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Fritz Engineering Laboratory Report No. 333.30

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ABSTRACT

A test program was recently completed which had as its objective the investigation of various symmetrically-loaded moment-resisting beam-to-column connections which are of extreme importance in design and construction of steel multi-story frames. This report discusses the results of three specimens included in the overall twelve-specimen program.

The three connections presented are: (a) a fully-welded connection where the beam flanges and web are groove welded to the column flange (b) a seated connection where the column flanges again are groove welded to the column flange but beam shear is carried by a beam seat which is fillet welded to the column flange, and (c) a connection in which only the flanges of the beam are groove welded to the column flange, leaving the beam flanges to carry both shear and moment. All three connections were fabricated of the same size beams and columns.

Presented in this report are comparisons of items such as load-deflection, load-rotation, and stresses at various locations on the three connections.

Test results show that Test (a) performed well achieving good stiffness, strength and ductility at maximum load. Test (b) also displayed good strength and stiffness but had very little ductility at maximum load due to buckling of the beam web. With the use of bearing stiffeners on the beam, the ductility of Test (b) was increased to a value close to that of Test (a). Test (c) attained only a very low strength level but exhibited initial stiffness and ductility at maximum load nearly equal to Test (a).

1. INTRODUCTION

With the many types of connections available for construction, one of the decisive factors in the choice of a particular type of connection is economy, especially in high-rise steel buildings. One of the most common types of connections used in high-rise steel buildings is the moment-resisting beam-to-column connection.

In 1971 a test program was initiated at Lehigh University to study the behavior and to develop the method of design of moment resisting beam-to-column connections. These are connections in which the beams are framed into the column with the beams causing bending of the column about the major axis. The test program consisting of twelve full-size beam-to-column connections was under the guidance of the Welding Research Council (WRC) Task Group on the Beam-to-Column Connections. The details of this test program are described elsewhere (7).

Presently, all twelve tests of the series of twelve specimens have been completed. Reference 12 presents the result of a complete analysis of a fully welded connection. This specimen (test C12 of Ref. 7) serves as a control specimen for the purpose of evaluating the performance of several other connections of different joint design in the series. References 4 and 9 summarize the theoretical and experimental results on the phase of the flange-welded web bolted connections (C1, C2 and C3) in the series. The test results on fully bolted connections (web-bolted and flange-bolted through moment plate, C6, C7, C8 and C9) is currently under preparation.

This report presents the test results on three interrelated welded connections in this series (C4, C5 and C12). Figure 1 shows the geometry of these three connections. All three connections are composed of the same size columns (W14x176) and beams (W27x94). The primary difference among the three is the method by which shear is carried. The fully-welded connection (C12, the control specimen) utilizes the beam web to carry shear; the second connection, the flange-welded, web unconnected with a beam seat (C4) carries shear by means of a beam seat; and finally the third connection, which is only flange-welded (C5) carries both moment and shear in the beam flanges.

Economy for field construction is the main factor in determining what types of the moment-resisting connections to be used. The fully-welded connections (C12) must be welded in the field including the expensive veritcal welding. Furthermore, the quality control for welding is hard to achieve in the field. Whereas, the beam seats in the connection (C4) can be welded to the column in the fabrication shop and thus reduce the expensive vertical field welding to only horizontal groove welding for the flanges. The connection (C5), which is flange-welded only is even more appealing for field construction. This study compares the performance of these three interrelated welded connections as shown in Fig. 1.

2. DESIGN CONCEPTS AND CRITERIA

The three connections discussed in this report represent actual interior beam-to-column moment connections and are designed according to plastic analysis procedures and comply with the AISC Specifications.[1] The moment-rotation curve in Fig. 2 schematically illustrates the behavior of a beam-to-column connection under symmetric loading. By properly designing the joint and preventing possible premature failure, the connection will be able to carry the plastic moment of the beam with sufficient rotation capacity and overall stiffness, as indicated by Curve A. However, if the design is unsatisfactory, the connection behavior will not be adequate. This is depicted by Curves B, C, and D. The connections tested are proportioned so that Curve A can be obtained.

As will be seen later, C4 (the connection with the beam seat) failed prematurely with the beam web buckling. This connection was then redesigned to include a beam web stiffener and labeled C4R. In the redesign the web was considered to act as a column with one end hinged and one end fixed.

The connections, along with all the others in the series, were designed so that the plastic moment of the beam section would be obtained. The column section chosen was that which had the least size permitted without requiring horizontal stiffeners. The connection members were proportioned such that at the beam-to-column junction the plastic moment and factored shear capacity would be achieved simultaneously.

The shear capacity used, 374 kips, was the shear capacity obtained for the bolts in the design of the shear plate for test C2 (see Ref. 7). The three connections were then designed using a 374 kip shear capacity and the beam span was then calculated as the ratio of moment to shear. The detailed design procedure of C12 is presented in Appendix 1 as an example.

The specimens were welded according to the AWS Building Code[2] with E70TG electrodes. The electrodes for fillet welds were E7028. NR311 filler metal was used for beam flange groove welds; and NR202 filler metal was used for beam web groove welds. All groove welds were inspected by ultrasonic testing and fillet welds by magnetic particle as per AWS Code.

3. TEST PROGRAM

3.1 Description of Connections

The joint detail of C12 is shown in Fig. 3; the beam flanges and beam web are connected by groove welds to the column flanges. To simulate field practice, an erection plate is tack welded to the column flange and A307 bolts are used temporarily to attach the beam to the column.

The joint detail of C4 is shown in Fig. 4. Vertical shear is resisted by a two-plate welded stiffener seat which is designed according to Table VIII of the AISC Manual. The beam flanges are groove welded to the column flanges, and the seat plates and stiffener plates are fillet welded to the column flanges. To simulate field conditions for this connection, the seat plates and stiffener plates are attached to the column at the fabrication shop first, and then the beam is held in place by the A307 bolts during the welding of the flanges.

As seen in Fig. 5, only the groove welds of the flanges connects the beam to the column flanges for test C5. It has neither an erection seat nor an erection clip. The strength of this connection should be weaker than that of Test C4.

A detailed description of these test specimens is given in Ref. 7.

3.2 Material Properties

The material used for both beams and column is ASTM A572 Grade 55 steel. Properties used in determining stresses are as follows:

Modulus of elasticity (E) = 29,570 ksi;

Yield strain (ϵ_y) = 0.001857 in/in;

Yield stress (σ_y) = 54.9 ksi;

Strain at onset of strain hardening (ϵ_{st}) = 0.0150 in/in;

Strain hardening modulus (E_{st}) = 581 ksi.

A detailed report of material properties is included in Ref. 11.

3.3 Instrumentation

Figures 6, 7 and 8 give an overall view of the instrumentation used for stress analysis in this report. SR-4 strain gages were placed on beam flanges to provide checks for possible lateral buckling, and to determine the stress distribution. SR-4 strain gages were also attached at upper portion of the column and were used to align the connection and testing machine crosshead. Deflection dial gages were located directly under the column for measuring overall deflection and in the column web compression region for determining web buckling. Level bars were attached near beam-to-column juncture to determine the rotation capacity of the joint.

In Figs. 9, 10 and 11 the panel zone instrumentation is shown. SR-4 strain gages were provided in the beam web to obtain the vertical stress distribution throughout this section. The strain gages in the column web panel zone were placed to provide the general stress distribution and flow throughout the zone. Strain gages were placed at a distance of $\frac{1}{2}$ (t_b + 5k) from beam flange centerline where t_b = thickness of beam flanges and k = distance from outer face of column flange to web toe of fillet. [In the present AISC Specifications,[1] formula

(1.15-1), which pertains to requirements for stiffening in the compression region, was developed from the concept that the column flange acts as a bearing plate. It distributes the load caused by the beam compression flange to the column web with a width of $t_b + 5k$.] The information from these, along with that in later tests, should provide data for determining the validity of present assumptions of stress distribution at the k-line in the column web. Hence, all strain gages shown along the column innerface were placed at the toe of fillet or the k-line. Strain rosettes labeled K in Figs. 9 and 11 or P in Fig. 10 were placed on opposite sides at the same location. These values were averaged to account for any early web buckling.

3.4 Test Setup

The test setup is shown in Fig. 6. A 5,000,000 pound-capacity hydraulic testing machine was used to apply axial load in the column. The beams were supported by two pedestals resting on the floor. Rollers were used to simulate simply supported end conditions. Because of the size of sections and the short span of the beam used, no lateral bracing was needed to provide stability. Bearing stiffeners were provided over supports to insure no web crippling would occur in the beam.

4. TEST RESULTS AND DISCUSSION

As described in Sec. 2, the connections simulate actual interior symmetrically-loaded beam-to-column moment connections. The test setup as shown in Fig. 6 is in an inverted position so that a concentrated load can be utilized.

4.1 Test Procedures and Observations

The applied load for the tests was increased continuously until failure, with all the strain and dial gage readings recorded after each load increment. Vertical alignment was checked by transit after each load to insure that no lateral buckling occurred.

The load deflection curve of each test, C12, C4, C4R and C5 as shown in Fig. 12 were plotted continuously so that general specimen behavior could be observed and compared. Figure 13 shows the corresponding load rotation curves for the specimens C4, C5 and C12.

4.1.1 Specimen C12

Load increments of 25 kips were used initially until 600 kips was attained. Then 20 kip increments were used until 680 kips was reached and the connection was unloaded completing the first loading cycle.

On the second loading cycle after reloading to 680 kips, the load was increased another 20 kips to 700 kips. From there on including the third loading cycle, the increments were changed from a load rate to a deflection rate of 0.20 in. After each deflection the load was allowed to stabilize until there was no further movement of the sensitive crosshead, with the loading valve closed.

The first yield lines began forming in the compression web of the column at an applied load of 475 kips. Both localized yielding at the toe of the fillet and yielding at the web center were observed. At this point the load deflection curve began to deviate from the linear.

At 600 kips yielding was observed in the tension region of the column web near the toe of the fillet. Yielding now appeared to extend completely through the web in the compression region and in the upper beam web area near the compression flange.

The connection attained a maximum load of 838 kips at a deflection of approximately 2.7 in. Figure 14 shows a view of the fracture of the weld at the tension flange. As seen by the picture, the weld did not fail but pulled out the surrounding column flange material. Figure 15 shows fracture of weld along the beam-web which occurred simultaneously. The connection is shown at the end of the test in Fig. 16.

4.1.2 Specimen C4

A 50^k load increment was used until a load of 450 kips was attained. Then a deflection increment was used until a load of 660 kips was reached. At this stage of loading the specimen began to unload.

Yielding was first observed at the cope of the beam web under the beam seat while loading the specimen from 150 to 200 kips as shown in Fig. 17. Maximum load was attained at 660 kips with a deflection of 0.386 in. At this point the beam web near the junction began to buckle and caused the specimen to unload. The beam web then tore at the cope and an overall view of the specimen at failure is shown in Fig. 18.

4.1.3 Specimen C4R

Because of the premature failure of C4 caused by excessive beam buckling, it was decided to retest the connection. The buckled beam was replaced and vertical stiffeners were added on both sides of both beams. The stiffeners, 5x5x3/8" angles attached with five A307 bolts, were designed assuming the beam web near the junction to act as a column which was hinged at one end and fixed at the other end. This assumption was based on the observations of the way the beam web plate of specimen C4 buckled.

Specimen C4R was then loaded in the same manner as specimen C4. The only data obtained for this retest specimen was deflection which is plotted in Fig. 12 along with the load-deflection curves of the other tests.

First signs of yielding occurred at a load of 450 kips in the beam web directly under the beam seat. Failure due to weld fracture at the heat-affected zone of the tension flange occurred while the specimen was unloading at a load of 768 kips. The maximum load attained was 776 kips with a corresponding deflection of 1.2 in. A picture of the specimen at the end of the test is shown in Fig. 19.

4.1.4 Specimen C5

The load increment for this test was also 50 kips until 350 kips was attained and then deflections were used.

Yielding first occurred at a load of 300 kips in both beam tension flanges and one compression flange. Yielding was also observed at the toe of the column fillet welds in the compression area. At a load of 375 kips, local plastic hinges formed. Figure 20 shows this connection at the end of the test.

4.2 Discussion of Results

Methods for determining the state of stresses and yielding from strain gages are presented in Appendix 2.

Stress analysis for the three connections tested will be presented in terms of the following sequence: column behavior, beam behavior and beam-to-column interaction.

4.2.1 Column Behavior

Figure 21 illustrates the horizontal stress variation (σ_x) along the column innerface (k-line) for the three connections, with compressive stresses occurring in the upper region and tensile stresses occurring in the lower region. The greater distribution of σ_x in test C12 is due to the effective use of the beam web to carry shear; whereas, in C4 and C5 the shear is carried by the beam seat and the beam flange respectively. The pattern of the stress distribution in the tension and compression zones of column web is seen to be the same for all connections.

Figure 22 illustrates the corresponding vertical stress variation (σ_y) along the column innerface or k-line for the three connections, with the stresses being primarily compression except in the lower region of

C12. This shows that biaxial tension can occur at the k-line near the beam tension flange. Indications are that the specimen C12 may be more critical in the sense of fracture failure than the other two connections.

The horizontal stress variations along the centerline of the column web as illustrated in Fig. 23 are in close proximity to being linear and approximately equal to zero at the centerline intersection. However, linearity is not maintained after the initial yielding has been attained which occurred in the compressive region of each connection.

4.2.2 Beam Behavior

Figure 24 shows the horizontal stress variation across a beam section six inches from the column flange. As seen, the σ_x values for C4 and C5 are very low; this owing to the fact that the beam web is not welded to the column flange as is C12.

Figures 25, 26 and 27 show the stress distribution for the beam flanges. A pattern can be seen when comparison of the distributions is made from C12 to C4 to C5; there is a tendency of reversal in the distributions from parabolic in one direction to the other direction. This degree of reversal is obviously caused by the amount of shear force carried by the beam flanges. The flanges in specimens C12, C4 and C5 carry the minor part, the significant part and the entire part of the shear force respectively. The diagrams in Fig. 28, from left to right, show the horizontal stress (σ_x) variation, vertical stress (σ_y) variation, and the shear stress (τ_{xy}) variation respectively in the vertical beam seat stiffener plate along the column flange (top diagrams) and the beam flange (bottom diagram) for C4. From the shear stress (τ_{xy})

diagram, it can be shown that the amount of load carried in shear in the stiffener plate increases from 9% at 150^k to 30% at 600^k with the remainder being carried by the two flanges and the seat plate. Using Fig. 28 in conjunction with Fig. 26, it can be seen that the amount of shear transferred to the beam seat has a significant effect on the stress distribution of beam flanges. Figure 27 shows the extreme condition of the distribution with the flanges carrying all the shear.

4.2.3 Beam-to-Column Interaction

As the load-deflection curve in Fig. 12 begins to deviate from linearity, yielding begins to occur either in the beam flanges or in the panel zone adjacent to the column flanges as shown in Figs. 29, 30, and 31. The stresses then redistribute to the adjacent area with the majority of the stress being distributed over a distance of $\frac{1}{2}$ $(t_b + 5k)$ from the centerline of each flange as shown in Fig. 21. Simultaneously, the connections begin to rotate inelastically with C12 and C4 being fairly equal.

From the load deflection curve of Fig. 12 it can be seen that the AISC Specifications[(1)] are adequate for the design of C4R as well as C12.

As can be seen in most of the plots, C12 and C4 or C4R have similar results; therefore, a designer has a choice of two connections to use. They are a fully-welded connection, C12, or a flange-welded connection, C4R, with a beam seat for shear carrying capabilities and a beam web stiffener to prevent buckling. Although the stress results are not available, load-deflection curve of C4R indicates that C4R is a much better connection than C4.

5. SUMMARY AND CONCLUSIONS

Herein, test results on three interrelated, welded, steel beam-to-column moment connections are reported. The size of columns and beams of these three connections is identical. The primary distinction among them is the way the beam shear force is carried. The fully-welded connection (C12) utilizes the beam web to carry a significant part of the shear force; the stiffened seated beam connection (C4) carries the shear force through both the beam seat and the beam flanges; and the connection (C5) which is beam flange-welded only, carries the entire shear force and moment capacity through the beam flanges. On the basis of the test results in this study, the following conclusions have been reached.

1. The AISC Specification provides adequate rules in design of such fully welded connections as C12 or stiffened seated beam connections as C4. For the latter case, however, the possibility of buckling of the beam web above the stiffened seat must be checked and beam web stiffeners may be added (C4R). This type of connection can be used in plastic design as the plastic limit load, sufficient rotation capacity, and adequate elastic stiffness are developed (Figs. 12 and 13).
2. Although the stiffened seated beam connection (C4) fails by excessive buckling of the beam web and eventually fractures at the cope hole of the beam web, specimen (C4) does exhibit sufficient stiffness under working load.
3. The fully welded connection (C12), and the stiffened seated beam connection with beam web stiffeners (C4R) are basically

identical in their general behavior to the applied loads and may be used interchangeably (Fig. 12).

