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NAME OF AUTHOR/NOM DE L'AUTEUR Alastair Kent Dunbar

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NAME OF SUPERVISOR/NOM DU DIRECTEUR DE THÈSE Dr. J. Warwaruk

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THE UNIVERSITY OF ALBERTA

PRESTRESSED CONCRETE TEE BEAMS CONTAINING
LARGE CIRCULAR WEB OPENINGS

by

©

ALASTAIR K. DUNBAR

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF MASTER OF SCIENCE

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EDMONTON, ALBERTA

FALL, 1976

THE UNIVERSITY OF ALBERTA
FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled "PRESTRESSED CONCRETE TEE BEAMS CONTAINING LARGE CIRCULAR WEB OPENINGS" submitted by Alastair K. Dunbar in partial fulfilment of the requirements for the degree of Master of Science.

J. Barwaruk
.....
J. Barwaruk Supervisor

J.G. MacGregor
.....
J.G. MacGregor

J.S. Kennedy
.....
J.S. Kennedy

DATE *October 4 / 1976*
.....

TO JANE

ABSTRACT

The research described in this thesis is the fourth such series of tests in a program initiated at the University of Alberta to study the behaviour of Prestressed Concrete tee beams containing large web openings. The experimental phase was carried out in the I. F. Morrison Structural Engineering Laboratory at the University of Alberta under the supervision of Dr. J. Warwaruk.

The present test series consisted of twelve Prestressed Concrete tee beams with large circular web openings, loaded to failure with two symmetrically placed point loads. All beams were 20 inches deep with a 20-inch flange and were prestressed with five prestressing strands of 3/8-inch diameter.

The major variables were: the amount and arrangement of shear reinforcement, the size and spacing of the web openings, and the positions of the load points.

Although further study is required to develop a rational design procedure for such beams, it was concluded that the most critical factor is the provision of adequate shear reinforcement across the minimum sections above and below the circular openings. Nominal reinforcement in the posts between holes seemed sufficient to avoid failure in those regions.

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CHAPTER I
INTRODUCTION

In building construction, the reduction in floor thickness made possible by passing building services through floor-beams, rather than under them, can cause a significant reduction in construction and maintenance costs, by reducing the total building height and volume. This is often accomplished using open-web steel joists or expanded-web steel girders. In concrete construction, the same effect can be achieved by either allowing the cutting of web openings by those involved in the installation of services which is a dangerous practise, or by designing beams containing web openings.

At the present time, however, there is not sufficient information available to allow an engineer to design such beams with any confidence in both their safety and economy. With the long-range goal of contributing to the development of design procedures, an experimental program was initiated at the University of Alberta to study the behaviour of prestressed concrete tee beams containing large web openings. The first part of the program was Sauve's study⁽¹⁰⁾ of prestressed concrete tee beams containing rectangular web openings and various arrangements of longitudinal and vertical shear reinforcement. Le Blanc⁽³⁾ continued with similar test beams, also varying the hole

shape, the prestress force, and the slope of the shear reinforcement. Finally, Linder(*) examined in more detail the effects of these variables, introduced further variations of shear reinforcement, and also varied hole size.

The present series shifts the focus from rectangular and parallelogram shaped openings to circular ones. The variables of principal concern were arrangement of shear reinforcement and loading configuration, although the effects of varying hole size and spacing were also briefly considered.

Twelve beams were tested, the results being recorded in the form of Demec strain gauge readings, electrical resistance strain gauge readings, and deflection readings. In addition, photographs were taken of the beams at various loading stages to record cracking patterns and failure modes.

CHAPTER II

REVIEW OF PREVIOUS WORK

The use of concrete members with large web openings is much less widespread than the use of open-web steel joists and other truss-type steel structures. In order to enable concrete to provide a better alternative than it has in the past, economical and reliable design procedures must be developed. To this end a limited amount of research has been done which suggests that such an alternative may be viable, but requires further research.

2.1 Prestressed Concrete Tee Beams with Large Web Openings

Sauve⁽¹⁰⁾ conducted tests in the Structural Engineering Laboratory of the University of Alberta on nine prestressed concrete tee beams with rectangular web openings. This program investigated nine 24 foot model beams tested under two point loads at various spacings. Other variables included: vertical shear reinforcement, as well as longitudinal reinforcement and supplementary lower web shear reinforcement.

Some observations and conclusions from this program were:

- a) Prestressed concrete tee beams containing large web openings cannot be designed for flexure using the ACI Code's minimum shear reinforcement conditions, even though the

actual ultimate shear capacity obtained could far exceed that designed for flexure.

b) Any additional shear reinforcement provided, served to increase the load carrying capacity of a beam containing large web openings by an amount ranging from 15% to 22%.

c) Additional shear reinforcement also confined failure to a location where there was an abrupt change in cross-sectional area of concrete.

d) An amount of vertical reinforcement, placed in the posts, gave these posts the capacity required to cause a localizing of the failure in the lower web, if this web had no vertical reinforcement. However, placement of a minimum amount of inclined shear reinforcement in the lower web caused the failure to be localized in the posts.

e) The addition of both post and lower web reinforcement resulted in a redistribution of stresses in the shear span such that all sections were more equally stressed in diagonal tension.

f) Considerable increase in supplementary longitudinal reinforcement did not significantly increase the shear capacity of the beams.

g) A decrease in the number of openings in the shear span increased considerably the shear capacity of the beams.

2.2 Parallelogram Shaped Openings in Prestressed Concrete Tee Beams

Le Blanc⁽³⁾, in a follow-up to Sauve's work, tested ten prestressed concrete tee beams with rectangular and parallelogram shaped web openings in the Structural Engineering Laboratory at the University of Alberta. The details of the beams were similar to Sauve's, and were tested under various load arrangements. The prestress force and the amount of shear reinforcement used were two important variables, however the main concern was the effects of the parallelogram type openings in the beams, compared with the rectangular openings or no openings.

Some of his observations and conclusions were:

a) Parallelogram shaped openings accompanied by inclined shear stirrups, in prestressed concrete tee beams containing web openings, result in a beam with a higher ultimate capacity relative to rectangular shaped openings and vertical shear stirrups, when the failure is a shear failure or a combined shear and bending type of failure.

b) Since shear failures occur through the hole sections, shear reinforcement in the posts is not sufficient in itself to prevent a shear failure.

c) Lower web reinforcement in the shear span below the web openings, increases the ability of the beam to withstand more severe shear stresses, thereby increasing the possibility of flexural failure.

d) Shear reinforcement, above the hole sections in the shear spans, would probably increase the possibility of a flexural failure in cases where the loading produces severe shear stress conditions, since shear failures tend to initiate in this region.

e) The ultimate load and moment calculated using the theoretical ACI Code procedure, in the case of flexural failures, is very conservative relative to the measured ultimate load and moment obtained from these tests.

f) Deflections at flexural ultimate loads were higher for beams having web openings than for the control beam having none, due to the decrease in initial stiffness caused by the introduction of the openings.

2.3 Prestressed Tee Beams with Large Web Openings

Ragan and Warwaruk^(s) tested six prestressed tee beams containing web openings. In this program, four model beams were tested in the Structural Engineering Laboratory of the University of Alberta and two full size beams were tested in the field. The design of the two full size beams was based on the results obtained from the model beams.

Some of their observations and conclusions were:

a) Cracking extended vertically downward from approximately the centre point of the web openings; hence it was concluded in the full size beams, to distribute the prestressing strands almost evenly across the vertical section

of the lower web.

b) Severe cracking at the connection of the "post" and flange led to the provision of steel in the posts at 1.3% of the horizontal area of these posts.

c) All failures were due to inclined cracking in the lower web and always in the half span which had the least amount of web reinforcement; hence the full size beams were provided with "U" stirrups in the lower web spaced at six to twelve inches.

d) The mode of failure of all beams with openings was by the formation of mechanisms. None of the beams, neither model or full size, failed in a flexural manner.

2.4 Reinforced Concrete Tee Beams with a Web Opening

Lorentsen⁽⁵⁾, of the Royal Institute of Technology in Stockholm, conducted analytical and experimental research on reinforced concrete girders having a single web opening. Four beams were tested under different loading conditions.

In general, his tests confirmed the behaviour pattern predicted by the elastic theory and also showed that there was a reserve of capacity available, due to the redistribution of moments between stressed sections located at the edges of the openings. Lorentsen also showed that for statically loaded structures, satisfactory structural capacity can be achieved if the sections near a hole are designed to resist the normal and bending forces. He pointed

out that in simply supported beams supporting a uniformly distributed load, there are two factors which make the location of a hole near the midspan desirable:

- a) The magnitude of the flange moment, and
- b) The low shear force relative to normal force

The flange moment refers to the moment in the upper flange at the left end of the hole, not the contribution of flange compression to overall moment resistance at a section. Since this moment is basically the sum of the fixed end moment of the flange itself induced by the portion of the load applied above the hole and thus dependent only on hole length, and a component produced by the transverse shear at the section, it follows that the flange moment will be minimal near the centerline. Similarly, the high normal force in the flange at centerline, relative to shear force, enhances the shear capacity of the flange, while the opposite situation at the supports induces high principal tension stresses in the flange.

An important conclusion which Lorentsen made is that, in the region of a hole, the members should be overdesigned with respect to crushing of the concrete. This guards against the occurrence of a plastic condition at working loads. Such a condition is not desirable, since the moment near a hole may change sign, depending on the position of the load. He also concluded that the principles presented can be applied to cases where more than one hole

exists.

2.5 Rectangular Reinforced Concrete Beams with Large Web Openings

Nasser, Acavalos and Daniel(?) of the University of Saskatchewan, tested nine rectangular beams containing web openings. They were able to verify that: large openings behave similarly to a Vierendeel panel, contraflexure points exist at the approximate midspan of the cross members, the diagonal force concentration at the corners of an opening is twice the simple shear force, and adequately reinforced large openings do not reduce the ultimate capacity of a beam, but reduce the stiffness and hence increase deflections.

2.6 Circular Openings in Webs of Continuous Beams

Somes and Corley(11) conducted tests in the Portland Cement Association Structural Development Laboratory on 19 reinforced concrete tee beams containing circular web openings. The specimens were proportioned to model the negative moment region at an interior support of a typical continuous joist and were cast with sanded lightweight-aggregate concrete. Some of their design recommendations were:

- a) An opening should encroach no closer to the extreme compression fibre than the depth of the equivalent

rectangular stress block "a" defined by Sec. 10.2.7(1). With this requirement satisfied, small isolated openings may be located anywhere in the web without strength reduction.

b) Large openings should have well-anchored stirrup reinforcement on either side. This reinforcement should be located as close to the opening as is consistent with the ACI standard requirements for concrete cover.

c) On each side of a large opening the area of stirrup reinforcement should be not less than the shear capacity of the joist at the location without the opening divided by the specified minimum yield strength of the stirrup steel. The shear capacity should be calculated according to Sec. 11.5(1).

d) Where multiple openings are used, the minimum horizontal dimension of the concrete between adjacent openings should be not less than 0.25 of the web depth or 4 inches, whichever is greater.

CHAPTER III
RESEARCH PROGRAM

This test series forms the fourth part of a program initiated to supply the experimental background necessary for the development of adequate design procedures for prestressed concrete tee beams containing large web openings. The first three series studied the effects of rectangular and parallelogram shaped openings on the behaviour of such beams, while this investigation introduces circular web openings.

Justification for the study of the effects of circular holes is two-fold. First, many of the building services that may be threaded through the holes are cylindrical, such as plumbing and some air-ducts, or would fit as easily through circular holes as any other shape, such as electrical conduit. Thus in practice a circular hole may be more desirable than a rectangular one. Second, intuition, supported by the results of the tests, suggests that the mode of behaviour of a beam is strongly influenced by web opening geometry. Thus a design of a beam with circular web openings based only on studies of rectangular holes would at best be a crude approximation.

The fabrication and testing of the beams of this study were carried out concurrently with those of Linder. Good co-ordination of the two test series avoided logistical

problems in the laboratory, and improved the efficiency of both operations with respect to time, material, and manpower.

Twelve specimens, described in Table 3.1 and Figure 3.5, were tested, each containing circular web openings. The section used was a simple tee with an overall depth of 20 inches, as illustrated in Figures 3.1 and 3.2. The location of the centerlines of the holes was at a depth of 8-1/2 inches from the top of the flange. Unlike previous sections used by Le Blanc and Sauve, no corners were beveled, although 1-1/2 by 1-1/2 inch fillets were formed at the web-flange junctions. Longitudinal mild steel reinforcement consisted of eight #3 deformed reinforcing bars, extending through the full length of all beams. Four of these were placed at mid-depth of the flange, two just above the holes, and two in the bottom of the web. In this way the steel was proportioned to resist initial tension stresses in the flange, to increase the moment capacity of the section, and to facilitate construction and handling of the reinforcing cages. Five seven-wire prestressing strands of 3/8 inch nominal diameter were positioned at the corners and centre of a 2 inch by 2 inch square at an average depth of 16 inches.

The first beam had ten holes spaced at 16 inches on centre, a span between support centerlines of 20 feet, and two loads placed symmetrically on the beam 8 feet apart.

Subsequent beams had holes spaced at 20 inches, spans between support centerlines of 20 feet 4 inches (for beams AD-2 to AD-4), or 16 feet 4 inches (for beams AD-5 to AD-12), but with the two loads located 8 feet 4 inches apart. These proportions enabled the loads to be placed midway between holes, while maintaining constant hole spacing throughout the perforated region and shear spans of either four or six feet. The length allowed for anchorage of the strands beyond the support centerlines at each end varied from 1 foot 6-1/2 inches to 2 feet, depending on the space available in the forms. There were seven or nine holes per beam (depending on beam length) of 8 inch (beams AD-7 and AD-8) or 10 inch diameter. Thus, while the average hole area per foot of beam length was less than that of beams with rectangular or parallelogram shaped holes by from 11% to 57%, the maximum depth of web removed by a hole was 25% higher.

The arrangement of shear reinforcement was the principal variable. It consisted of full-depth open U-stirrups of #3 deformed bars, and smaller closed stirrups of #2 bars. The U-stirrups placed at 45° and 90° to the longitudinal axes of the beams were bent to the specified shape by the supplier, while those placed at 60° were produced in the laboratory by re-bending the 45° stirrups. The smaller stirrups were fabricated in the laboratory from straight #2 bars bent on a simple bending jig, and were placed at 45° above and below the holes. The lower web

stirrups were anchored by means of wire ties where their ends lapped around a corner. Figures 3.3 and 3.5 show stirrup details and locations in the test beams.

The prestressing operation was carried out between two reinforced concrete abutments anchored to the load floor. The first four beams were cast singly, the second four were each cast with one of Linder's beams in tandem, while the last four were cast as two pairs in tandem. The formwork used was that designed by Linder, consisting of built-up steel forms lined with plywood treated to resist damage from fresh concrete. The openings were formed by styrofoam blocks held in place by small truncated plywood "cones" which were wedged into sockets in the blocks. These cones were attached to the form liners with wood-screws such that they could be easily relocated.

The concrete mix used was proportioned to produce a nominal strength of 5000 psi, and contained High Early Strength cement. Typically, three standard test cylinders were cast from each batch of concrete, allowed to cure under the same conditions as the beams, and were then tested for compressive strength and tensile splitting strength on the same day as the corresponding beam test. Transfer of the prestress force was accomplished after either a four or five day moist curing period. The beams and cylinders were then stored in an exposed condition in the laboratory, until being tested at ages ranging from 15 days to 36 days.

The instrumentation of the beams was of three types: electrical resistance strain gauges, Demec points, and metal scales graduated in hundredths of an inch. The strain gauges were mounted on the shear reinforcement at locations that appeared to be critical, judging from the cracking behaviour of the first beams, and on the prestressing strands at selected sections. These were carefully waterproofed with a coating of epoxy, which also served to protect them from being damaged by the aggregate during casting. Pairs of Demec points were mounted at a section near the beam centerline as shown in Figure 3.4 to determine the strain changes induced by creep and elastic shortening at transfer, especially at the level of the prestressing strands in the lower web. Strain distributions across the depth of the section were also derived as the beams were loaded. Deflection readings were obtained for each increment of load with the use of a surveying level and scales suspended from the bottom of the web at the load points and centerline.

The loading apparatus, as pictured in Figure 3.7, consisted of two hydraulic jacks mounted on a steel frame secured to the load floor, and two concrete pedestals. The jacks were controlled from an Amsler loading apparatus, such that they would exert equal loads, and were positioned to bear down on two steel plates, plastered to the top of the flange. The rounded knife-edges, which provided the point of

contact between the beams and the supports, were mounted on small roller-wheeled carts that traveled on bearing plates bolted to the concrete pedestals. To provide structural stability, one of these carts was restrained by a threaded rod, while the other was free to roll as the bottom fibre of the beam elongated. This motion of the rolling support upset the symmetry of loading by allowing the centre portion of the beams to move in the same direction. To counteract this undesirable effect, the position of the fixed support could be adjusted during testing by means of the threaded rod. The underside of the web was protected from stress concentrations by the steel beam seat of Figure 3.6. To guard against the slight possibility of a beam tipping over sideways under high loads, a frame of 5 inch channels was bolted across each end of the loading frame above the test beam to restrain the flange.

TABLE 3.1
GENERAL BEAM DATA

BEAM NO.	CLEAR SPAN ft, in	SHEAR SPAN ft, in	NO. OF HOLES	HOLE SIZE in	HOLE SPACING in	AGE AT RELEASE days	AGE AT TESTING days
AD-1	20-0	6-0	10	10	16	5	27
AD-2	20-4	6-0	9	10	20	5	23
AD-3	20-4	6-0	9	10	20	5	23
AD-4	20-4	6-0	9	10	20	4	22
AD-5	16-4	4-0	7	10	20	5	34
AD-6	16-4	4-0	7	10	20	5	30
AD-7	16-4	4-0	7	8	20	5	15
AD-8	16-4	4-0	7	8	20	5	20
AD-9	16-4	4-0	7	10	20	5	29
AD-10	16-4	4-0	7	10	20	5	36
AD-11	16-4	4-0	7	10	20	4	19
AD-12	16-4	4-0	7	10	20	4	21

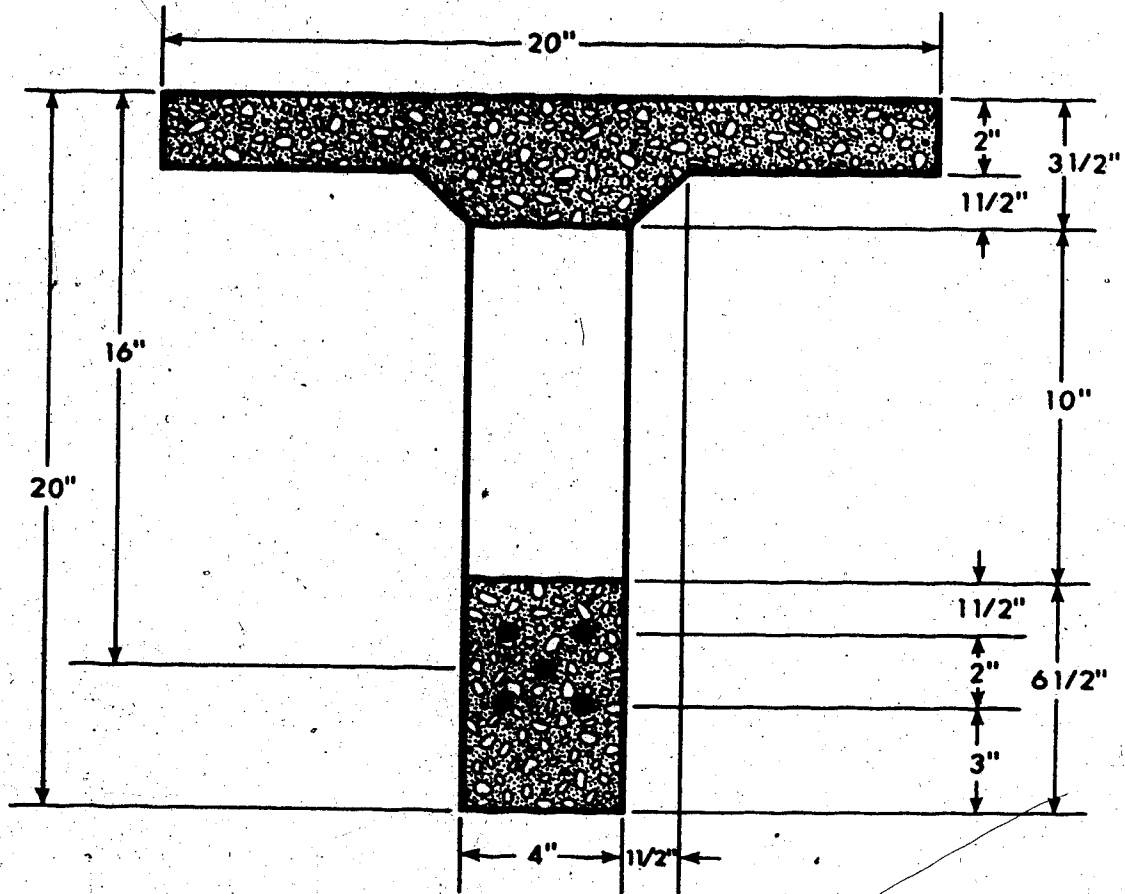


FIGURE 3.1

TYPICAL CONCRETE CROSS-SECTION

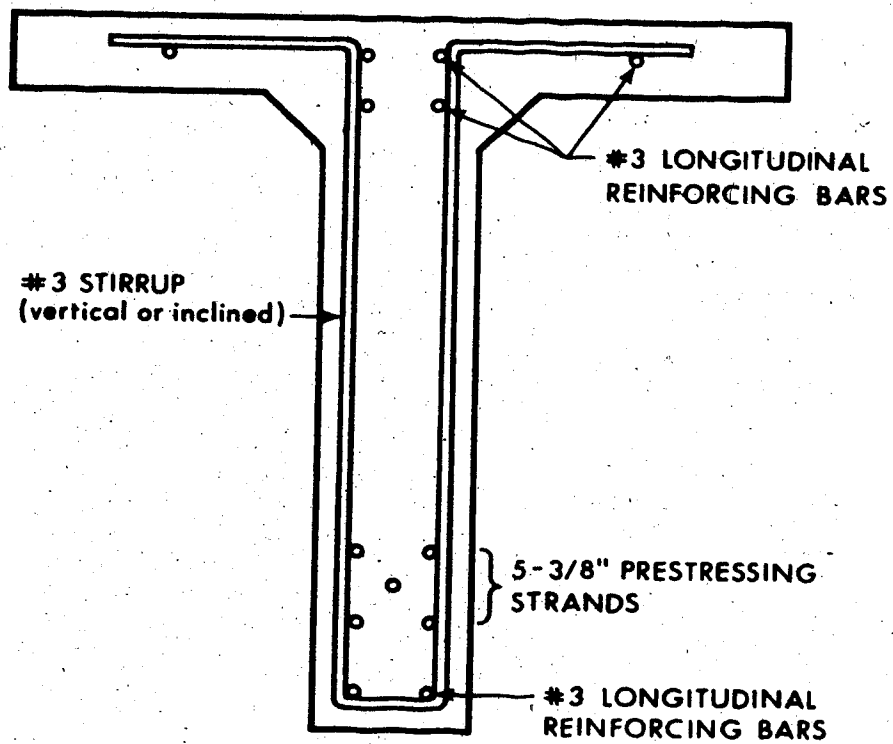


FIGURE 3.2

TYPICAL REINFORCEMENT DETAILS

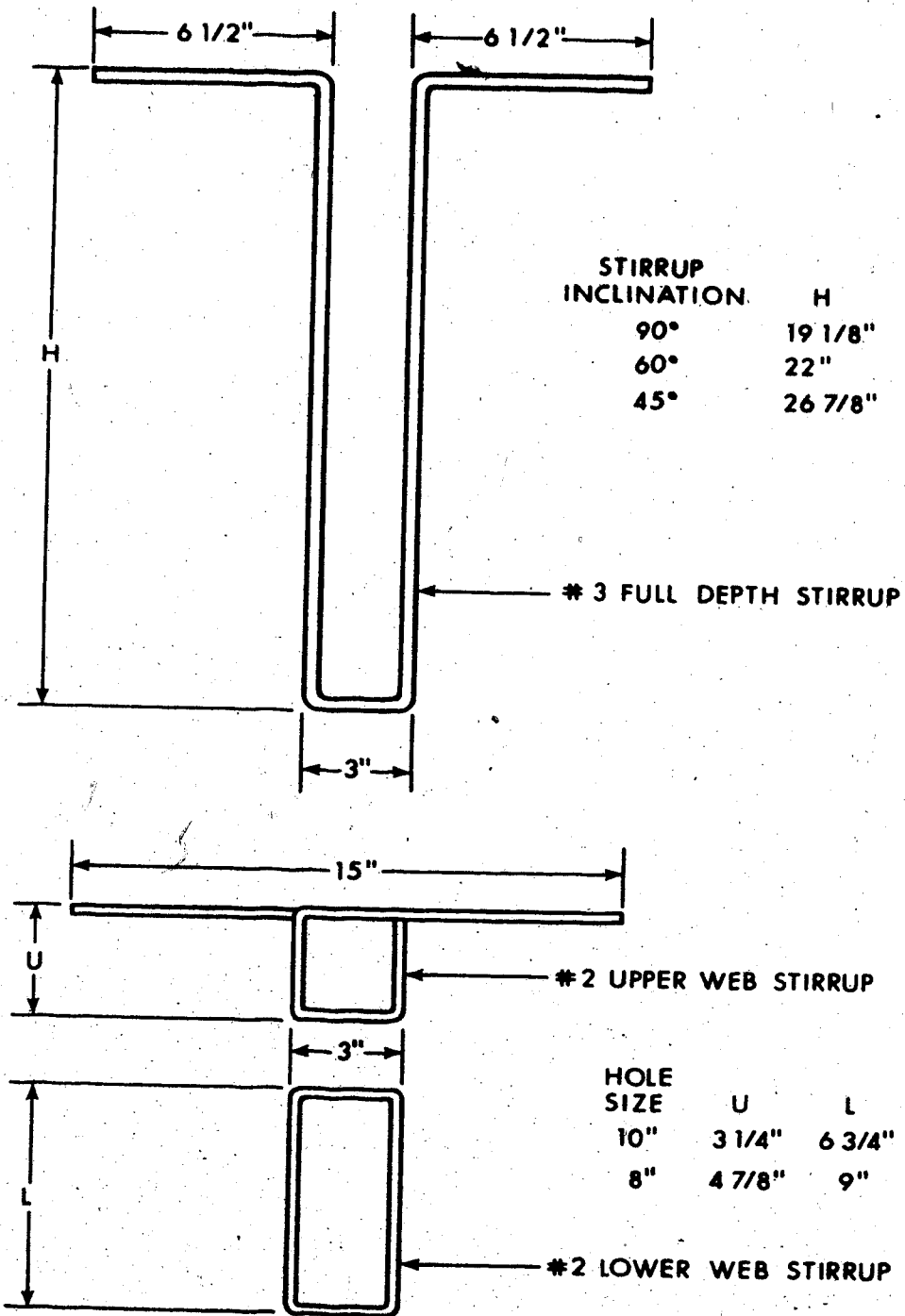


FIGURE 3.3
STIRRUP DETAILS

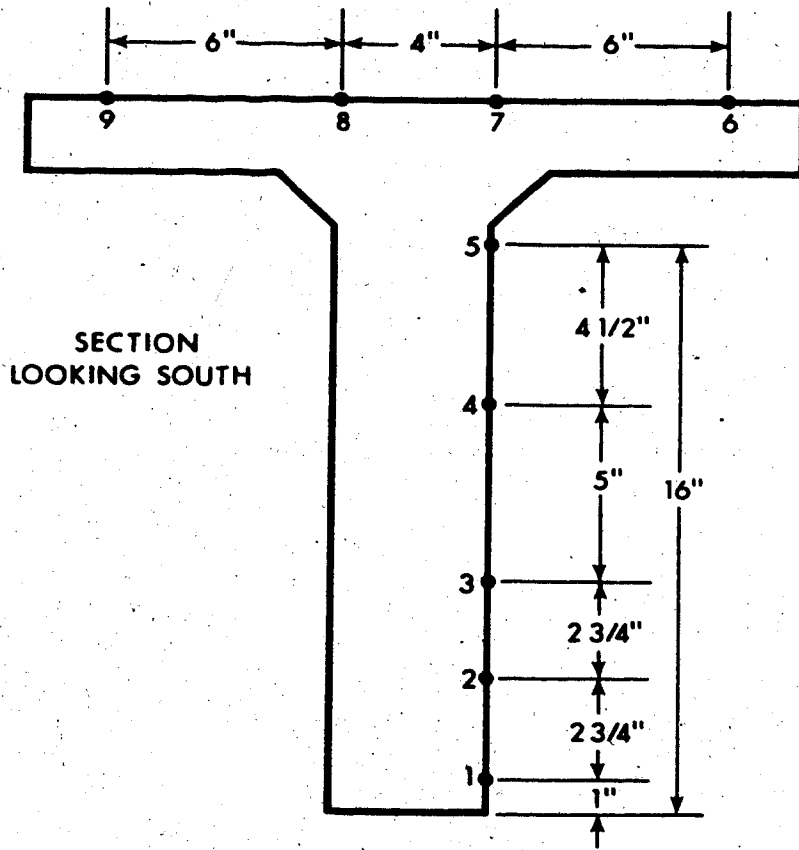
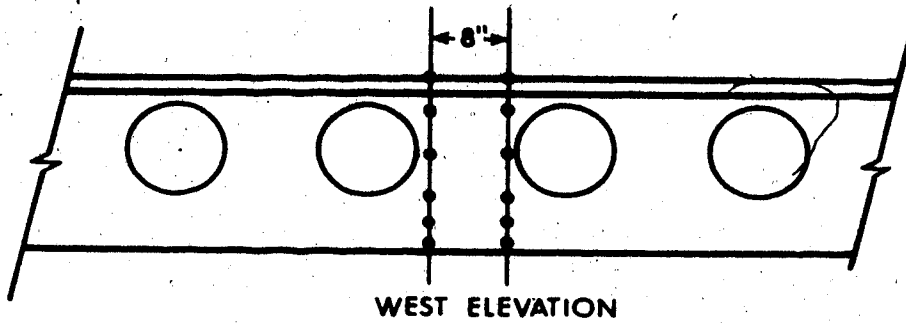


FIGURE 3.4
DEMEC POINT LOCATIONS

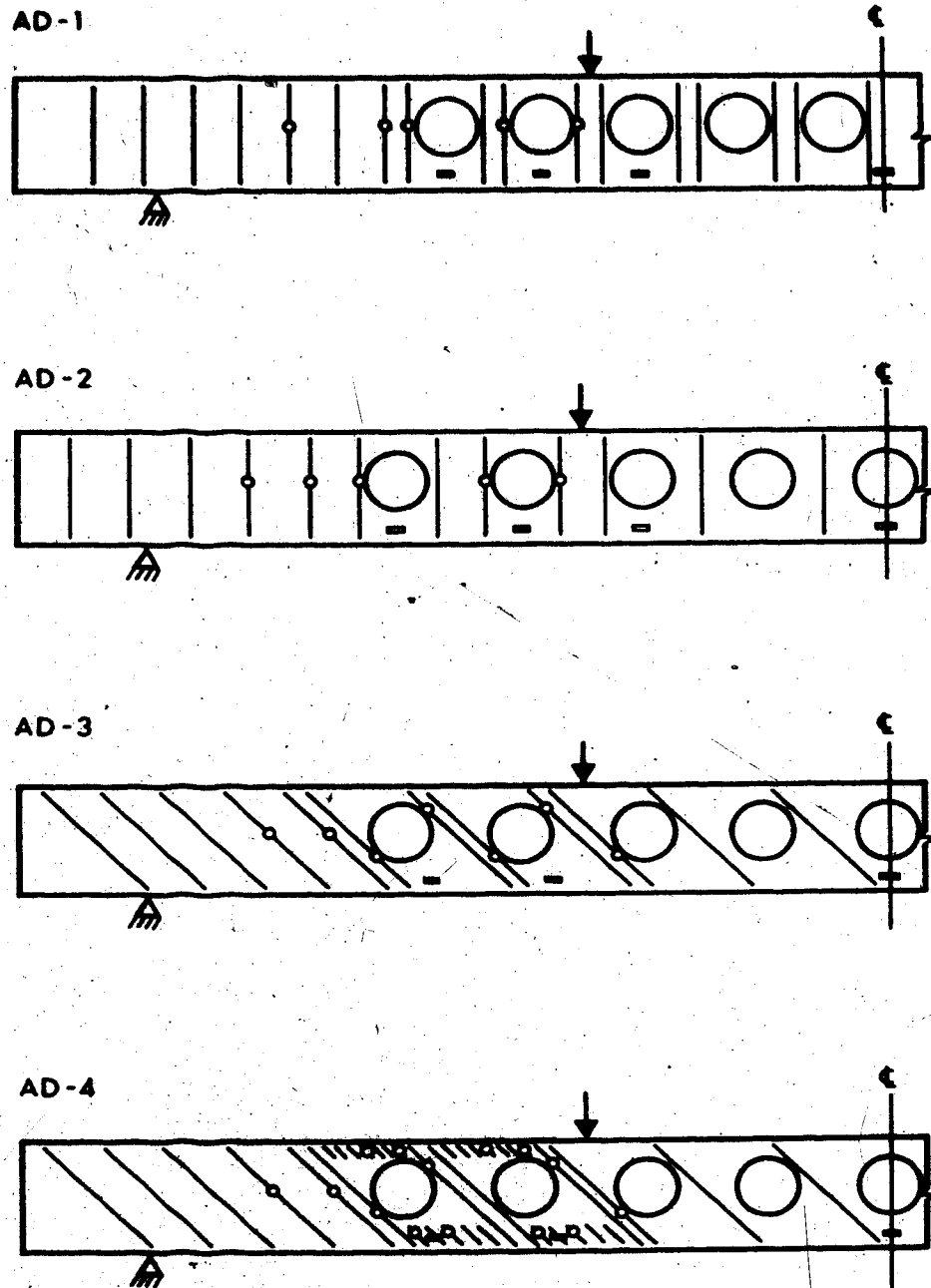


FIGURE 3.5.1

SHEAR REINFORCEMENT DETAILS CASTING GROUP ONE

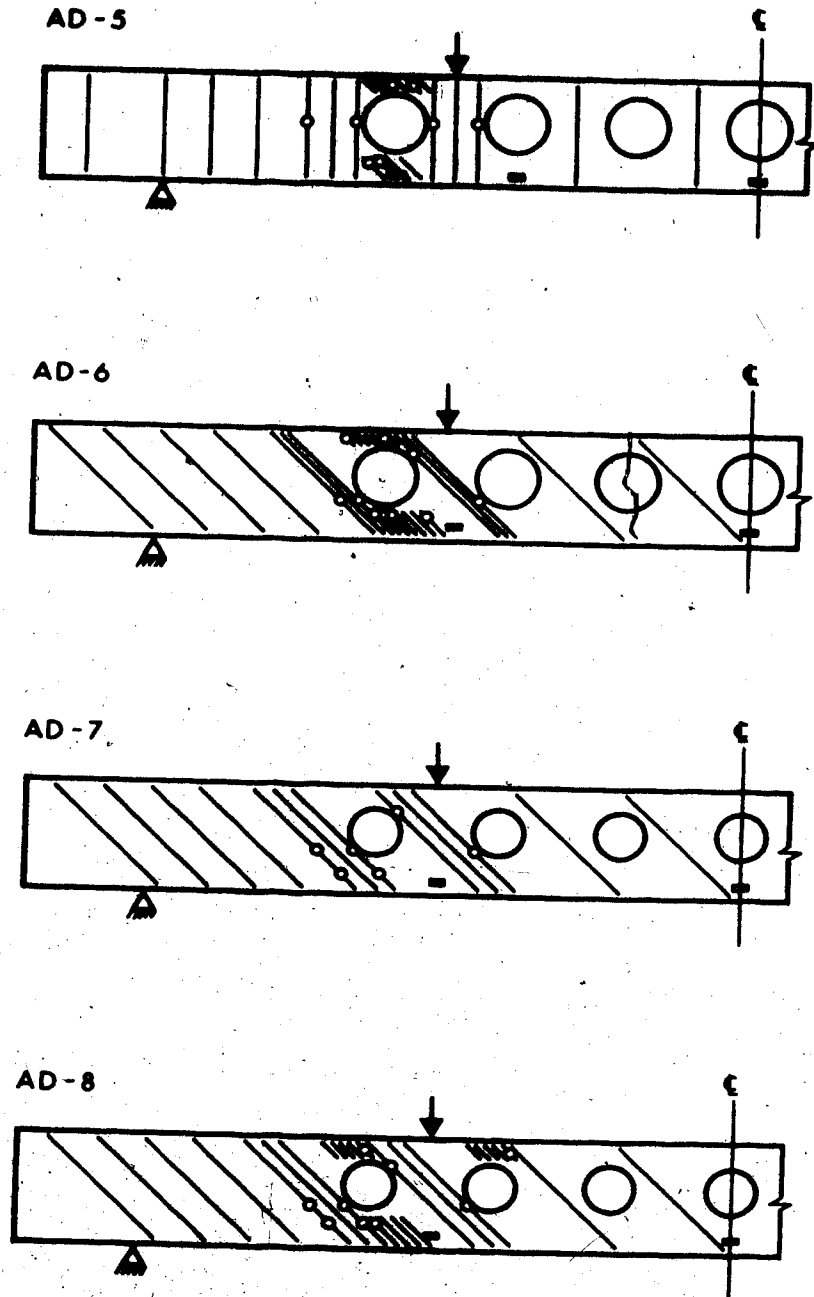


FIGURE 3.5.2
SHEAR REINFORCEMENT DETAILS CASTING GROUP TWO

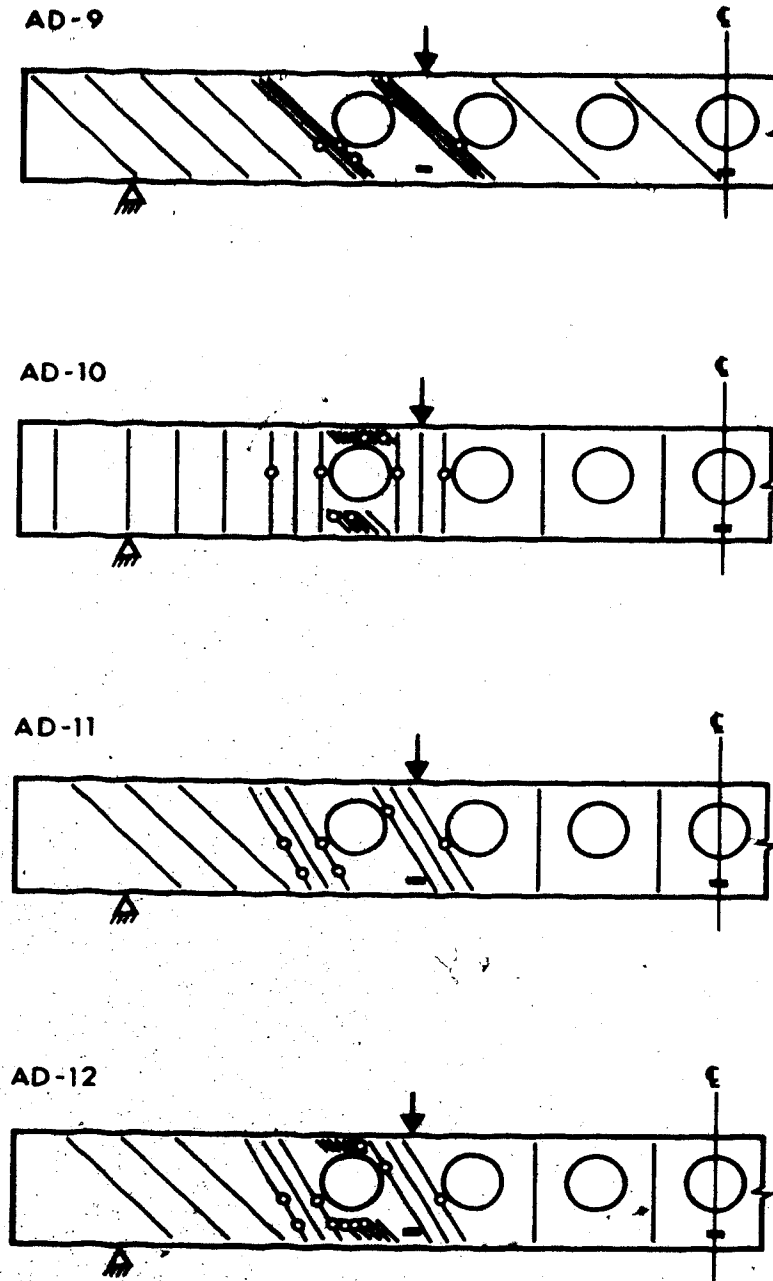


FIGURE 3.5.3

SHEAR REINFORCEMENT DETAILS CASTING GROUP THREE

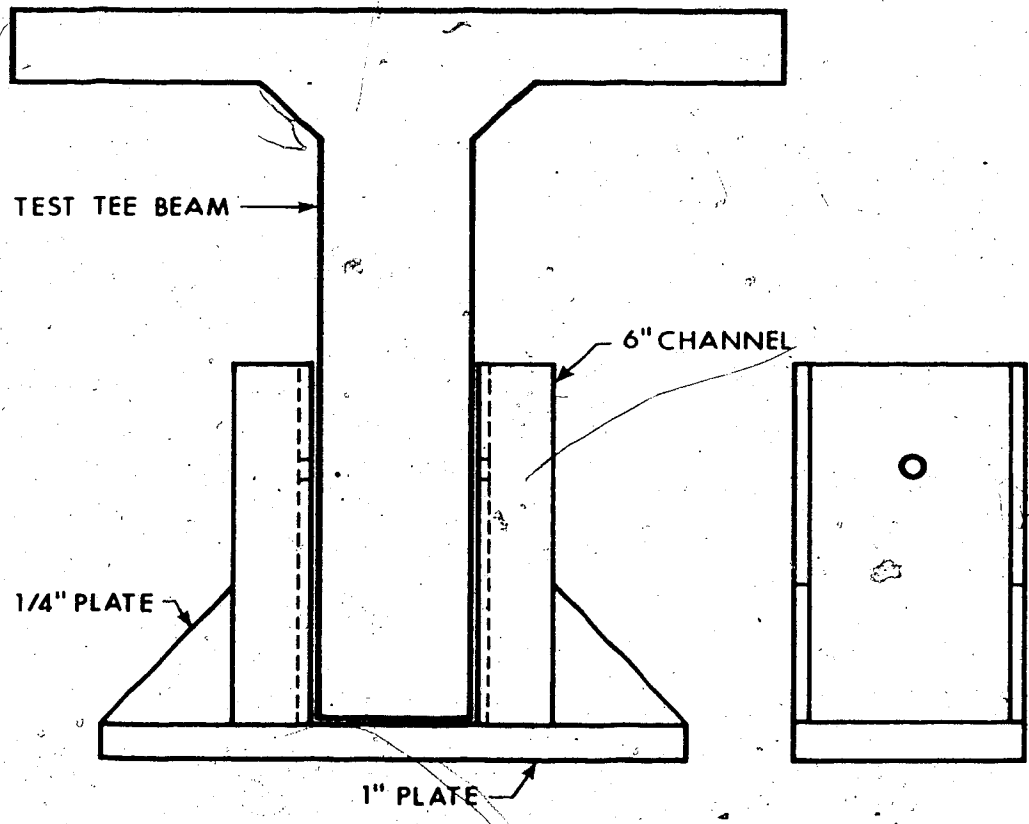
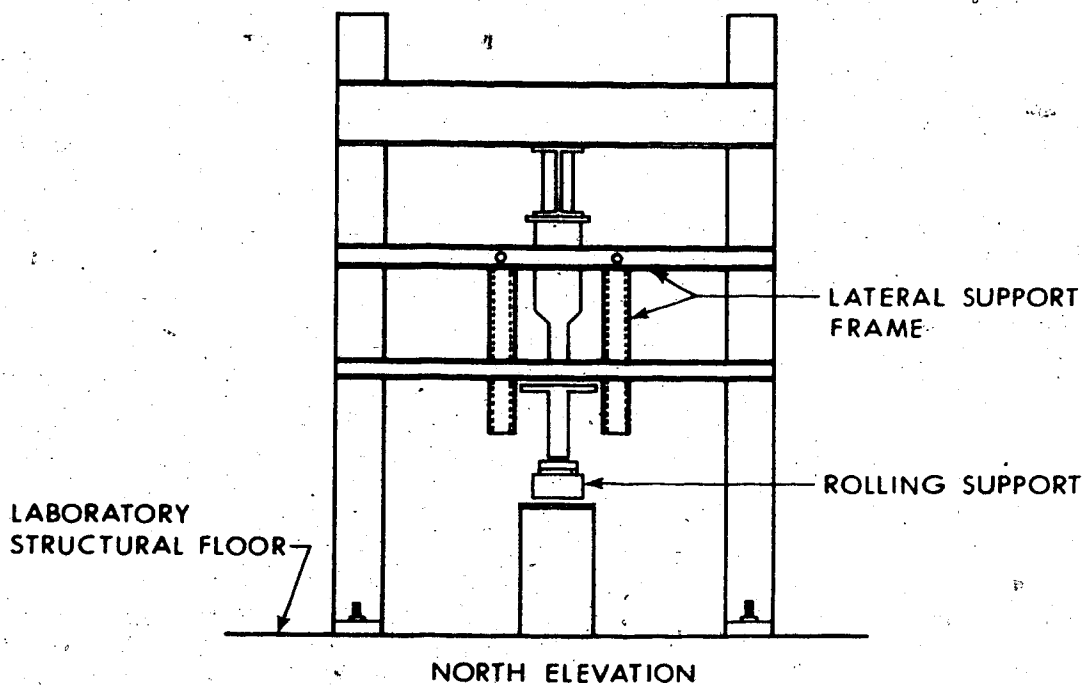
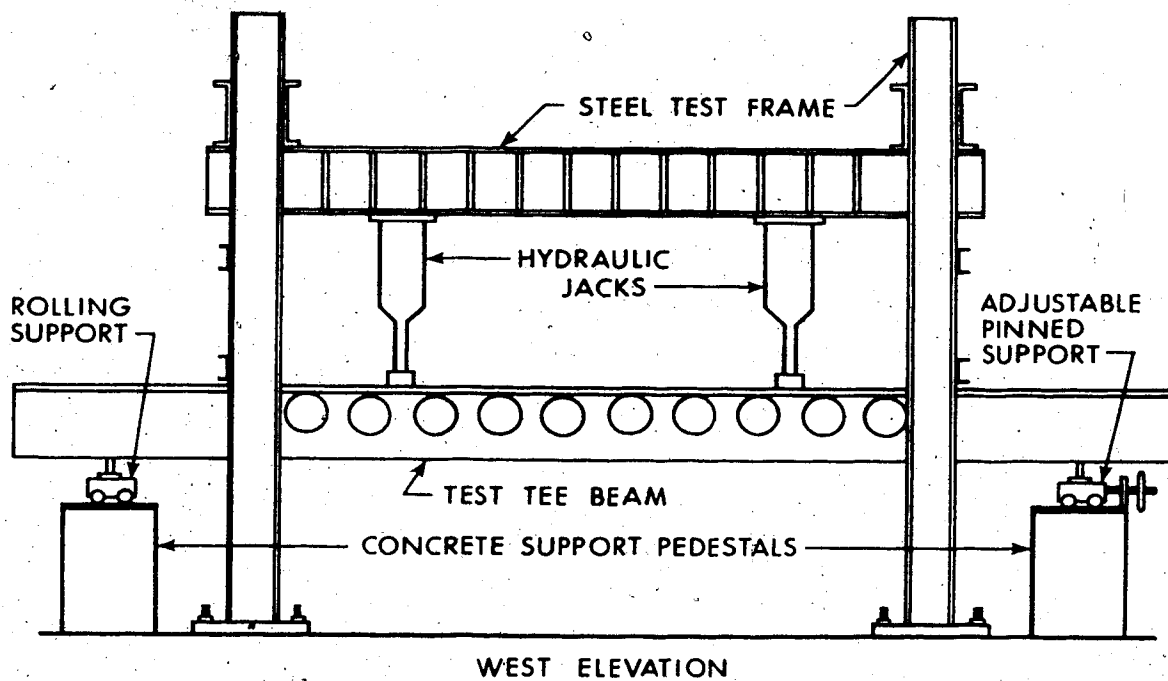


FIGURE 3.6
BEAM SEAT DETAIL



WEST ELEVATION
NORTH ELEVATION
FIGURE 3.7
TYPICAL TEST SET UP

CHAPTER IV

TEST RESULTS

The results of the tests of twelve tee beams are presented in this chapter in tabular, graphical and photographic form. The values plotted were derived from readings taken during testing with the use of a survey level, electrical resistance strain gauges, Demec strain gauges, and the Amsler loading apparatus, and are tabulated in Appendix B.

