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BEHAVIOR AND DESIGN OF REINFORCED CONCRETE T-BEAMS

WITH LARGE OPENINGS IN THE WEBS

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ABSTRACT

Fifteen simply supported reinforced microconcrete T-Beams with large openings in the webs were loaded monotonically with point loads to collapse. The variables were: the size of the openings, the type of loading, shear span ratio, and type of special web reinforcement.

The responses of the beams to the applied loads are compared with two control beams and a theoretical prediction.

Beams with their openings reinforced with special web reinforcement as developed in this study behaved very similar to beams without openings. Shear span ratios and size of openings influence the strength of beams with large openings. An empirical formula, which is based on the results of this study, for the calculation of steel area of special web reinforcement was established.

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BEHAVIOR AND DESIGN OF REINFORCED CONCRETE T-BEAMS WITH LARGE OPENINGS IN THE WEBS

CHAPTER I

INTRODUCTION

1.1 General

In modern multistory buildings air conditioning ducts, plumbing, and access openings consume a significant amount of expensive space. Conventionally, the total story height has been increased above the usable story height to provide the necessary space and thus producing an accompanying cost increase. Unless proper provision is made in the design for these openings, there is a risk that each of the subtrades will block out for the openings necessary to fit their installations, and possibly critically weaken the structural frame of the building. Therefore, the designer is often faced with the necessity of providing adequate openings in the structural frame without reducing load capacity or exceeding allowable deflection limits

Unfortunately, to date only a limited amount of research concerned with large openings in reinforced concrete flexural members has been performed.

1.2 Survey of Previous Studies

Until 1962, there was essentially no research which was directly applicable to this problem. Burton (4)* tested to failure two wide shallow reinforced concrete beams subject to uniform loading over an 18-ft. span with a negative restraining moment at one end. The test specimens were identical in all respects except that one of the beams had 7 x 1-3/8 - inch ducts spaced at 12 - inch centers. The results of the tests made on the two beams indicated that:

A. Embedment of ducts did not significantly reduce the beam's load carrying capacity nor impair its performance when compared with the beam without embedded ducts.

B. The major difference in the behavior was that higher stresses developed in the compression steel of the specimen containing the ducts.

C. At failure, the stirrups were more highly stressed in the beam with embedded ducts.

D. Load deflection responses of both beams were almost identical.

Nasser, et al. (7) tested nine rectangular reinforced concrete beams. The flexural reinforcement was 3-# 6 bars (i.e., steel percentage, p = 0.85%) with openings that reduced the cross-section area from 41.7 to 44.4 percent. The beams dimensions were 18 - inches deep 9 - inches wide and 13 feet long, they were supported 12 feet on centers. One beam which had an opening at mid-span, was loaded first in pure bending, then under a concentrated load applied at mid-span of its top chord of the opening and finally under pure bending again until failure

*Number in parentheses refers to references listed in the Bibliography.

took place. The beam failed in shear in its solid portion. A second beam with an opening was loaded with a concentrated load which was moved to eight different positions. Three beams, each containing two openings, were loaded with one concentrated load at the center of the span, and three companion beams, each with one opening, were tested by loading with one concentrated load 6 - inches away from the opening. One of the three companion beams failed due to diagonal tension in the chords and the other two failed due to flexural bending. The purpose of this research was to study the following assumptions which were first proposed by Segner (9) from tests of wide flange steel members with large openings:

1. The top and bottom cross members of the opening are assumed to behave similar to the chords of a vierendeel panel.
2. The cross members of the openings, when they are not subject to transverse loads, have contraflexure points at their mid-span.
3. The cross members carry the external shear in proportion to their cross-section areas.
4. There is a diagonal force concentration at the corners induced by the chord shear, and its magnitude is double the simple shear force.

Baker (3) tested six rectangular reinforced concrete beams, three of them had rectangular openings ranging from 2/3 d to 1/3 d and the remaining members were solid for comparison purposes. Steel percentages were 0.85, 1.62 and 2.61 percent, respectively. A two-point loading system was used to give a constant moment section through the center section where the opening was present. Baker's conclusions were as follows:

A. Openings do not reduce the maximum moment capacity.

B. The openings did reduce the stability or rotation capacity of the beams, and the beams failed suddenly due to a lack of stability in the compression region.

C. Openings did not increase the deflections.

Baker stated:

> "Unless data to the contrary is made available, no openings, even if adequately designed should be allowed in regions where hinging is likely to occur if the structural framework should be over loaded. This statement is valid even if limit analysis has not been applicable to the design of the framework, because it reflects the overall integrity of the structural system and its true collapse load."

Lorentsen (5) tested four identical reinforced T-Beams with openings (12.4 x 71.0 inches) at the mid-span section. The first member was loaded symmetrically so that shear force between the load points was zero. The mode of failure was yielding of the main reinforcement. The second beam was also loaded with two point loads, but unsymmetrically, so that shear force was present over the opening. The beam failed due to yielding of the main reinforcement. The last two beams were loaded at only one of the third points. Both beams failed due to corner cracking. Lorentsen assumed that the bottom chord was subject to tension only. He derived a solution based on the Theory of Elasticity for the influence line of moment at any location in the upper chord.

1.3 Discussion

Burton's (4) conclusion that embedment of ducts will not influence beam strength is based on the following restrictions:

A. The size of the ducts are small.

B. The beam is not a true T-Beam but a rectangular section reinforced for tension only.

C. The ducts are placed in the tension zone.

Unless these restrictions and limitations are understood, faulty design could result.

The beam tests of Nasser, et al. (7) were limited by the following conditions:—

(a) The percentage of the main reinforcement was low (0.82%) for all the beams.

(b) The author did not consider the effects of the size of opening.

(c) Load-deflection responses were not compared.

Lorentsen (5) tested only four reinforced concrete beams, identical in all respects, with main reinforcement percentage equal to 0.8 percent. No load-deflection responses were reported.

In summary, there are many parameters which may influence the strength and behavior of reinforced concrete beams with openings. A list of these parameters would probably include the following:

(a) Type and amount of reinforcement around the openings.

(b) Type of loading (e.g., one-point load, two-point loading, uniformly distributed load, symmetrical or unsymmetrical loading).

(c) Shape of the cross section of beams (e.g., rectangular, T-Beam etc.)

(d) Absolute values of dimensions of the cross section (e.g., d, b, b′).

(e) Materials qualities.

(f) Percentage of steel.

(g) Support condition (simply supported, continuous, etc.).

(h) Load history.

It should be noted that, in a test project designed to study the influence of the above factors and assuming each factor at four levels would require several thousand specimens to establish a reasonable degree of confidence in the design of openings. One may easily conclude from a study of the tests which have been carried out to date that the problem of openings in reinforced concrete beams is as yet poorly understood.

1.4 Current Recommended Practice

At present, the most widely recognized condifying body for design of reinforced concrete structures in this country, the American Concrete Institute, has indicated a dearth of information concerning the subject matter discussed here by offering no specifications, procedures, or recommendations. However, this should not prevent the designer from designing reinforced concrete beams with large openings providing adequate test data are available.

1.5 Objectives and Scope

The primary objectives for this research project were as follows:

(a) To determine the most effective type of reinforcement around the openings.

(b) To determine the amount of reinforcement with respect to the length of openings.

(c) To determine the effect of loading type (e.g., one-point and two-point loading).

(d) To determine the effect of variations in shear span (a/d).

(e) To study the effects of multiple openings.

(f) To study and develop a theoretical approach to provide adequate reinforcement for large openings in reinforced concrete T-Beams.

The scope of the experimental program undertaken has been limited to the observation of load-deflection, moment-rotation responses and failure mechanisms of 15 reinforced microconcrete simple span model T-Beams loaded monotonically with point loads to collapse. All of the model beams tested had a constant beam section.

The depth of the openings as compared to the depth of the beam was limited to a ratio of 0.45 and the vertical location of the openings was constant. The variables were length of the openings, type of loading, and shear span.

