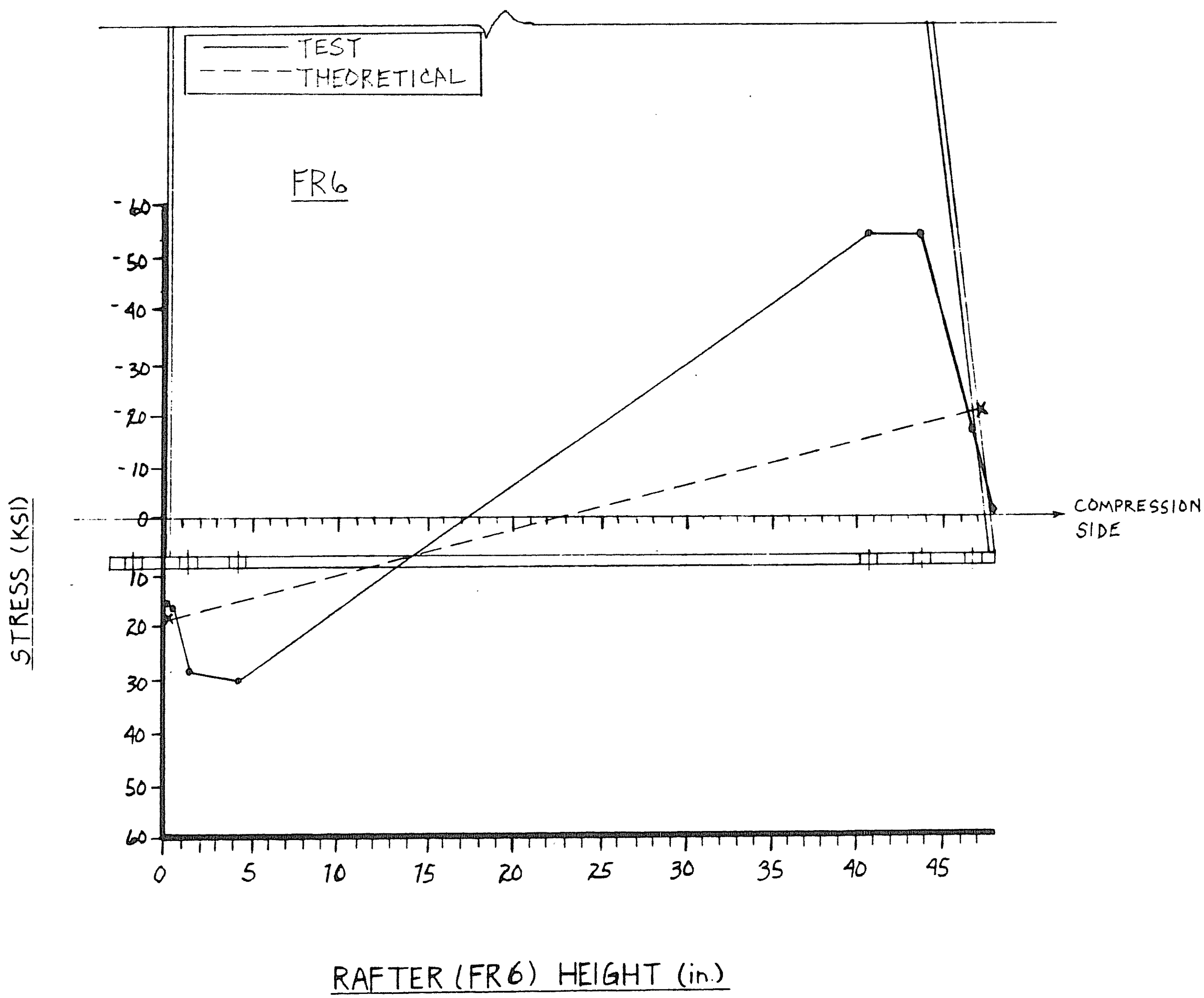


FIGURE F.14 STRESS VARIATION ACROSS RAFTER AT WORKING LOAD, TEST FA-6

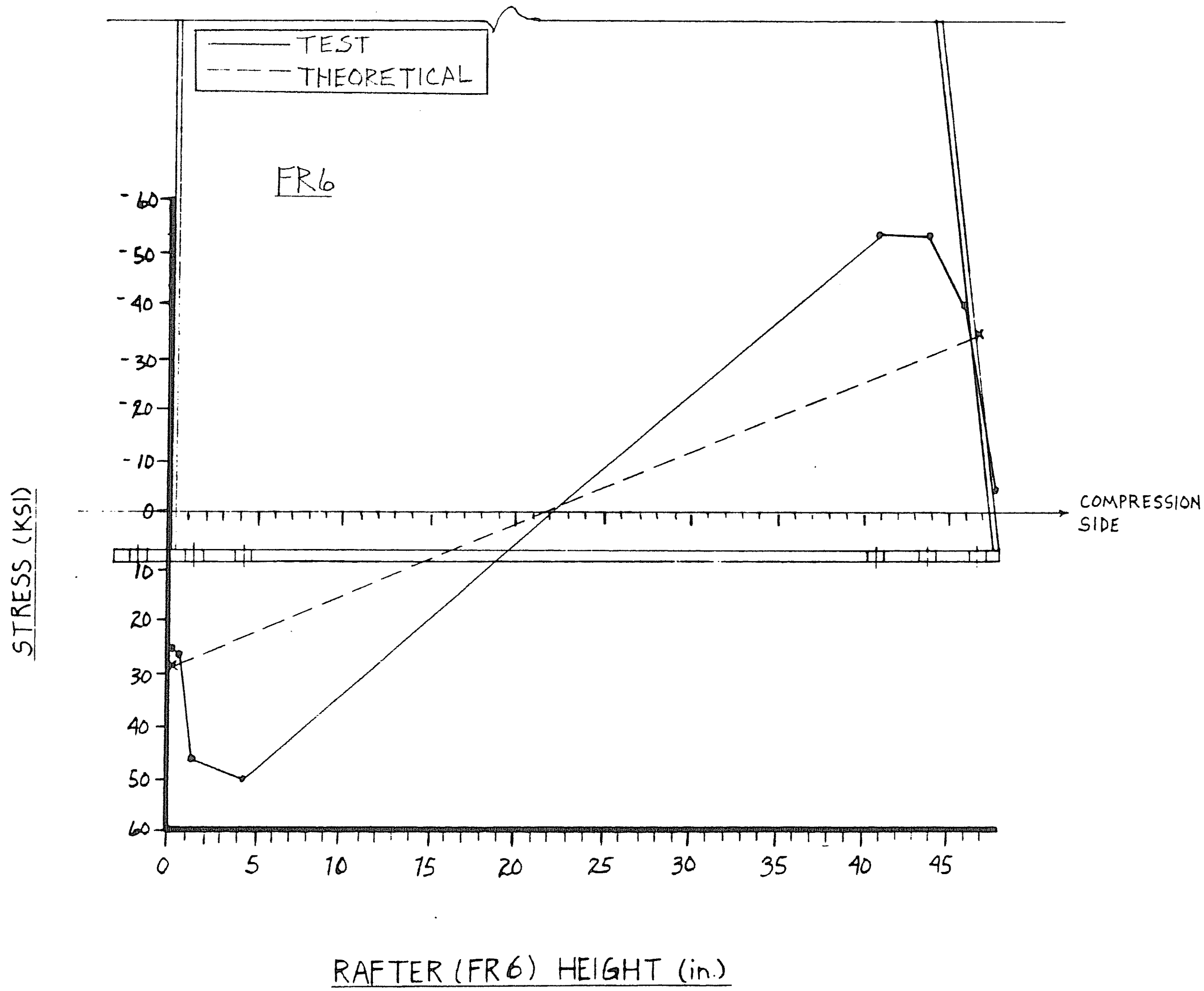


FIGURE F.15 STRESS VARIATION ACROSS RAFTER AT FAILURE LOAD, TEST FA-6

APPENDIX G

FA-7 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: Mesco Knee Test

Test No.: FA-7

Test Date: 8/28/84

Purpose: Test of knee area

Number of Tension Bolts: 4 Bolt Gage (g): 3" Pitch: 3"

Bolt Diameter: 3/4" End Plate Thickness (t): .501"

End Plate Width (w): 8.0" End Plate Length (de): 30.0"

Panel Zone Web Plate Thickness: .150"

Initial Out-of-Straightness at the Center of Panel Zone: .29"

Pretension Force per Bolt: 28 kips

Failure Load, (Total Load): 26.4 kips

Failure Mode: Buckling of Panel zone

Predicted Failure Loads:

Method: Bending and compr. of column Total Load: 50.39 kips

Method: Buckling of panel, TFA Total Load: 26.55 kips

Buckling of panel, No TFA 7.6 kips

Discussion:

- Prior to a test load of 5 kips the observed chord deflections agreed closely with the theoretical prediction. Also, the force in the southwest compression bolt remained close to the pretension force while the force in the northeast compression bolt increased in a linear manner with the application of test load.

- At 5 kips, the chord deflections began to increase at a greater rate with respect to load than was predicted; the load deflection relationship became curvilinear.

- At 12 kips, flaking of the whitewash was observed indicating yielding in the compression flange of the column in the area immediately adjacent to the rafter. Similar yielding was observed of the rafter compression flange at 15.1 kips of test load.

- With the application of 21.0 kips of test load, buckling of the panel zone began. In addition, flaking of the whitewash was observed on the column compression flange along the length from the rafter to the web splice. The force in the northeast compression bolt began to increase at a greater rate at this load.

- At 24 kips of test load, noticeable end plate seperation was observed along the length between the tension and compression bolts. Buckling of the panel became more severe. In addition, the force in the southwest compression bolt began to increase substantially above the pretension force.

- At 26.4 kips, the panel buckling became to severe that the specimen was unable to sustain any load increase and the test was terminated. The end plate separation at the center of the panel zone was 3/8" at this point.

