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# TWO BEAM-TO-COLUMN WEB CONNECTIONS

by

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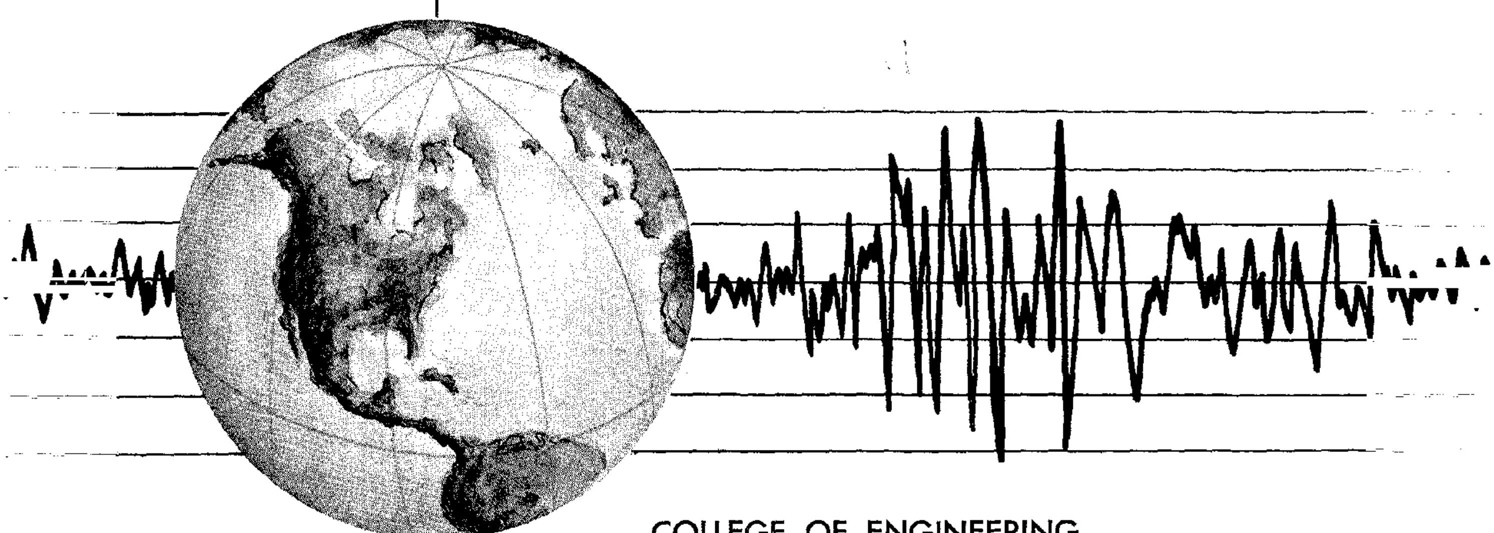

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# TWO BEAM-TO-COLUMN WEB CONNECTIONS

## ALTERNATIVE DESIGNS AND TESTS

## OF TWO

## BEAM-TO-COLUMN WEB SEISMIC MOMENT CONNECTIONS

by

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# ABSTRACT

In this report test results for experiments on two half-scale steel beam-to-column web moment connections are described. In the first experiment an innovative concept aimed at enhancing the ductility and strength of the connection by adding two pairs of reinforcing ribs was explored. In the second experiment the behavior of similar connections but without reinforcing ribs—a type that is common in practice—was evaluated. The new design was relatively simple to fabricate and the reinforced connection exhibited excellent strength and ductility characteristics. The results show that these moment connections are suitable for severe seismic service.

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# CHAPTER 1 INTRODUCTION

## 1.1 General

In current U.S. practice, the analysis and design of building structures under loads are typically carried out using elastic theory [1,2]. The design of earthquake-resistant structures is therefore challenging because it is not economical to design structures to resist severe earthquakes elastically, especially since such severe ground motions rarely occur during the service life of a structure. A compromise approach is consequently employed [3,4]. First, under moderate earthquake shaking, serviceability and functionality are to be maintained, requiring that a structure be sufficiently stiff to limit the story drift so that damage to nonstructural elements is minimized. Secondly, under major earthquake shaking, a structure is permitted to undergo inelastic action, but must not collapse. This requirement establishes the need for a structural system capable of absorbing and dissipating energy. Current building codes adopt the above concept and specify a minimum lateral load required for elastic analysis and design [2,5,6]. In building structures designed according to this approach inelastic deformation of members is allowable during major earthquakes, thereby dissipating the energy induced by the ground motion.

## 1.2 Moment-Resisting Steel Frame

The steel moment-resisting frame (MRF) is the structural steel framing system most frequently used in earthquake-resistant design [22]. The MRF can be designed to be ductile and to dissipate large amounts of energy during a severe earthquake [8,22,23]. The demand for energy dissipation on steel MRFs designed according to current building code requirements may, however, be very high in the event of a major earthquake [22]. The capacity of an MRF to dissipate energy will therefore primarily depend on the adequacy of the strength and ductility of the MRF at beam-to-column joints. Two types of connection are commonly encountered in beam-to-column joints of steel moment-resisting frames (Fig. 1.1). One is a connection to the column flange, and the other occurs at the corners of any three-dimensional framing system where beams are framed into the webs of columns. The latter type of connection is very important for tall tubular structural systems in which corner spandrel beams simultaneously apply moments and shears to corner columns to develop tube action [24].

### 1.3 Objective and Scope

Experimental research on steel beam-to-column moment connections has focused primarily on beam framing into column flanges [7-14], and little research on the behavior of beam-to-column web connections has been carried out [15-19]. Available experimental results have shown, unfortunately, that certain types of connection detail commonly used to connect beams to column webs perform very poorly under cyclic loading[16,17,26]. Accordingly, there is a great need to improve present methods used to detail such connections. A test program in which the behavior of a novel design of beam-to-column web moment connection was assessed and its behavior was compared to the best design used in current practice was therefore carried out. The concept developed for the new connection consists in reducing the stress concentration at the beam-to-column juncture by adding special ribs. The general details of this new design are shown in Fig. 1.2. The scope of the test program was as follows:

1. To obtain experimentally the cyclic strength and ductility for the reinforced connection;
2. To repeat the experiments for a connection commonly used in practice, and to compare the results; and
3. In both experiments to obtain experimental data on bolt slippage under cyclic loading, an especially important objective.

# CHAPTER 2 EXPERIMENTAL SYSTEM

## 2.1 Selection of Subassemblages

Examination of the force distribution in a typical MRF under severe lateral load reveals that it is reasonable to assume that the inflection points are located at mid-height of the columns and mid-span of the beams. The subassemblage shown in Fig. 2.1 was made from a W18x40 section. The member sizes were restricted by the need to limit the complexity of the test specimen and by the available materials. The column used for Specimen 1 was re-used in Specimen 2, and the two beams were cut from the same piece of rolled section. The material used for the two specimens—including beam sections, column, continuity plates, shear tabs, and reinforcing ribs—was ASTM A36 steel. All welding was accomplished using AWS E70 electrodes employing the shielded metal arc process [20]. The experimental set-up, including a test specimen, is shown in Fig. 2.2. Considering strain hardening of the beam, it was estimated that the column would remain elastic during the test. Although the effect of axial load in the column was not considered in this investigation, it is believed that the responses of the subassemblages provide a good indication of behavior that can be anticipated in actual assemblages in building frames.

## 2.2 Description of Test Specimens

### Specimen 1

The beam and column were of A36 steel fabricated from W18X40 and W12X133 sections, respectively. Two pairs of continuity plates and one shear tab were welded to the column with fillet welds. For each specimen, the W18X40 beam was then bolted to the column using four 1-in. diameter A325-X bolts through 1/16-in. oversize holes on the beam web and shear tab. The beam flanges were groove-welded to the continuity plates using full-penetration welds with 1/4-in. root openings. The back-up plates, 3/8 in. by 1 in., remained in place after the welds were completed. The overall fabrication details are shown in Fig. 2.3.

Note that the top continuity plates were 5/8 in. thick while the bottom plates were 3/4 in. This arrangement is used by some fabricators and creates a sound full-penetration weld regardless of the unavoidable variation in beam depth. Finally, two pairs of reinforcing ribs, 1/2 in. by 2 in. by 9 in. each, were welded to the connection as shown in Fig. 2.3. The plates were tapered at the ends to reduce stress concentration in the beam flange. Three 3/8-in. fillet welds each 2 in. long were made to attach the beam web to the shear tab to reduce joint slippage. All welding was visually inspected, and appeared to be comparable to that commonly seen in good fabrication shops.

**Specimen 2**

After Specimen 1 had been tested, the beam was cut off along the plane of the column-flange tips, and a new segment of W18x40 was attached by means of a new shear tab on the opposite side of the column web. The same details were used for Specimen 2 as for Specimen 1, except that no reinforcing ribs or welding on the beam web-to-shear tab was used. The fabrication details for Specimen 2, shown in Fig. 2.4, are typical of those used in the western part of the United States.

## 2.3 Experimental Set-Up

The experiment was conducted by applying cyclic loads to the horizontally mounted specimens at the tip of the cantilever, with no axial load on the column. The general arrangement is illustrated in Fig. 2.5. The ends of the column were anchored to the flange of a W24X145 rolled section that had been attached to a massive concrete block by post tension rods. Four 1-1/4 in. diameter grade A354-BD bolts were used at each end of the column to resist the shearing and tension forces induced by the applied load. In order to allow the column to deform under load, a 3/4-in. gap was provided between the flange tips of the column and the flange face of the supporting W24X145 by two bearing plates at each end of the column. The bearing plates were tapered at the ends to reduce the column end fixity and prying action of the anchor bolts (Fig. 2.2). A hydraulic actuator was used to apply load to the beam in the horizontal plane. A clevis was bolted to the end of the beam and a 3-in.

For Specimen 2, the cyclic loading was incrementally increased approximately as for Specimen 1 until the specimen fractured. Figures 2.9 and 2.10 illustrate the loading histories for Specimens 1 and 2, respectively. During the entire process, the loading was stopped at selected points to take readings with a low-speed scanner. A log was maintained during each test to record critical observations, such as slippage of bolts, flaking of the whitewash, etc.

# CHAPTER 3 EXPERIMENTAL RESULTS

## 3.1 Material Properties

All steel shapes and plates used to construct the specimens were of ASTM A36. Two tensile coupon tests were carried out to determine the steel properties using ASTM procedures [21]. The coupons were taken one each from the flange and web of the W18X40 beam. The test results are listed in Table 1. The stress-strain curves for the coupon tests were typical of those for A36 mild steel. The yield plateau before strain hardening occurred at 1.6% and 2.6% of strain for the flange and web coupons, respectively. The actual dimensions of the beam section were measured and found to be in good agreement with the values in the AISC Manual [1]. The small differences in size were therefore neglected and the section properties for W18X40 in the AISC manual were adopted. For Specimen 1, in which two pairs of ribs were added at the beam-column joint, the section properties of the beam at the juncture were obtained from the nominal plate dimensions and section properties of a bare W18X40. The section properties of the W12X133 column stub are given in the AISC Manual, 7th Edition.

## 3.2 Test Results

For both experiments the load-deformation data were obtained for cyclically applied loads. The resulting hysteretic loops provide the basic data for determining the behavior of the specimens. The maximum attained loads, the onset of yielding, and the ultimate inelastic deformations of the beams were measured in both tests and provided the data for the comparison of the two specimens. The column rotations at the beam-column joint versus the applied load are shown in Fig. 3.1 for both experiments. Note that the column behaved elastically during both tests. The same column stub could therefore be used for both experiments.

### Specimen 1

The relationship between the cantilever tip load and the beam end deflection for Specimen 1 is shown in Fig. 3.2. The beam responded elastically until the load was slightly beyond that required to reach the maximum beam bending stress of 36 ksi. The flaking of whitewash was first observed during the third loading cycle on the beam top flange outside the tips of the reinforcing rib. As cyclic loading progressed, the corresponding areas enclosed by hysteretic loops indicate the capacity of a member and its connections to absorb and dissipate energy. The loops consistently exhibited stable characteristics. During the sixth cycle, no significant deterioration of the loop occurred, although local buckling of the bottom flange outside the reinforcing ribs developed. Subsequently, both the top and bottom flanges had pronounced local buckling that appeared and disappeared cyclically. However, the beam-column assemblage maintained load-carrying capacity even when severe local buckling occurred in either of the flanges.

The web of the beam outside the shear tab buckled during the ninth cycle and was straightened and buckled cyclically from then on. The hysteretic loop remained stable. During the tenth cycle, some LVDTs were removed to prevent damage. The cantilever load was terminated after pronounced ductility had been observed. The maximum load applied by the actuator was about 67 kips. A slight reduction of peak load was detected during the last cycle. No visible slippage of the bolts was detected in the shear tab. Cyclic buckling of the top or bottom flanges and beam web was observed in the region outside the line connecting the tips of the top and bottom reinforcing ribs. The buckling of the bottom flange during advanced stages of testing is shown in Fig. 3.3, and the buckling of the top flange is shown in Fig. 3.4. The buckling of the beam web is shown in Fig. 3.5. The extent of the yield pattern at the top continuity plate is shown in Fig. 3.6. The yield pattern in the shear tab is illustrated in Figs. 3.7 and 3.8.

**Specimen 2**

The cantilever tip load versus beam end deflection for Specimen 2 is shown in Fig. 3.9. The beam again responded elastically up to the load slightly beyond that required to reach the maximum nominal beam bending stress of 36 ksi. During the second cycle, the whitewash on the top continuity plate cracked. During the third cycle, a relative movement between the beam web and the shear tab was observed, causing the bolt to slip. Up to the end of eighth cycle, extensive slippage of the beam web, flaking of whitewash on both the top and bottom flanges as well as on the top continuity plate were observed (Fig. 3.10). The whitewash on the bottom continuity plate did not crack. During the ninth cycle, the beam top flange near the end of the web-cope cracked beginning at the center of the flange and propagating toward each side of the flange as the load was increased (Figs. 3.11 and 3.12). At this point, some clip gages were removed to prevent damage. The beam top flange subsequently fractured and the beam lost its load-carrying capacity. The crack closed and opened when subjected to subsequent cyclic loading. During this test the maximum load attained by the actuator was about 61 kips. The fractured specimen is shown in Fig. 3.13. Note the severity of the flaking of whitewash on the top continuity plate. The relative movement between beam web and shear tab due to the bolt-slippage is shown in Fig. 3.14.

**3.3 Summary**

Specimen 1 exhibited superb capacities to absorb and dissipate energy. Specimen 2 failed quite abruptly after cracking had begun at the top flange. In general, both specimens carried load well above the 36 ksi nominal yield strength of the beam. In both experiments, the hysteretic loops exhibited a considerable amount of strain hardening of the material. The experimental data are summarized in Table 2.

**TABLE 1 MATERIAL PROPERTIES OF W18X40**

| Coupon Location | Material Type | Yield Stress (ksi) | Tensile Stress (ksi) | Young's Modulus (ksi) | Elongation at Fracture (%) |
|---|---|---|---|---|---|
| Flange | A36 | 38.2 | 60.6 | 29525 | 25 |
| Web | A36 | 50.3 | 64.3 | 30198 | 28 |

**TABLE 2 SUMMARY OF OBSERVED RESPONSE**

| Specimen | Max. Attained Load (kips) | Max. Cantilever Deflection Before Failure (inch) | Failure Mode |
|---|---|---|---|
| 1 | 67.13 | 3.49 | Local Buckling But No Loss of Capacity |
| 2 | 61.34 | 2.09 | Fracture of Beam Top Flange |

# CHAPTER 4 DISCUSSION OF RESULTS

## 4.1 General

In both experiments, the load-deformation hysteretic loops were stable and remained remarkably reproducible during consecutive cycles. In Specimen 1, the added web welding that was capable of developing 22% of the web plastic moment capacity essentially eliminated bolt slippage. The flanges alone of the W18X40 had 70% of the plastic section modulus of the entire section. A task force of the SEAOC Seismology Committee, currently working on a revision of the Blue Book [5], has called for a requirement that the web welding be capable of developing 20% of the plastic capacity of the girder web if the girder flanges alone have less than 70% of the plastic section modulus of the entire girder section. Although a requirement of 20% appears to be justifiable, if the 70% rule is to be used, further investigation seems warranted. Excessive web slippage might have contributed to the fracture of the beam top flange in Specimen 2. Although it would have been beneficial to have had the results of testing a subassemblage using added web welds but without the top and bottom reinforcing ribs, there were, unfortunately, insufficient funds to conduct further investigations.

## 4.2 Analytical Comparisons

In order to gain further insight into the behavior of the two test specimens, some design parameters are examined below. The section properties and the moment capacities of the beam sections of both specimen are given in Table 3. The elastic moment capacities, $M_y$s, were obtained using a yield strength of 38.2 ksi from the flange coupon test. The moment capacities, $M_p$s, were obtained from an averaged yield strength of 44.25 ksi from the flange coupon and the web coupon. For Specimen 1, with rib reinforcement at the built-in end, it is useful to plot the variation of moment capacity along the axis of the beam (Fig. 4.1). The applied moment at any section can be determined from statics:

$$M = P \cdot X$$

where $P$ is the actuator force and $X$ is the distance from the loading point to the section of interest. Since the reinforcing ribs increase the section moduli substantially, the critical section will occur at the edge of the ribs (Fig. 4.1). Therefore, the elastic yield load, $P_y$, can be determined:

$$P_y = \frac{M_y}{L_c}$$

where $L_c$ is the distance measured from the applied load to the critical section. The plastic load, $P_p$, can be determined similarly:

$$P_p = \frac{M_p}{L_c}$$

The same approach can be used for Specimen 2. The critical section of this specimen is at the edges of the column flanges. The results calculated are given in Table 4. To appraise more accurately the inelastic behavior of the two test beams, the contribution due to the rotation of the column stub must be subtracted from the total displacements, such as those shown in Fig. 3.2. The applied load versus the beam displacement can thus be plotted for each specimen. Moreover, both the load and deflection with respect to their corresponding elastic limits can be normalized. Normalized results for Specimens 1 and 2 are shown in Figs. 4.2 and 4.3, respectively.