1.6 Notations

A_c = Area of concrete

A_s = Area of tensile reinforcement

A_v = Area of shear reinforcement

a = Shear span (length of region of constant shear)

b = Width of cross section

b' = Web width in T-section

C = Compression force pounds

E_s = Modulus of elasticity of steel

f_e = Compressive strength of concrete

f_s = Tensile steel stress

f_v = Stress in web reinforcement

f_y = Yield point of steel

j_d = Internal moment

L = Length of beam

M = Bending moment in-lb

M_y = Yield bending moment in-lb

$n = E_s/E_c$

R = Reaction

S = Spacing of web reinforcement along longitudinal axis of member

T = Force in tension reinforcement

t = Total depth of the section

V = Total shear force

V_c = Shear force carried by concrete

V_u = Total ultimate shear

V_u' = Ultimate shear carried by the special web reinforcement

τ = Shear stress

τ_u = Ultimate shear stress

α = Inclination of web reinforcement

Q = Capacity reduction factor

CHAPTER II

DESIGN OF THE TEST SPECIMENS

2.1 Prototype Analysis

The dimensions of prototype T-Beams may very within wide limits in multistory buildings. Therefore, the dimensions of the T-Beam test specimens for this study were selected to be in the general range of those occurring most frequently in large framed structures. Thus the following constants were preselected:

(a) Width of web to depth ratio (b'/t) of 0.50.

(b) Four inch slab thickness.

(c) Depth span ratio (t/L) of 0.10.

(d) The percentage of main reinforcement (p) of 1.22.

(e) Depth of the opening to depth of the beam ratio of 0.45.

The variables for the program were:

(a) Length of the opening to depth of the beam ratio (limited to 0.5, 1.0, 1.5).

(b) Shear span (a/d) ratio (limited to 3.8, 5.4, 7.2).

(c) Type of load (one-point and two-point loading for one size of opening).

2.2 Structural Engineering Use of Models

(a) General

The concept of using a reduced scale member constructed from the same materials as used in the prototype in the tests of individual structural members has been fairly widely accepted as a sound basis for structural research purposes.

Alami and Ferguson (2) used a series of beams with various scale factors down to 0.2215 in a study of the reliability of model testing techniques. Failure modes, cracking similitude, load-deflection response, and load-strain characteristics were studied and shown to be in good agreement for the various scales with the exception of the cases in which bond stresses had an influence on the failure.

(b) Theory of Models

A model is a device which is so related to a physical system that observations on the model may be used to predict accurately the performance of the physical system of prototype dimensions.

The methods of structural model testing are generally referred to as "Direct" or "Indirect". The direct method, which was used in this research problem, generally employs a true-to-scale model which reproduces the prototype to a constant linear scale with only minor details (which do not affect the structural behavior of the prototype) occasionally omitted. In the direct method of analysis all of the important dimensions of the prototype are reduced by an arbitrary scale factor (f).

Prototype stresses, strain, moment, deflection, etc. may then be determined from observations of the model when it is subjected to the appropriately scaled loadings. The soundness of the procedure is verified by the laws of similitude (1, 6, 8).

In order to establish similarity between prototype and model structures three conditions are necessary and sufficient (2): (a) geometric, (b) kinematic, and (c) mechanical similitudes. The first condition is satisfied if some constant ratio exists between linear dimensions in both structures. The second condition is satisfied if analogous points in the prototype and the model can be located by application of some constant ratio at any time for the entire load cycle. The third condition requires that surface forces and body forces be expressible in terms of some constant ratio.

The relationships of similitude necessary for this study are:

$d_m = f_r \, d_p$	$W_m = f_E \, W_p / f_r$
$\sigma_m = f_E \, \sigma_p$	$U_m = f_r \, U_p$
$\epsilon_m = \epsilon_p$	$E_m = f_E \, E_p$
$P_m = f_E \, P_p$	$\mu_m = \mu_p$

Where

f_E = mechanical scale factor

P = force/unit area

W = unit weight

U = displacement

ϵ = strain

μ = Poisson's ratio

m,p, (subscripts) = model, prototype

f_r = linear scale factor

2.3 Prototype Versus Model Correlations

The linear scale factor (f_r) selected for this study was 0.273 and the mechanical factor (f_E) was considered to be equal to unity (as commonly assumed by many investigators).

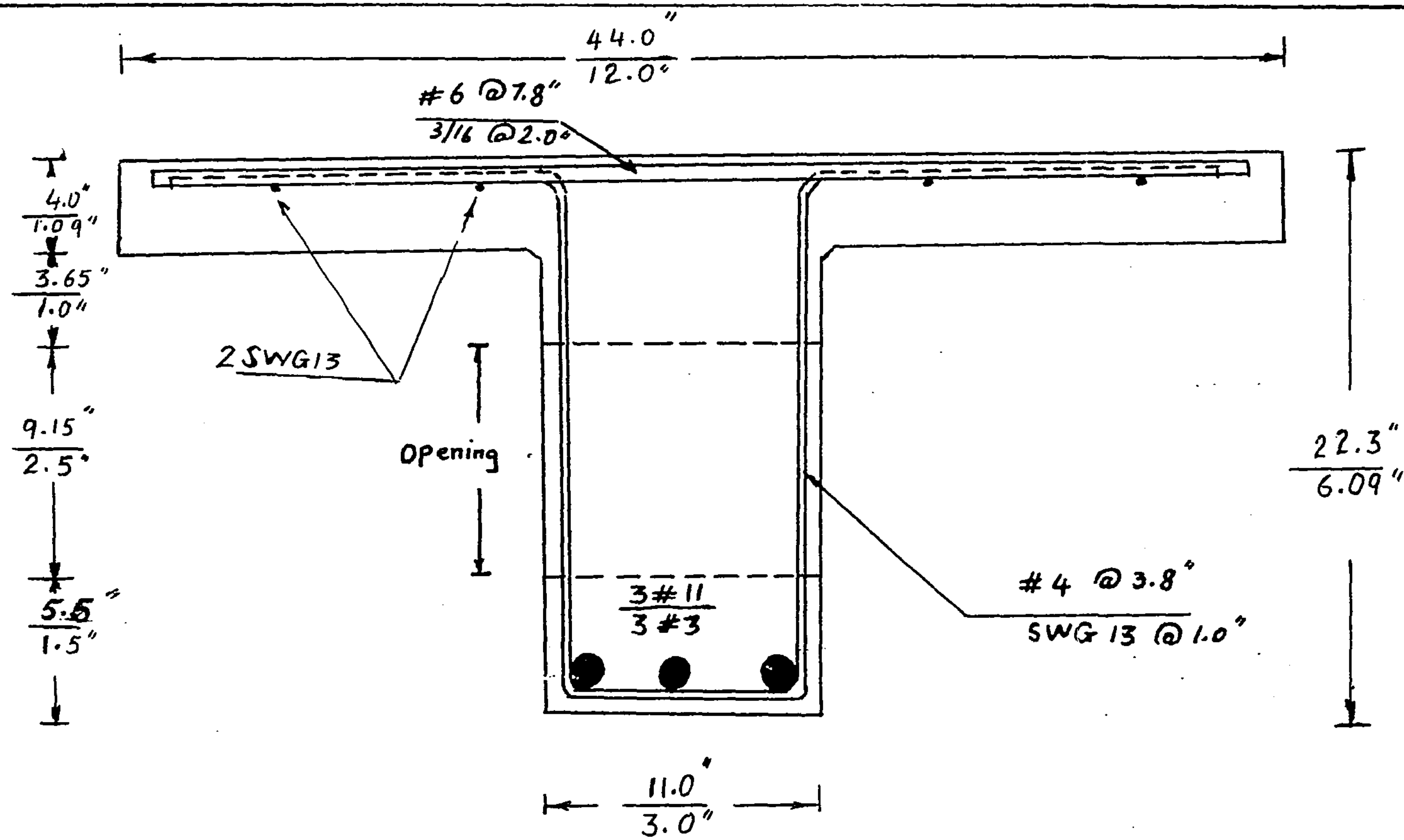
The laws of similitude may be simplified and applied to this study as follows: a variable in the model is equivalent to "K" times the same variable in the prototype, where "K" is given in Table 2.1.

Variable	K
Linear dimension and deflection	0.273
Stresses and strains	1
Concentrated load	0.0745
Dead weight/linear dimension	0.0745
Modulus of elasticity	1

Table 2.1 Similitude factors for this study.

2.4 Model Dimensions and Reinforcement

Following the dimension proportions given in Prototype Analysis the dimensions and reinforcement for a prototype T-Beam were selected. Thus, the model T-Beams were proportioned by applying the linear scale factor of 0.273 to all of the linear dimensions of the prototype. A typical section with dimensions for both the prototype and model is shown in Figure 2.1.