4. The flange-welded only connection (C5) attains 51 percent of its predicted plastic limit load based upon whole section, and showed substantial deformation and rotation capacity. This type of connection can be used in a design where the initial stiffness and deformation capacity rather than the full strength of connections is the controlling factor.

5. The basic patterns of stress distribution in the panel zone of the column are essentially the same for all the connections tested. However, the stress distributions in the beam flanges and web are effected significantly by the amount of shear force transferred to the beam flange.

6. ACKNOWLEDGMENTS

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7. APPENDICES

APPENDIX 1: DESIGN OF CONNECTION C12

(W27x94 beam and W14x176 column)

1. Determine beam span.

Plastic Moment

$$M_p = F_y Z_x = (55K/in.^2)(278\ in.^3) = 15290\ kip\text{-}in.$$

Design Ultimate Shear

Design from test C2 (Ref. 6): 7-1" A490-X bolts in single shear, $V = 7(1.7)(0.7854\ in.^2)(40K/in.^2) = 374K$. [See Ref. 6 for explanation of 40 ksi shear stress.]

Check: $V_p \leq 0.55\ F_y\ td = (0.55)(55K/in.^2)(0.490\ in.)(26.91\ in.)$

$$= 399.0\ K \approx 94.7\%\ V_p \quad O.K.\ [AISC,\ 2.5\text{-}1]$$

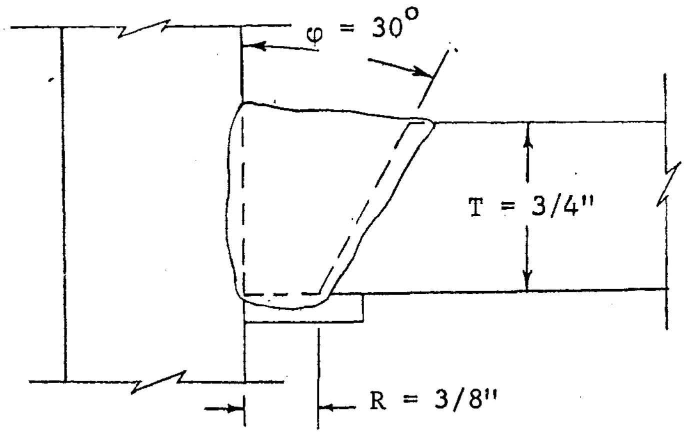
Beam Span

$$L = M_p/V = 15290\ K\text{-}in./374\ K = 40.8\ in.$$

Use 41 in. length (3'-5")

2. Determine groove welds.

Allowable tension normal to effective throat of complete-penetration groove weld is same as allowable tensile stress for base metal. Use E70XX electrodes and weld TC-U4-S of the AISC Specification.



for flanges

For web, use $\varphi = 45^{\circ}$, R = ¼".

3. Check horizontal stiffener requirements.

Opposite compression flange:

Using AISC Specification,

$$t < \frac{C_1 A_f}{t_b + 5k} \qquad \text{[AISC, 1.15-1]}$$

$$t < \frac{(1)(9.990 \text{ in.})(0.747 \text{ in.})}{0.747 \text{ in.} + 5\ (2.0 \text{ in.})} = 0.694 \text{ in.}$$

t for W14x176 column is 0.820 in. ∴ O.K.

$$t \leq \frac{d_c \sqrt{F_y}}{180} \qquad \text{[AISC, 1.15-2]}$$

$$t \leq \frac{(15.25 \text{ in.} - 4.00 \text{ in.})\ \sqrt{55 \text{ K/in}^2}}{180} = 0.464 \text{ in.} < 0.820 \text{ in.} \quad \text{O.K.}$$

Using Fritz Engineering Laboratory Report 333.14(13),

$$t \leq \frac{d_c^2 \sqrt{F_y} + 180\ C_1\ A_f}{125\ d_c\ \sqrt[4]{F_y}}$$

$$t \leq \frac{(11.25 \text{ in.})^2 \sqrt{55 \text{ K/in.}^2} + 180\ (1)(9.990 \text{ in.})(0.747 \text{ in.})}{125\ (11.25 \text{ in.})\ \sqrt[4]{55 \text{ K/in.}^2}}$$

$$= 0.596 \text{ in.} < 0.820 \text{ in.} \quad \therefore \text{O.K.}$$

Stiffeners are not required opposite the compression flange.

Opposite tension flange:

$$t_f < 0.4\sqrt{C_1 A_f} \qquad \text{[AISC, 1.15-3]}$$

$$t_f < 0.4\sqrt{(1)(9.990 \text{ in.})(0.747 \text{ in.})} = 1.093 \text{ in.}$$

t_f for W14x176 column is 1.313 in.; therefore stiffeners not required.

4. Design erection plate.

Connection is to be designed as a field welded connection. Therefore, an erection plate is to be attached to the column to facilitate field welding.

(a) Choose a plate.

Erection plate must be able to carry dead load of beam

Dead load ≈ (0.094 K/ft)(3.42 ft) = 0.32 K

Try a 3/8 in. x 23 1/2 in. plate. (A572 Grade 55)

$$F_v = 0.40\ F_y \qquad \text{[AISC, 1.5.1.2]}$$

Shear plate can resist = $0.40\ F_y\ td$ = 0.40 (55 K/in.^{2})(.375 in.)(23.5 in.)

= 194 K >> 0.32 K ∴O.K.

Use a <u>3/8 in. x 23 1/2 in.</u> plate.

(b) Weld plate to column.

Tack weld using allowable shear stress of 21 ksi (E70XX Electrodes). From AISC 1.17.5, minimum weld size is 5/16 in. Using intermittent welds and conforming to AISC 1.17.8, try 3 two-inch fillet welds.

Allowable shear = (1.7)(21 K/in.^{2})(6.0 in.)(0.3125 in.)(0.707) = 47.4 K

47.4 K >> 0.32 K ∴O.K.

(c) Transfer load by bolts.

Try 2-3/4"φ A307 erection bolts.

Allowable shear = 2 (1.7)(0.4418 in.^{2})(4.42 K/in.^{2}) = 6.64 K

6.64 K > 0.32 K O.K.

See Fig. 3 for design sketch of connection C12.

APPENDIX 2: STRESS-STRAIN RELATIONSHIP

1. For Strain Rosettes

(a) Tension or Compression

Using the Von Mises yield criterion, the effective stress is defined as

$$\sigma_e = \frac{1}{\sqrt{2}} [(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2 + 6(\tau_{12}^2 + \tau_{23}^2 + \tau_{31}^2)]^{1/2}$$

The effective strain is defined as

$$\epsilon_e = \frac{\sqrt{2}}{2(1+\mu)} [(\epsilon_1-\epsilon_2)^2 + (\epsilon_2-\epsilon_3)^2 + (\epsilon_3-\epsilon_1)^2 + 6(\epsilon_{12}^2 + \epsilon_{23}^2 + \epsilon_{31}^2)]^{1/2}$$

For a simple tension test,

$\sigma_2 = \sigma_3 = \tau_{12} = \tau_{23} = \tau_{31} = 0$, $\epsilon_{12} = \epsilon_{23} = \epsilon_{31} = 0$, and $\epsilon_2 = \epsilon_3 = -\mu\epsilon_1$

These equations reduce to $\sigma_e = \sigma_1$ and $\epsilon_e = \epsilon_1$, respectively.

($\sigma_1 = \sigma_y$ and $\epsilon_1 = \epsilon_y$ from tensile tests) (See Ref. 5)

From linear elasticity,

$$\epsilon_x = \frac{1}{E} [\sigma_x - \mu (\sigma_y + \sigma_z)]$$

$$\epsilon_y = \frac{1}{E} [\sigma_y - \mu (\sigma_x + \sigma_z)]$$

$$\epsilon_z = \frac{1}{E} [\sigma_z - \mu (\sigma_x + \sigma_y)]$$

For the connection web portions, assume plane stress condition, i.e. $\sigma_z = 0$. Therefore,

$$\epsilon_z = - \frac{\mu (\epsilon_x + \epsilon_y)}{(1 - \mu)}$$

$$\sigma_x = \frac{(1 - \mu) E}{(1 + \mu)(1 - 2\mu)} \epsilon_x + \frac{\mu E}{(1 + \mu)(1 - 2\mu)} (\epsilon_y + \epsilon_z)$$

$$\sigma_y = \frac{(1 - \mu) E}{(1 + \mu)(1 - 2\mu)} \epsilon_y + \frac{\mu E}{(1 + \mu)(1 - 2\mu)} (\epsilon_x + \epsilon_z)$$

(b) Shear

For cases of high shear, the effective stress and strain equations reduce to

$$\sigma_e = \sqrt{3}\ \tau_{12}$$

$$\epsilon_e = \frac{\sqrt{3}}{2\ (1 + \mu)}\ \gamma_{12}$$

where $\gamma_{12} = 2\ \epsilon_{12}$.

(c) Shear and Axial Stresses in Panel Zone

From Ref. 5, for high shear and axial stresses in a connection panel, the effective stress and effective strain are:

$$\sigma_e = \frac{1}{\sqrt{2}}\ [2\ \sigma_1^2 + 6\ \tau_{12}^2]^{1/2}$$

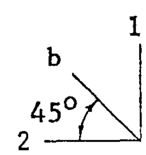
$$\epsilon_e = \frac{\sqrt{2}}{2\ (1 + \mu)}\ [2\ (1 + \mu)^2\ \epsilon_1^2 + \frac{3}{2}\ \gamma_{12}^2]^{1/2}$$

Using either Mohr's circle for stress and comparing the principal stresses to the appropriate effective stress, or Mohr's circle for strain and comparing the principal strains to the appropriate effective strain, yielding at the strain rosette can be determined.

It was found that by neglecting ϵ_z results of rosette stresses changed insignificantly so that in future tests, data could be analyzed considering only a two-dimensional system.

In determining ϵ_{12} from the strain rosette,

$$\epsilon_{12} = 2\ \epsilon_b - (\epsilon_1 + \epsilon_2)$$



2. For 90° Gages

The effective stress used was $\sigma_e = \sigma_1$ (where $\sigma_1 = \sigma_y$ of tensile tests). Stresses in the 90° gages were determined by

$$\sigma_x = \frac{E}{1 - \mu^2} (\varepsilon_2 + \mu\ \varepsilon_1)$$

$$\sigma_y = \frac{E}{1 - \mu^2} (\varepsilon_1 + \mu\ \varepsilon_2)$$

3. Linear Gages

Strain readings were compared directly to ε_y and ε_{st}. Below the elastic limit $\sigma = E\varepsilon$; between ε_y and ε_{st}, $\sigma = \sigma_y$; above ε_{st}, $\sigma = \sigma_y + E_{st} (\varepsilon - \varepsilon_y)$.