The so-called ductility factor, $\mu$, has been widely used as a measure of the inelastic behavior of structures. This quantity has been variously defined [23,25], and thus its value strongly depends on the definition used to derive $\mu$. When ductility factors are used for purposes of comparison, it is therefore important to consider the definition used to derive $\mu$, as well as the loading and the structural parameters involved. While a unique loading sequence would ideally have been applied to both specimens for the results to be truly comparable, it was decided that a quantitative as well as qualitative comparison would be valid based on the loading and structural consistency of the two experiments. Two ductility factors commonly derived from data such as that plotted in Figs. 4.2 and 4.3 are indicated in Table 4. In column 5 of this table, the maximum displacements $\mu_1$ were measured from the origin to the peak load, shown as point B in Figs. 4.2 and 4.3, during the last cycle. In column 6, the maximum displacements $\mu_2$ were measured from the zero load intercepts, shown as point A, to point B at the peak load. The beam in Specimen 1 was capable of deforming, without significant loss of capacity, into the inelastic range far beyond the point where the peak load had occurred. The ductility factors for Specimen 1 are therefore conservative. The active role of the reinforcing ribs in resisting the load is clear from Figs. 4.4 and 4.5, which show the strains in the reinforcing ribs during the cyclic loading. Note that the strain gages were located 1 in. from the top edge of the reinforcing ribs.

**TABLE 3 MECHANICAL PROPERTIES OF BEAM SECTIONS**

| Beam Section | Elastic Section Modulus $S_x(in^3)$ | Plastic Section Modulus $Z_x(in^3)$ | Elastic Moment Capacity $M_y(k-in)$ | Plastic Moment Capacity $M_p(k-in)$ |
|---|---|---|---|---|
| W18X40 With Ribs | 87.0 | 114.0 | 3320 | 5010 |
| W18X40 | 68.4 | 78.4 | 2610 | 3470 |

**TABLE 4 CRITICAL LOADS AND DUCTILITY FACTORS**

| Specimen (1) | Critical Distance $L_c$ (in) (2) | Elastic Yield Load $P_y(kips)$ (3) | Plastic Yield Load $P_p(kips)$ (4) | Ductility Factor $\mu_1$ (5) | Ductility Factor $\mu_2$ (6) |
|---|---|---|---|---|---|
| 1 | 60.3 | 43.3 | 57.7 | 5.2 | 10.6 |
| 2 | 65.3 | 40.0 | 53.1 | 4.0 | 6.5 |

# CHAPTER 5 CONCLUSIONS

## 5.1 Summary

Structural steel framing is widely used to resist lateral loads in regions of high seismic risk. Various structural framing systems have been evolved by which the overall stiffness and strength necessary for seismic resistance are provided. In particular, framed-tube structural systems consisting of closely spaced columns interconnected by deep spandrel beams have gained wide acceptance as the perimeter frame for tall buildings. The overall stiffness and strength of a typical rectangular tubular structure rely on the integrity between the planar frames in two mutually perpendicular directions. Joints and connections in the corners of such tubular structures are thus the most critical elements in the system. If wide flange columns are used in these corners, they must be connected to the spandrel girders in both strong and weak axes of the columns.

Research to date has provided limited data on the behavior of beam-to-column web moment connections. Moreover, experimental evidence has shown that some of the connection details commonly used may be dangerous during a major earthquake. This motivated an alternative concept in which reinforcing ribs are added to the conventional connection. To investigate the performance of this connection, an experimental program was conducted to assess the behavior as well as the effectiveness of the web-to-shear tab welding. A comparative test on a conventional connection of the best available type was also carried out.

## 5.2 Conclusions

The following conclusions can be drawn from the experiments:

1. Both specimens carried load well above the 36 ksi nominal yield strength of the beam, and a considerable amount of strain hardening took place in the post-yield range.

2. With added reinforcing ribs at the beam-to-column web juncture, the connection can sustain a large number of loading reversals without failure.
3. With a small amount of web welding, the slippage of the beam web with respect to the shear tab can essentially be eliminated in the relevant range of loading.
4. For beam flanges with more than 70% of the plastic section modulus of the entire beam section, some web welding might still be needed to mitigate against the slippage of the bolts under severe cyclic loading.
5. A good conventional connection detail similar to that used in Specimen 2 may fracture during a major earthquake.
6. With reinforcing ribs, the critical section of the beam is shifted away from the heat-affected zones next to the groove welds to the tips of the reinforcings ribs.
7. The concept of adding ribs to conventional beam-to-column web connections appears to enhance the seismic resistance of buildings. The height of the ribs is such that they become fireproof after the slab has been cast.

### 5.3 Future Research Needs

The pilot experimental work described in this paper was aimed at a specific set of geometric parameters using available materials. A test program covering a wider spectrum of sections, particularly deeper beams, is needed. The requirements of the beam web welding for a wide range of beam sections requires further investigation. Alternative details (as shown in Fig. 5.1) were considered for those beams with either wider or narrower flange widths than the beams tested. Since full penetration welds project above the top of the continuity plates, a smooth notch on the bottom of each rib is required as shown in Fig. 5.1. For Specimen 1, small crescent shape notches were made by grinding to allow placement of continuous welds along the ribs.

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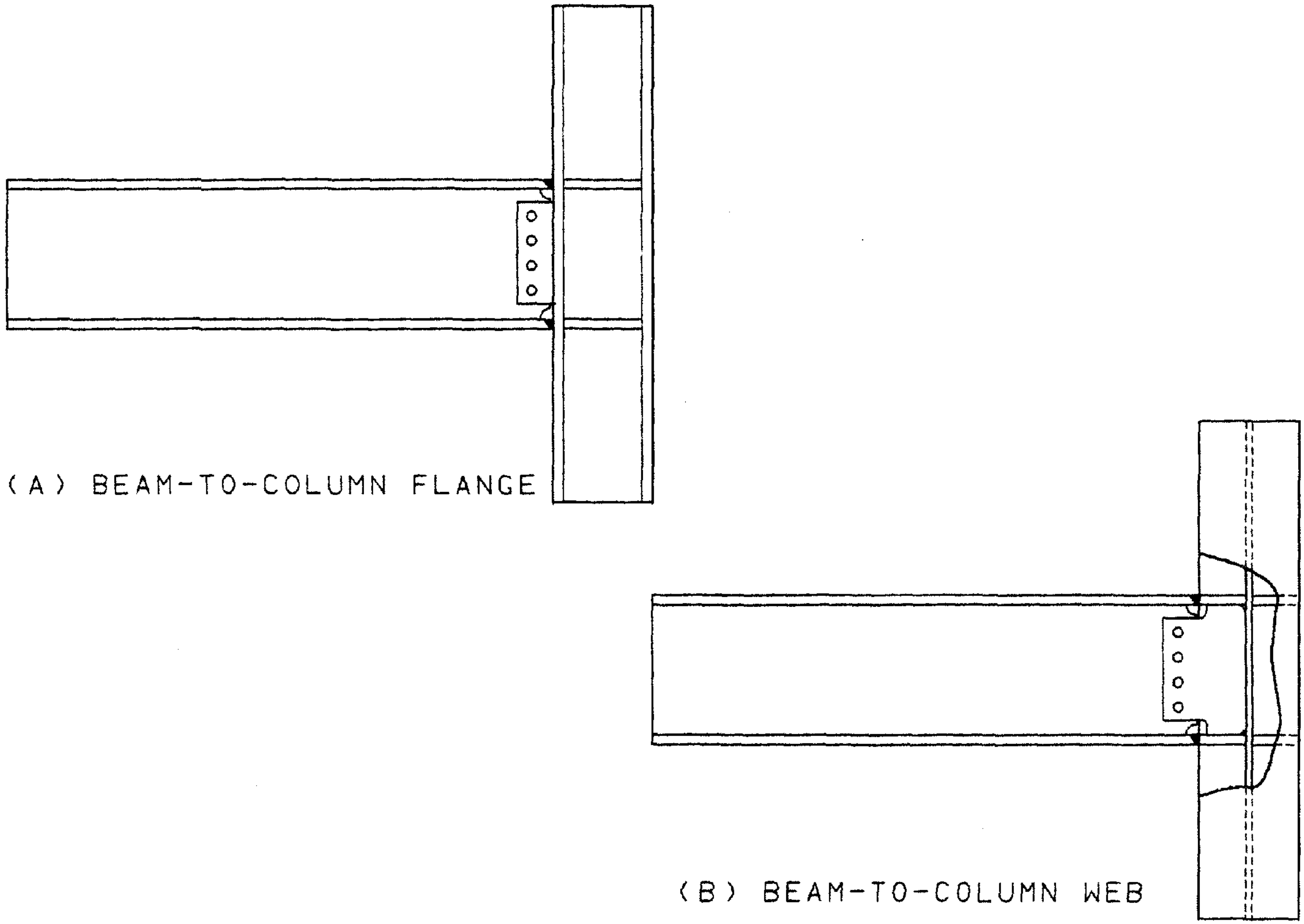