4.1 Summary of Test Results

Table 4.1 gives simplified descriptions of the shear reinforcement details of the beams, and corresponding failure loads. Table 4.2 compares the observed ultimate moments with theoretical values based on Eq. 18-3 of the 1971 ACI Building Code⁽¹⁾, a strain-compatibility analysis, and the assumption of a stress of 275 ksi in the prestressing strand at failure. Sample calculations of these moments are given in Appendix C. In all cases the non-prestressed tensile reinforcement was considered to contribute to moment capacity according to Sec. 18.7.2 of the ACI Code⁽¹⁾. Also in this table the failure mode of each beam is described as either shear, shear-compression, or flexural. This classification was made on the simplistic, descriptive basis explained in Sec. 5.2 to facilitate the

evaluation of the shear reinforcement arrangements used.

4.2 Load-Deflection Relationships

Figures 4.1.1 to 4.1.3 illustrate the relationships between moment and centerline deflections. For clarity, the beams are divided into three groups of four and each curve is referenced with the corresponding beam number. These groups are termed "Casting Groups" one (long beams cast singly), two (short beams cast in tandem with Linder's), and three (short beams cast in pairs in tandem). Deflection readings were also taken at the load points such that the three readings could be used to calculate an average curvature in the pure moment region. The resulting moment-curvature relationships are plotted on Figures 4.2.1 to 4.2.3.

4.3 Strain Distribution

Strain distributions within beam sections, as calculated from Demec strain gauge readings, are shown for selected loads in Figures 4.3.1 to 4.3.3. Demec point locations near beam centerlines are as depicted in Figure 3.4. In addition, the strain in the top fibre of the flange given by Demec readings is plotted against moment in Figures 4.4.1 to 4.4.3.

4.4 Moment-Reinforcement Strain Relationships

The remaining figures show the relationships between applied moment and strain in the reinforcement obtained from readings of the strain gauges mounted on the prestressing strand and shear reinforcement. General locations of these gauges are shown in Figure 4.5. Individual curves are referenced with beam number and gauge number in order that the exact location of any gauge can be found in Appendix B.

Strains in the prestressing strand are plotted on Figures 4.6.1 to 4.7.4. The first three figures deal with strain in the strand at centerline, while the rest deal with the other four general locations. Figures 4.8.1 to 4.11 illustrate the relationships between moment and strain in the full-depth #3 stirrups, the upper web #2 stirrups, and the lower web #2 stirrups.

4.5 Cracking and Failure Patterns

Plates 4.1 to 4.12 consist of photographs of each of the beams at two stages of loading. The first load is one at which the general cracking pattern has developed, and the second is the failure load. Close-up views of the failure regions of beams AD-1, AD-7, and AD-8 are shown in Plates 4.13 to 4.15 to illustrate the three modes of failure by which the beams are classified.

TABLE 4.1

SUMMARY OF TEST RESULTS

BEAM NO.	SHEAR SPAN ft	STIRRUP SLOPE degrees	STIRRUPS PER POST	AVERAGE SPACING		FAILURE LOAD kips
				UPPER in	LOWER in	
AD-1	6	90	2	none	none	19.0
AD-2	6	90	2	none	none	20.0
AD-3	6	45	2	none	none	29.0
AD-4	6	45	2	2.13	3.00	29.1
AD-5	4	90	3	1.25	2.00	35.0
AD-6	4	45	3	1.25	1.50	42.0
AD-7	4	45	4	none	none	42.5
AD-8	4	45	4	1.50	1.75	43.25
AD-9	4	45	3	none	none	38.0
AD-10	4	90	3	1.00	1.50	33.0
AD-11	4	60	3	none	none	31.0
AD-12	4	60	3	1.00	1.50	39.0

TABLE 4.2
COMPARISON OF OBSERVED AND
THEORETICAL ULTIMATE MOMENTS

BEAM NO.	FAILURE TYPE	FAILURE LOAD kips	FAILURE MOMENT in kips	THEOR. ULTIMATE MOMENT		
				ACI CODE in kips	STRAIN COMPAT. 275 ksi in kips	MOMENT in kips
AD-1	SHEAR	19.0	1368.0	1650.0	1896.2	1900.2
AD-2	SHEAR	20.0	1440.0	1650.0	1910.9	1900.2
AD-3	SHEAR-COMP	29.0	2088.0	1650.0	1904.2	1900.2
AD-4	FLEXURE	29.1	2095.2	1650.0	1900.1	1900.2
AD-5	SHEAR	35.0	1680.0	1650.0	1900.6	1900.2
AD-6	FLEXURE	42.0	2016.0	1650.0	1915.7	1900.2
AD-7	SHEAR-COMP	42.5	2040.0	1650.0	1904.5	1900.2
AD-8	FLEXURE	43.25	2076.0	1650.0	1900.2	1900.2
AD-9	SHEAR-COMP	38.0	1824.0	1650.0	1903.8	1900.2
AD-10	SHEAR	33.0	1584.0	1650.0	1906.4	1900.2
AD-11	SHEAR	31.0	1488.0	1650.0	1897.8	1900.2
AD-12	SHEAR-COMP	39.0	1872.0	1650.0	1911.9	1900.2

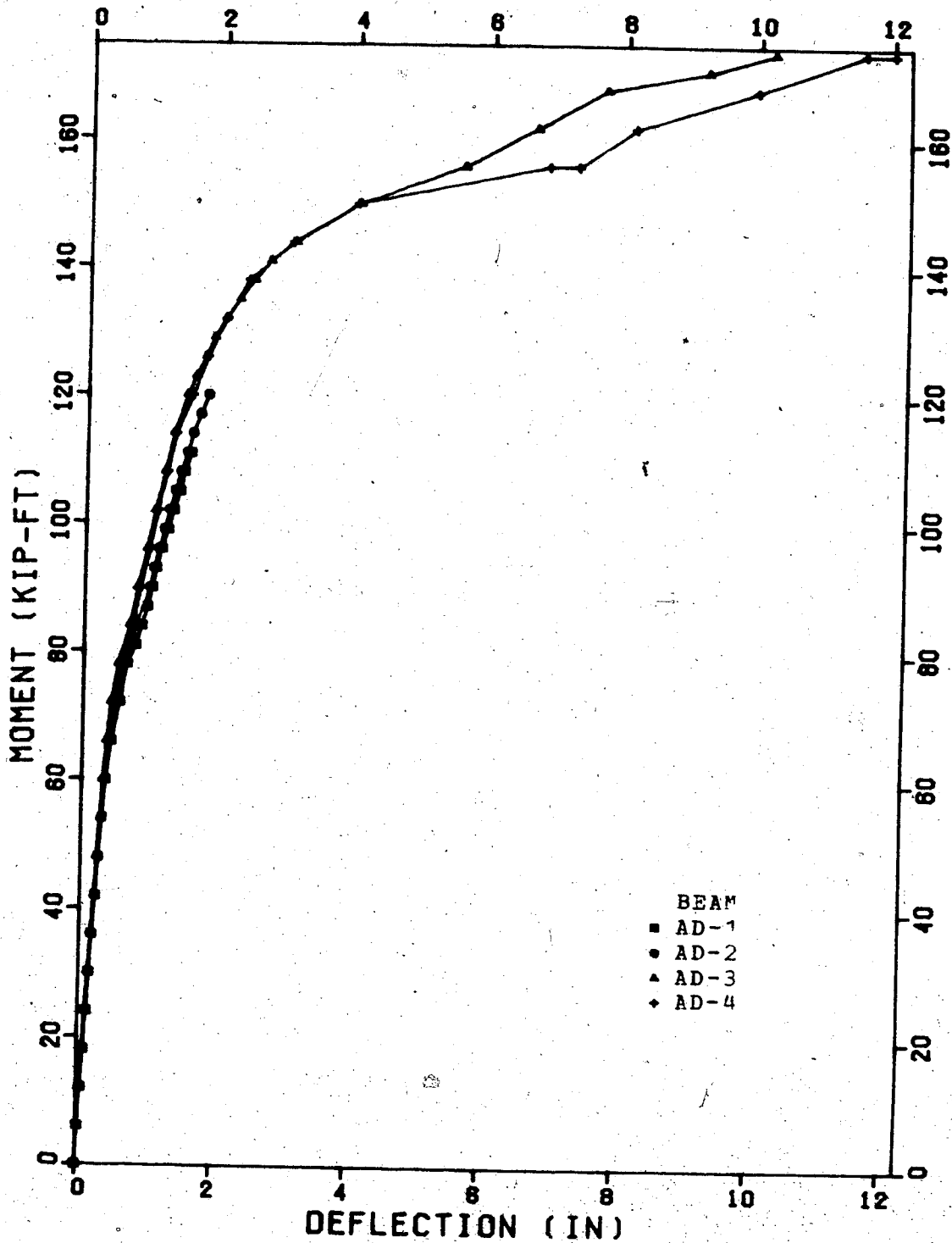


FIGURE 4.1.1 CASTING GROUP ONE
APPLIED MOMENT VS. CENTERLINE DEFLECTION

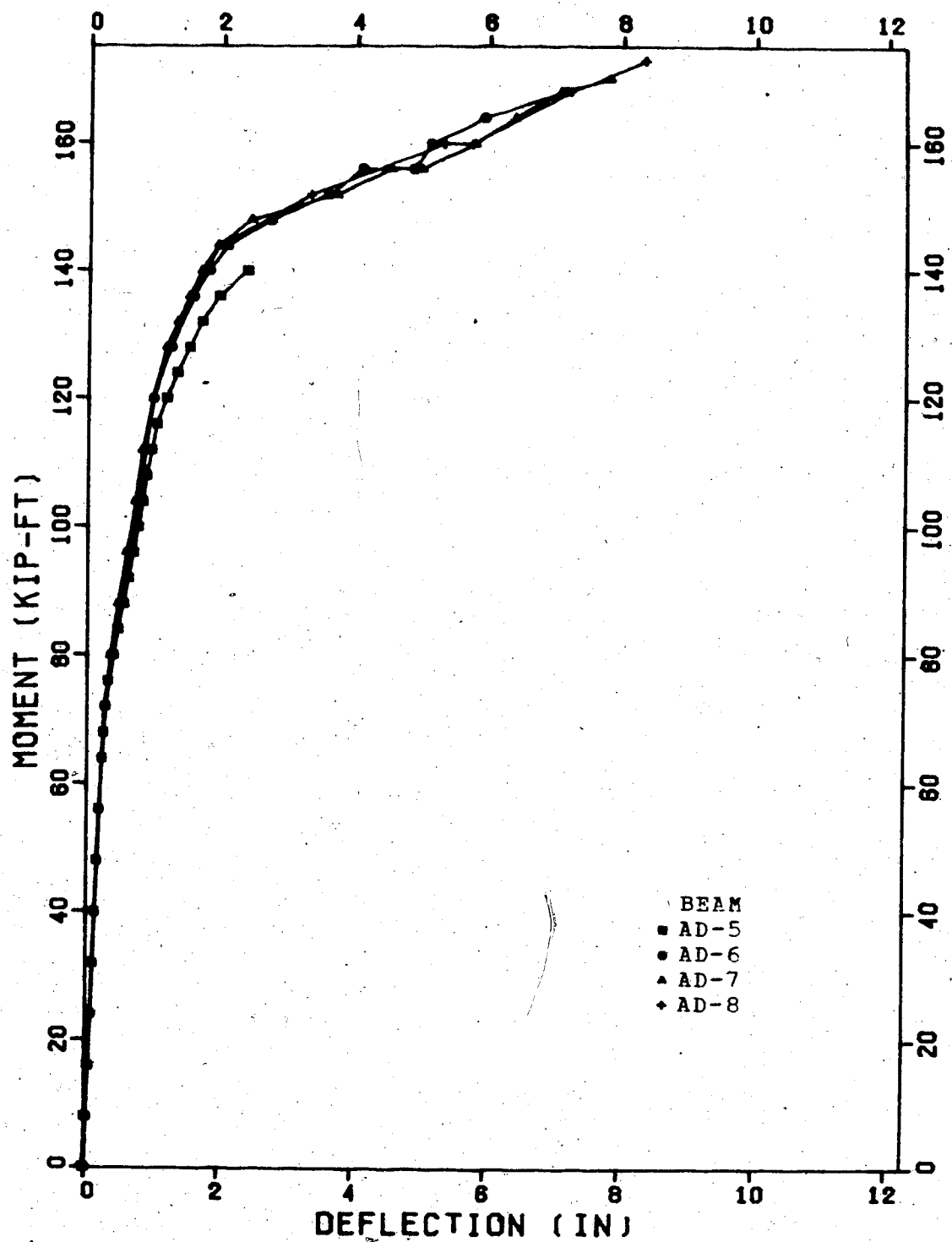


FIGURE 4.1.2 CASTING GROUP TWO
APPLIED MOMENT VS. CENTERLINE DEFLECTION

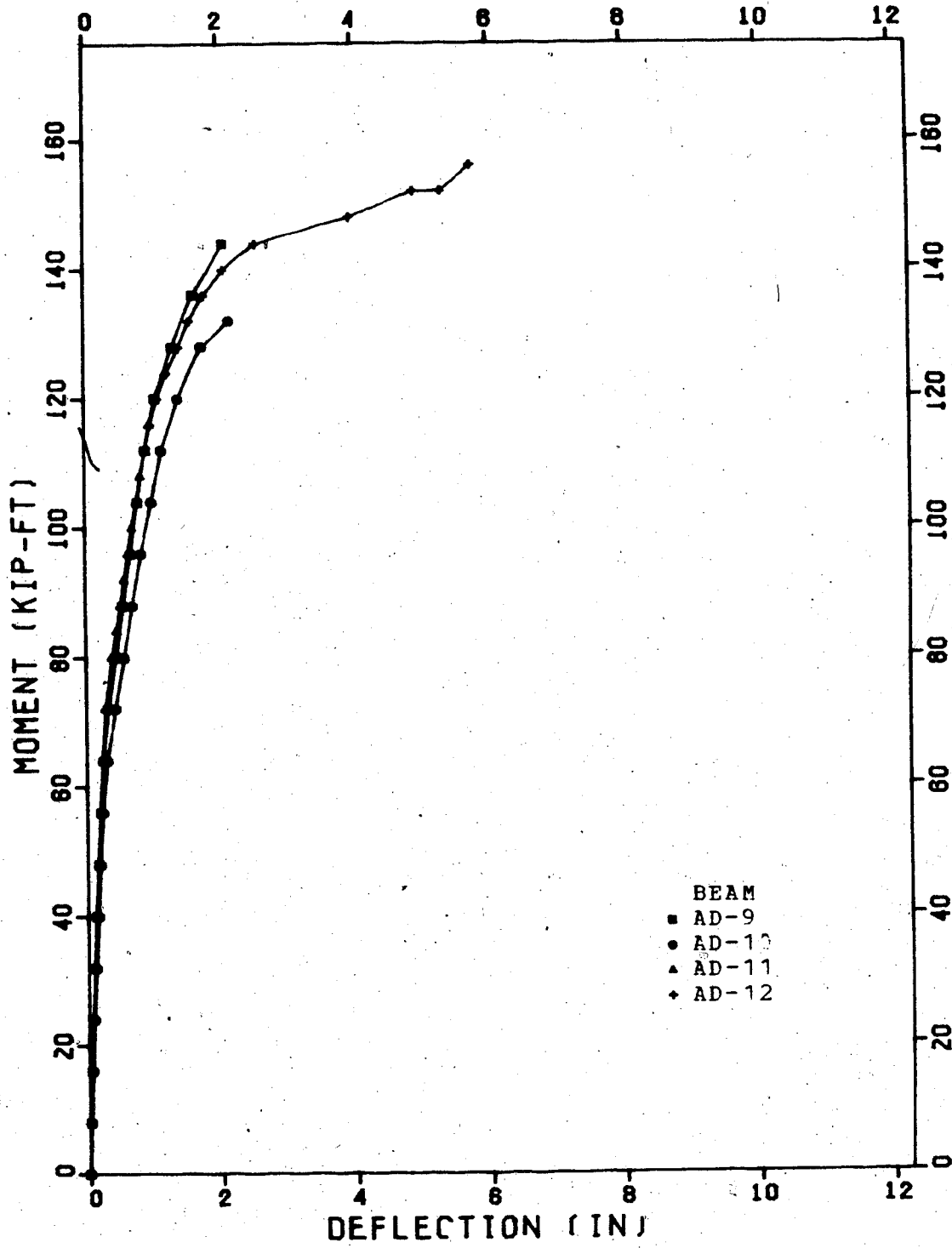


FIGURE 4.1.3 CASTING GROUP THREE
APPLIED MOMENT VS. CENTERLINE DEFLECTION

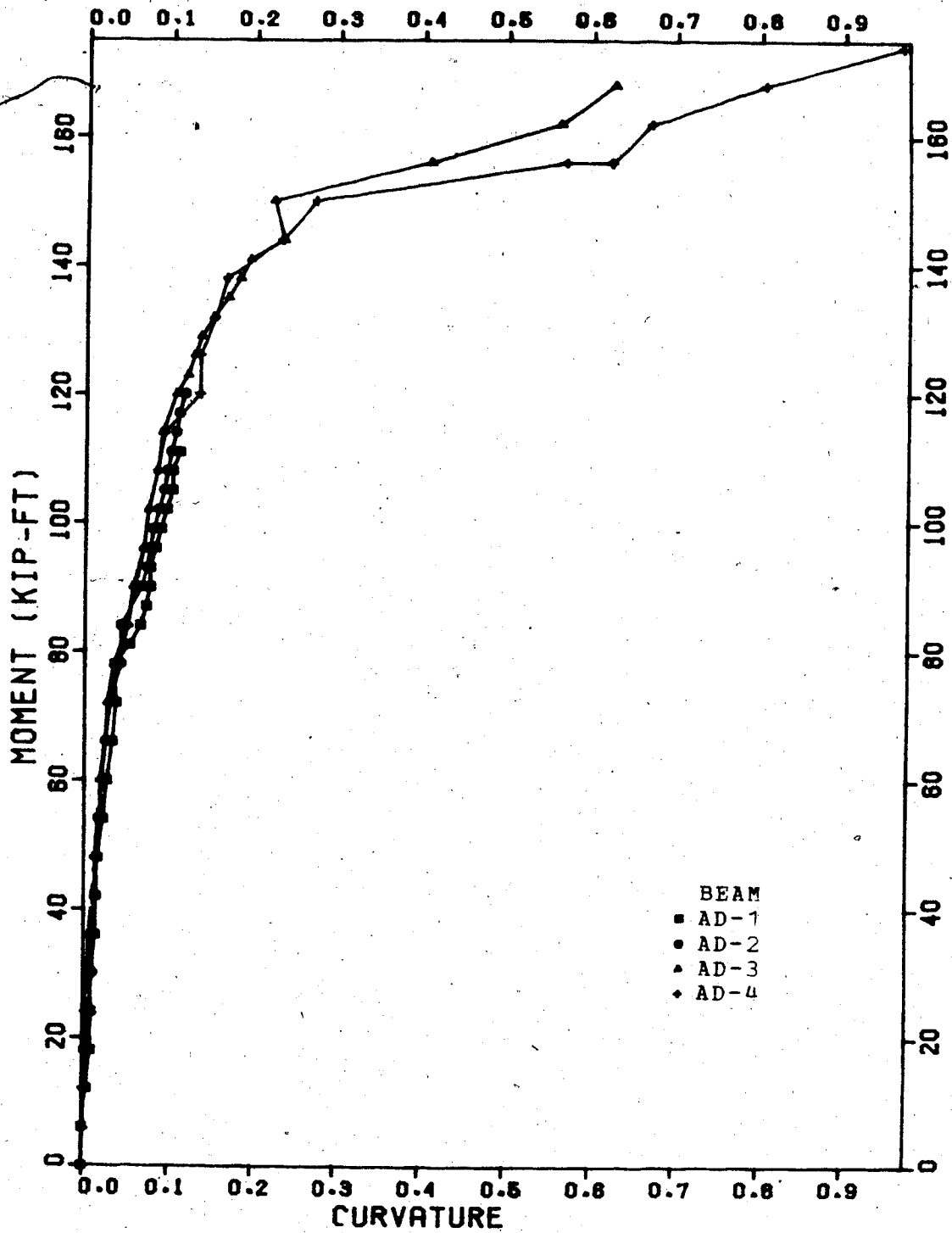


FIGURE 4.2.1 CASTING GROUP ONE

APPLIED MOMENT VS. AVERAGE CURVATURE ($\text{in}^{-1} \cdot 10^{-3}$)

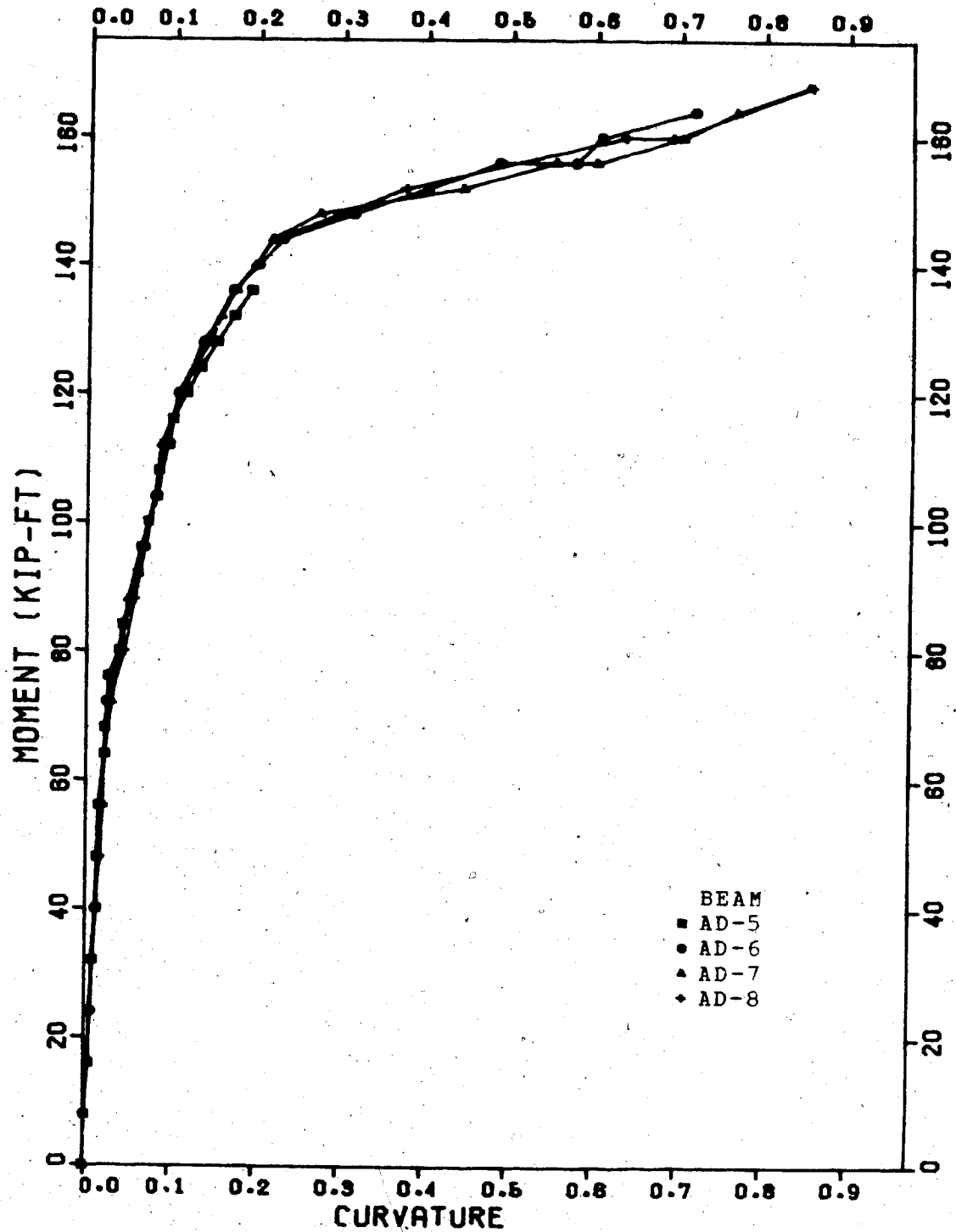


FIGURE 4.2.2 CASTING GROUP TWO

APPLIED MOMENT VS. AVERAGE CURVATURE (in⁻¹*10⁻³)

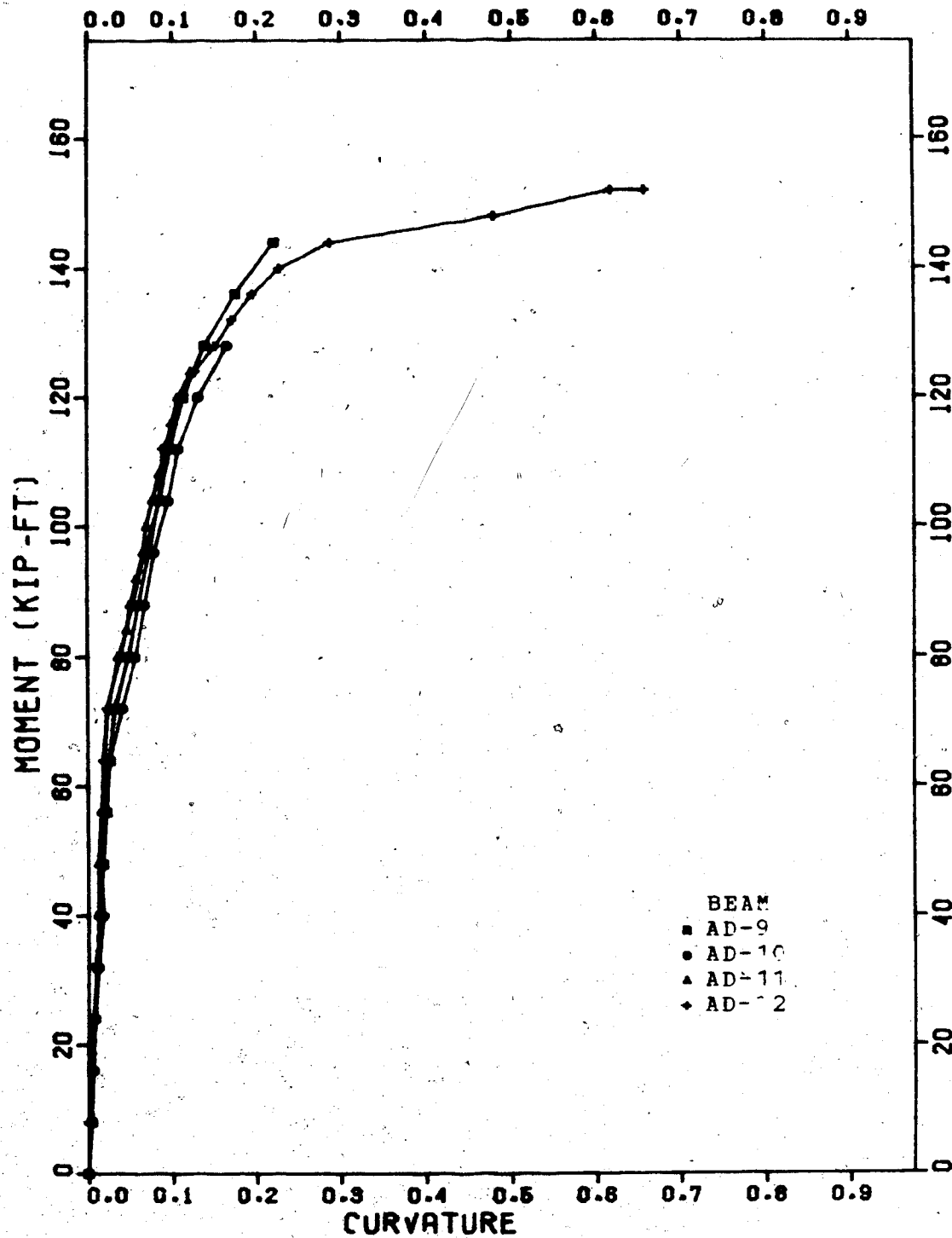


FIGURE 4.2.3 CASTING GROUP THREE
 APPLIED MOMENT VS. AVERAGE CURVATURE ($\text{in}^{-1} \cdot 10^{-3}$)