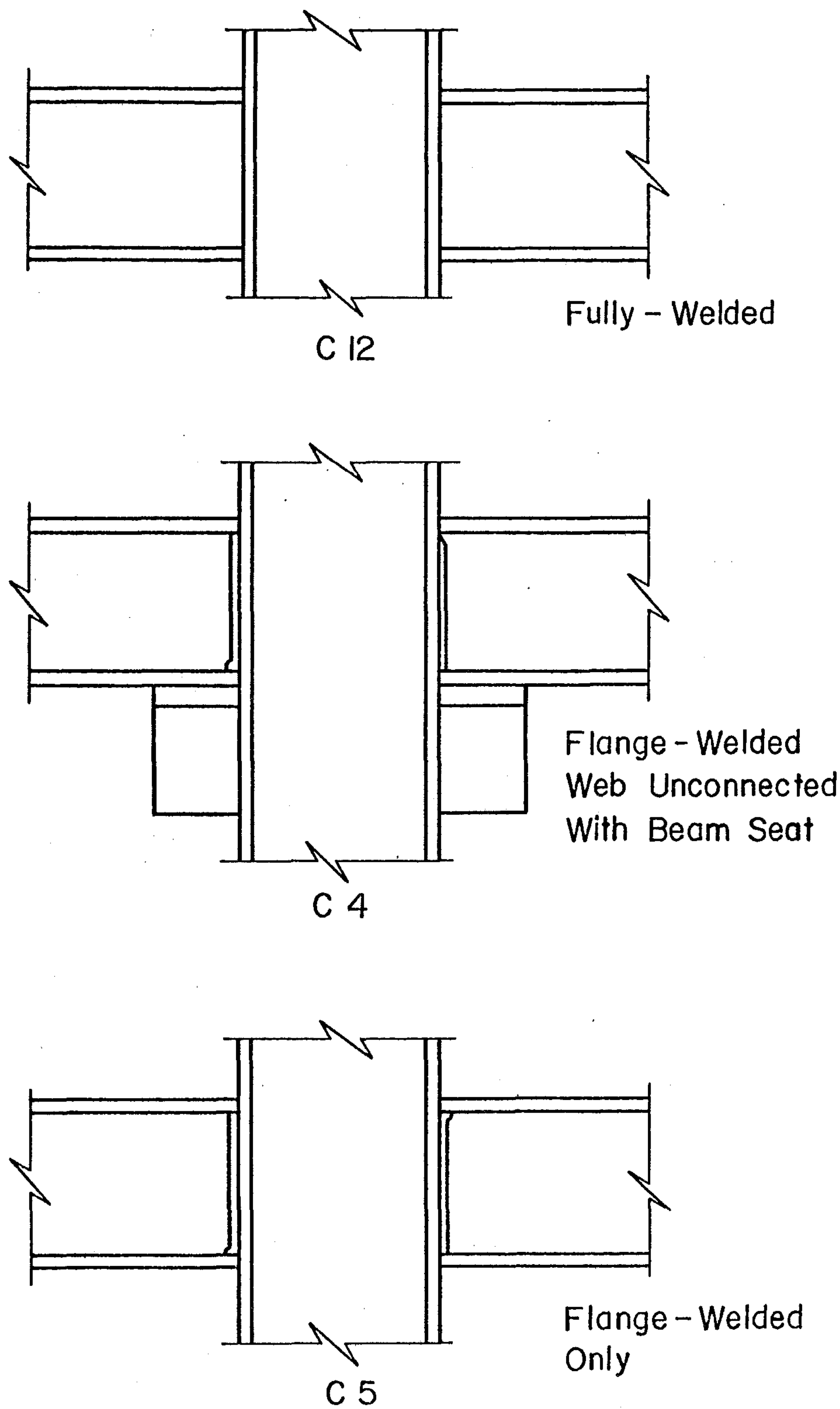


Fig. 1 Connection Geometries

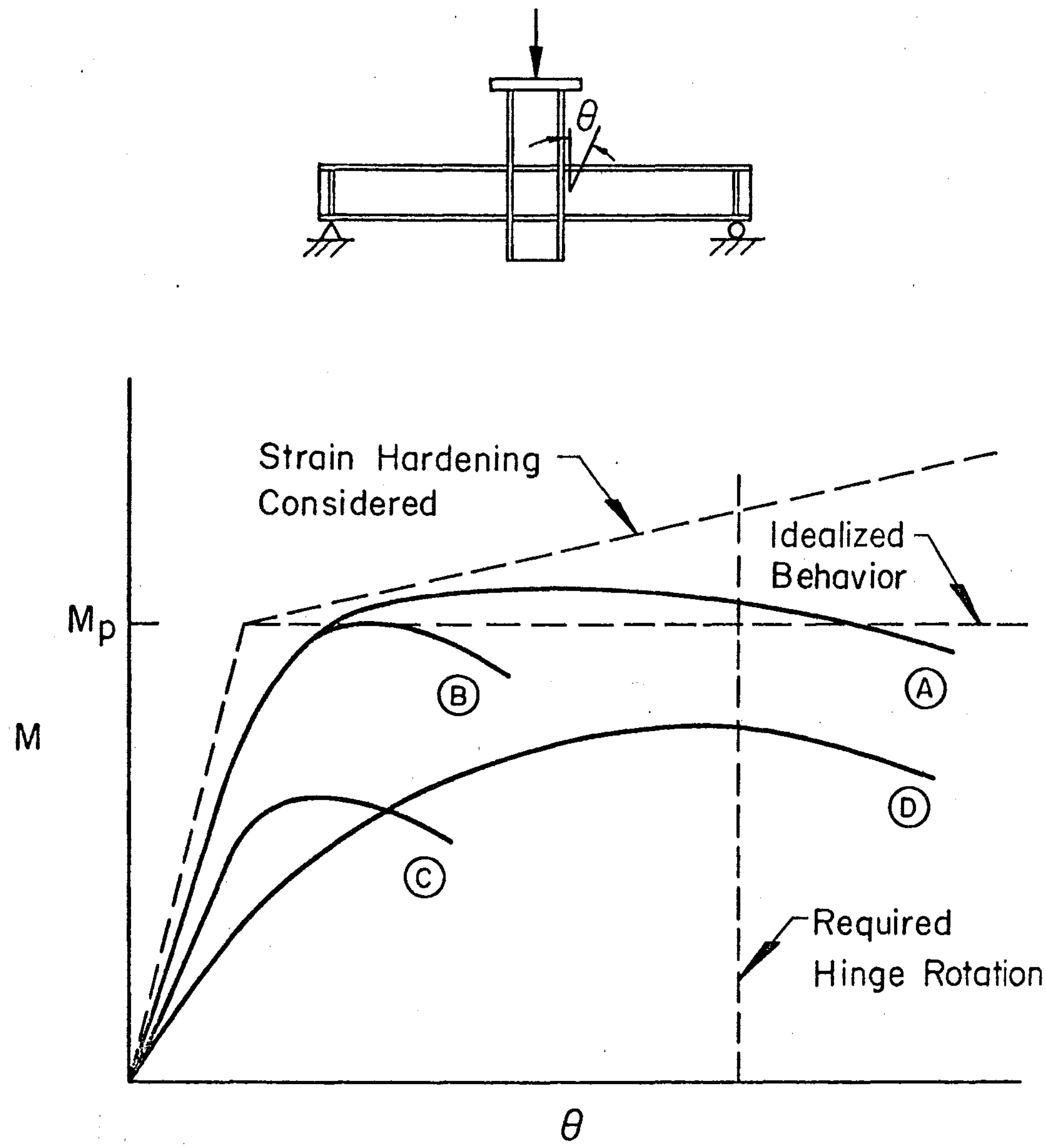


Fig. 2 Moment Rotation Curves

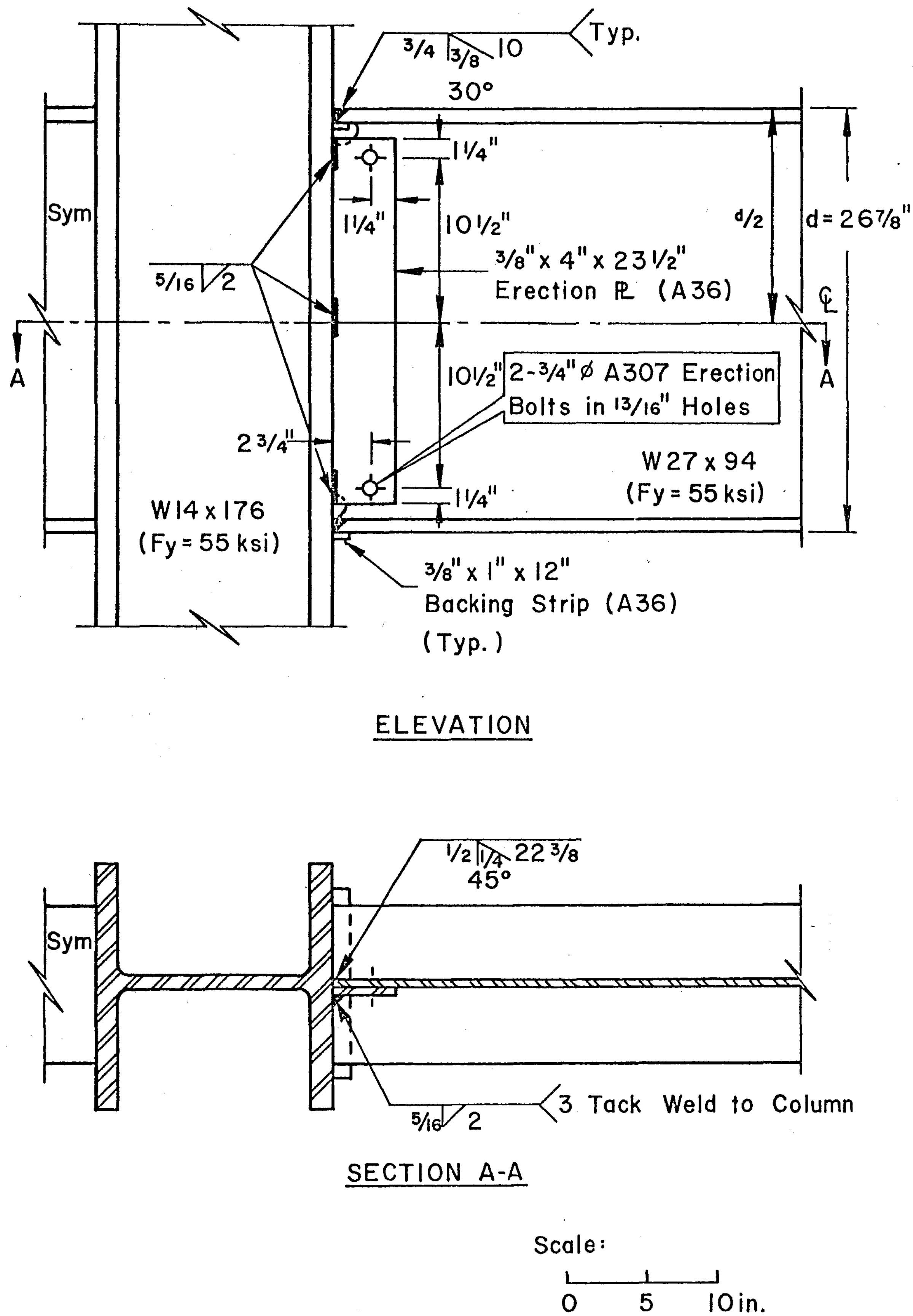


Fig. 3 Test C12 Detail

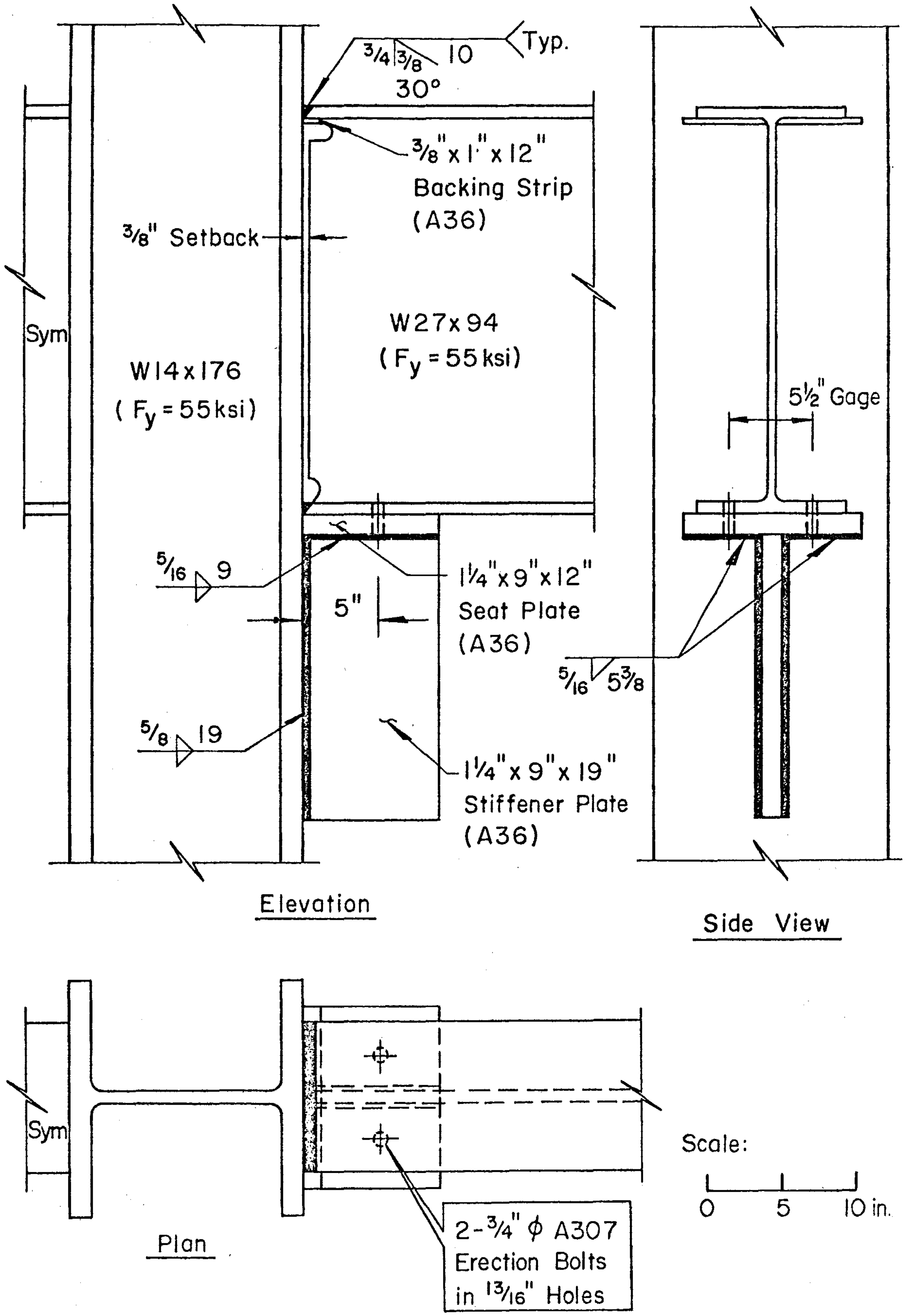


Fig. 4 Test C4 Detail

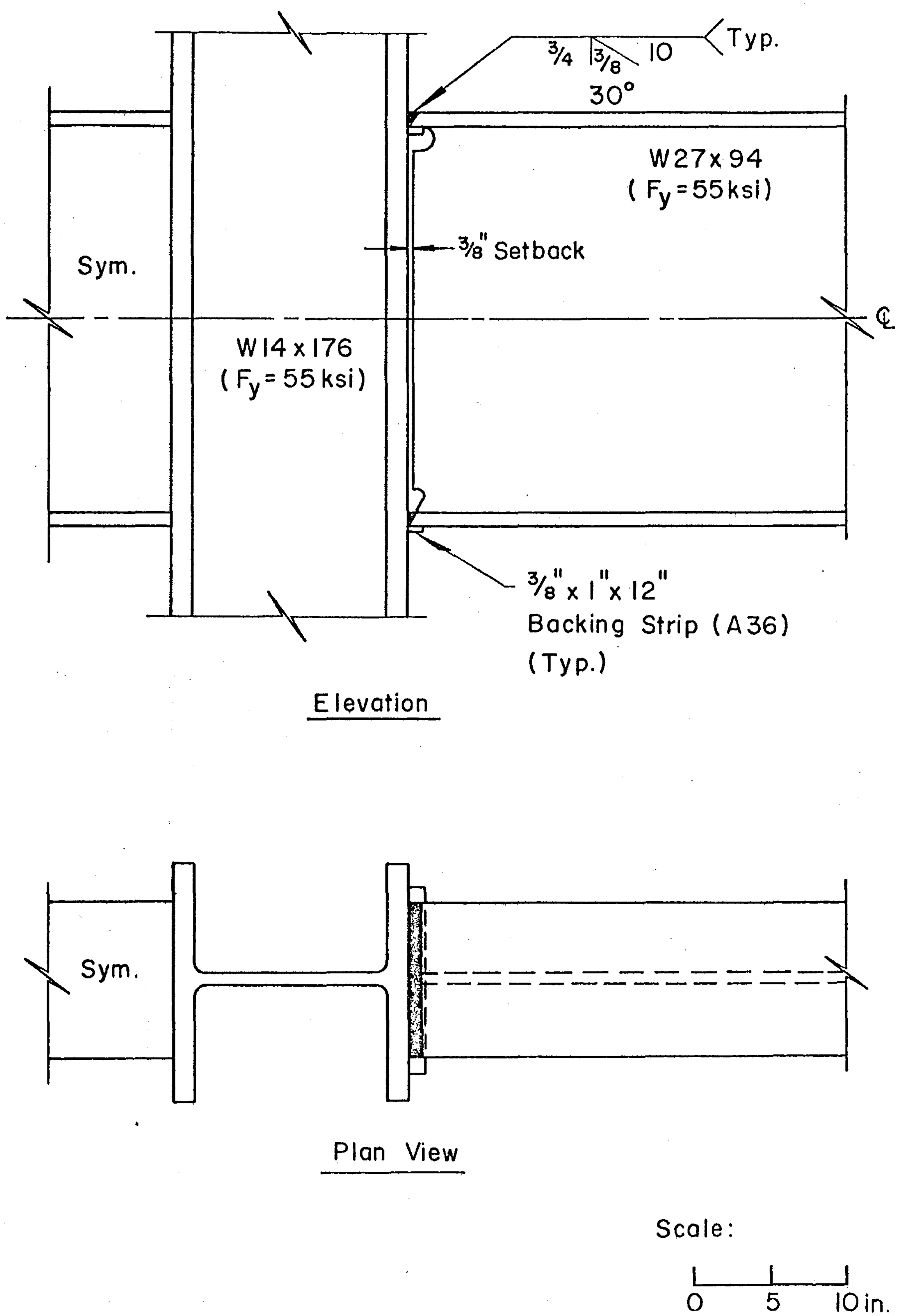


Fig. 5 Test C5 Detail

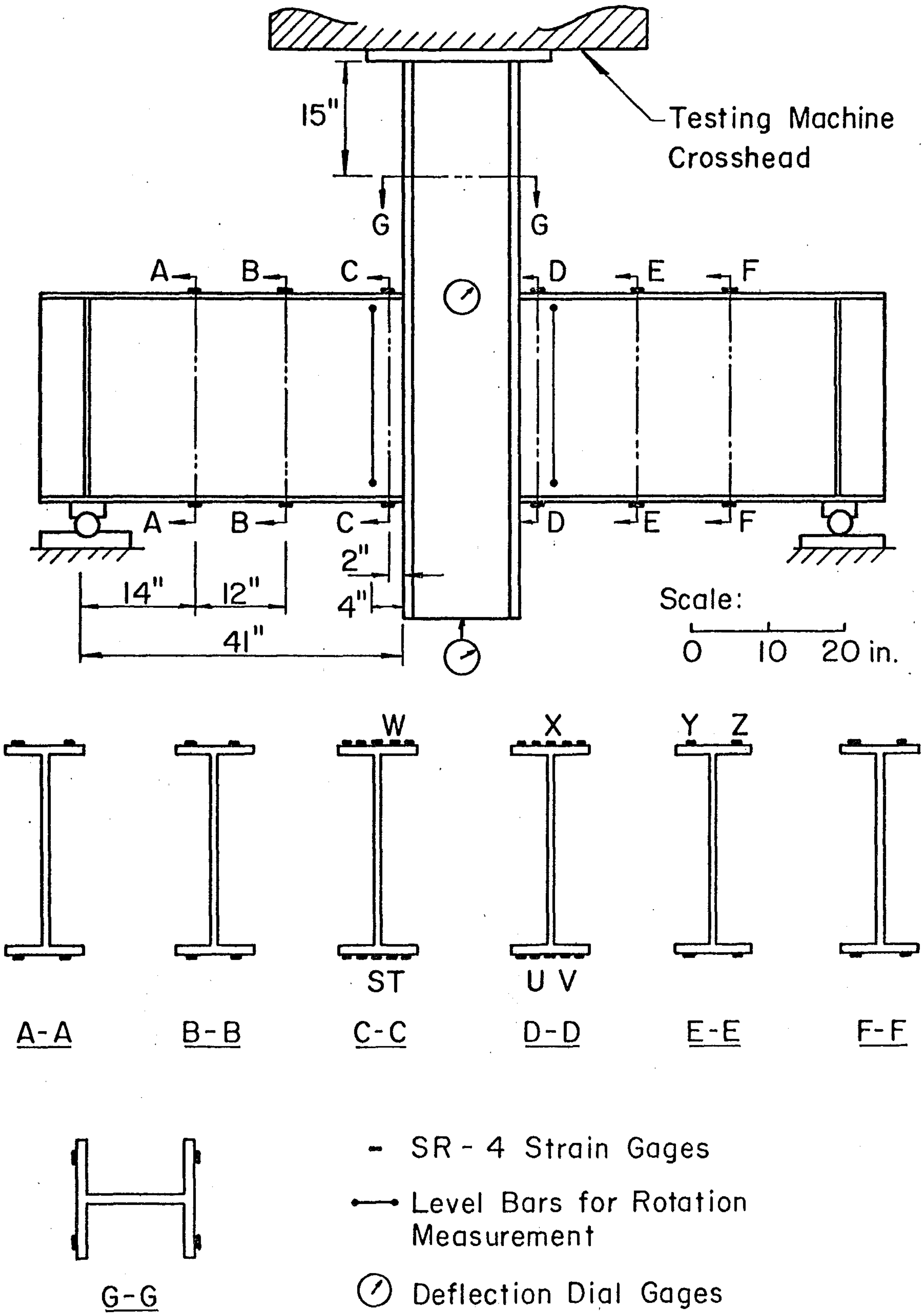


Fig. 6 Test C12 Instrumentation

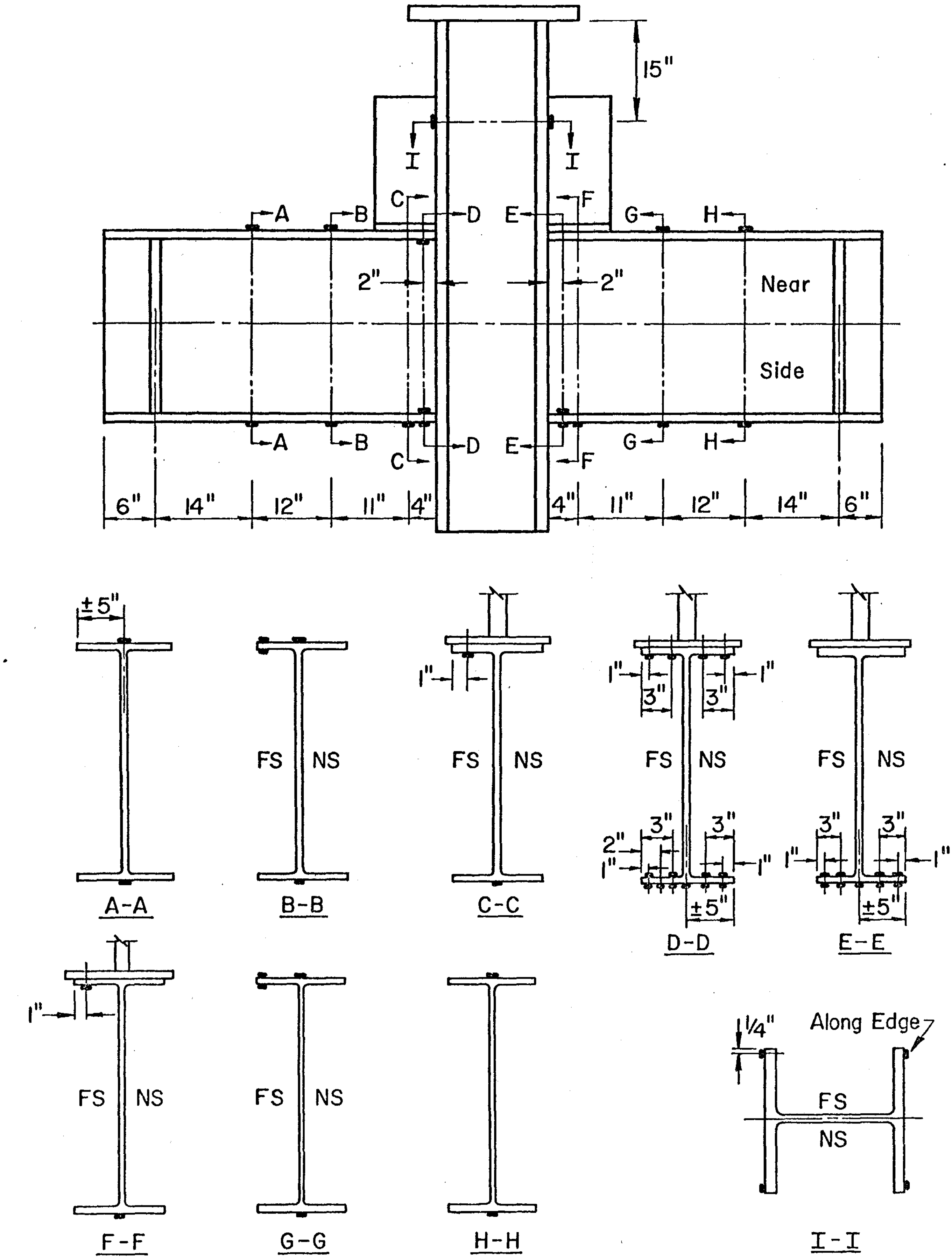