Fig. 1.1 Steel Beam-to-Column Moment Connections

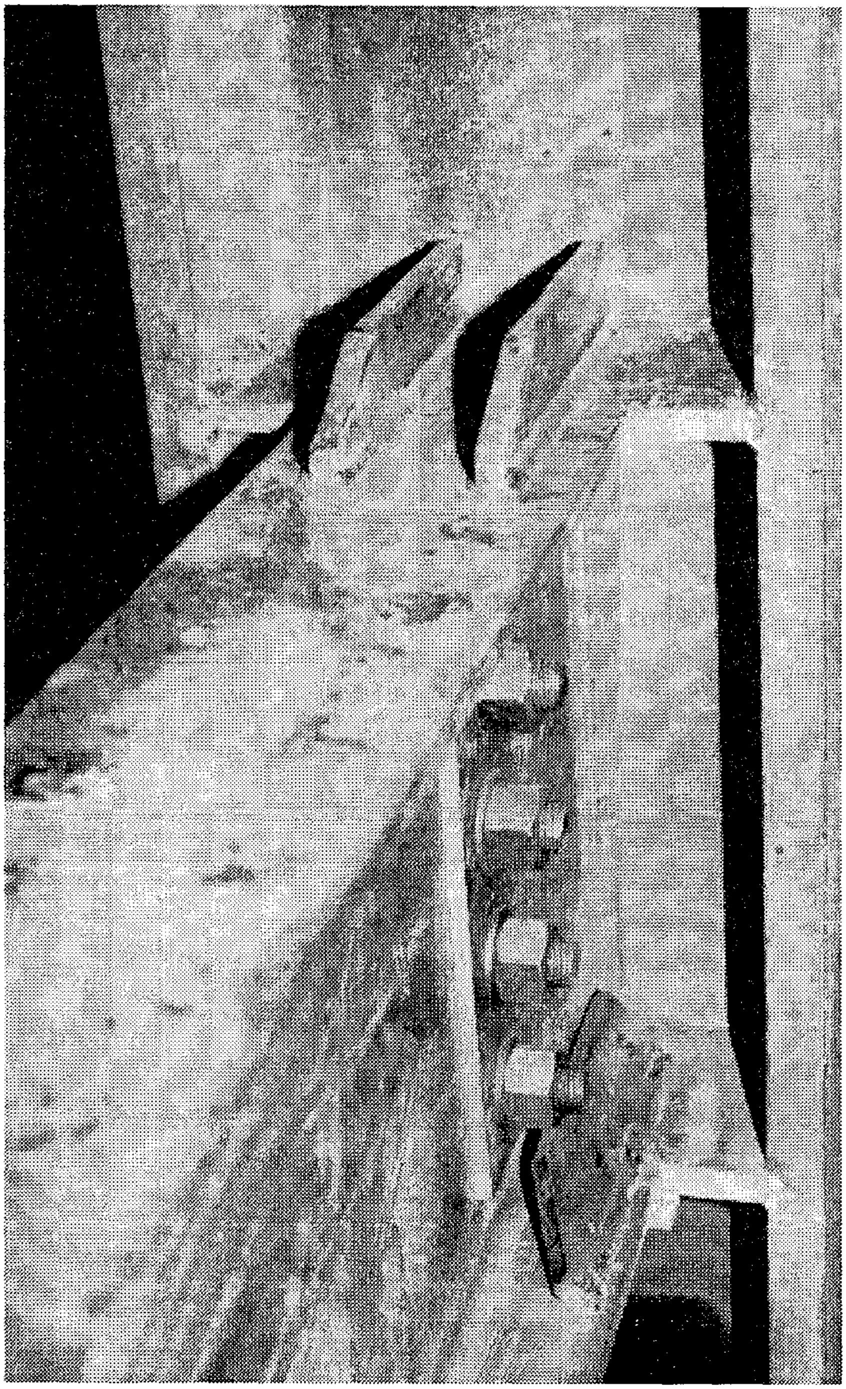


Fig. 1.2 General View of New Design

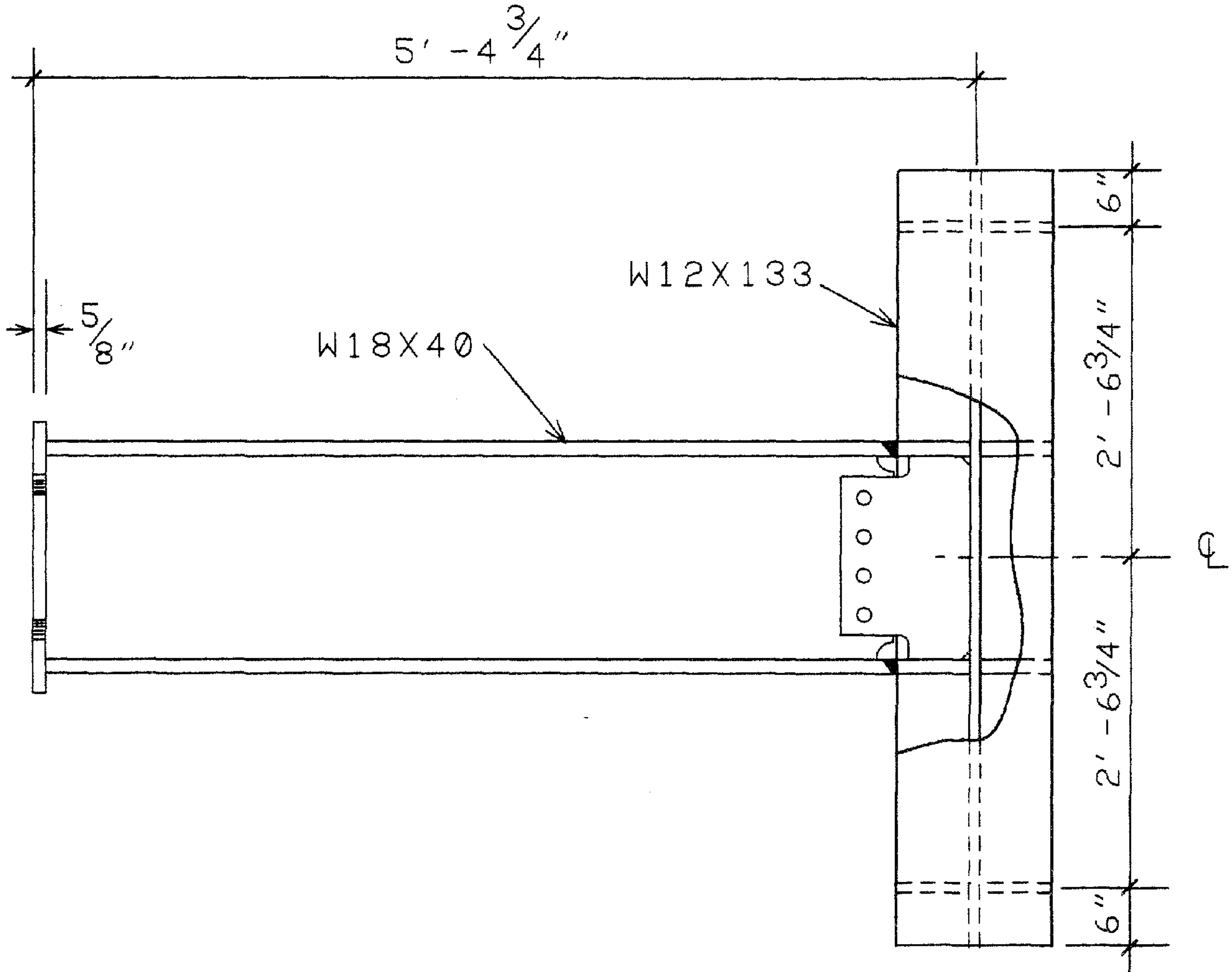



Fig. 2.1 Subassemblage of Beam-to-Column Web Moment Connection

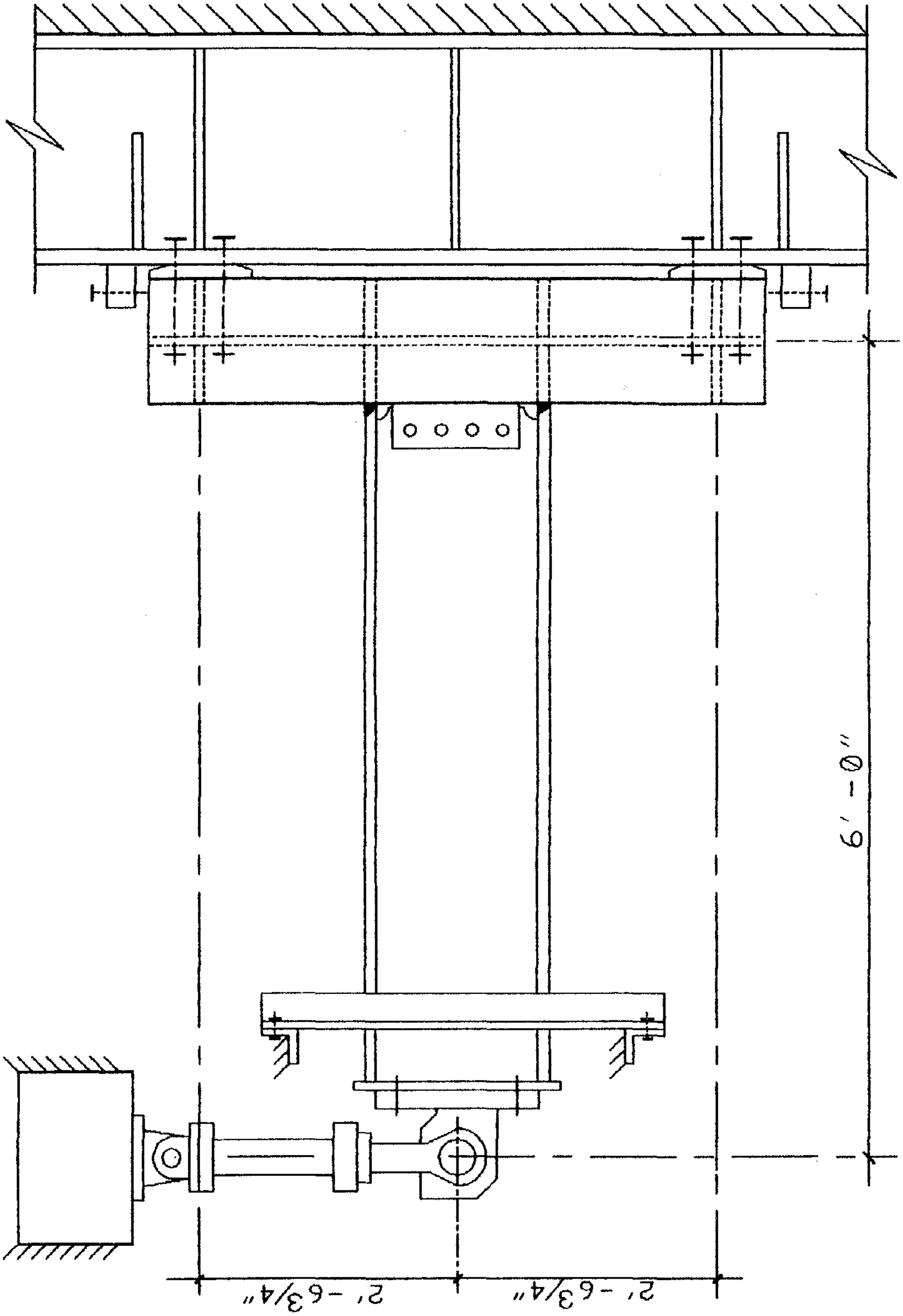



Fig. 2.2 Mounting Arrangement of Specimen

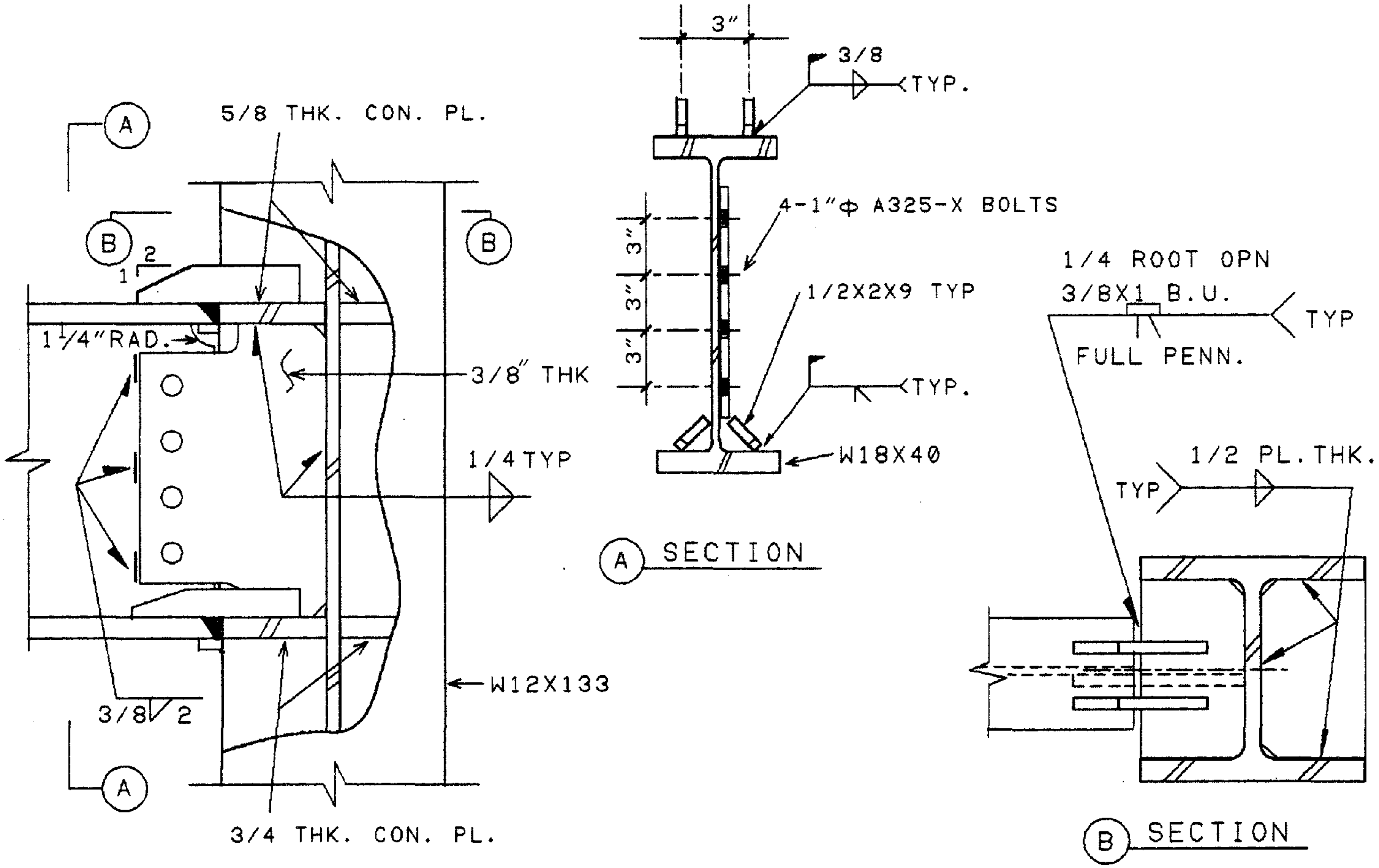



Fig. 2.3 Connection Detail (Specimen 1)

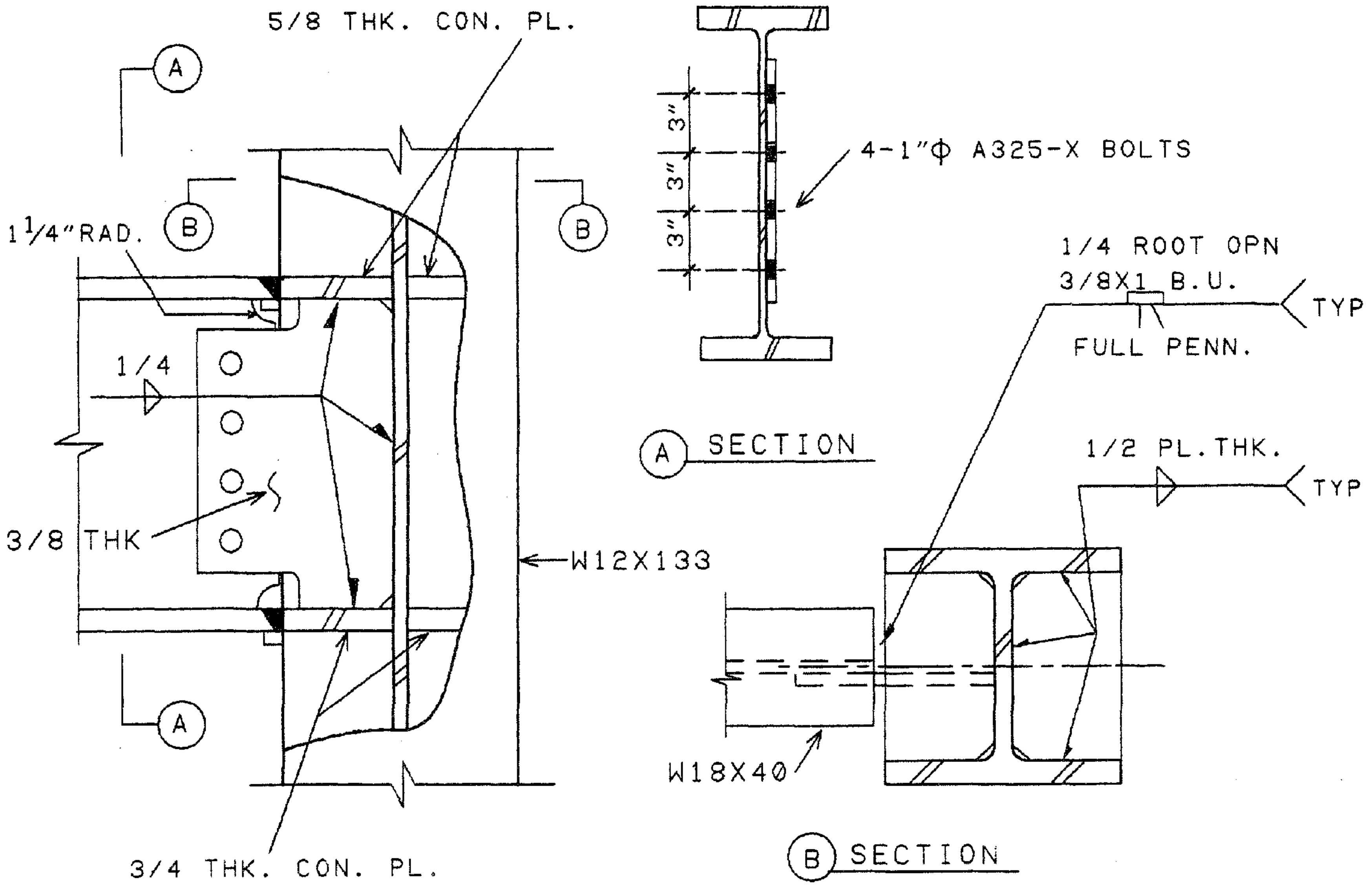



Fig. 2.4 Connection Detail (Specimen 2)