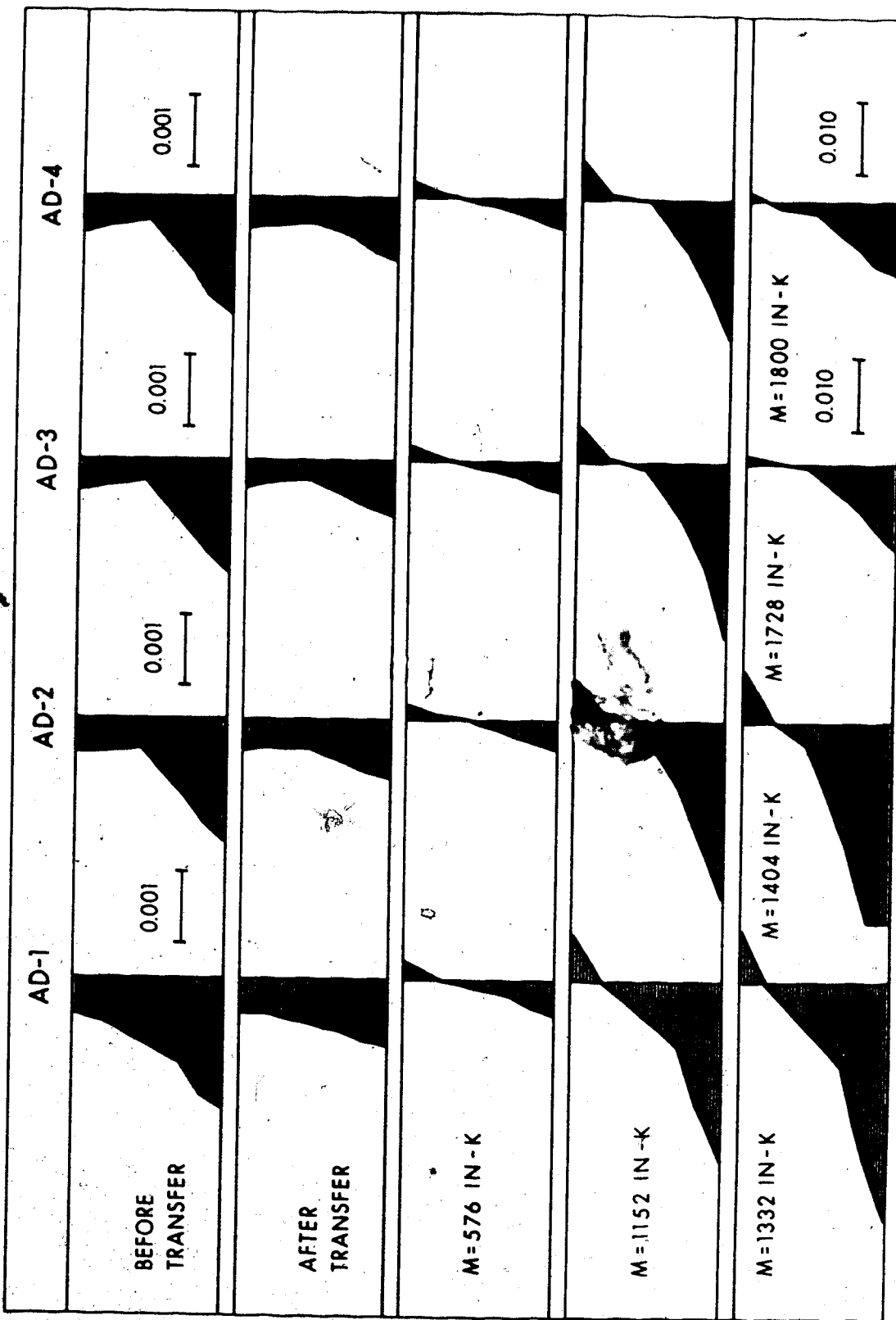


FIGURE 4.3.1 CASTING GROUP ONE STRAIN DISTRIBUTION

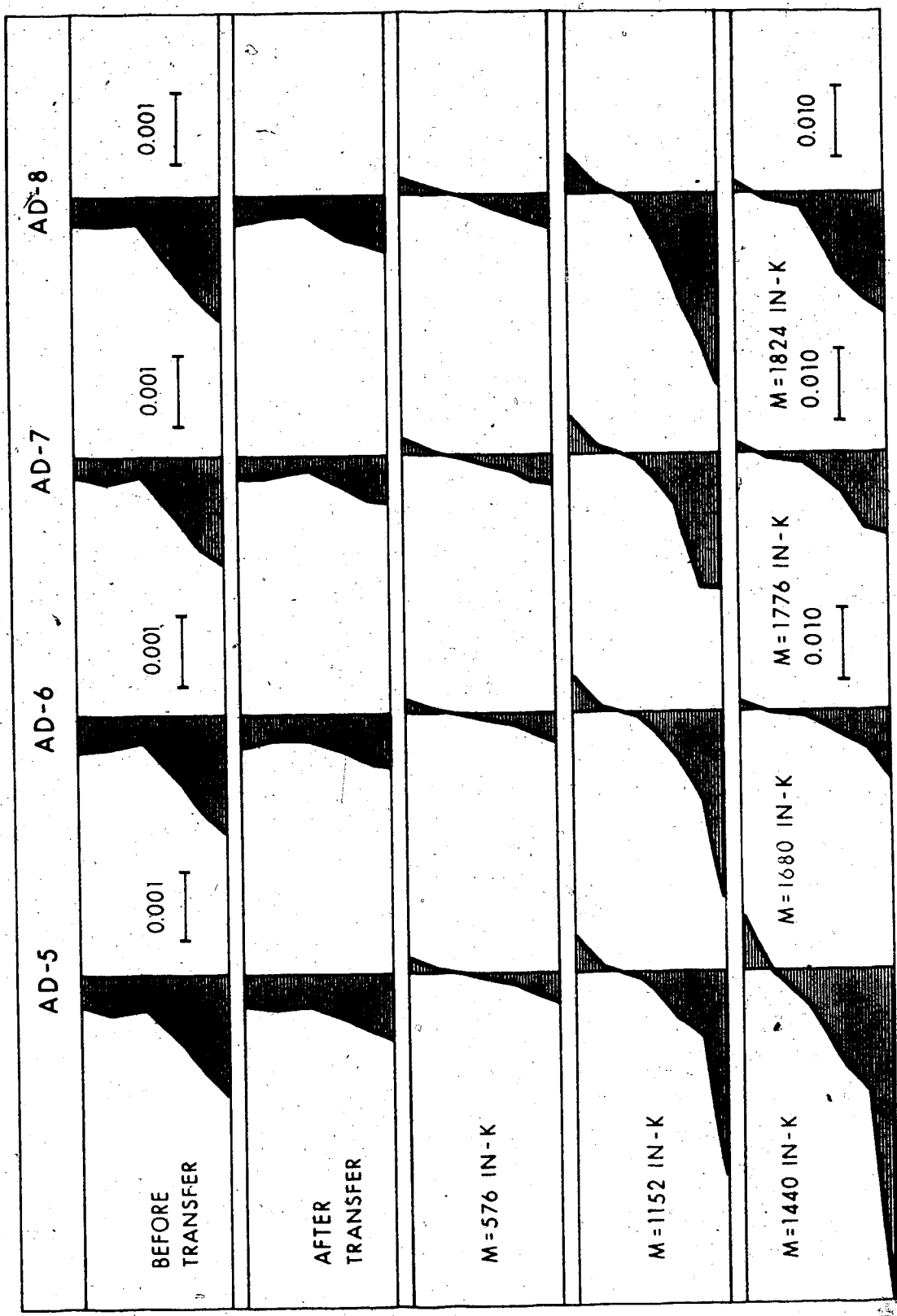


FIGURE 4.3.2 CASTING GROUP TWO STRAIN DISTRIBUTION

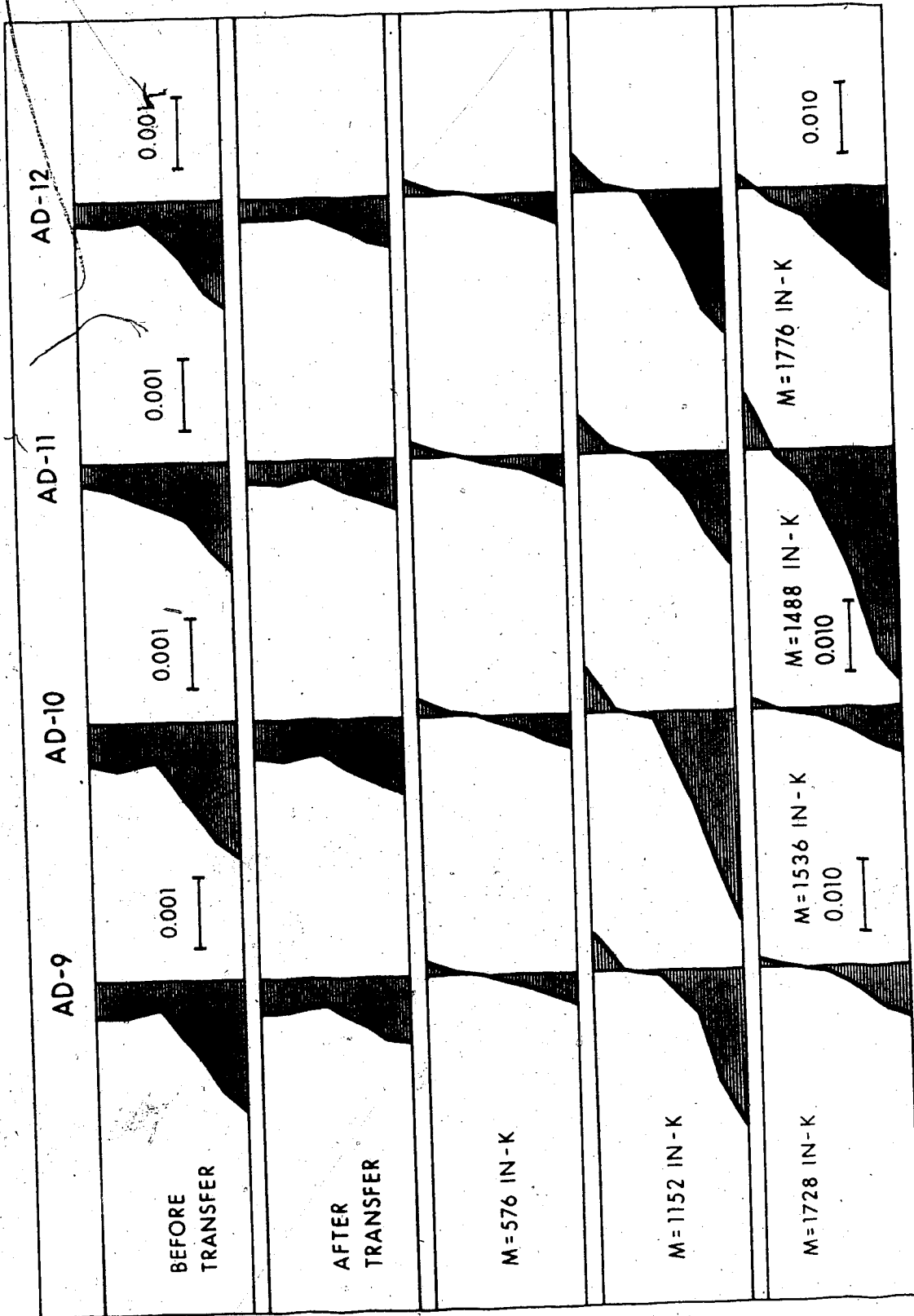


FIGURE 4.3.3 CASTING GROUP THREE STRAIN DISTRIBUTION

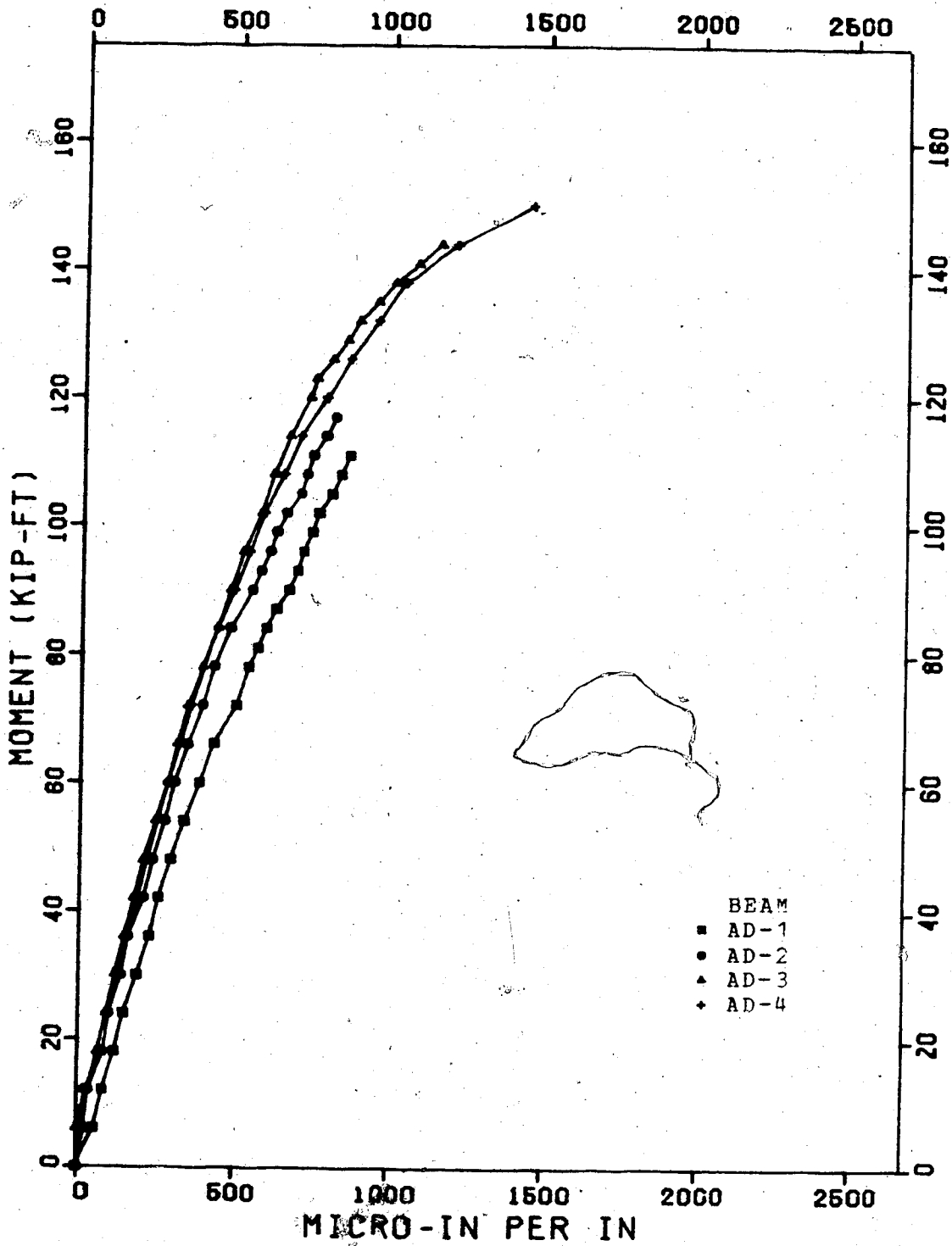


FIGURE 4.4.1 CASTING GROUP ONE

APPLIED MOMENT VS. STRAIN ON TOP SURFACE OF FLANGE

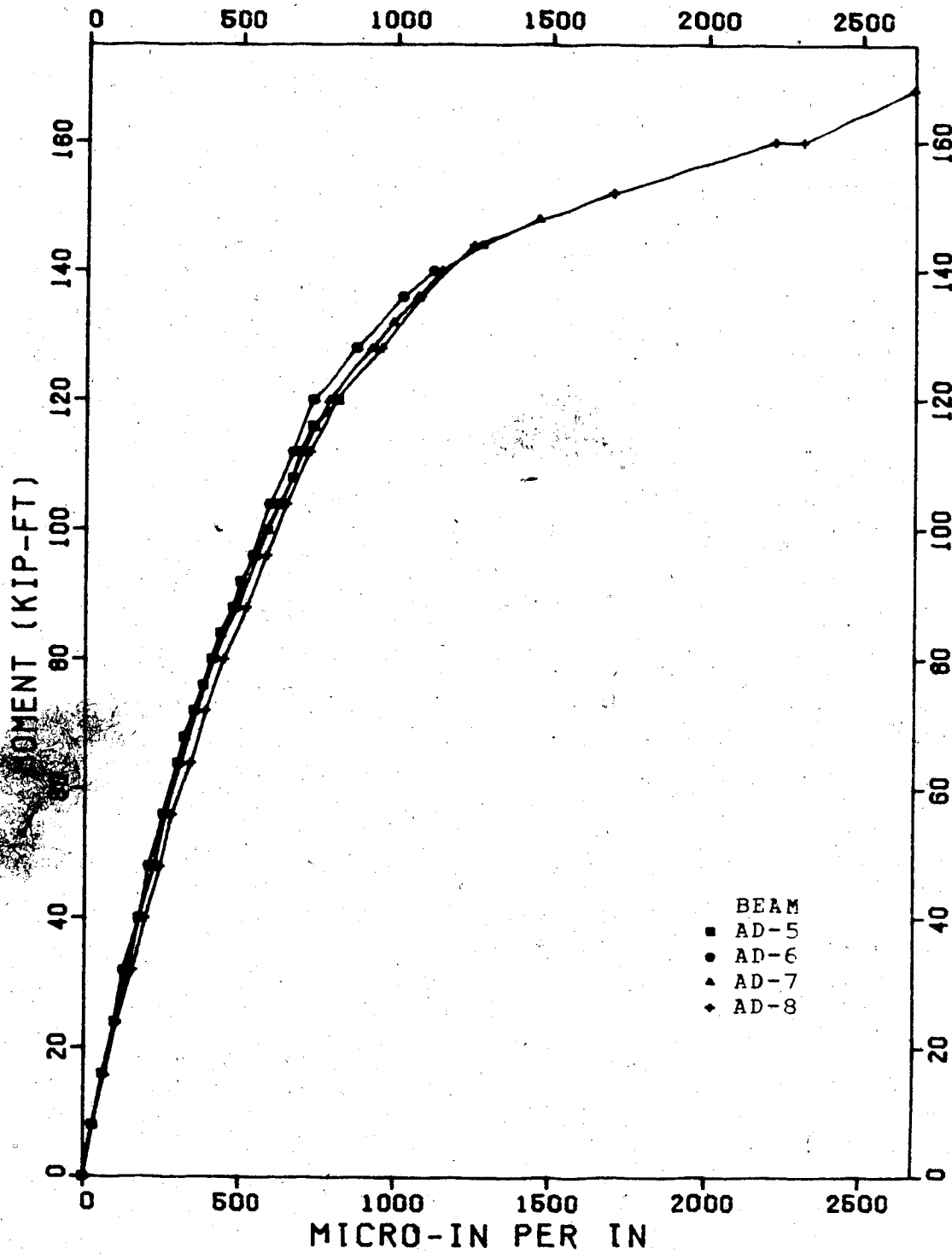


FIGURE 4.4.2 CASTING GROUP TWO

APPLIED MOMENT VS. STRAIN ON TOP SURFACE OF FLANGE

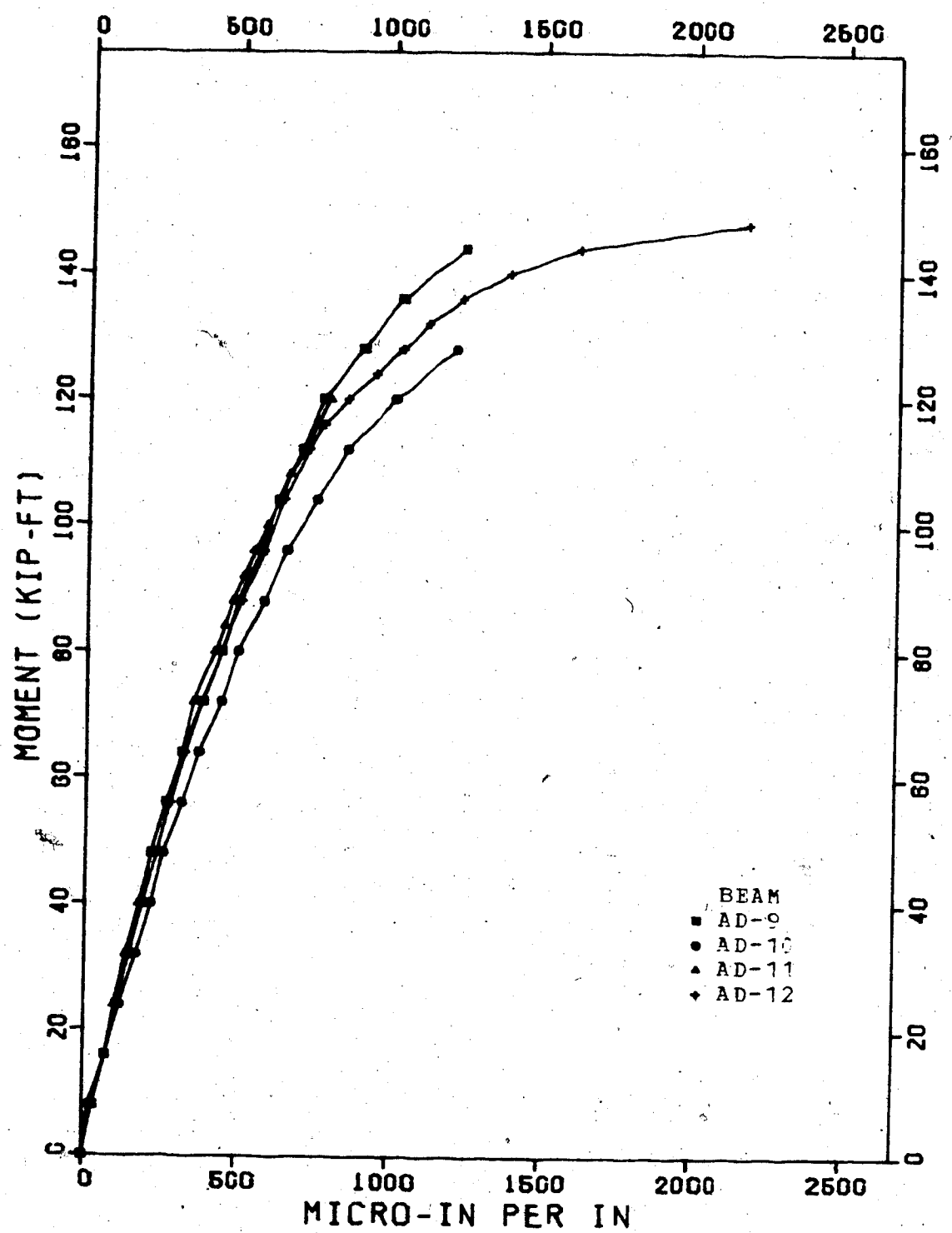
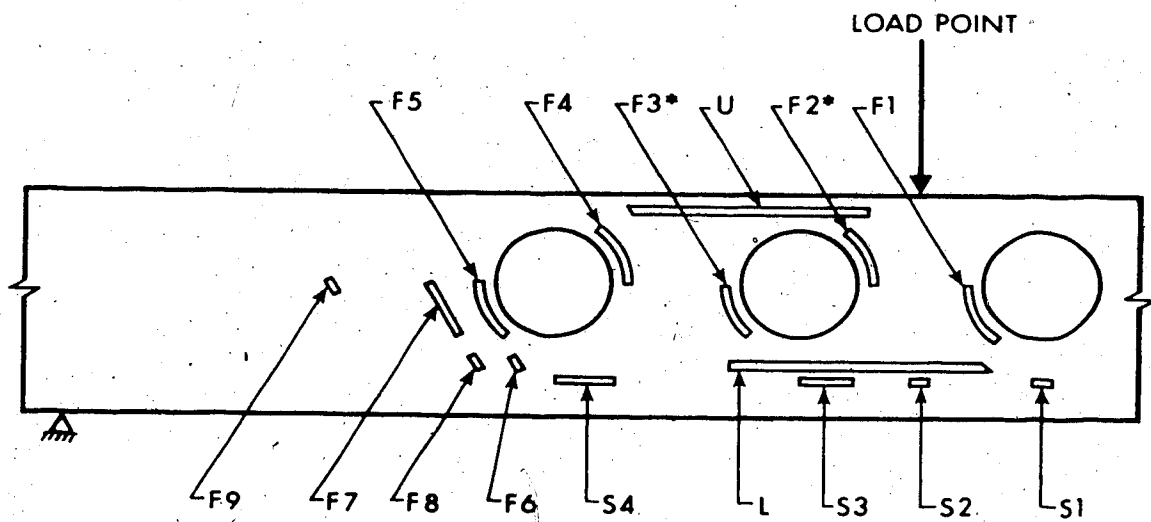


FIGURE 4.4.3 CASTING GROUP THREE

APPLIED MOMENT VS. STRAIN ON TOP SURFACE OF FLANGE



* CASTING GROUP #1 ONLY

S1 to S4 STRAIN GAUGES ON PRESTRESSING STRAND

F1 to F9 STRAIN GAUGES ON FULL-DEPTH #3 STIRRUPS

U STRAIN GAUGES ON UPPER WEB #2 STIRRUPS

L STRAIN GAUGES ON LOWER WEB #2 STIRRUPS

FIGURE 4.5

GENERAL GAUGE LOCATIONS

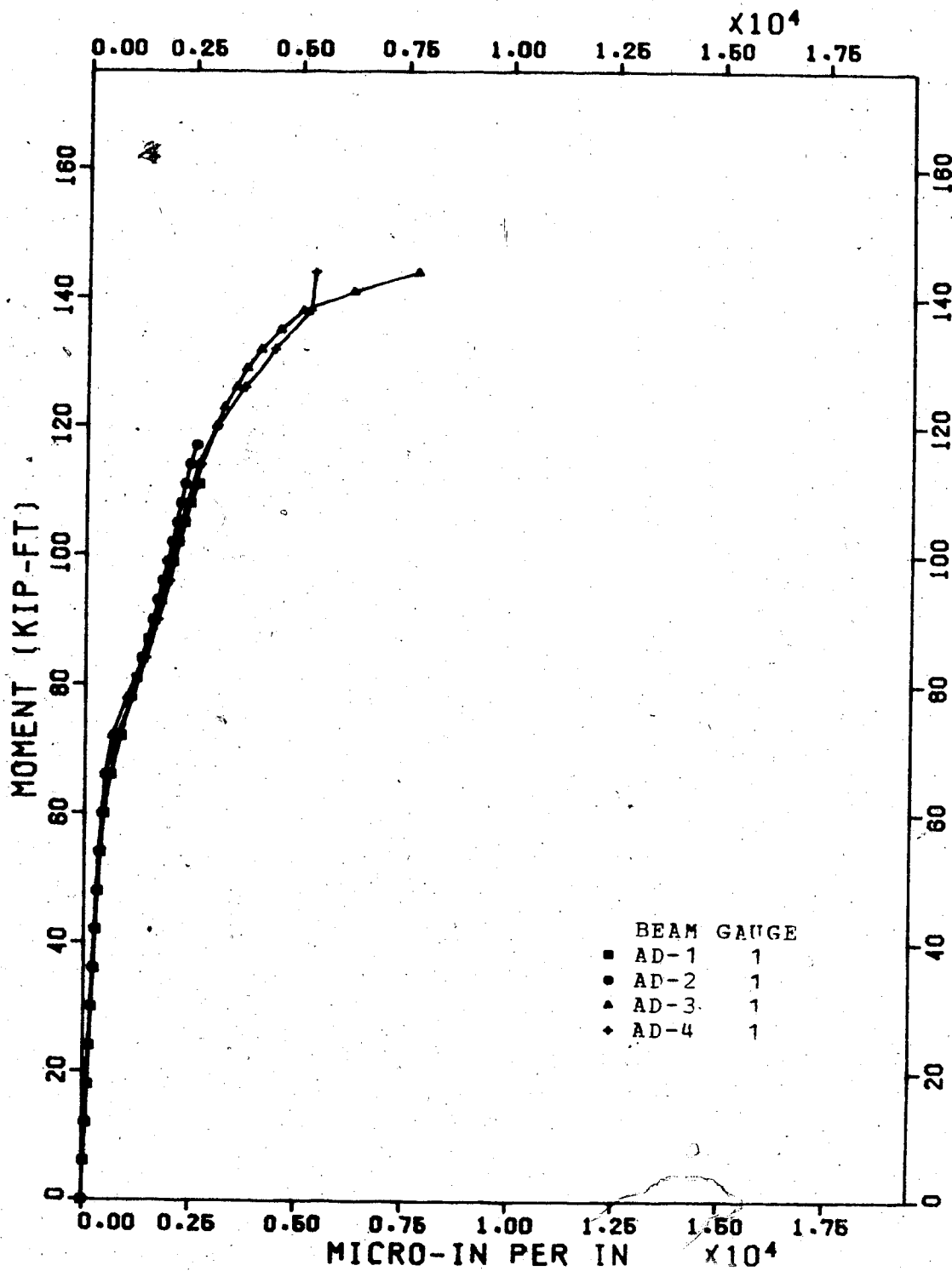


FIGURE 4.6.1 CASTING GROUP C

APPLIED MOMENT VS. STRAND STRAIN AT CENTERLINE

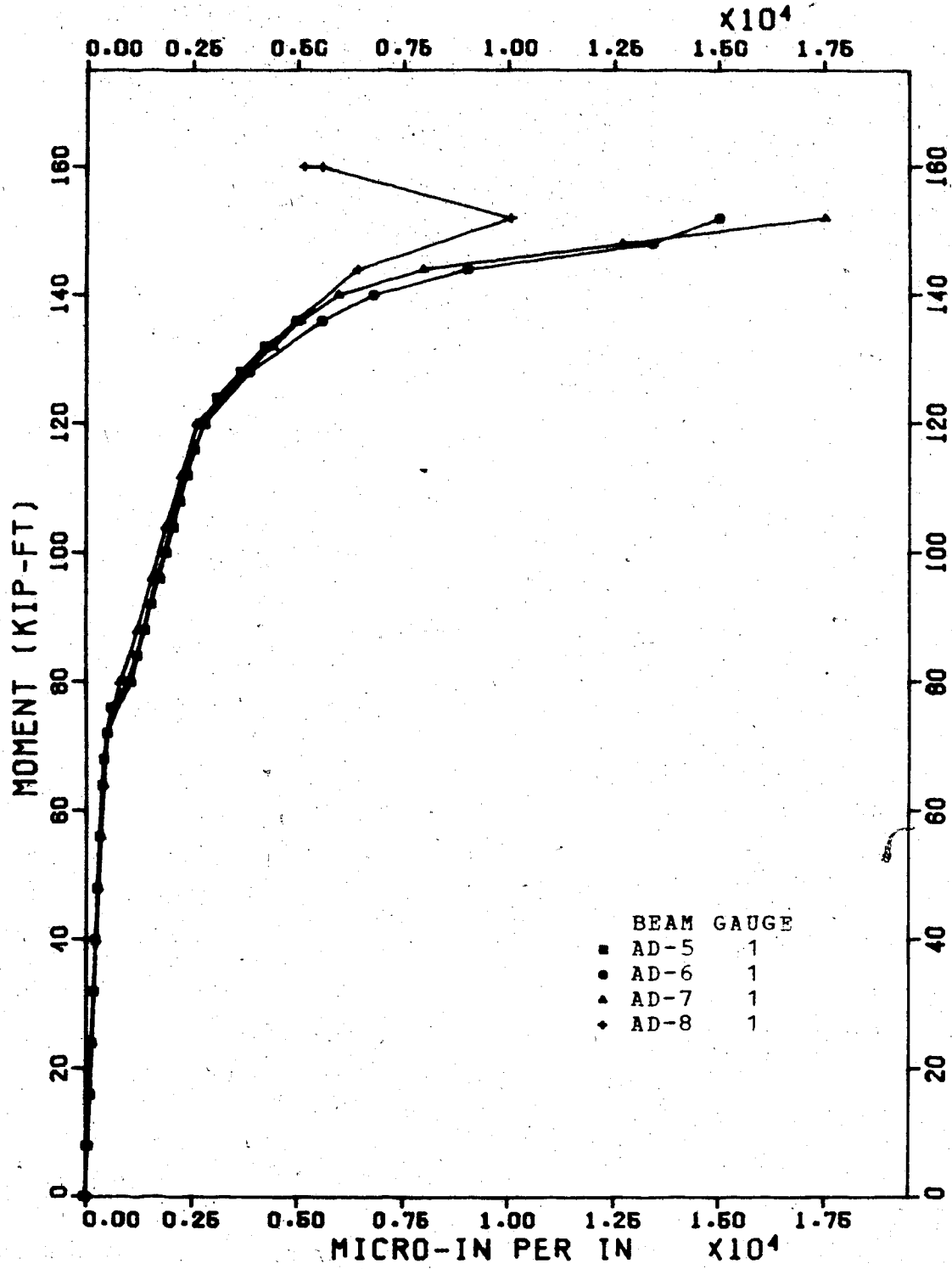


FIGURE 4.6.2 CASTING GROUP TWO
APPLIED MOMENT VS. STRAND STRAIN AT CENTERLINE

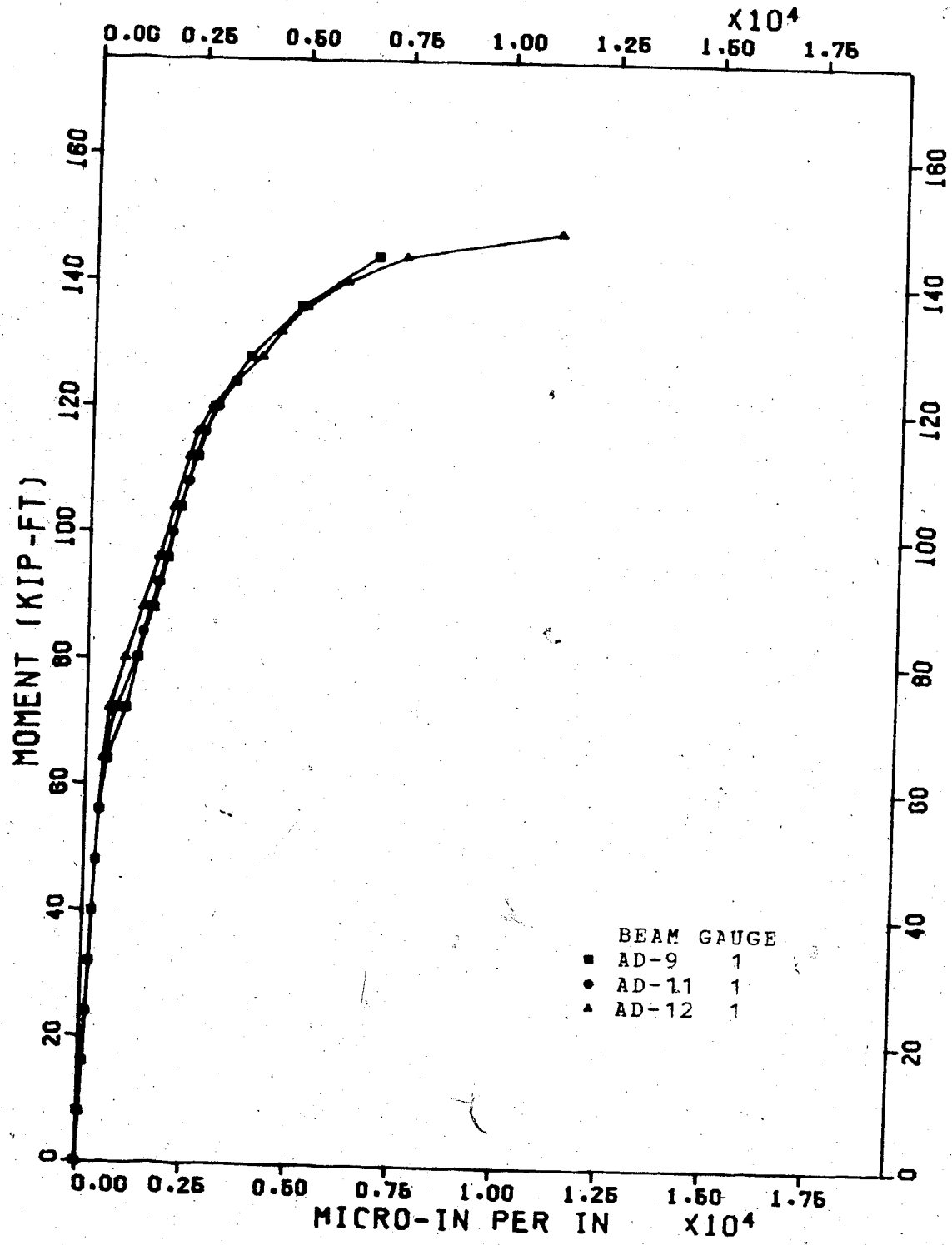


FIGURE 4.6.3 CASTING GROUP THREE
APPLIED MOMENT VS. STRAND STRAIN AT CENTERLINE

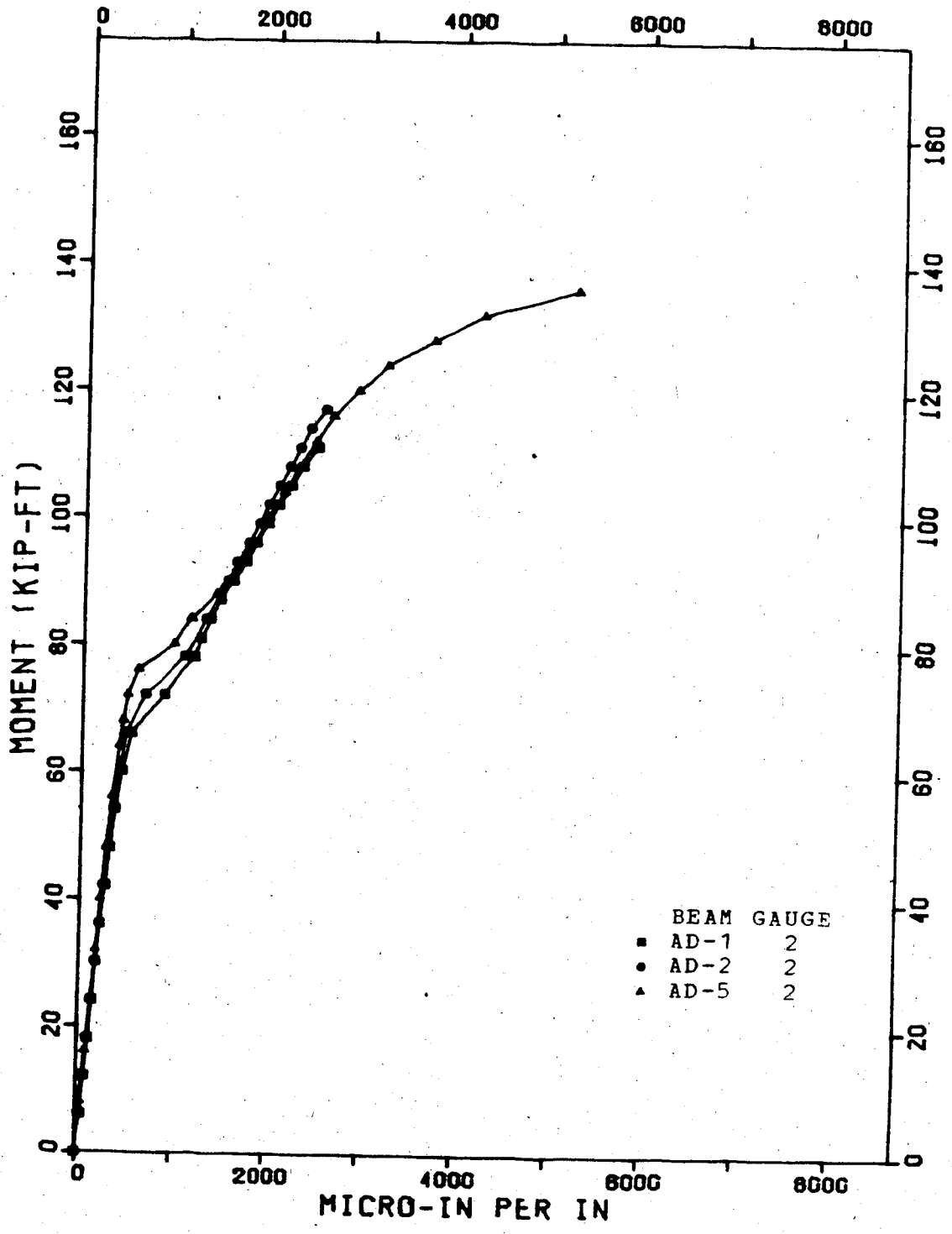


FIGURE 4.7.1

APPLIED MOMENT VS. STRAND STRAIN AT LOCATION 'S1'

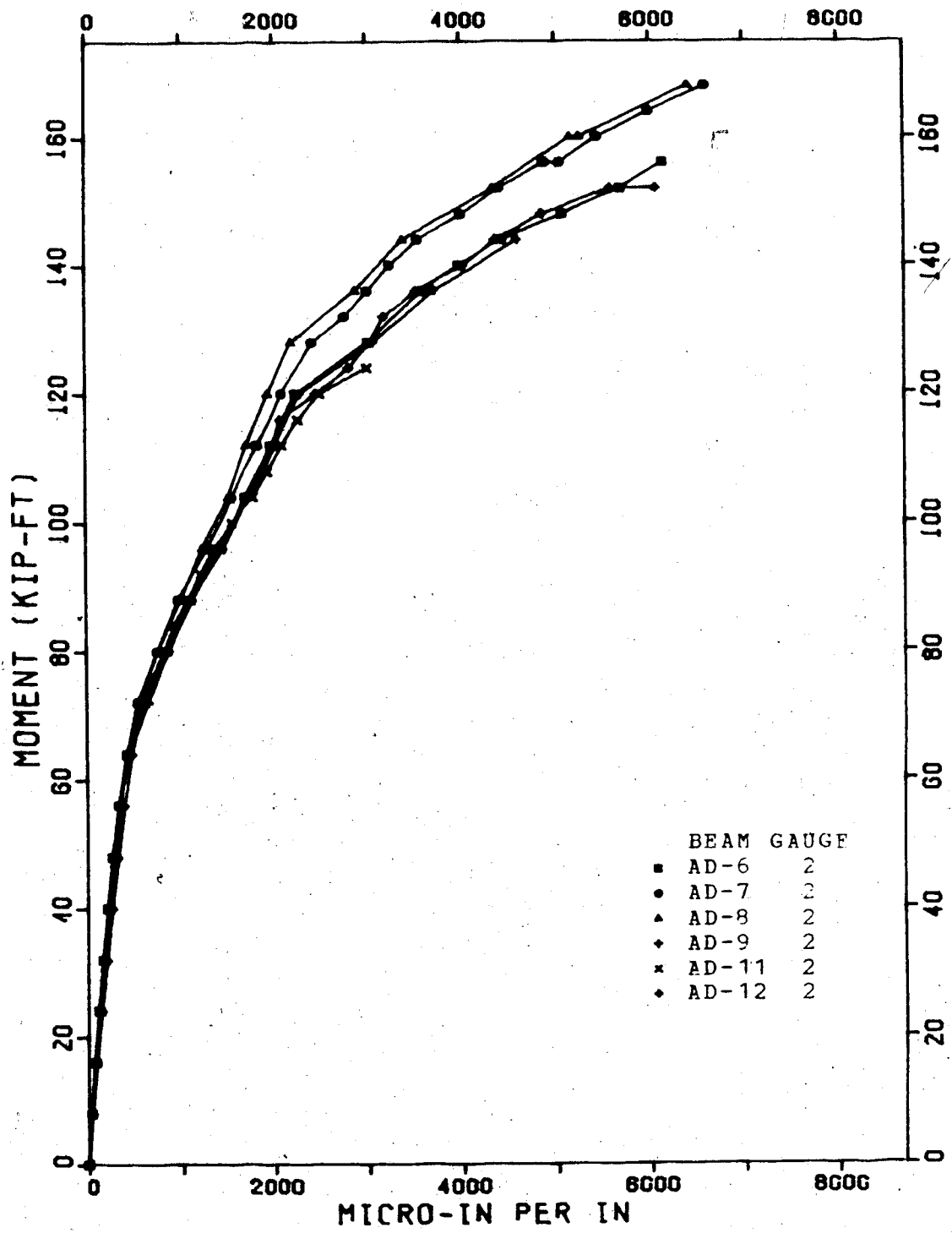


FIGURE 4.7.2

APPLIED MOMENT VS. STRAND STRAIN AT LOCATION 'S2'

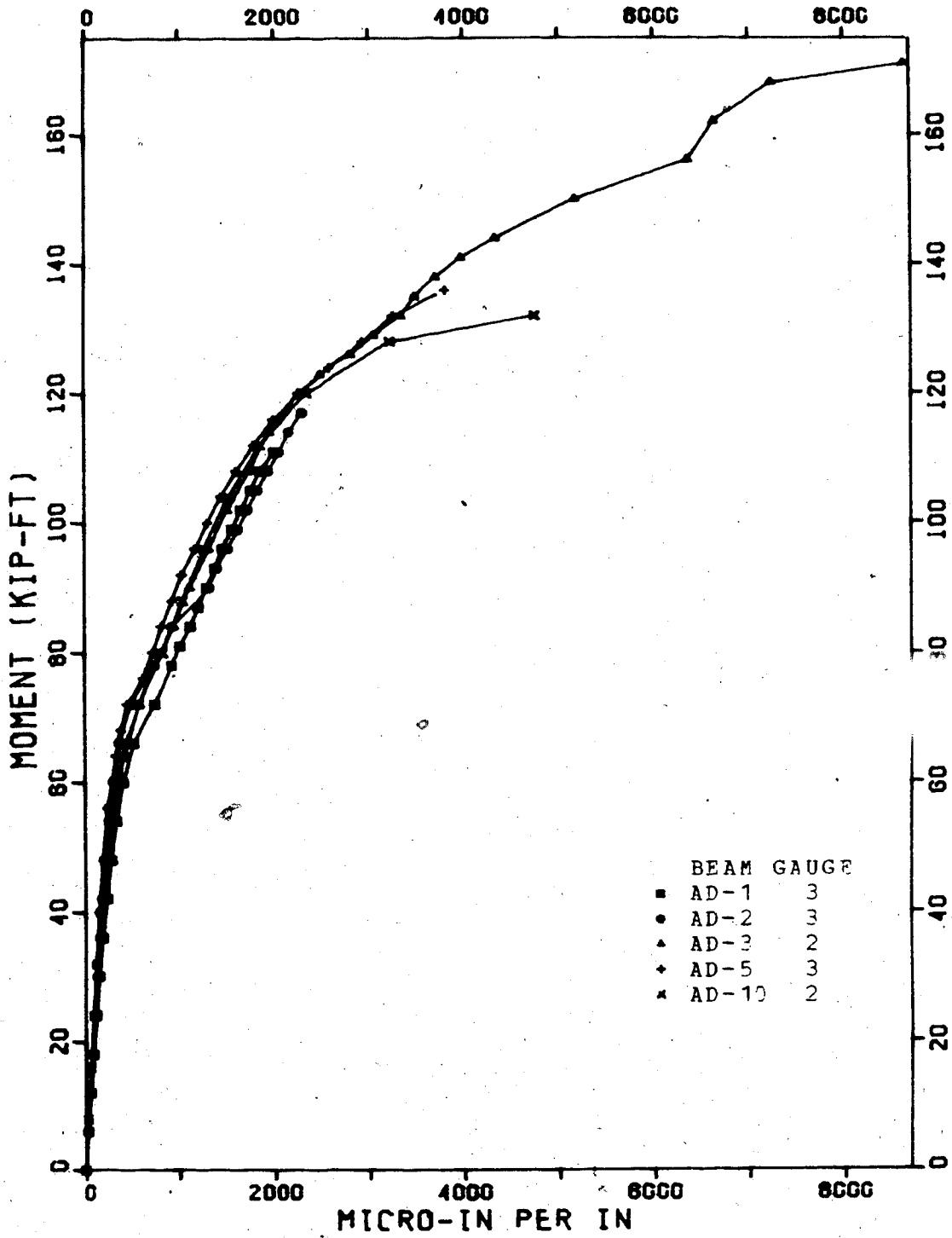


FIGURE 4.7.3

APPLIED MOMENT VS. STRAND STRAIN AT LOCATION 'S3'

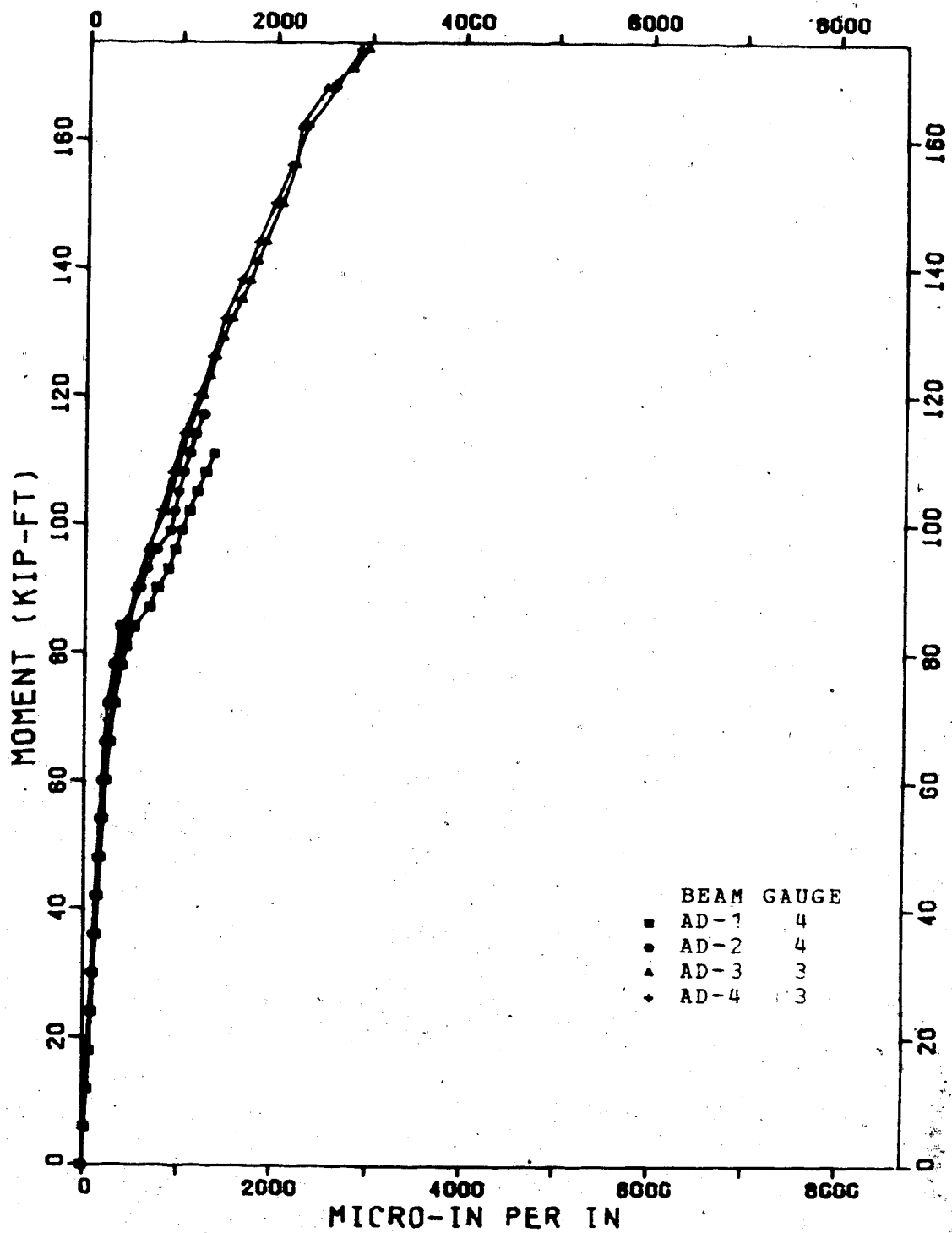


FIGURE 4.7.4

APPLIED MOMENT VS. STRAND STRAIN AT LOCATION 'S4'

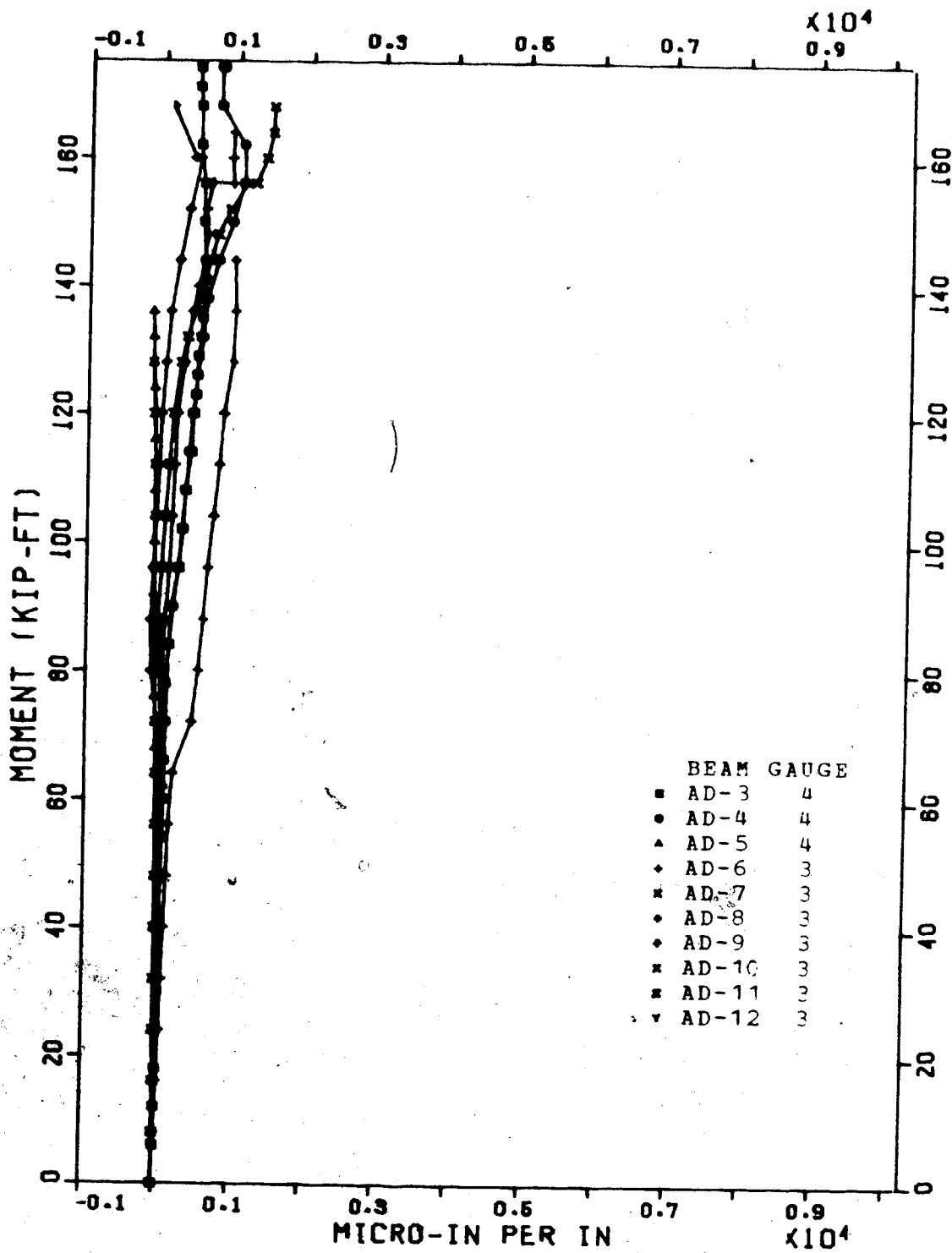


FIGURE 4.8.1

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F1'

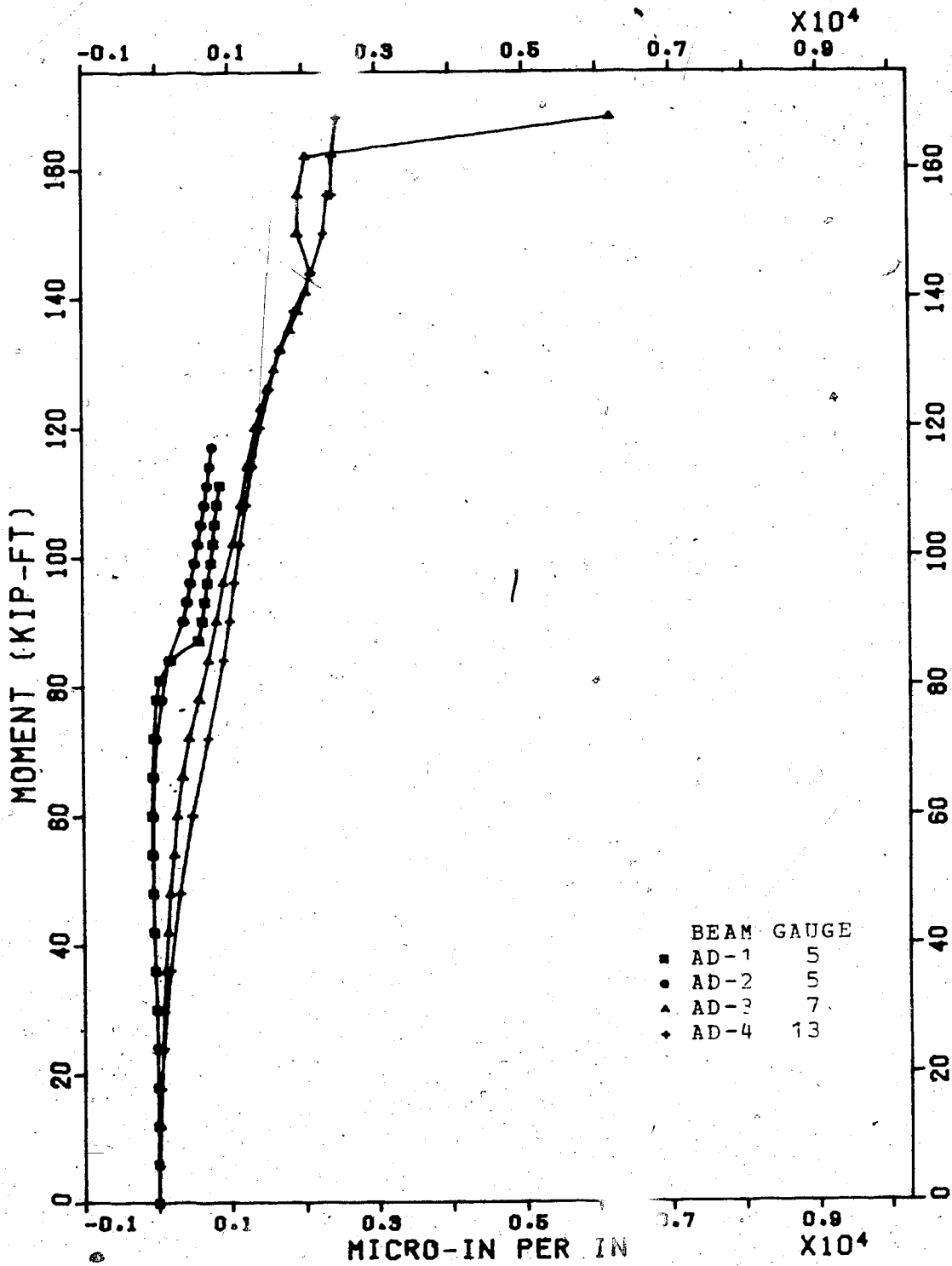


FIGURE 4.8.2

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F2'