Total Length 22 Ft/6 Ft
Supported Length 18.3 Ft/5 Ft

Figure 2.1 Typical Cross-Section Showing Reinforcement and Dimensions. The top and Bottom Dimensions are for the Prototype and Model Respectively.

CHAPTER III

EXPERIMENTAL PROGRAM FOR THIS STUDY

3.1 General

Fifteen reinforced microconcrete simply supported T-Beams having large openings were constructed and loaded monotonically to collapse with concentrated type loadings.

Hydraulic rams were used for applying loads. After application of each loading increment, the load was maintained constant as vertical and rotational displacements were recorded. Cracking of the beams was observed and marked for each load increment. Sequential cracking information was recorded by means of photographs of the beams.

3.2 Model Construction

(a) The Form Work

The form work for T-Beams was made of plywood covered on the inside by sheet metal in order to keep the moisture in the concrete and away from the plywood. The form was cleaned prior to casting and was oiled with commercial form oil. Rubber pads were used as "block outs" for the opening as shown in Figure 3.8 d.

(b) The Reinforcement

The reinforcement was tied according to the patterns shown in Figures 3.1 through 3.8. The reinforcement around the openings was

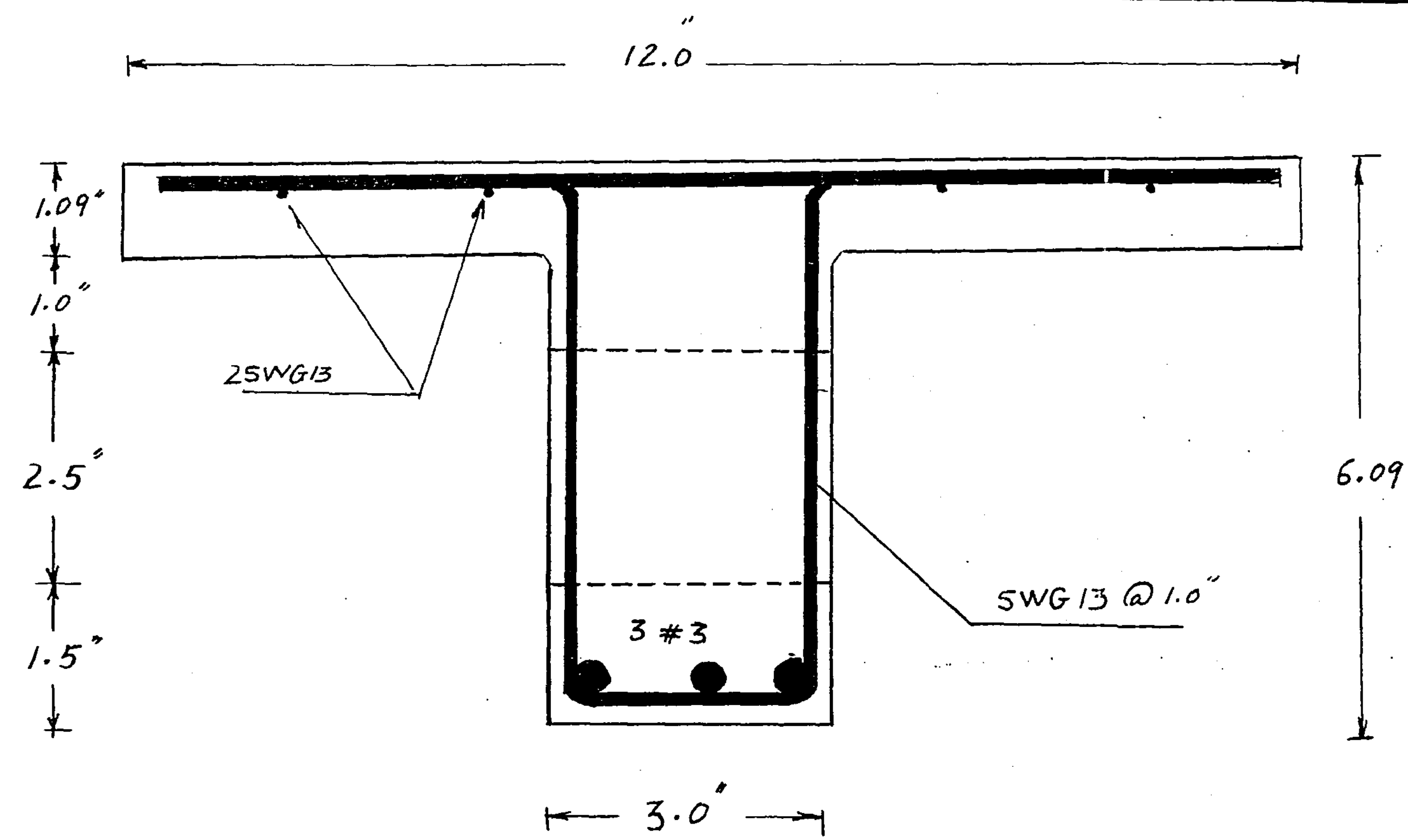


Figure 3.1 A Typical Cross Section for all Beams

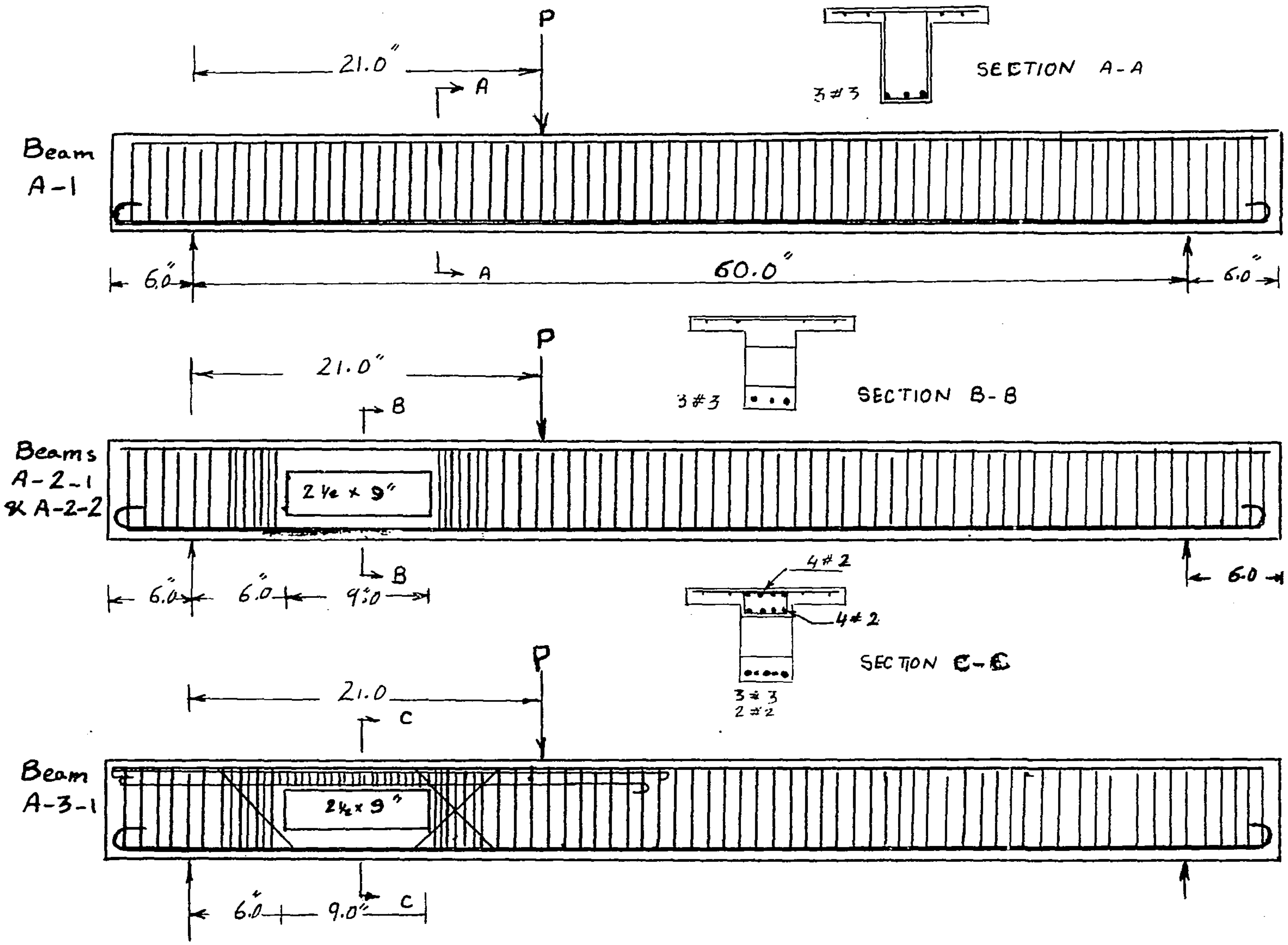


Figure 3.2 Beams Type A Reinforcement and Dimensions

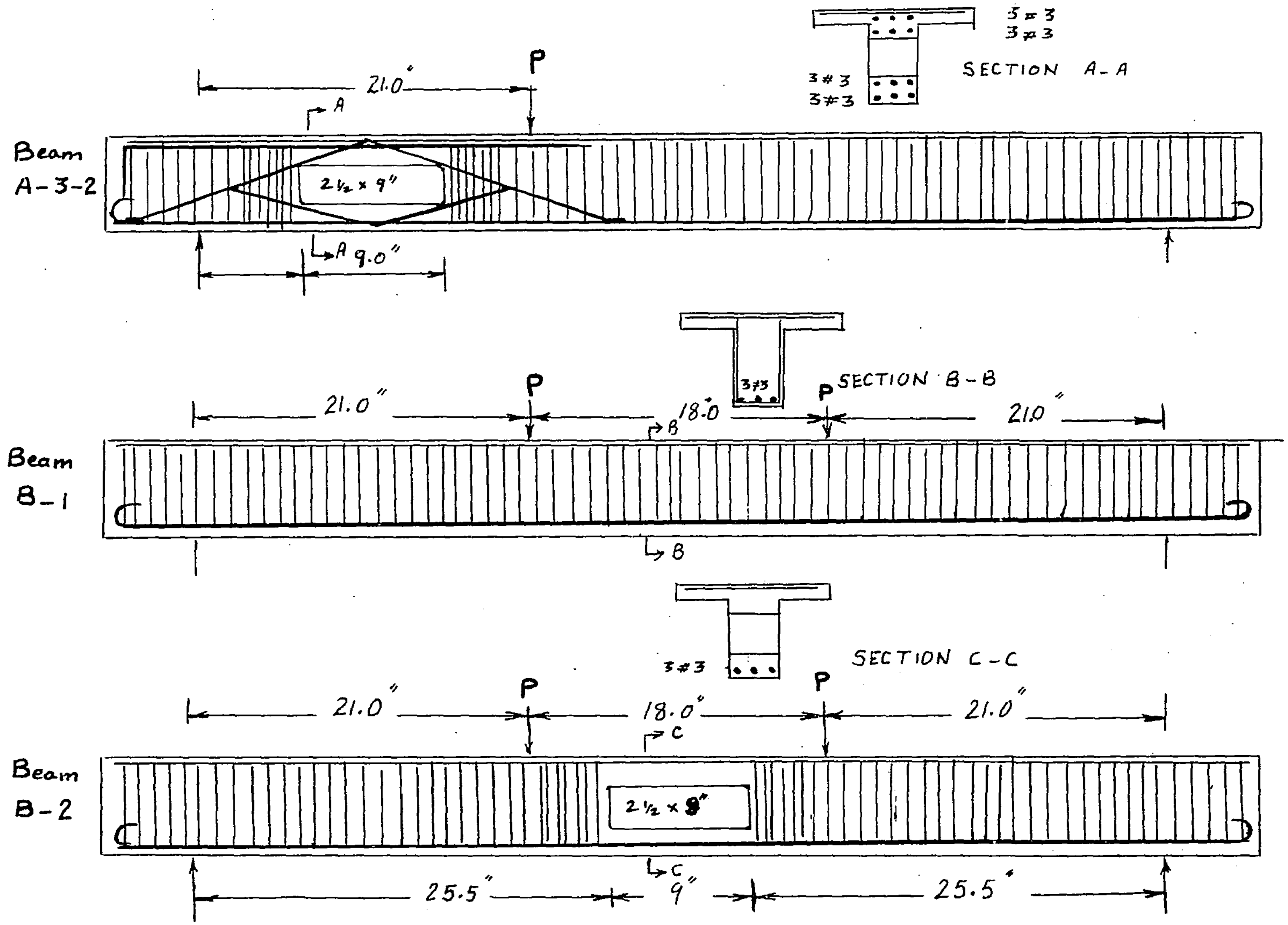


Figure 3.3 Beams Type B and Beam A-32 Reinforcement and Dimensions

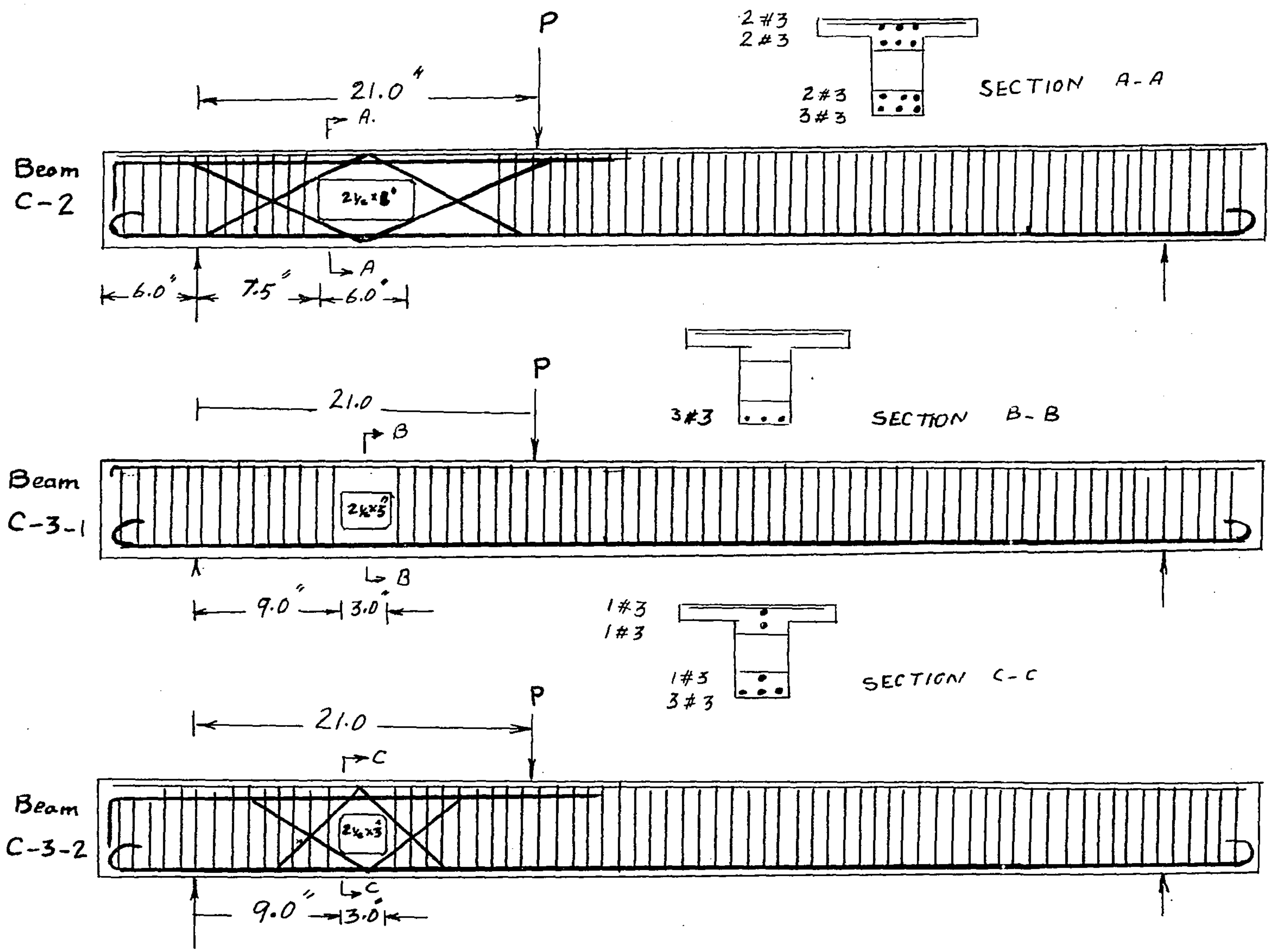


Figure 3.4 Beams Type C Reinforcement and Dimensions

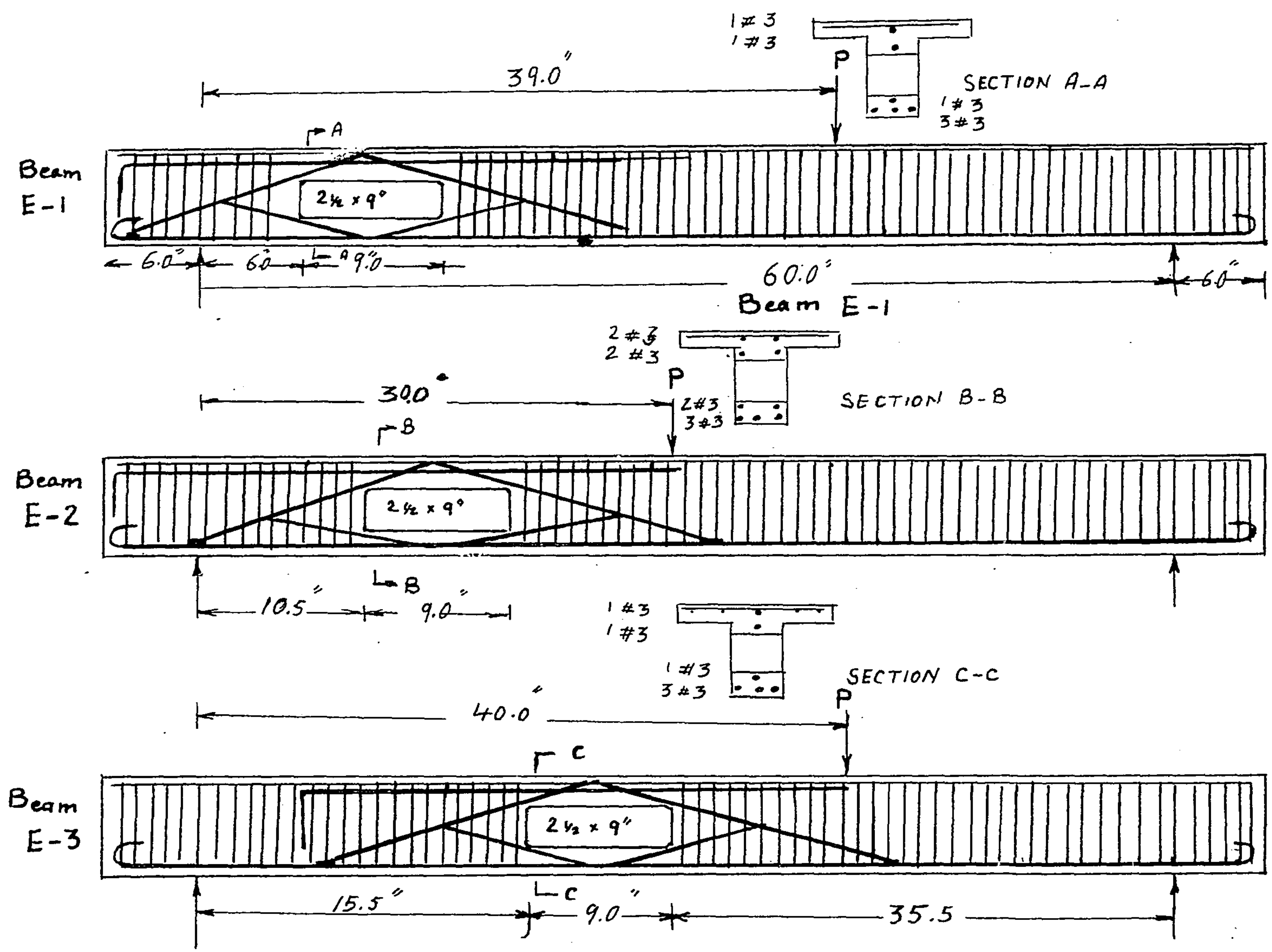


Figure 3.5 Beams Type E Reinforcement and Dimensions

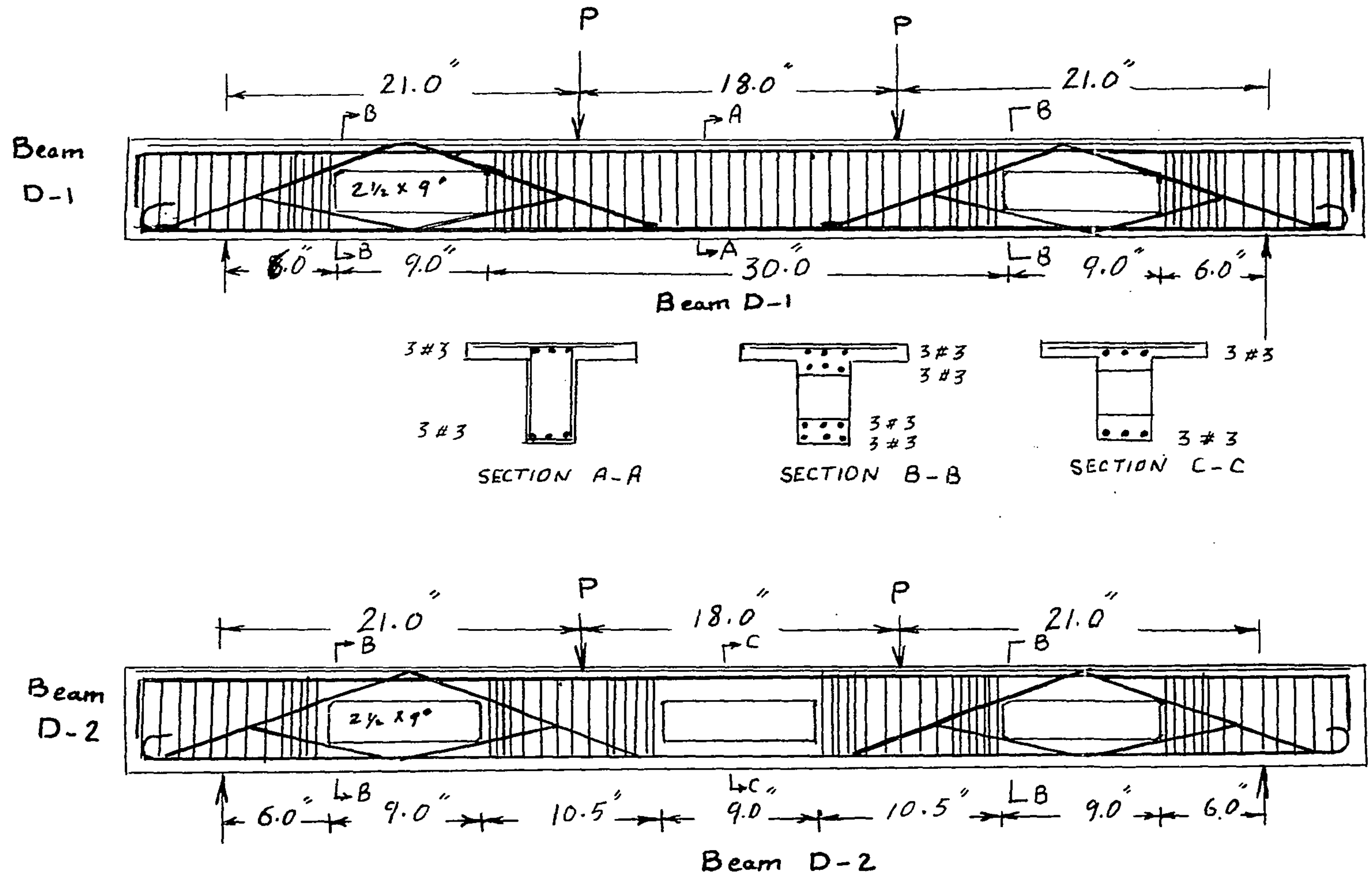
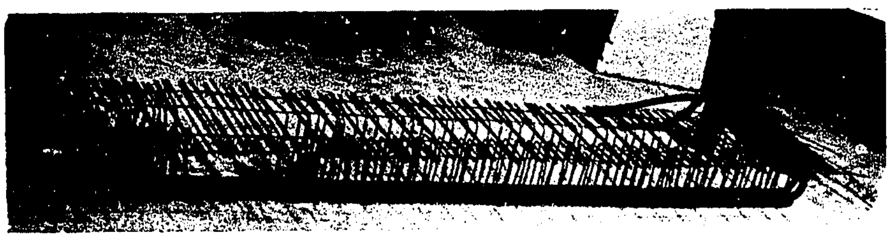
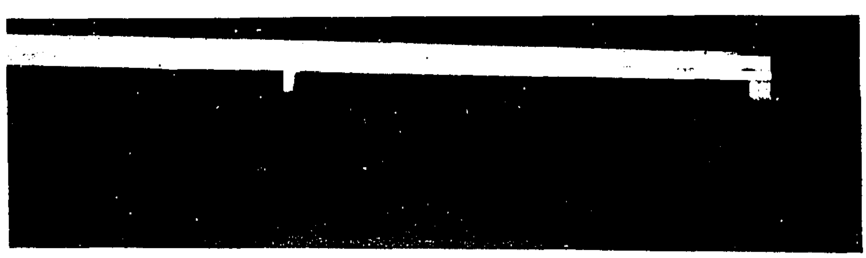