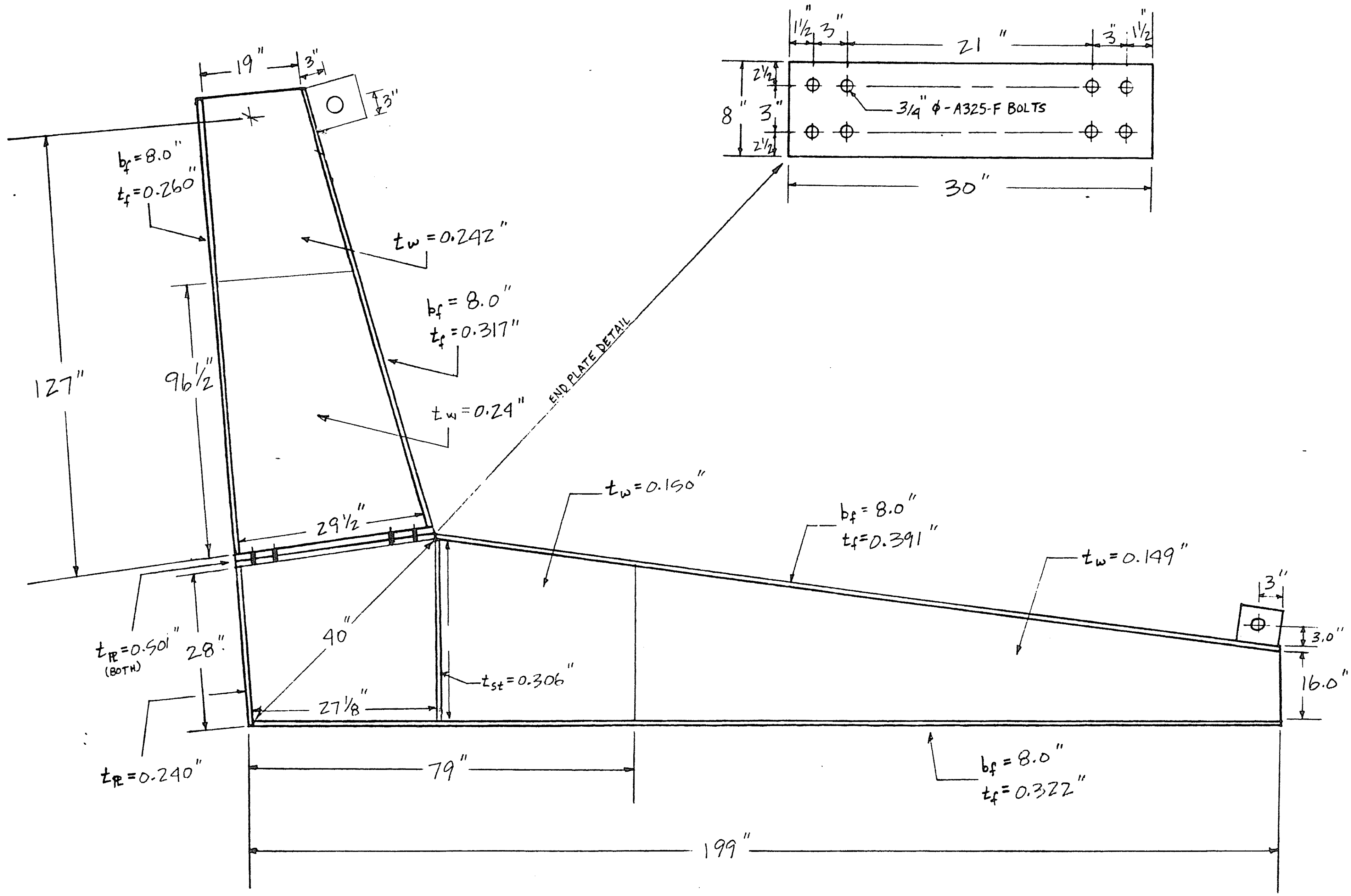


FIGURE G.1 SPECIMEN DETAILS, TEST FA-7

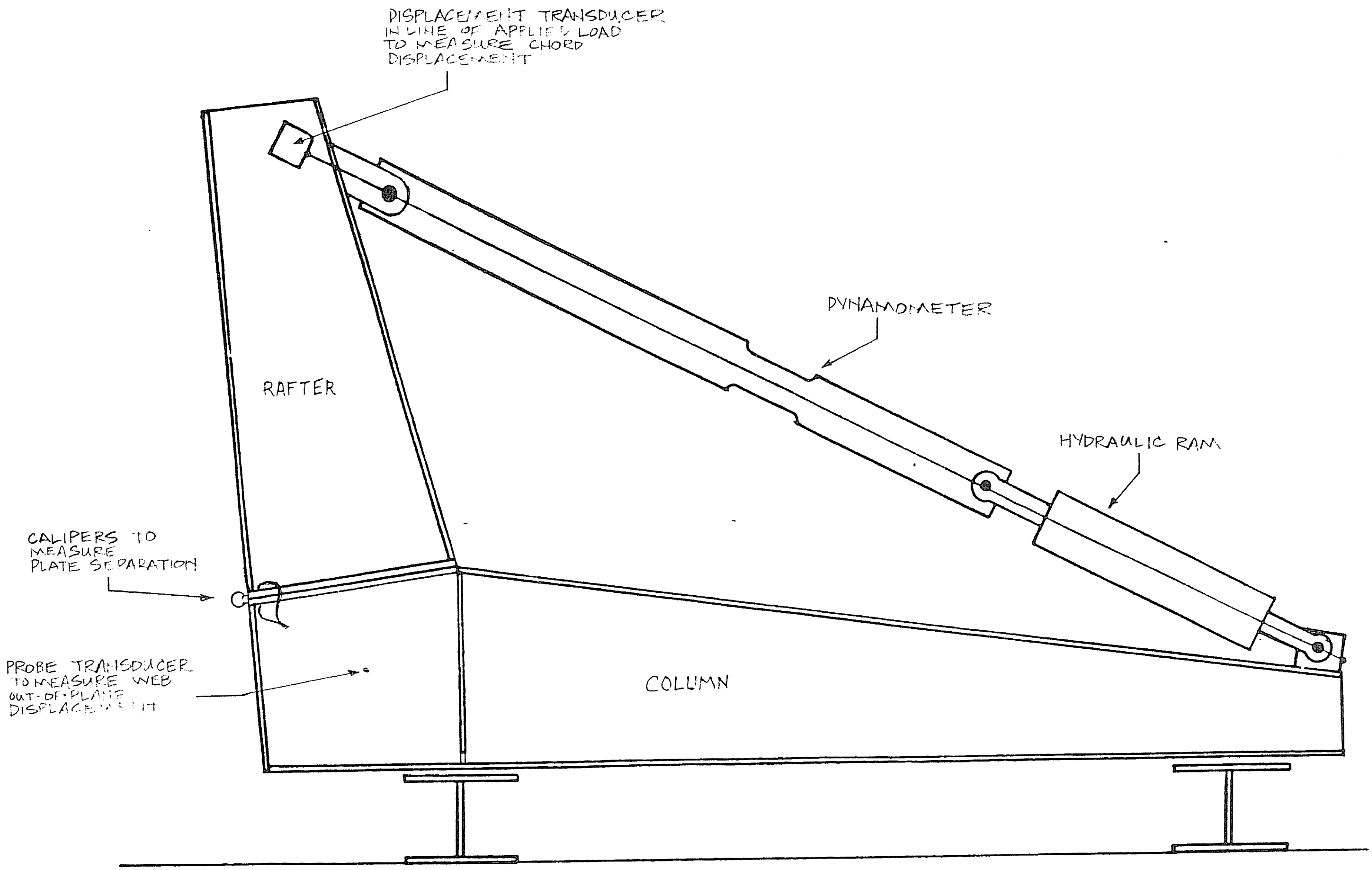


FIGURE G.2 TEST SET-UP, TEST FA-7

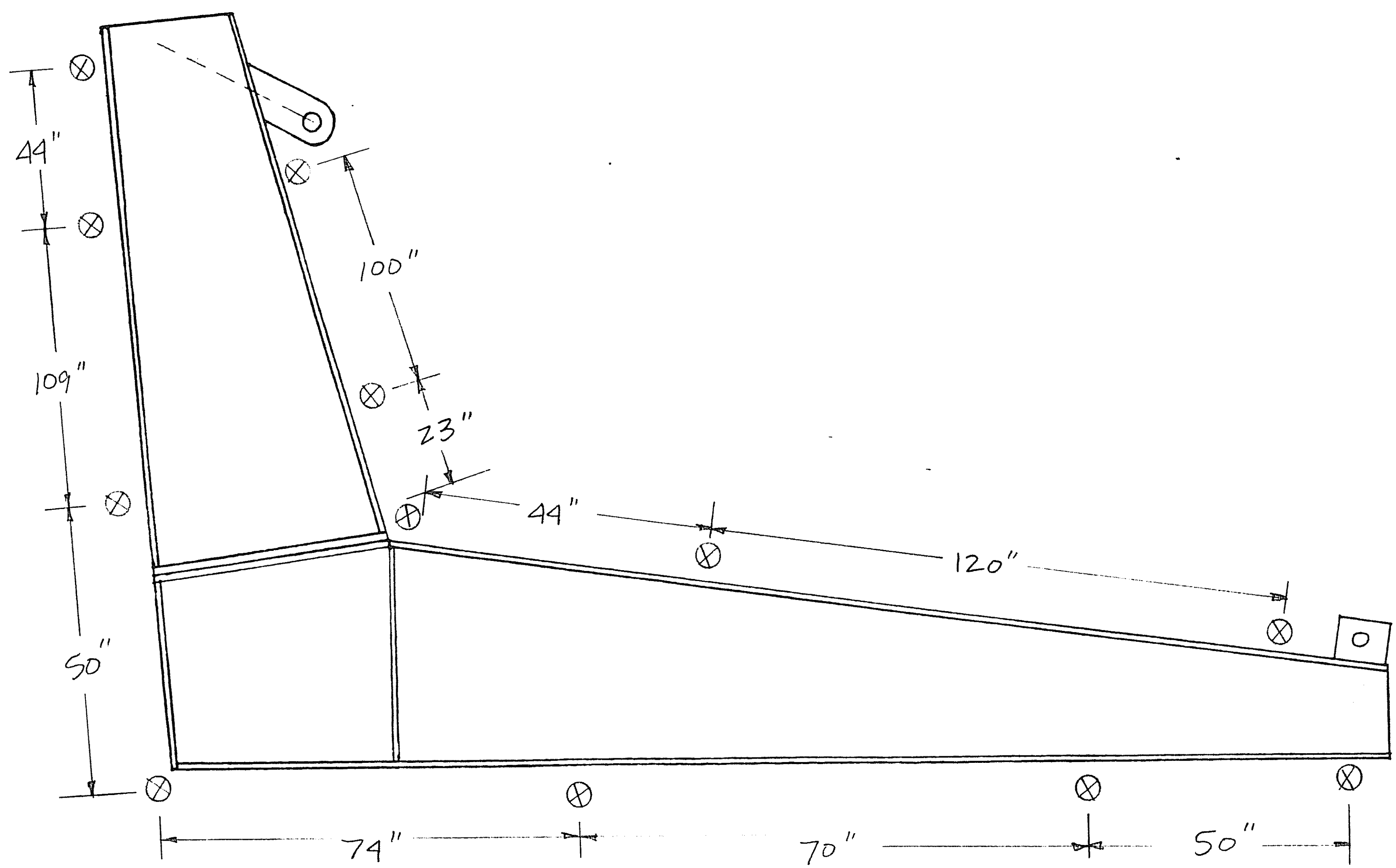


FIGURE G.3 LATERAL BRACE LOCATIONS, TEST FA-7

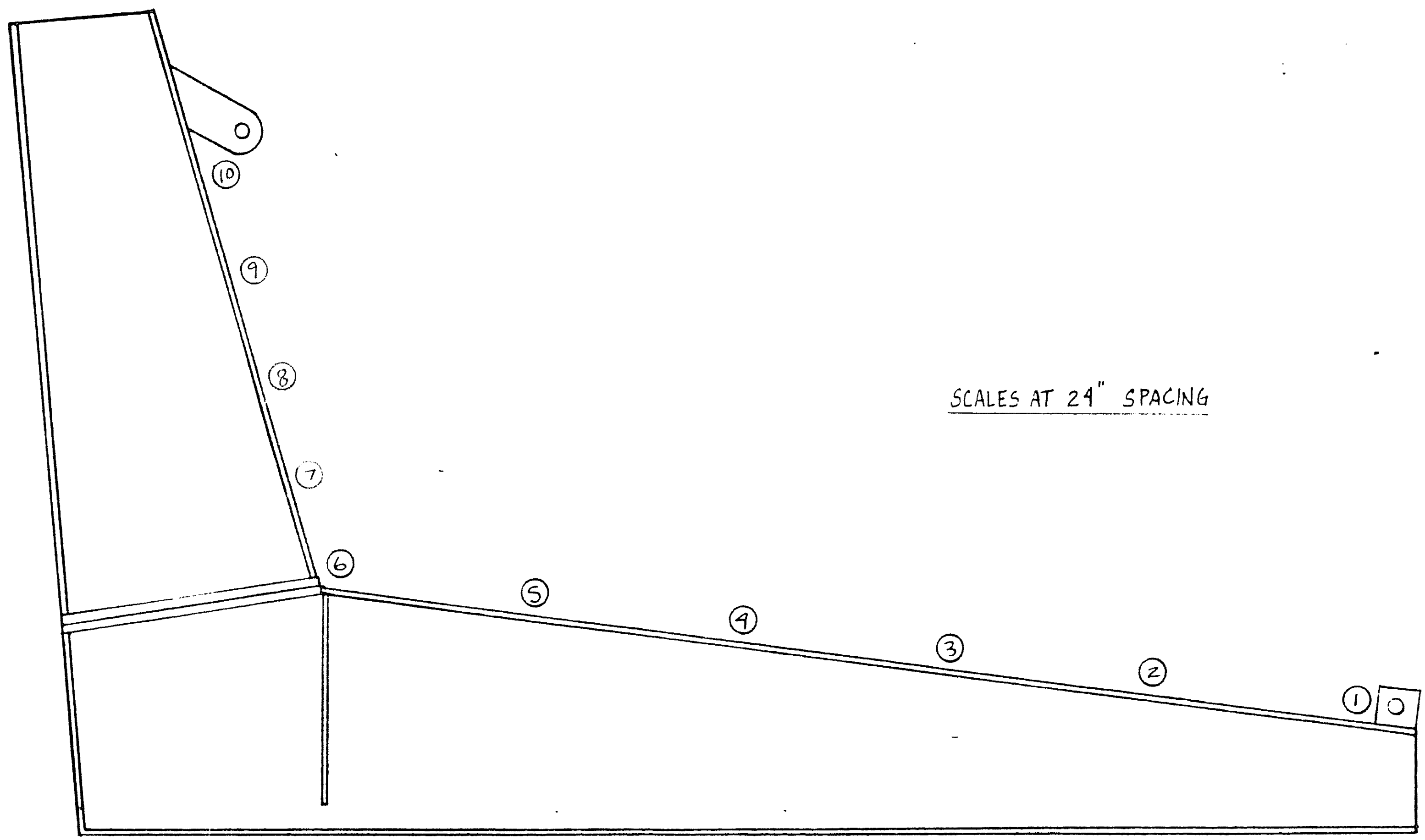


FIGURE G.4 LATERAL DISPLACEMENT SCALE LOCATION, TEST FA-7

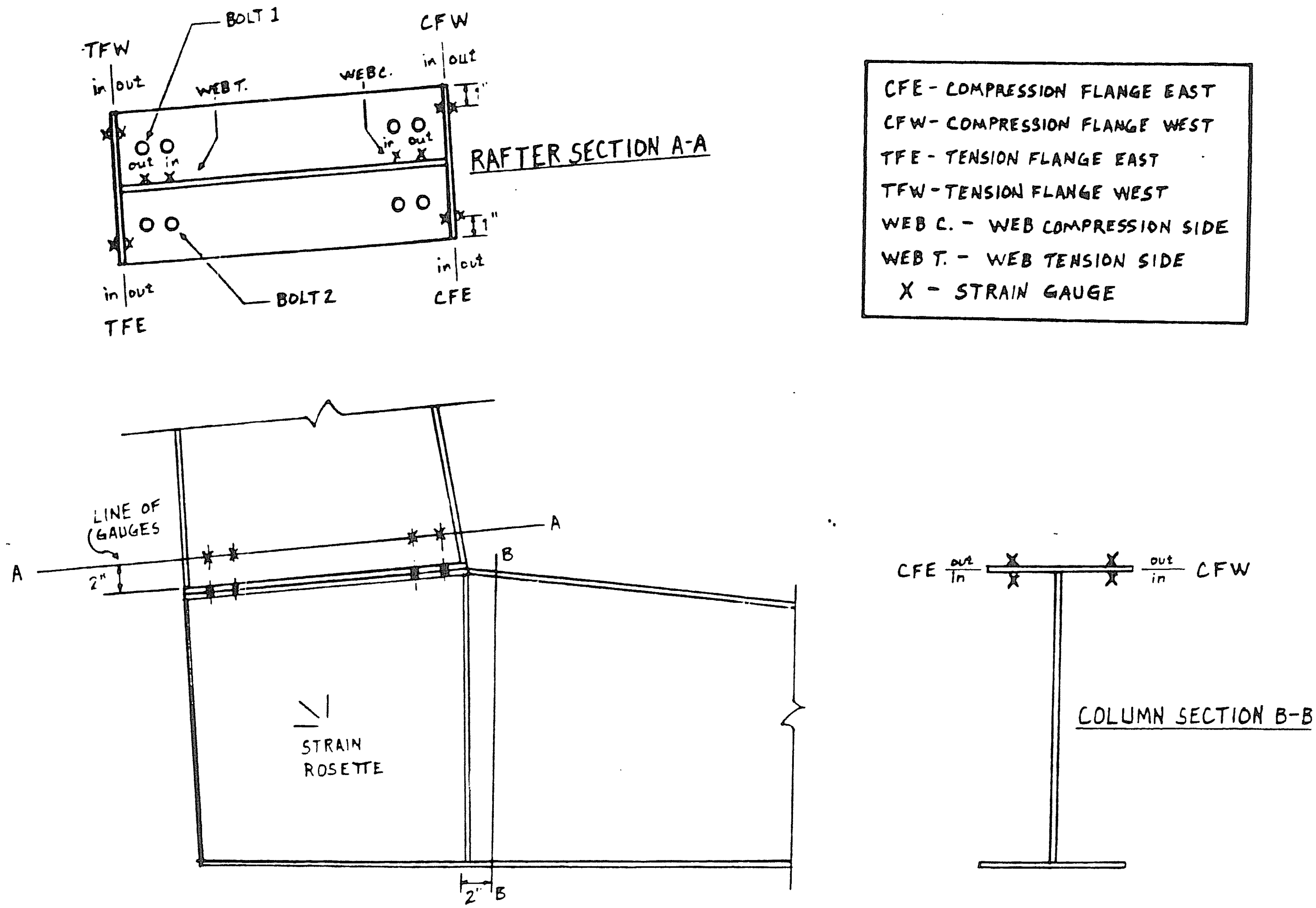


FIGURE G.5 STRAIN GAGE LOCATIONS, TEST FA-7

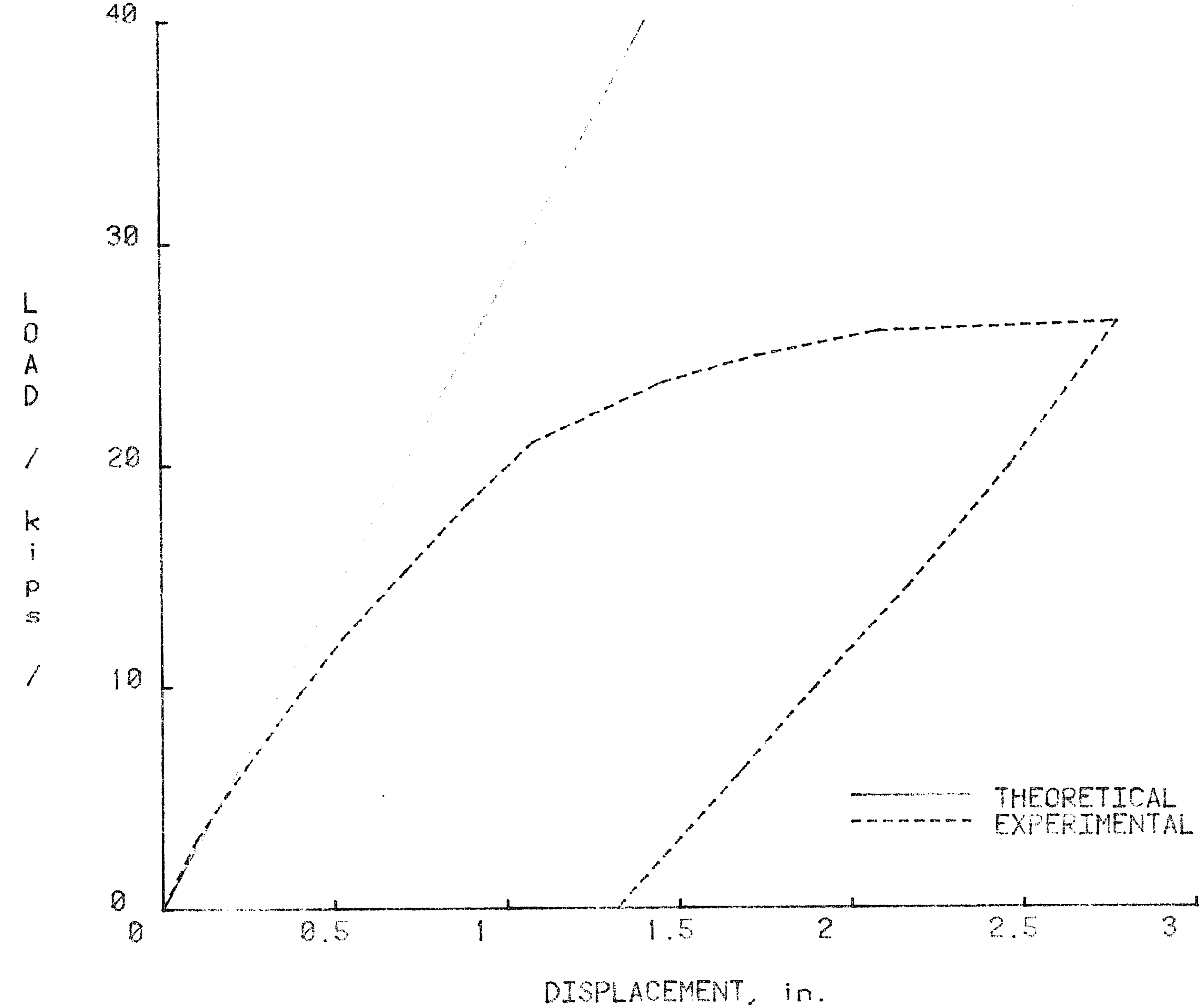


FIGURE G.6 LOAD VS. CHORD DISPLACEMENT, TEST FA-7

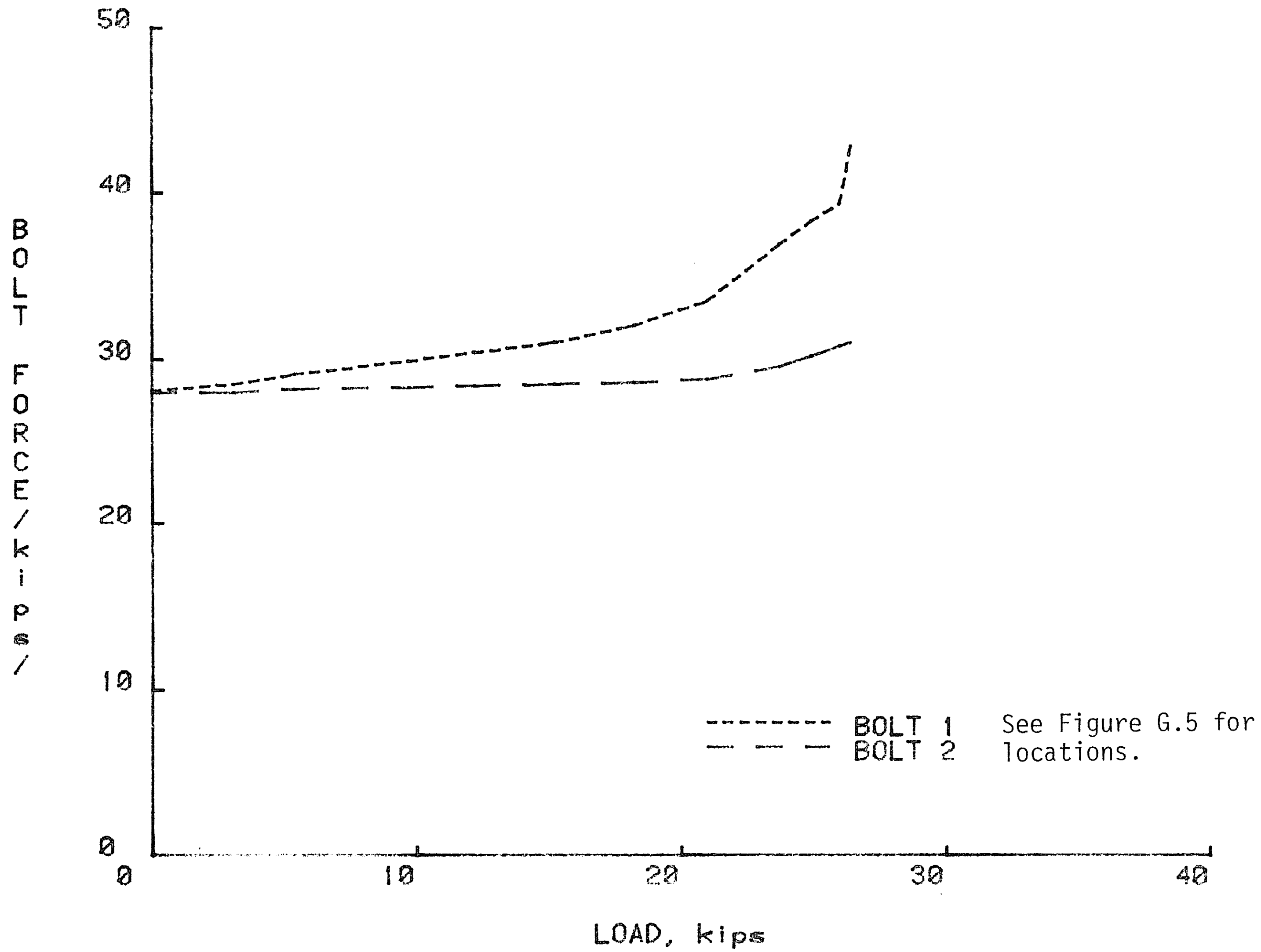


FIGURE G.7 BOLT FORCE VS. LOAD, TEST FA-7

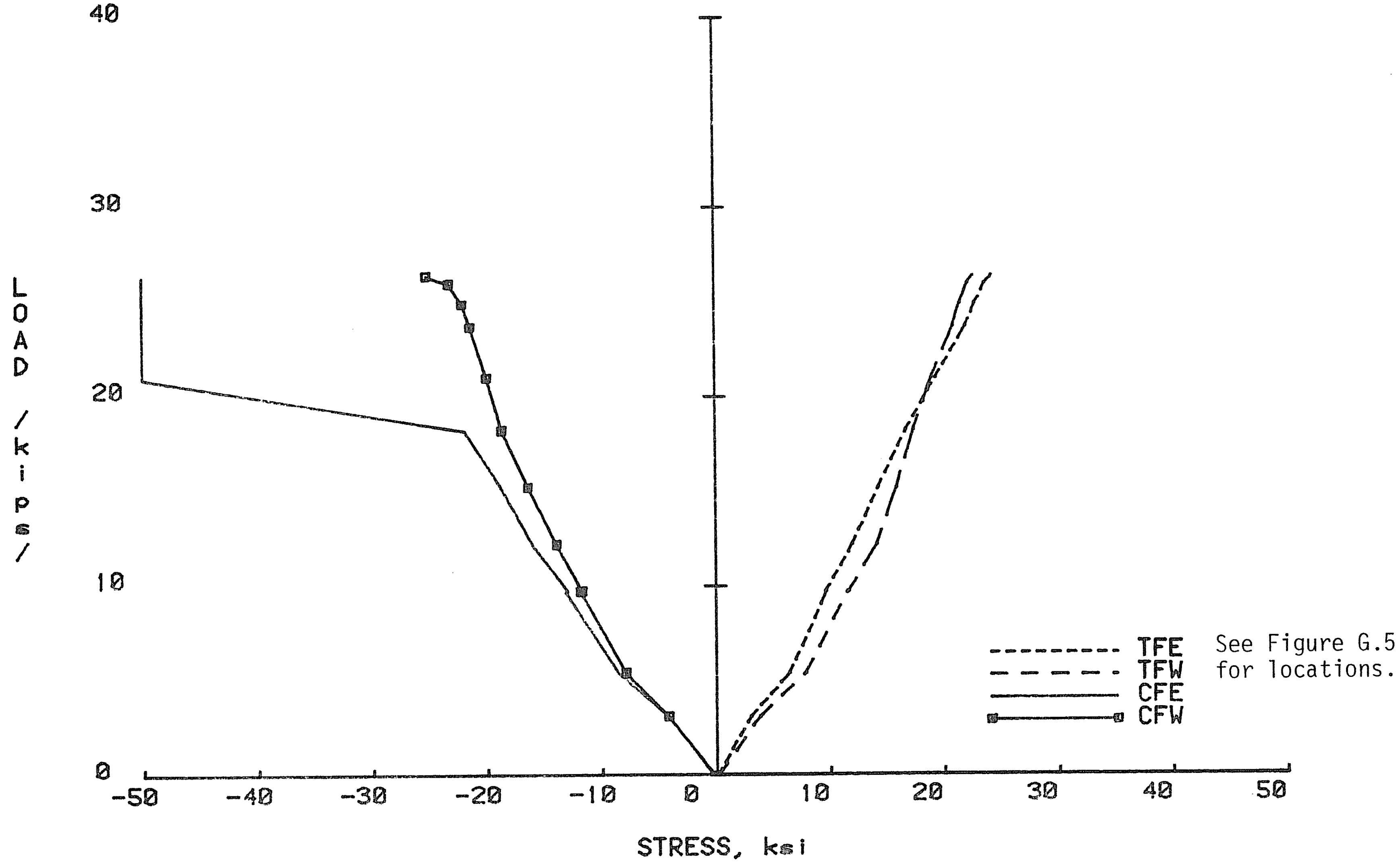


FIGURE G.8 **LOAD VS. RAFTER FLANGE STRESSES, TEST FA-7**

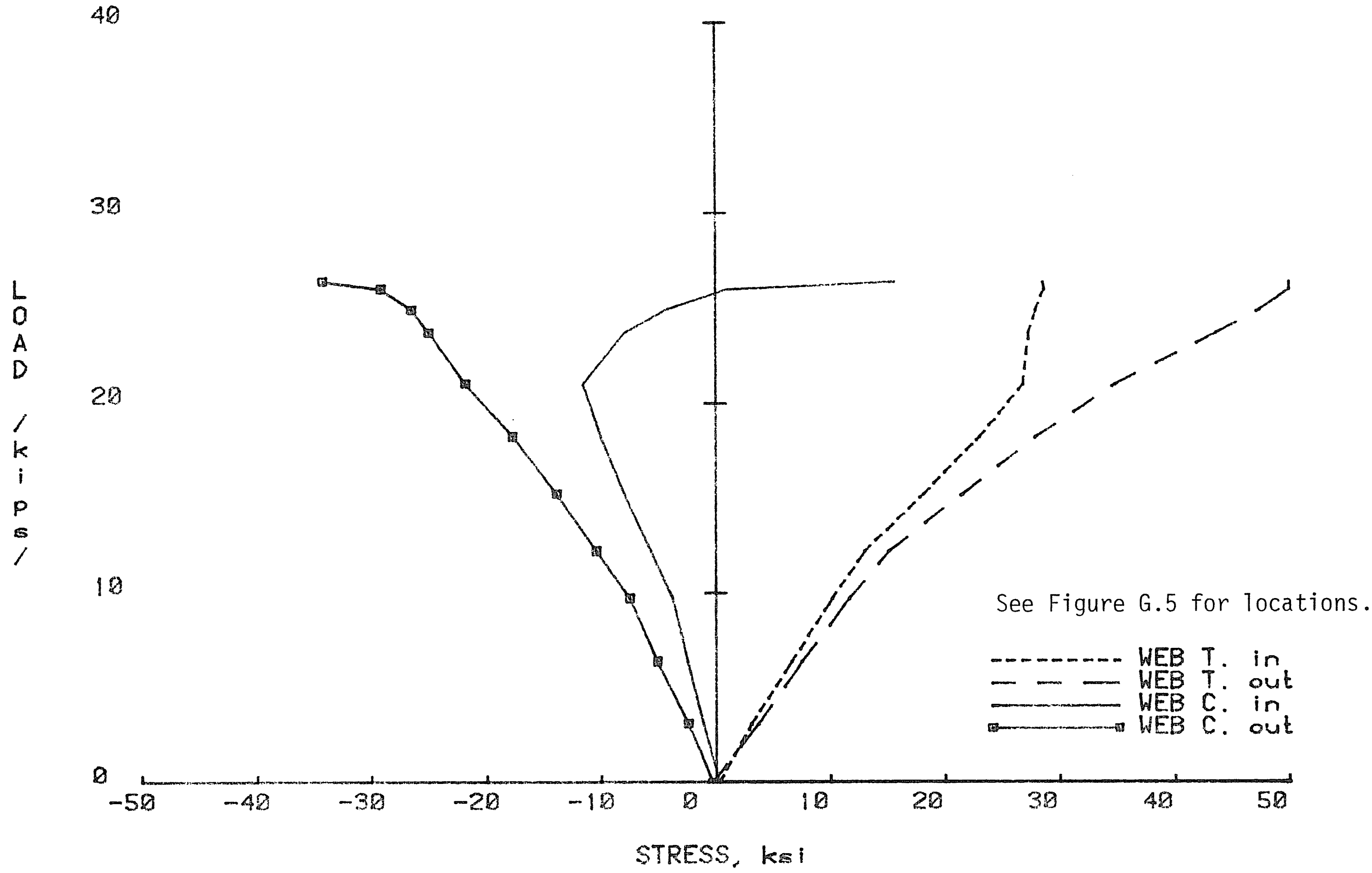


FIGURE G.9 **LOAD VS. RAFTER WEB STRESS, TEST FA-7**

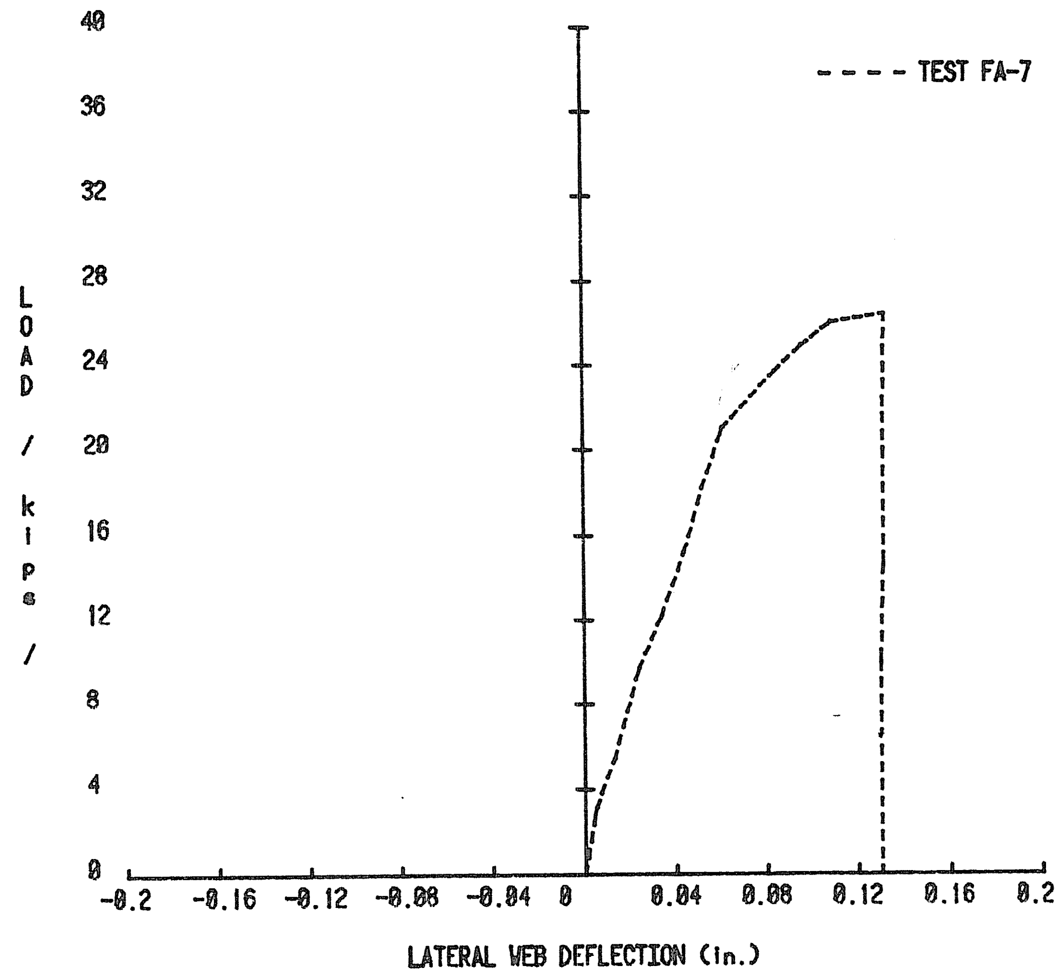


FIGURE G.10 LOAD VS CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-7

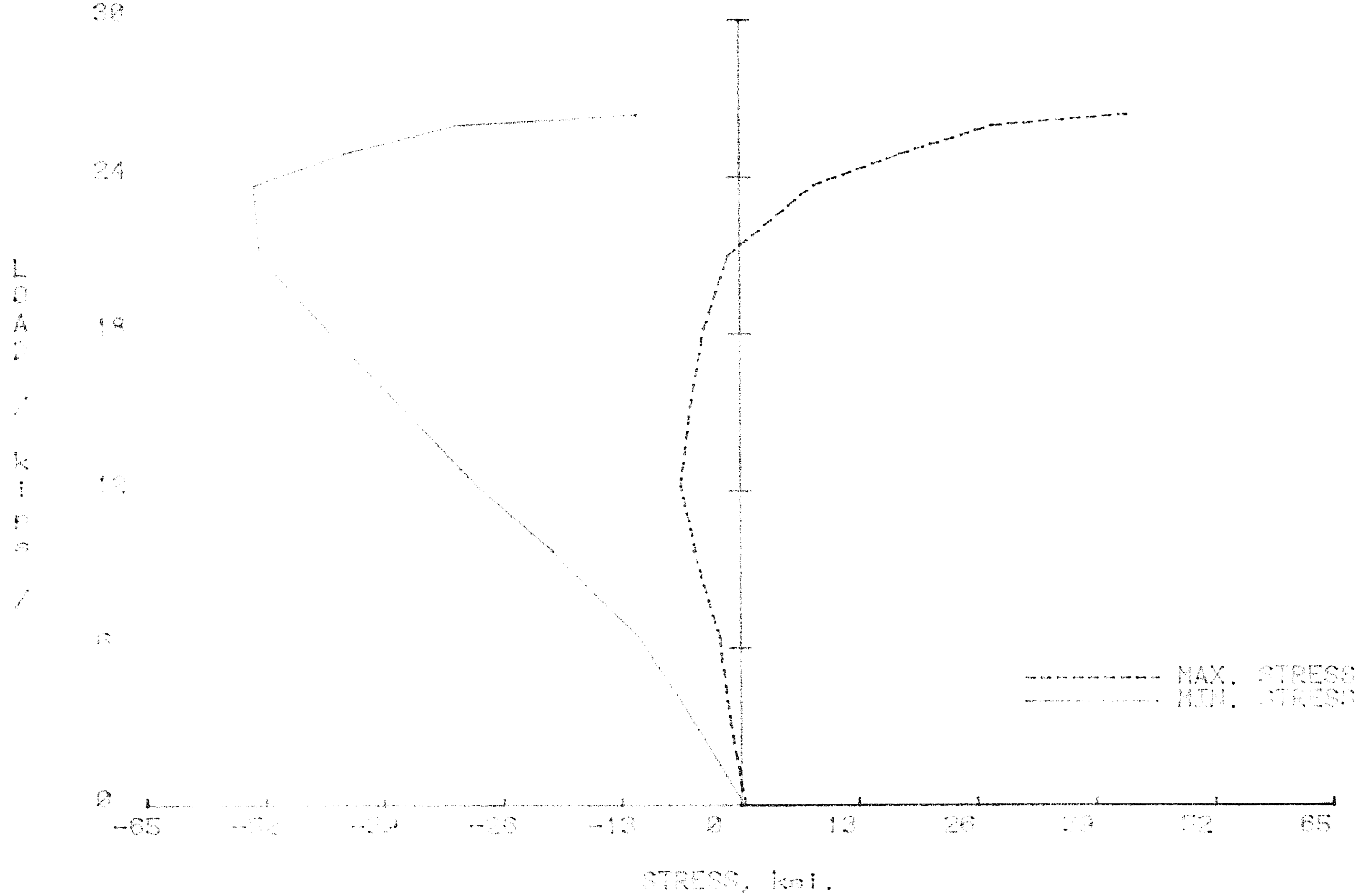


FIGURE G.11 LOAD VS. PRINCIPAL STRESSES, TEST FA-7
(Measured at Center of Panel Zone)

test FA- 7