Fig. 7 Test C4 Instrumentation

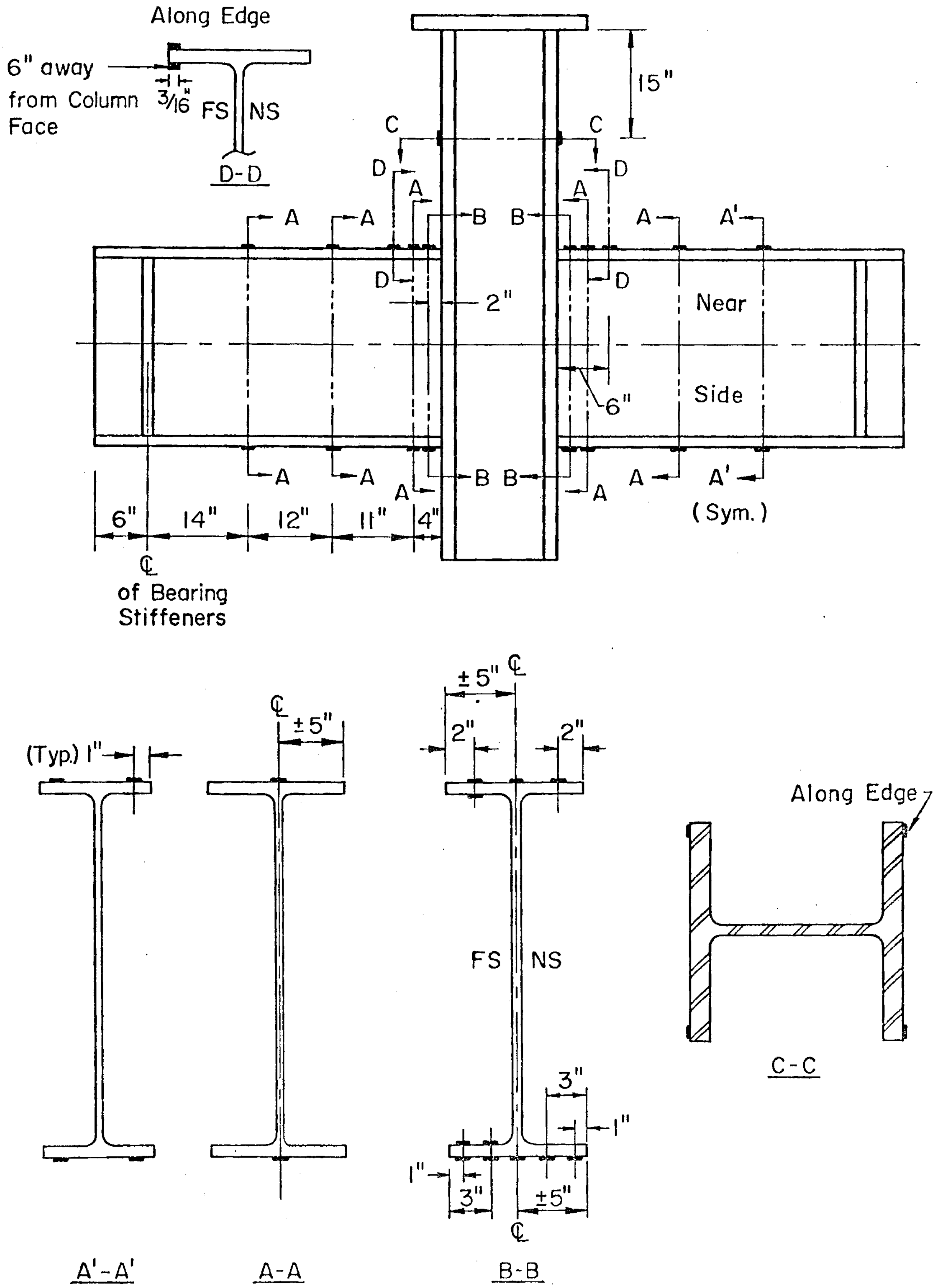


Fig. 8 Test C5 Instrumentation

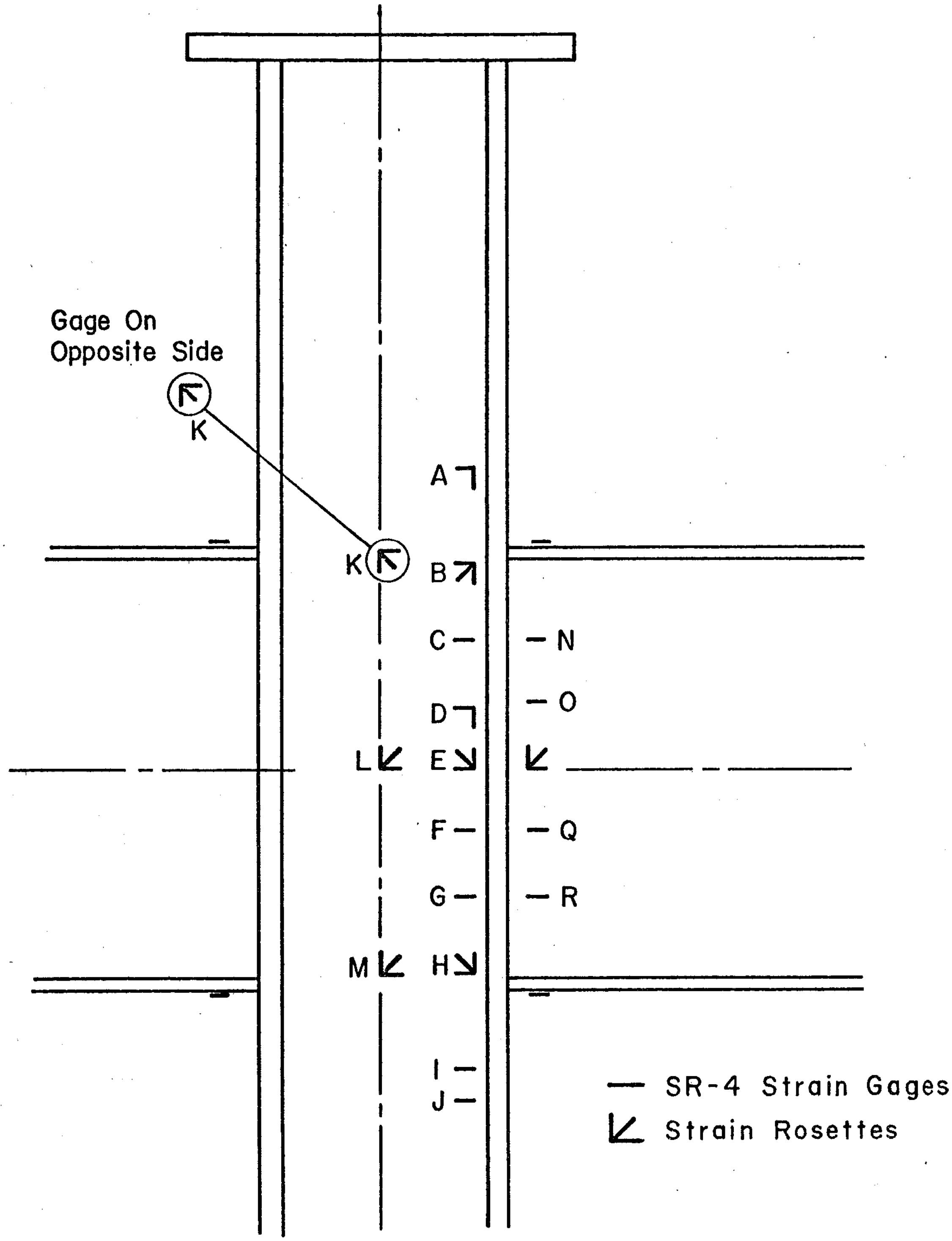


Fig. 9 Panel Zone Instrumentation--Test C12

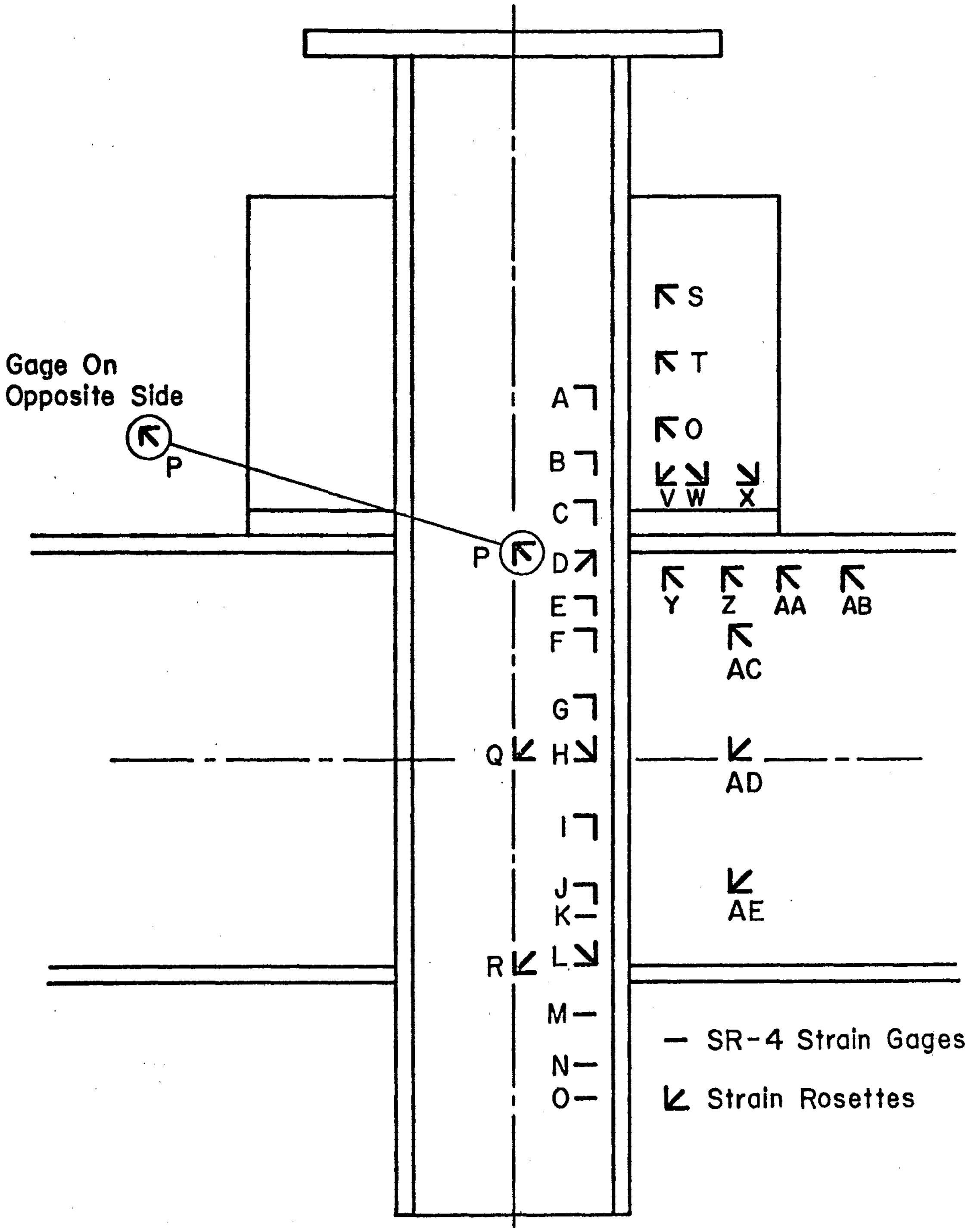


Fig. 10 Panel Zone Instrumentation--Test C4

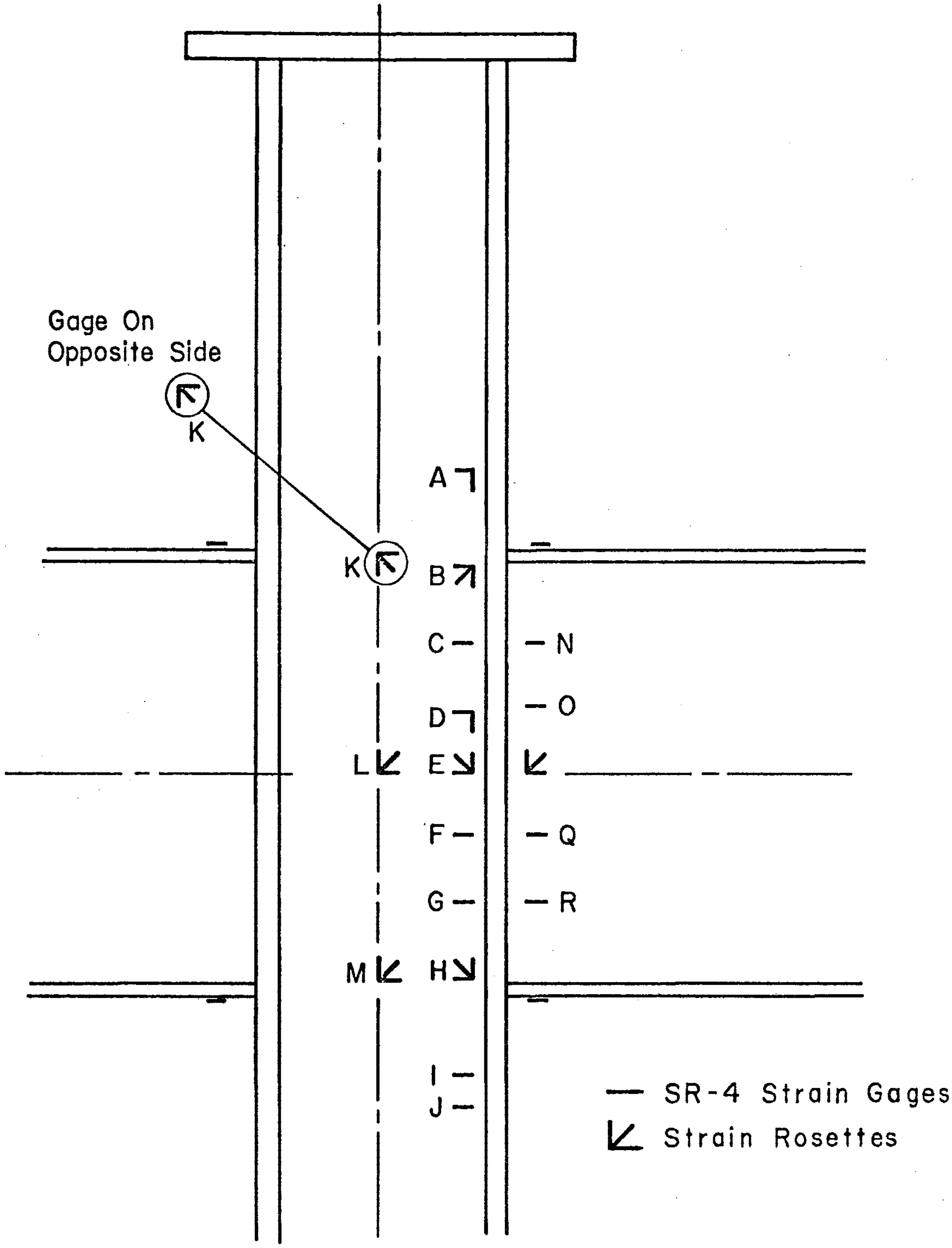


Fig. 9 Panel Zone Instrumentation--Test C12

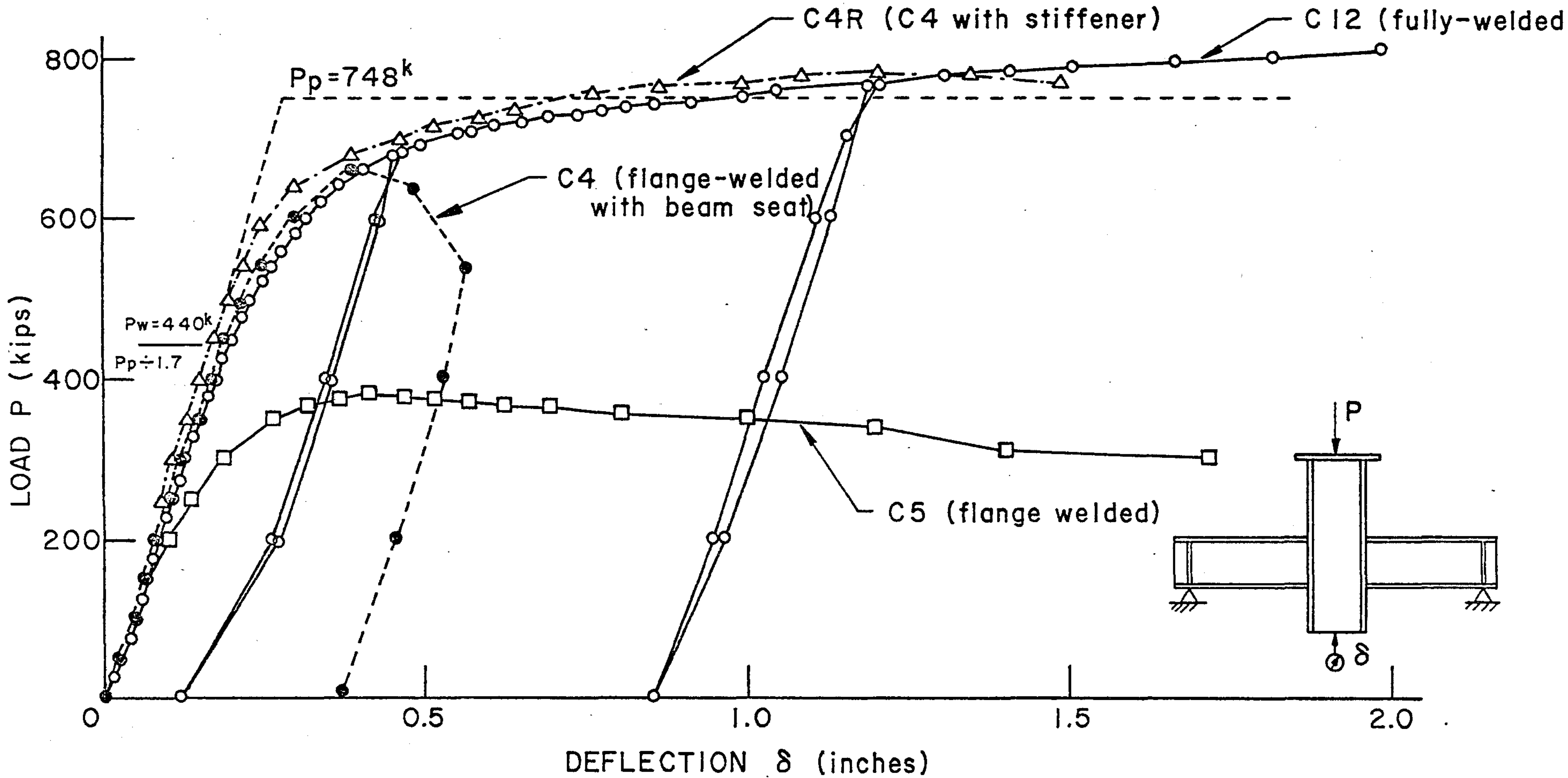


Fig. 12 Load-Deflection Curves