Figure 3.6 Beams Type D Reinforcement and Dimensions

(a) Beams A-1 and B-1



(b) Beams A-2-1 and A-2-2

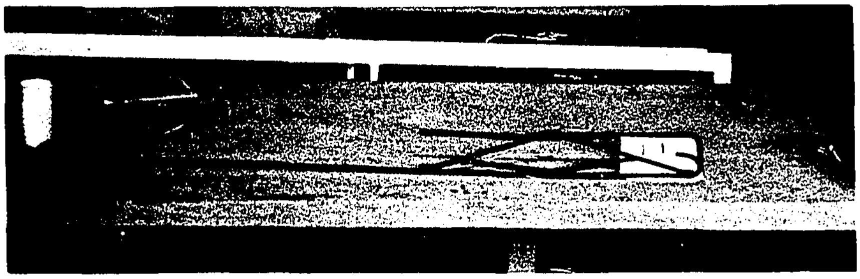


(c) Beam C-2

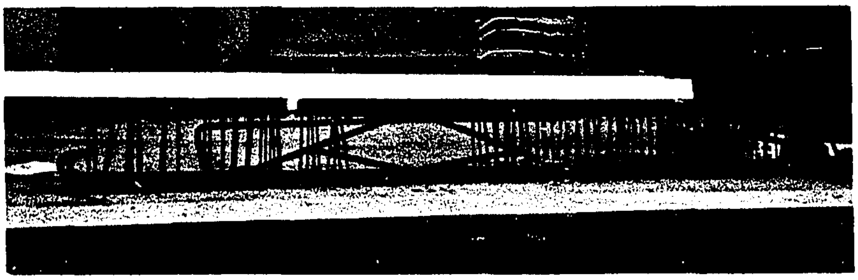
(d) Beam C-2

Figure 3.7 Reinforcement of Beams

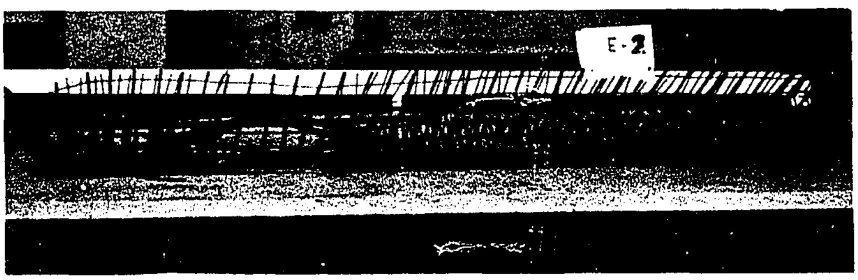
A-1, B-1, A-2-1, A-2-2 and C-2.



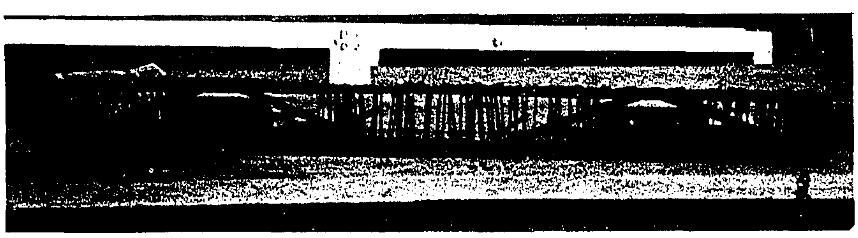
(a) Beam E-1



(b) Beam E-3



(c) Beam E-2



(d) Beam D-1

Figure 3.8 Reinforcement of Beams

E-1, E-2, E-3 and D-1.

assembled by welding as shown in Figure 3.8.

In general the principal longitudinal reinforcement used in the model T-Beams was made up from #3 bars. Tests on specimens of this bar (Figure C-1, Appendix C) yielded the following properties: yield point-57.3 Ksi, the ultimate strength - 80.0 Ksi, and percentage elongation - 17.3%. The reinforcement around the opening was also fabricated from #3 bars. Tests on this bar indicated a yield point of 51.3 Ksi, and an ultimate strength of 74.6 Ksi. Reinforcing bars (#2) used in beam A-3-1 had a yield point of 56.0 Ksi and an ultimate strength of 77.3 Ksi (Figure C-3, Appendix C). Tests on smooth black annealed wire (AISI C 1008) 0.0915 in. diameter (steel wire gage - SWG 13) gave a yield point of 28.3 Ksi, an ultimate strength of 47.1 Ksi, and a percentage elongation of 35.0% (Figure C-2, Appendix C). This wire was used as stirrups and as slab reinforcement for the T-Beam flanges.

(c) Microconcrete

A series of tests were initially performed to determine the proportions required to produce a microconcrete mixture with a design strength of 3000 psi. These tests indicated that local gravel and sand with materials greater than 3/8 removed in combination with water and Type III portland cement in the proportions 0.74:1.0:2.0:2.0 (water : cement : sand : gravel)by weight would yield a workable mixture with a 7-day strength of approximately 3000 psi. Quality control for the microconcrete was maintained by testing four 3.3 x 6.0" cylinders cast from each batch used to cast a beam.

(d) Fabrication and Curing

The concrete in the beams was vibrated with an internal vibrator. The top surface was screeded to the level of the top of the form work and finished with a steel trowel. The beam and the companion cylinders were covered with Griffolyn plastic sheeting material immediately after finishing. Twenty-four hours after placement the cylinders were stripped and placed on top of the test specimen, still under the plastic sheeting. The beam was removed from the forms in two to six days after casting and was then covered with the plastic sheet. The beam was checked each day and wetted if necessary.

(e) Dimensions

Dimensions of all the test specimens are shown in Figures 3.1 through 3.6.

A detailed comparison of the agreement between the design and as-built dimensions of the beams, is presented in Appendix B.

3.3 Instrumentation

(a) General

The primary aim of the design of the instrumentation for the test beams of this study was to provide data which illustrate load-vertical-deflection response of selected points on the top of the models and moment-rotation response for sections along the beams. The vertical deflection data were obtained from observations of dial gages mounted on a frame which was independently supported and thus free from the influence of the loading.

(b) Deflection Systems

Vertical deflections were recorded at each loading increment for fourteen points along the center line. The locations of the dial gages were such that the distances along the span between the gages and over the opening were approximately $1\frac{1}{2}$ inches and those away from the opening were 9 inches as shown in Figure 3.10.

One dial gage was mounted directly over each support on the top surface of the beam. The relative deflections of the intermediate points along the span were obtained by deducting the supports deflections. To obtain these deflections one-thousandth inch least count dial gages were mounted on a frame which was supported independently.

3.4 Moment-Rotation Response

Moment-rotation response was measured by a Rotation Meter specially designed and built of two yokes each made of two angles and two rods. Both yokes were fitted against the cross section of the beam perpendicular to the center line and 3.5 inches apart. The rotation of the section between the two yokes decreases the distance between the two yokes on the top and increases it at the bottom. To obtain this horizontal movement, ten-thousandth inch least count dial gages were mounted at the top and bottom of one of the yokes. The dial gage plunger was then brought into contact with similarly located points on the other yoke (Figure 3. 9).