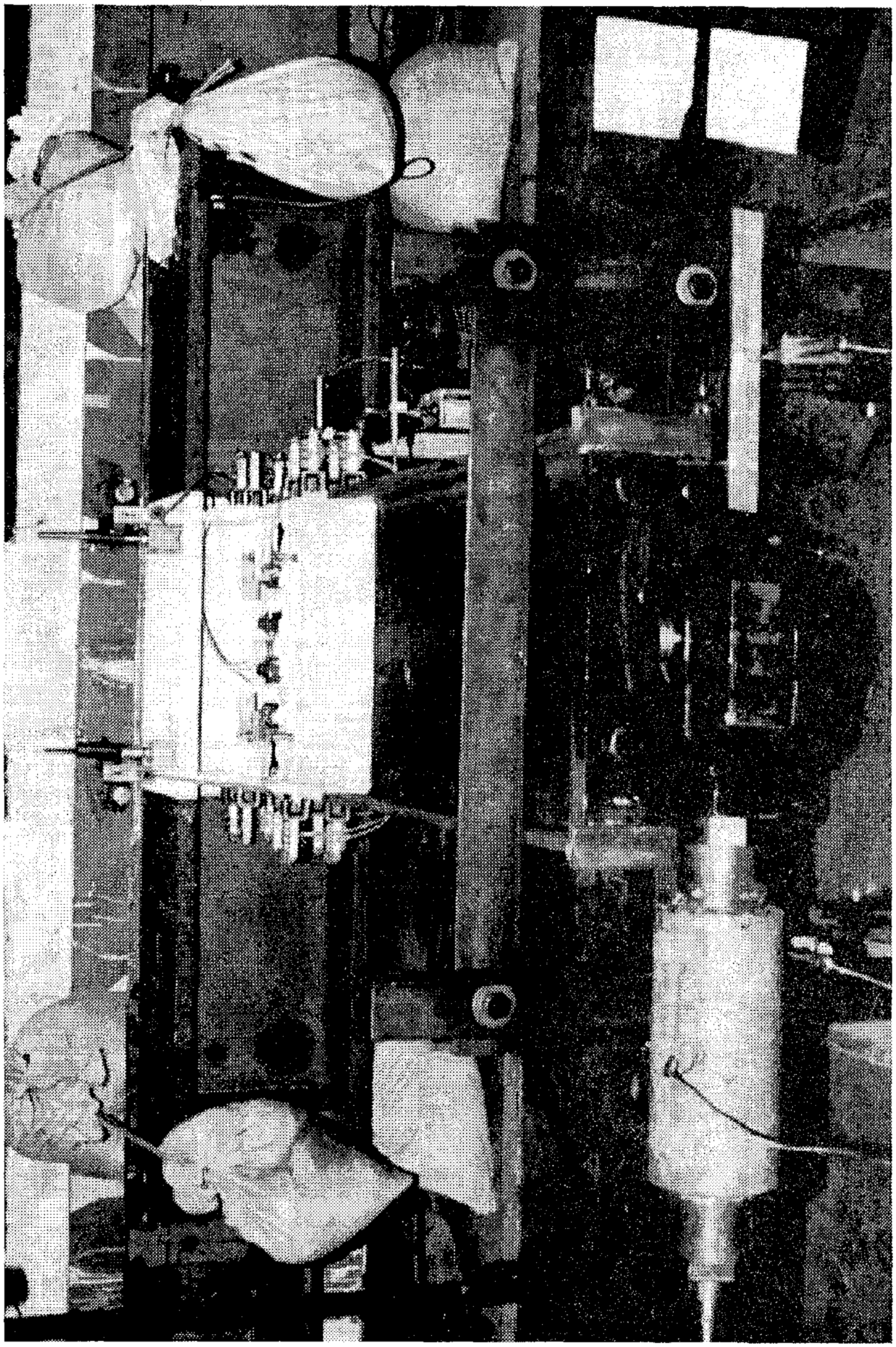


Fig. 2.5 General View of Specimen

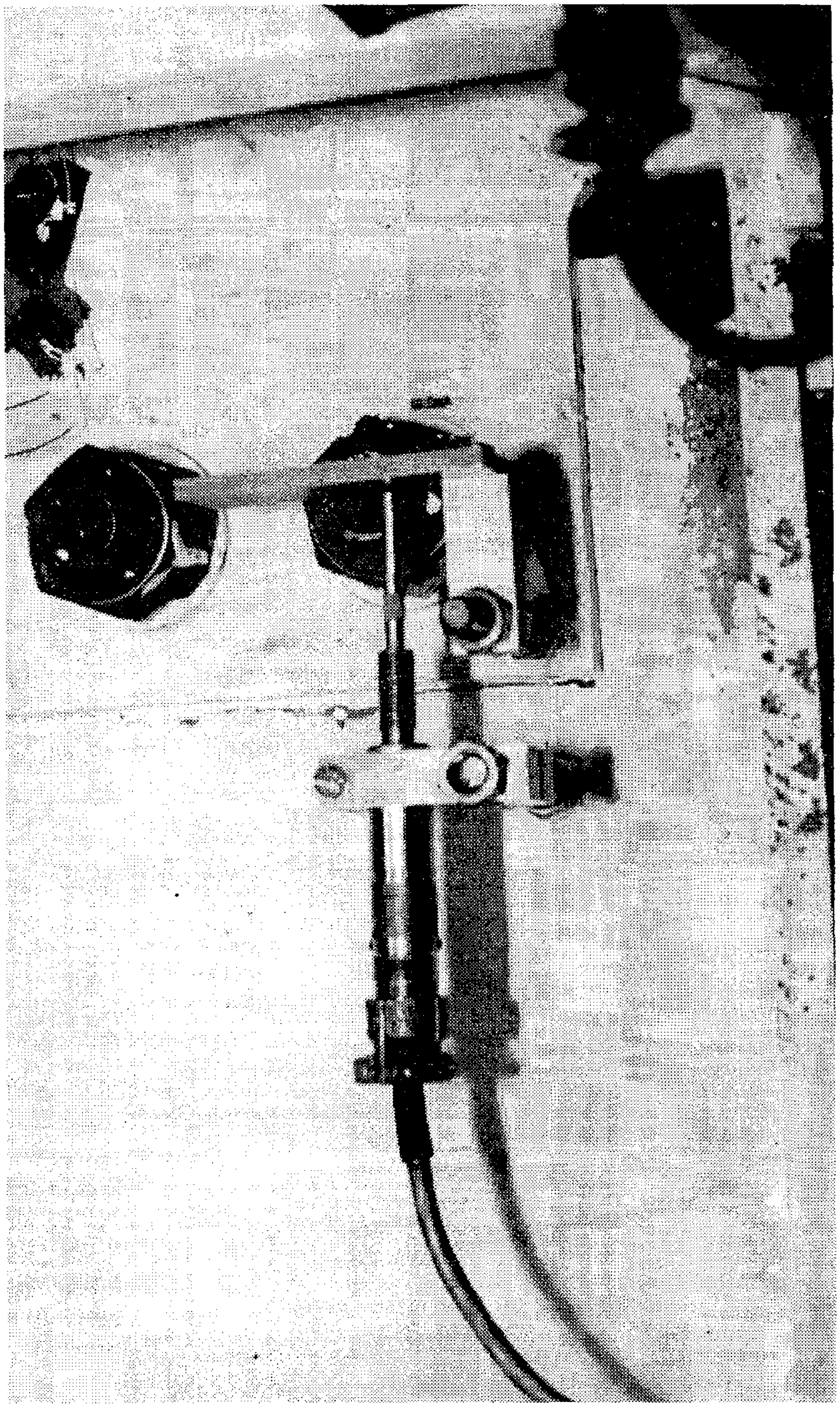


Fig. 2.6 Typical Slip Gage

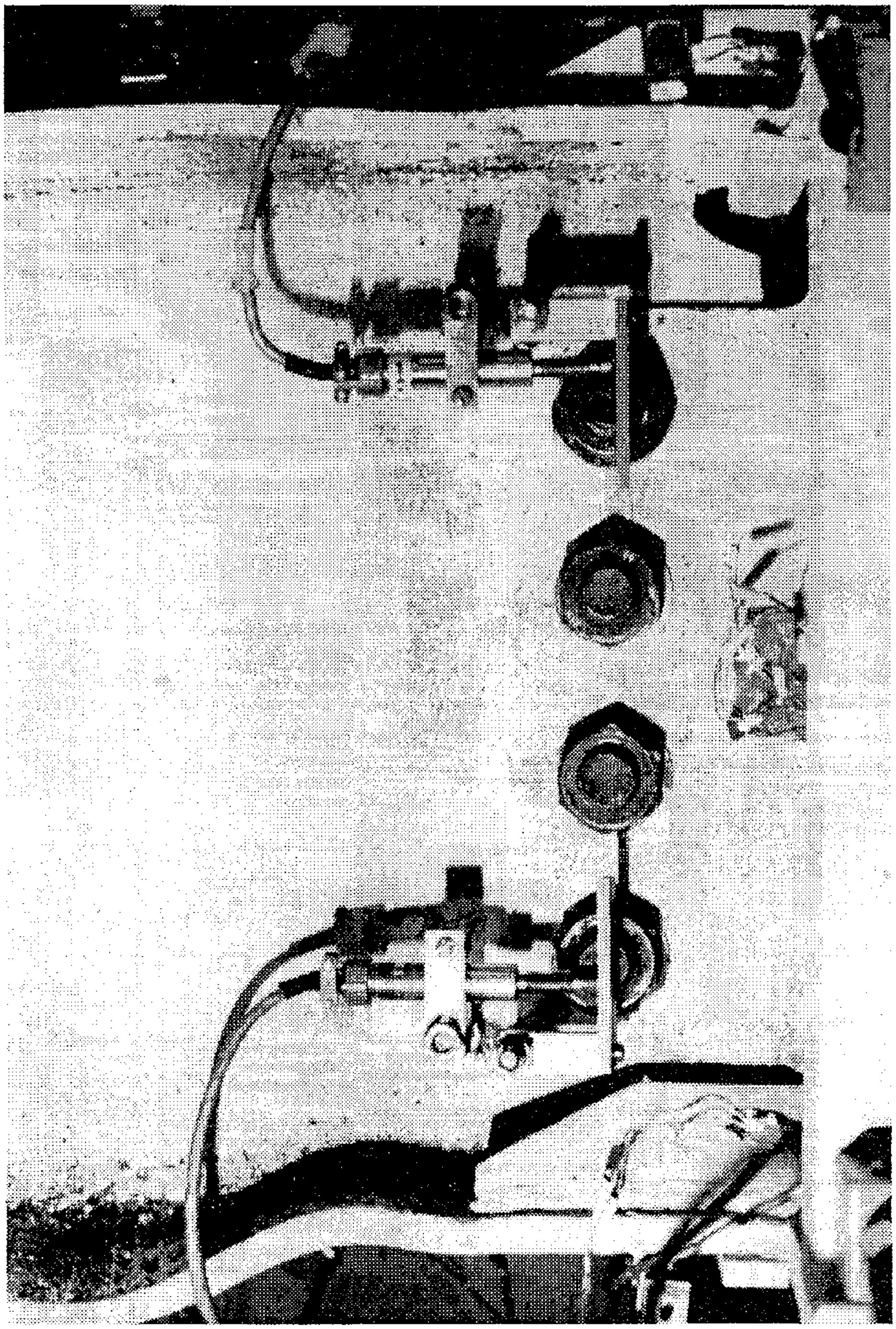


Fig. 2.7 Typical Set-Up for Rotation Measurement

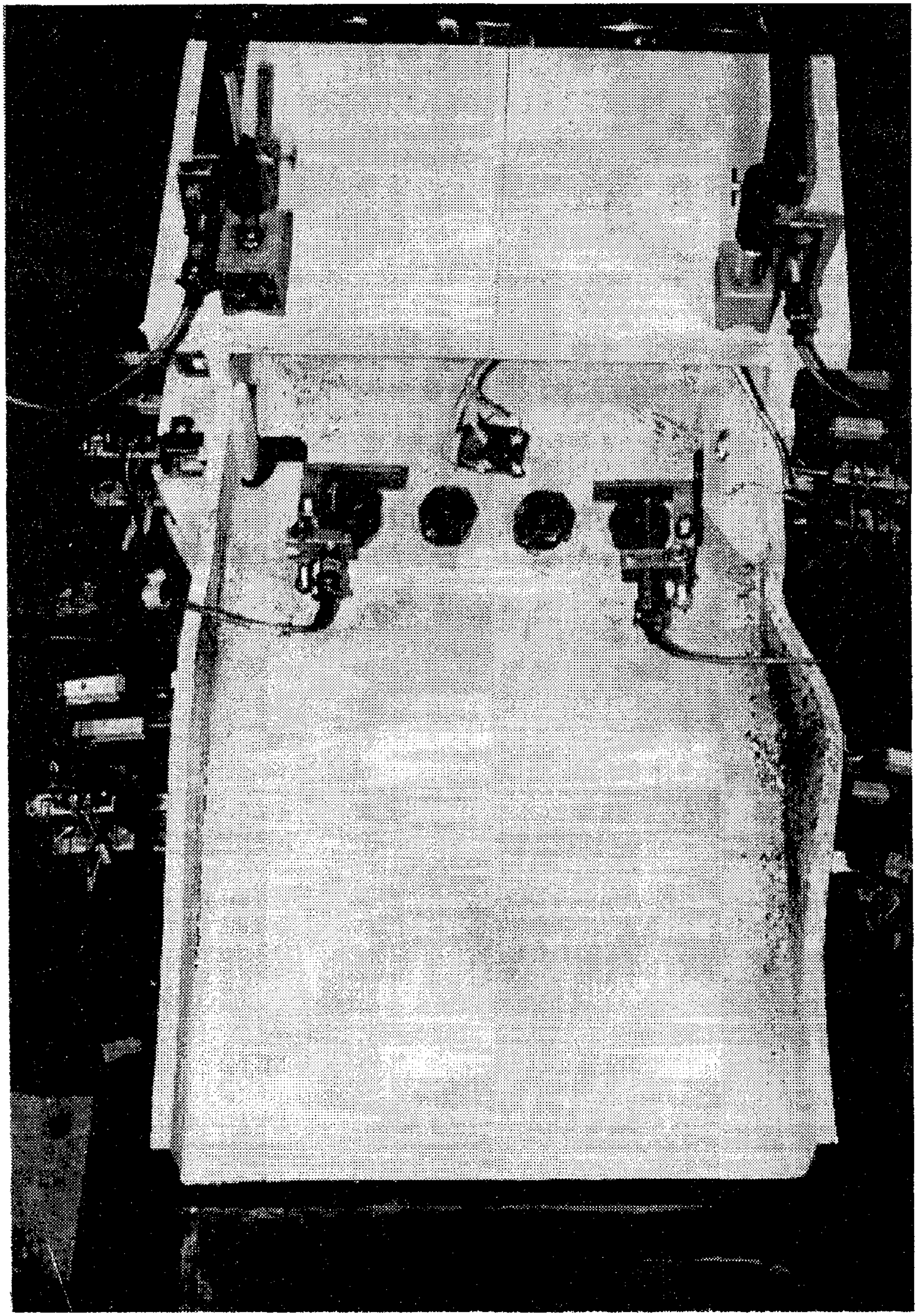


Fig. 2.8 Instrumentation at Joint

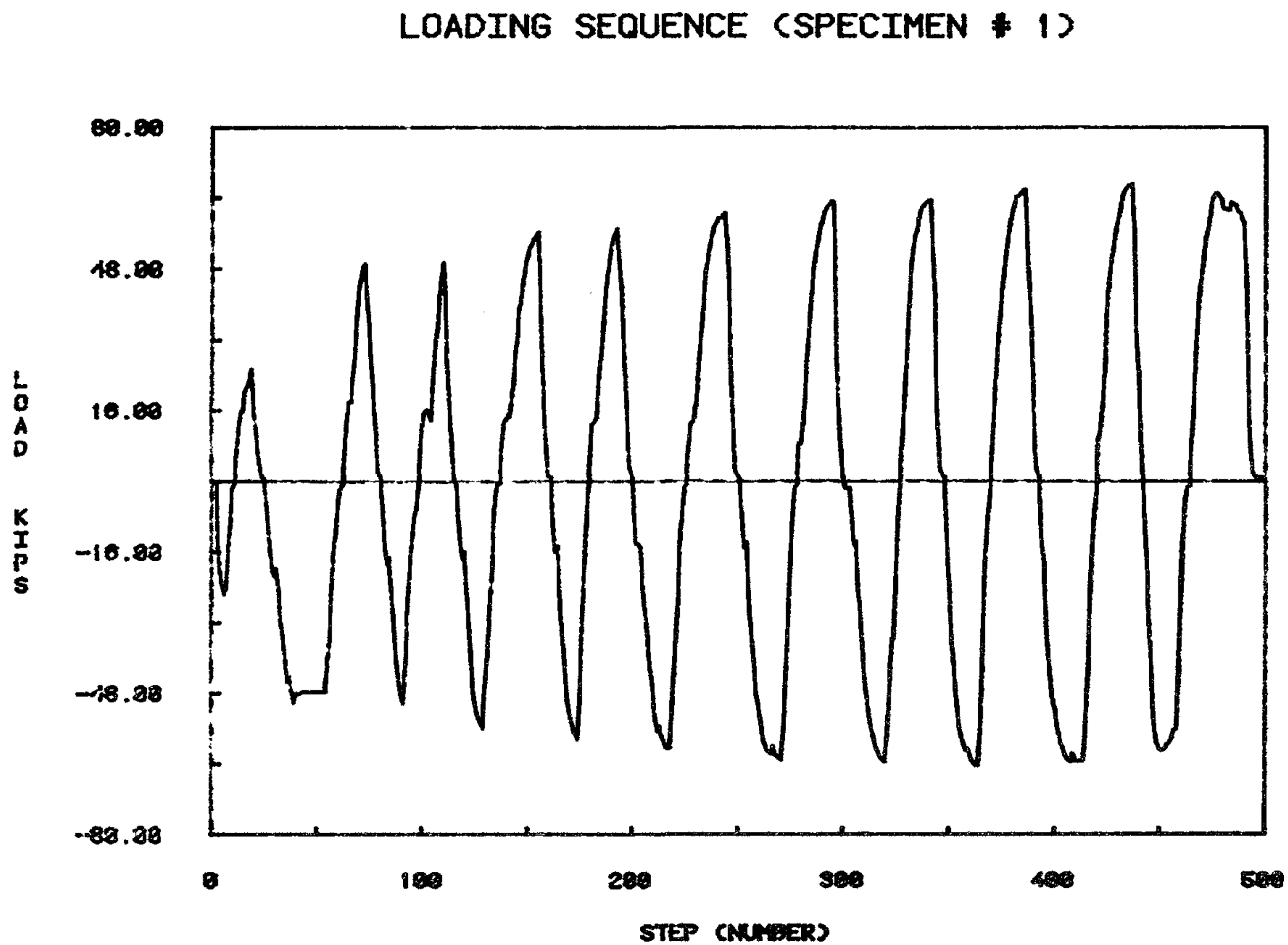



Fig. 2.9 Loading Sequence (Specimen 1)

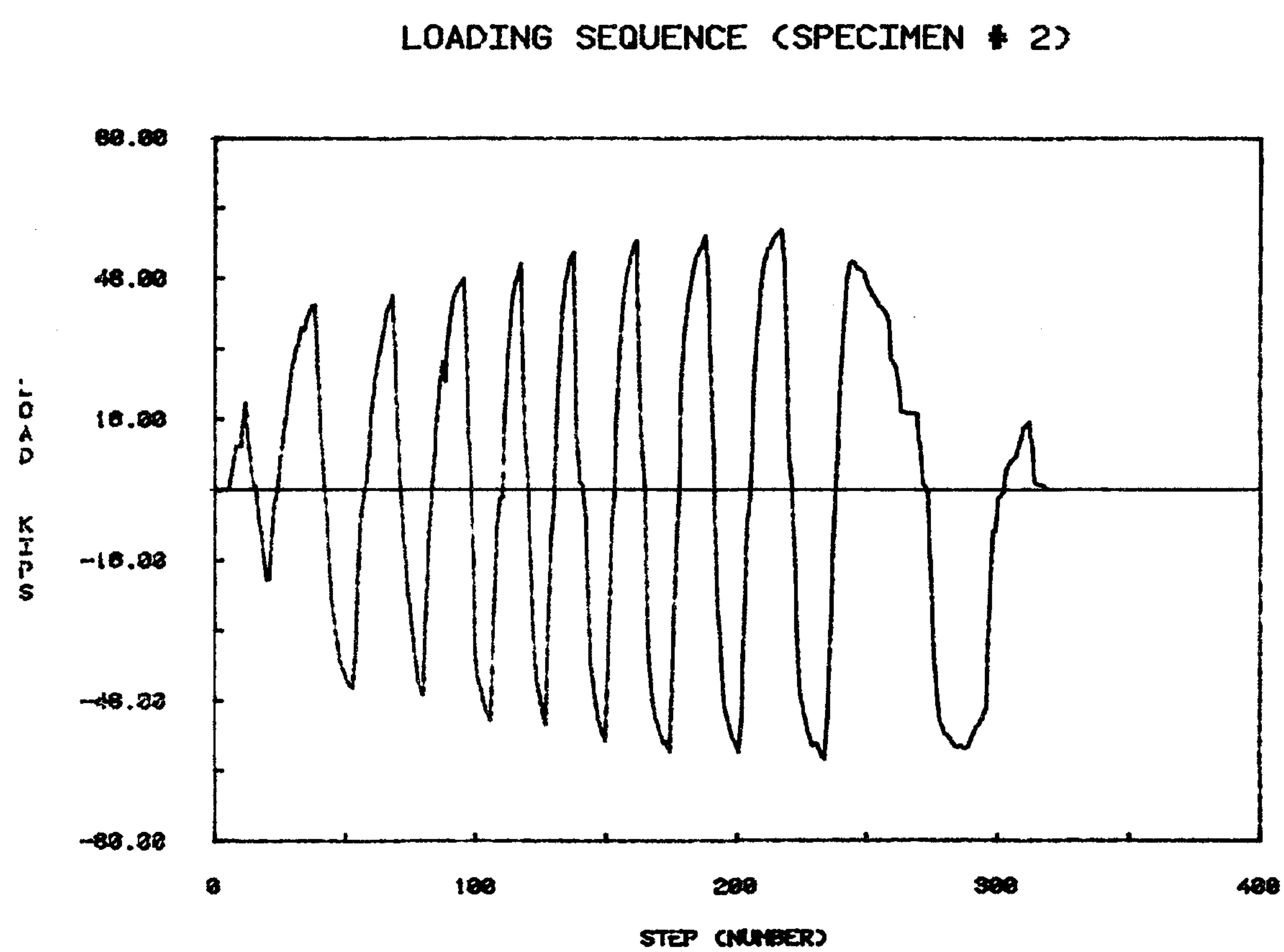
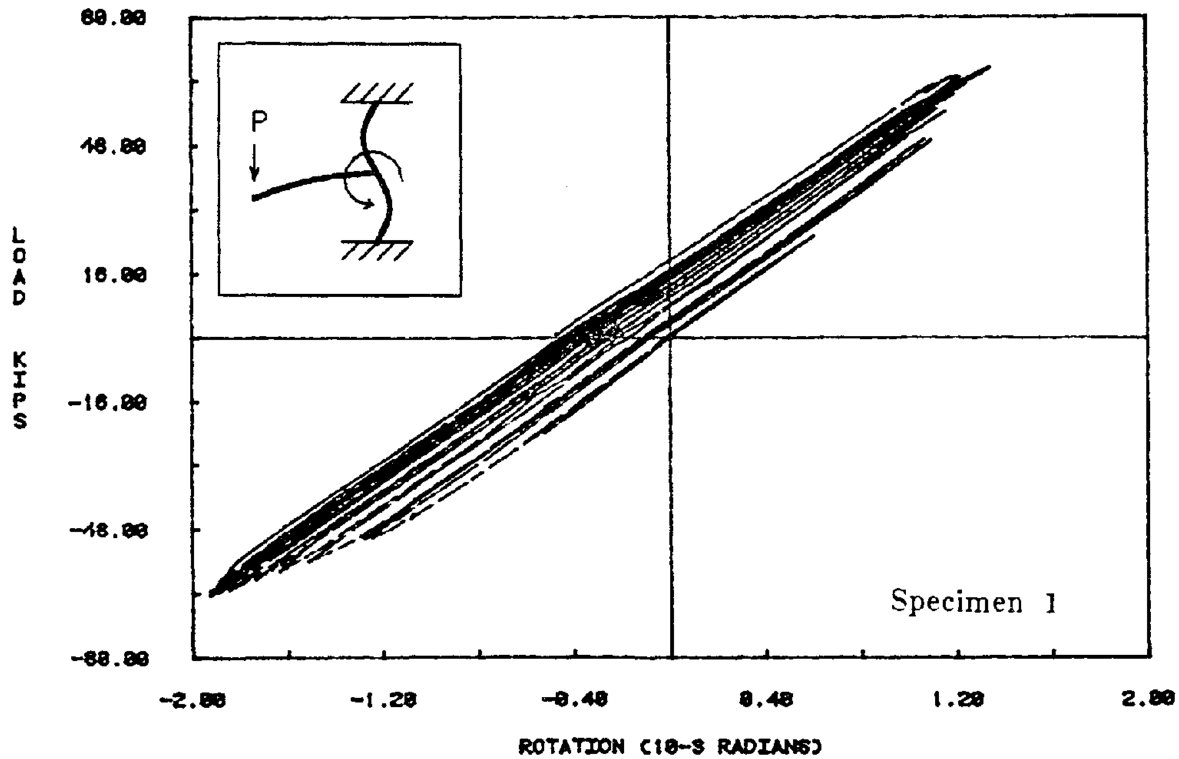



Fig. 2.10 Loading Sequence (Specimen 2)