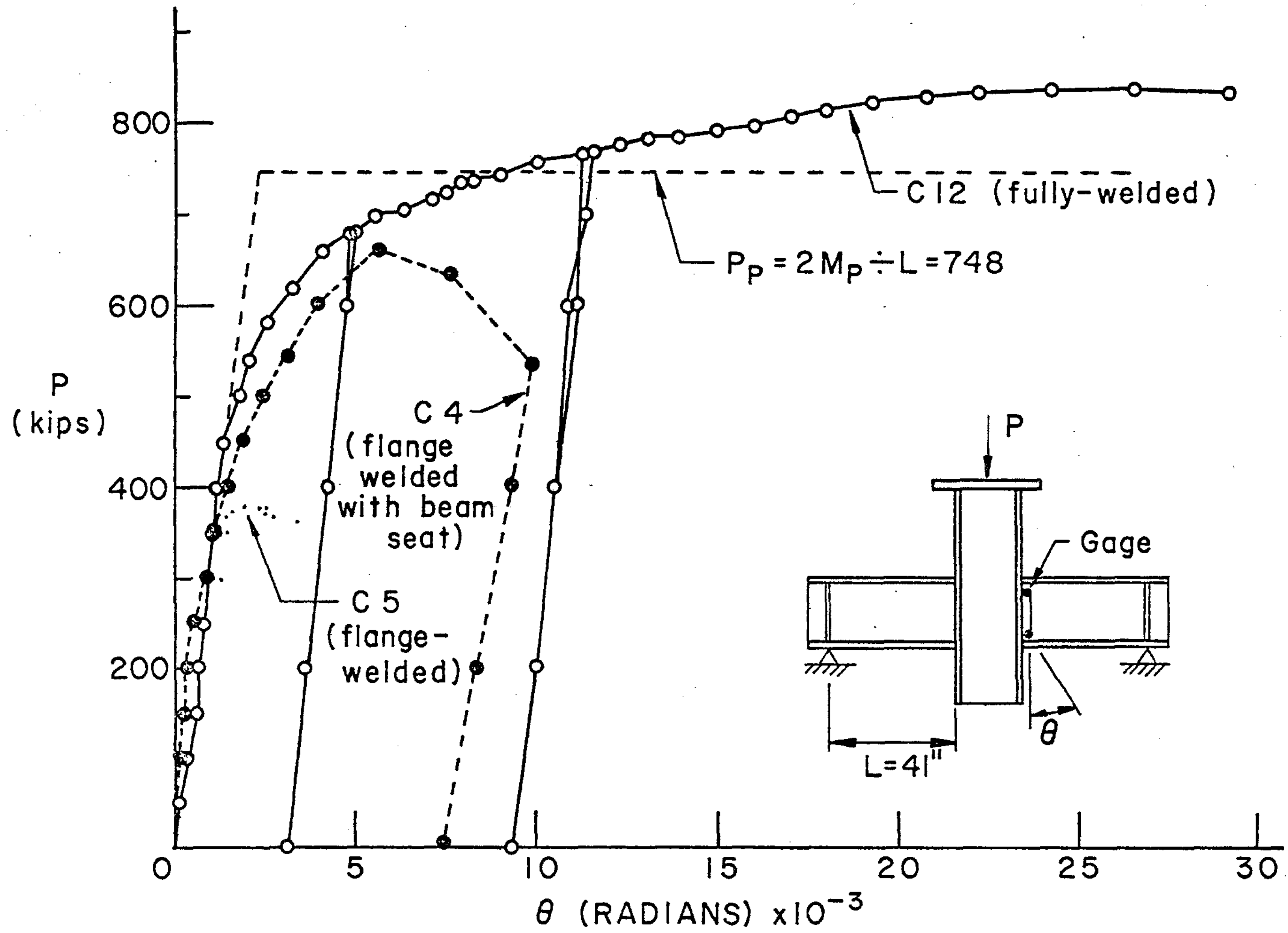


Fig. 13 Load-Rotation Curves

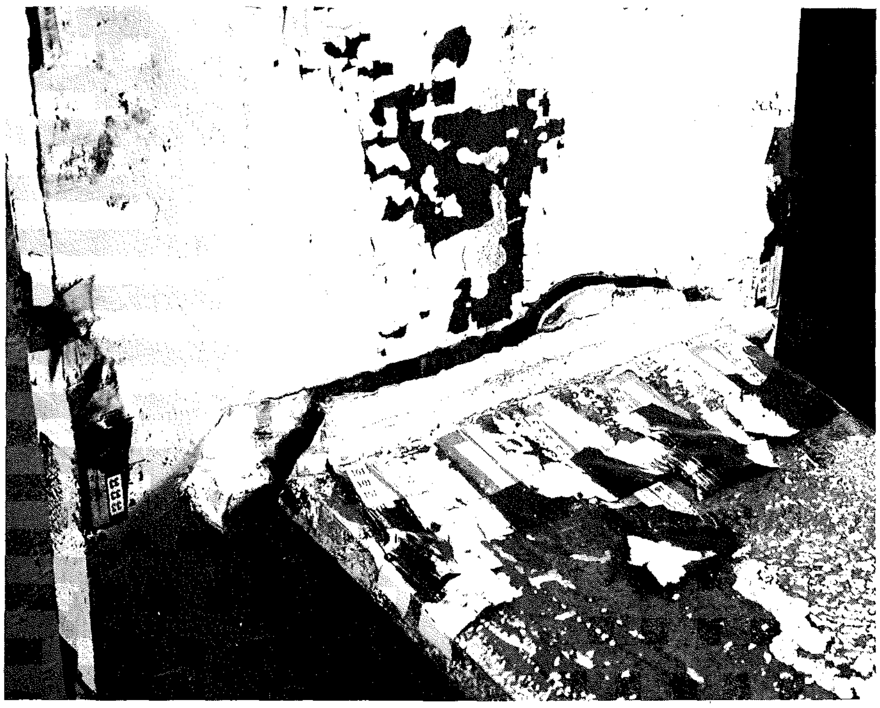


Fig. 14 Weld Fracture at Tension Flange of C12

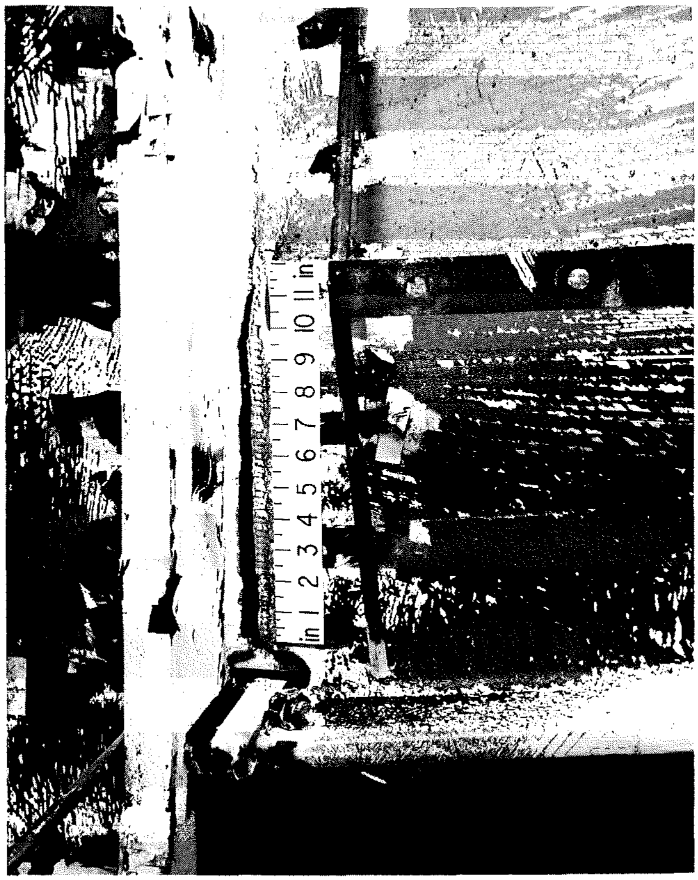


Fig. 15 Fracture of Weld Along Beam Web of C12

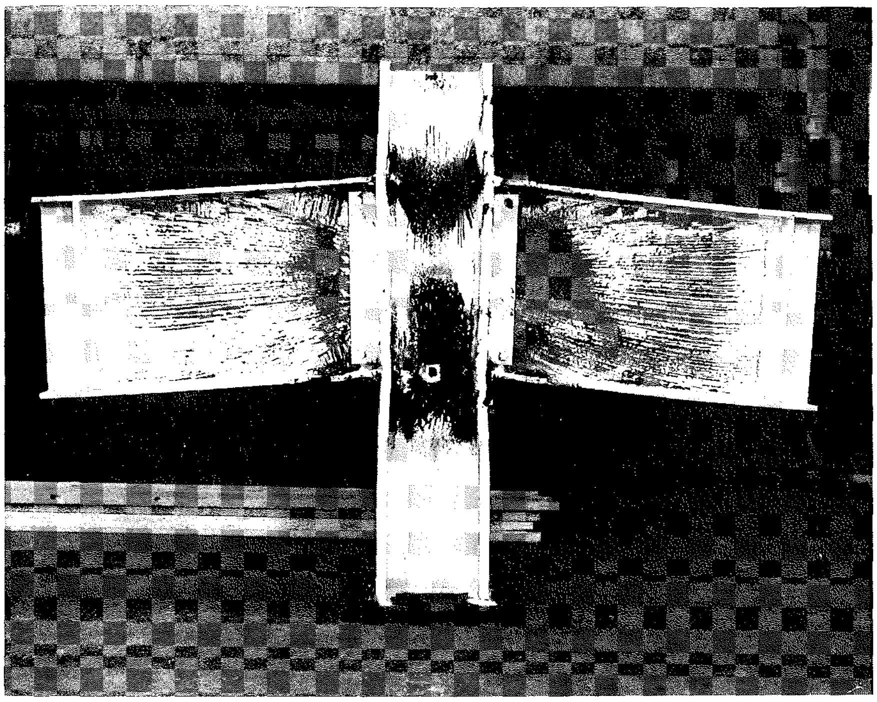


Fig. 16 Connection C12 at End of Test

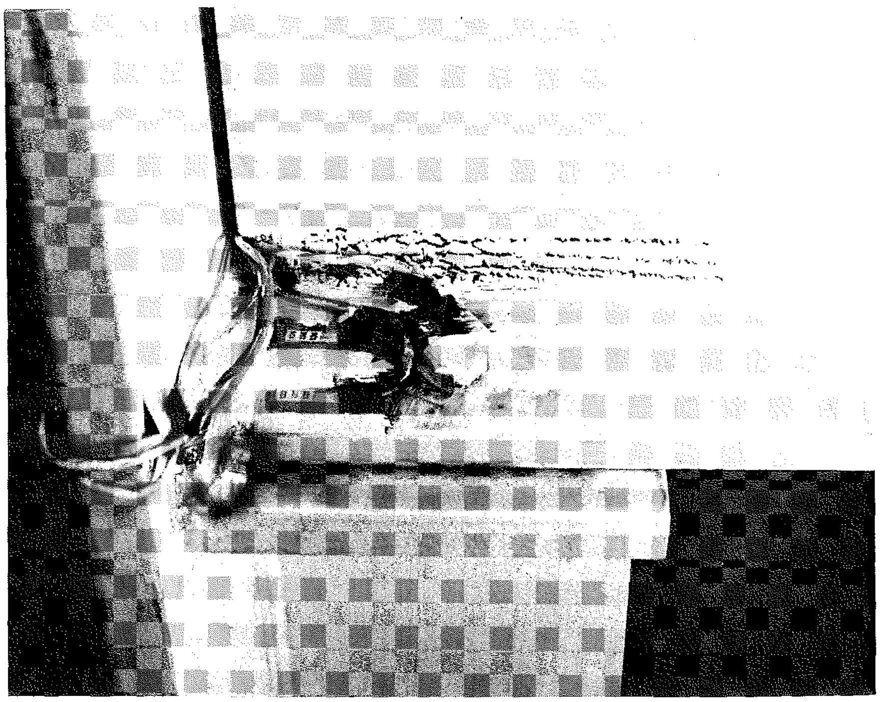


Fig. 17 Yielding at Cope of Test C4