The relative deflection of the top and bottom dial gages was used to obtain the rotation of the beam according to the following:

Let ϕ equal ϵ_t minus ϵ_b divided by t i.e.

$$\phi = \frac{\epsilon_t - \epsilon_b}{t}$$

$$\text{then} \quad \phi = \frac{\Delta L_t - \Delta L_b}{hL}$$

Where

ϵ_t = Unit strain at the lowest fiber of the cross section

ϵ_b = Unit strain at the upper most fiber of the cross section

ΔL_t = Deflection recorded from the top dial gage

ΔL_b = Deflection recorded from the bottom dial gage

h = Horizontal distance between the two yokes

L = The vertical distance between the two dial gages

3.4 Loading

In the first phase of the experimental program the test specimens were loaded with a single concentrated load which was applied by a hydraulic ram. This ram was controlled by a pump which was equipped with a 5,000 psi pressure gage. The load was monitored by both the pressure gage and a self-temperature compensating load cell with read out taken on a portable strain indicator. The static check provided by the pressure gage and the load cell demonstrated excellent control of applied loadings. The loading system (hydraulic pump, ram, pressure gage and load cell) was calibrated by comparison with a universal testing machine which had previously been calibrated. Precision well beyond the least division of the read out equipment was indicated by a linear least squares fit of the calibration data.

For the second phase of this study the loading system consisted of two identical hydraulic rams and a single hydraulic pump. Thus two concentrated loads were applied to the test specimens. Two calibrated load cells with read out taken on a portable strain indicator equipped with a switch and balance unit were used to monitor the two loadings.

The two loading cells, the strain indicator and the switch and balance unit are shown in Figure 3.11. Load locations for each beam are shown in Figures 3.2 through 3.6.

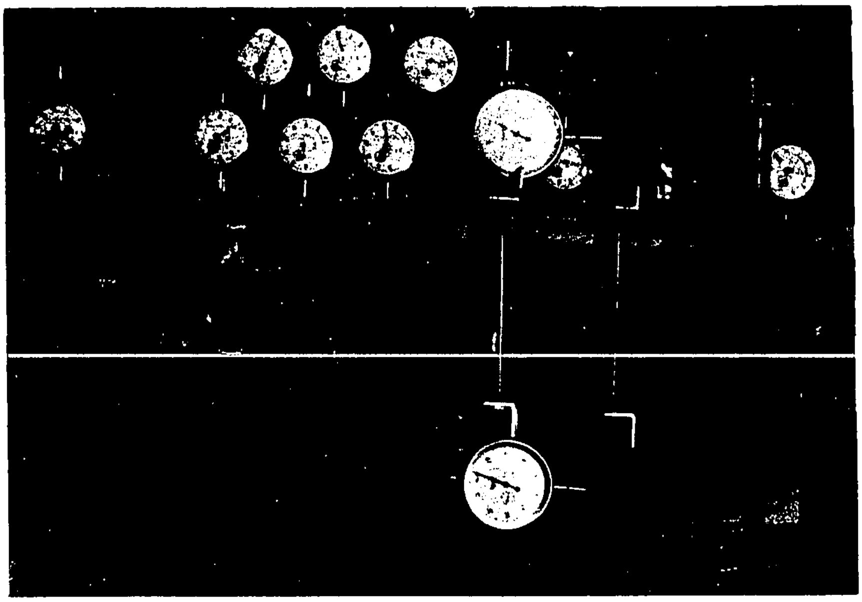


Figure 3.9 Rotation Meter

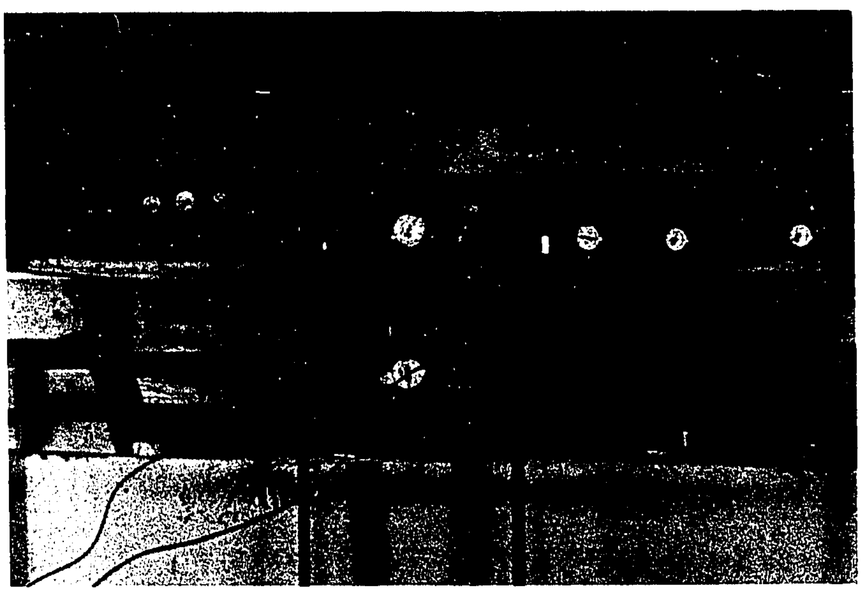


Figure 3.10 Deflection System

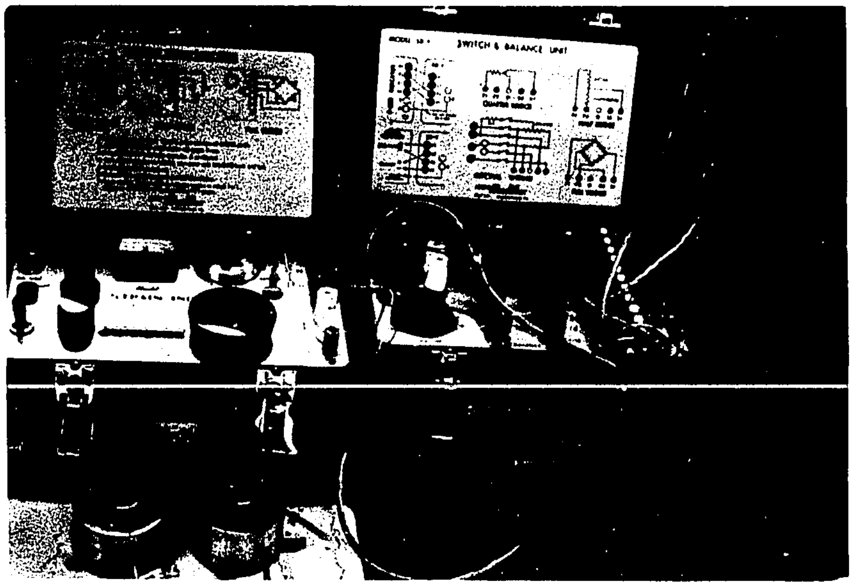


Figure 3.11 Load cells, strain indicator and switch and balance unit.

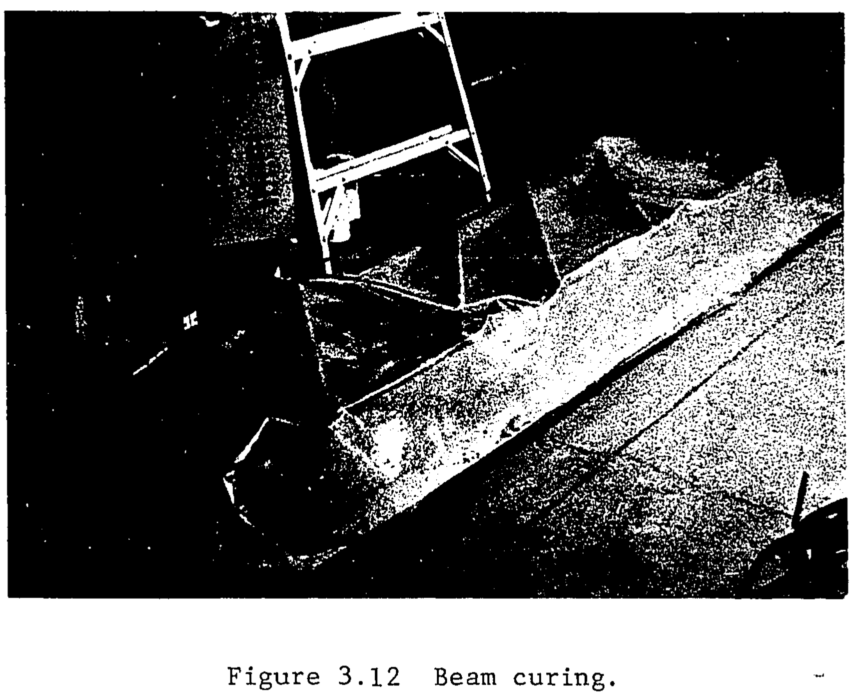


Figure 3.12 Beam curing.