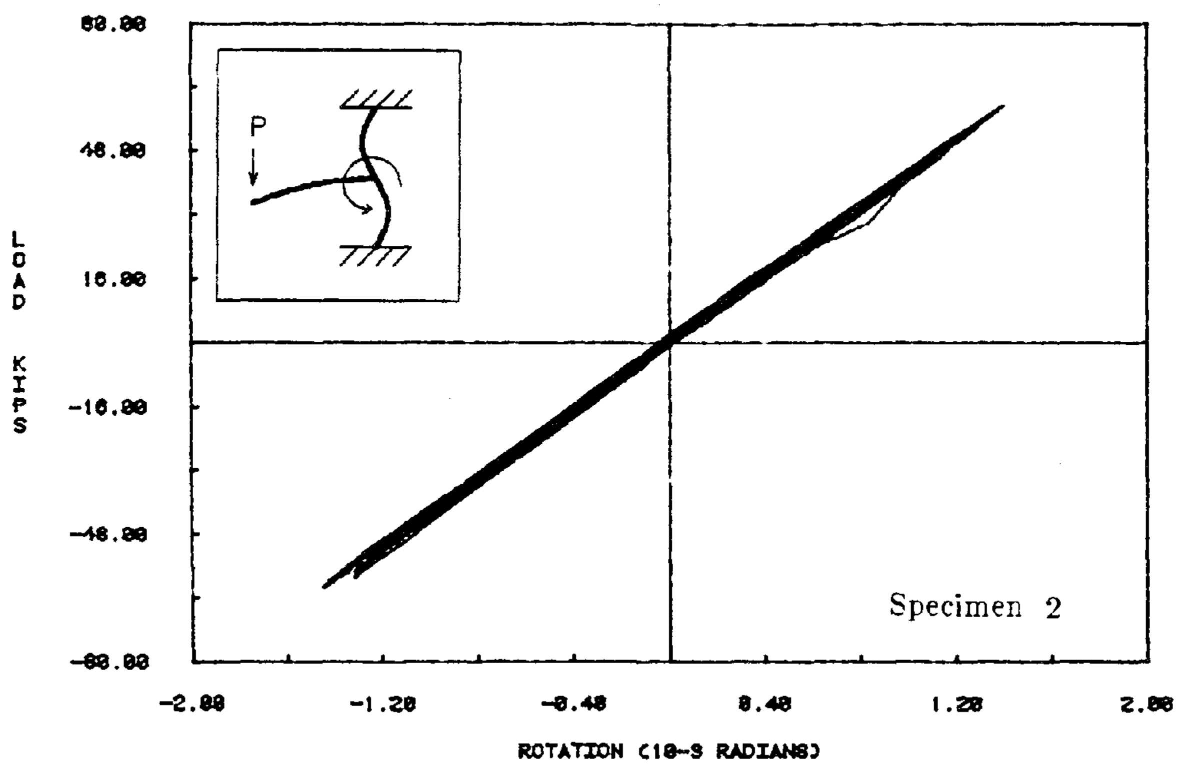




Fig. 3.1 Column Rotation at Beam-Column Joint for Specimen 1 and 2

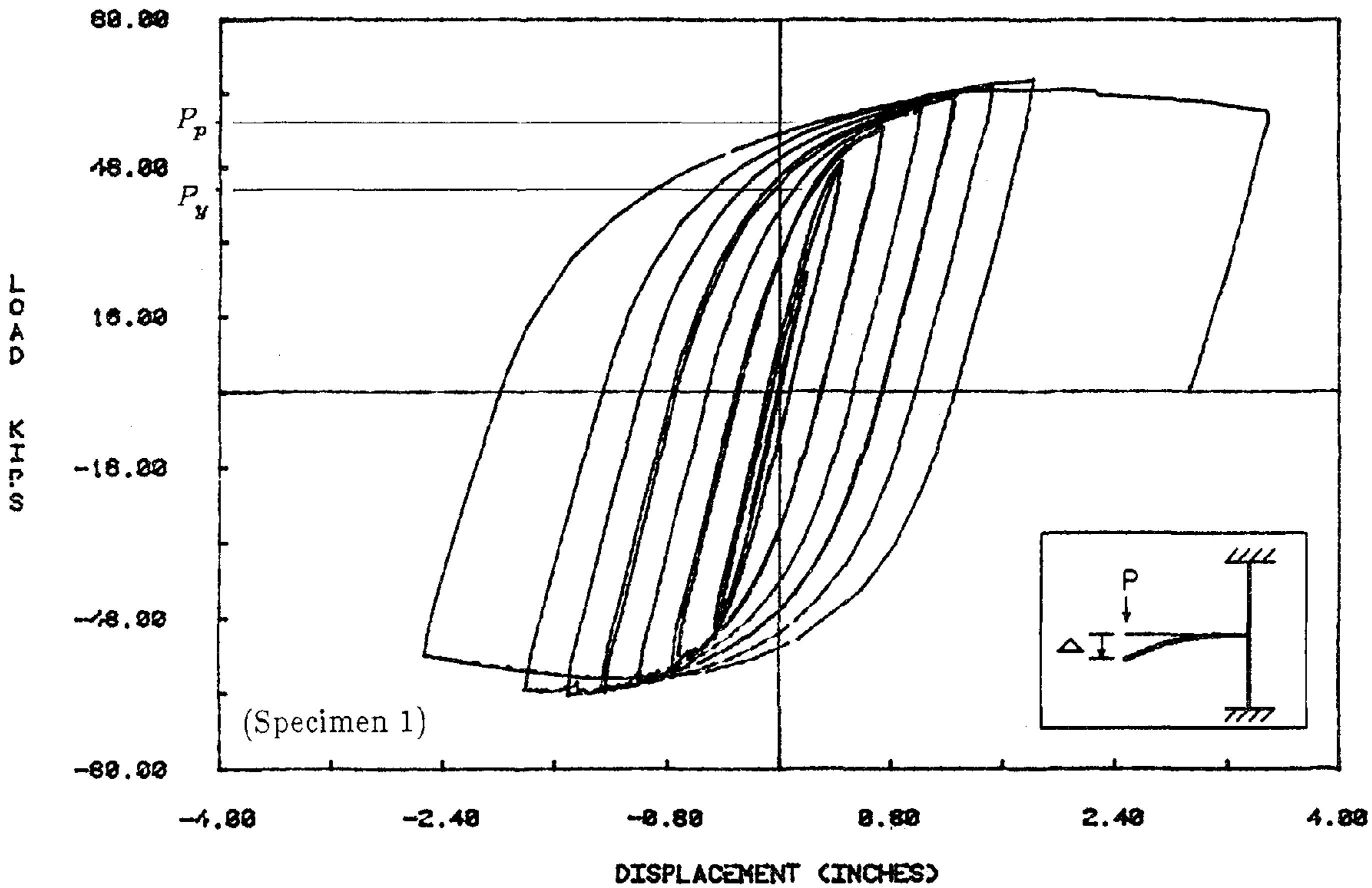



Fig. 3.2 Cantilever Beam Load vs. Beam End Displacement (Specimen 1)

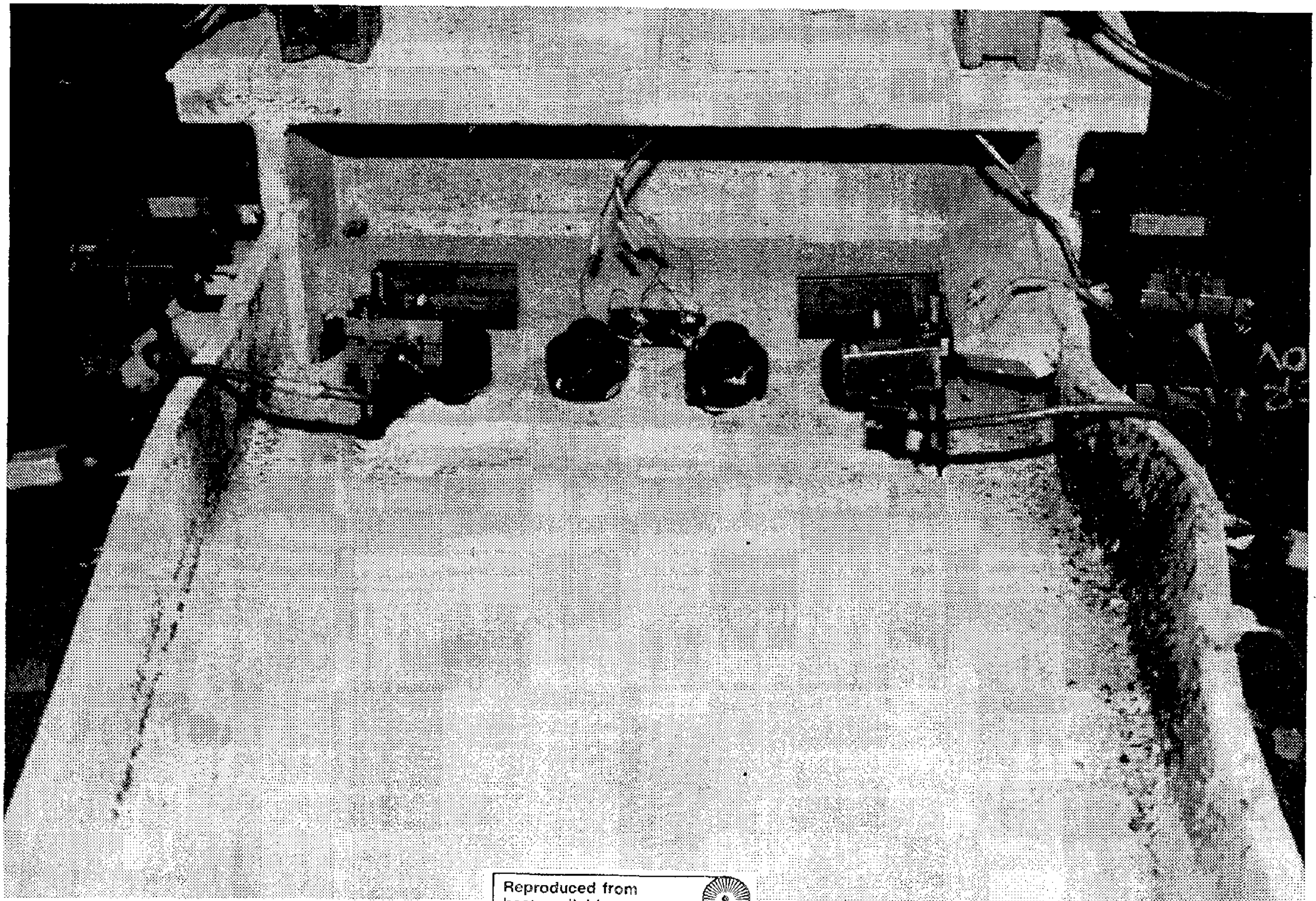




Fig. 3.3 Local Buckling of Beam Bottom Flange (Specimen 1)

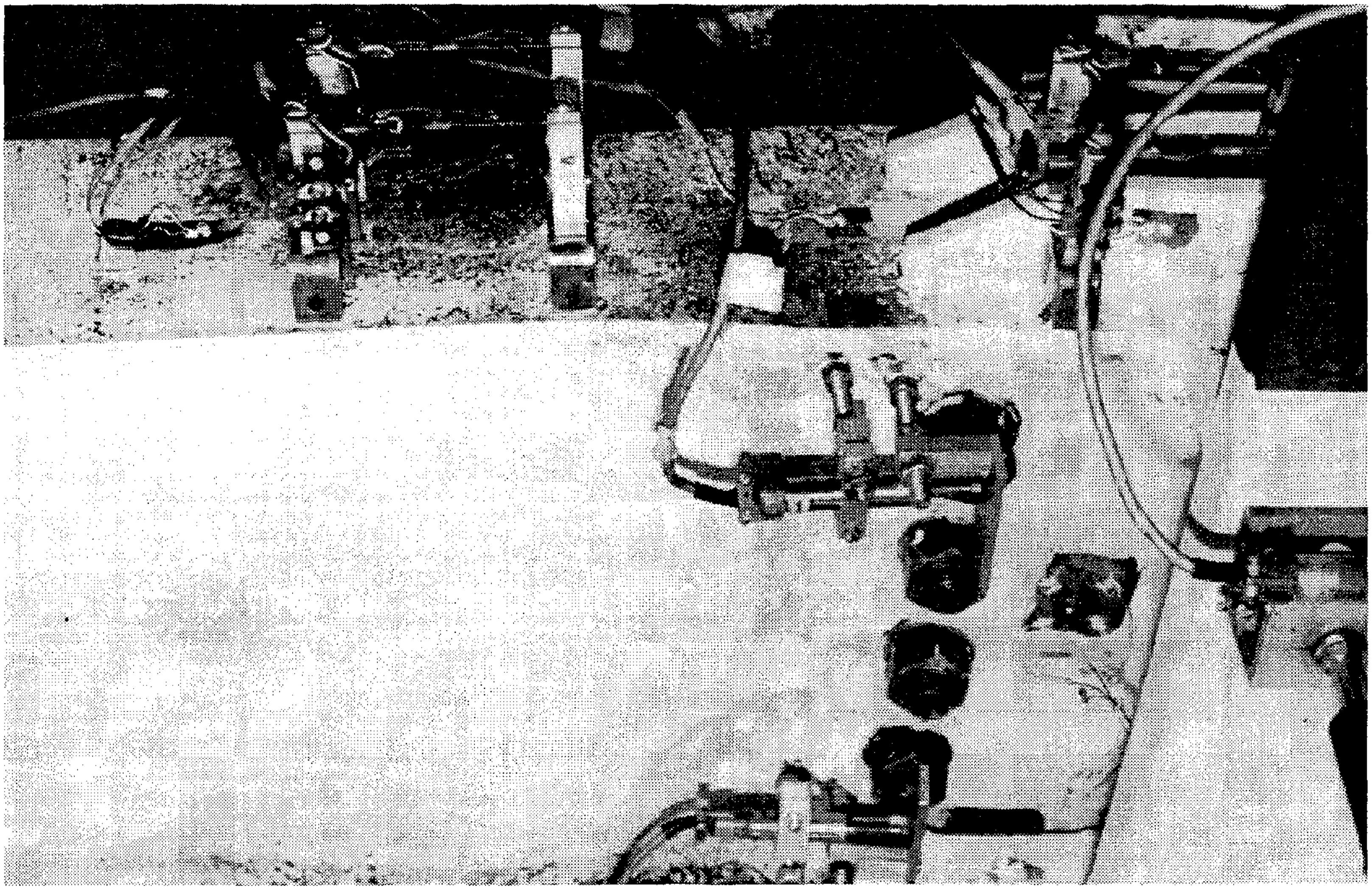


Fig. 3.4 Local Buckling of Beam Top Flange (Specimen 1)

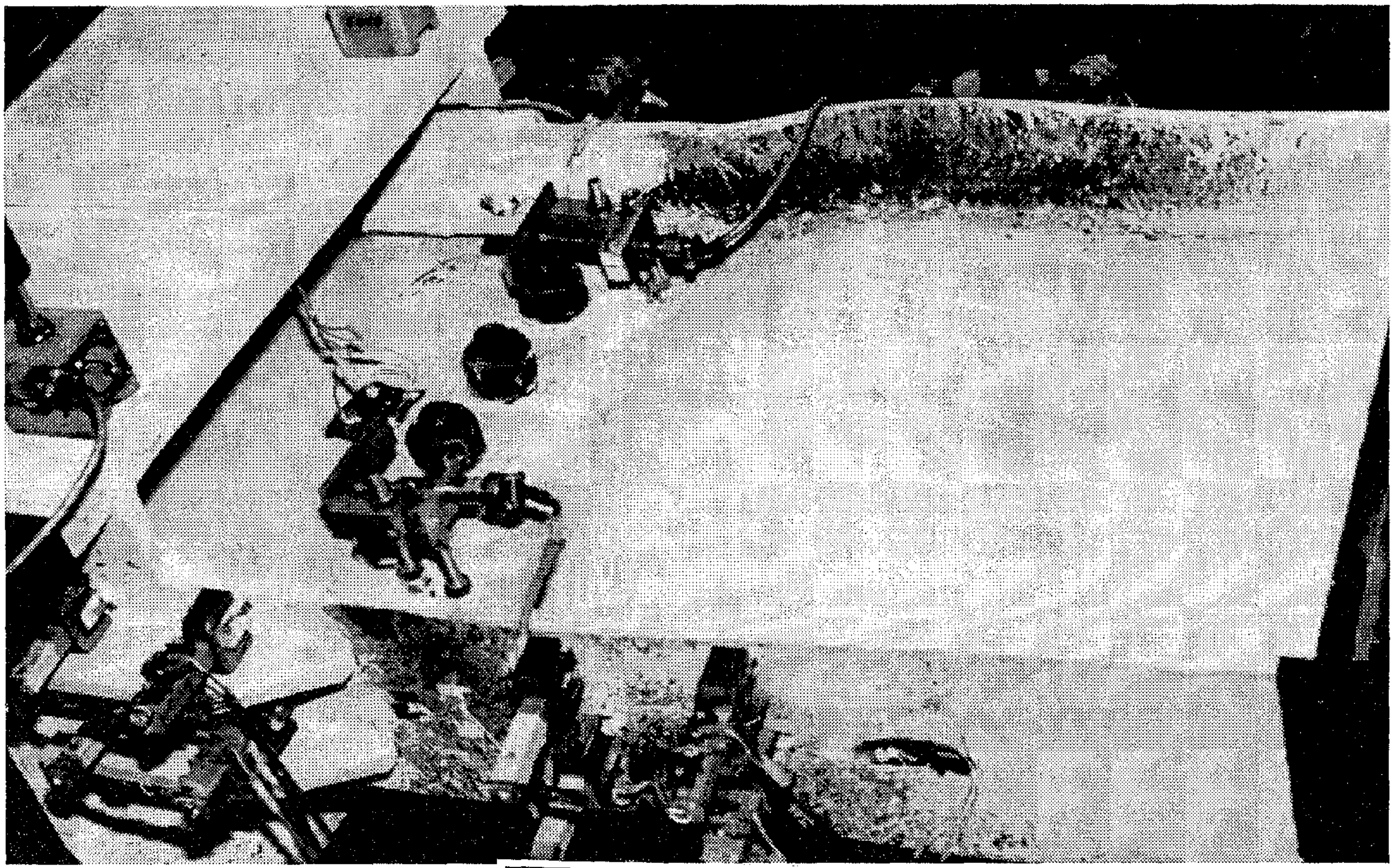




Fig. 3.5 Local Buckling of Beam Web (Specimen 1)

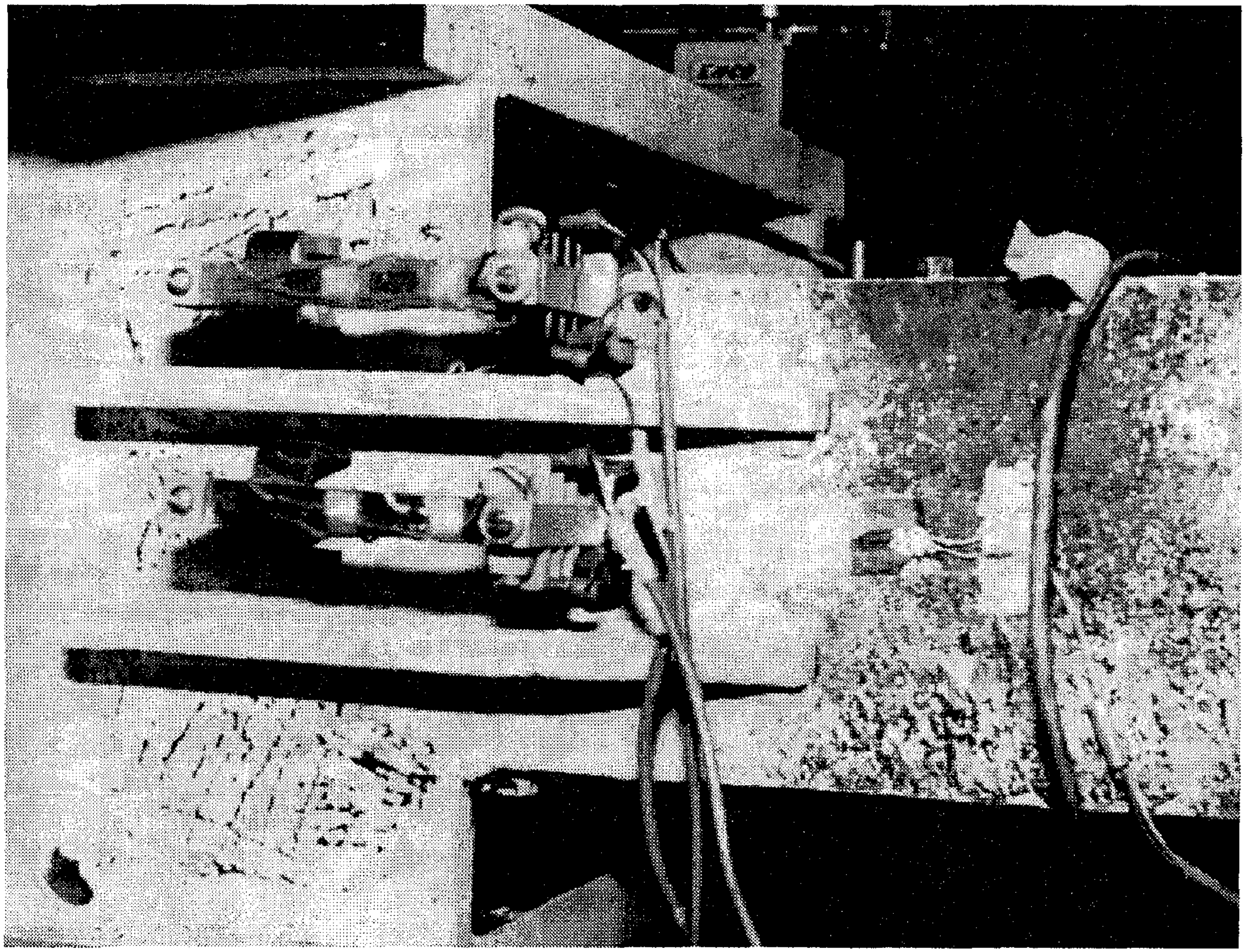


Fig. 3.6 Yield Pattern at Top Continuity Plate (Specimen 1)