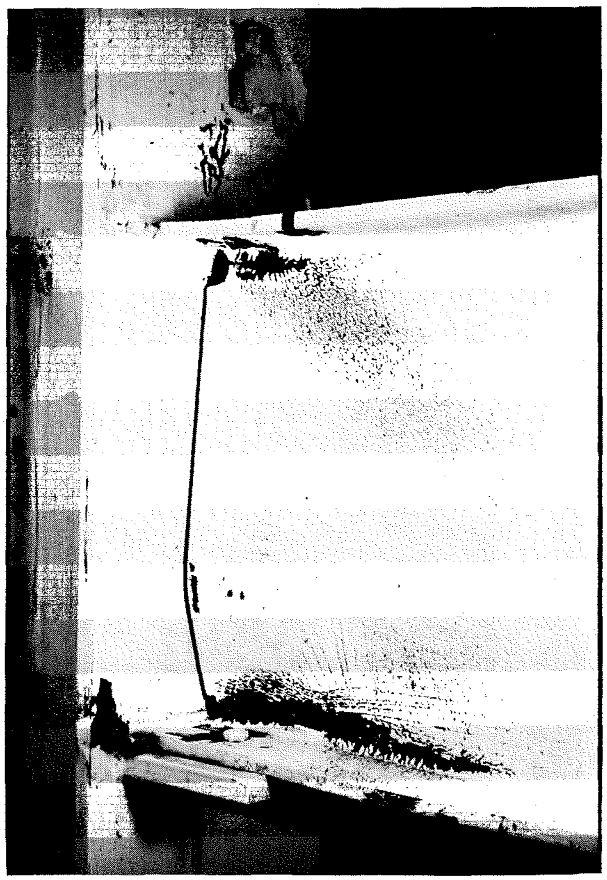


Fig. 18 C4 at Failure

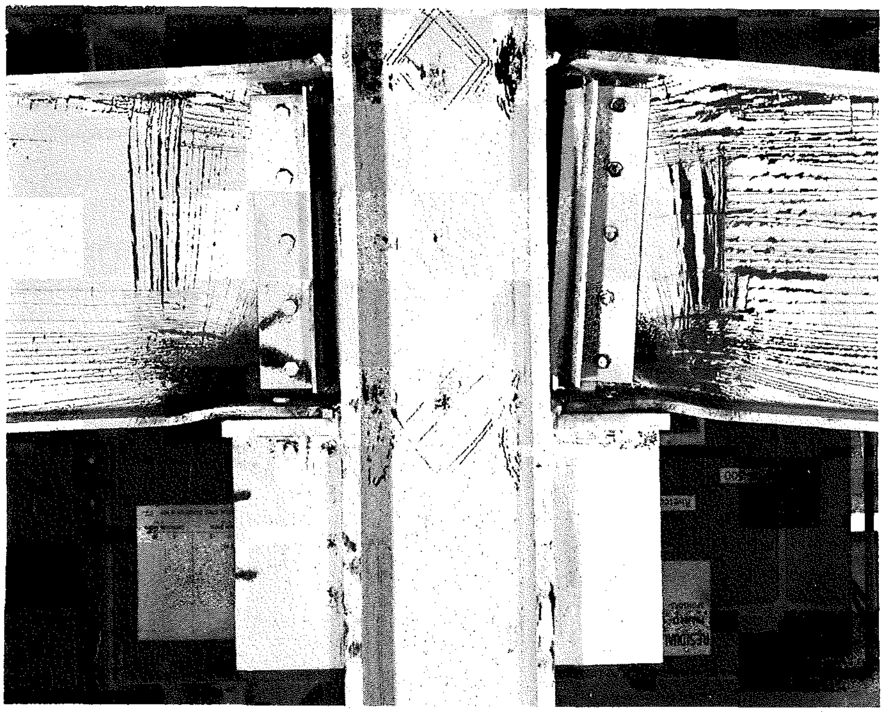


Fig. 19 Connection C4R at End of Test

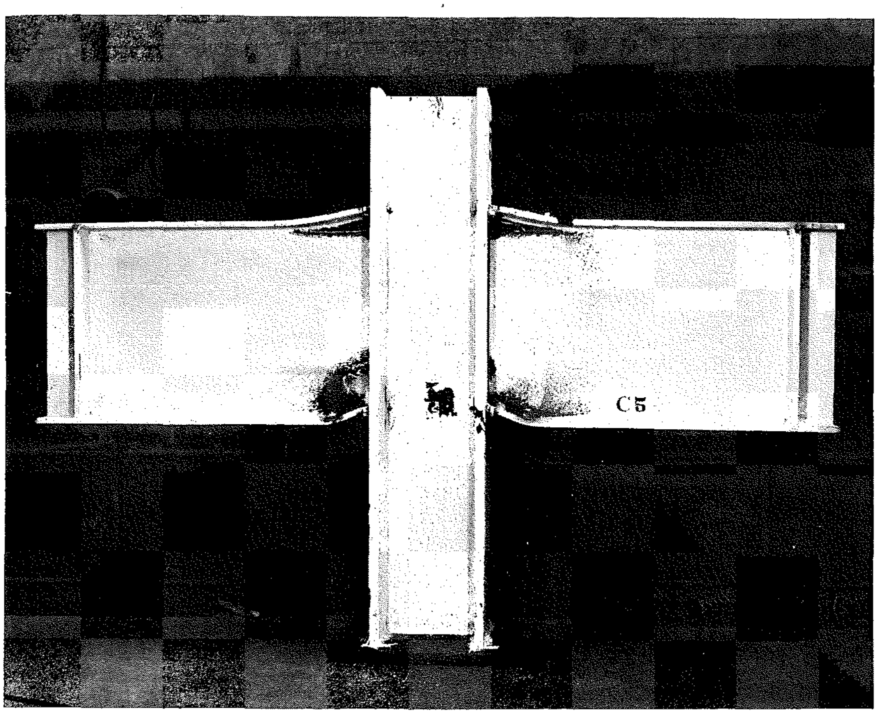


Fig. 20 Connection C5 at End of Test

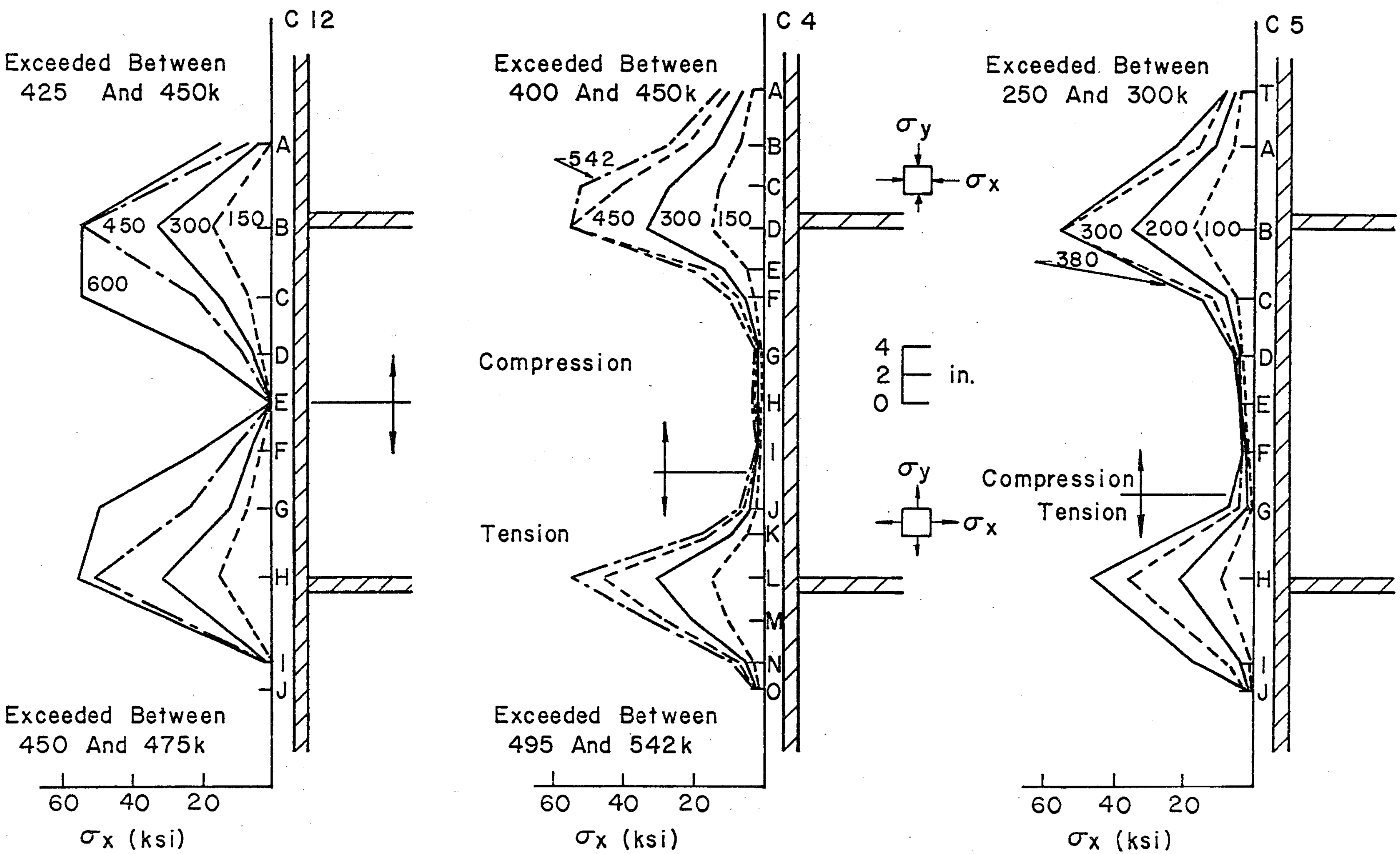


Fig. 21 Variation of Horizontal Stress (σ_x) Along Column Innerface (K-Line)

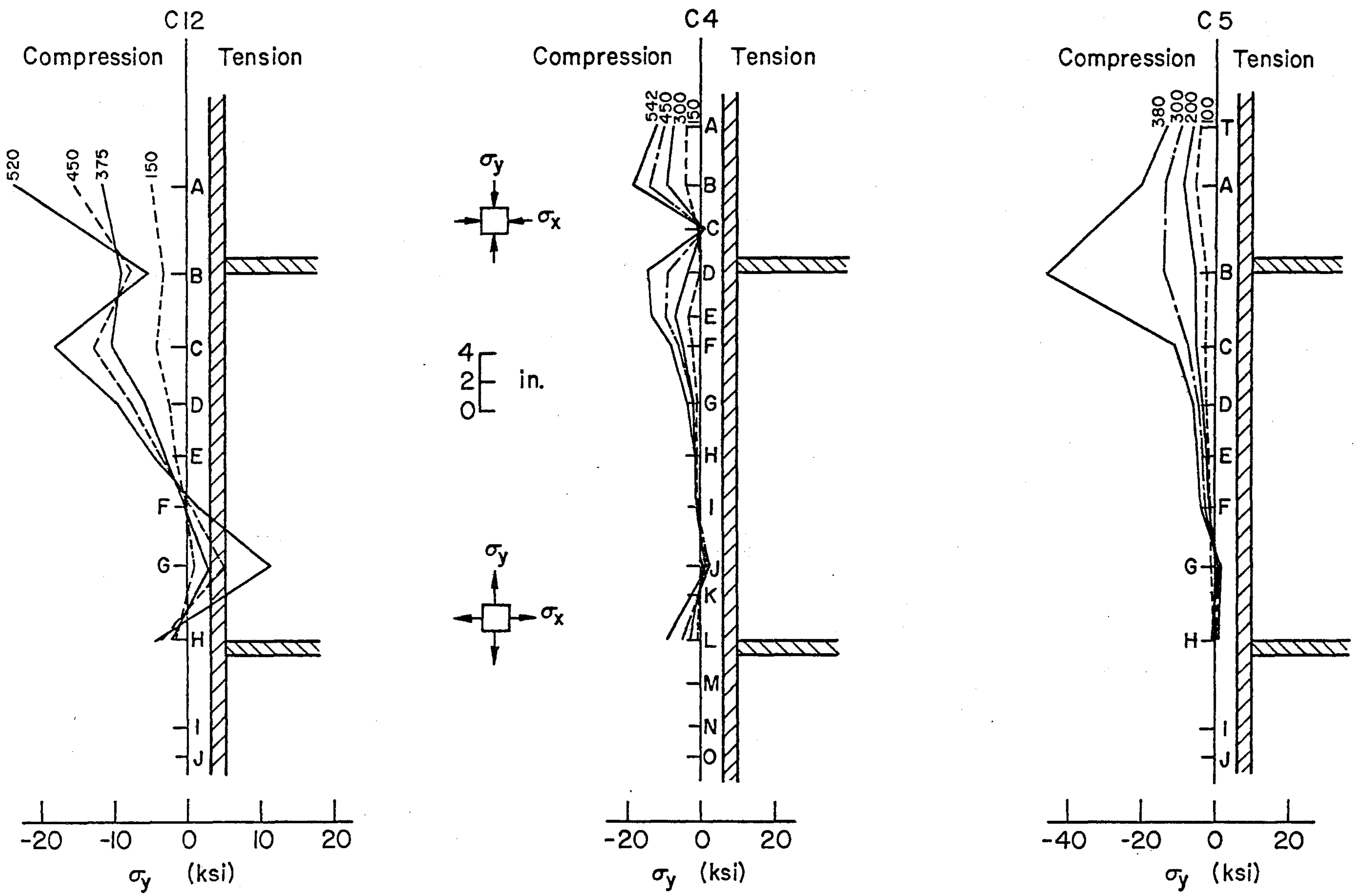


Fig. 22 Variation of Vertical Stress (σ_y) Along Column Innerface (K-Line)