load	horizontal stress	verticle stress	effective stress	shear stress
0.	0.37	0.30	0.39	[illegible]
[illegible]	-2.74	-3.03	4.54	1.94
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	-12.43	[illegible]	[illegible]
9.70	[illegible]	-15.40	16.06	7.82
[illegible]	-15.04	-19.10	26.15	10.53
[illegible]	[illegible]	-23.31	34.74	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	-7.04	34.47	31.43
[illegible]	6.07	2.99	50.90	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	38.45	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	-4.00	[illegible]	19.75	[illegible]
[illegible]	[illegible]	4.93	13.01	[illegible]

LOAD	PRINCIPLE STRESS 1	PRINCIPLE STRESS 2	PRINCIPLE STRAIN 1	PRINCIPLE STRAIN 2	THETA
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE G.1 Stress and Strain at Center of Panel Zone

APPENDIX H

FA-8 TEST RESULTS

MESCO KNEE TEST SUMMARY

Project: Mesco Knee Test

Test No.: FA-8

Test Date: 9/28/84

Purpose: Test of Knee Area

Number of Tension Bolts: 4 Bolt Gage (g): 3" Pitch: 3"

Bolt Diameter: 3/4" End Plate Thickness (t): 0.500"

End Plate Width (w): 8.0" End Plate Length (de):

Panel Zone Web Plate Thickness: .151

Initial Out-of-Straightness at the Center of Panel Zone: .28"

Pretension Force per Bolt: 28 kips

Failure Load, (Total Load): 29.3 kips

Failure Mode: Buckling of Panel

Predicted Failure Loads:

Method: Bending of Column Total Load: 45.3 kips

Method: Buckling of Panel TFA Total Load: 28.96 kips

Buckling of Panel No TFA 10.00 kips

Discussion:

- Prior to a test load of 6 kips the observed chord deflections agreed closely with the theoretical prediction. Also, the lateral web displacement at the center of the panel zone increased gradually in a linear manner during this interval. Bolt forces changes only slightly.

- At 6 kips of test load, the chord deflections began to increase at a greater rate with respect to load. This increase in chord deflections was accompanied by an increase in the rate of lateral deflection of the panel zone. Additionally, the northeast tension bolt continued its gradual increase while the southwest bolt began a gradual decrease.

- At 18.1 kips, yielding was indicated by flaking of the whitewash. This flaking occurred on the compression flanges of both the rafter and column immediately adjacent to the knee.

- At 20.8 kips, buckling in the panel zone was observed. The chord deflections at this load began to increase at a substantially greater rate with respect to load. In addition, the force in the northeast tension bolt increased greatly while that in the southwest bolt continued its gradual decrease.

- At 24.0 kips, yielding, indicated by flaking of the whitewash, increased on the rafter and column compression flanges. Also, slight flaking of the whitewash was observed on the tension flange of the rafter. Plate separation was also visible at this load.

- With the application of 29.3 kips of test load buckling of the panel zone was so severe that the knee lost its structural integrity and no further loading was possible. Yield lines were visible in the whitewash across the entire panel zone, continuously along the column compression flange from the knee to the web splice and on the tension flange of the rafter.