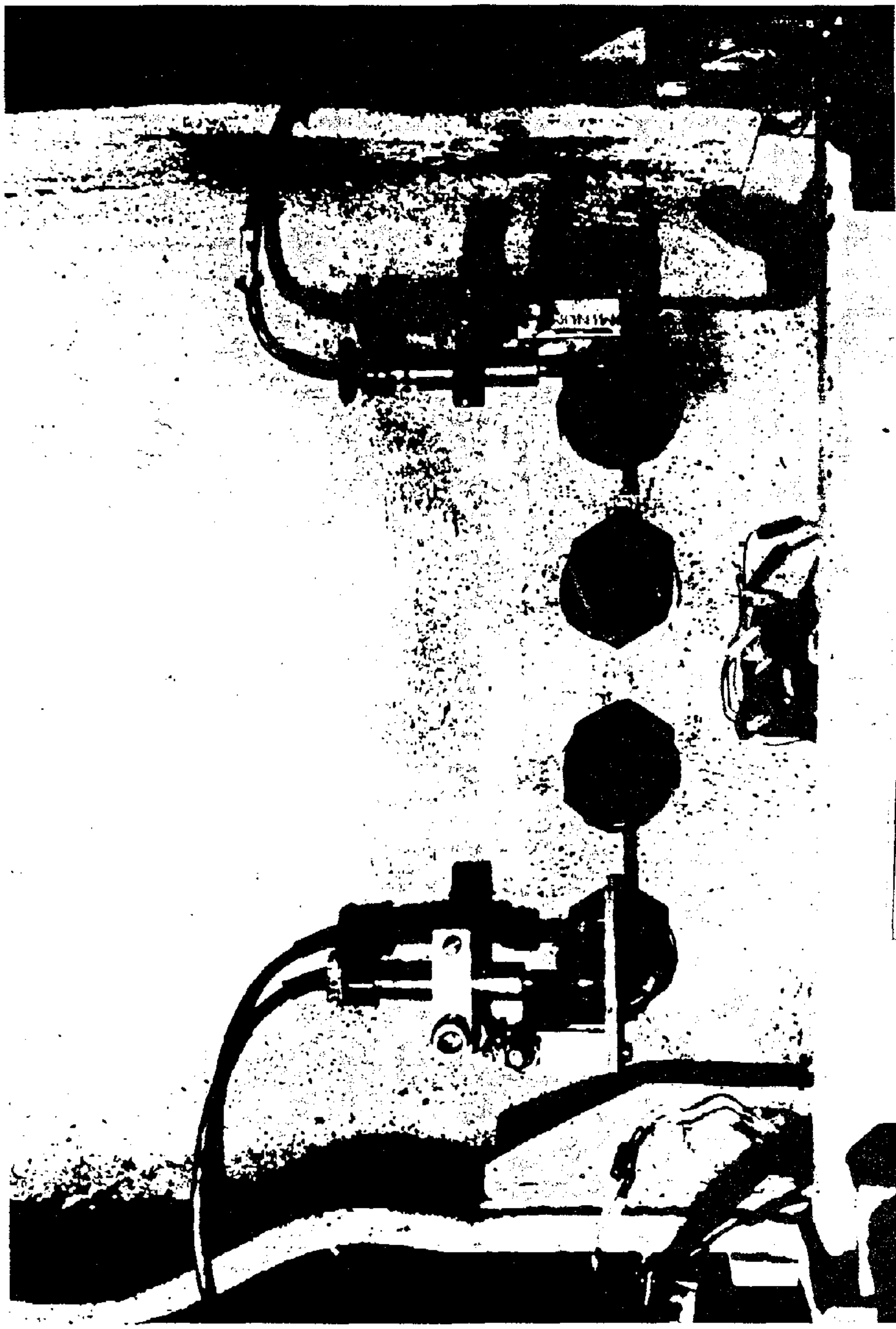




Fig. 3.7 Yield Pattern at Beam Web (Specimen 1)

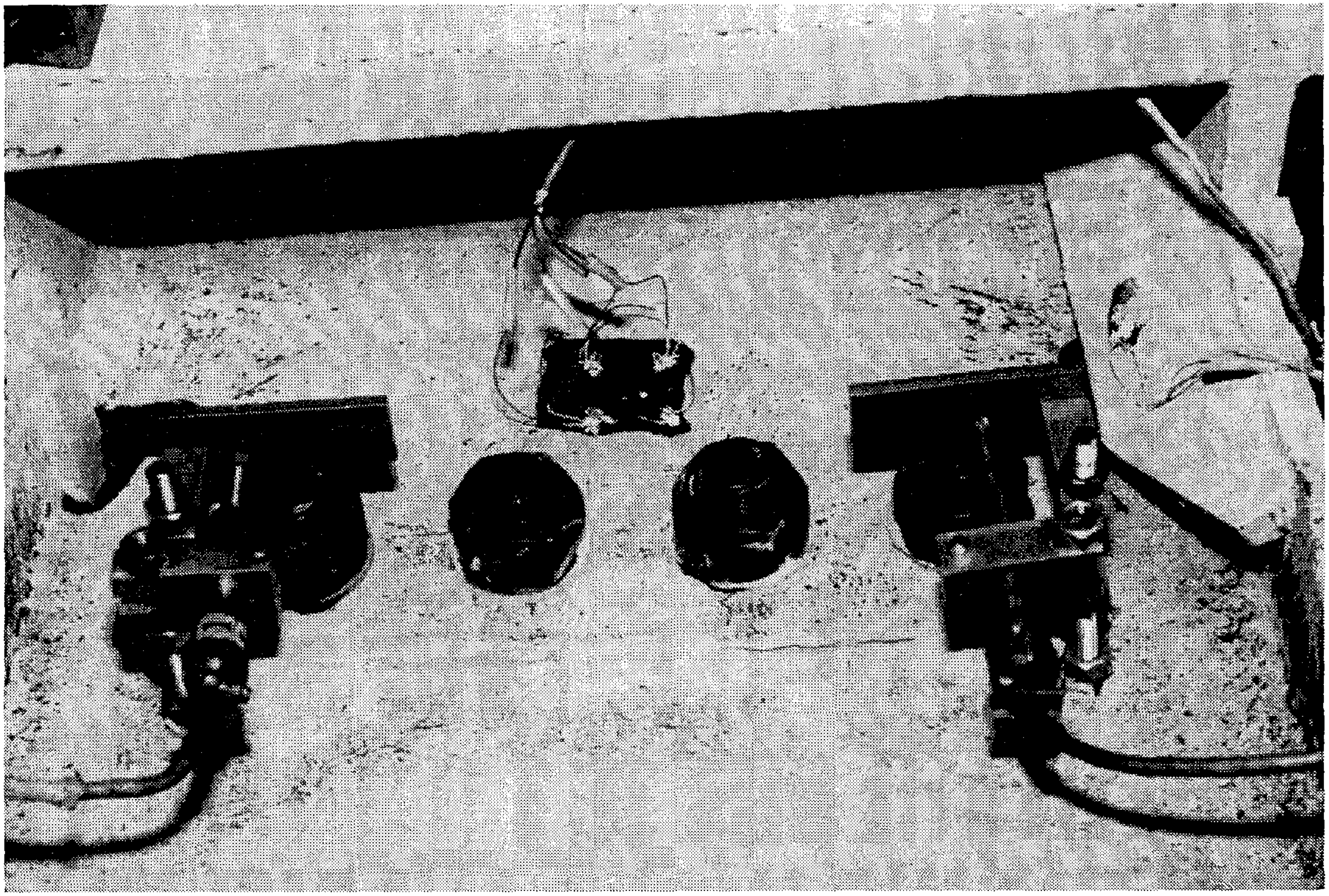


Fig. 3.8 Yield Pattern at Shear Tab (Specimen 1)

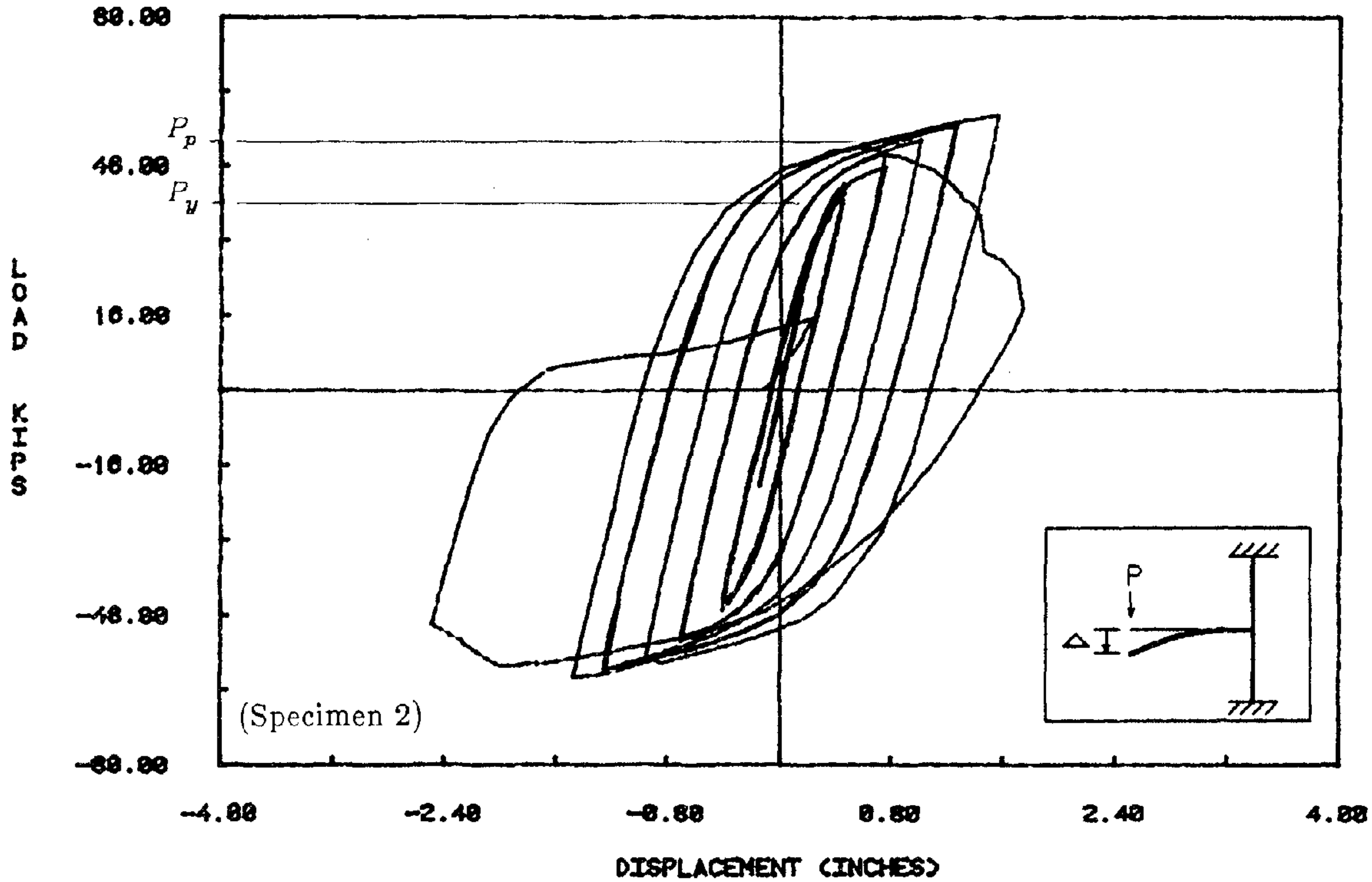



Fig. 3.9 Cantilever Beam Load vs. Beam End Displacement (Specimen 2)

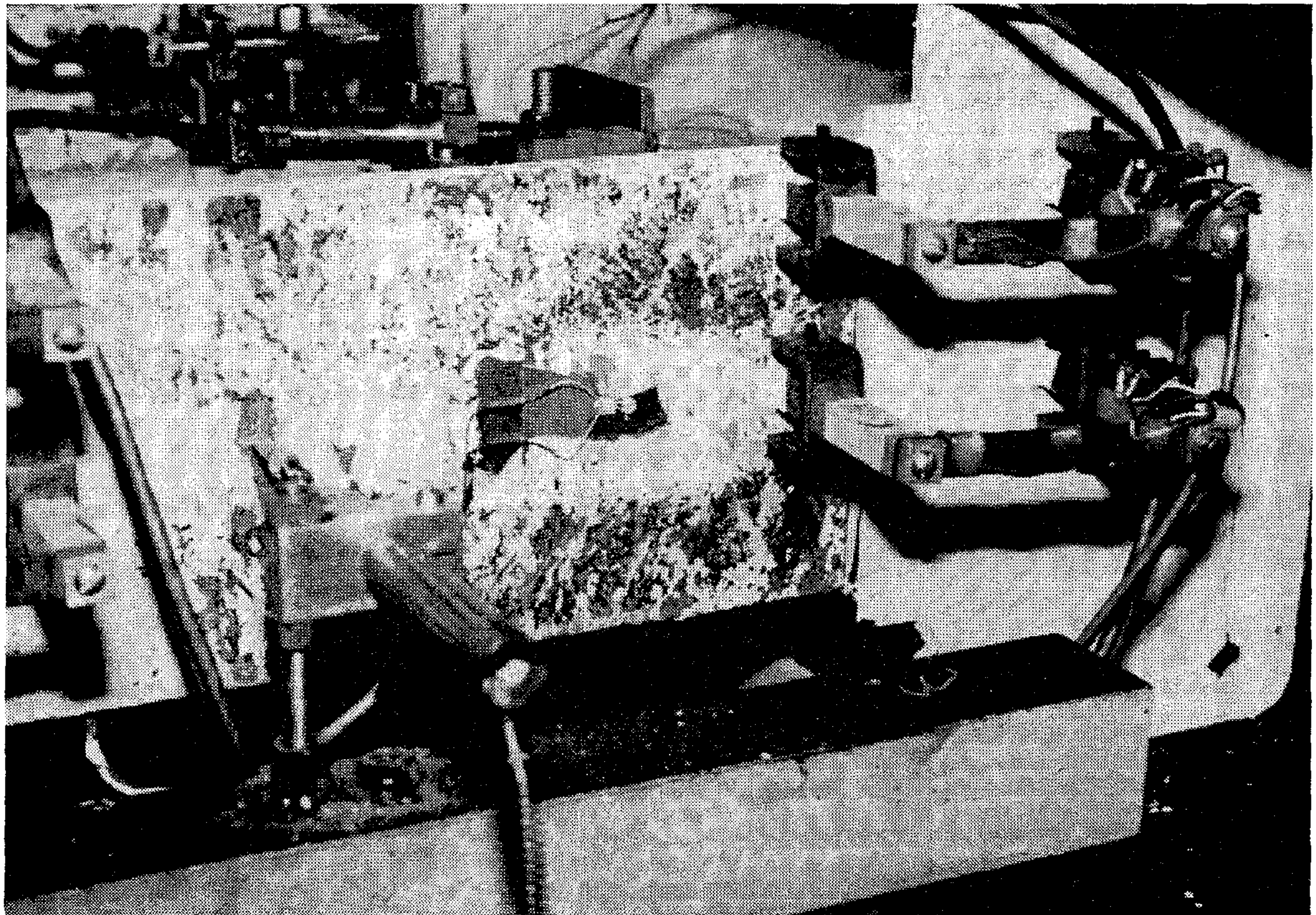




Fig. 3.10 Flaking of Whitewash on Bottom Flange (Specimen 2)

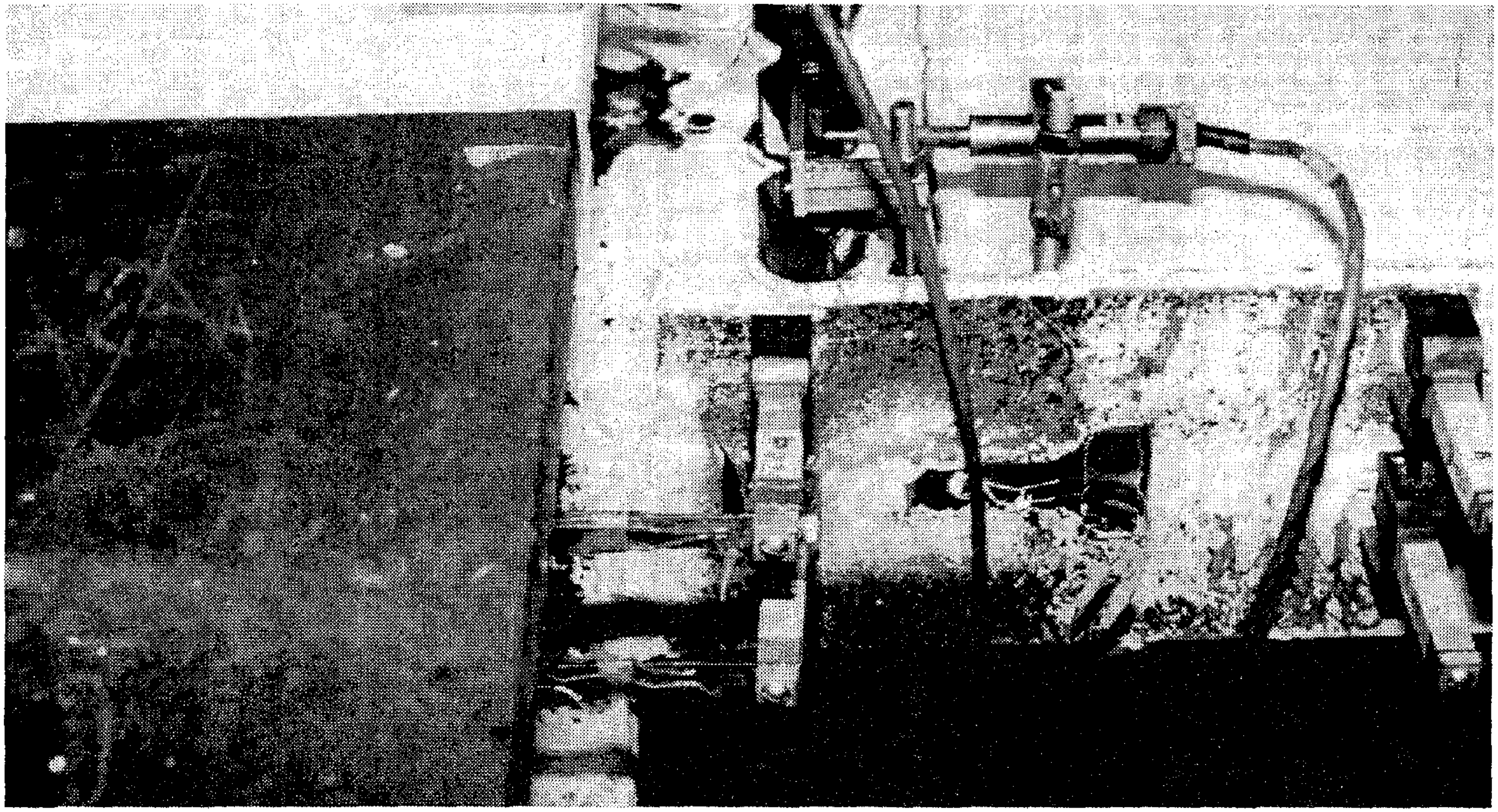


Fig. 3.11 Crack Initiation on Beam Top Flange (Specimen 2)