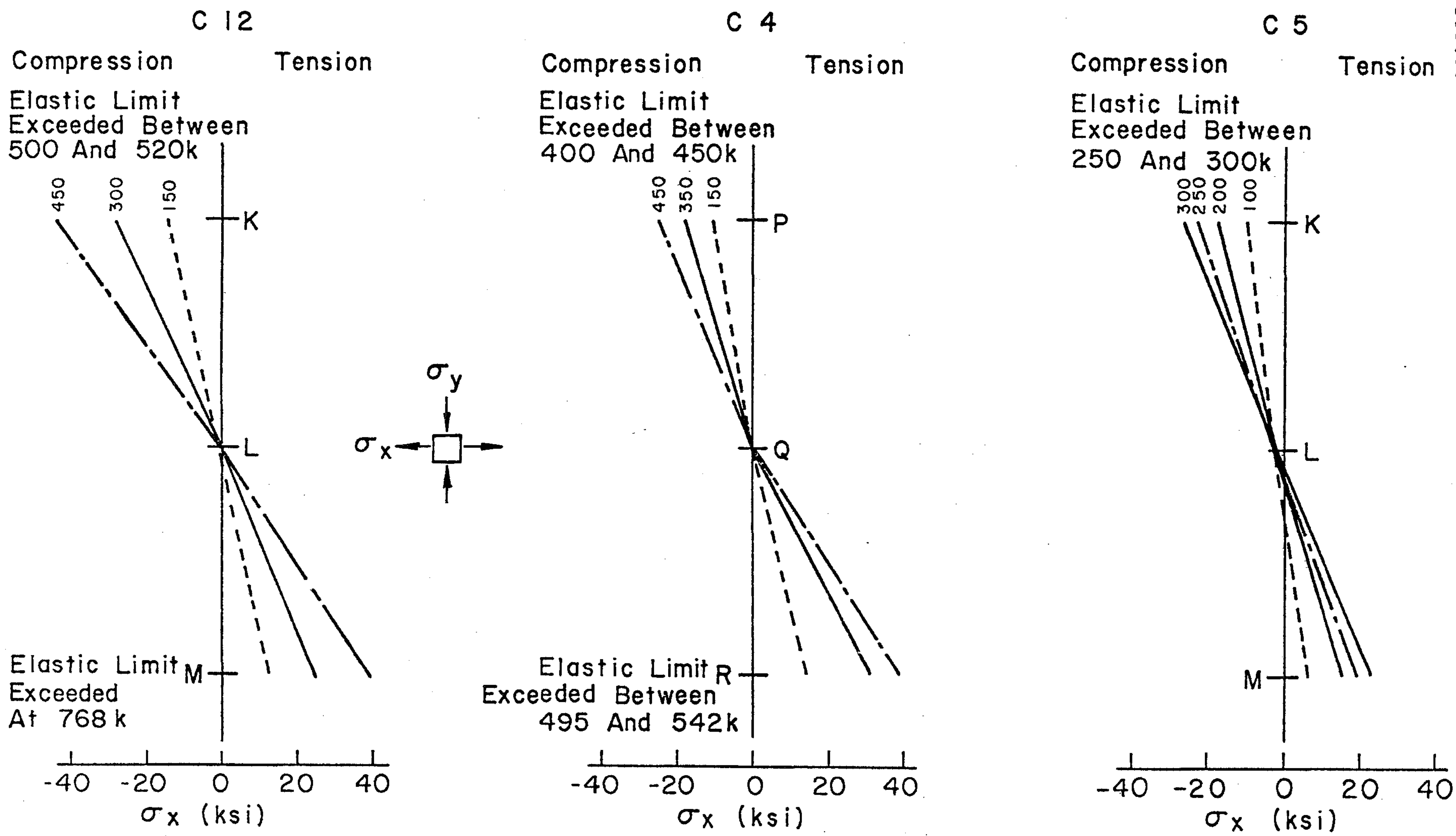


Fig. 23 Variational Horizontal Stress Along Column Centerline

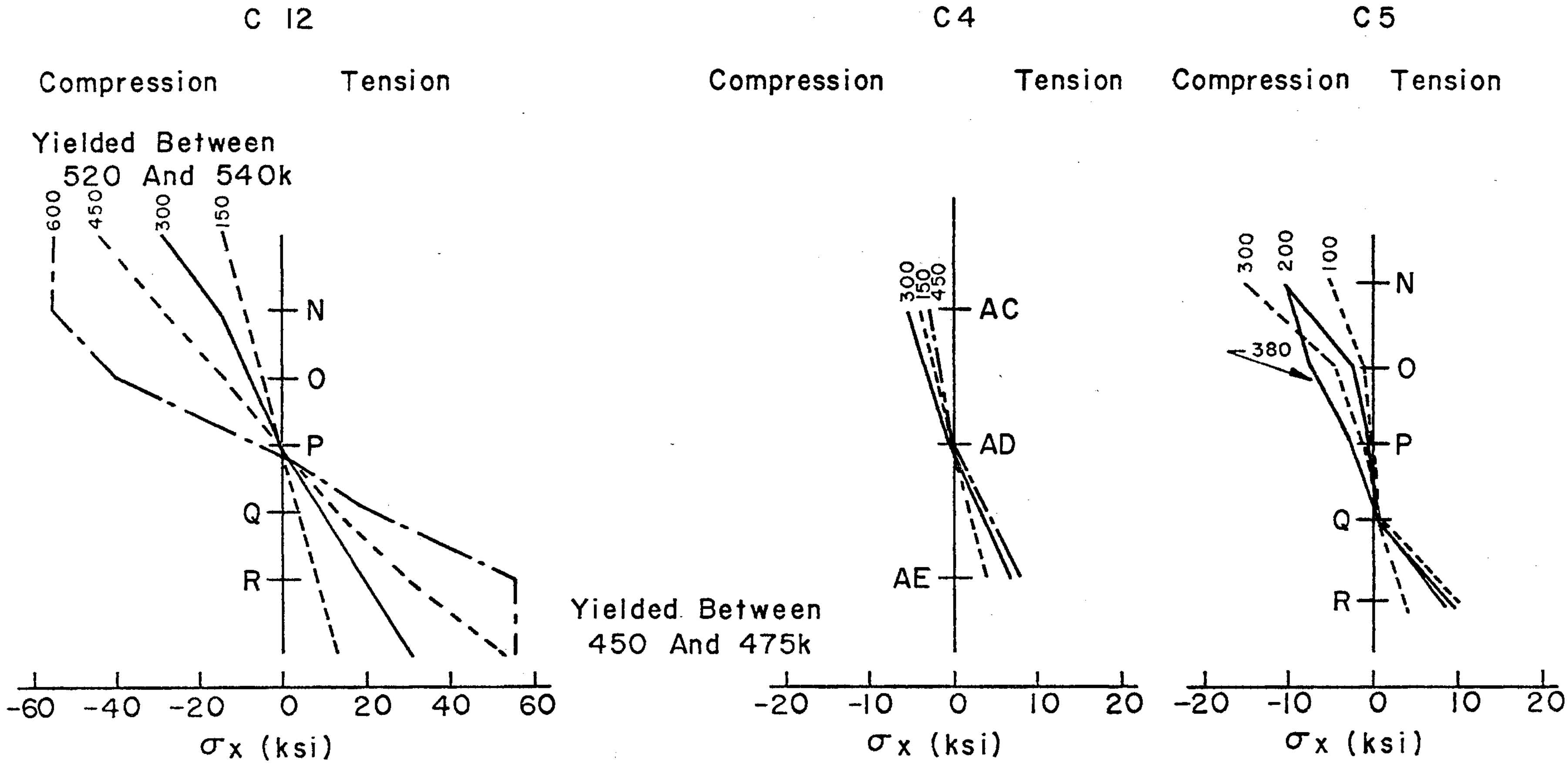


Fig. 24 Horizontal Stress (σ_x) Variation Through a Beam Section

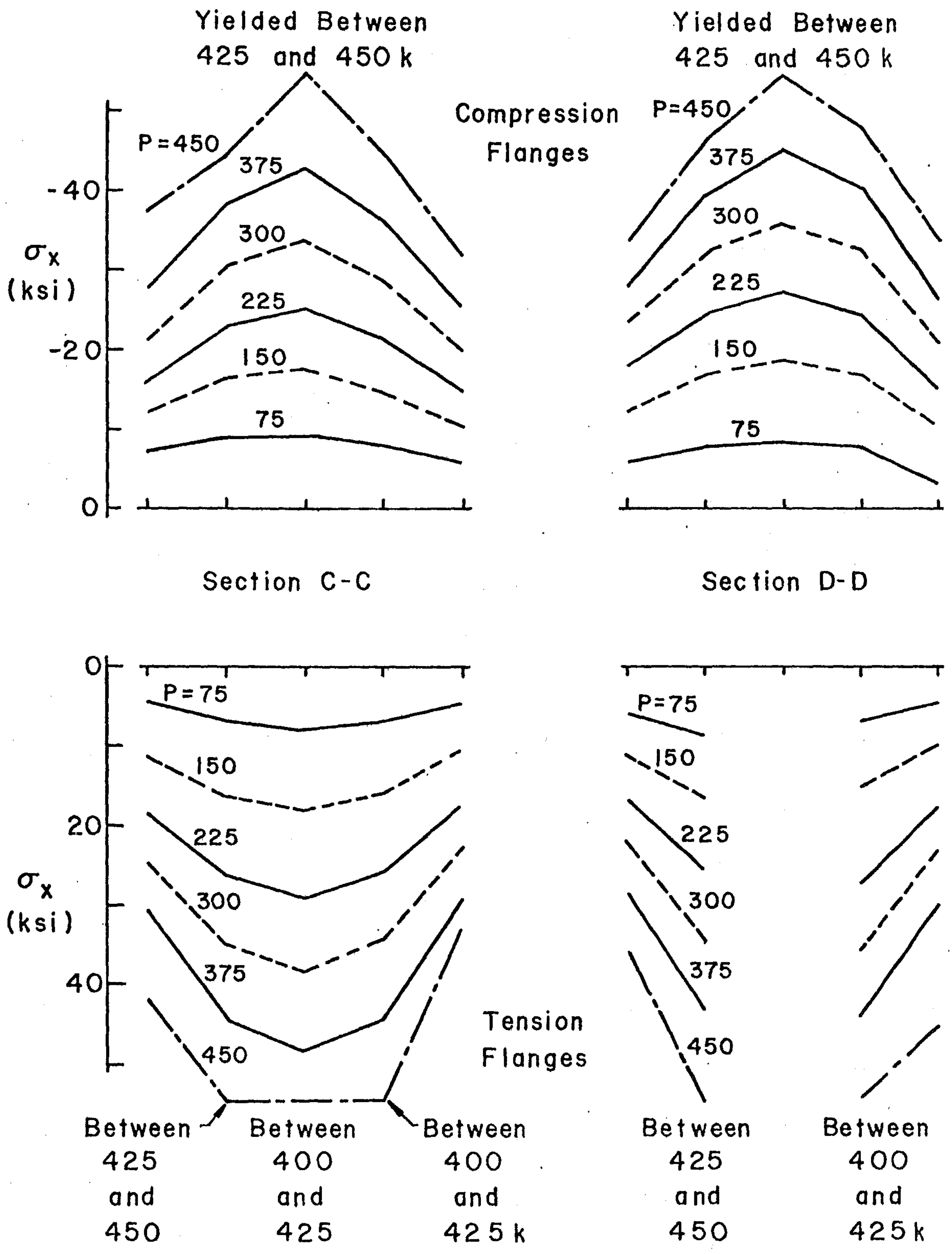


Fig. 25 Variation of Stress Across Beam Flanges Adjacent to Column--C12

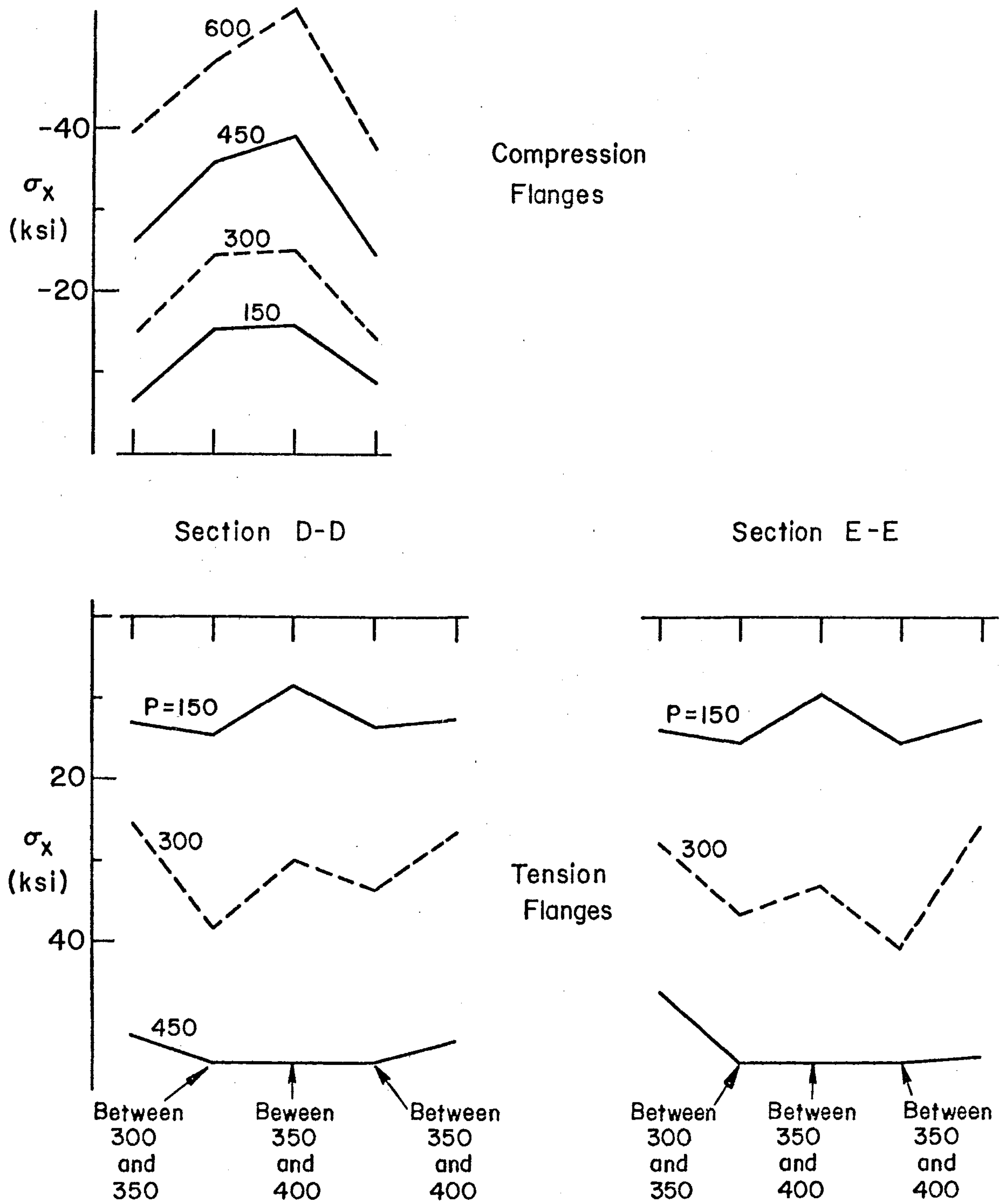


Fig. 26 Variation of Stress Across Beam Flanges Adjacent to Column--C4

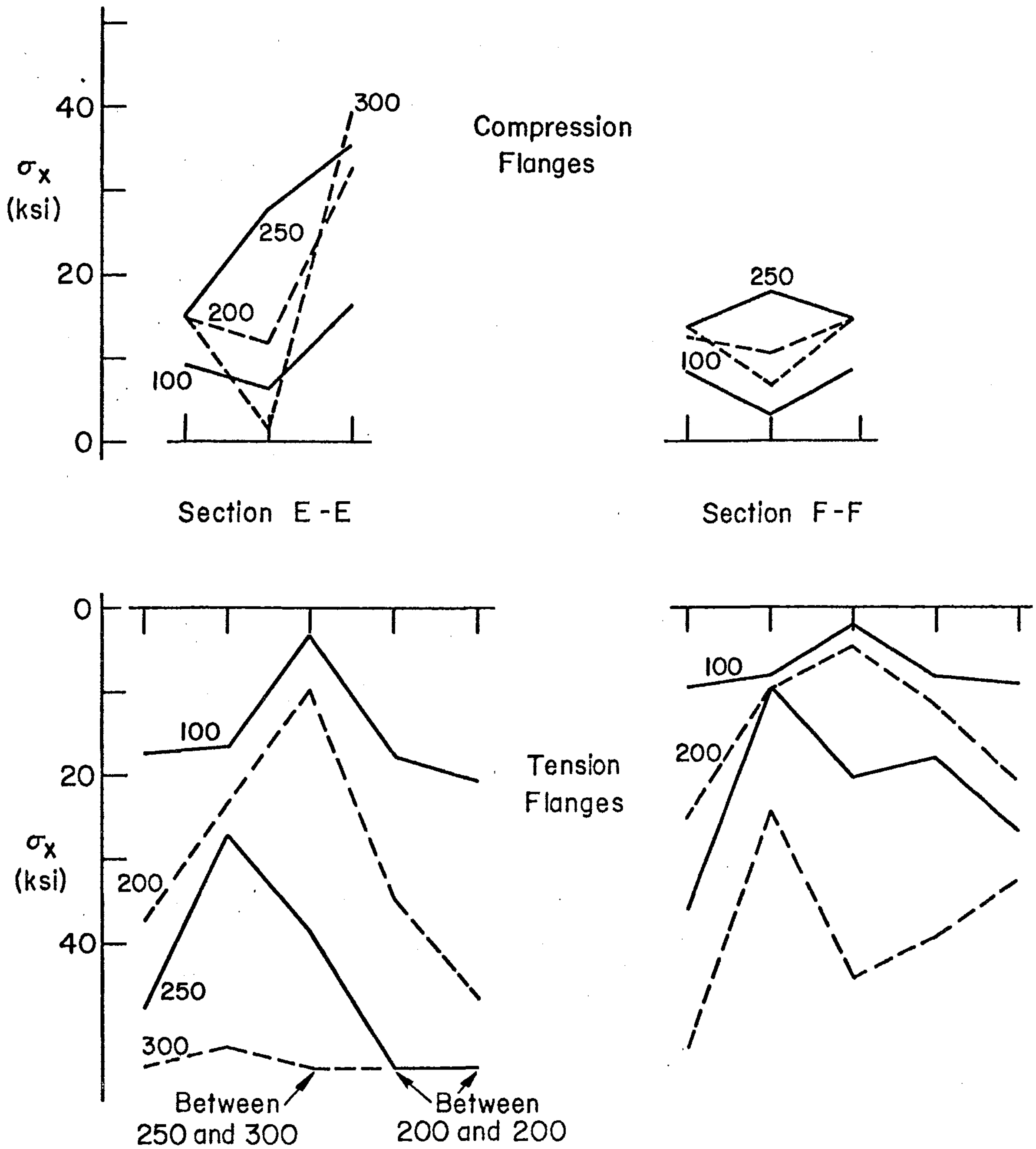