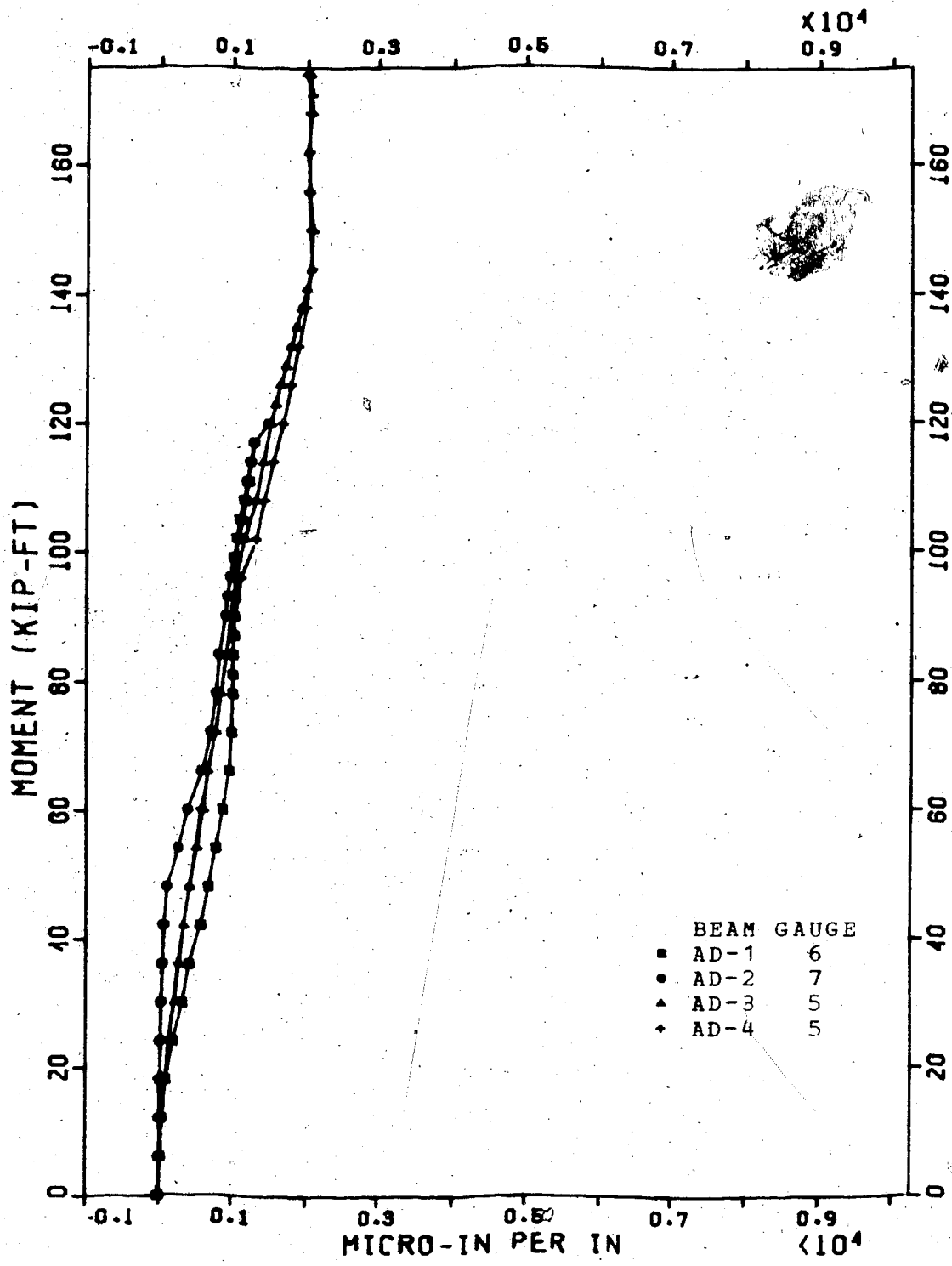


FIGURE 4.8.3

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F3'

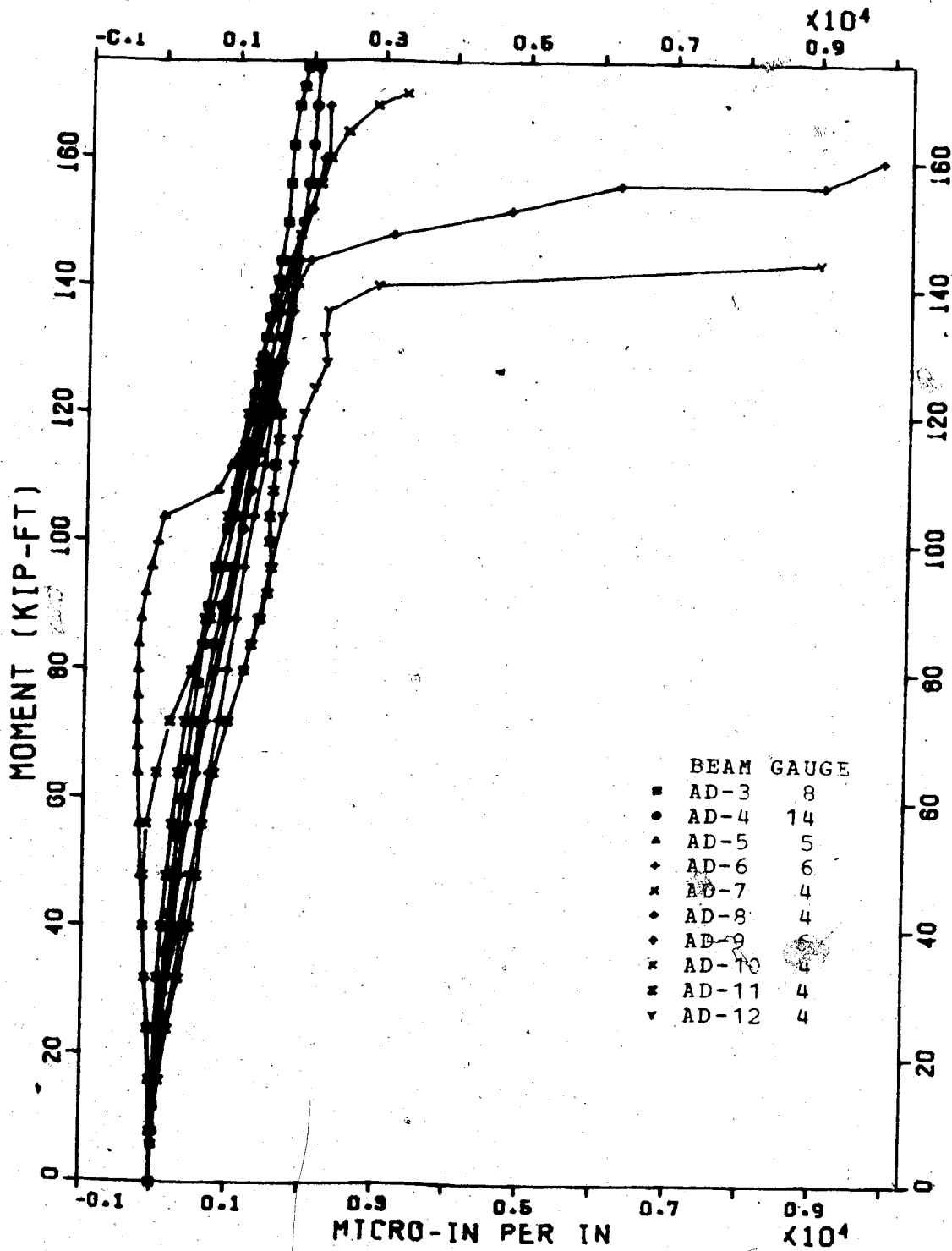


FIGURE 4.8.4

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F4'

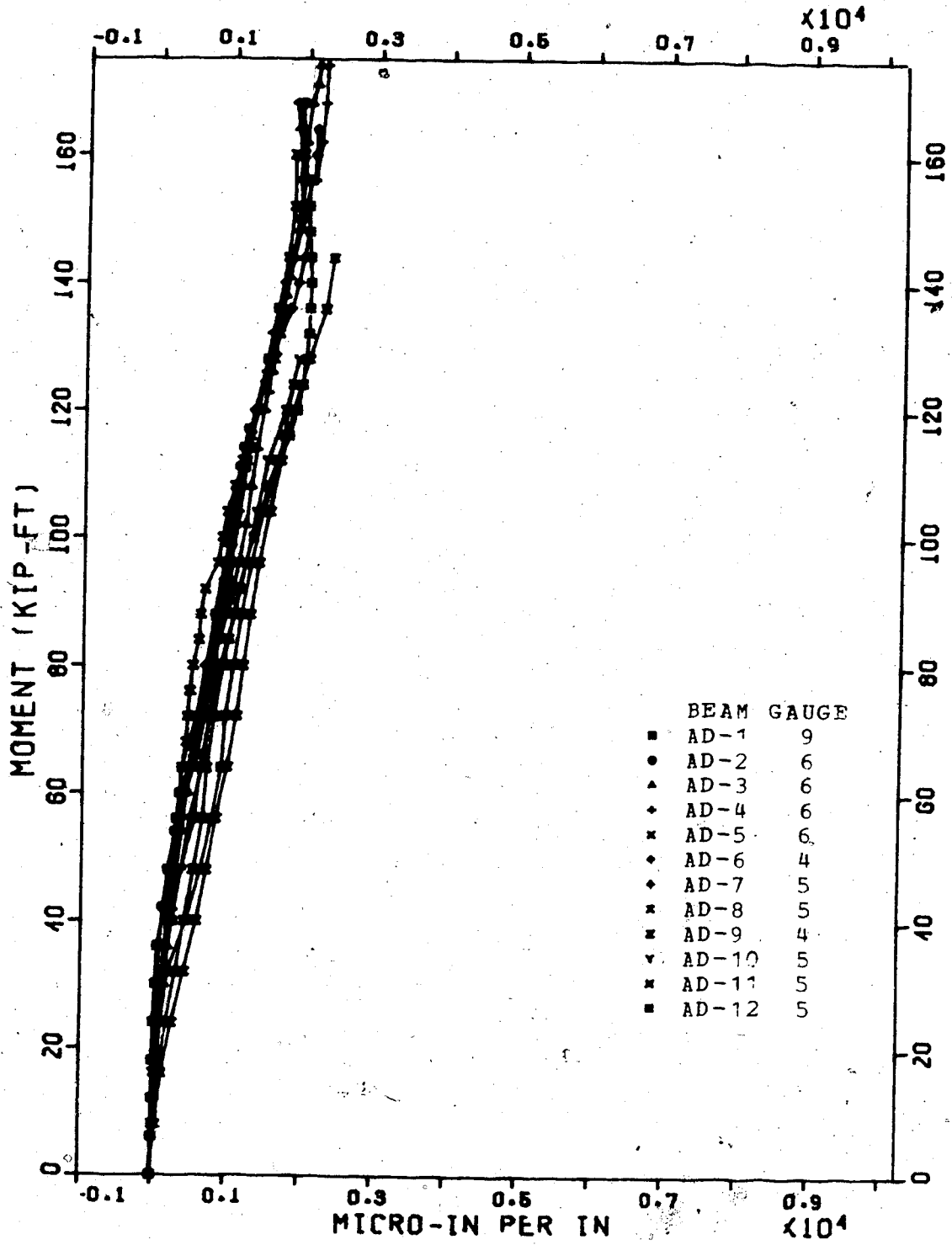


FIGURE 4.8.5

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F5'

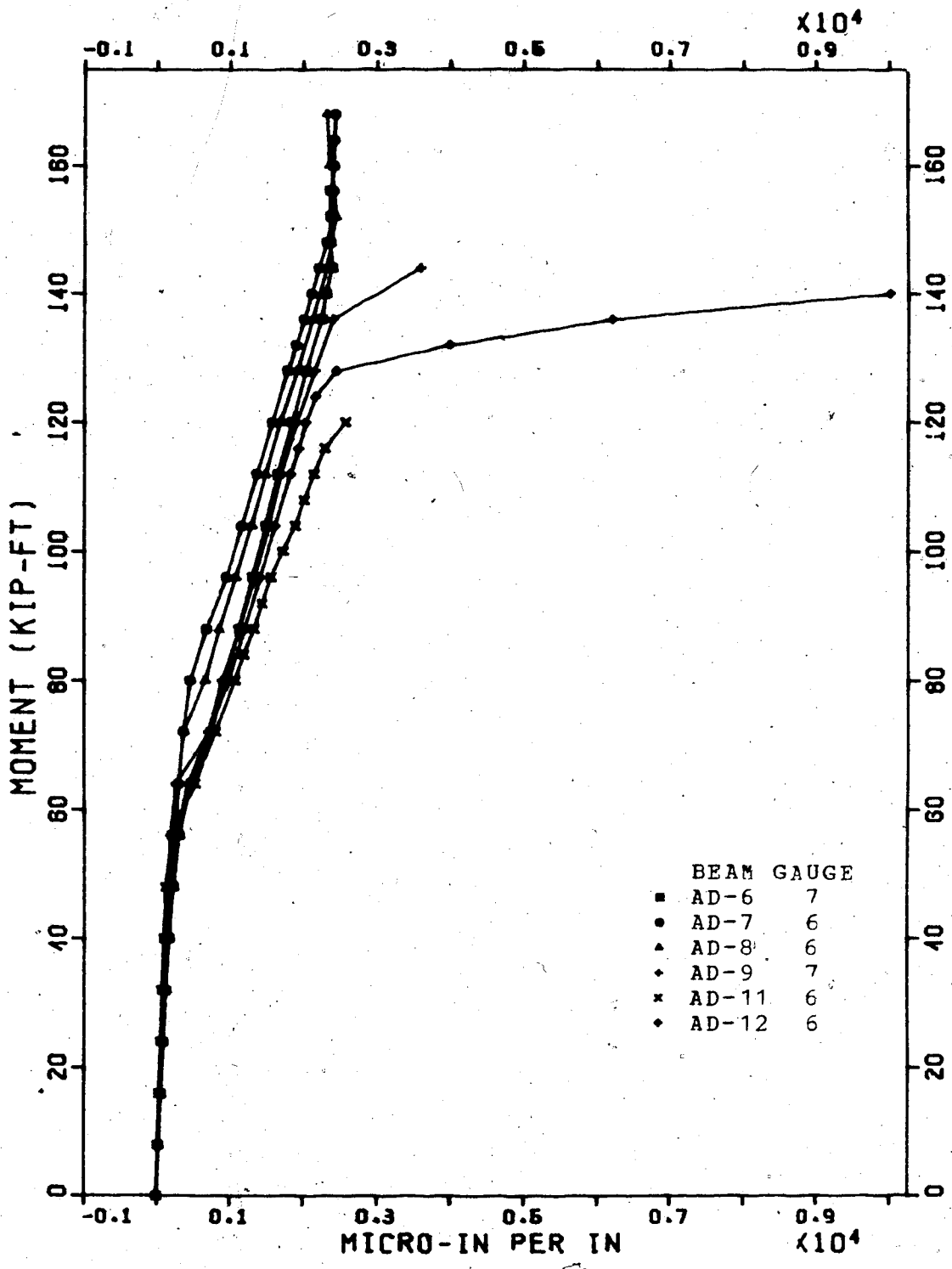


FIGURE 4.8.6

APPLIED MOMENT VS. STIRRUP TRAIN AT LOCATION 'F6'

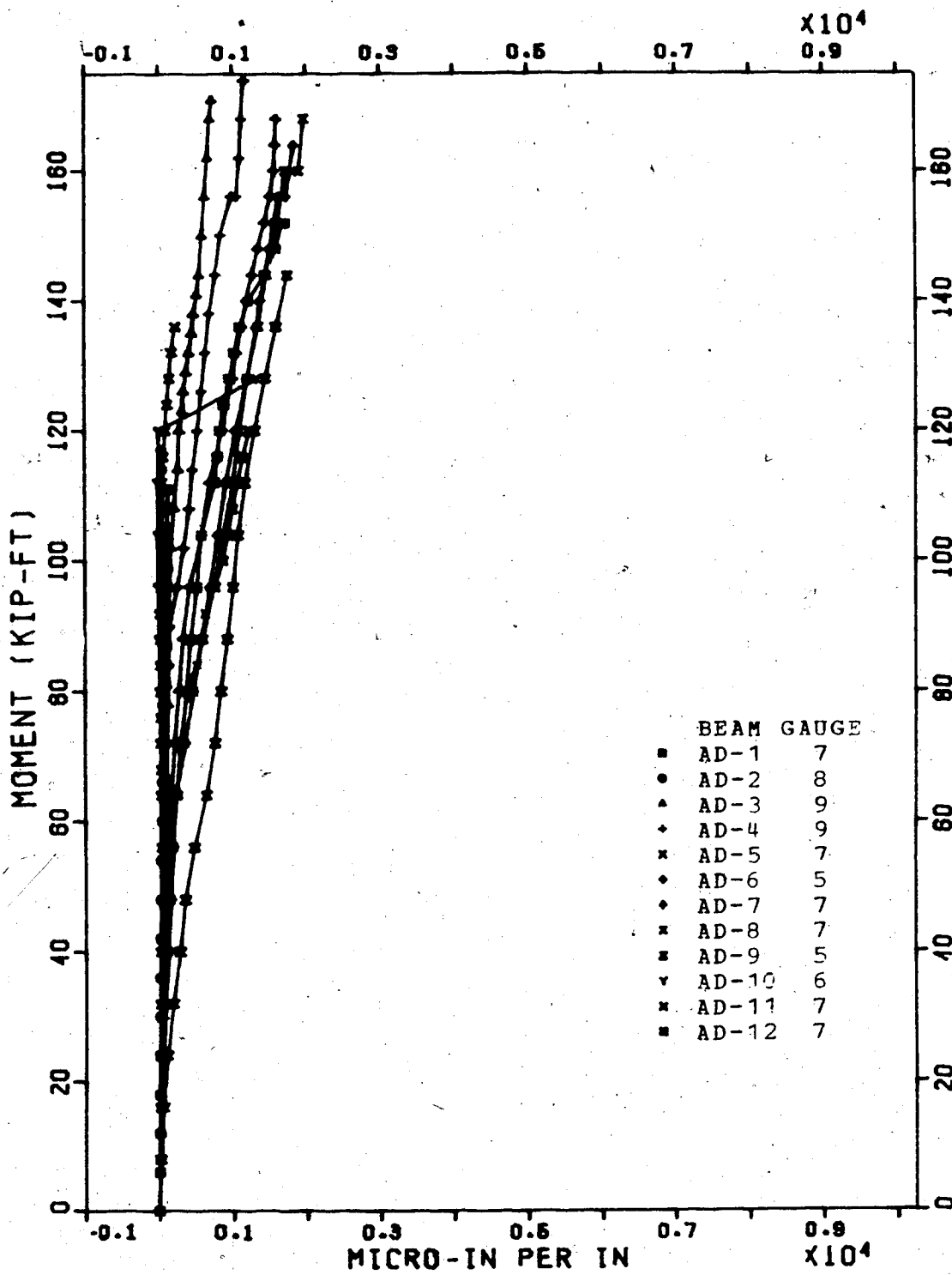


FIGURE 4.8.7

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F7'

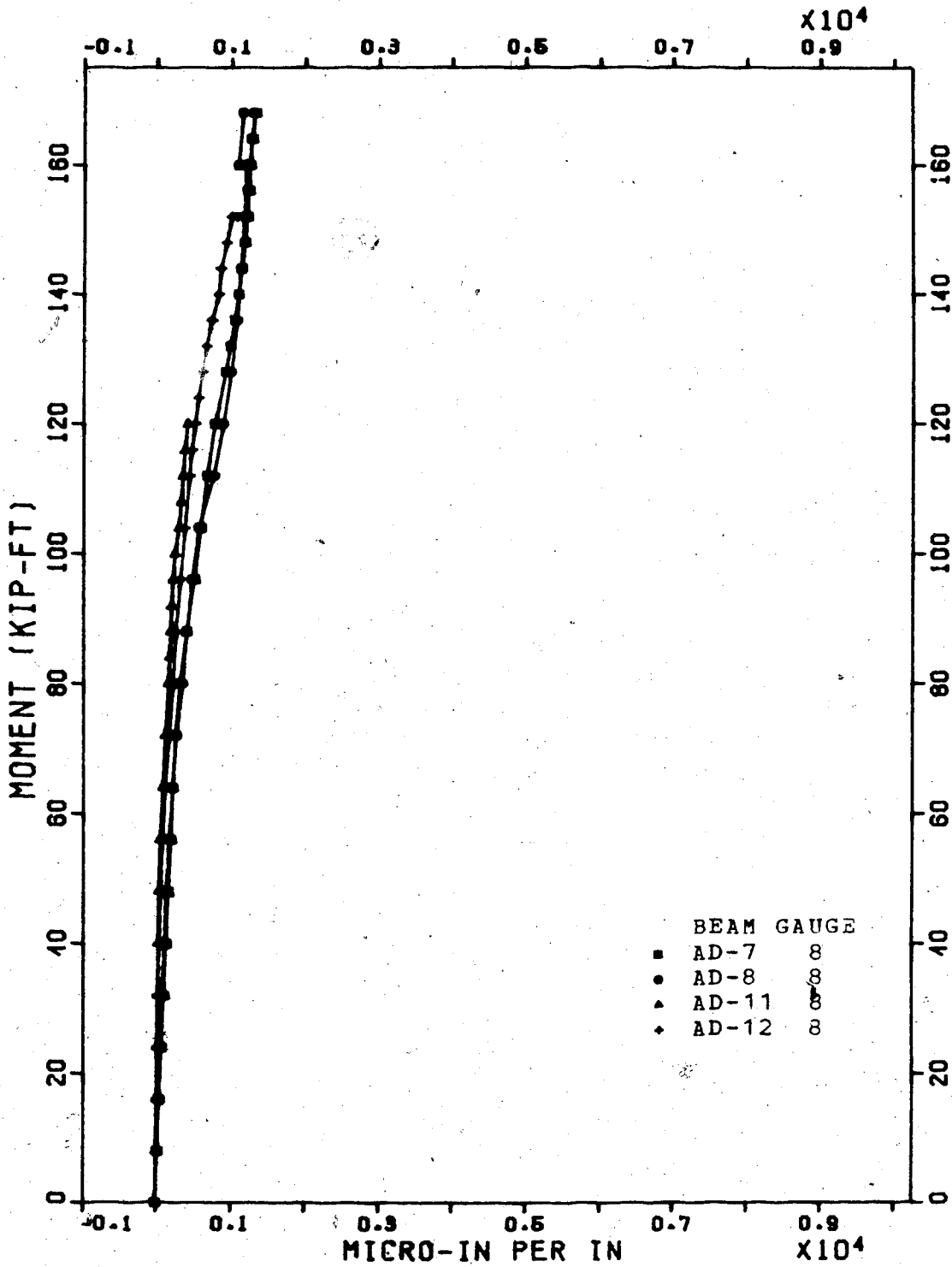


FIGURE 4.8.8

APPLIED MOMENT VS. STIRUP STRAIN AT LOCATION 'F8'

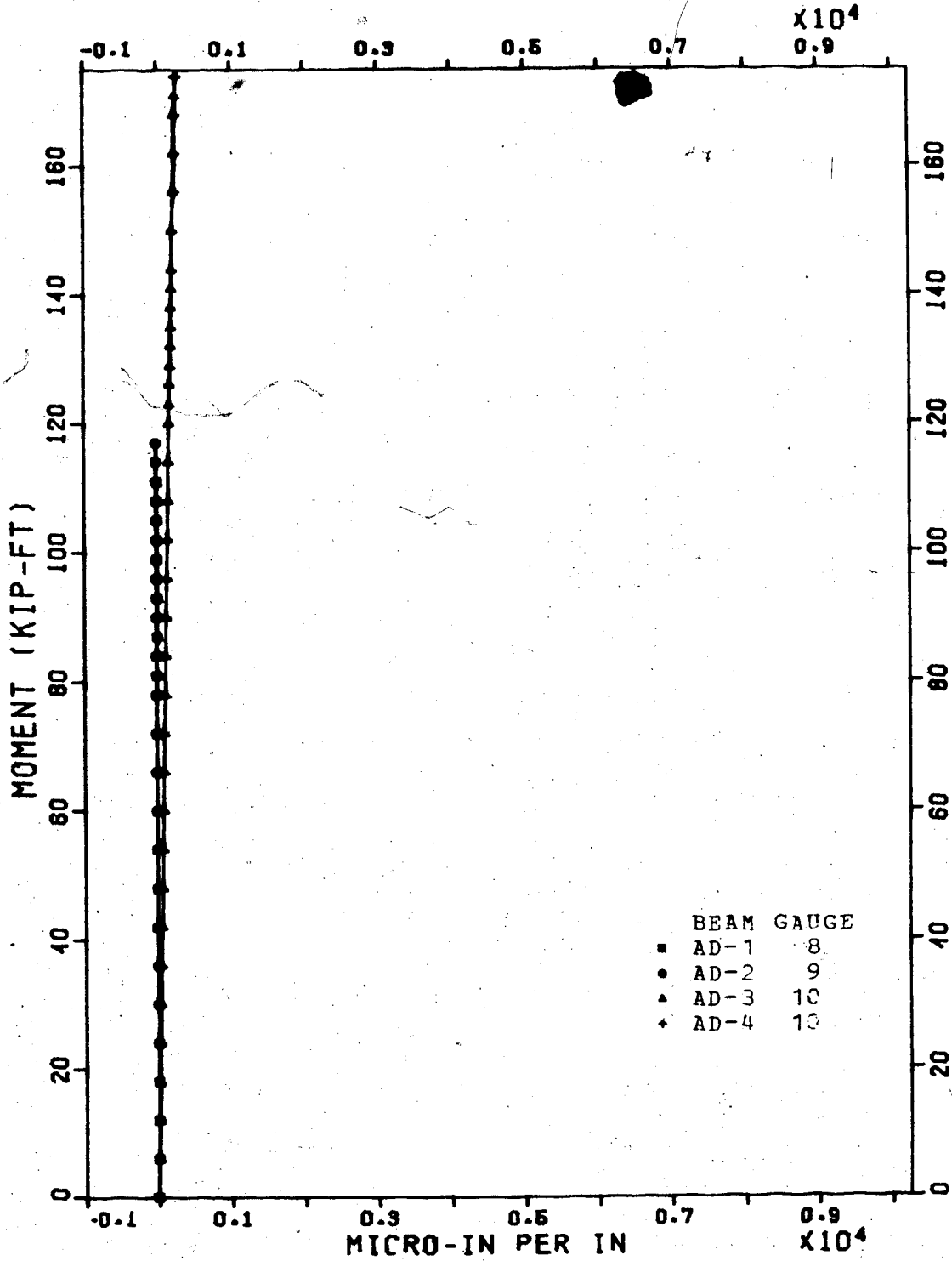


FIGURE 4.8.9

APPLIED MOMENT VS. STIRRUP STRAIN AT LOCATION 'F9'