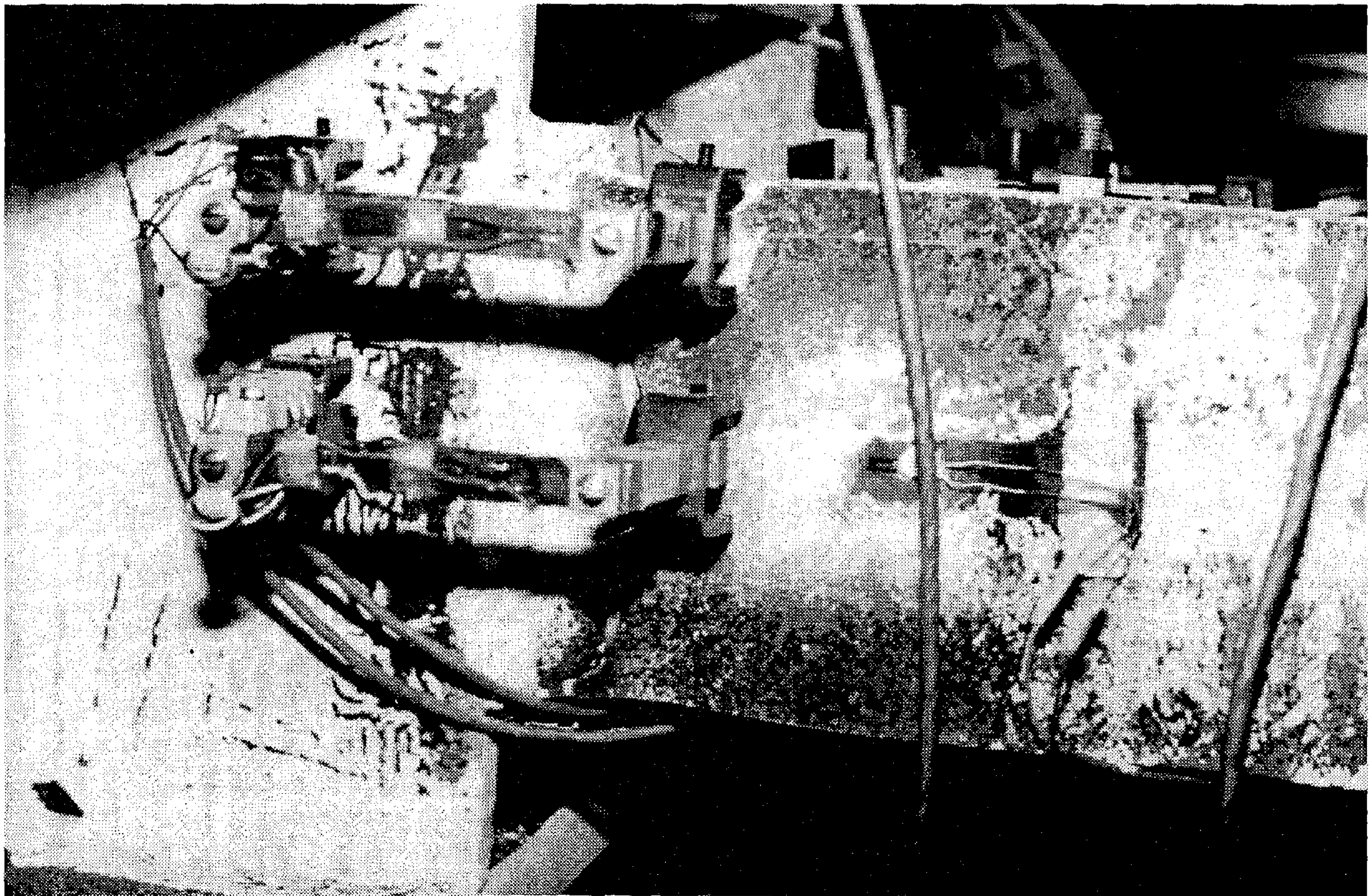




Fig. 3.12 Crack Propagation on Beam Top Flange (Specimen 2)

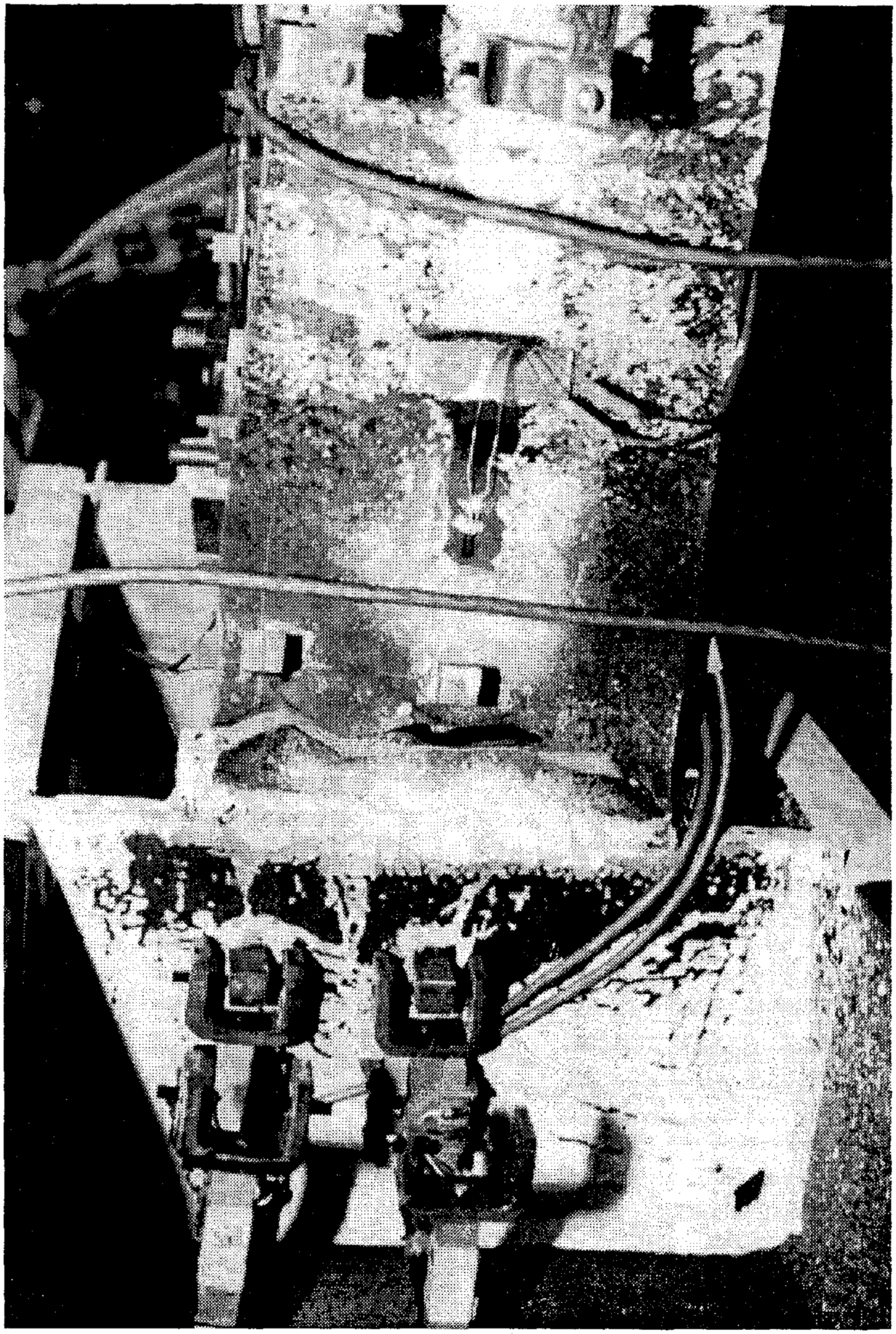


Fig. 3.13 Specimen 2 after Failure

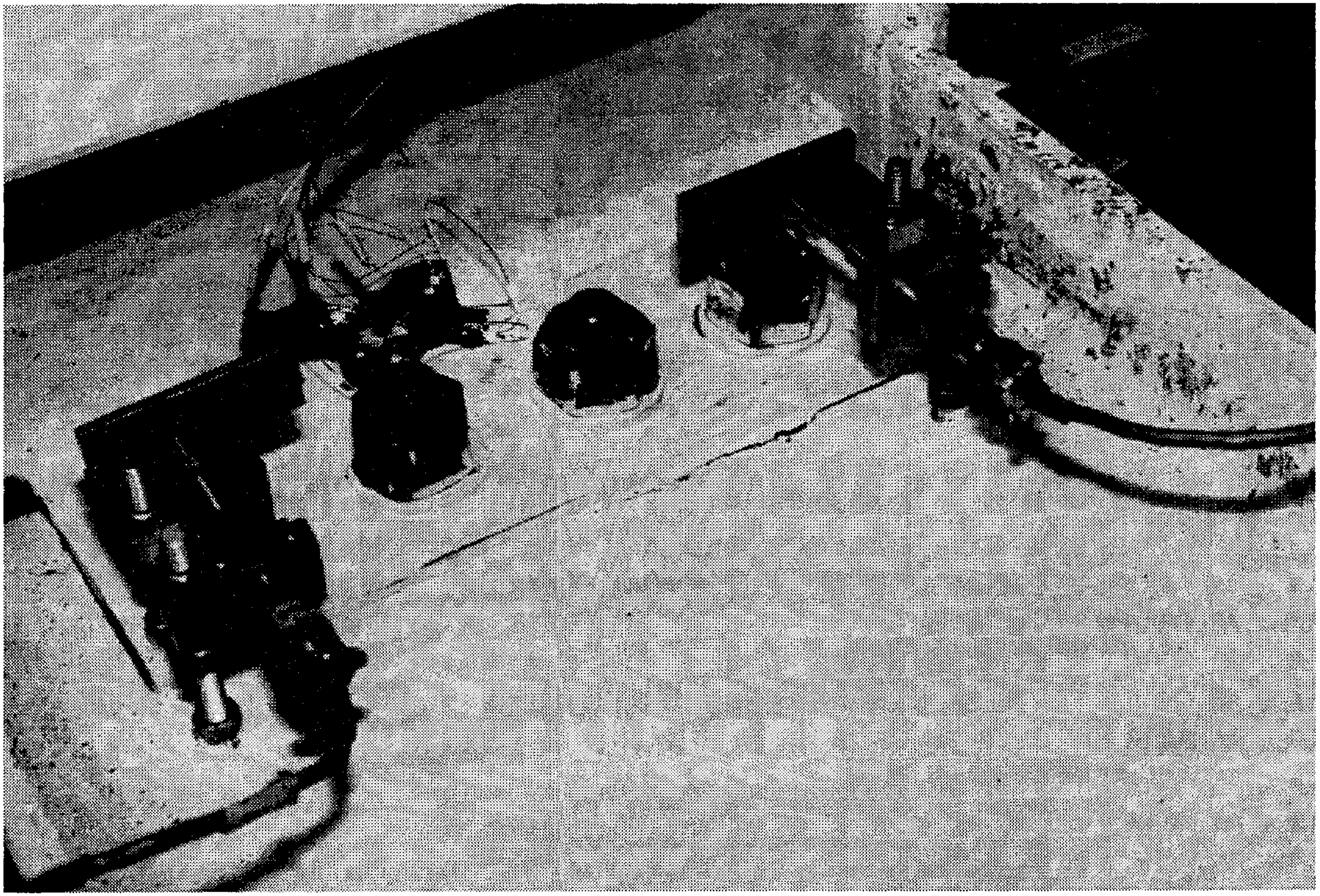


Fig. 3.14 Slippage between Beam Web and Shear Tab

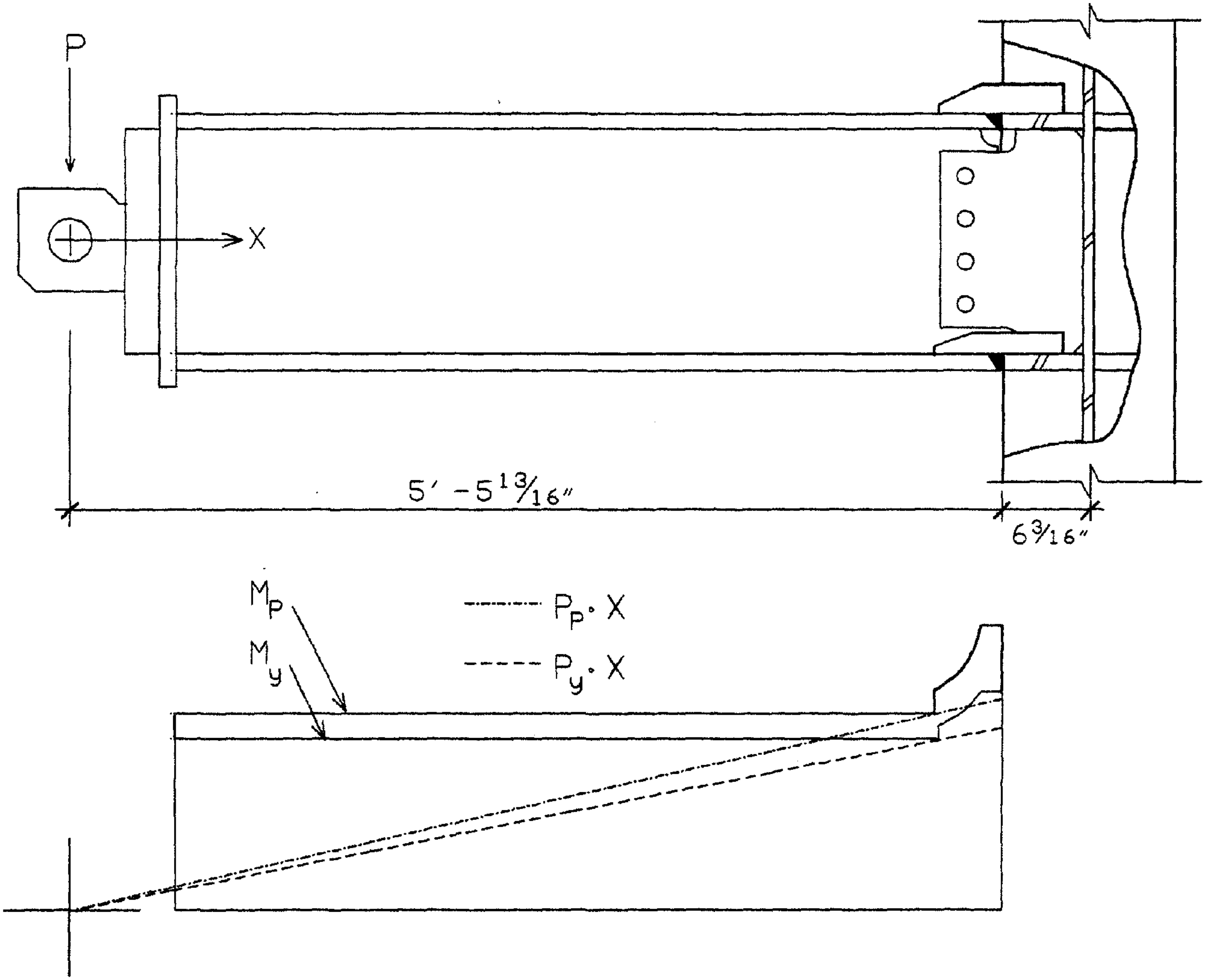



Fig. 4.1 Beam Moment Capacity and Applied Moment

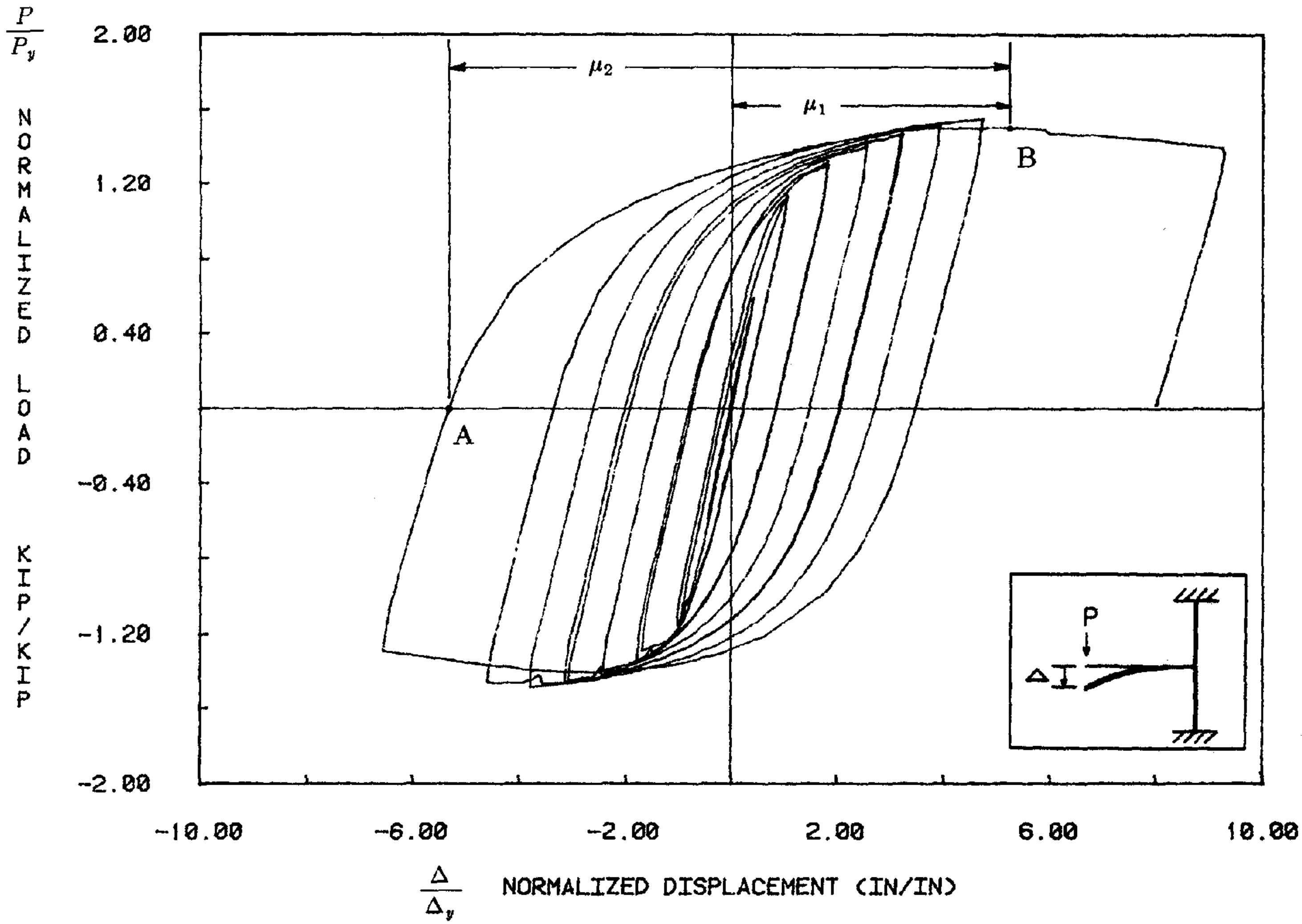



Fig. 4.2 Normalized Load vs. Beam Deflection Curve (Specimen 1)