Fig. 27 Variation of Stress Across Beam Flanges Adjacent to Column--C5

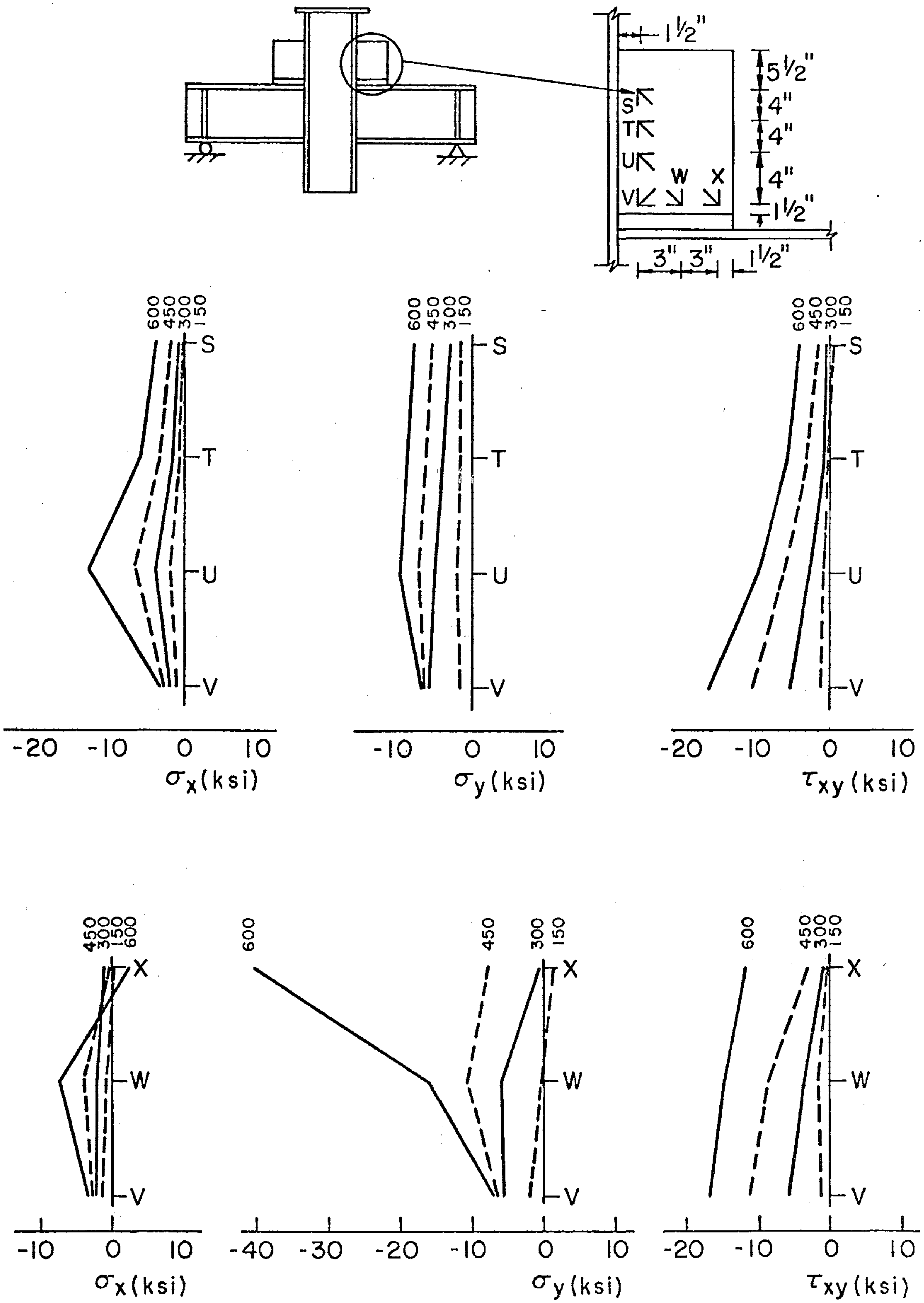


Fig. 28 Stress Variation in Beam Seat--C4

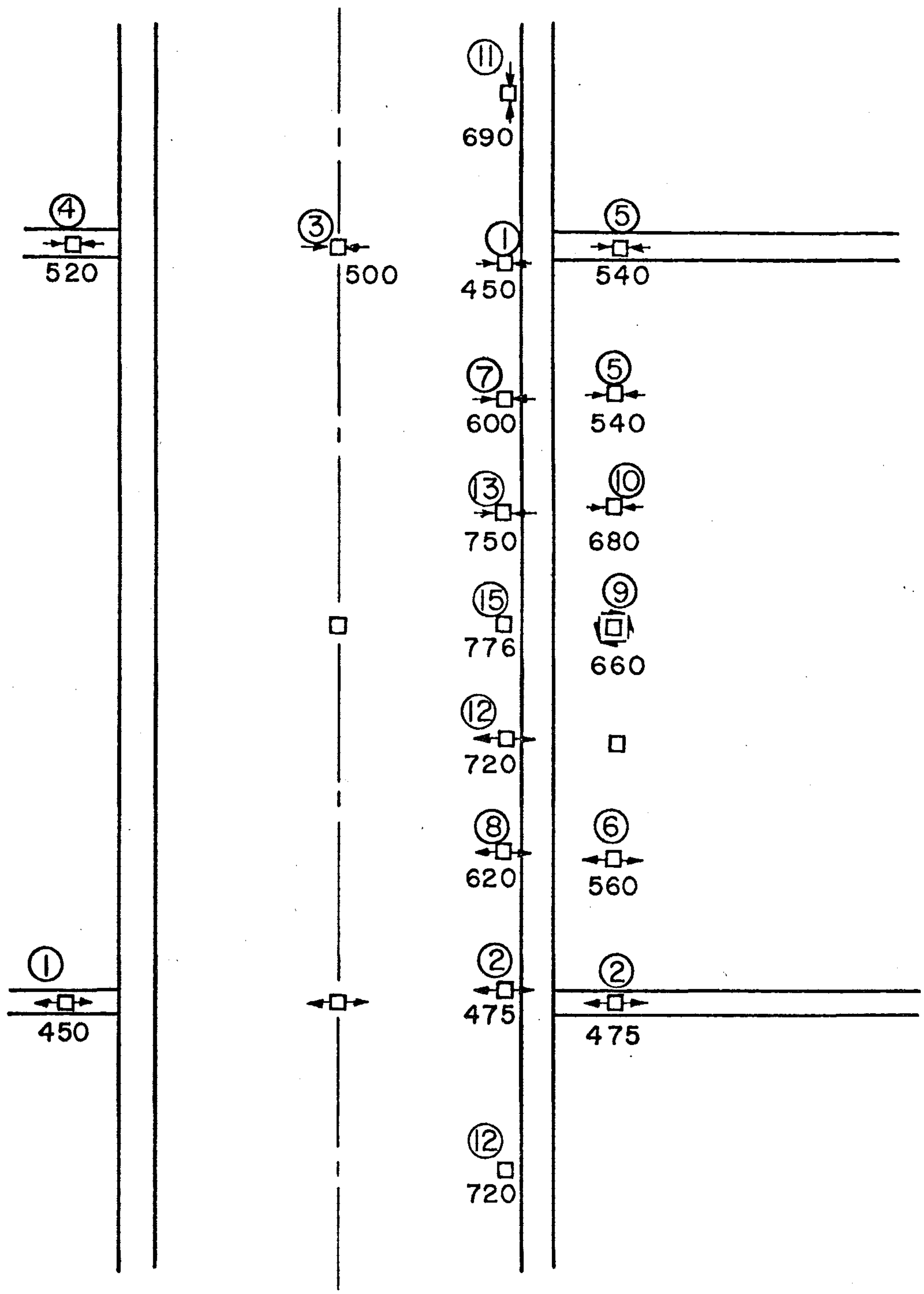


Fig. 29 Sequence of Panel Zone Yielding for Specimen C12

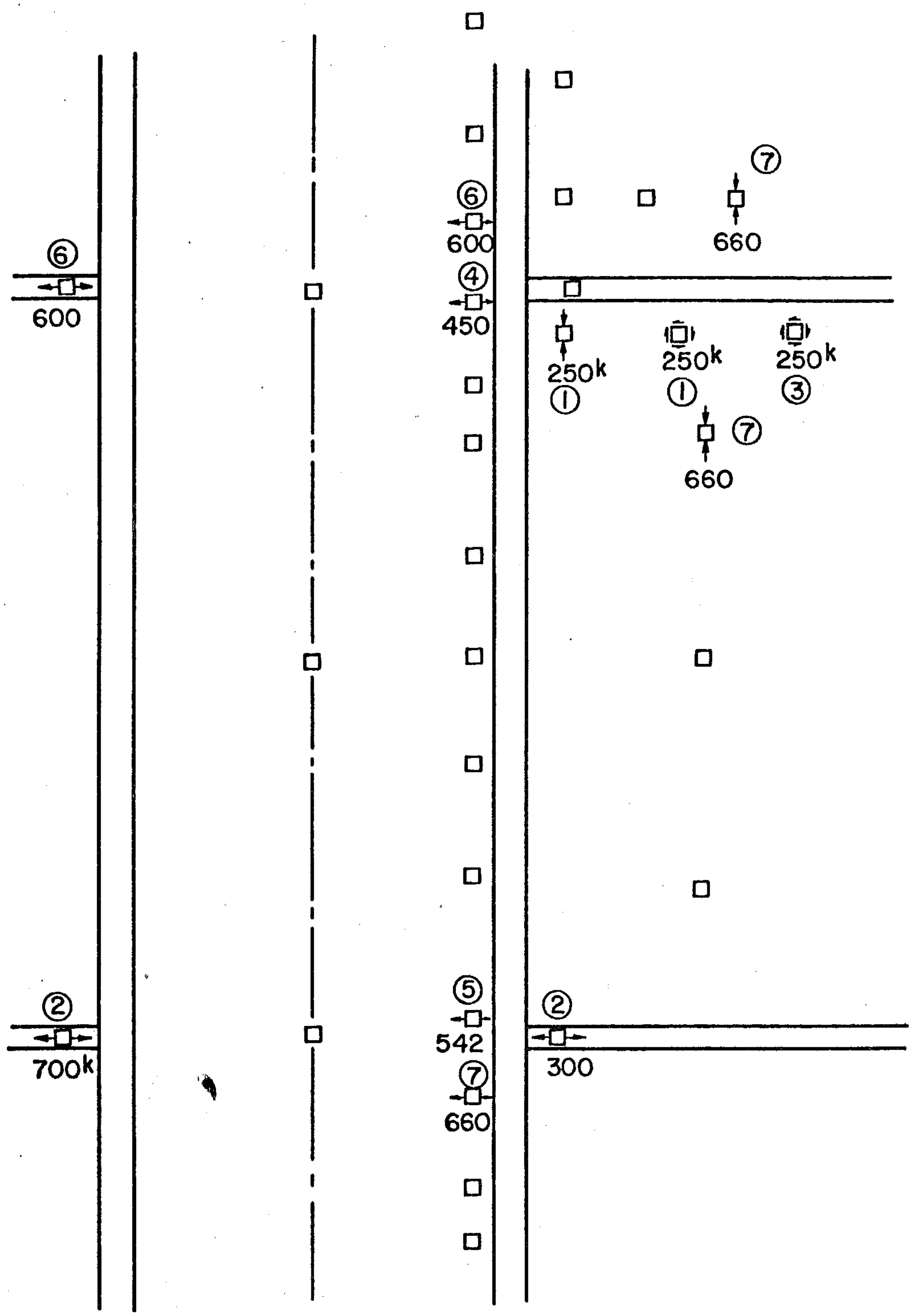


Fig. 30 Sequence of Panel Zone Yielding for Specimen C4

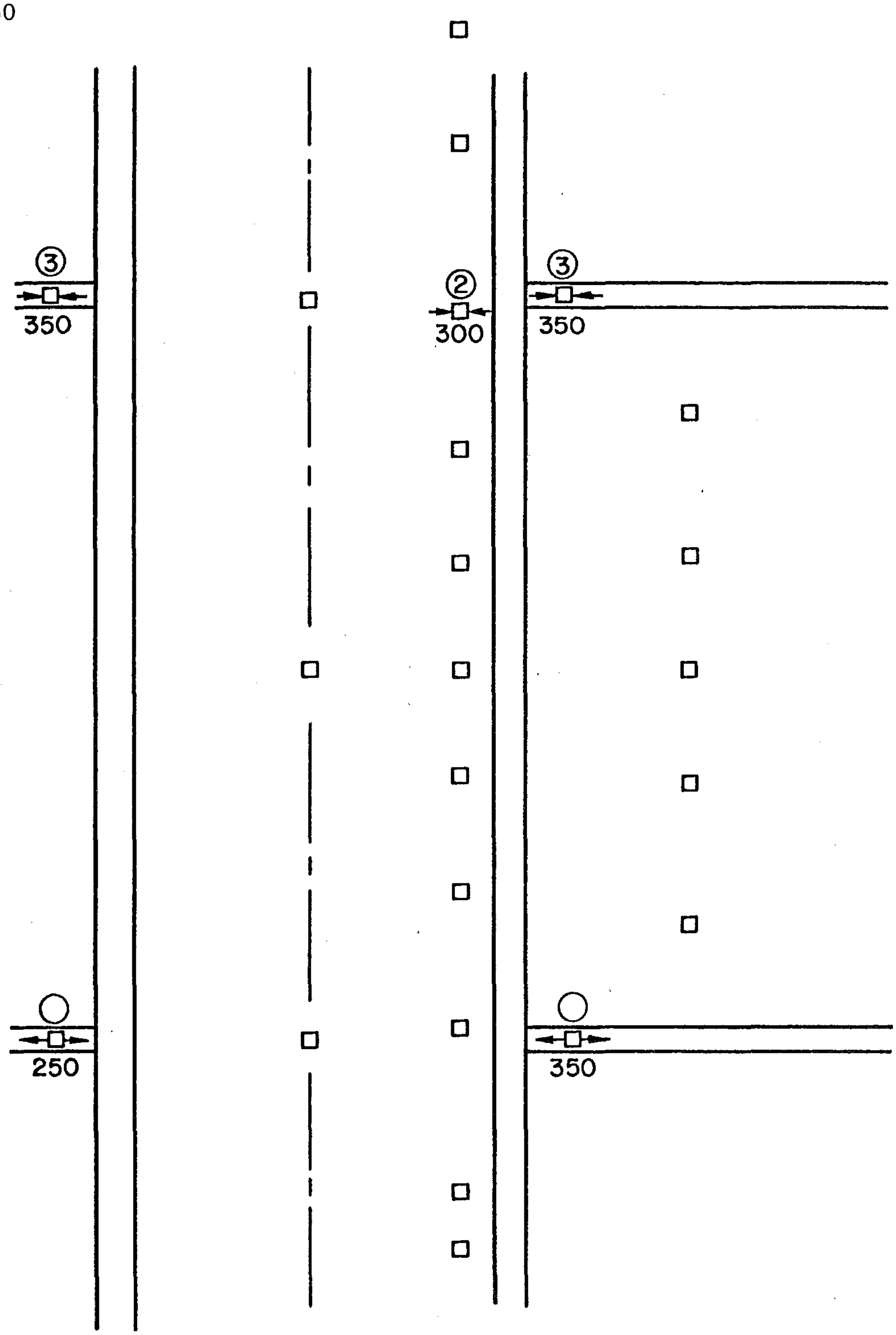


Fig. 31 Sequence of Panel Zone Yielding for Specimen C5

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