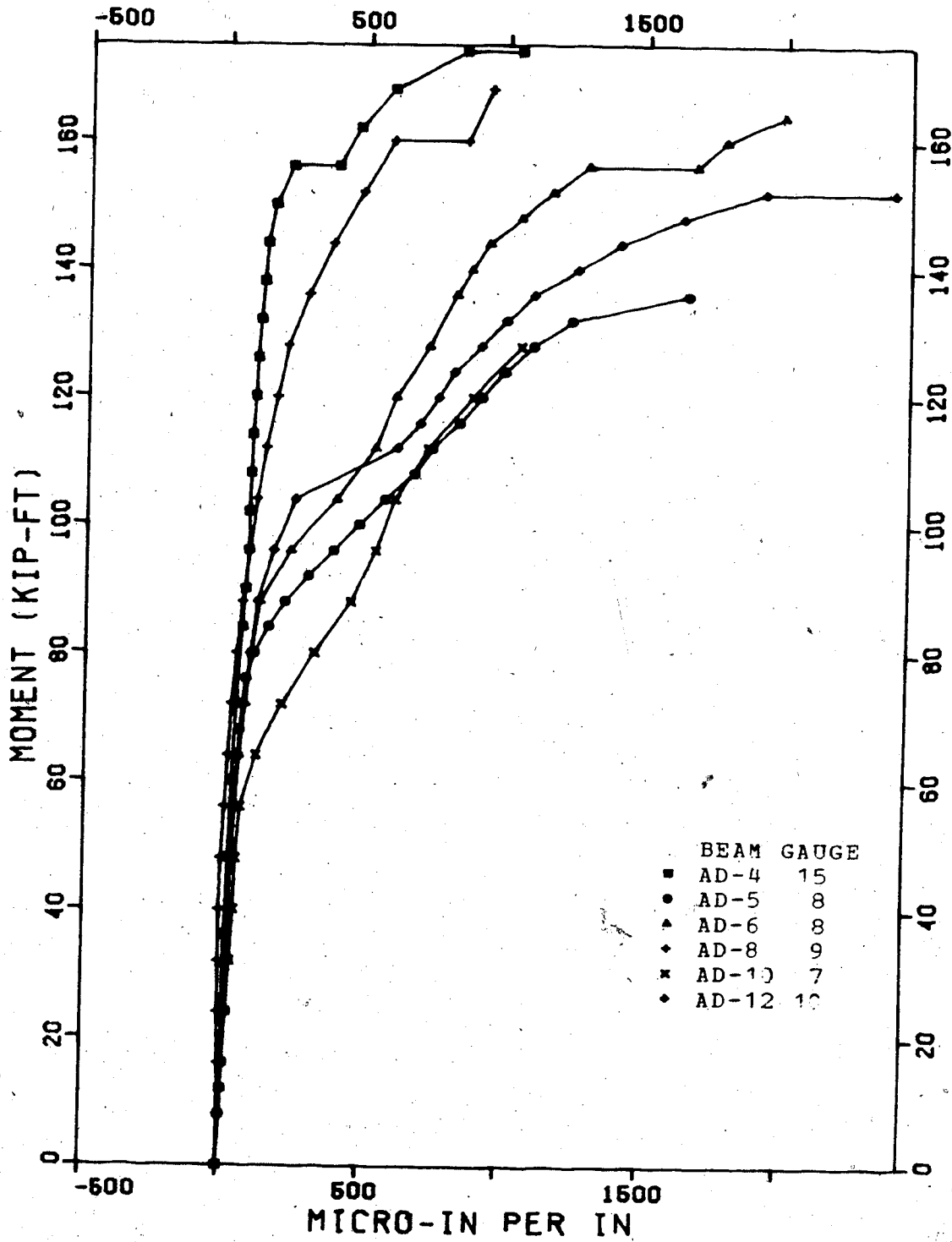


FIGURE 4.9.1 UPPER WEB STIRRUPS
APPLIED MOMENT VS. MOST CRITICAL STRAINS

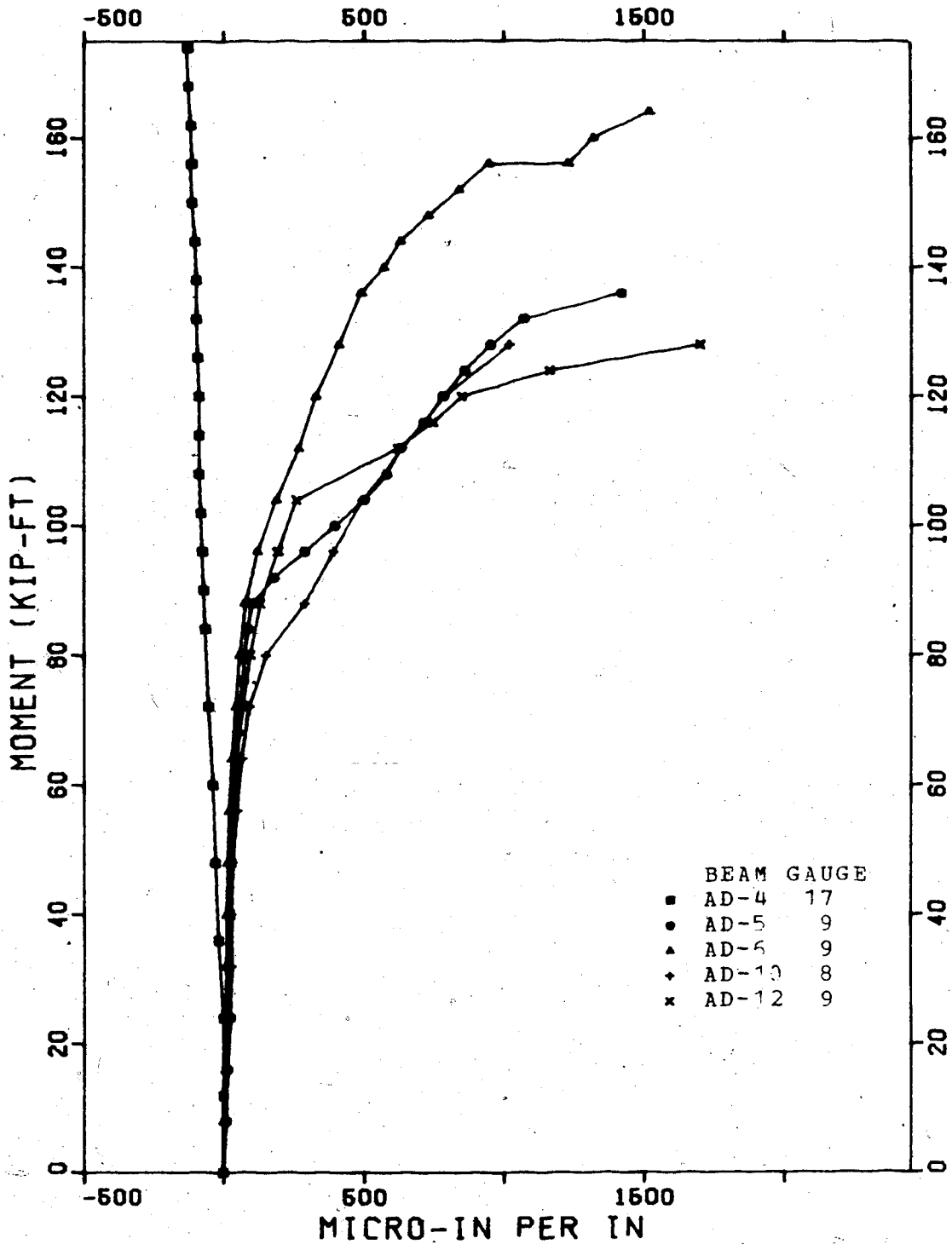


FIGURE 4.9.2 UPPER WEB STIRRUPS
APPLIED MOMENT VS. INTERMEDIATE STRAINS

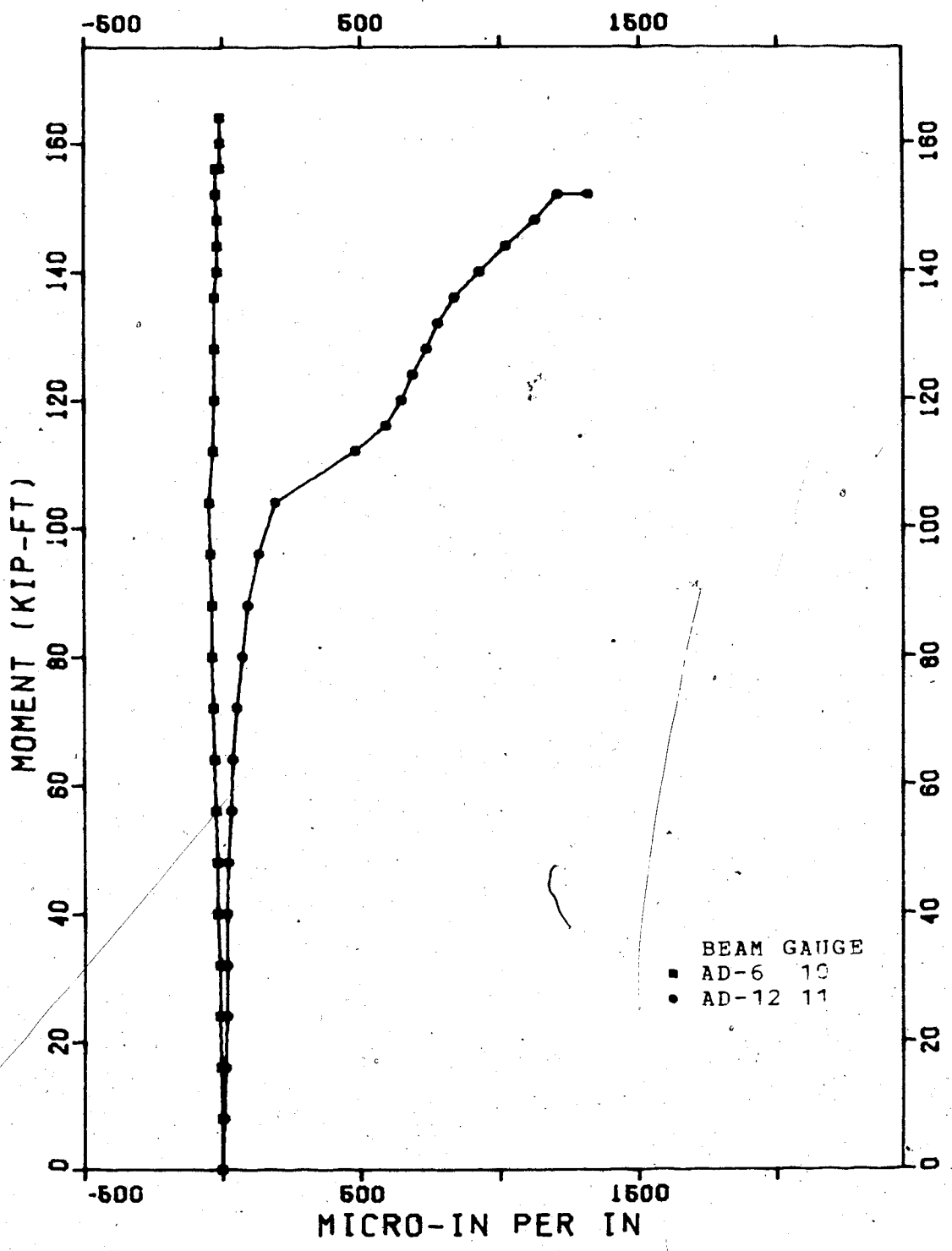


FIGURE 4.9.3 UPPER WEB STIRRUPS
APPLIED MOMENT VS. LEAST CRITICAL STRAINS

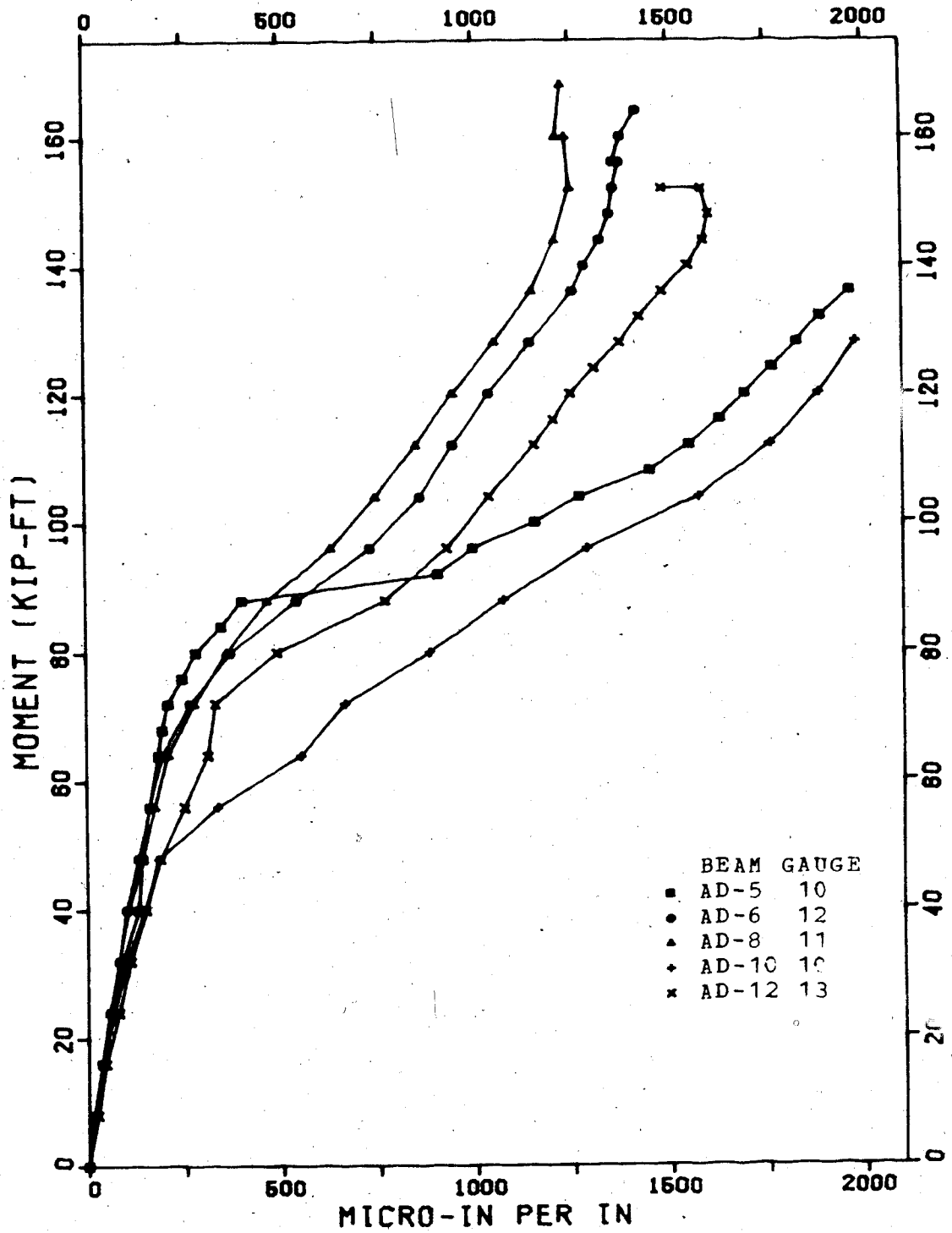


FIGURE 4.10.1 LOWER WEB STIRRUPS
APPLIED MOMENT VS. MOST CRITICAL STRAINS

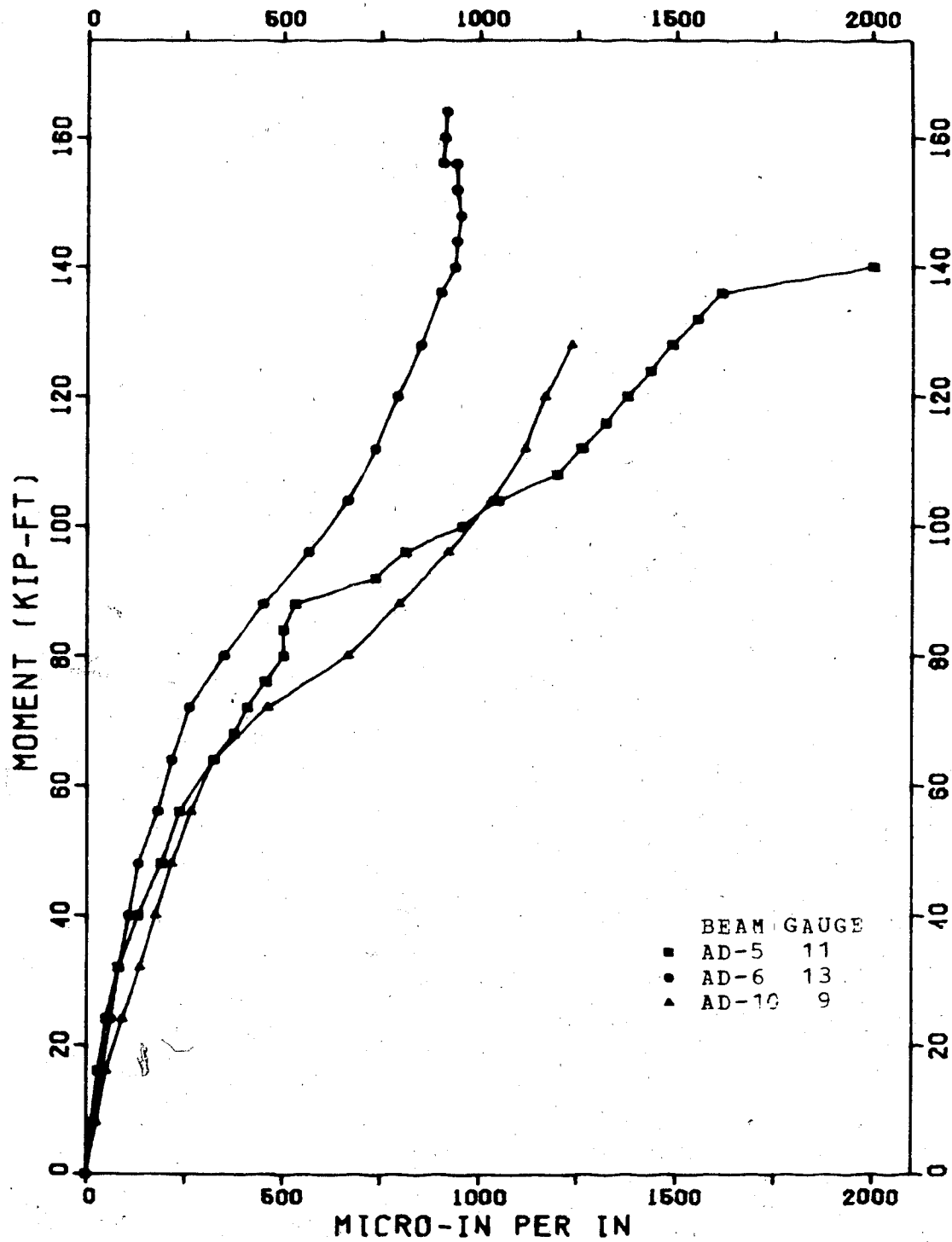


FIGURE 4.10.2 LOWER WEB STIRRUPS
APPLIED MOMENT VS. INTERMEDIATE STRAINS

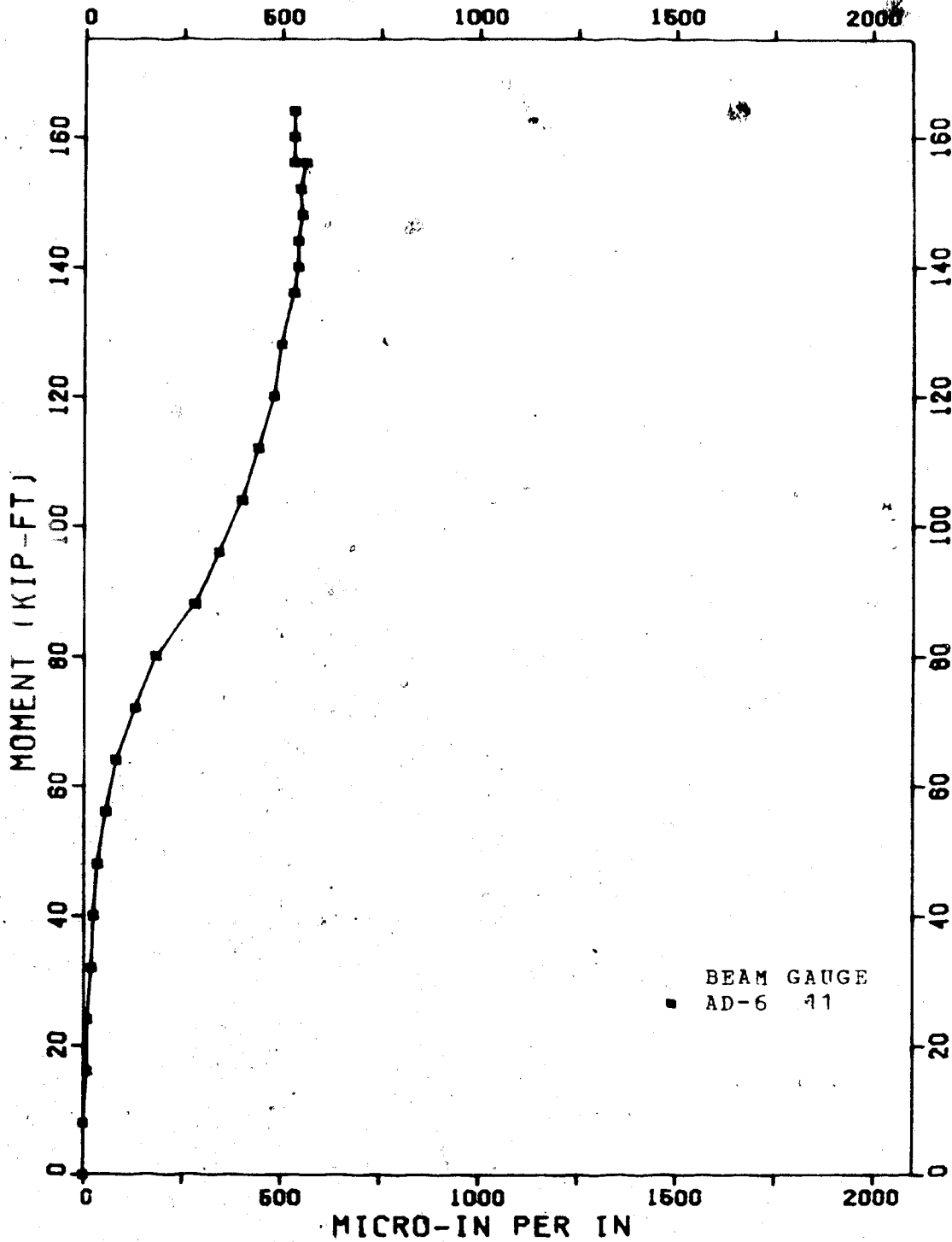


FIGURE 4.10.3 LOWER WEB STIRRUPS
APPLIED MOMENT^o VS. LEAST CRITICAL STRAINS

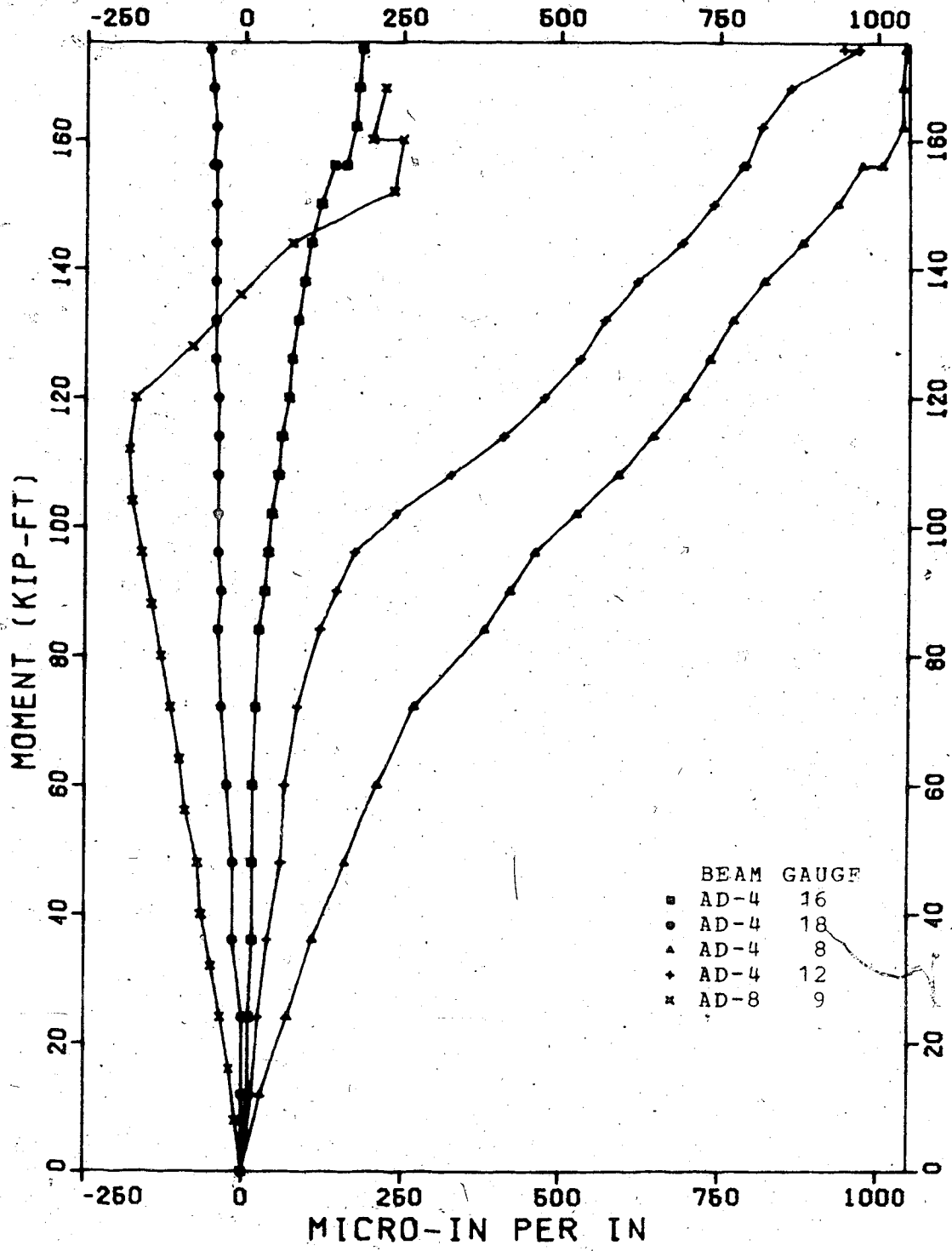


FIGURE 4.11

APPLIED MOMENT VS. MISCELLANEOUS #2 STIRRUP STRAINS

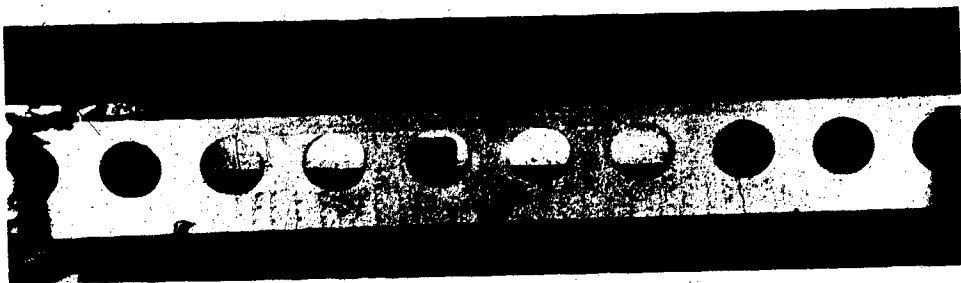


PLATE 4.1 BEAM AD-1 CRACKING AND FAILURE PATTERNS

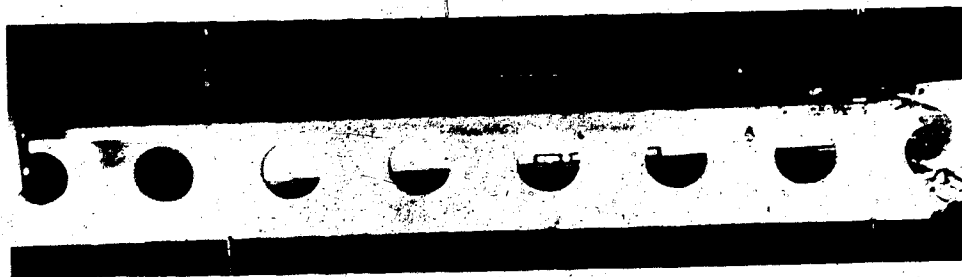


PLATE 4.2 BEAM AD-2 CRACKING AND FAILURE PATTERNS

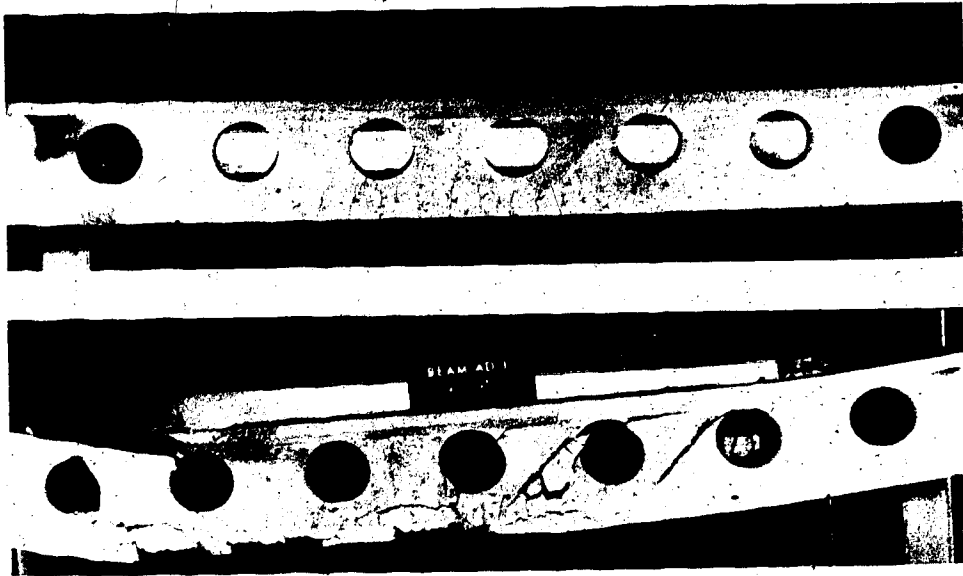


PLATE 4.3 BEAM AD-3 CRACKING AND FAILURE PATTERNS

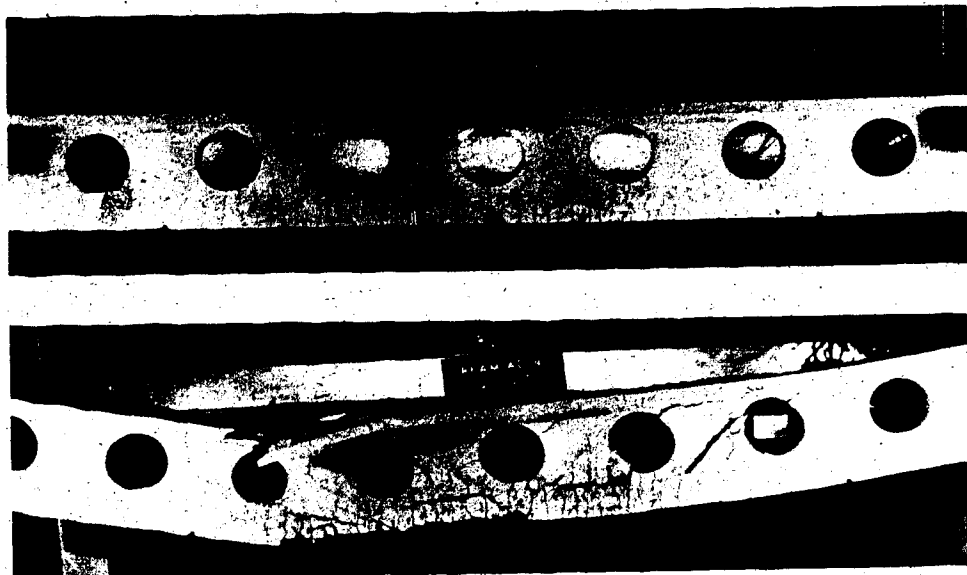


PLATE 4.4 BEAM AD-4 CRACKING AND FAILURE PATTERNS

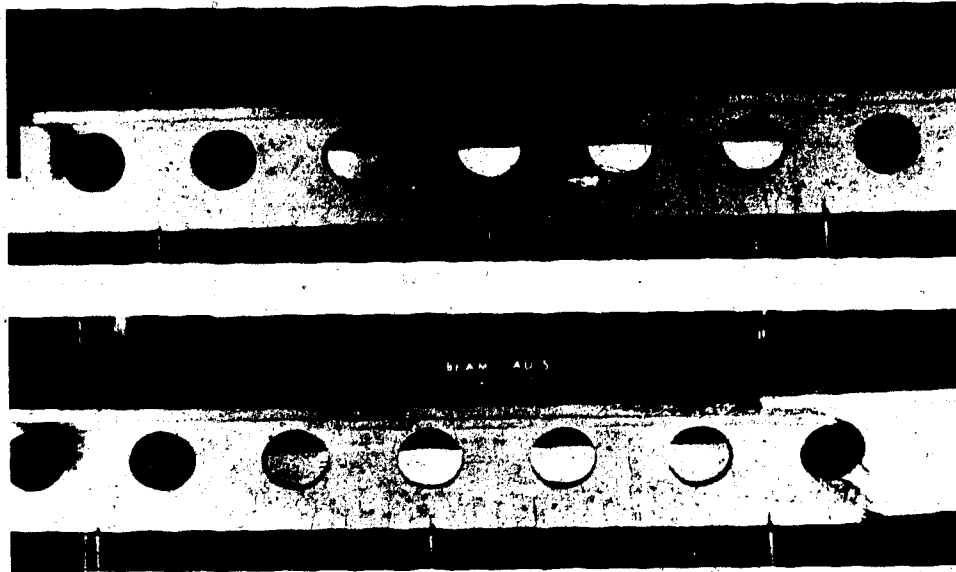


PLATE 4.5 BEAM AD-5 CRACKING AND FAILURE PATTERNS

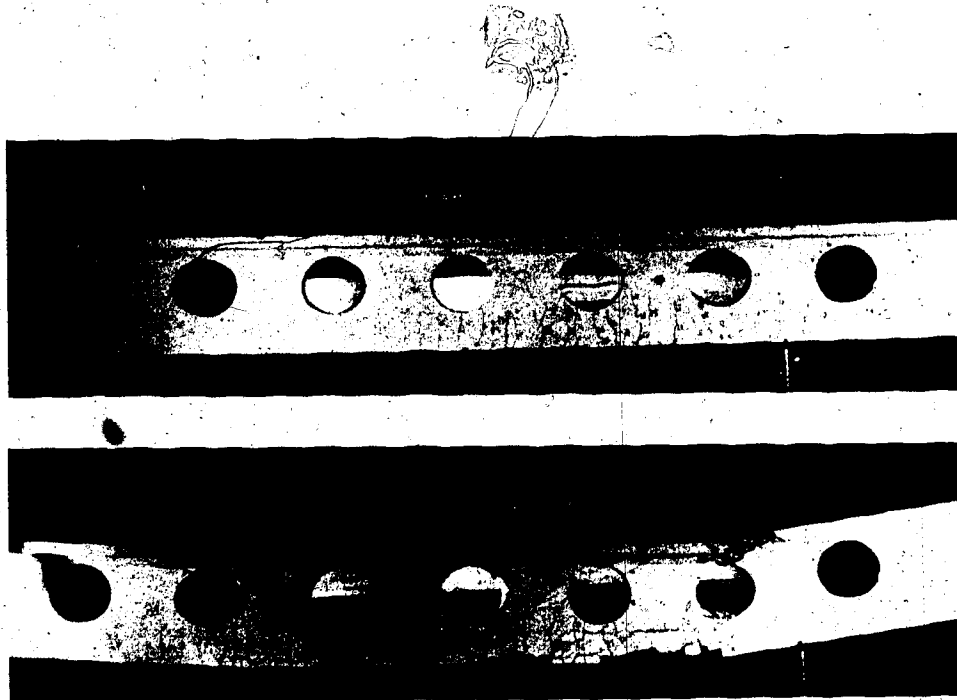


PLATE 4.6 BEAM AD-6 CRACKING AND FAILURE PATTERNS

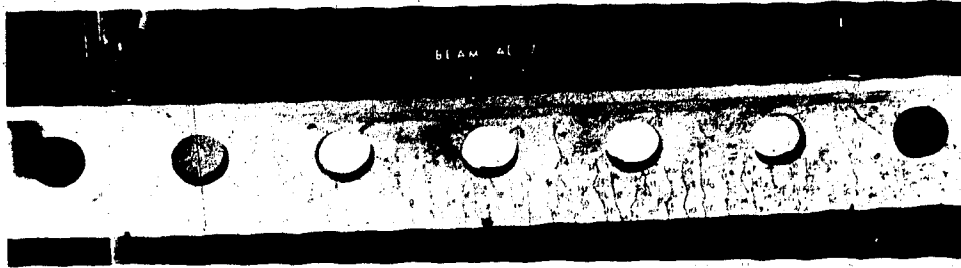


PLATE 4. BEAM AD-7 CRACKING AND FAILURE PATTERNS

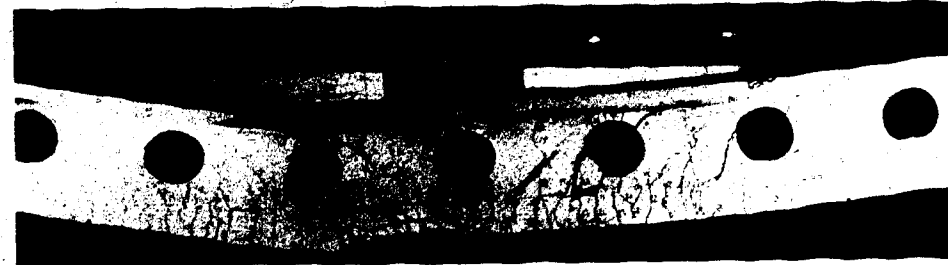


PLATE 4.8 BEAM AD-8 CRACKING AND FAILURE PATTERNS

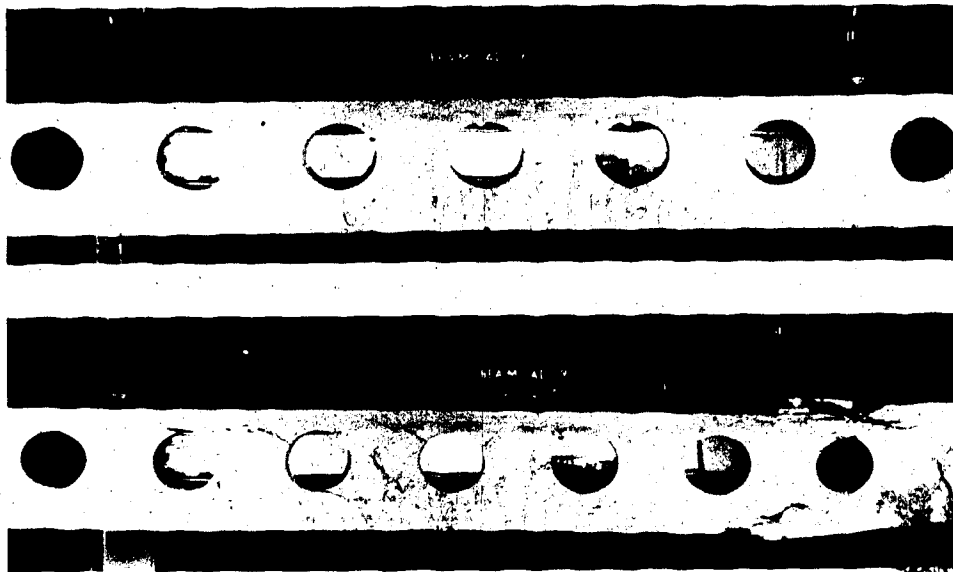


PLATE 4.9 BEAM AD-9 CRACKING AND FAILURE PATTERNS

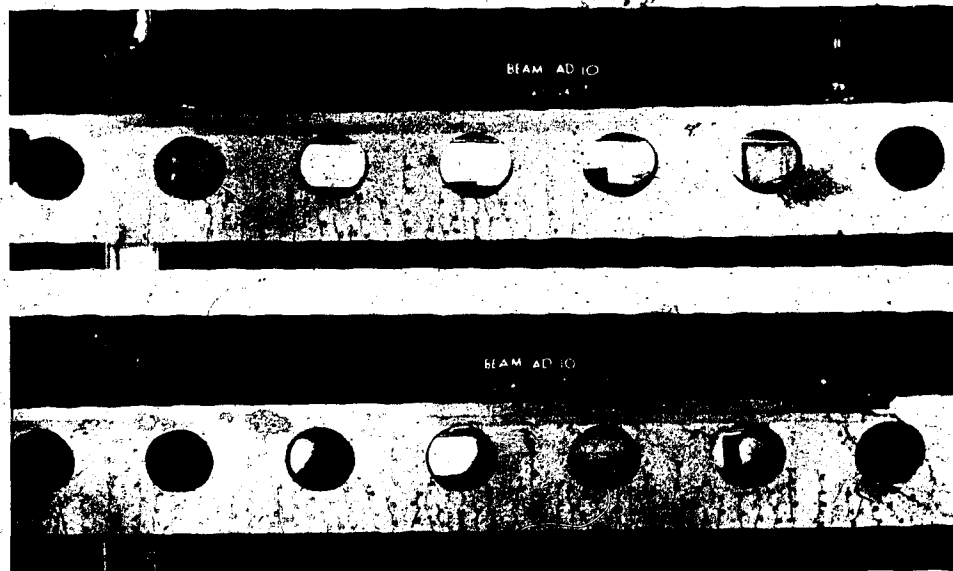


PLATE 4.10 BEAM AD-10 CRACKING AND FAILURE PATTERNS

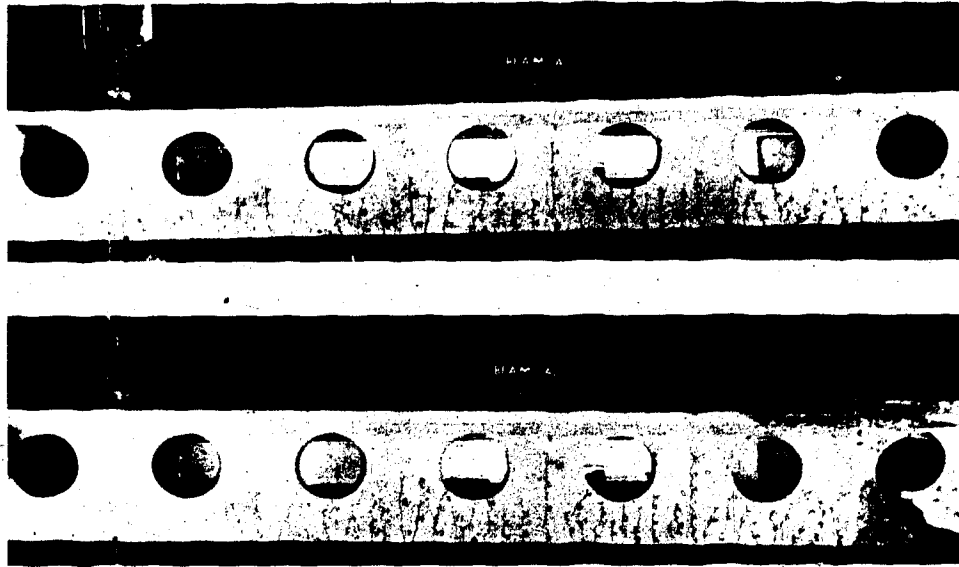


PLATE 4.11 BEAM AD-11 CRACKING AND FAILURE PATTERNS

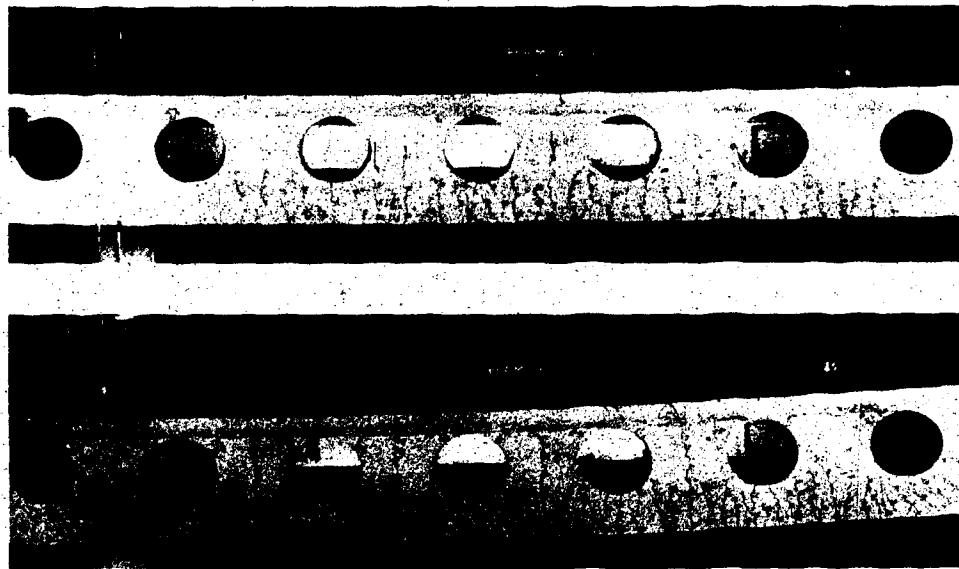


PLATE 4.12 BEAM AD-12 CRACKING AND FAILURE PATTERNS

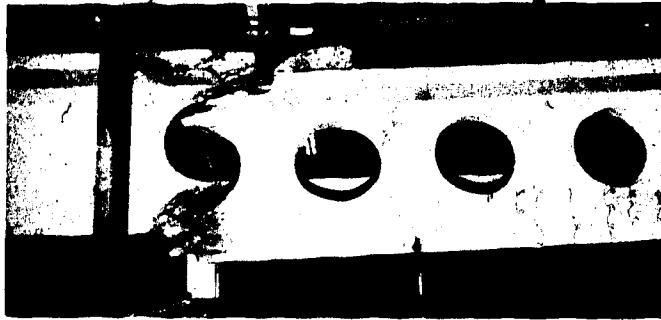


PLATE 4.13 BEAM AD-1
TYPICAL SHEAR FAILURE DETAIL

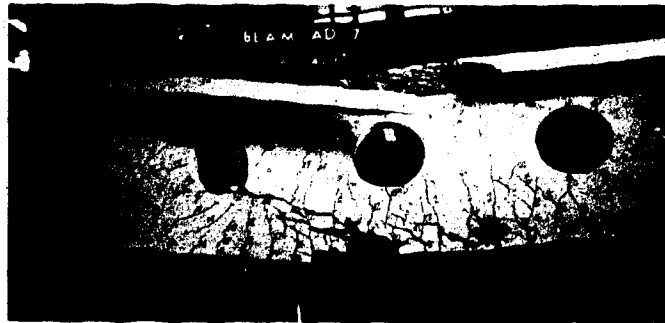


PLATE 4.14 BEAM AD-7
TYPICAL SHEAR-COMPRESSION FAILURE DETAIL

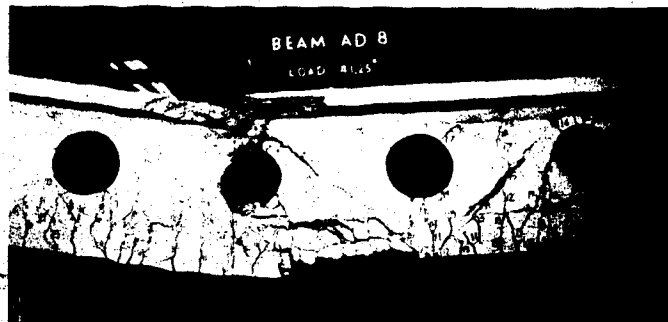


PLATE 4.15 BEAM AD-8
TYPICAL FLEXURAL FAILURE DETAIL

CHAPTER V

DISCUSSION

The test series reported herein consisted of twelve prestressed concrete tee beams, each containing circular web openings. The principal variables included were arrangement of shear reinforcement and loading configuration, while variables of secondary concern were hole size and spacing. The following discussion, supplemented by the figures in Chapters III and IV, describes the behaviour of the beams under load and relates it to the variables mentioned above.

5.1 Parameters

In all cases two equal point loads were positioned symmetrically at either four or six feet from the supports. Thus to reach the ultimate moment capacity of the section, two different severities of shear stress would also be developed. Because no failure mechanisms were formed involving more than one hole, the presence of an extra hole in the longer shear spans of the first four beams had little influence on behaviour and was not considered as a variable.

Ten inch diameter holes were spaced at 20 inches on centre in all beams except AD-1, in which a 16 inch spacing was used. This beam failed at 19 kips whereas AD-2, similar in all respects but hole spacing, failed at 20 kips. The difference is not large enough to attribute it solely to the

difference in spacing, particularly considering the higher measured compressive and tensile strengths of the concrete of AD-2.

The inclusion in the series of beams AD-7 and AD-8, containing the smaller 8 inch diameter holes, did little to isolate the effects of hole size on shear strength as they both had more stirrups per post than other beams. These two beams failed at or near flexural capacity, making it difficult to determine their actual shear strength. The failure at or near flexural capacity of AD-6 and AD-9, the beams most similar to AD-7 and AD-8, also made the combined strengthening effect of smaller holes and more stirrups difficult to assess. It is intuitively obvious, however, that smaller holes in the shear span will decrease the possibility of shear failure, by removing less concrete, and by providing more room for shear reinforcement.

The variations of shear reinforcement formed the most important parameter of these tests. Initial calculations for shear reinforcement requirements ignoring the holes showed that vertical #3 stirrups could be placed in the longer shear spans at 15 inches, and in the shorter shear spans at 10 inches. As well as weakening the beams, the presence of the web openings placed restrictions on possible stirrup locations. While the average spacing of stirrups in the first four beams was 8 or 10 inches, the distance between two stirrups surrounding a hole was 12 or

17 inches depending on their slope.

Full-depth #3 "U" stirrups, as shown in Figure 3.3, were employed in all beams as the primary shear reinforcement. They were fabricated in different heights to allow their placement at 90° , 60° , or 45° to the longitudinal beam axis. In the case of beams AD-7 and AD-8, stirrups of twice the area were produced by tack welding two regular stirrups together.

Secondary shear reinforcement was also provided in six beams in the form of closed #2 stirrups above and below holes in critical shear regions. They were modeled after those of Linder, who developed them pursuant to the recommendations of Le Blanc. Because of the small web depths available, especially for the upper stirrups, they were all placed at 45° to take advantage of the resulting efficiency and overlapping effect of inclined stirrups.

5.2 Cracking

Besides mode of failure, the progression of cracking through loading history is an aspect of beam behaviour that is affected by the presence of web openings. The first cracks noticeable in each of the beams were a form of web shear crack extending from a shear hole in two opposite directions along a 45° line. This cracking developed at lower loads than web shear cracking in the unperforated portions of the shear spans because of the

higher average diagonal tension stress across the line of the cracks caused by the presence of the holes. The next to appear were the first flexural cracks, progressing upwards from the bottom surface of the web near the load points. The total of superimposed and dead load moments at which this occurred ranged from about 640 inch-kips to 900 inch-kips, while the calculated cracking moment, based on the modulus of rupture, was about 870 inch-kips. Within a short time of its appearance the flexural cracking pattern had developed a uniform density and crack length, both of which increased with successive loading until the concrete in the lower webs became unable to contribute to lower web tensile strength. A similar series of flexure-shear cracks also developed in the shear spans, curving upwards and away from the beam ends.

As the neutral axis migrated higher into the web, cracking began to occur above the openings. In the moment spans this often took the initial form of two small cracks ascending radially from each hole at about 45° from the vertical, and generally stopping at the web to fillet junction. These were flexural cracks, and their deviation from the vertical was the only evidence of stress concentration noted in the tests. In some instances these two were the first of a small group of similar cracks radiating from the top quarter of the opening perimeters. Those beams failing in flexure and shear-compression also experienced the propagation of single flexural cracks up the middle of the posts in the moment spans from the bottom

fibre into the fillet.

By the latter stages of loading the first cracks that had appeared above the first hole outside the moment span usually had grown into the fillet towards the load point, still inclined at about 45° . Even at considerably high loads the upper webs immediately above these holes remained relatively free from visible cracking, although this was the location of the final failure crack.

5.3 Classification of Failure Types

The beams were divided into three categories depending on their mode of failure. The flexural group contains all the beams that failed in the desirable flexural manner, indicating that their shear reinforcement was sufficient for them to attain their ultimate capacity. The shear group consists of those beams that failed at loads considerably below the expected flexural capacities, due to insufficient shear strength. The shear-compression group includes the beams that failed at relatively high loads, but in a manner similar to the shear failures. Failures typical of these three types can be seen in Plates 4.13, 4.14, and 4.15.

The last category was labelled shear-compression because of the combination of high shear and compressive stresses in the upper web near the load points prior to failure. This stress condition apparently was the cause of

the failures, as evidenced by the occurrence of cracks inclined less than the 45° of a typical shear crack. While these failures were basically shear in nature, they displayed a behaviour almost as desirable as a flexural failure, since they sustained loads and deflections in some cases greater than those of the flexural group.

The definition of flexural failure as "one occurring in the pure moment region", though somewhat strict, was intended to avoid including a failure not clearly a result of flexural effects. As predicted by strain-compatibility analyses, all flexural failures occurred by the crushing of concrete in the flange and upper web. Thus it was difficult to identify the basic cause of a failure in a shear span as shear or compressive stress on the evidence of appearance alone. This confusion did not arise in previous tests(3)(4)(10) in which all flexural failures were brought about by rupture of the prestressing strand, because this mode of behaviour is indicative of a flexural failure regardless of the amount of shear stress present.

The shear failures were the simplest to distinguish, not only by the low loads involved, but by the form that they took. Because of the low moments sustained, shear stresses dominated over compressive stresses, and the failures took the form of two diagonal cracks, inclined at about 45° . The first of these appeared in the lower web

below a hole in the shear span where the concrete was already weakened by flexural tension cracks. Upon the loss of the ability of the lower web to resist shear stress, the portion it had carried was transferred to the upper web. When the shear capacity of the upper web and flange was reached the second diagonal crack appeared, rending the upper web, usually in a violent manner. Both cracks tended to be positioned such that they penetrated the upper and lower webs at their minimum sections, thus minimizing their total length.

The beams were tested in numerical order, but to provide a more logical sequence for comparing their behaviour, the tests will be described in the next three sections by failure type in ascending order of ultimate capacity, expressed in terms of maximum moment at collapse. The ranges of values of maximum moment of the three failure groups are: shear 1368 to 1680 inch-kips, shear-compression 1824 to 2088 inch-kips, and flexural 2016 to 2095 inch-kips. Comparisons of the shear capacities of beams containing different arrangements of shear reinforcement are included even in cases where the shear spans were of different lengths. Because of the dependence of concrete shear strength on normal stress, and thus on applied moment, these comparisons are intended only as qualitative indications of the effects of altering shear reinforcement.

5.4 Beams Failing in the Shear Mode

This group is the largest, consisting of five beams, failing at moments of from 1368 to 1680 inch-kips. It is interesting to note that only one beam with inclined primary shear reinforcement, AD-11, fell in this category. The stirrups in this beam were placed at 60°, rather than at the more efficient 45°, and were not supplemented by upper or lower web shear reinforcement. A further observation is that all beams containing vertical full-depth stirrups were included in this category.

The beam attaining the lowest maximum moment was AD-1. This was expected, because as well as being lightly reinforced with two vertical #3 stirrups per post, this was the only beam with a hole spacing as small as 16 inches. The failure, however, did not occur in the resulting 6 inch posts but in the web portions above and below the first hole from the South end in the manner described in the previous section. The posts in the shear spans, being only slightly cracked, showed that they could have resisted higher loads but it could not be concluded that their strength would be sufficient to allow a flexural failure.

Exceeding AD-1 in strength by about 5%, AD-2 failed at a maximum moment of 1440 inch-kips. AD-2 was detailed exactly as AD-1, except for its 20 inch hole spacing, and the resulting increased length of the moment span from 8 feet to 8 feet 4 inches. Similar too, was the mode of

failure, except that it occurred at the second hole from the North end. As noted before, the concrete in AD-2 had higher measured compressive and tensile strengths, which alone may have been sufficient to cause the difference in strength between the beams. AD-2 was flexurally stiffer than AD-1, probably due to the combination of larger hole spacing and the higher modulus of elasticity associated with higher compressive strength. At a load of 18.5 kips (1332 inch-kips, maximum moment) the centerline deflection of AD-1 was 1.56 inches, while that of AD-2, though longer by 4 inches, was only 1.50 inches.

AD-11 failed at a maximum moment of 1488 inch-kips, only 3.3% higher than that of AD-2. Because it had shorter shear spans this represents a maximum shear force 55% higher. Besides having one less hole in each shear span, AD-11 differed from AD-2 in having three stirrups per post inclined at 60° rather than two vertical stirrups. While AD-11 failed in a manner similar to AD-2, the general inclination of the principal cracks was steeper in the case of AD-11. By intersecting the path that a crack would follow in the absence of shear reinforcement, the stirrups adjacent to the hole tended to force the webs to crack in a different location, and thus at a higher load. In this way the stirrup that is barely visible at the bottom of the web below the hole in Plate 4.11 forced the bottom ends of the cracks to the left, away from the support, while the top ends were still free to remain close to the lowest point on the

circumference of the hole. This effect also tended to restrict the volume of lower web that was split severely by the cracks, consequently reducing the amount of residual vertical shear deformation across the hole.

Beams AD-10 and AD-5 were the strongest of the group, reaching maximum moments of 1584 and 1680 inch-kips, respectively. AD-5 was designed and tested first, and when it failed below ultimate flexural capacity, its design was repeated in AD-10, but with a closer spacing of #2 stirrups. It was anticipated that this would increase the shear capacity of the upper and lower web regions and thus allow AD-10 to reach higher loads. This did not occur for two reasons. The #2 bar used for the stirrups in AD-10 came from a batch having a yield stress of only 36 ksi, 8 ksi less than that used in AD-5, resulting in $(A_v f_y / s)$ values for AD-10 only 2% and 9% higher in the upper and lower webs respectively. Also beam AD-10 was not seated properly with the centre surface of the web coplanar with the axes of the jacks, as well as adversely affecting the strength of AD-10, this misalignment modified the mode of failure from the type exhibited by all of the other beams in the shear group, by inducing transverse components of load. There was far less visible evidence of distress in the lower web. No cracks could be seen to be completely severing the lower web, and none of the reinforcement contained therein was revealed by surface spalling which was so prominent in other beams. The upper web was cracked in the usual way, although the crack

opened only slightly, and the failure section did not present as striking a discontinuity in the horizontal lines of the beam as it had in other beams.

The flange, however, was badly damaged by the weak-axis bending and torsion resulting from the transverse components of the loads. These effects added to the already high compressive and shearing stresses in the East side of the flange, causing this region to fail in compression between the load and the hole in the shear span. Collapse progressed slowly enough that it could be seen that the crushing of the flange occurred first, precipitating the final failure of the upper web by a shear-type crack. Slippage along this crack (in a counter-clockwise sense in Plate 4.10) caused the crushing and spalling at the bottom of the hole and the series of unmarked cracks in the lower web, extending from the spalled area to the flake protruding from the bottom of the web towards the support.

Beam AD-5 was the only beam containing upper and lower web stirrups that failed in the shear mode typical of the other beams in this group. The lower web was split by a series of cracks inclined at about 45° , and its surface was broken away exposing three of the enclosed stirrups. In Plate 4.5 these stirrups are inclined more steeply than the 45° at which they were placed because of the counter-clockwise slippage that occurred along the cracks. At the penultimate load of 34 kips a crack appeared at a slope of

less than 45° , extending from the topmost point of the hole through the fillet and into the flange. The failure of the beam was brought about by the shear failure of the upper web along this crack after application of the last increment of load.

If it could be assumed that the shear capacity of the test beams was dependent only on the shear reinforcement present, and not at all on the loading configuration, then a beam similar to AD-5 but without upper and lower web stirrups, would fail at 20 kips, the failure load of AD-2. The inclusion of this extra reinforcement would then be responsible for an increase of 75% in the capacity of AD-5. While this figure is rather high, it indicates that the effect of such stirrups is of the same order as that of the 60° full-depth stirrups of AD-11. The manner in which these two schemes of reinforcement achieved higher strengths was different, however. In the case of AD-11 the stirrups adjacent to the hole extended into the upper and lower web regions and intersected the line of the most probable shear crack, thus forcing it to detour along a new route at a higher load. The #2 stirrups of AD-5, by being distributed throughout the upper and lower webs, allowed no such possible detour but required the failure cracks to appear in the usual location at a higher load because of the numerous stirrups to be crossed.

Generally, all of the shear failures in this test

series occurred at locations predetermined by the geometry of the circular web openings. The upper and lower webs formed by these holes were necessarily very stocky, and, though only a few inches deep, acted as haunched deep beams failing as such in shear rather than flexure. In contrast, the beams cast in previous tests with long rectangular web openings, were such like Viereendeel trusses having slender beam-column members. In regions of high shear force these members would develop considerable secondary flexural stresses which in some cases led to mechanism failures before the flexural capacity of the beam as a whole could be attained.

5.5 Beams Failing in the Shear-Compression Mode

The four beams in this category made use of two different schemes of shear reinforcement. One combined upper and lower web stirrups with three full-depth stirrups per post inclined at 60° . The other three had two, three, or four 45° stirrups in the posts, with no additional shear reinforcement in the upper and lower webs.

The beam reaching the lowest maximum moment of 1824 inch-kips was AD-9, containing three 45° stirrups per post. Although collapse occurred with little warning, it was apparent that the progression of deterioration of this beam was different from that of the shear failure group. Due to the intrusion of the #3 stirrups into the lower web, the

shear cracks developed there opened only slightly and had no serious effects on the shear capacity of that region. Similarly, the upper web remained relatively free of cracks until the latter stages of loading, although it was less protected by the stirrups because of the proximity of the hole to the top surface of the flange. Failure occurred in the upper web due to the combination of high compressive and shear stresses, but at a location above the edge of the hole nearer the support, rather than at the minimum section. Much of the damage visible in Plate 4.9 was caused by the sudden release of the large amount of strain energy contained in the beam, rather than by static stresses. The most obvious examples of this type of damage were the cracks in the lower web aligned along the direction of principal tension instead of across it, and the separation of the fillet from the web to the right of the hole. The loss of portions of the flange and the appearance of cracks coincident with some of the reinforcement in the other end of the beam, were also results of this energy release.

Although having shear spans of different length, AD-2, AD-11, and AD-9 illustrate well the strengthening effect of sloping shear reinforcement in beams containing circular web openings. In these beams the stirrups were placed at 90° , 60° , and 45° respectively, producing ultimate loads of 20, 31, and 38 kips. These successive improvements were not the result of the alignment of the sloped reinforcement with the diagonal tension field alone, but

also of their presence in the critical areas of the upper and lower webs made possible by their inclination.


Although failing at a maximum moment of 1872 inch-kips, only slightly higher than that of AD-9, AD-12 seemed to suffer less damage, especially that caused by the dynamic release of strain energy. Crushing occurred above the hole in the South shear span, breaking off small pieces of the flange. The lower web was cracked in the same way as was the lower web of AD-9, but not excessively enough to reveal the steel within. The remainder of the beam retained its well developed regular crack pattern after failure, giving no visual indication that it had resisted stresses capable of destroying its load-carrying capacity.

Beam AD-12, containing the 60° #3 stirrups, was similar to AD-11 except for the addition of #2 stirrups in the upper and lower web in the shear spans. This supplementary reinforcement caused an increase in strength of about 26% over that of AD-11. Comparison with AD-5 shows that the effect of changing the slope of the primary shear reinforcement from 90° to 60°, while retaining the secondary shear reinforcement, was to increase the ultimate load by about 11% from 35 to 39 kips. This was not as much as the increase that would be caused by the same transformation in a beam with no secondary shear reinforcement, because the small stirrups in the upper and lower webs were displaced away from the critical sections to make room for the larger

full-depth stirrups.

The maximum moment of 2040 inch-kips reached by AD-7, while not the largest, represents the highest value of shear applied to a beam in the shear-compression group. The design of AD-7 followed the testing of AD-3, in which the effectiveness of sloped primary shear reinforcement was first observed. In anticipation of the higher shear force required to develop the ultimate flexural strength of the beams with shorter shear spans, the upper and lower webs were strengthened in two ways. A double stirrup was placed under each load point extending through the critical sections of the upper webs, providing more protection in these areas than would a single stirrup. As well, the diameter of the openings was reduced to 8 inches, making the webs more accessible to the sloped stirrups, and providing a larger area of concrete to resist shear stresses. Together, these two factors caused the failure load of AD-7 to exceed that of AD-9 by about 12%.

Failure of beam AD-7 was similar to that of AD-9, except that the extent of damage caused by the release of strain energy was greater. Stirrups were uncovered by the cracks opening along their length, and portions of the bottom surface were broken off by cracks following the longitudinal steel in the lower web. Longitudinal cracks also appeared on the top surface of the flange approximately above the compression steel in the upper web, severing a



large portion of the flange near the South load point, though not detaching it completely from the beam.