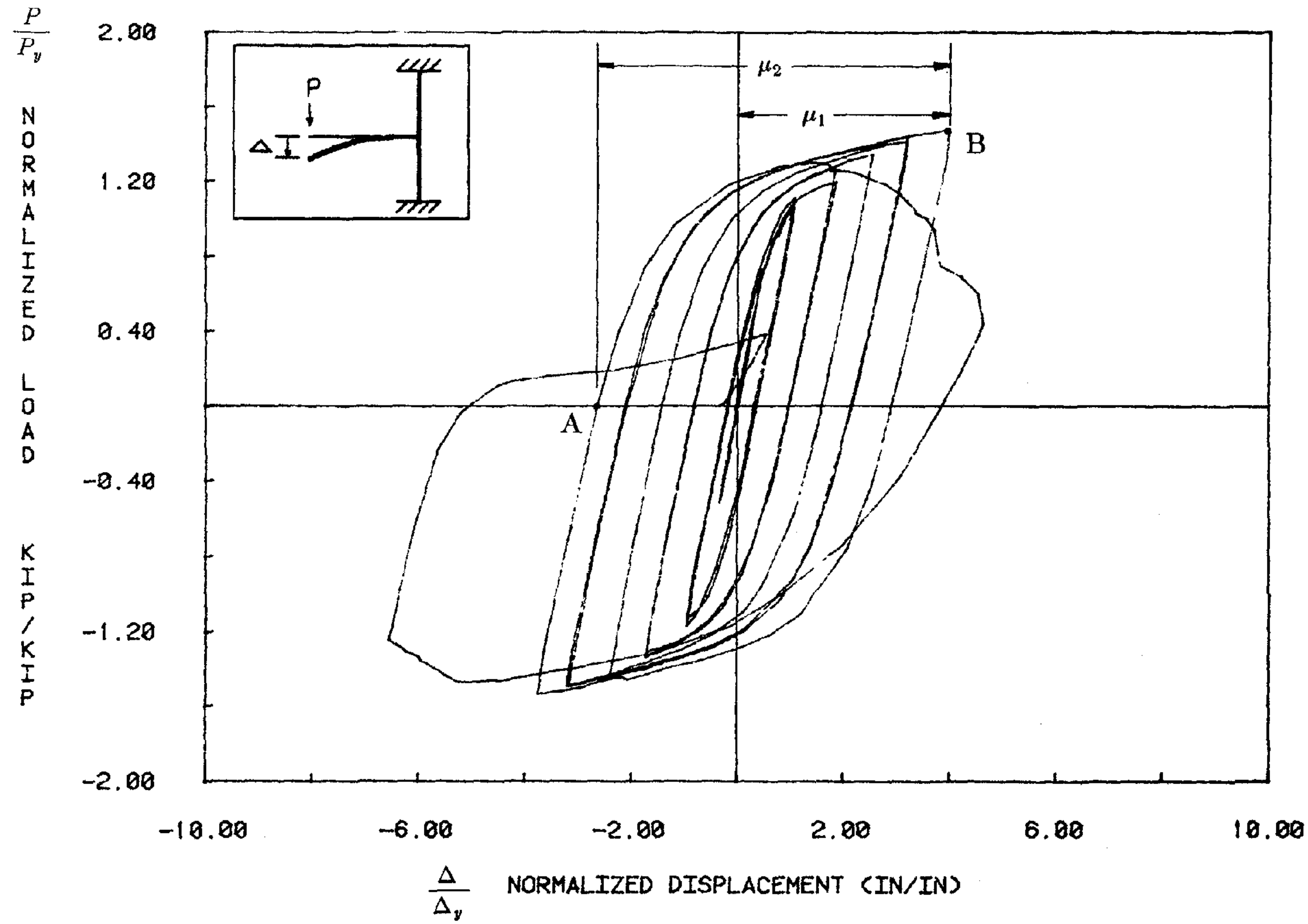



Fig. 4.3 Normalized Load vs. Beam Deflection Curve (Specimen 2)

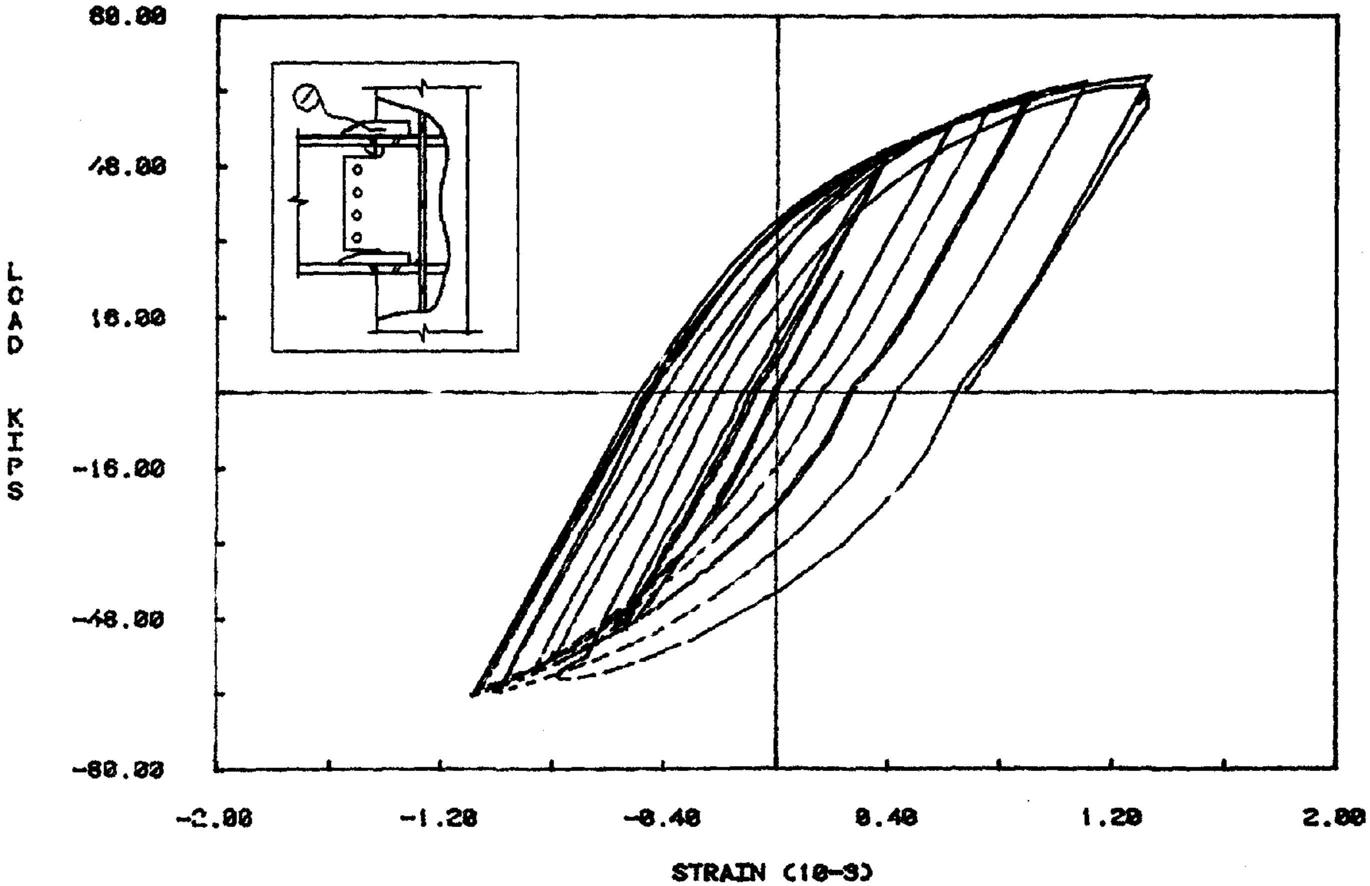



Fig. 4.4 Strain in Top Reinforcing Rib (Specimen 1)

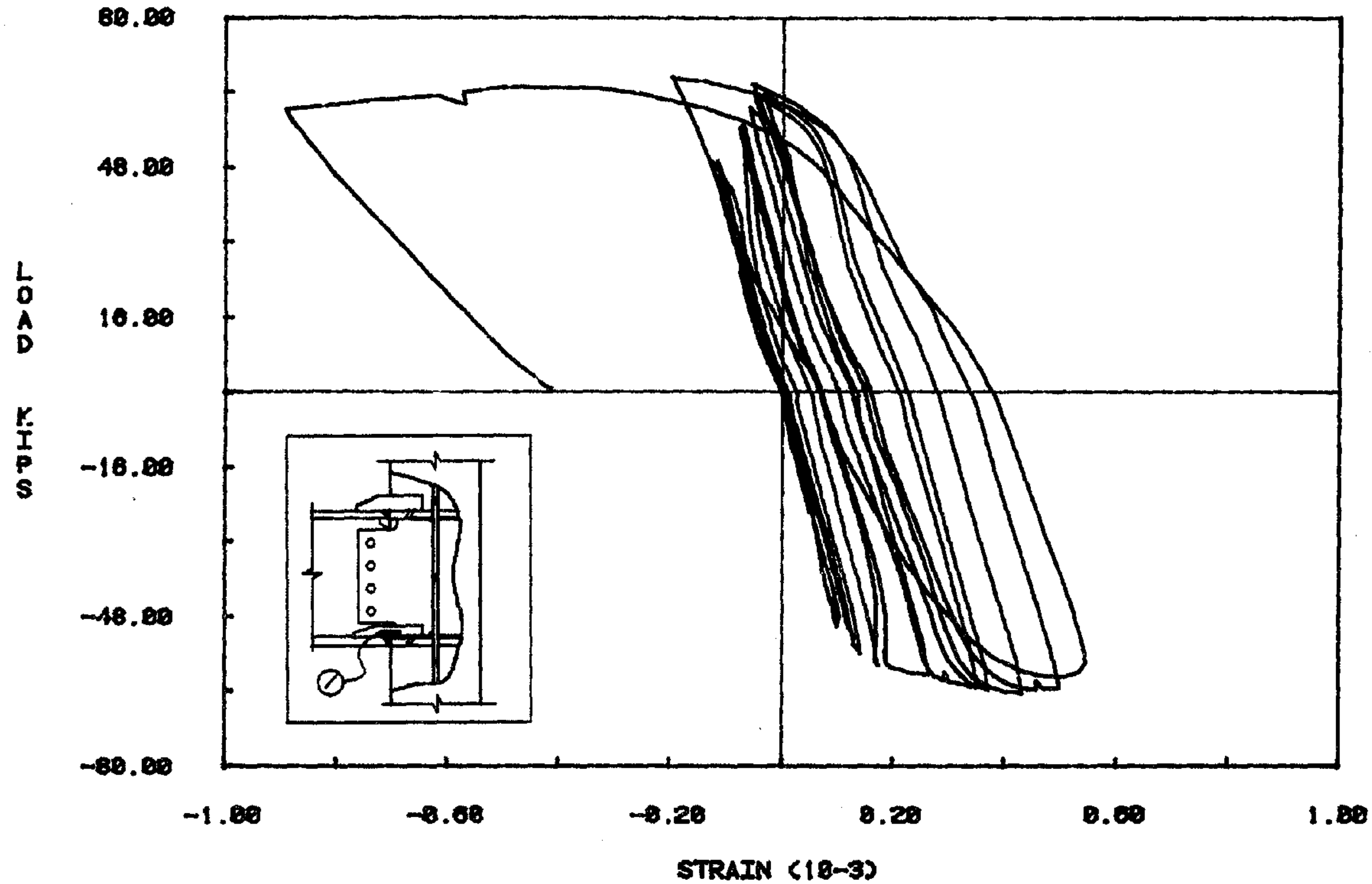



Fig. 4.5 Strain in Bottom Reinforcing Rib (Specimen 1)

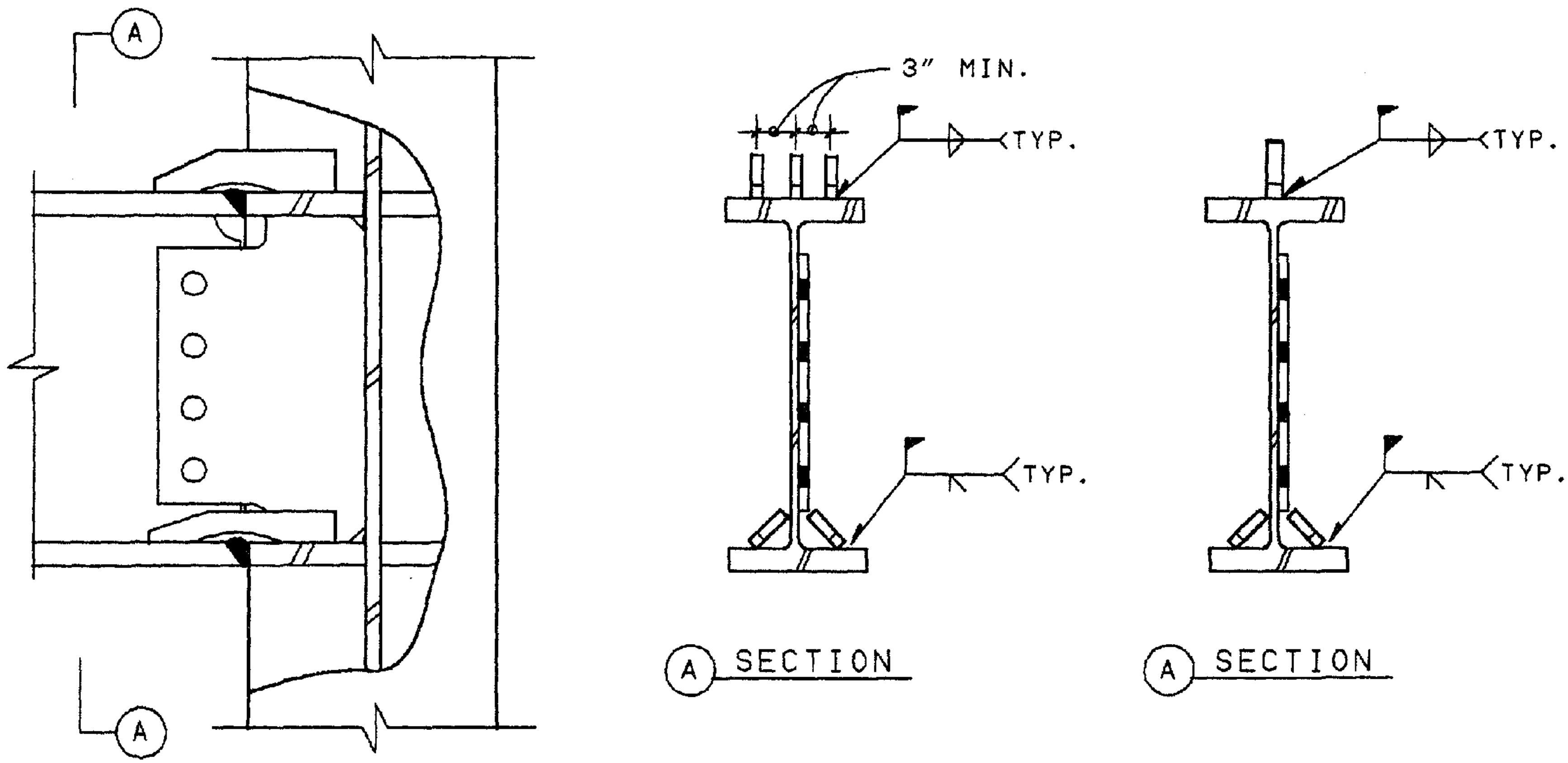



Fig. 5.1 Alternative Reinforcing Rib Details

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UCB/EERC-79/02 "The Development of a Mathematical Model to Predict the Flexural Response of Reinforced Concrete Beams to Cyclic Loads, Using System Identification," by J. Stanton & H. McNiven - Jan. 1979(PB 295 875)A10

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UCB/EERC-79/07 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 1 - Summary Report," by P.N. Spencer, V.F. Zackay, and E.R. Parker - Feb. 1979(UCB/EERC-79/07)A09

UCB/EERC-79/08 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 2 - The Development of Analyses for Reactor System Piping,""Simple Systems" by M.C. Lee, J. Penzien, A.K. Chopra and K. Suzuki "Complex Systems" by G.H. Powell, E.L. Wilson, R.W. Clough and D.G. Row - Feb. 1979(UCB/EERC-79/08)A10

UCB/EERC-79/09 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 3 - Evaluation of Commercial Steels," by W.S. Owen, R.M.N. Pelloux, R.O. Ritchie, M. Faral, T. Ohhashi, J. Toplosky, S.J. Hartman, V.F. Zackay and E.R. Parker - Feb. 1979(UCB/EERC-79/09)A04

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