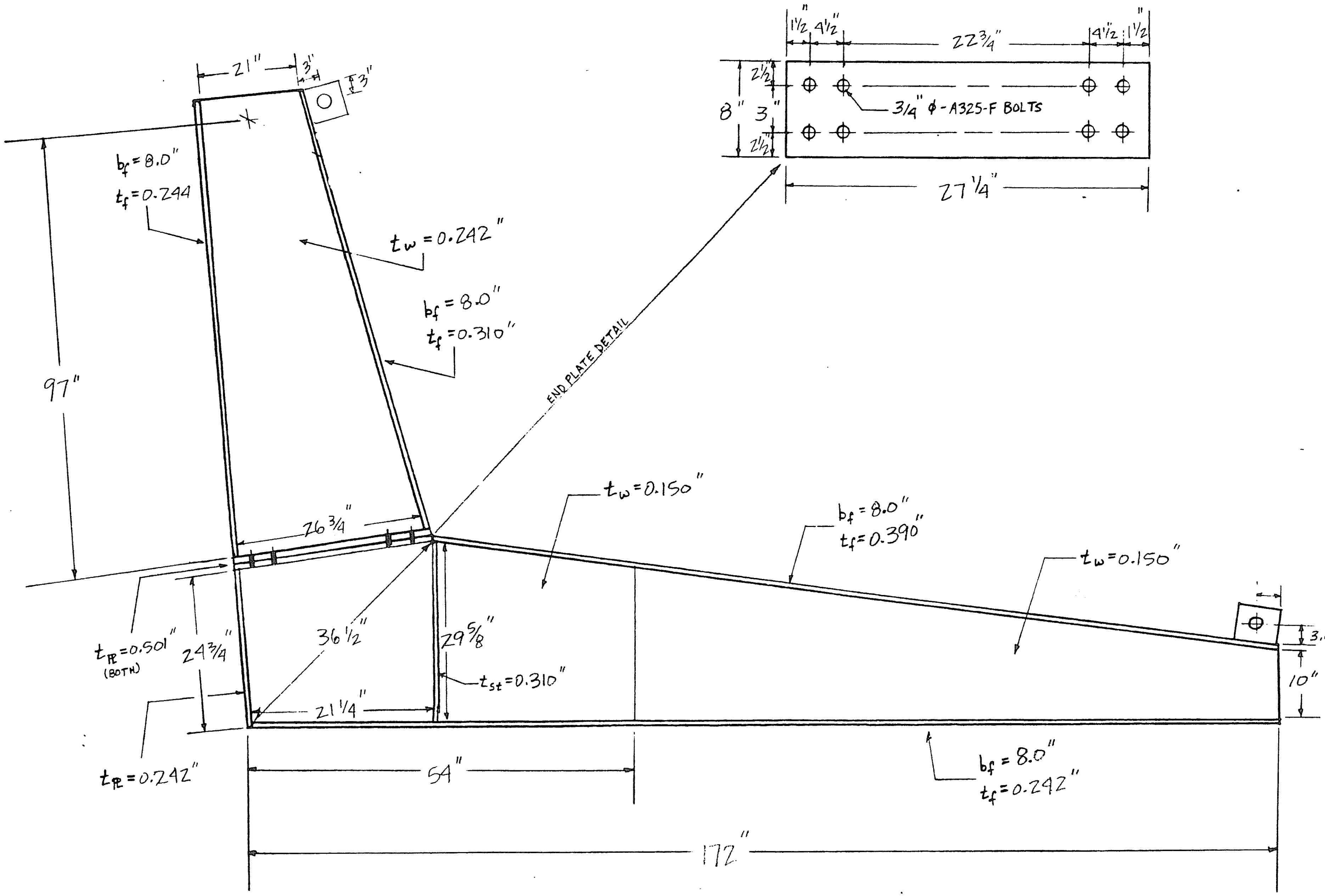


FIGURE H.1 SPECIMEN DETAILS, TEST FA-8

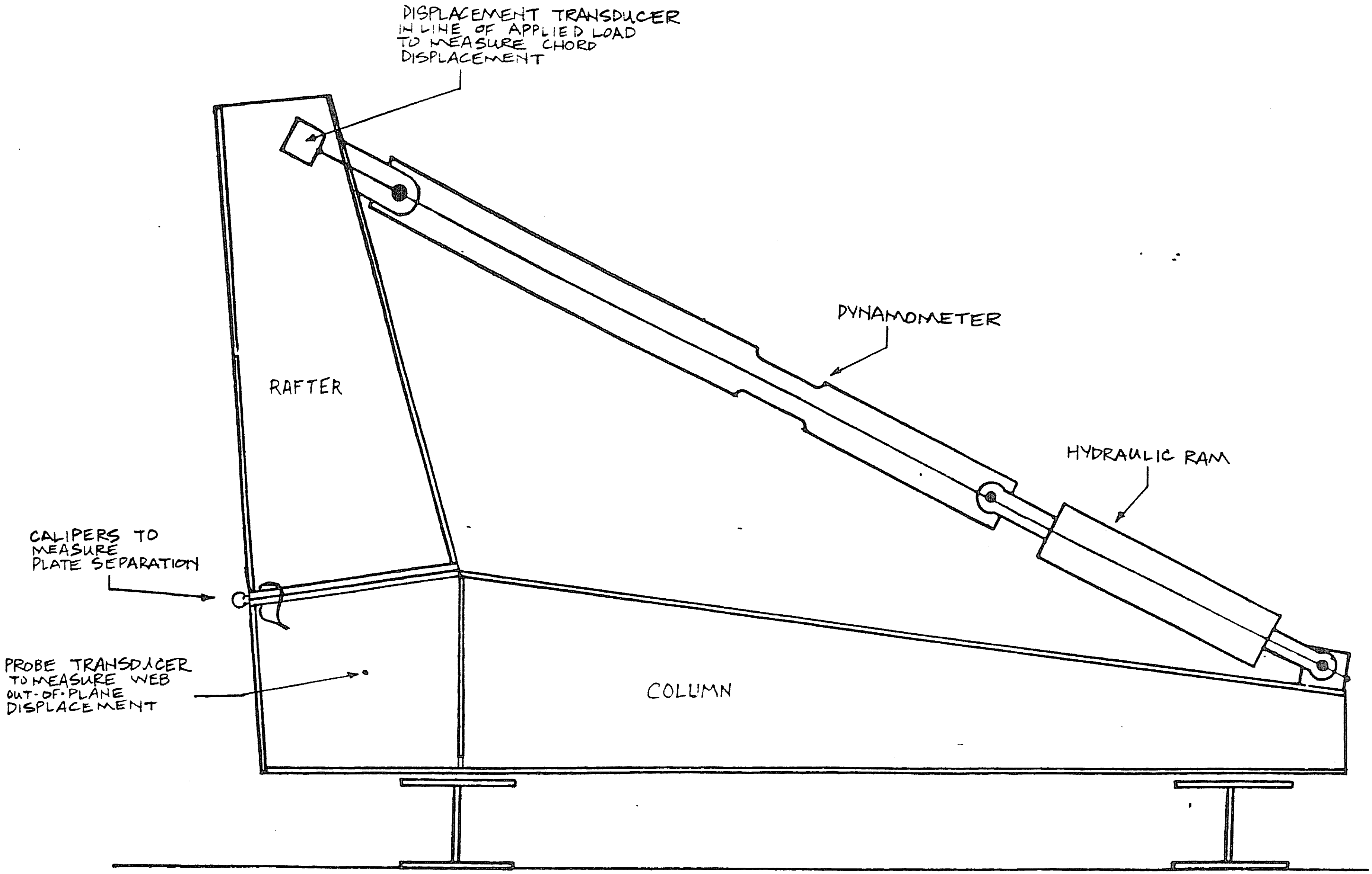


FIGURE H.2 TEST SET-UP, TEST FA-8

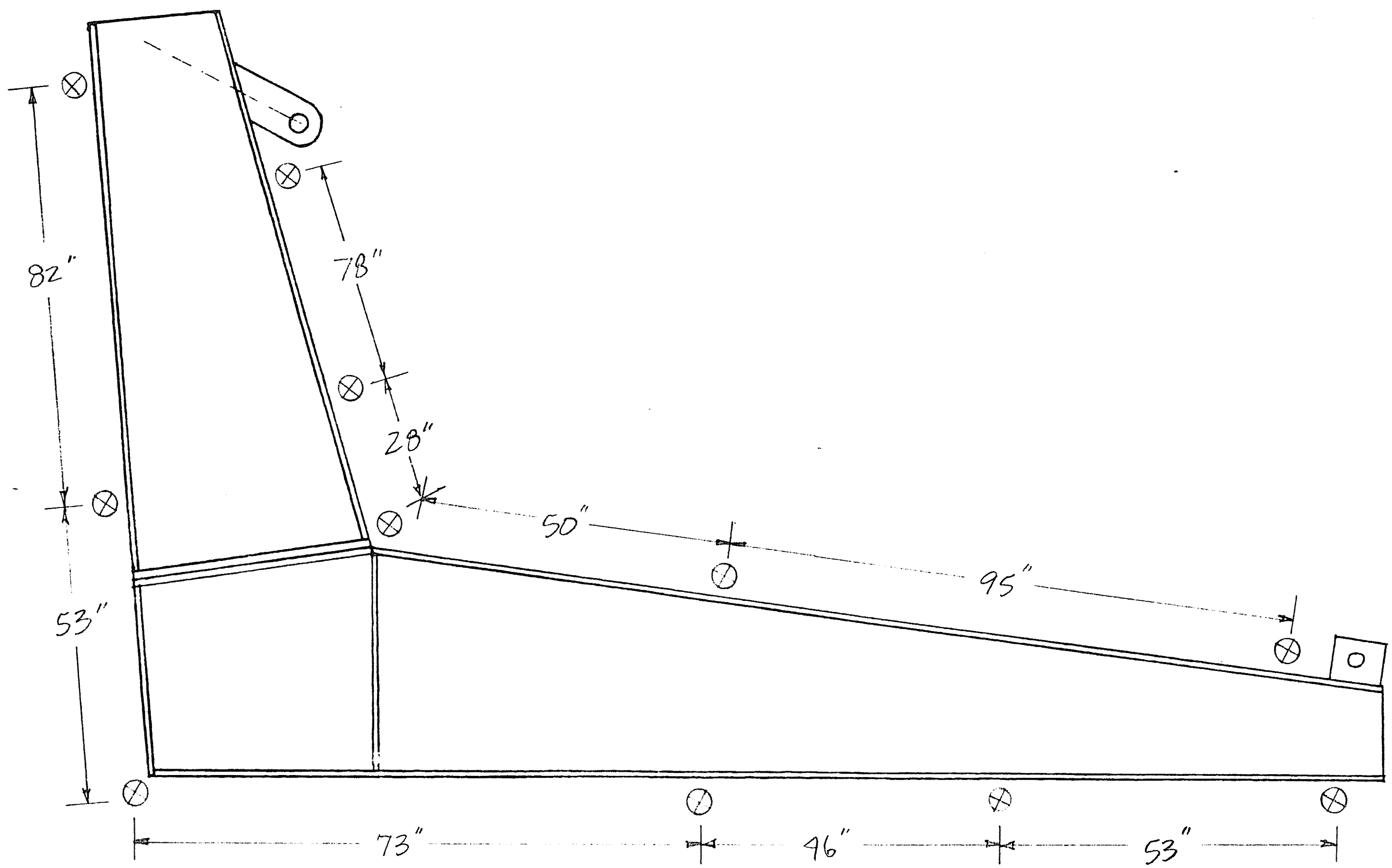


FIGURE H.3 LATERAL BRACE LOCATIONS, TEST FA-8

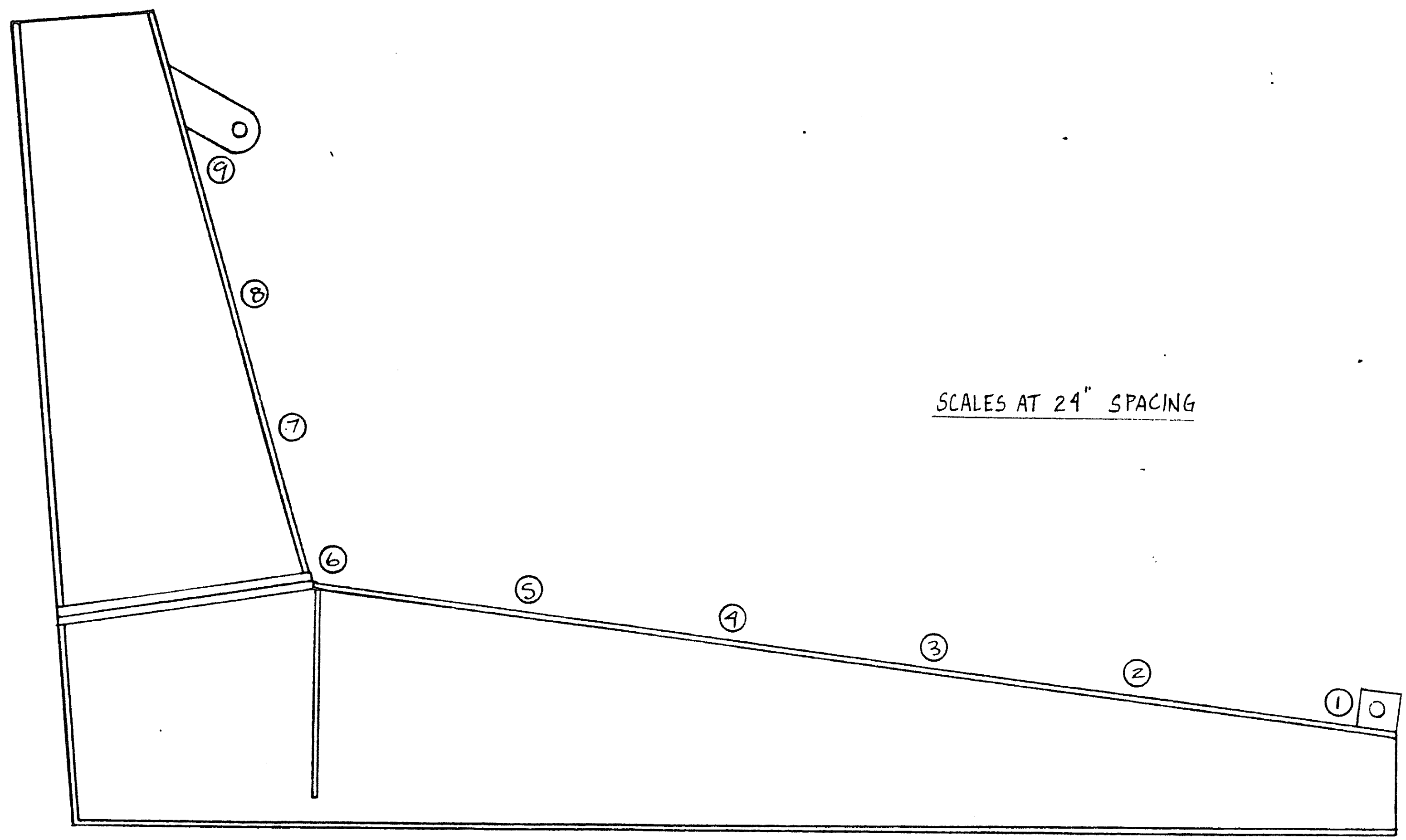


FIGURE H.4 LATER DISPLACEMENT SCALE LOCATIONS, TEST FA-8

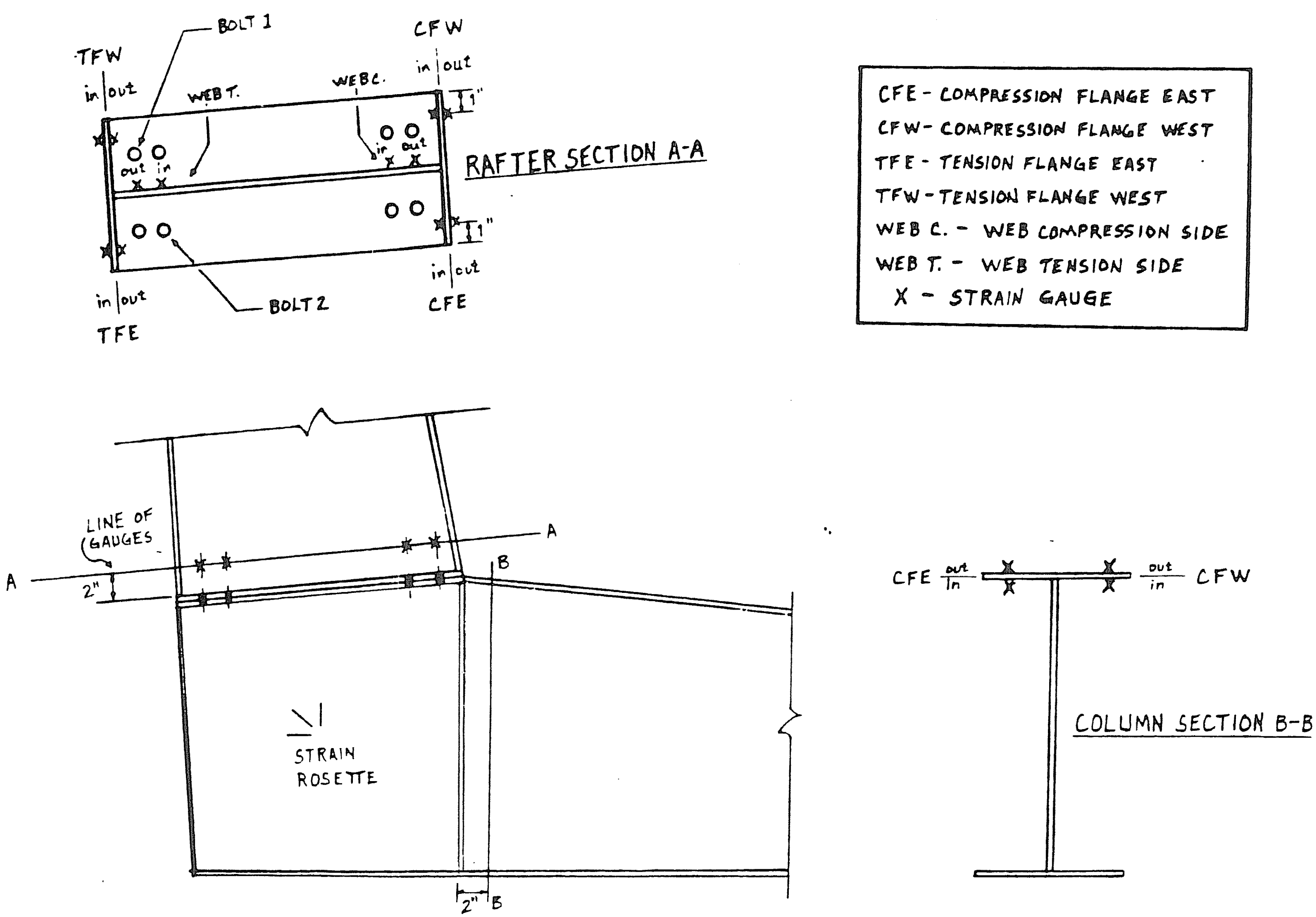


FIGURE H.5 STRAIN GAGE LOCATIONS, TEST FA-8

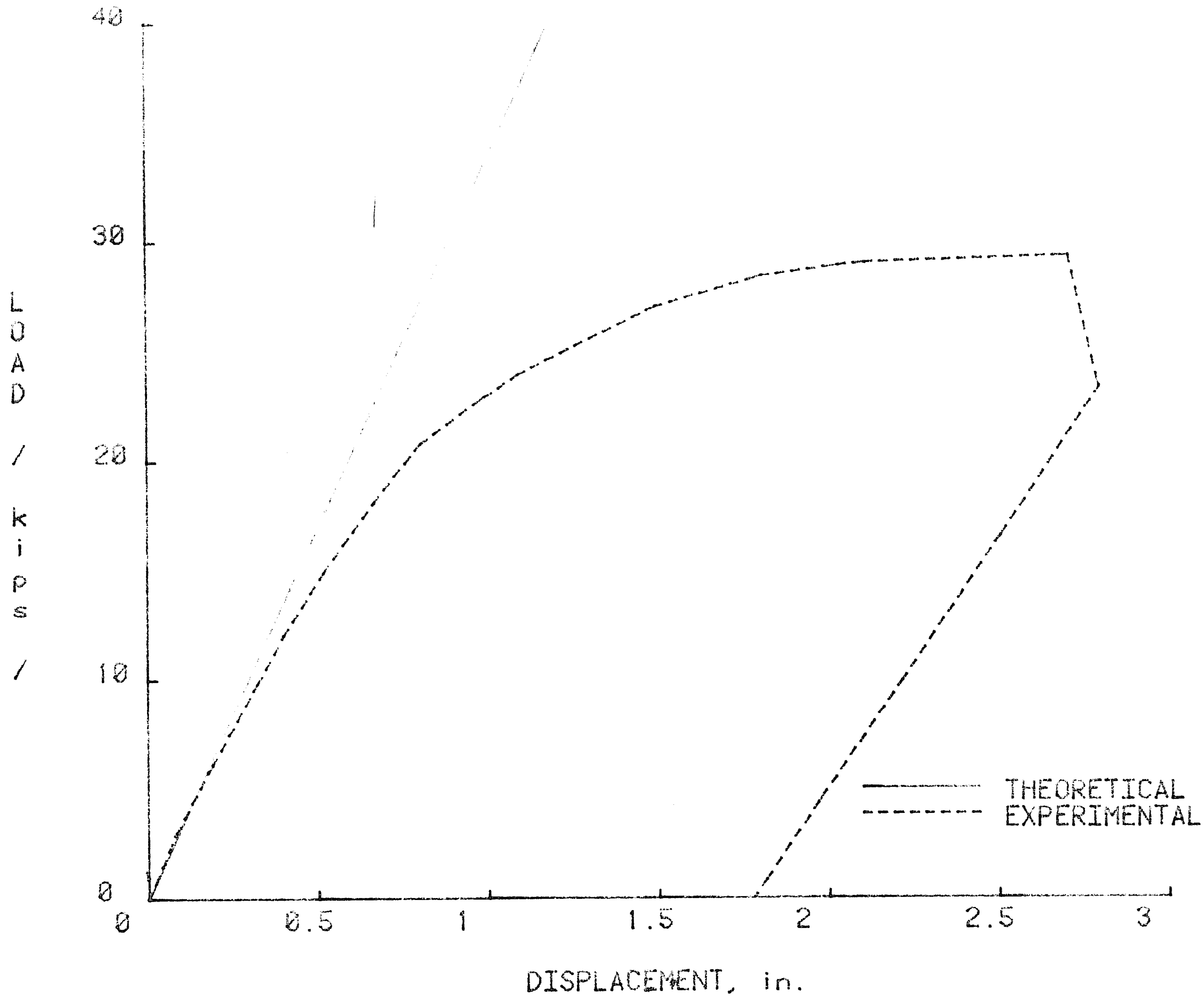


FIGURE H.6 LOAD VS. CHORD DISPLACEMENT, TEST FA-8

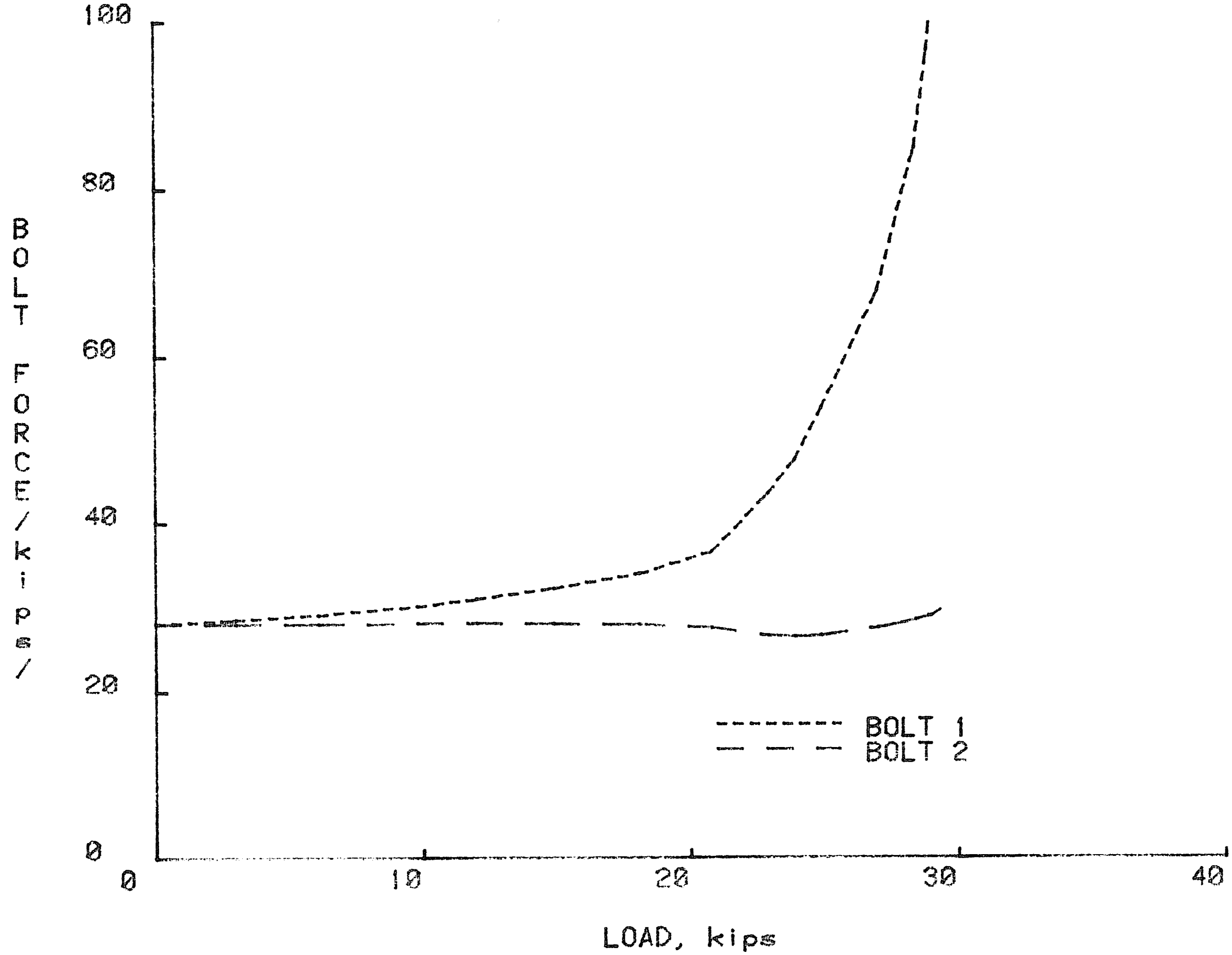


FIGURE H.7 BOLT FORCE VS. LOAD, TEST FA-8

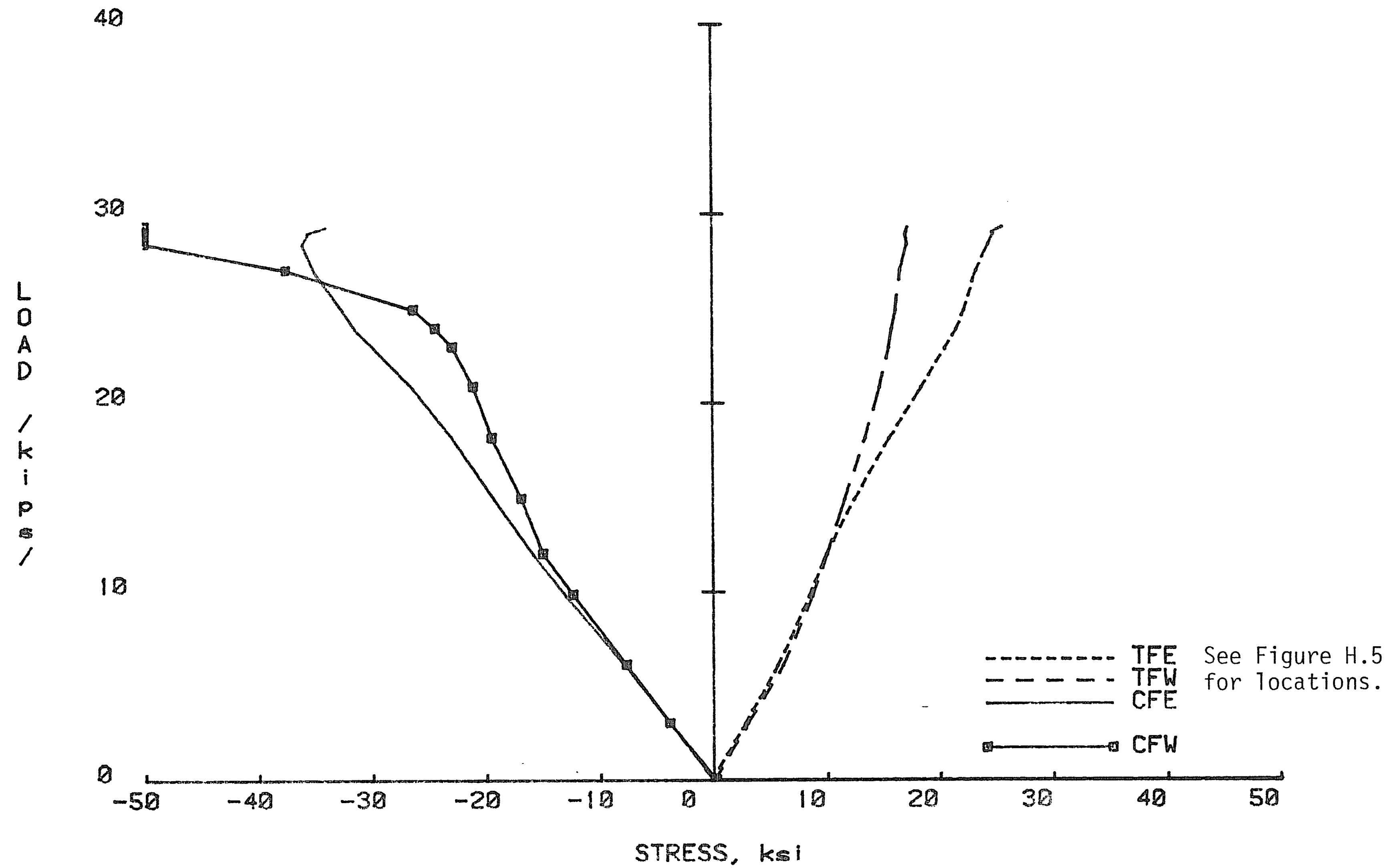


FIGURE H.8 **LOAD VS. RAFTER FLANGE STRESSES, TEST FA-8**

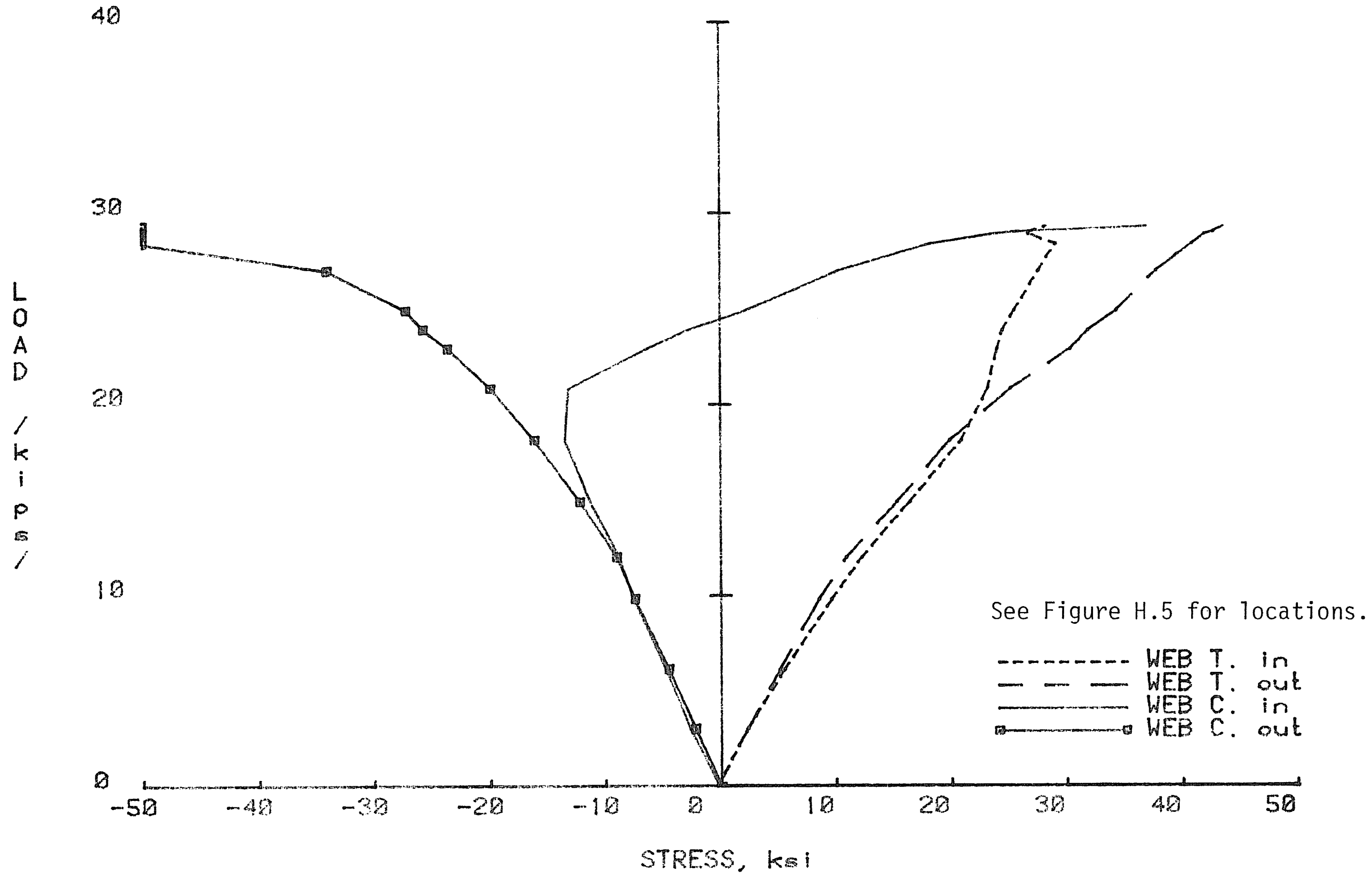


FIGURE H.9 **LOAD VS. RAFTER WEB STRESS, TEST FA-8**

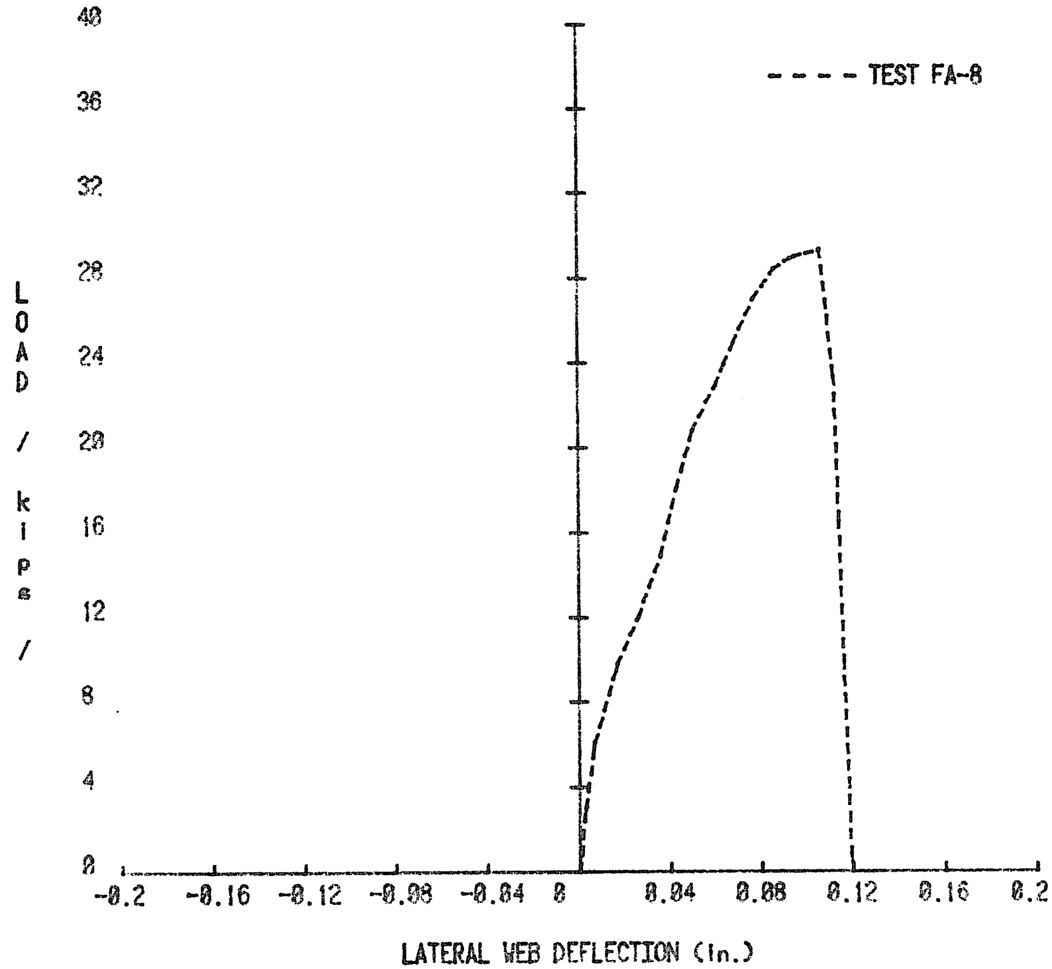


FIGURE H.10 LOAD VS CENTER OF PANEL ZONE DEFLECTIONS, TEST FA-8