Beam AD-3 failed at a maximum moment of 2088 inch-kips, in a manner strikingly similar to AD-7, as seen in Plates 4.3 and 4.7. The shear force involved, however, was much lower because of the longer shear spans of AD-3, and for this reason two #3 stirrups per post were nearly sufficient to induce a flexural failure. A comparison of this beam with AD-2 provides the best illustration of the effect of changing the slope of the full-depth shear reinforcement, as these two beams were similar in all other respects. AD-2 failed at 20 kips and AD-3 failed at 29 kips. This 45% increase in strength, though not as large as others reported, is significant in that the maximum moment involved is larger than those sustained by two beams failing in flexure.

In general, the beams in the shear-compression group behaved in a more desirable manner than did the beams of the shear group. All failure loads were higher in the shear-compression group, thus ultimate deflections tended to be higher also. The one exception to this rule was AD-9 which reached a deflection of only 2.09 inches, slightly less than the 2.38 inch deflection of AD-5 of the shear group. The ultimate deflections of the rest of the shear-compression group ranged from 5.76 to 10.22 inches, with the value of 7.80 inches for AD-7 exceeding the lowest

ultimate deflection of a beam of the same length in the flexural group.

5.6 Beams Failing in the Flexural Mode

The three beams failing in this category failed at maximum moments ranging from 2016 to 2095 inch-kips. These were the only beams containing upper and lower web stirrups in conjunction with full-depth #3 stirrups placed at 45°. In all three cases the shear spans remained relatively intact, while the pure moment regions suffered damage from the dynamic release of strain energy as noted before. The mechanism triggering collapse was different, however, as the upper web and flange failed in compression at a location within the moment region.

Beam AD-6 failed at a maximum moment of 2016 inch-kips, lower than the failure moments of AD-3 and AD-7 of the shear-compression group. Location of the failure section was made evident, prior to complete loss of strength, by the slow spalling of the top surface of the flange which occurred near the first hole in the North end of the moment region. Because of the lower moment, beam AD-6 sustained less damage upon the release of strain energy than did the rest of this group and some of the shear-compression group. The longitudinal reinforcement above the hole was buckled out as in a column-type failure. Two of the #3 stirrups and portions of the longitudinal bars in the bottom of the web

were revealed by the cracking.

Most closely related to beam AD-6 are beams AD-5 and AD-9. Beam AD-5 had upper and lower web stirrups at about the same spacing as those in AD-6, but the three full-depth stirrups in each post were placed vertically. The use of the inclined stirrups in AD-6 caused this beam to reach a load of 42 kips per jack, or 20% higher than the 35 kip ultimate load of AD-5. Beam AD-9 was detailed as AD-6 but without upper and lower web shear reinforcement. Its ultimate load was 38 kips, so that the addition of the supplementary shear reinforcement increased the strength of AD-6 by about 11%.

Beam AD-8 failed at a maximum moment of 2076 inch-kips, or at an ultimate load of 43.25 kips per jack. This was the beam most heavily reinforced against shear failure and as a result, it sustained the highest shear force applied to any of the beams. In addition to the use of smaller holes and doubled stirrups, as in AD-7, this beam contained upper and lower web stirrups at all hole locations in the shear spans. The upper webs above the end holes in the moment span were also protected by stirrups. This was done in an attempt to shift the location of the failure region towards the beam centerline where failure should theoretically occur because of the dead load moment. It was thought that some mechanism action was inducing erratic shear stresses in the upper web near the load points in the moment span which, combined with

the high compressive stress, brought about collapse prematurely. These extra stirrups shifted the failure section to the second hole from the end of the moment span, and kept the region between there and the near load point as intact as the shear span regions. The damage was more severe than in AD-6, as the residual deflection was larger.

Beams AD-7 and AD-8 provide another illustration of the effect of adding supplementary upper and lower web stirrups to a beam containing full-depth stirrups placed at 45°. Because of the efficiency of the primary shear reinforcement, the supplementary shear reinforcement increased the strength of AD-8 by only about 2%. Since it failed in flexure, the shear strength of this beam would probably have been somewhat higher.

Because the shear strengths of beams AD-8 and AD-6 are not known, owing to their failure in the flexural mode, a comparison of their strengths based on differences in their shear reinforcement would have no practical value. That AD-8 reached a higher load than AD-6 was not due to a difference in concrete strengths, as these were 5400 and 6100 psi, respectively. The larger cross-sectional area of the upper web and flange of AD-8 may have had some effect, although the depth of the equivalent rectangular stress block of the strain-compatibility analyses is less than the depth of either upper web. While the stirrups in the upper web in the moment span of AD-8 may have strengthened that

section by confining the compression zone, it is unlikely that this alone could have increased the strength of the whole beam. Most probably the explanation is a combination of effects, including local concrete weaknesses, fabrication inaccuracies, and prestressing strand inconsistencies.

The final beam of this study to be reported is AD-4, which reached a maximum moment of 2095 inch-kips. This beam was lightly reinforced with upper and lower web stirrups and only two #3 stirrups per post, but this was sufficient to avoid a shear failure because of the longer shear spans and subsequently lower shear forces. Crushing occurred above the first hole in the South end of the moment span and was accompanied by the usual cracking caused by release of strain energy. Cracks also appeared along the top surface of the flange above the longitudinal reinforcement contained therein.

The difference in ultimate load between AD-4 and AD-3, being only about 0.3%, is an insignificant one. Because of the lower concrete strength of AD-4, 5400 psi compared with 5600 psi for AD-3, it would have had a lower ultimate load than AD-3 if fabricated without the supplementary reinforcement. But since the possible failure locations in the shear spans were protected, mainly by the upper web stirrups, the failure occurred in the moment span at a load about equal to the ultimate load of AD-3. Because of the proximity of the failure load of AD-3 to the flexural

ultimate load, these stirrups probably acted to confine the compression zone as well as resist the principal tension stresses.

The results of the three tests reported in this section indicate that the concrete section used, although under-reinforced in the sense that the tensile steel was well yielded, tended to fail in flexure by crushing of the flange, rather than by rupture of the prestressing strand. Thus, from the standpoints of strength and ductility, beams AD-3 and AD-7 could be considered to have failed satisfactorily in flexure, although their failure regions developed in the shear spans. Including these two beams as having failed in flexure, the deflections at ultimate load of this group ranged from about 10.2 to 12.4 inches, and 7.1 to 8.3 inches for the longer and shorter beams respectively.

5.7 General Discussion

The present test series consisted of twelve prestressed concrete tee beams containing large circular web openings. While it was found that the openings decreased beam shear strength and flexural stiffness, the behaviour of these beams was different from that of beams containing rectangular or parallelogram shaped openings.

The mode of flexural failure exhibited by the present series was quite different from that of previous tests. Noticeably lacking was the occurrence of tensile failure of the prestressing strands, the criterion used by

Le Blanc⁽³⁾ Linder⁽⁴⁾ and Sauve⁽¹⁰⁾ to distinguish between shear and flexural failure. This made the classification of failure types more difficult in some cases, and led to the adoption of a third category, shear compression failures. Although failure by strand rupture was probably incipient in some of the beams, it is a little odd, considering the similarity of the test specimens used in all the tests conducted at the University of Alberta, that no beam with round holes failed in this manner, and that no beam with angular holes was reported to have failed in flexure by crushing of the flange. There is not enough evidence, however, to conclude that tensile strand failure was made less likely by the use of circular web openings.

The main problem encountered in introducing openings into the web of a tee beam is to maintain sufficient shear strength to allow the member to attain its flexural capacity. This is because the centre portion of the web, where openings are usually placed, carries most of the shear on the section. Thus the main concern of this test series was to study the behaviour of the beams in shear, rather than in flexure, and to compare this to the effects of more elongated holes of quadrilateral shape.

The tests confirm that the geometry of the web openings had a significant influence on the mode in which shear failures took place. Previous test beams containing long quadrilateral openings often experienced localized

flexural failure of the upper and lower webs rather than shear stress-diagonal tension failure, because of the slenderness of these sub-members. Also, the sharp opening corners, although filleted, gave rise to stress concentrations at locations where the secondary bending stresses were already high. In contrast, the circular openings produced upper and lower webs which were shorter and stockier, and thus apparently less susceptible to secondary flexural failure. Shear failures typically occurred by diagonal tension cracking, exhibiting little evidence of distress due to secondary flexural effects or stress concentrations.

As well as lessening the likelihood of secondary flexural failure of the upper and lower webs, the geometry of the round holes governed the locations of possible shear failure cracks. Each of these sub-members had one minimum section located above or below the centre of a hole, at which the shear stress was a maximum. As expected, the beams without upper and lower web shear reinforcement sustained 45° shear failure cracks precisely at these sections. The addition of shear reinforcement in these regions, although increasing shear capacity, was able to shift the crack locations only slightly.

The circular openings also tended to preclude the possibility of shear failure in the posts, which occurred in some of the tests on beams with rectangular or parallelogram

shaped openings. Each post had one minimum section, as did the upper and lower webs, but because of the curvature of both sides of the posts, an inclined crack would not be able to take advantage of this. In a beam in which the spacing of circular openings is greater than about 1.4 times the opening diameter, two adjacent openings cannot be connected with a 45° line. Thus any potential diagonal tension crack must be inclined at less than 45° , such that the tension across it is not the principal tensile stress.

While the basic behaviour of the beams was determined by the presence of the circular web openings, it was modified by the use of shear reinforcement. The beams failing at the lowest loads had a very minimal amount of shear reinforcement, which was furthermore confined to the posts. The beams failing in flexure contained shear reinforcement in the posts which also extended into the more critical upper and lower web regions, plus smaller, closely spaced inclined stirrups distributed throughout the upper and lower webs in the shear spans. Arrangements of reinforcement between these two extremes produced beams having strengths between the minimum and maximum strengths recorded.

CHAPTER VI
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

In this investigation, twelve prestressed concrete tee beams containing circular web openings were tested. The principal variables included were arrangement of shear reinforcement, and loading configuration, although hole size and spacing were varied slightly in some of the beams. In all cases two equal point loads were positioned symmetrically on the beams at either four or six feet from the support centerlines. Flexural reinforcement was the same in all beams, consisting of eight #3 bars distributed throughout the section, and five 3/8 inch diameter 250K prestressing strands. Shear reinforcement consisted of various combinations of full-depth #3 "U" stirrups placed at 90°, 60°, or 45°, and #2 closed stirrups in the upper and lower web regions placed at 45°. The concrete section used was of a depth of 20 inches with a 2-inch by 20-inch flange. The openings had diameters of 8 or 10 inches, and were spaced regularly at 16 or 20 inches.

The behaviour of the beams is discussed with respect to visual observation and measurements of deflection, reinforcement strain, and concrete strain. Each beam is classified by failure type into the categories of

shear, shear-compression, or flexural to facilitate evaluation of the various shear reinforcement schemes.

6.2 Conclusions

The data and observations obtained as a result of the twelve beam tests led to the following conclusions:

a) The behaviour of the twelve beams with circular web openings was similar in some respects to the behaviour exhibited by the beams tested by Le Blanc⁽³⁾ Linder⁽⁴⁾ and Sauve⁽¹⁰⁾ which contained rectangular and parallelogram shaped openings.

i) The presence of web openings in regions of high shear force tended to make the beams more susceptible to shear failure than similar beams without openings would have been.

ii) The openings decreased flexural stiffness, but in cases where shear reinforcement was sufficient to prevent a shear failure, they had no detrimental effect on the ultimate moment capacity of the section.

b) Geometrically, the effect of the circular web openings on the beams was to shorten the posts and the upper and lower web regions, and to make them stockier by the haunching effect of the curve of the openings. These factors contributed to the lack of shear failures in the posts, and upper or lower web flexural failures.

c) Although initial shear cracks developed at hole sections along a 45° line through the hole centres, the

failure cracks tended to penetrate the upper and lower webs at their minimum sections. Strain gauges placed on the supplementary shear reinforcement in the upper and lower webs suggested that shear stress was high at these minimum sections, but also that it decreased significantly away from them. Thus, shear reinforcement concentrated at such locations would probably be more efficient than if distributed evenly throughout the length of the upper and lower webs.

d) The use of supplementary #2 upper and lower web stirrups increased the shear capacity of the beams by significant amounts. The magnitude of such strength increases was less in cases where the #2 stirrups could not be placed at the minimum sections because of the presence of full-depth inclined stirrups.

e) The contribution towards beam shear capacity of the full-depth #3 stirrups depended greatly on their slope. Those placed at 45° proved most effective due both to their efficiency in resisting diagonal tension, and to their presence at the minimum upper and lower web sections, made possible by their inclination.

6.3 Recommendations

a) Different methods of reinforcing the upper and lower web should be considered on the basis of bridging the minimum sections of these areas. Further test data should be collected to aid in the development of design procedures for

the upper and lower webs above and below circular openings. Considering these areas as compression or tension members seems a reasonable approach, but further study is needed to determine effective shear areas, and the relative amounts of shear force taken by them.

b) Future testing programs should study the shear strength of the posts by inducing the occurrence of post shear failures. This could be achieved by reducing the spacing of the openings or the area of shear reinforcement provided in the posts.

c) A test series using the same amount of prestressing, but proportioned to fail in flexure by rupture of the prestressing strands, should be tested in order to allow further comparison with previous tests in which this was the normal mode of failure.

d) Test beams containing two web openings in each shear span could be tested with shorter shear spans, to study the effects of a post being completely within a shear span in a beam with severe shear reinforcement requirements.

e) To optimize economy and total hole area, combinations of different hole shapes could be used. In low shear force regions rectangular openings could be used, while in high shear areas parallelogram shaped or circular openings would be more efficient. In addition, oval shaped openings should be tested in an attempt to combine the advantages of circular and rectangular openings.

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APPENDIX A
MATERIALS, PROCEDURES, AND APPARATUS

A.1 Materials

a) **Cement**

In order to allow a short curing period, the cement used in all of the beams was Type III, high-early strength Portland Cement.

b) **Aggregate**

The coarse aggregate used consisted of pea-gravel with a maximum size of 3/8-inch. Typical sieve analyses for this material and the sand used are shown in Tables A.1 and A.2. Both aggregates were obtained from the stockpiles in the laboratory, and have been found satisfactory through their use in previous tests.

c) **Concrete Mix**

The mix design used by previous investigators(3)(4) was repeated in the present series.

The proportions comprising this design were, by weight:

Cement	1.0
Sand	2.2
Gravel	1.6
Water	0.4 to 0.5

While these proportions were maintained for each batch, total batch weights were varied to avoid material wastage. The water content of the mix was varied to offset the effect of variations in the moisture content of the aggregate so that a slump of about 3 inches would result. Workability of the fresh concrete was an important consideration due to the crowded arrangement of reinforcement in some areas of the forms.

The nominal strength of the cured concrete was 5000 psi, although the measured strengths ranged from 5300 psi to 6100 psi. These values were based on the tests of two standard cylinders per batch. A third cylinder was cast from each batch and tested on its side to obtain the tensile splitting strength of the concrete. These values ranged from 300 psi to 500 psi. Cylinder data for each beam are presented in Table A.3

d) Prestressing Strand

Each beam contained five seven-wire, 3/8-inch diameter strands of grade 250K, complying with ASTM-A-416 specifications. The initial portion of the manufacturer's

stress-strain curve is shown in Figure A.1.

e) Reinforcement

The non-prestressed longitudinal reinforcement consisted of #3 deformed bars having a measured yield stress of 55 ksi. The full-depth shear reinforcement consisted of #3 deformed bars of the same yield stress bent into a "U" shape by the supplier. The stirrups placed at 60° were produced in the laboratory by re-bending the longer stirrups that were proportioned for placement at 45°.

The smaller upper and lower web stirrups were manufactured in the laboratory from #2 smooth bars. As noted in the discussion, the steel used for these stirrups was not all from the same heat, and had yield strengths of either 44 ksi or 36 ksi.

A.2 Procedures and Apparatus

a) Formwork

The forms used were designed by Linder. They consisted of pairs of ten-foot long sections built up from 1/8-inch plate and small angles, and lined with varnished 1/2-inch plywood. Small truncated plywood cones were attached to the plywood liners with wood-screws and served to hold the styrofoam void-forms in position by engaging sockets cut into the styrofoam blocks. These cones could be

relocated easily without wasting any wood, as the screw-holes were patched to prevent the seepage of water into the form-liners. The forms were bolted together and to a steel channel base which formed the bottom surface of the web. The proper shape was maintained by pipe clamps holding the flange tips at the specified width, and by adjustable braces attached to the forms at the flange tips and to the loading floor.

b) Prestressing Procedure

The shear reinforcement cage was first set up with one half of the form in place between the two reinforced concrete abutments used to transmit the prestress force to the load floor. The prestressing strands were then threaded through the abutments, the form bulkheads, the reinforcement cages, and steel centre-hole dynamometers, and anchored at one abutment with CCL anchoring devices. At the other end they were threaded singly through an electrically driven hydraulic centre-hole jack. The power to the jack was switched off when readings of the strain gauges on the dynamometer indicated that the desired level of prestress had been reached. Another anchoring device was then seated on the strand against the abutment to allow the removal of the jack. When all of the strands had been stressed, the rest of the formwork was assembled and adjusted.

c) Casting Procedure

The concrete was mixed in the laboratory in a nine cubic foot vertical drum mixer. Three batches were required for the longer beams, and two for the shorter ones. The concrete was placed carefully using shovels and a pencil vibrator to avoid any serious honey-combing under the holes and under the prestressing strands. Three cylinders were cast from each batch using the standard procedures for compaction, and then left to set beside the beam.

After about 18 hours the forms were removed and the styrofoam void-forms were punched through the openings. The beam, with the cylinders on its flange, was then covered with wet burlap and plastic tarps, to minimize shrinkage cracking. This condition was maintained for five days, at which time the strands were released, and the beam and cylinders were stored in the laboratory uncovered until the time of the test.

d) Release of Prestress

After the five day moist curing period the strands were cut at one end of the beam. To avoid possible damage by sudden transfer of the prestress force, the strands were evenly heated one at a time over a length of about four feet with an oxy-acetylene torch, to cause relaxation before fracture. This method was apparently effective, as none of the beams failed by loss of bond. When all of the strands

had been released, they were cut off close to the ends of the beam with the torch.

e) **Prestress Losses**

The Demec points used for measuring concrete strain during the tests were mounted on the beam at a section near centerline shortly after removal of the forms. Readings of these were taken before and after release, and just prior to testing. Also just before release, a set of readings of the strain gauges on the dynamometers was taken to determine the strains in the strands after any losses that would occur because of the slippage of the anchorage devices. The changes in strain were used to calculate prestress losses caused by elastic shortening of the beam and inelastic effects such as creep and shrinkage of the concrete, and relaxation of the steel. These calculated values are shown in Table A.4.

f) **Testing Procedure**

Each beam was tested under two point loads applied by hydraulic jacks acting in compression above the beam. The jacks were mounted on a horizontal distributing beam parallel to the test beam, and held above it by two steel frames straddling the test beam. The jacks could be moved to any location on this distributing beam to accommodate the various loading geometries of the beams. Also attached to

each of the two frames was a frame of 5-inch channels arranged so as to restrain the flange tips if the beam tended to tip sideways.

Supports for the beam were provided by two steel roller-wheeled carts riding on steel baseplates bolted to reinforced concrete pedestals. One of these carts was free to roll while the other one was restrained by a threaded rod, thus simulating a simply supported condition.

The loads were applied and controlled with an Ansler hydraulic pump located next to the test setup. When each successive increment of load had been applied, it was maintained at a constant level until the beam reached a stable equilibrium and all of the readings had been recorded. At times it was noticed that the jacks were leaning toward the rolling support due to the elongation of the bottom fibre. In such cases the position of the fixed support was adjusted to minimize this condition by means of the threaded rod. At large deflections, however, the jacks tended to lean excessively towards each other because of the shortening of the top fibre. To avoid damage to the jacks it was sometimes necessary to carefully remove the load, shift the jack heads into new positions on the beam, and then re-apply the load before continuing.

TABLE A.1
SIEVE ANALYSIS OF SAND

SIEVE SIZE	WEIGHT RETAINED (GRAMS)	PERCENT RETAINED.	CUMULATIVE PERCENT RETAINED	A.S.T.M. STANDARD
#4	17.5	3.0	3.0	0-5
#8	85.2	14.7	17.7	-
#16	54.6	9.5	27.2	20-55
#30	60.0	10.3	37.5	-
#50	208.4	35.8	73.3	70-90
#100	122.9	21.1	94.4	90-98
PAN	17.8	3.1	-	-
SILT	14.4	2.5	-	-
TOTAL	580.8	100.0		

FINENESS MODULUS 2.53

TABLE A.2
SIEVE ANALYSIS OF COARSE AGGREGATE

SIEVE SIZE	PERCENT RETAINED	CUMULATIVE PERCENT RETAINED
3/4	0.0	0.0
3/8	5.9	5.9
#4	87.1	93.0
PAN	7.0	100.0
TOTAL	100.0	

TABLE A.3
SUMMARY OF CONCRETE STRENGTHS

BEAM	MIX NO.	AGE (days)	MEAN CYLINDER STRENGTH (psi)	SPLITTING STRENGTH (psi)
AD-1	1	27	5137	407
	2		5429	371
	3		5394	380
AD-2	1	23	5977	380
	2		5924	473
	3		5760	508
AD-3	1	23	4739	398
	2		5977	345
	3		6145	398
AD-4	1	22	5553	539
	2		5146	495
	3		5544	464
AD-5	1	34	4943	451
	2		5933	451
AD-6	1	30	6119	464
	2		5995	429
AD-7	1	15	5491	407
	2		5624	371
AD-8	1	20	5385	371
	2		5447	380
AD-9	1	29	5438	424
	2		5464	398
AD-10	1	36	5460	442
	2		5915	469
AD-11	1	19	5084	314
	2		5562	292
AD-12	1	21	5800	398
	2		6057	442

TABLE A.4
SUMMARY OF PRESTRESS LOSSES

BEAM	INITIAL PRESTRESS (ksi)	ELASTIC LOSS (ksi)	TIME LOSS (ksi)	TOTAL LOSS (ksi)	EFFECTIVE PRESTRESS (ksi)
AD-1	172.7	21.0	22.9	43.9	128.8
AD-2	172.7	19.6	19.2	38.8	133.9
AD-3	172.6	16.5	17.7	34.2	138.4
AD-4	172.2	16.7	20.4	37.1	135.1
AD-5	177.4	16.1	22.0	38.1	139.3
AD-6	177.9	18.6	19.1	37.7	140.2
AD-7	175.8	18.9	16.5	35.4	140.4
AD-8	177.0	22.0	18.2	40.2	136.8
AD-9	176.5	19.1	23.3	42.4	134.1
AD-10	176.5	20.0	26.3	46.3	130.2
AD-11	175.9	17.1	16.1	33.2	142.7
AD-12	175.9	17.7	16.8	34.5	141.4

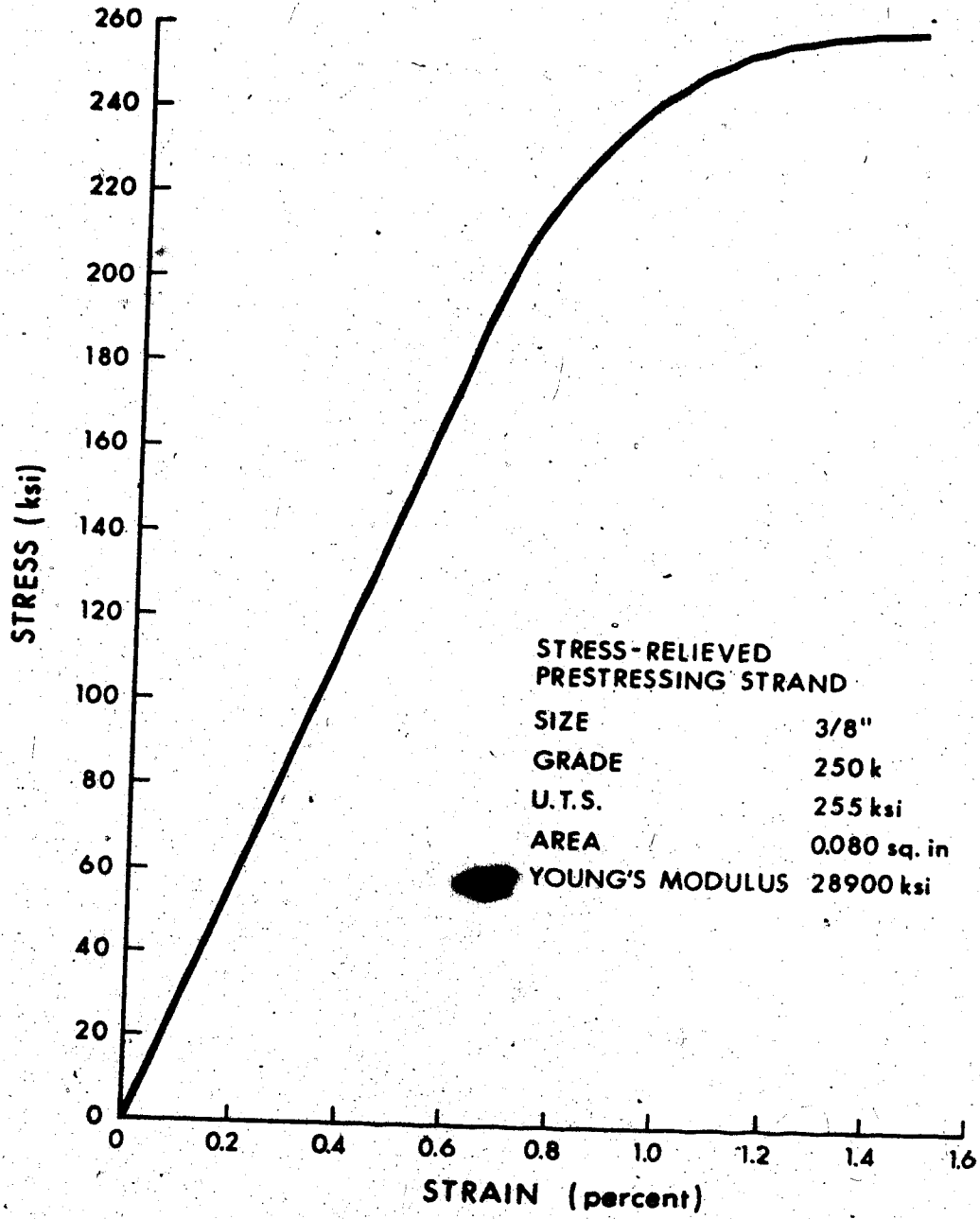


FIGURE A.1
STRAND STRESS-STRAIN CURVE

APPENDIX B

DATA

This appendix contains a drawing of each beam showing the locations of the shear reinforcement and strain gauges, as well as tables of data recorded during the tests. For clarity the readings are referenced to zero applied load, and are given in the basic units of inches for deflection and microinches per inch for strain. Tensile strains and downward deflections are noted as positive.

The electrical strain gauges used had a rated resistance of 120 ohms and a gauge factor of 2.02 for those placed on the prestressing steel, and 2.095 for those placed on non-prestressed reinforcement. The desec points were located at 8-inch gauge lengths at or near beam centerline as depicted in Figure 3.4.

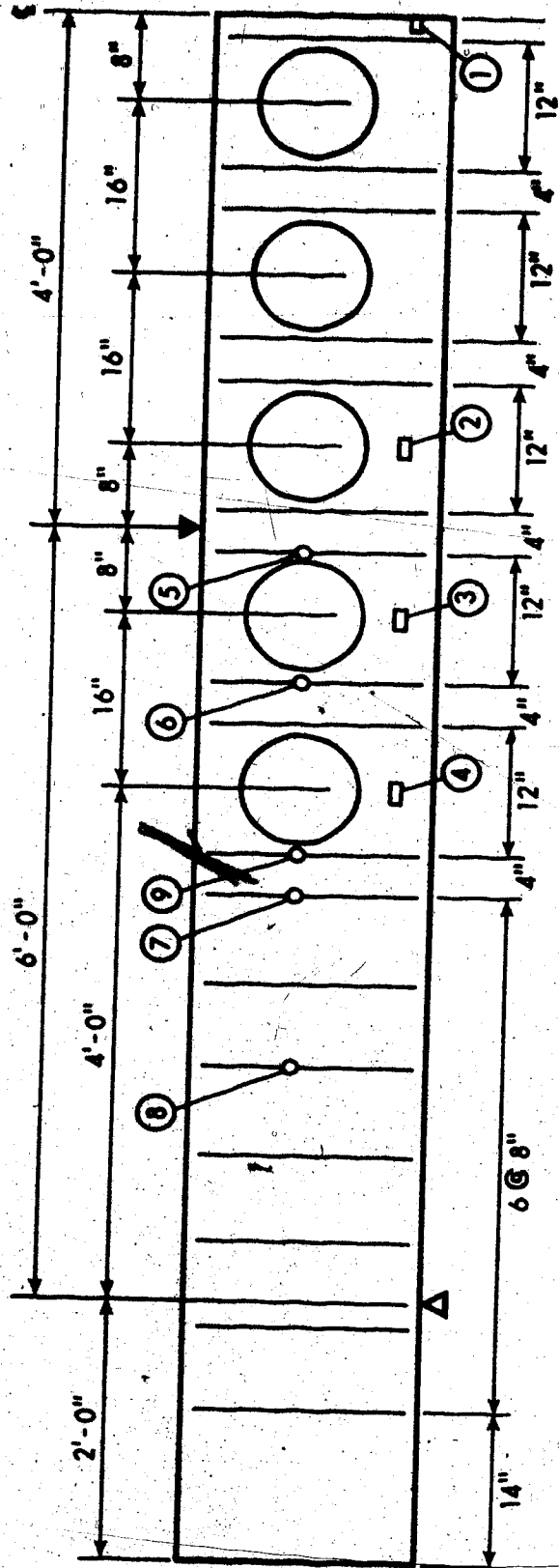


FIGURE B.1 BEAM AD-1 REINFORCEMENT AND GAUGE DETAILS

TABLE B.1.1.1

STRAIN GAUGE DATA BEAM AD-1

LOAD (kips)	(1)	(2)	(3)	(4)
0.0	0	0	0	0
1.0	40	45	35	25
2.0	80	75	65	50
3.0	110	110	100	70
4.0	150	145	130	90
5.0	185	180	160	110
6.0	230	220	200	135
7.0	280	270	245	160
8.0	325	315	285	185
9.0	385	365	340	215
10.0	465	435	410	245
11.0	625	535	525	285
12.0	860	875	750	330
13.0	1075	1185	925	390
13.5	1220	1250	1015	435
14.0	1355	1365	1120	530
14.5	1490	1470	1200	700
15.0	1620	1590	1290	775
15.5	1770	1720	1380	880
16.0	1890	1835	1460	955
16.5	2040	1955	1560	1030
17.0	2165	2065	1650	1110
17.5	2315	2195	1755	1190
18.0	2450	2320	1855	1275
18.5	2650	2480	2000	1365
19.0				

TABLE B.1.1.2
STRAIN GAUGE DATA BEAM AD-1

LOAD (kips)	(5)	(6)	(7)	(8)	(9)
0.0	0	0	0	0	0
1.0	-5	15	5	0	5
2.0	-10	30	5	0	10
3.0	-15	90	5	0	10
4.0	-15	195	10	0	20
5.0	-30	335	25	0	40
6.0	-45	430	35	0	60
7.0	-60	580	50	-5	145
8.0	-70	680	50	-5	190
9.0	-80	780	65	-5	300
10.0	-80	880	70	-5	345
11.0	-70	970	70	-5	545
12.0	-50	1005	75	-5	640
13.0	-15	1015	90	-5	740
13.5	35	1015	95	-5	775
14.0	205	1020	105	-5	830
14.5	570	1035	110	0	850
15.0	620	1035	110	0	890
15.5	655	1045	115	0	925
16.0	690	1050	120	0	960
16.5	740	1055	125	0	1000
17.0	770	1075	125	0	1040
17.5	795	1110	130	0	1060
18.0	825	1165	135	0	1125
18.5	870	1215	140	0	1180
19.0					1265

TABLE B. 1. 2. 1

DEMEC POINT STRAIN DATA (WEB) BEAM AD-1

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1744	1558	1098		519
A.T.	862	804	666		441
0.0	0	0	0		0
1.0	49	49	29		-10
2.0	88	69	49		-20
3.0	147	118	88		-10
4.0	176	147	98		-39
5.0	225	176	108		-39
6.0	265	235	137		-20
7.0	343	265	157		-49
8.0	431	333	186		-49
9.0	519	382	196		-69
10.0	647	470	245		-78
11.0	892	608	284		-69
12.0	1196	813	314		-69
13.0	1470	1029	441		-39
13.5	1637	1117	480		-49
14.0	1813	1274	559		-39
14.5	1960	1392	608		-49
15.0	2107	1490	666		-29
15.5	2264	1627	755		-29
16.0	2411	1744	813		-29
16.5	2577	1872	872		-20
17.0	2744	1980	941		-10
17.5	2920	2146	1009		0
18.0	3067	2264	1068		20
18.5	3273	2430	1137		29
19.0					

B. T. BEFORE TRANSFER

A. T. AFTER TRANSFER

TABLE B.1.2.2
 DEMEC POINT STRAIN DATA (PLANGE) BEAM AD-1

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	431	470	402	382
A.T.	402	441	392	372
0.0	0	0	0	0
1.0	-49	-49	-49	-49
2.0	-69	-78	-78	-78
3.0	-108	-118	-118	-118
4.0	-147	-157	-147	-157
5.0	-176	-176	-186	-186
6.0	-206	-216	-225	-225
7.0	-245	-255	-255	-265
8.0	-274	-294	-294	-294
9.0	-314	-333	-333	-343
10.0	-363	-363	-382	-382
11.0	-402	-421	-431	-431
12.0	-461	-470	-500	-500
13.0	-510	-519	-539	-559
13.5	-549	-549	-568	-588
14.0	-578	-578	-598	-627
14.5	-608	-598	-627	-657
15.0	-637	-598	-666	-696
15.5	-666	-666	-696	-725
16.0	-696	-696	-715	-755
16.5	-725	-725	-745	-794
17.0	-755	-735	-764	-823
17.5	-784	-774	-804	-853
18.0	-813	-794	-833	-882
18.5	-853	-833	-862	-911
19.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.1.3

DEFLECTION DATA BEAM AD-1

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
1.0	0.03	0.03	0.03
2.0	0.05	0.06	0.05
3.0	0.07	0.09	0.07
4.0	0.10	0.12	0.10
5.0	0.13	0.15	0.13
6.0	0.15	0.18	0.15
7.0	0.19	0.22	0.19
8.0	0.22	0.26	0.23
9.0	0.26	0.31	0.26
10.0	0.30	0.36	0.30
11.0	0.37	0.44	0.36
12.0	0.47	0.56	0.48
13.0	0.58	0.66	0.58
13.5	0.65	0.77	0.65
14.0	0.71	0.86	0.71
14.5	0.77	0.94	0.78
15.0	0.83	1.01	0.84
15.5	0.89	1.07	0.90
16.0	0.95	1.15	0.97
16.5	1.02	1.23	1.03
17.0	1.08	1.31	1.10
17.5	1.16	1.40	1.17
18.0	1.22	1.46	1.23
18.5	1.30	1.56	1.31
19.0			

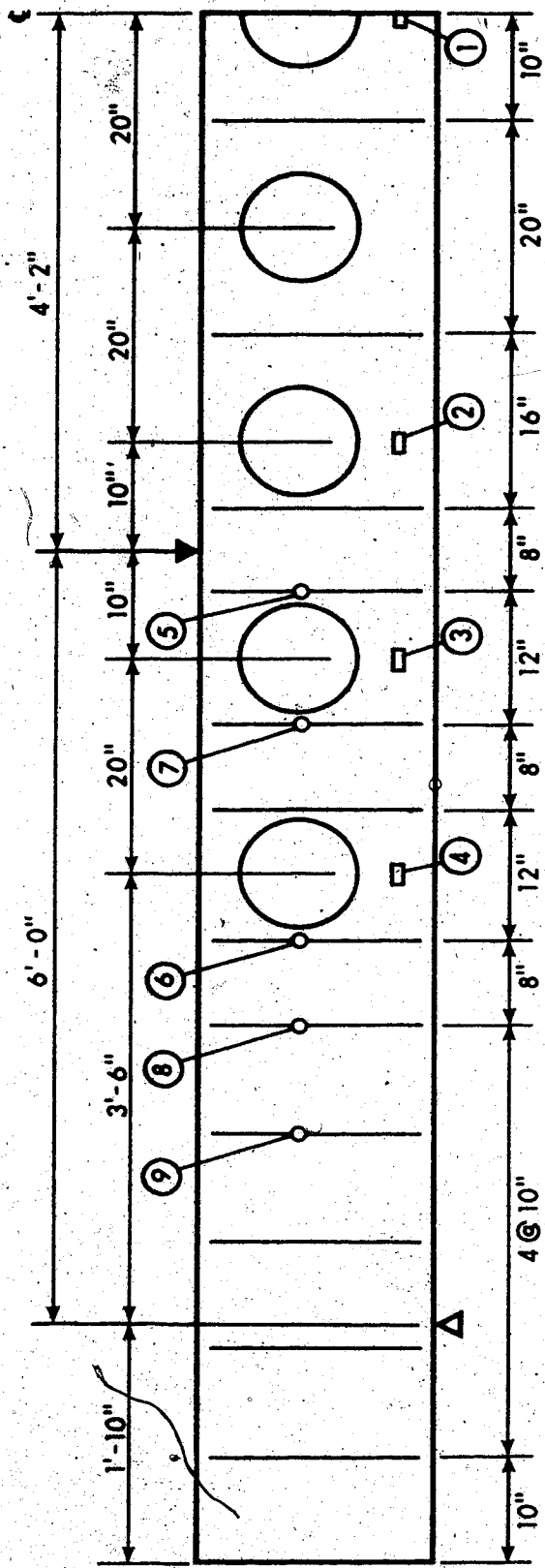


FIGURE B.2 BEAM AD-2 REINFORCEMENT AND GAUGE DETAILS

TABLE B-2.1.1
STRAIN GAUGE DATA BEAM AD-2

LOAD (kips)	(1)	(2)	(3)	(4)
0.0	0	0	0	0
1.0	25	30	20	15
2.0	55	55	45	30
3.0	90	95	70	50
4.0	125	130	100	65
5.0	160	165	130	85
6.0	200	205	160	100
7.0	240	245	190	120
8.0	290	295	225	140
9.0	335	345	265	165
10.0	400	405	315	190
11.0	475	470	370	220
12.0	765	675	480	255
13.0	1100	1088	740	320
14.0	1325	1300	925	385
15.0	1600	1545	1315	600
15.5	1690	1630	1400	670
16.0	1800	1745	1510	755
16.5	1910	1860	1620	900
17.0	2010	1960	1720	945
17.5	2120	2070	1825	990
18.0	2210	2175	1935	1050
18.5	2320	2285	2050	1110
19.0	2430	2400	2160	1170
19.5	2585	2550	2300	1250
20.0				

TABLE B.2.1.2
STRAIN GAUGE DATA BEAM AD-2

LOAD (kips)	(5)	(6)	(7)	(8)	(9)
0.0	0	0	0	0	0
1.0	-5	5	5	0	0
2.0	-10	10	5	0	0
3.0	-20	20	10	0	0
4.0	-25	30	20	5	-5
5.0	-30	50	30	5	-5
6.0	-45	80	45	5	-5
7.0	-60	120	60	10	-10
8.0	-65	185	100	20	-10
9.0	-65	280	270	25	-10
10.0	-60	420	405	40	-15
11.0	-50	565	595	45	-15
12.0	-10	665	705	50	-20
13.0	75	745	780	50	-20
14.0	170	820	820	50	-20
15.0	375	955	910	50	-20
15.5	425	990	940	50	-20
16.0	465	1025	980	50	-20
16.5	520	965	1025	50	-15
17.0	565	995	1085	50	-15
17.5	605	1030	1135	50	-15
18.0	650	1080	1170	50	-10
18.5	690	1120	1200	50	-10
19.0	725	1160	1240	50	-10
19.5	765	1225	1285	45	-10
20.0		1370	1480		

TABLE B.2.2.1
 DEMEC POINT STRAIN DATA (WEB) BEAM AD-2

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1656	1382	941	343	392
A.T.	735	676	539	333	314
0.0	0	0	0	0	0
1.0	20	29	20	0	-29
2.0	59	59	39	-10	-39
3.0	98	88	59	-20	-59
4.0	157	118	88	-10	-59
5.0	206	157	98	-10	-69
6.0	265	196	118	-10	-88
7.0	333	245	147	-10	-88
8.0	392	284	176	-10	-98
9.0	480	353	216	-10	-118
10.0	568	412	225	-20	-147
11.0	715	490	265	-20	-137
12.0	911	588	304	-29	-137
13.0	1274	813	412	-29	-137
14.0	1666	1098	549	-20	-127
15.0	2078	1431	725	0	-118
15.5	2195	1519	804	29	-108
16.0	2381	1656	872	39	-108
16.5	2548	1803	980	69	-108
17.0	2715	1931	1068	98	-88
17.5	2901	2087	1176	137	-88
18.0	3077	2225	1274	147	-88
18.5	3254	2372	1362	186	-78
19.0	3450	2519	1470	216	-69
19.5	2724	2715	1597	255	-69
20.0					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.2.2.2

DEHEC POINT STRAIN DATA (FLANGE) BEAM AD-2

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	402	372	382	412
A.T.	372	333	343	402
0.0	0	0	0	0
1.0	-20	-20	-20	-20
2.0	-39	-29	-39	-39
3.0	-69	-78	-59	-69
4.0	-98	-98	-98	-108
5.0	-127	-137	-127	-127
6.0	-167	-157	-157	-167
7.0	-196	-206	-196	-216
8.0	-235	-235	-216	-245
9.0	-265	-274	-255	-284
10.0	-294	-304	-294	-333
11.0	-333	-343	-333	-372
12.0	-372	-392	-382	-421
13.0	-431	-431	-421	-490
14.0	-480	-480	-480	-549
15.0	-549	-549	-549	-617
15.5	-568	-578	-568	-647
16.0	-598	-608	-598	-686
16.5	-627	-627	-627	-715
17.0	-647	-657	-647	-735
17.5	-686	-706	-696	-794
18.0	-715	-725	-715	-813
18.5	-745	-745	-735	-843
19.0	-774	-784	-774	-872
19.5	-804	-813	-813	-902
20.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.2.3
DEFLECTION DATA BEAM AD-2

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
1.0	0.02	0.02	0.02
2.0	0.04	0.05	0.04
3.0	0.08	0.08	0.07
4.0	0.10	0.11	0.09
5.0	0.13	0.15	0.12
6.0	0.16	0.18	0.15
7.0	0.19	0.22	0.18
8.0	0.23	0.26	0.22
9.0	0.27	0.30	0.25
10.0	0.31	0.35	0.29
11.0	0.36	0.41	0.34
12.0	0.44	0.50	0.41
13.0	0.56	0.65	0.53
14.0	0.67	0.76	0.64
15.0	0.82	0.97	0.79
15.5	0.86	1.03	0.84
16.0	0.92	1.10	0.90
16.5	0.99	1.18	0.97
17.0	1.05	1.25	1.02
17.5	1.12	1.33	1.08
18.0	1.19	1.41	1.15
18.5	1.26	1.50	1.24
19.0	1.32	1.58	1.31
19.5	1.42	1.69	1.41
20.0	1.53	1.81	1.51

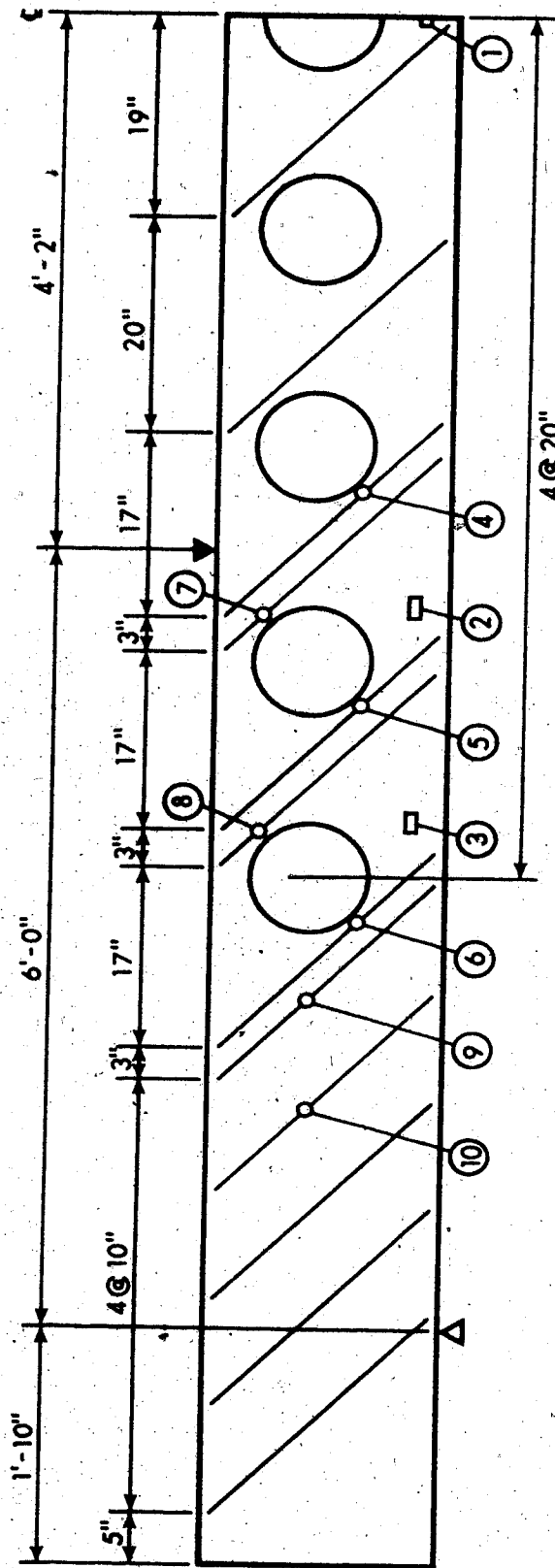


FIGURE B.3 BEAM AD-3 REINFORCEMENT AND GAUGE DETAILS

TABLE B.3.1.1
STRAIN GAUGE DATA BEAM AD-3

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
0.0	0	0	0	0	0
1.0	25	20	15	10	15
2.0	60	45	35	15	40
3.0	90	80	60	20	85
4.0	130	110	80	30	135
5.0	170	140	105	40	215
6.0	215	180	130	45	285
7.0	255	215	150	55	355
8.0	305	255	175	65	435
9.0	350	305	205	75	530
10.0	420	370	235	80	610
11.0	480	435	265	90	670
12.0	640	590	310	95	780
13.0	1000	740	360	95	855
14.0	1350	950	440	140	935
15.0	1590	1110	530	175	990
16.0	1880	1315	680	260	1060
17.0	2090	1510	840	300	1170
18.0	2380	1740	970	340	1310
19.0	2640	1950	1090	410	1415
20.0	3030	2275	1240	435	1520
20.5	3210	2490	1310	460	1580
21.0	3510	2810	1385	475	1650
21.5	3750	3060	1460	490	1720
22.0	4060	3340	1550	520	1790
22.5	4540	3490	1640	540	1860
23.0	5070	3710	1730	560	1930
23.5	6250	3980	1810	565	2000
24.0	7770	4340	1900	565	2060
25.0		5180	2060	540	2080
26.0		6360	2210	540	2035
27.0		6640	2270	490	2010
28.0		7250	2530	485	2070
28.5		8650	2800	460	2060
29.0			2960	460	2040