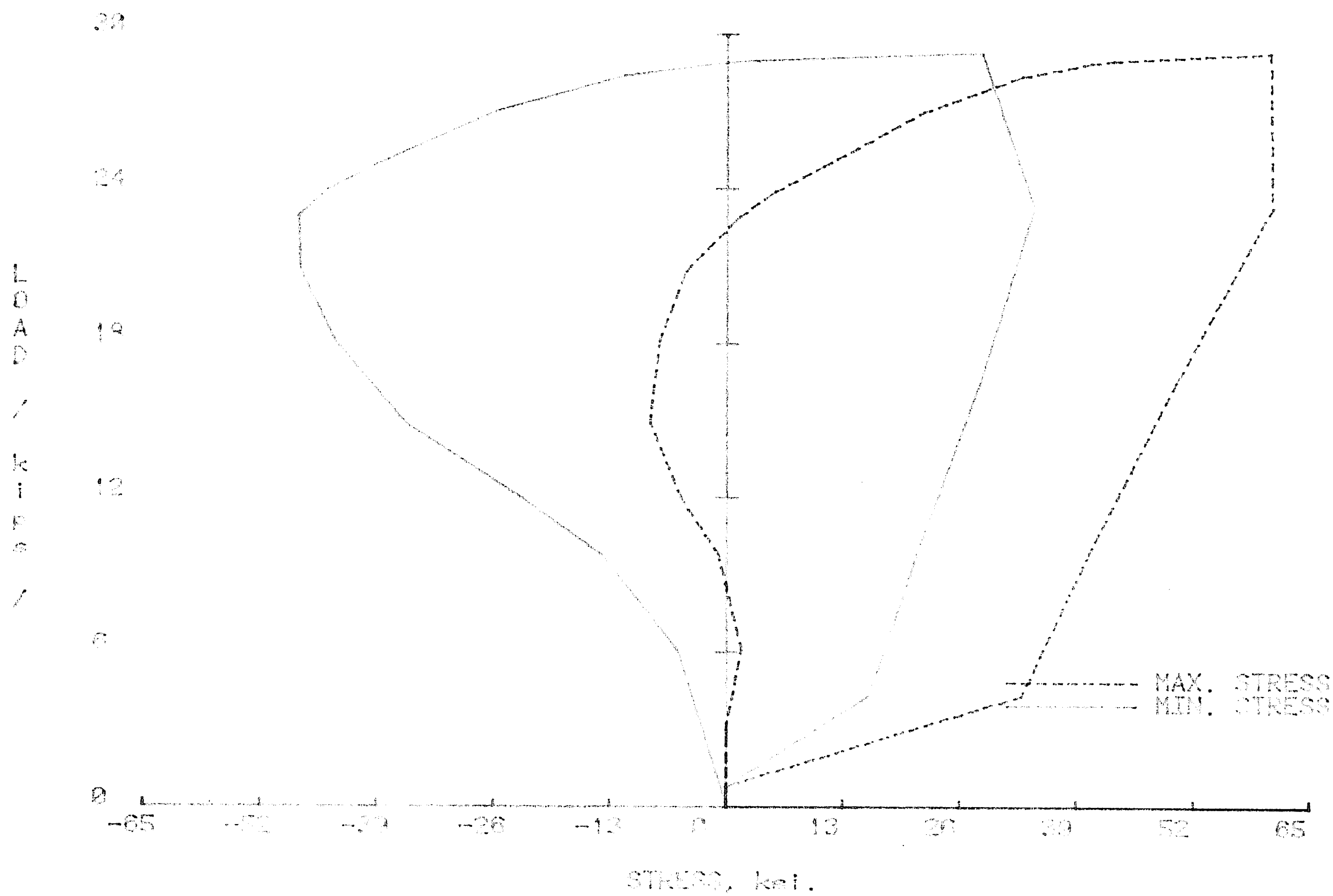


FIGURE H.11 LOAD VS. PRINCIPAL STRESSES, TEST FA-8
(Measured at Center of Panel Zone)

test FA- 8

load	horizontal stress	verticle stress	effective stress	,shear stress
0.	0.00	11.10	0.00	0.00
3.00	0.00	9.93	0.00	1.81
6.10	-3.36	-0.36	6.33	3.3[illegible]
9.90	-9.69	-4.78	13.57	6.17
12.00	-15.30	-11.84	20.96	8.89
14.90	-21.69	-21.15	32.36	13.61
18.10	-24.83	-24.68	40.40	16.06
20.80	-24.18	-23.89	41.52	21.55
22.70	-2[illegible].31	-18.40	42.42	[illegible].34
23.90	-22.94	-13.58	47.51	24.61
24.90	-15.64	-7.66	45.57	24.35
27.00	-9.37	5.69	41.23	22.57
28.40	1.95	18.89	39.88	20.74
28.90	12.12	26.72	40.80	16.[illegible]
29.00	15.34	29.19	41.58	[illegible]
29.30	37.03	49.07	52.55	14.54
23.30	39.67	52.18	53.77	10.69
16.00	31.13	44.52	43.43	7.34
9.90	22.46	37.16	34.97	4.33
4.30	15.63	31.27	26.49	-0.48
0.50	11.16	0.00	0.00	-11.10
0.00	9.93	26.93	0.00	-9.43

LOAD	PRINCIPLE STRESS 1	PRINCIPLE STRESS 2	PRINCIPLE STRAIN 1	PRINCIPLE STRAIN 2	THETA