TABLE B.3.1.2
STRAIN GAUGE DATA BEAM AD-3

LOAD (kips)	(6)	(7)	(8)	(9)	(10)
0.0	0	0	0	0	0
1.0	10	10	15	15	10
2.0	40	25	35	20	10
3.0	75	40	60	30	25
4.0	110	60	85	40	30
5.0	160	85	135	60	40
6.0	205	110	165	65	45
7.0	255	145	215	75	55
8.0	310	180	255	85	65
9.0	375	235	315	95	70
10.0	440	280	360	105	80
11.0	525	355	430	115	90
12.0	610	440	485	120	95
13.0	730	570	560	130	110
14.0	870	700	625	135	115
15.0	1015	815	690	140	120
16.0	1110	915	770	160	135
17.0	1210	1060	925	180	140
18.0	1280	1170	1040	220	155
19.0	1360	1260	1140	270	160
20.0	1450	1360	1235	290	165
20.5	1490	1440	1280	330	175
21.0	1540	1525	1320	345	180
21.5	1580	1630	1370	380	185
22.0	1620	1720	1410	415	190
22.5	1665	1840	1460	455	195
23.0	1710	1940	1510	480	195
23.5	1760	2050	1560	520	205
24.0	1810	2115	1600	550	210
25.0	1890	1940	1685	590	215
26.0	1960	1950	1720	630	230
27.0	1970	2050	1750	670	240
28.0	2040	6200	1820	700	240
28.5	2110		1890	730	260
29.0	2140		1930		

TABLE B.3.2.1

DEMEC POINT STRAIN DATA (WEB) BEAM AD-3

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1490	1215	833	216	294
A.T.	715	627	451	225	235
0.0	0	0	0	0	0
1.0	20	10	0	29	0
2.0	78	59	39	29	-10
3.0	127	98	59	29	-20
4.0	176	127	78	29	-39
5.0	225	157	98	29	-39
6.0	274	206	108	29	-49
7.0	323	245	127	39	-59
8.0	382	294	157	39	-69
9.0	441	333	176	29	-78
10.0	510	392	216	39	-88
11.0	568	451	245	29	-98
12.0	657	519	284	49	-98
13.0	1068	715	363	29	-88
14.0	1637	980	510	39	-88
15.0	1970	1147	608	39	-88
16.0	2381	1411	755	49	-88
17.0	2754	1676	931	49	-78
18.0	3116	1921	1098	69	-78
19.0	3479	2185	1284	78	-69
20.0	3636	2813	1676	98	-49
20.5	4214	3224	1872	108	-29
21.0	5841	3469	2087	118	-10
21.5	6468	3822	2342	127	10
22.0	7183	5204	2607	147	29
22.5	7860	5655	2960	186	39
23.0	8781	6252	3361	196	59
23.5	9898	7085	3940	255	98
24.0	11290	8095	4635	343	147
25.0					
29.0					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.3.2.2

DEHEC POINT STRAIN DATA (FLANGE) BEAM AD-3

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	353	382	382	372
A.T.	333	343	343	353
0.0	0	0	0	0
1.0	0	-20	0	-20
2.0	-20	-20	-29	-39
3.0	-49	-69	-59	-78
4.0	-78	-88	-88	-108
5.0	-98	-118	-118	-137
6.0	-127	-137	-147	-167
7.0	-157	-176	-176	-206
8.0	-176	-206	-206	-235
9.0	-216	-245	-245	-284
10.0	-245	-274	-274	-323
11.0	-274	-304	-304	-353
12.0	-314	-323	-323	-392
13.0	-363	-382	-382	-441
14.0	-421	-431	-431	-510
15.0	-451	-441	-480	-549
16.0	-500	-480	-519	-598
17.0	-539	-529	-578	-666
18.0	-588	-578	-617	-715
19.0	-637	-627	-666	-774
20.0	-715	-696	-735	-843
20.5	-755	-725	-755	-892
21.0	-794	-764	-804	-941
21.5	-833	-813	-853	-990
22.0	-872	-853	-892	-1039
22.5	-931	-902	-951	-1098
23.0	-990	-970	-1009	-1166
23.5	-1078	-1039	-1078	-1245
24.0	-1156	-1117	-1156	-1323
25.0				
29.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.3.3
DEFLECTION DATA BEAM AD-3

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
1.0	0.01	0.02	0.02
2.0	0.03	0.04	0.04
3.0	0.06	0.08	0.07
4.0	0.08	0.11	0.09
5.0	0.11	0.13	0.12
6.0	0.14	0.17	0.15
7.0	0.17	0.20	0.17
8.0	0.20	0.23	0.20
9.0	0.23	0.28	0.23
10.0	0.26	0.31	0.27
11.0	0.30	0.36	0.30
12.0	0.35	0.42	0.36
13.0	0.44	0.53	0.45
14.0	0.56	0.69	0.57
15.0	0.65	0.79	0.65
16.0	0.75	0.92	0.75
17.0	0.85	1.04	0.86
18.0	0.97	1.18	0.97
19.0	1.07	1.30	1.08
20.0	1.22	1.49	1.23
20.5	1.31	1.62	1.33
21.0	1.43	1.76	1.45
21.5	1.54	1.89	1.56
22.0	1.67	2.05	1.68
22.5	1.82	2.25	1.84
23.0	1.99	2.46	2.02
23.5	2.20	2.70	2.23
24.0	2.46	3.07	2.51
25.0	3.47	4.04	3.50
26.0	4.47	5.59	4.67
27.0	5.21	6.66	5.30
28.0	6.08	7.69	6.17
28.5		9.21	
29.0		10.22	

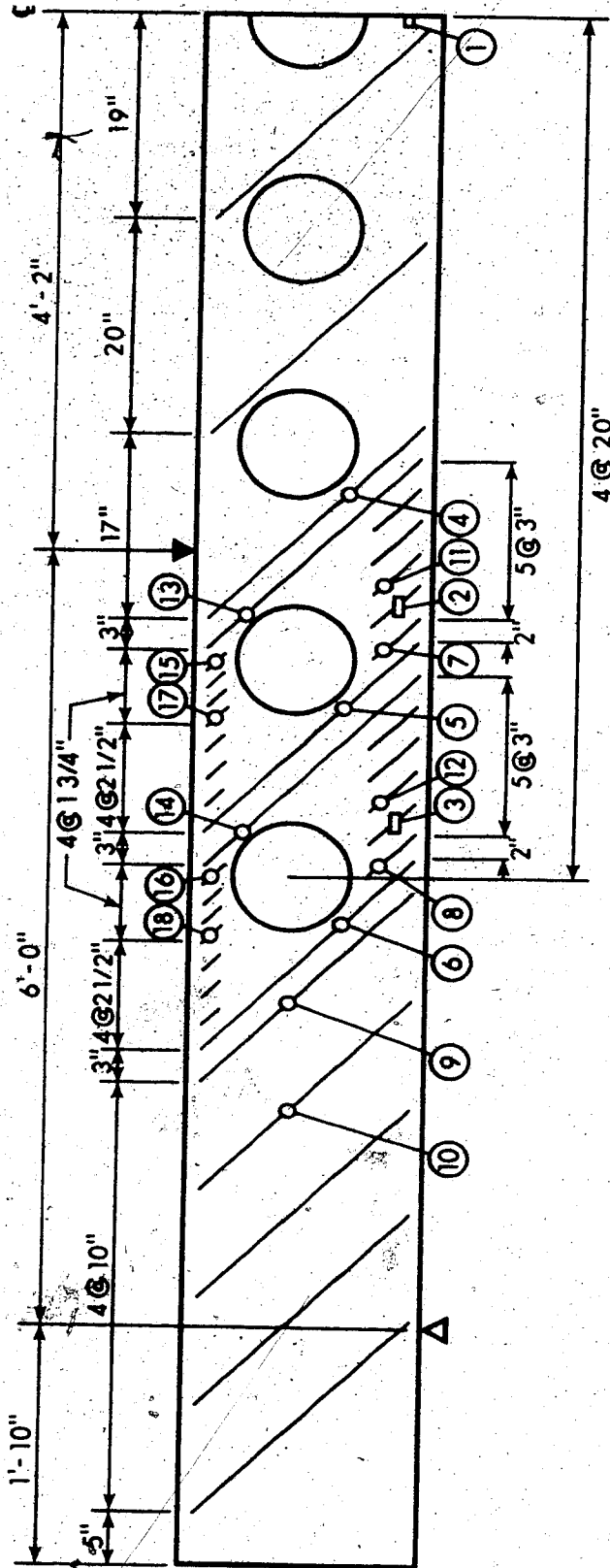


FIGURE B.4 BEAM AD-4 REINFORCEMENT AND GAUGE DETAILS

TABLE B.4.1.1
STRAIN GAUGE DATA BEAM AD-4

LOAD (kips)	(1)	(2)	(3)	(4)	(5)	(6)
0.0	0		0	0	0	0
2.0	60		40	15	45	40
4.0	125		80	30	125	100
6.0	200		130	45	260	180
8.0	290		180	65	430	280
10.0	400		240	90	585	395
12.0	810		325	110	770	580
14.0	1440		465	140	920	830
15.0	1710		565	190	1040	920
16.0	1960		690	260	1120	1000
17.0	2200		800	310	1310	1085
18.0	2440		910	335	1425	1160
19.0	2700		1040	375	1550	1240
20.0	3050		1200	415	1680	1365
21.0	3700		1330	480	1790	1465
22.0	4400		1470	550	1890	1590
23.0	5240		1640	605	1990	1700
24.0	5340		1815	740	2065	1820
25.0			1990	925	2045	1945
26.0			2150	1080	2020	2045
26.0			2190	1095	2030	2100
27.0			2335	1080	2030	2170
28.0			2630	755	2035	2225
29.0			2880	750	2010	2240
29.0				775	1985	2245
29.1						

TABLE B.4.1.2
STRAIN GAUGE DATA BEAM AD-4

LOAD (kips)	(7)	(8)	(9)	(10)	(11)	(12)
0.0		0	0	0		0
2.0		30	20	15		15
4.0		70	40	30		25
6.0		110	60	40		40
8.0		160	85	60		60
10.0		210	105	75		65
12.0		270	120	90		85
14.0		380	140	110		120
15.0		420	155	120		145
16.0		460	260	125		175
17.0		525	340	140		240
18.0		590	410	145		325
19.0		645	455	155		410
20.0		695	525	170		475
21.0		735	580	175		530
22.0		770	635	185		570
23.0		820	690	195		620
24.0		880	775	210		690
25.0		935	845	215		740
26.0		975	990	240		785
27.0		1005	1055	250		790
28.0		1040	1100	260		815
29.0		1040	1125	265		860
29.0		1045	1150	265		970
29.0			1175	275		945
29.1						

TAB; E B. 4. 1. 3

STRAIN GAUGE DATA BEAM AD-4

LOAD (kips)	(13)	(14)	(15)	(16)	(17)	(18)
0.0	0	0	0	0	0	0
2.0	20	30	10	10	0	0
4.0	70	80	10	10	0	0
6.0	175	190	15	15	-20	-15
8.0	310	315	25	15	-30	-15
10.0	470	435	35	15	-40	-25
12.0	685	595	45	20	-55	-35
14.0	910	770	65	25	-65	-40
15.0	1000	880	75	35	-70	-35
16.0	1070	1010	85	40	-75	-40
17.0	1150	1130	85	45	-80	-40
18.0	1230	1235	90	55	-85	-40
19.0	1310	1335	95	60	-85	-40
20.0	1415	1450	105	70	-85	-40
21.0	1555	1555	110	75	-90	-45
22.0	1710	1640	120	85	-95	-45
23.0	1900	1730	130	95	-95	-45
24.0	2130	1830	140	105	-100	-45
25.0	2300	1900	165	120	-110	-45
26.0	2360	1960	225	140	-115	-45
26.0	2410	1990	390	160	-110	-50
27.0	2420	2025	465	175	-115	-45
28.0	2480	2055	590	180	-125	-50
28.0	2495	2085	845	185	-130	-55
29.0	2420	2080	1045		-125	
29.1						

TABLE B.4.2.1

DEMEC POINT STRAIN DATA (WEB) BEAM AD-4

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1597	1323	892	294	363
A.T.	833	725	510	304	323
0.0	0	0	0	0	0
2.0	59	69	29	10	-10
4.0	167	127	78	10	-20
6.0	265	196	118	20	-39
8.0	372	284	167	20	-69
10.0	490	372	216	20	-88
12.0	686	510	284	10	-98
14.0	1333	853	470	-10	-78
15.0	1588	1019	559	-10	-78
16.0	1842	1215	686	0	-59
17.0	2195	1499	804	10	-59
18.0	2352	1744	921	0	-59
19.0	2617	2009	1068	0	-49
20.0	3205	2499	1323	29	-10
21.0	3587	2911	1597	78	39
22.0	4155	3459	1999	196	98
23.0	4929	3930	2538	323	186
24.0	6742	5939	3567	637	304
25.0	9771	8732	5527	1362	598
26.0					
26.0					
27.0					
28.0					
29.0					
29.0					
29.1					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.4.2.2
 DEMEC POINT STRAIN DATA (FLANGE) BEAM AD-4

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	382	441	480	451
A.T.	363	392	431	421
0.0	0	0	0	0
2.0	-39	-39	-29	-39
4.0	-88	-88	-88	-88
6.0	-147	-157	-157	-137
8.0	-206	-216	-216	-216
10.0	-274	-284	-284	-294
12.0	-343	-363	-353	-372
14.0	-451	-451	-441	-490
15.0	-500	-510	-490	-539
16.0	-539	-549	-539	-598
17.0	-588	-598	-588	-657
18.0	-647	-657	-647	-715
19.0	-696	-715	-706	-774
20.0	-784	-794	-784	-862
21.0	-853	-862	-862	-941
22.0	-951	-951	-951	-1049
23.0	-1029	-1039	-1039	-1156
24.0	-1186	-1196	-1205	-1323
25.0	-1450	-1421	-1450	-1578
26.0				
26.0				
27.0				
28.0				
29.0				
29.0				
29.1				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.4.3
DEFLECTION DATA BEAM AD-4

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.04	0.04	0.04
4.0	0.09	0.10	0.09
6.0	0.15	0.16	0.14
8.0	0.20	0.23	0.20
10.0	0.26	0.31	0.27
12.0	0.38	0.45	0.38
14.0	0.59	0.71	0.58
15.0	0.68	0.83	0.69
16.0	0.79	0.96	0.79
17.0	0.90	1.08	0.89
18.0	1.00	1.21	1.00
19.0	1.10	1.33	1.10
20.0	1.29	1.57	1.18
21.0	1.45	1.79	1.46
22.0	1.69	2.06	1.67
23.0	1.98	2.38	1.95
24.0	2.40	3.03	2.50
25.0	3.38	4.00	3.26
26.0	5.46	6.84	5.38
26.0	5.73	7.28	5.72
27.0	6.50	8.13	6.41
28.0	8.06	9.96	7.84
29.0	9.20	11.58	9.12
29.0		12.00	
29.1			

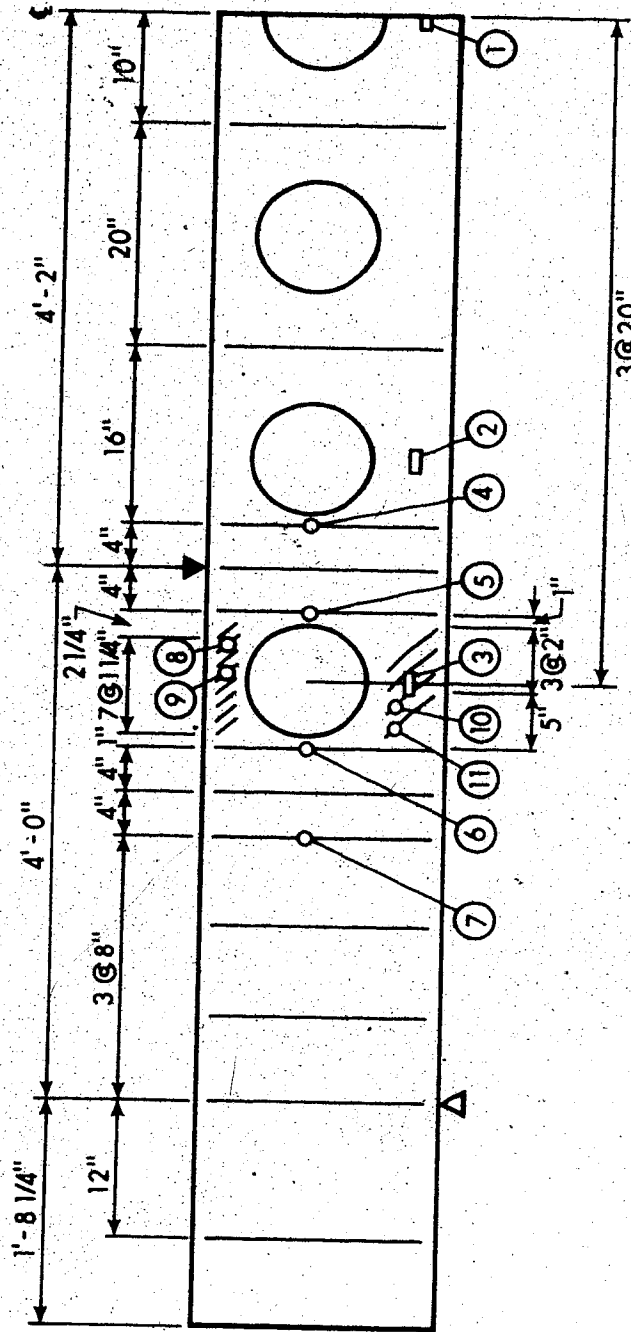


FIGURE B.5 BEAM AD-5 REINFORCEMENT AND GAUGE DETAILS

TABLE B.5.1.1
STRAIN GAUGE DATA BEAM AD-5

BEAM (kips)	(1)	(2)	(3)	(4)	(5)
0.0	0	0	0	0	0
2.0	40	40	30	-5	-15
4.0	80	85	60	-10	-30
6.0	125	125	90	-20	-50
8.0	170	175	120	-15	-105
10.0	215	220	155	-15	-145
12.0	265	275	200	-20	-185
14.0	320	330	250	-25	-210
16.0	390	400	340	-30	-235
17.0	435	440	385	-35	-245
18.0	520	485	450	-40	-250
19.0	625	600	625	-50	-250
20.0	1035	975	725	-50	-250
21.0	1210	1145	815	-60	-245
22.0	1400	1425	925	-75	-215
23.0	1550	1630	1030	-80	-160
24.0	1730	1815	1165	-75	-80
25.0	1880	1965	1300	-80	0
26.0	2050	2125	1440	-80	80
27.0	2200	2275	1600	-75	800
28.0	2380	2460	1790	-70	980
29.0	2550	2635	1990	-80	1155
30.0	2790	2900	2260	-80	1280
31.0	3095	3205	2585	-95	1415
32.0	3650	3700	2930	-115	1560
33.0	4250	4230	3265	-120	1665
34.0	5000	5230	3810	-130	1715
35.0					

TABLE B.5.1.2
STRAIN GAUGE DATA BEAM AD-5

BEAM (kips)	(6)	(7)	(8)	(9)	(10)	(11)
0.0	0	0	0	0	0	0
2.0	15	0	5	10	15	15
4.0	35	5	15	15	35	30
6.0	65	5	25	25	60	50
8.0	160	5	30	15	90	80
10.0	260	15	30	20	130	130
12.0	245	35	35	25	140	190
14.0	310	35	45	30	160	235
16.0	370	30	55	45	185	325
17.0	425	30	60	50	195	375
18.0	440	25	70	60	210	410
19.0	465	25	80	70	245	455
20.0	500	25	110	75	280	500
21.0	580	20	160	85	345	500
22.0	600	20	215	100	400	530
23.0	660	15	300	180	905	735
24.0	840	40	390	290	995	810
25.0	900	45	480	395	1155	955
26.0	965	50	570	500	1270	1045
27.0	1060	50	675	580	1450	1195
28.0	1150	60	740	630	1550	1260
29.0	1270	80	835	715	1630	1320
30.0	1360	100	920	780	1695	1375
31.0	1450	130	995	860	1765	1435
32.0	1570	160	1100	950	1830	1490
33.0	1635	190	1235	1070	1890	1555
34.0	1725	240	1650	1420	1965	1615
35.0						2000

TABLE B.5.2.1

DEMEC POINT STRAIN DATA (WEB) BEAM AD-5

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1676	1352	990	441	519
A.T.	892	774	627	431	470
0.0	0	0	0	0	0
2.0	39	39	20	0	-10
4.0	108	88	39	0	-20
6.0	167	127	69	10	-29
8.0	235	167	88	10	-39
10.0	294	206	127	10	-59
12.0	382	265	137	10	-69
14.0	480	323	167	0	-78
16.0	598	402	196	0	-98
17.0	686	451	225	0	-98
18.0	774	519	255	0	-108
19.0	960	588	294	0	-108
20.0	1431	608	343	0	-118
21.0	1803	637	412	0	-118
22.0	2146	686	441	10	-88
23.0	2489	784	539	29	-88
24.0	2822	902	637	69	-88
25.0	3116	1000	755	137	-69
26.0	3420	1078	853	186	-69
27.0	3704	1186	941	255	-59
28.0	3998	1284	1029	314	-39
29.0	4332	1401	1137	363	-39
30.0	5233	1666	1333	470	10
31.0					
32.0					
33.0					
34.0					
35.0					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.5.2.2

DEHEC POINT STRAIN DATA (PLANGE) BEAM AD-5

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	431	392	431	363
A.T.	441	372	412	382
0.0	0	0	0	0
2.0	-39	-29	-29	-29
4.0	-69	-59	-59	-69
6.0	-98	-88	-98	-98
8.0	-147	-127	-137	-147
10.0	-186	-176	-176	-186
12.0	-225	-206	-225	-225
14.0	-274	-255	-255	-274
16.0	-304	-294	-304	-323
17.0	-343	-323	-323	-353
18.0	-470	-353	-353	-372
19.0	-392	-372	-382	-402
20.0	-431	-402	-412	-441
21.0	-461	-431	-441	-470
22.0	-500	-470	-480	-519
23.0	-539	-500	-500	-549
24.0	-578	-539	-549	-588
25.0	-617	-568	-588	-627
26.0	-647	-598	-627	-657
27.0	-706	-627	-666	-696
28.0	-735	-676	-696	-735
29.0	-774	-715	-735	-784
30.0	-843	-784	-813	-853
31.0				
32.0				
33.0				
34.0				
35.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.5.3

DEFLECTION DATA BEAM AD-5

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.01	0.02	0.02
4.0	0.03	0.05	0.04
6.0	0.05	0.07	0.06
8.0	0.07	0.10	0.08
10.0	0.09	0.13	0.10
12.0	0.12	0.16	0.13
14.0	0.14	0.19	0.16
16.0	0.18	0.24	0.19
17.0	0.20	0.26	0.21
18.0	0.22	0.29	0.23
19.0	0.25	0.33	0.27
20.0	0.31	0.41	0.32
21.0	0.36	0.47	0.37
22.0	0.41	0.55	0.43
23.0	0.46	0.62	0.48
24.0	0.52	0.69	0.54
25.0	0.57	0.76	0.59
26.0	0.62	0.83	0.63
27.0	0.67	0.89	0.69
28.0	0.72	0.97	0.74
29.0	0.79	1.05	0.81
30.0	0.89	1.19	0.91
31.0	1.00	1.34	1.02
32.0	1.13	1.52	1.15
33.0	1.27	1.71	1.29
34.0	1.47	1.96	1.49
35.0		2.38	

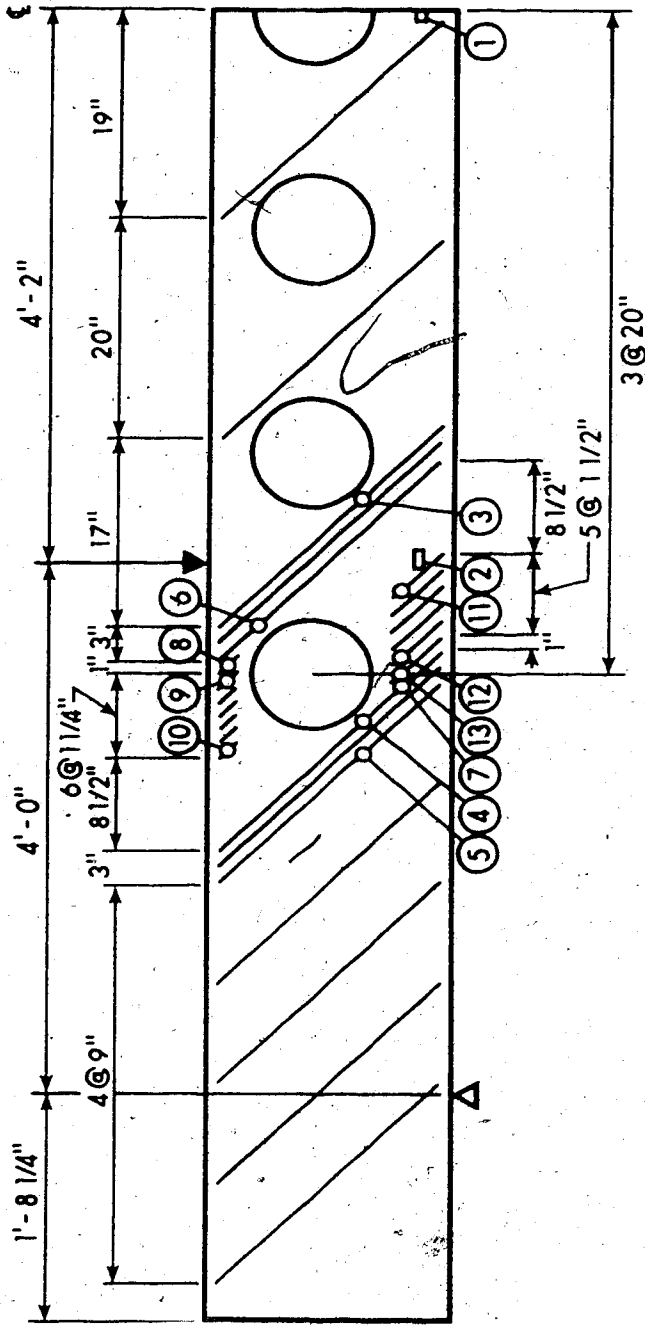


FIGURE B.6 BEAM AD-6 REINFORCEMENT AND GAUGE DETAILS

TABLE B.6.1.1

STRAIN GAUGE DATA BEAM AD-6

LOAD (kips)	(1)	(2)	(3)
0.0	0	0	0
2.0	35	40	5
4.0	80	70	10
6.0	120	115	10
8.0	170	160	20
10.0	220	210	25
12.0	270	260	35
14.0	330	330	45
16.0	400	420	60
18.0	505	590	60
20.0	1050	845	45
22.0	1360	1110	80
24.0	1675	1430	135
26.0	2015	1710	165
28.0	2360	1975	195
30.0	2770	2250	220
32.0	3860	3020	300
34.0	5570	3600	405
35.0	6800	3990	470
36.0	9040	4430	545
37.0	13400	5080	590
38.0	15000	5700	570
39.0		6150	645
39.0			920
40.0			910
41.0			920
42.0			

TABLE B.6.1.2

STRAIN GAUGE DATA BEAM AD-6

LOAD (kips)	(4)	(5)	(6)	(7)	(8)
0.0	0	0	0	0	0
2.0	25	20	20	25	5
4.0	55	40	60	60	10
6.0	100	60	115	90	15
8.0	145	90	185	130	20
10.0	190	130	255	175	30
12.0	240	165	330	230	35
14.0	305	195	410	315	45
16.0	365	230	540	480	60
18.0	550	355	670	745	80
20.0	670	460	810	950	100
22.0	795	580	950	1130	130
24.0	930	690	1060	1310	235
26.0	1050	810	1150	1490	400
28.0	1175	910	1270	1670	535
30.0	1310	1040	1370	1860	610
32.0	1500	1200	1560	2050	725
34.0	1800	1340	1730	2260	820
35.0	1890	1390	1830	2315	875
36.0	1960	1450	2000	2395	940
37.0	2015	1520	3150	2370	1050
38.0	2020	1570	4770	2370	1160
39.0	2020	1625	6250	2365	1290
39.0	2090	1730	9040	2390	1675
40.0	2110	1780	9850	2410	1780
41.0	2120	1840		2400	1990
42.0					

TABLE B.6.1.3

STRAIN GAUGE DATA BEAM AD-6

LOAD (kips)	(9)	(10)	(11)	(12)	(13)
0.0	0	0	0	0	0
2.0	0	0	0	20	20
4.0	5	-5	10	40	40
6.0	5	-10	10	55	60
8.0	10	-10	20	80	80
10.0	10	-20	25	100	105
12.0	15	-20	35	130	130
14.0	20	-25	55	160	180
16.0	30	-30	80	195	215
18.0	45	-35	130	265	260
20.0	60	-40	180	370	350
22.0	80	-40	280	540	450
24.0	120	-45	340	730	565
26.0	190	-50	400	860	665
28.0	270	-35	440	945	735
30.0	330	-30	480	1040	790
32.0	410	-30	500	1145	850
34.0	490	-30	530	1255	900
35.0	570	-20	540	1285	935
36.0	630	-20	540	1325	940
37.0	730	-20	550	1350	950
38.0	840	-25	545	1360	940
39.0	945	-25	560	1375	940
39.0	1230	-10	530	1360	905
40.0	1320	-10	530	1380	910
41.0	1520	-10	530	1420	915
42.0					

TABLE B.6.2.1
 DENEK POINT STRAIN DATA (WEB) BEAM AD-6

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1617	1343	921	333	412
A.T.	715	676	510	323	343
0.0	0	0	0	0	0
2.0	49	49	20	10	0
4.0	108	69	39	10	-20
6.0	167	127	69	10	-29
8.0	235	167	98	10	-29
10.0	304	216	108	10	-49
12.0	382	265	137	10	-69
14.0	451	225	167	10	-78
16.0	549	294	206	10	-98
18.0	666	372	255	10	-108
20.0	1176	617	333	10	-108
22.0	1793	892	441	10	-98
24.0	2528	1225	617	39	-88
26.0	3077	1558	794	49	-78
28.0	3665	1852	1000	78	-69
30.0	4361	2244	1284	108	-59
32.0	5713	3009	1872	225	0
34.0	7438	4047	2724	490	78
35.0	9026	4910	3459	657	137
36.0					
37.0					
38.0					
39.0					
39.0					
40.0					
41.0					
42.0					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.6.2.2

DENEK POINT STRAIN DATA (FLANGE) BEAM AD-6

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	490	500	480	431
A.T.	461	441	441	431
0.0	0	0	0	0
2.0	-10	-29	-29	-20
4.0	-39	-39	-59	-49
6.0	-69	-88	-98	-88
8.0	-98	-108	-127	-127
10.0	-147	-157	-176	-167
12.0	-186	-186	-206	-206
14.0	-216	-235	-255	-255
16.0	-265	-274	-304	-304
18.0	-323	-323	-353	-353
20.0	-392	-402	-421	-431
22.0	-451	-451	-480	-500
24.0	-519	-529	-539	-568
26.0	-578	-578	-598	-637
28.0	-647	-627	-666	-706
30.0	-706	-696	-735	-794
32.0	-853	-843	-872	-941
34.0	-990	-970	-1019	-1098
35.0	-1078	-1058	-1117	-1205
36.0				
37.0				
38.0				
39.0				
39.0				
40.0				
41.0				
42.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.6.3
DEFLECTION DATA BEAM AD-6

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.02	0.02	0.02
4.0	0.04	0.05	0.04
6.0	0.06	0.08	0.06
8.0	0.08	0.10	0.08
10.0	0.10	0.13	0.10
12.0	0.12	0.16	0.13
14.0	0.14	0.19	0.15
16.0	0.17	0.23	0.18
18.0	0.21	0.27	0.21
20.0	0.29	0.38	0.28
22.0	0.37	0.50	0.37
24.0	0.45	0.62	0.45
26.0	0.54	0.74	0.54
28.0	0.62	0.85	0.62
30.0	0.72	0.99	0.73
32.0	0.90	1.24	0.90
34.0	1.15	1.58	1.16
35.0	1.30	1.81	1.32
36.0	1.51	2.08	1.51
37.0	1.92	2.72	1.96
38.0	2.56	3.57	2.58
39.0	2.87	4.09	2.88
39.0	3.41	4.84	3.39
40.0	3.61	5.11	3.58
41.0	4.18	5.92	4.08
42.0		7.12	

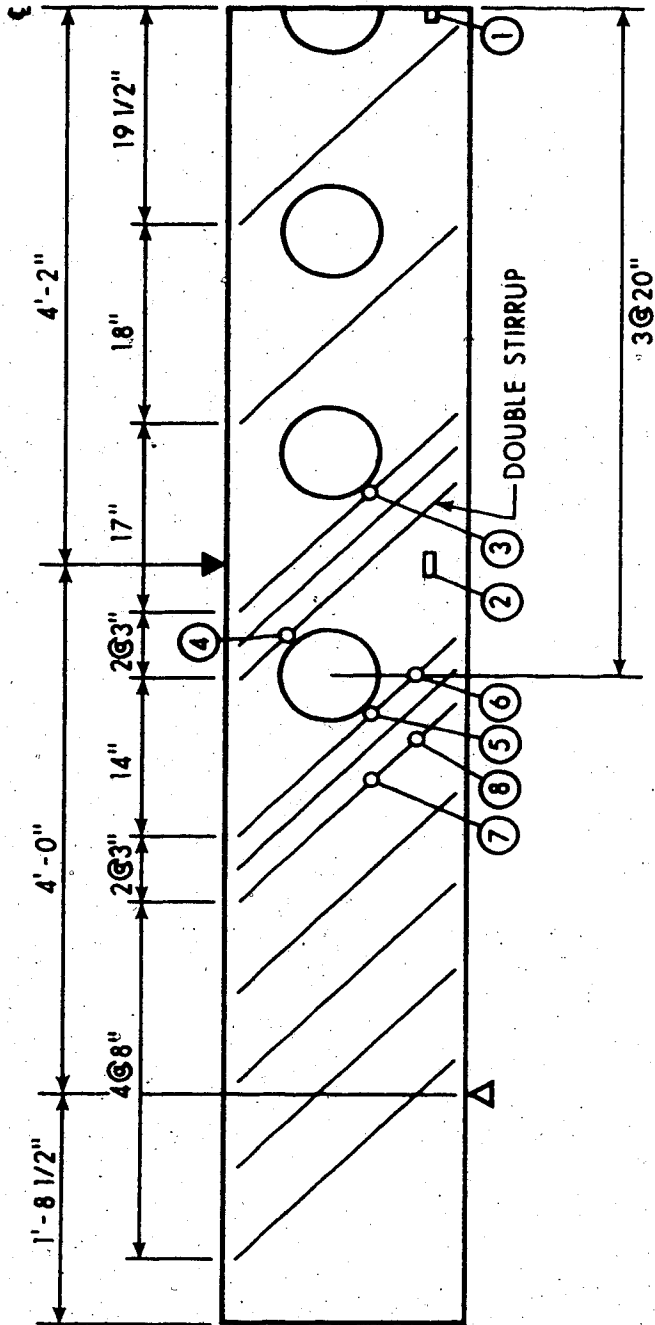


FIGURE B.7 BEAM AD-7 REINFORCEMENT AND GAUGE DETAILS

TABLE B.7.1.1

STRAIN GAUGE DATA BEAM AD-7

LOAD (kips)	(1)	(2)	(3)	(4)
0.0	0	0	0	0
2.0	40	35	0	15
4.0	90	80	5	30
6.0	140	130	5	45
8.0	190	180	10	65
10.0	245	230	15	105
12.0	310	290	20	175
14.0	370	350	30	235
16.0	450	430	35	315
18.0	545	545	40	400
20.0	830	740	15	520
22.0	1205	970	-10	650
24.0	1580	1285	30	800
26.0	1885	1550	75	940
28.0	2245	1825	120	1070
30.0	2620	2090	175	1195
32.0	3745	2415	270	1350
33.0	4430	2770	340	1430
34.0	5075	3010	445	1545
35.0	5990	3250	545	1630
36.0	8000	3550	640	1740
37.0	12700	4005	730	1860
38.0	17500	4410	895	1990
39.0	20000	4890	1140	2115
39.0		5055	1250	2130
40.0		5450	1385	2265
41.0		6000	1465	2500
42.0		6600	1475	2900
42.5				3300

TABLE B.7.1.2
STRAIN GAUGE DATA BEAM AD-7

LOAD (kips)	(5)	(6)	(7)	(8)
0.0	0	0	0	0
2.0	20	30	10	20
4.0	55	60	25	45
6.0	110	95	40	70
8.0	175	130	60	95
10.0	270	170	80	125
12.0	375	210	110	155
14.0	485	255	140	185
16.0	635	300	180	215
18.0	745	365	220	250
20.0	810	460	275	315
22.0	900	680	330	405
24.0	985	950	415	510
26.0	1100	1165	585	595
28.0	1205	1365	690	680
30.0	1340	1590	870	780
32.0	1490	1800	1005	925
33.0	1545	1910	1060	990
34.0	1620	2020	1130	1050
35.0	1700	2120	1190	1105
36.0	1790	2220	1275	1150
37.0	1870	2320	1355	1190
38.0	1930	2400	1445	1220
39.0	1910	2415	1525	1230
39.0	1905	2390	1520	1240
40.0	1915	2420	1570	1260
41.0	1880	2425	1585	1280
42.0	1840	2440	1600	1320
42.5				

TABLE B.7.2.1

DEMEC POINT STRAIN DATA (WEB) BEAM AD-7

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1470	1264	853	216	323
A.T.	637	588	412	157	255
0.0	0	0	0	0	0
2.0	49	39	20	10	-10
4.0	108	88	39	10	-20
6.0	186	127	59	10	-49
8.0	245	196	108	20	-49
10.0	314	245	137	20	-59
12.0	382	314	176	20	-78
14.0	451	363	216	29	-98
16.0	549	461	255	29	-108
18.0	647	568	294	29	-118
20.0	902	862	353	29	-127
22.0	1333	1313	490	20	-118
24.0	1852	1813	666	49	-108
26.0	2313	2254	813	69	-98
28.0	2783	2754	1000	88	-78
30.0	3244	3273	1245	118	-69
32.0	4175	4351	1784	284	-10
33.0	4773	4939	2078	412	20
34.0	5772	5762	2411	568	69
35.0	6791	6546	2715	696	118
36.0	8506	7958	3508	921	196
37.0	11231	10319	5263	1284	294
38.0					
39.0					
39.0					
40.0					
41.0					
42.0					
42.5					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.7.2.2

DEMEC POINT STRAIN DATA (FLANGE) BEAM AD-7

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	245	284	274	196
A.T.	265	284	274	216
0.0	0	0	0	0
2.0	-39	-29	-29	-39
4.0	-69	-69	-69	-78
6.0	-108	-108	-108	-108
8.0	-147	-147	-147	-157
10.0	-176	-186	-176	-206
12.0	-225	-235	-225	-245
14.0	-265	-372	-265	-304
16.0	-314	-323	-314	-353
18.0	-353	-363	-363	-402
20.0	-412	-412	-421	-470
22.0	-490	-480	-490	-549
24.0	-568	-559	-559	-637
26.0	-637	-617	-627	-715
28.0	-706	-686	-706	-804
30.0	-764	-764	-784	-882
32.0	-911	-892	-921	-1029
33.0	-980	-960	-990	-1098
34.0	-1049	-1039	-1068	-1186
35.0	-1127	-1117	-1147	-1284
36.0	-1235	-1235	-1274	-1411
37.0	-1421	-1411	-1460	-1607
38.0				
39.0				
39.0				
40.0				
41.0				
42.0				
42.5				

B. T. BEFORE TRANSFER

A. T. AFTER TRANSFER

TABLE B.7.3

DEFLECTION DATA BEAM AD-7

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.01	0.02	0.02
4.0	0.03	0.04	0.03
6.0	0.05	0.07	0.05
8.0	0.08	0.10	0.08
10.0	0.11	0.14	0.10
12.0	0.13	0.17	0.13
14.0	0.15	0.20	0.15
16.0	0.19	0.24	0.18
18.0	0.22	0.29	0.21
20.0	0.28	0.37	0.26
22.0	0.37	0.48	0.35
24.0	0.45	0.60	0.43
26.0	0.53	0.72	0.51
28.0	0.61	0.82	0.60
30.0	0.71	0.98	0.69
32.0	0.87	1.19	0.84
33.0	0.97	1.35	0.95
34.0	1.09	1.51	1.06
35.0	1.22	1.70	1.20
36.0	1.42	1.94	1.38
37.0	1.79	2.43	1.71
38.0	2.67	3.70	2.52
39.0	3.22	4.53	3.08
39.0	3.53	4.97	3.41
40.0	4.07	5.78	3.98
41.0	4.49	6.39	4.46
42.0	4.96	7.07	4.92
42.5		7.80	

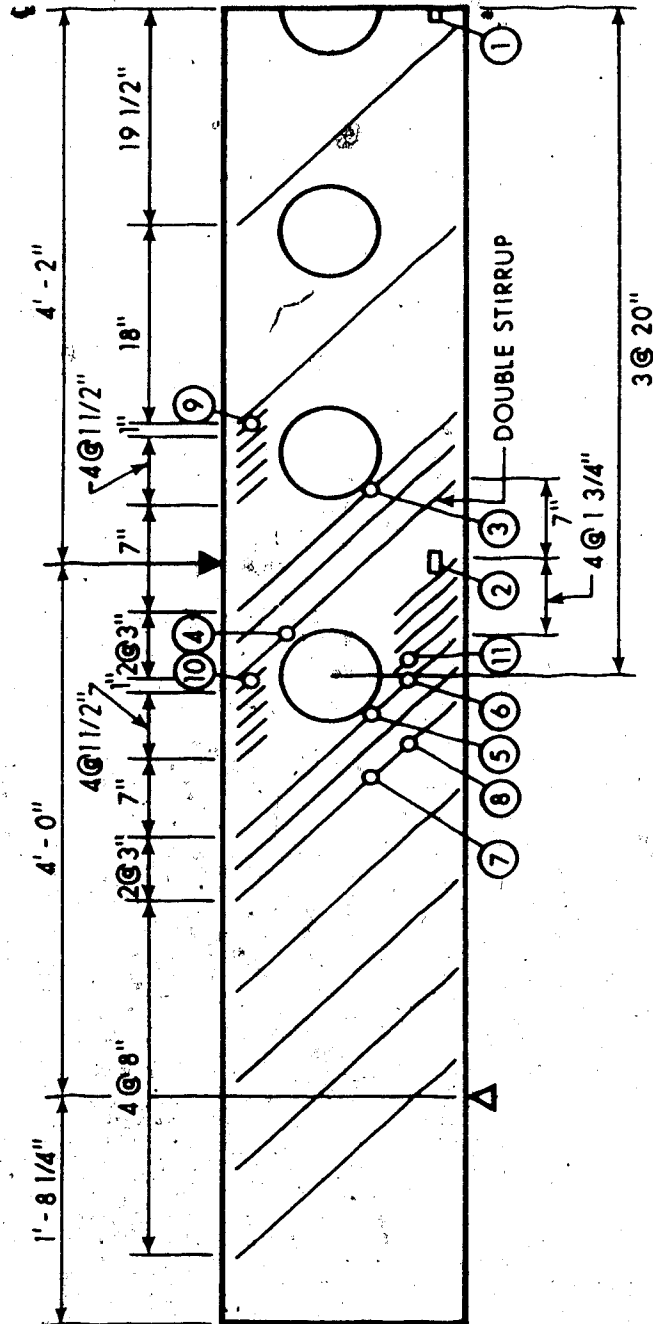


FIGURE B.8 BEAM AD-8 REINFORCEMENT AND GAUGE DETAILS

TABLE B.8.1.1

STRAIN GAUGE DATA BEAM AD-8

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
0.0	0	0	0	0	0
2.0	35	40	0	15	20
4.0	75	80	0	65	50
6.0	125	125	-5	125	85
8.0	170	175	-5	185	125
10.0	225	225	-5	260	180
12.0	280	285	0	340	240
14.0	345	350	0	430	315
16.0	420	430	0	550	515
18.0	525	540	-15	655	645
20.0	975	750	-105	785	710
22.0	1350	1000	-115	915	810
24.0	1675	1240	-95	1025	940
26.0	1985	1510	-45	1125	1065
28.0	2285	1720	-20	1235	1215
30.0	2615	1940	5	1345	1355
32.0	3800	2200	55	1460	1495
34.0	4950	2885	115	1600	1620
36.0	6410	3390	225	1790	1740
38.0	10060	4350	345	2015	1820
40.0	5590	5170	490	2175	1820
40.0	5170	5270	410	2230	1940
42.0		6420	110	2235	1915
43.2					