0.	0.0	0.0	409.	851.	-11.[illegible]
3.00	0.0	0.0	57.	797.	-9.[illegible]
6.10	1.7	-5.3	112.	-201.	-32.6
9.90	-0.9	-14.0	110.	-458.	-35.7
12.00	-5.1	-23.0	62.	-717.	-3[illegible].7
14.90	-6.0	-30.8	73.	*****	-4[illegible]
18.10	-7.1	-43.7	164.	*****	[illegible]
20.80	-4.0	-47.6	304.	*****	[illegible]
22.70	1.4	-47.7	324.	*****	-40.[illegible]
23.90	5.4	-44.6	627.	*****	-3[illegible]
24.90	10.8	-39.2	751.	*****	-3[illegible]
27.00	21.9	-17.7	984.	*****	-3[illegible]
28.40	32.9	-1[illegible].4	1211.	-72[illegible].	[illegible]
28.90	40.6	-0.4	1359.	[illegible]	[illegible]
29.00	43.0	3.0	1404	-329.	-34.[illegible]
29.30	60.6	28.4	1756.	342.	-35.0
23.30	60.8	34.0	1687.	534.	-30.[illegible]
16.00	50.0	28.2	1380.	443.	-[illegible]
9.90	[illegible]	21.[illegible]	1121.	507.	[illegible]
4.30	[illegible]	1[illegible].9	957	501.	[illegible]
0.50	0.0	0.0	*****	93.	[illegible]
0.00	0.0	0.0	*****	69.	-4.[illegible]