TABLE B.8.1.2
STRAIN GAUGE DATA BEAM AD-8

LOAD (kips)	(6)	(7)	(8)	(9)	(10)	(11)
0.0	0	0	0	0	0	0
2.0	25	15	15	-10	0	15
4.0	55	25	35	-20	0	35
6.0	85	45	60	-35	-5	55
8.0	120	70	80	-50	-5	80
10.0	150	100	105	-65	-5	105
12.0	190	130	130	-70	0	140
14.0	235	175	165	-90	10	175
16.0	300	240	210	-100	20	210
18.0	380	290	260	-115	30	275
20.0	670	380	345	-130	45	360
22.0	860	595	405	-145	65	465
24.0	1085	750	475	-160	90	630
26.0	1300	880	570	-175	115	745
28.0	1495	1005	765	-180	145	850
30.0	1710	1090	890	-170	180	945
32.0	1930	1210	1000	-80	215	1055
34.0	2140	1360	1085	-5	290	1150
36.0	2330	1470	1135	75	375	1210
38.0	2450	1615	1180	235	480	1250
40.0	2390	1720	1210	250	590	1240
40.0	2355	1910	1100	200	855	1215
42.0	2325	1980	1165	220	945	1230
43.2						

TABLE B.8.2.1

DENEPC POINT STRAIN DATA (WEB) BEAM AD-8

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1695	1421	1078	314	382
A.T.	725	637	549	225	274
0.0	0	0	0	0	0
2.0	59	39	29	10	-10
4.0	118	98	59	0	-29
6.0	186	147	88	10	-49
8.0	255	196	118	10	-78
10.0	333	255	167	10	-88
12.0	412	323	216	10	-108
14.0	500	392	255	20	-118
16.0	637	480	314	69	-137
18.0	951	676	412	78	-157
20.0	1529	1127	578	59	-157
22.0	2127	1627	902	98	-147
24.0	2685	2107	1235	118	-137
26.0	3067	2509	1558	147	-127
28.0	3508	2891	1872	206	-118
30.0	3969	3342	2234	294	-108
32.0	5410	4439	3058	490	-29
34.0	7223	5811	4038	676	39
36.0	9575	7644	5400	892	137
38.0	16768	14465	11143	1784	598
40.0					
40.0					
42.0					
43.2					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.8.2.2

DEMEC POINT STRAIN DATA (PLANGE) BEAM AD-8

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	441	343	333	323
A.T.	451	323	323	333
0.0	0	0	0	0
2.0	0	69	-29	-29
4.0	-29	-69	-59	-59
6.0	-69	-108	-108	-88
8.0	-108	-157	-157	-137
10.0	-157	-196	-196	-176
12.0	-196	-245	-245	-216
14.0	-245	-294	-284	-265
16.0	-284	-343	-343	-314
18.0	-323	-392	-392	-363
20.0	-412	-476	-451	-441
22.0	-470	-529	-519	-519
24.0	-549	-598	-588	-598
26.0	-637	-676	-647	-676
28.0	-696	-755	-725	-755
30.0	-774	-833	-804	-833
32.0	-911	-980	-951	-980
34.0	-1049	-1127	-1078	-1127
36.0	-1205	-1284	-1245	-1294
38.0	-1666	-1735	-1695	-1754
40.0	-2244	-2274	-2215	-2244
40.0	-2283	-2352	-2303	-2352
42.0	-2656	-2734	-2666	-2685
43.2				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.8.3
DEFLECTION DATA BEAM AD-8

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.02	0.02	0.02
4.0	0.04	0.05	0.04
6.0	0.06	0.08	0.06
8.0	0.08	0.11	0.09
10.0	0.11	0.14	0.11
12.0	0.14	0.18	0.13
14.0	0.16	0.21	0.16
16.0	0.19	0.25	0.20
18.0	0.23	0.30	0.23
20.0	0.31	0.42	0.31
22.0	0.39	0.53	0.39
24.0	0.48	0.65	0.48
26.0	0.56	0.76	0.55
28.0	0.63	0.87	0.64
30.0	0.73	1.00	0.73
32.0	0.89	1.25	0.90
34.0	1.12	1.55	1.12
36.0	1.40	1.95	1.41
38.0	2.36	3.32	2.41
40.0	3.79	5.31	3.66
40.0	4.05	5.73	3.96
42.0	5.21	7.21	4.93
43.2		8.32	

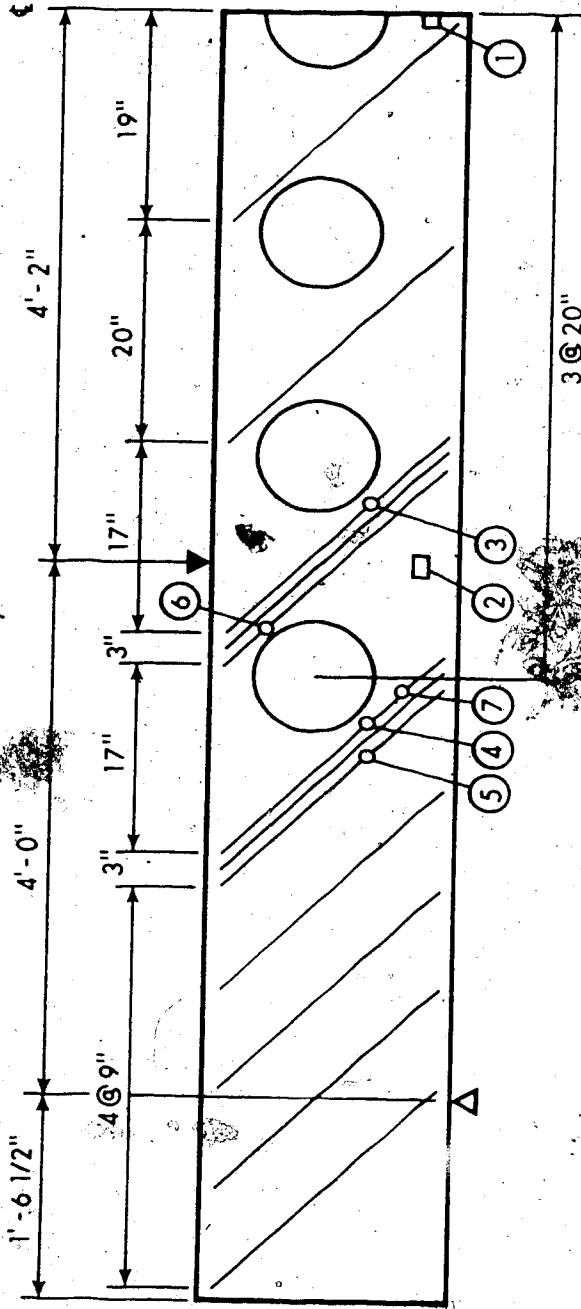


FIGURE B. 9 BEAM AD-9 REINFORCEMENT AND GAUGE DETAILS

TABLE B.9.1.1
STRAIN GAUGE DATA BEAM AD-9

LOAD (kips)	(1)	(2)	(3)	(4)
0.0	0	0	0	0
2.0	30	40	25	60
4.0	80	80	45	135
6.0	130	125	70	270
8.0	185	170	90	425
10.0	240	220	115	585
12.0	310	270	140	715
14.0	380	330	165	850
16.0	540	455	215	990
18.0	935	640	455	1100
20.0	1240	860	535	1190
22.0	1570	1125	595	1295
24.0	1880	1450	655	1405
26.0	2170	1745	725	1540
28.0	2530	2015	795	1675
30.0	2930	2280	845	1845
32.0	3750	3075	950	2055
34.0	4960	3725	985	2270
36.0	6790	4600	965	2375
38.0				

TABLE B.9.1.2
STRAIN GAUGE DATA BEAM AD-9

LOAD (kips)	(5)	(6)	(7)
0.0	0	0	0
2.0	35	35	15
4.0	70	90	40
6.0	125	180	70
8.0	195	300	95
10.0	285	400	140
12.0	355	500	180
14.0	475	615	260
16.0	640	730	430
18.0	765	850	705
20.0	845	950	890
22.0	935	1065	1130
24.0	1010	1170	1335
26.0	1080	1290	1540
28.0	1175	1420	1710
30.0	1310	1510	1915
32.0	1455	1655	2165
34.0	1600	1780	2410
36.0	1755	1865	3600
38.0			

TABLE B.9.2.1
 DEMEC POINT STRAIN DATA (WEB) BEAM AD-9

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1754	1509	1029	343	451
A.T.	862	823	617	363	421
0.0	0	0	0	0	0
2.0	49	29	29	0	-20
4.0	98	88	59	0	-10
6.0	176	118	88	0	-29
8.0	235	176	108	0	-29
10.0	304	245	147	0	-59
12.0	382	294	176	0	-49
14.0	470	353	206	0	-78
16.0	568	431	245	0	-98
18.0	784	598	274	0	-69
20.0	1186	470	314	0	-78
22.0	1558	1166	402	10	-59
24.0	2107	1578	588	20	-69
26.0	2509	1901	774	20	-59
28.0	2940	2264	1000	39	-49
30.0	3391	2636	1215	69	-29
32.0	4655	3587	1784	147	29
34.0	5664	4498	2411	206	157
36.0	7262	6037	3410	333	353
38.0					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.9.2.2

DEMEC POINT STRAIN DATA (FLANGE) BEAM AD-9.

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	451	490	451	412
A.T.	451	470	421	421
0.0	0	0	0	0
2.0	-29	-29	-29	-39
4.0	-78	-59	-69	-69
6.0	-98	-98	-108	-108
8.0	-147	-147	-147	-147
10.0	-176	-186	-186	-186
12.0	-216	-225	-216	-225
14.0	-255	-274	-265	-274
16.0	-294	-314	-314	-333
18.0	-363	-372	-382	-402
20.0	-421	-431	-441	-461
22.0	-500	-500	-500	-539
24.0	-568	-568	-568	-617
26.0	-657	-627	-627	-686
28.0	-715	-706	-706	-755
30.0	-794	-784	-774	-833
32.0	-911	-902	-902	-960
34.0	-1049	-1029	-1029	-1098
36.0	-1254	-1225	-1235	-1303
38.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.9.3
DEFLECTION DATA BEAM AD-9

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.01	0.02	0.01
4.0	0.03	0.04	0.03
6.0	0.05	0.07	0.06
8.0	0.07	0.10	0.08
10.0	0.09	0.13	0.10
12.0	0.12	0.17	0.13
14.0	0.15	0.21	0.16
16.0	0.18	0.25	0.19
18.0	0.25	0.33	0.25
20.0	0.33	0.45	0.33
22.0	0.41	0.56	0.41
24.0	0.49	0.68	0.50
26.0	0.57	0.79	0.58
28.0	0.66	0.91	0.67
30.0	0.76	1.05	0.77
32.0	0.96	1.31	0.96
34.0	1.20	1.64	1.20
36.0	1.54	2.09	1.53
38.0			

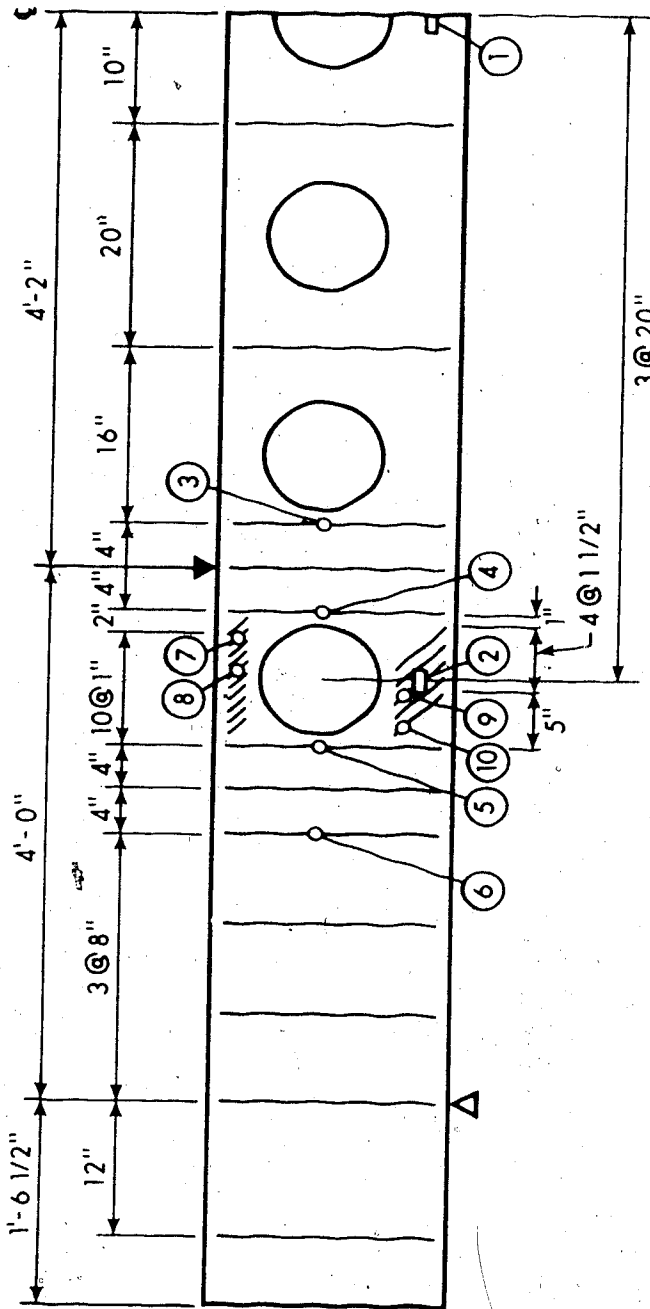


FIGURE B.10 BEAM AD-10 REINFORCEMENT AND GAUGE DETAILS

TABLE B.10.1.1
 STRAIN GAUGE DATA BEAM AD-10

LOAD (kips)	(1)	(2)	(3)	(4)	(5)	(6)
0.0	0	0	0	0	0	0
2.0		30	-5	-15	15	0
4.0		60	-10	-35	25	5
6.0		90	-20	-70	50	5
8.0		130	-20	-115	130	10
10.0		170	-20	-140	250	20
12.0		220	-20	-145	370	35
14.0		280	-20	-105	540	30
16.0		420	-25	20	610	25
18.0		585	-40	185	735	20
20.0		830	-45	465	885	15
22.0		1040	-60	720	1060	5
24.0		1265	-65	875	1205	0
26.0		1525	-65	1030	1360	-5
28.0		1855	-80	1160	1505	-5
30.0		2350	-95	1320	1730	5
32.0		3230	-120	1425	1905	1340
33.0		4750				

TABLE B.10.1.2
STRAIN GAUGE DATA BEAM AD-10

LOAD (kips)	(7)	(8)	(9)	(10)
0.0	0	0	0	0
2.0	5	5	25	15
4.0	15	15	50	35
6.0	25	25	90	65
8.0	35	30	135	100
10.0	45	30	175	145
12.0	50	35	215	185
14.0	65	50	265	335
16.0	120	65	325	550
18.0	210	90	460	665
20.0	325	150	665	885
22.0	455	290	795	1075
24.0	540	390	920	1290
26.0	610	495	1030	1575
28.0	725	635	1115	1760
30.0	885	785	1165	1885
32.0	1055	1020	1235	1980
33.0				

TABLE B-10.2.1

DENEK POINT STRAIN DATA (WEB) BEAM AD-10

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1882	1637	1235	510	617
A.T.	1019	921	784	480	539
0.0	0	0	0	0	0
2.0	39	49	20	0	0
4.0	127	88	49	0	0
6.0	186	137	78	0	-10
8.0	265	196	108	0	-20
10.0	343	255	147	0	-39
12.0	421	333	186	0	-49
14.0	529	402	225	0	-59
16.0	676	510	255	0	-69
18.0	1294	892	480	0	-69
20.0	1872	1333	735	10	-49
22.0	2352	1695	970	20	-59
24.0	2842	2087	1245	69	-49
26.0	3391	2421	3538	206	-39
28.0	3920	2950	1911	421	0
30.0	4851	3704	2499	608	69
32.0	6272	4773	3283	970	206
33.0					

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.10.2.2

DEMEC POINT STRAIN DATA (FLANGE) BEAM AD-10

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	598	588	539	392
A.T.	568	549	490	382
0.0	0	0	0	0
2.0	-39	-29	-29	-29
4.0	-69	-69	-69	-69
6.0	-98	-108	-118	-127
8.0	-137	-157	-167	-167
10.0	-176	-196	-216	-225
12.0	-216	-235	-255	-274
14.0	-265	-294	-314	-343
16.0	-323	-343	-372	-402
18.0	-402	-412	-441	-490
20.0	-470	-470	-500	-559
22.0	-539	-549	-578	-657
24.0	-617	-627	-657	-745
26.0	-706	-725	-755	-843
28.0	-804	-813	-853	-941
30.0	-921	-960	-1009	-1098
32.0	-1098	-1147	-1205	-1274
33.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.10.3
DEFLECTION DATA BEAM AD-10

LOAD (Kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.02	0.03	0.02
4.0	0.05	0.06	0.04
6.0	0.08	0.09	0.06
8.0	0.11	0.13	0.09
10.0	0.13	0.17	0.12
12.0	0.17	0.20	0.15
14.0	0.20	0.24	0.19
16.0	0.25	0.31	0.24
18.0	0.34	0.43	0.32
20.0	0.44	0.57	0.42
22.0	0.54	0.70	0.52
24.0	0.65	0.84	0.63
26.0	0.76	0.99	0.74
28.0	0.88	1.14	0.86
30.0	1.08	1.40	1.06
32.0	1.35	1.75	1.32
33.0		2.17	

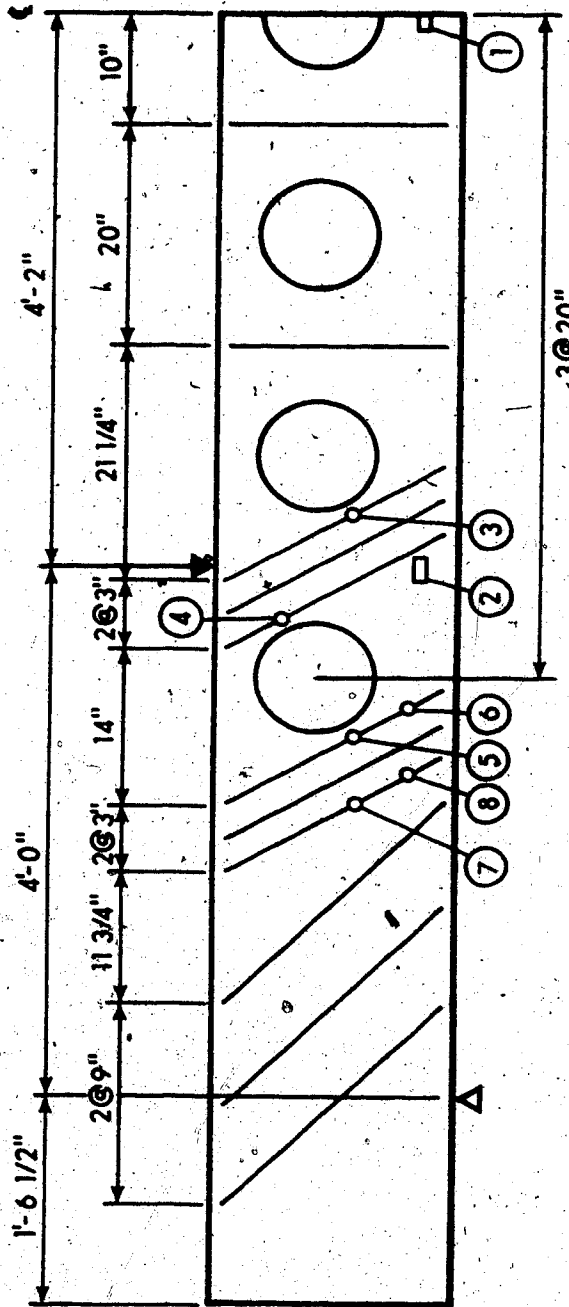


FIGURE B.11 BEAM AD-11 REINFORCEMENT AND GAUGE DETAILS

TABLE B.11.1.1
 STRAIN GAUGE DATA BEAM AD-11

LOAD (kips)	(1)	(2)	(3)	(4)
0.0	0	0	0	0
2.0	48	35	5	25
4.0	85	80	10	100
6.0	140	125	20	205
8.0	190	175	25	345
10.0	250	225	35	475
12.0	305	275	45	580
14.0	375	345	50	650
16.0	475	425	50	790
18.0	700	555	45	965
20.0	1185	815	5	1185
21.0	1350	935	15	1275
22.0	1535	1080	55	1385
23.0	1685	1225	75	1475
24.0	1835	1370	105	1530
25.0	1990	1555	140	1500
26.0	2160	1785	165	1495
27.0	2350	1935	185	1535
28.0	2500	2100	200	1565
29.0	2690	2270	215	1605
30.0	3005	2500	230	1620
31.0	3410	3010	245	1500

TABLE B.11.1.2
 STRAIN GAUGE DATA BEAM AD-11

LOAD (kips)	(5)	(6)	(7)	(8)
0.0	0	0	0	0
2.0	35	10	15	0
4.0	100	30	30	5
6.0	185	50	50	10
8.0	315	70	80	20
10.0	425	95	110	30
12.0	540	125	145	40
14.0	655	215	195	55
16.0	705	540	265	90
18.0	810	810	340	120
20.0	925	1080	455	155
21.0	990	1195	515	170
22.0	1060	1335	560	185
23.0	1125	1445	635	195
24.0	1205	1575	775	215
25.0	1320	1735	875	240
26.0	1435	1895	945	295
27.0	1520	2020	1000	325
28.0	1630	2155	1075	350
29.0	1725	2300	1140	375
30.0	1755	2590	1235	415
31.0	1835			

TABLE B.11.2.1

DEMEC POINT STRAIN DATA (WEB) BEAM AD-11

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1460	185	304		343
A.T.	657	362	487	216	294
0.0	0	0	0	0	0
2.0	59	39	20	20	-10
4.0	118	88	39	40	-20
6.0	176	127	69	10	-29
8.0	245	186	98	10	-39
10.0	314	225	127	20	-59
12.0	392	284	157	20	-59
14.0	470	343	186	20	-69
16.0	559	421	225	20	-78
18.0	715	519	265	20	-88
20.0	882	598	363	20	-98
21.0	1009	676	412	20	-98
22.0	1196	813	461	10	-88
23.0	1352	931	510	20	-88
24.0	1519	1127	598	39	-88
25.0	1705	1303	696	78	-78
26.0	1999	1548	862	147	-78
27.0	2254	1774	1019	225	-69
28.0	2528	1980	1166	304	-49
29.0	2754	2185	1313	372	-29
30.0	2969	2411	1480	441	-10
31.0	3254	2813	1784	578	39

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.11.2.2

DENEK POINT STRAIN DATA (FLANGE) BEAM AD-11

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	294	294	294	235
A.T.	294	265	284	235
0.0	0	0	0	0
2.0	-20	-39	-20	-29
4.0	-39	-69	-69	
6.0	-88	-108	-98	
8.0	-127	-157	-137	
10.0	-167	-196	-176	-186
12.0	-196	-225	-216	-235
14.0	-245	-265	-265	-284
16.0	-284	-314	-314	-333
18.0	-333	-363	-353	-392
20.0	-392	-421	-421	-461
21.0	-421	-441	-451	-510
22.0	-461	-480	-480	-549
23.0	-490	-510	-519	-578
24.0	-519	-549	-549	-627
25.0	-568	-578	-588	-666
26.0	-598	-617	-637	-715
27.0	-637	-666	-666	-755
28.0	-676	-706	-715	-804
29.0	-715	-745	-755	-843
30.0	-755	-735	-794	-892
31.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.11.3
DEFLECTION DATA BEAM AD-11

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.02	0.02	0.02
4.0	0.04	0.05	0.04
6.0	0.06	0.08	0.06
8.0	0.08	0.11	0.08
10.0	0.11	0.14	0.11
12.0	0.14	0.17	0.14
14.0	0.16	0.20	0.16
16.0	0.19	0.24	0.19
18.0	0.23	0.29	0.23
20.0	0.30	0.39	0.30
21.0	0.35	0.46	0.34
22.0	0.40	0.52	0.39
23.0	0.44	0.58	0.43
24.0	0.48	0.64	0.47
25.0	0.53	0.70	0.52
26.0	0.58	0.77	0.57
27.0	0.63	0.84	0.62
28.0	0.68	0.91	0.67
29.0	0.73	0.97	0.71
30.0	0.80	1.06	0.78
31.0	0.91	1.21	0.89

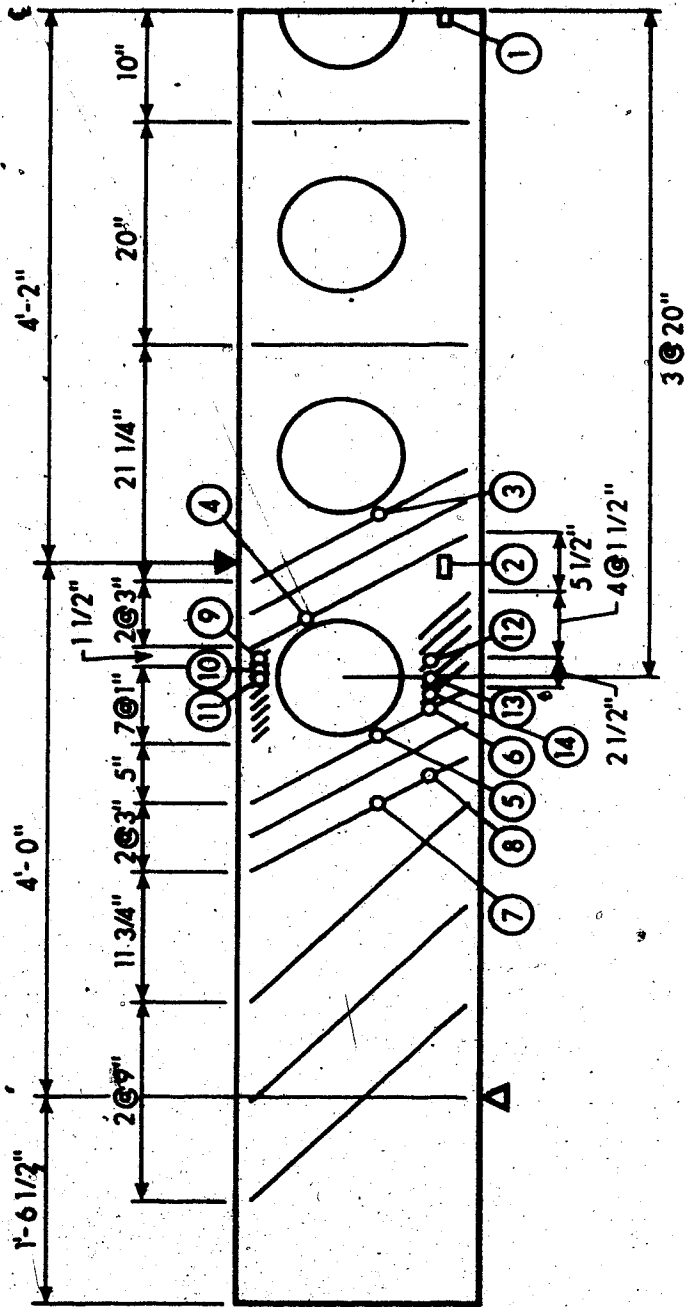


FIGURE B.12 BEAM AD-12 REINFORCEMENT AND GAUGE DETAILS

TABLE B.12.1.1
STRAIN GAUGE DATA BEAM AD-12

LOAD (kips)	(1)	(2)	(3)	(4)
0.0	0	0	0	0
2.0	45	45	5	20
4.0	90	90	0	35
6.0	135	145	0	50
8.0	190	200	0	195
10.0	240	255	-5	340
12.0	305	320	0	480
14.0	365	395	0	625
16.0	455	480	-5	770
18.0	570	590	-10	970
20.0	895	870	-35	1170
22.0	1325	1105	-15	1360
24.0	1685	1405	55	1510
26.0	2005	1740	115	1670
28.0	2360	1985	155	1815
29.0	2535	2075	175	1855
30.0	2860	2455	190	1955
31.0	3390	2815	225	2090
32.0	4035	3035	270	2250
33.0	4460	3190	290	2210
34.0	5070	3530	345	2255
35.0	6050	4030	410	2950
36.0	7450	4375	470	9000
37.0	11200	4865	600	
38.0		5590	715	
38.0		6075	705	
39.0				

TABLE B.12.1.2

STRAIN GAUGE DATA BEAM AD-12

LOAD (kips)	(5)	(6)	(7)	(8)
0.0	0	0	0	0
2.0	30	15	15	10
4.0	60	35	30	10
6.0	105	55	45	15
8.0	195	75	65	20
10.0	445	100	100	40
12.0	625	140	140	55
14.0	775	190	180	80
16.0	910	265	235	110
18.0	950	720	305	160
20.0	1055	985	370	210
22.0	1150	1200	425	250
24.0	1290	1410	510	315
26.0	1450	1625	580	375
28.0	1655	1830	765	440
29.0	1780	1940	805	470
30.0	1890	2040	840	515
31.0	1960	2175	890	560
32.0	2000	2460	970	620
33.0	2035	4000	1030	670
34.0	2050	6200	1100	735
35.0	2060	10000	1220	820
36.0	2050		1450	855
37.0	2030		1590	930
38.0	2025		1675	1000
38.0			1720	1080
39.0				

TABLE B.12.1.3
STRAIN GAUGE DATA BEAM AD-12

LOAD (kips)	(9)	(10)	(11)	(12)	(13)	(14)
0.0	0	0	0		0	
2.0	5	10	5		25	
4.0	10	15	10		45	
6.0	15	20	15		80	
8.0	20	20	15		110	
10.0	25	30	15		150	
12.0	30	35	20		190	
14.0	40	45	30		250	
16.0	50	55	35		310	
18.0	70	75	50		330	
20.0	95	95	70		490	
22.0	130	120	90		770	
24.0	195	175	130		930	
26.0	260	250	190		1040	
28.0	620	615	480		1155	
29.0	745	695	590		1205	
30.0	850	760	645		1250	
31.0	1165	815	685		1310	
32.0	1700	915	735		1375	
33.0		1000	775		1425	
34.0		1100	835		1485	
35.0		1255	925		1550	
36.0		1405	1020		1590	
37.0		1630	1125		1605	
38.0		1925	1205		1585	
39.0		2395	1315		1485	

TABLE B.12.2.1
 DEMEC POINT STRAIN DATA (WEB) BEAM AD-12

LOAD (kips)	(1)	(2)	(3)	(4)	(5)
B.T.	1460	1235	764	255	314
A.T.	657	598	412	235	265
0.0	0	0	0	0	0
2.0	49	29	20	0	-20
4.0	98	78	39	0	-29
6.0	157	127	59	0	-39
8.0	235	176	88	0	-49
10.0	304	225	118	0	-69
12.0	372	274	137	0	-88
14.0	461	343	167	0	-98
16.0	568	412	206	0	-108
18.0	666	529	265	0	-118
20.0	784	676	343	0	-118
22.0	1441	1196	637	0	-108
24.0	1950	1617	902	10	-98
26.0	2372	1989	1166	59	-88
28.0	2832	2401	1480	167	-69
29.0	3067	2617	1627	225	-59
30.0	3352	2960	1000	314	-20
31.0	3861	3450	2332	490	39
32.0	4263	3969	2764	676	108
33.0	4567	4459	3165	843	167
34.0	5135	5096	3714	1078	255
35.0	6321	6145	4547	1450	382
36.0	8604	7909	5860	2068	627
37.0	14661	12867	9663	3793	1284
38.0					
38.0					
39.0					

B. T. BEFORE TRANSFER

A. T. AFTER TRANSFER

TABLE B.12.2.2

DEMIC POINT STRAIN DATA (FLANGE) BEAM AD-12

LOAD (kips)	(6)	(7)	(8)	(9)
B.T.	274	314	284	225
A.T.	265	294	274	255
0.0	0	0	0	0
2.0	-29	-29	-29	-39
4.0	-59	-69	-69	-78
6.0	-98	-98	-108	-118
8.0	-137	-137	-147	-157
10.0	-176	-176	-186	-196
12.0	-225	-225	-235	-245
14.0	-265	-265	-274	-284
16.0	-314	-314	-323	-333
18.0	-353	-353	-382	-392
20.0	-421	-412	-441	-451
22.0	-490	-480	-510	-529
24.0	-559	-539	-578	-608
26.0	-637	-617	-647	-686
28.0	-715	-696	-725	-764
29.0	-755	-735	-774	-813
30.0	-823	-804	-853	-882
31.0	-902	-892	-941	-960
32.0	-980	-980	-1029	-1068
33.0	-1068	-1068	-1117	-1147
34.0	-1156	-1166	-1225	-1235
35.0	-1294	-1313	-1382	-1401
36.0	-1499	-1529	-1607	-1607
37.0	-2048	-2087	-2166	-2176
38.0				
38.0				
39.0				

B.T. BEFORE TRANSFER

A.T. AFTER TRANSFER

TABLE B.12.3
DEFLECTION DATA BEAM AD-12

LOAD (kips)	NORTH (in)	CENTRE (in)	SOUTH (in)
0.0	0.0	0.0	0.0
2.0	0.02	0.02	0.02
4.0	0.04	0.05	0.04
6.0	0.06	0.08	0.06
8.0	0.09	0.11	0.09
10.0	0.11	0.14	0.11
12.0	0.13	0.17	0.13
14.0	0.16	0.20	0.16
16.0	0.19	0.24	0.20
18.0	0.23	0.29	0.24
20.0	0.30	0.40	0.31
22.0	0.40	0.53	0.40
24.0	0.49	0.66	0.49
26.0	0.57	0.78	0.58
28.0	0.67	0.90	0.68
29.0	0.72	0.98	0.73
30.0	0.79	1.08	0.81
31.0	0.89	1.22	0.91
32.0	1.02	1.41	1.04
33.0	1.14	1.58	1.16
34.0	1.28	1.78	1.30
35.0	1.51	2.09	1.53
36.0	1.83	2.56	1.85
37.0	2.72	3.96	2.80
38.0	3.32	4.90	3.39
38.0	3.64	5.31	3.69
39.0		5.76	

APPENDIX C
DESIGN EXAMPLES AND NOTATION

C.1 Material and Section Properties

f'_c	=	5000 psi
f'_{ci}	=	4000 psi
f_{pu}	=	255 ksi
f_y	=	40 ksi
E	=	28100 ksi
E_{ps}	=	28900 ksi
b	=	20 in
b_w	=	4 in
d	=	16 in
y_b	=	12.67 in
y_t	=	7.33 in
A_g	=	114.25 in ²
I_g	=	4583 in ⁴
w_{dl}	=	0.12 kips per foot
M_{dl}	=	72 in kips
f_i	=	174 ksi
f_e	=	140 ksi
P_i	=	70 kips
P_e	=	56 kips

C.2 Maximum Concrete Stresses at Transfer

Top at a support:

$$f_c = \frac{P_i}{A_g} - \frac{P_i (d-y_t) y_t}{I_g}$$

$$= -0.358 \text{ ksi (tension)}$$

Top at centerline:

$$f_c = -0.358 + \frac{M_d l y_t}{I_g}$$

$$= -0.243 \text{ ksi (tension)}$$

Bottom at a support:

$$f_c = \frac{P_i}{A_g} + \frac{P_i (d-y_b) y_b}{I_g}$$

$$= 2.291 \text{ ksi (compression)}$$

Bottom at centerline:

$$f_c = 2.291 - \frac{M_d l y_b}{I_g}$$

$$= 2.092 \text{ ksi (compression)}$$

The allowable concrete stresses at transfer by the ACI 318-71 Code, Section 18.4, are:

$$\text{compression} = 0.6 f'_c = 2400 \text{ psi}$$

$$\text{tension} = 3(f'_c)^{0.5} = 190 \text{ psi}$$

Because the allowable tensile stress is exceeded, reinforcement must be provided to resist the net tensile force on the section:

$$T = 9.1 \text{ kips}$$

$$\text{Steel Area required} = 9.1/f_y = 0.23 \text{ in}^2$$

Four #3 bars in the flange provide an area of:

$$4(0.11) = 0.44 \text{ in}^2$$

C.3 Working Load Moments

The concrete stress in the bottom fibre of the beam can be expressed as:

$$f_c = \frac{P_e}{A_g} + \frac{P_e (d-y_t) y_b}{I_g} - \frac{M_w y_b}{I_g}$$

or, rearranging the terms:

$$\begin{aligned} M_w &= P_e \left[\frac{I_g}{A_g y_b} + d - y_t \right] - \frac{f_c I_g}{y_b} \\ &= 662.8 - 361.7 f_c \end{aligned}$$

Thus for a maximum tensile stress at a working load of:

- 0 , $M_w = 662.8$ in kips
- 424.3 psi, $M_w = 816.3$ in kips (Section 18.4.2(b))
- 530.3 psi, $M_w = 854.6$ in kips (modulus of rupture)
- 848.6 psi, $M_w = 969.7$ in kips (Section 18.4.2(c))

C.4 Ultimate Moment

$$f_{ps} = f_{pu} - \frac{0.5 A_{ps} f_{pu}^2}{b d f'_c}$$

$$= 246.9 \text{ ksi}$$

$$P_{su} = A_{ps} f_{ps}$$

$$= 98.8 \text{ kips}$$

Assuming that the longitudinal reinforcement at depths of 19 and 2.5 inches yields in tension, the total tensile force is:

$$\begin{aligned} T &= 98.8 + 0.44 f_y \\ &= 116.4 \text{ kips} \end{aligned}$$

Thus the depth of the equivalent rectangular stress block is:

$$\begin{aligned} a &= \frac{T}{0.85 b f'_c} \\ &= 1.37 \text{ in} \end{aligned}$$

and the ultimate flexural capacity by the ACI 318-71 Code (assuming a capacity reduction factor of unity) is:

$$\begin{aligned} M_u &= P_s u d + 0.22 f_y (19 + 2.5) - T a/2 \\ &= 1690.3 \text{ in kips} \end{aligned}$$

In addition to the equations prescribed by the ACI 318-71 Code for estimating ultimate flexural capacity, strain-compatibility analyses were done on each of the beams using, for the most part, measured material properties rather than the design values. The stress-strain relationship of the prestressing strand was assumed to be bi-linear, with the inelastic portion extended to 281.25 ksi at 4% strain. Thus the total prestressing force could be expressed as:

$$P_s = 101.23 + 2.2555 (\epsilon_s)$$

where ϵ_s is the effective strain in the prestressing strand (in percent) given by:

$$\epsilon_s = \frac{P_s 100\%}{A_{ps} E_{ps}}$$

and e_s is the estimate of the difference between e_e and the strain in the strand at ultimate load.

Using the measured value of $f_y = 55$ ksi, the force in the longitudinal reinforcement at different depths in the beam was also expressed in terms of e_s :

$$F(x) = A E \frac{x (0.3 + e_s) - 0.3}{d}$$

with a maximum of $A f_y$. Finally the equivalent rectangular stress block parameters were also described:

$$c = \frac{0.3 d}{0.3 + e_s}$$

$$C = -0.85 f'_c (0.85 - 0.005(f'_c - 4)) \cdot c b$$

$$= \frac{-339.5}{0.3 + e_s} \quad (\text{for Beam AD-1})$$

$$a/2 = c (0.85 - 0.005(f'_c - 4))/2$$

$$= \frac{1.884}{0.3 + e_s} \quad (\text{for Beam AD-1})$$

A sample set of calculation results follows for Beam AD-1:

$$P_e = 51.898 \text{ kips}, \quad e_e = 0.45\%$$

$$f'_c = 5.3 \text{ ksi}$$

$$\text{assume } e_s = 2.63\%:$$

$$P_s = 108.18 \text{ kips}$$

$$F(19) = 12.1 \text{ kips}$$

$$F(2.5) = 9.76 \text{ kips}$$

$$F(1) = -14.45 \text{ kips} \quad (\text{compression})$$

$$C = -115.87 \text{ kips}$$

$$\text{Sum of axial forces} = 0.28 \text{ kips}$$

$$\begin{aligned}
 M_u &= -0.643 (115.87) - 1 (14.45) + 2.5 (9.76) \\
 &\quad + 19 (12.1) + 16 (108.18) \\
 &= 1896.2 \text{ in kips}
 \end{aligned}$$

A third estimate of the ultimate moment capacity of the beams is a simplification of the strain compatibility method. assume:

$$f_{ps} = 275 \text{ ksi}$$

$$f'_c = 5 \text{ ksi}$$

and considering only the reinforcement below the prestressing strands:

$$P_s = 0.4 (275)$$

$$= 110 \text{ kips}$$

$$F(19) = 12.1 \text{ kips (assume yielded)}$$

$$C = -122.1 \text{ kips}$$

$$a/2 = \frac{C}{2 (0.85 f'_c) b}$$

$$= 0.718 \text{ in}$$

$$M_u = -0.718 (122.1) + 16 (110) + 19 (12.1)$$

$$= 1902.2 \text{ in kips}$$

C.5 Shear Reinforcement

For a beam with a 16 foot span and 4 foot shear spans:

$$M_{d1} = 46 \text{ in kips (moment at centerline)}$$

$$P_u = \frac{(M_u - M_{d1})}{4 (12)}$$

$$= 34.25 \text{ kips}$$

The critical section for shear stress is just outside the load point where:

$$\begin{aligned} V_u &= 34.25 + 4 (0.12) \\ &= 34.73 \text{ kips} \end{aligned}$$

and:

$$N_u = 1678.6 \text{ in kips}$$

By Section 11.5.1 the nominal shear stress carried by the concrete is:

$$\begin{aligned} v_c &= 0.6 (70.71) + \frac{700 (34.73) 16}{1678.6} \\ &= 274.2 \text{ psi} \end{aligned}$$

Assuming a capacity reduction factor of unity:

$$\begin{aligned} v_u - v_c &= \frac{34730}{4 (16)} - 274.2 \\ &= 268.5 \text{ psi} \end{aligned}$$

Using #3 double leg stirrups, the required spacing at the critical section is:

$$\begin{aligned} s &= \frac{A_v f_y}{(v_u - v_c) b_w} \\ &= 8.2 \text{ in} \end{aligned}$$

Shear reinforcement calculations were also carried out at other sections of beams of both lengths by the method illustrated here, and also using the provision of a higher value of v_c allowed under Section 11.5.2. The results of these calculations suggested that vertical stirrups spaced at 10 and 15 inches would be acceptable in the shear spans of the 16 foot and 20 foot beams respectively.

Because of the lack of information about shear

reinforcement requirements in the presence of circular web openings the proportioning of stirrups in the test beams was done on an intuitive basis. By placing two, three or four stirrups in all posts in the shear spans, average spacing values of less than the design values were maintained, although it was understood that this was not a rationally complete approach.

Tentative calculations for the proportioning of upper and lower web stirrups were carried out assuming a net section approach and then that the upper and lower webs acted as compression and tension members respectively. This last method seemed the most rational although the values used were so arbitrary and indefinite that no further conclusion could be made. The small size of the upper and lower web sections had a more direct effect on the stirrup spacing chosen as it reduced the maximum spacing to the same order of magnitude as the minimum spacing based on ease of concrete placement.

C.6 Notation

- A_g = area of the gross concrete section
- A_{ps} = total area of prestressing strand
- A_v = area of one stirrup
- a = depth of equivalent rectangular stress block
- b = width of compression face (flange)
- b_w = thickness of web

- C** = total compressive force in equivalent rectangular stress block
c = depth to a fibre of zero stress
d = depth of centroid of prestressing strands
E = elastic modulus of non-prestressed reinforcement
E_{ps} = elastic modulus of prestressing strands
e_e = strain in prestressing strand corresponding to effective prestress
e_s = difference between e_s and strand strain at ultimate load
F_s = total force in prestressing strand
F(x) = force in non-prestressed reinforcement located at depth x
f_c = general term for concrete stress
f'_c = compressive strength of concrete
f'_{ci} = compressive strength of concrete at transfer
f_e = effective prestress after losses
f_i = initial prestress after transfer
f_{ps} = calculated stress in prestressing strand at design load
f_{pu} = ultimate strength of prestressing strand
f_y = specified yield strength of non-prestressed reinforcement
I_g = moment of inertia of gross concrete section
M_{dl} = maximum moment induced by beam self-weight

- M_u** = ultimate flexural capacity of a section or moment at a section in a beam supporting its ultimate load
- M_v** = Working load moment
- P_e** = effective prestress force after losses
- P_i** = initial prestress force after transfer
- P_{su}** = force in prestressing strands corresponding to f_{ps}
- P_u** = ultimate jack load
- s** = spacing of shear reinforcement stirrups
- T** = net tensile force on an uncracked gross section at transfer
- V_u** = shear force at a section in a beam supporting its ultimate load
- v_c** = nominal permissible shear stress carried by concrete
- v_u** = nominal total shear stress corresponding to V_u
- W_{dl}** = weight of a one-foot length of beam
- y_b** = distance to bottom fibre from neutral axis of gross concrete section
- y_t** = distance to top fibre from neutral axis of gross concrete section