CHAPTER IV

PRESENTATION OF EXPERIMENTAL RESULTS

4.1 General

The objectives of this study as stated in Chapter I were to investigate the behavior of reinforced microconcrete T-Beams with large openings with and without special web reinforcement. The variables were: the size of openings, the type of loading, the shear-span ratio and percentage of special web reinforcement. A detailed explanation of coding notations used in describing the fifteen beams tested in this study is given in Table 4-1.

Fifteen models of reinforced microconcrete simply supported T-Beams were loaded monotonically to failure by applying one or two-point loadings. The load-deflection and moment-rotation responses of thirteen beams were compared with two beams without openings and a theoretical load-deflection solution for beams. This theoretical solution is written as a Fortran IV computer program LDDFN and is listed in Appendix A.

The responses of these beams to applied loads are presented in terms of load versus deflection curves, moment versus rotation curves, cracking, and collapse mechanisms.

4.2 Type A Beams

No.	T-Beam Code Notation	Number of Openings	Size of Openings Inches	Type of Loading	Shear Span Ratio a/d	% of Special Reinforcement in top chord
1	A-1	0	None	One-point	3.78	None
2	A-2-1	1	$2\frac{1}{2} \times 9$	One-point	3.78	None
3	A-2-2	1	$2\frac{1}{2} \times 9$	One-poinr	3.78	None
4	A-3-1	1	$2\frac{1}{2} \times 9$	One-point	3.78	1.48 + Stirrups
5	A-3-2	1	$2\frac{1}{2} \times 9$	One-point	3.78	2.44
6	B-1	1	None	Two-points	3.78	None
7	B-2	1	2×9	Two-points	None	None
8	C-2	1	$2\frac{1}{2} \times 6$	One-point	3.78	1.62
9	C-3-1	1	$2\frac{1}{2} \times 3$	One-point	3.78	None
10	C-3-1	1	$2\frac{1}{2} \times 3$	One-point	3.78	0.82
11	D-1	2	$2\frac{1}{2} \times 9$	Two-points	3.78	2.44
12	D-2	3	$2\frac{1}{2} \times 9$	Two-points	3,78	2.44
13	E-1	1	$2\frac{1}{2} \times 9$	One-point	7.00	0.82
14	E-2	1	$2\frac{1}{2} \times 9$	One-point	5.4	1.62
15	E-3	1	$2\frac{1}{2} \times 9$	One-point	7.2	0.82

Table 4-1. Detailed explanation of coding notations used in describing the fifteen beams.

The purpose of the design of the beams classified as Type A was to study the behavior of beams with an opening which were subjected to one-point loading and to find an effective type of reinforcement for placement around the opening. All of the beams in the A series had openings of $2\frac{1}{2}$ x 9 inches.

In addition to the control beam A-1 which did not have an opening four beams were tested in the A series. Beams A-2-1 and A-2-2 were companion specimens each having a $2\frac{1}{2}$ x 9 inch opening without special reinforcement. As a result of the behavior of these beams, the openings of two other beams (A-3-1 and A-3-2) were "reinforced" by two different methods. The first method (of reinforcement) was based on the formation of cracks around the openings in beams A-2-1 and A-2-2. The second method of reinforcement was based on the concept of restraining the rotation of the section containing the opening.

Load-Deflection Responses

Fourteen dial gages were mounted along the top of the beam (Figure 3.10). The point of maximum deflection was found to be roughly 2 inches toward the centerline of the span from the concentrated load for the control beam A-1 and 2 to 4 inches toward the support from the concentrated load for the remaining four specimens.

The load-deflection responses are shown in Figure 4.1. The load-span deflections are shown in Figures 4.10 through 4.13.

Moment-Rotation Responses

The curvature (ϕ) values and the corresponding moments are mainly dependent on the physical dimensions of the reinforced concrete section

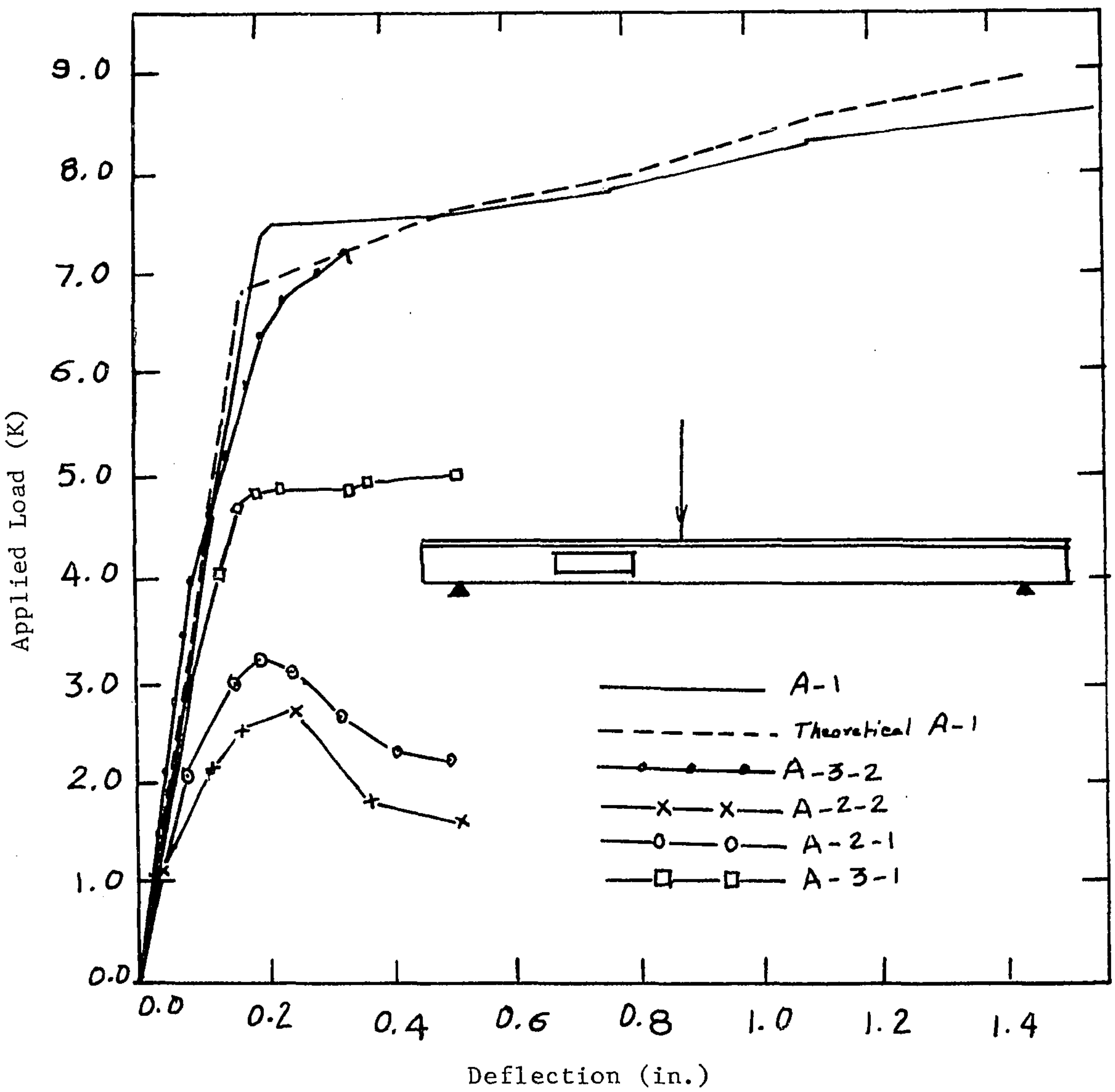


Figure 4.1 Applied Load Versus Vertical

Deflection for Type A Beams.

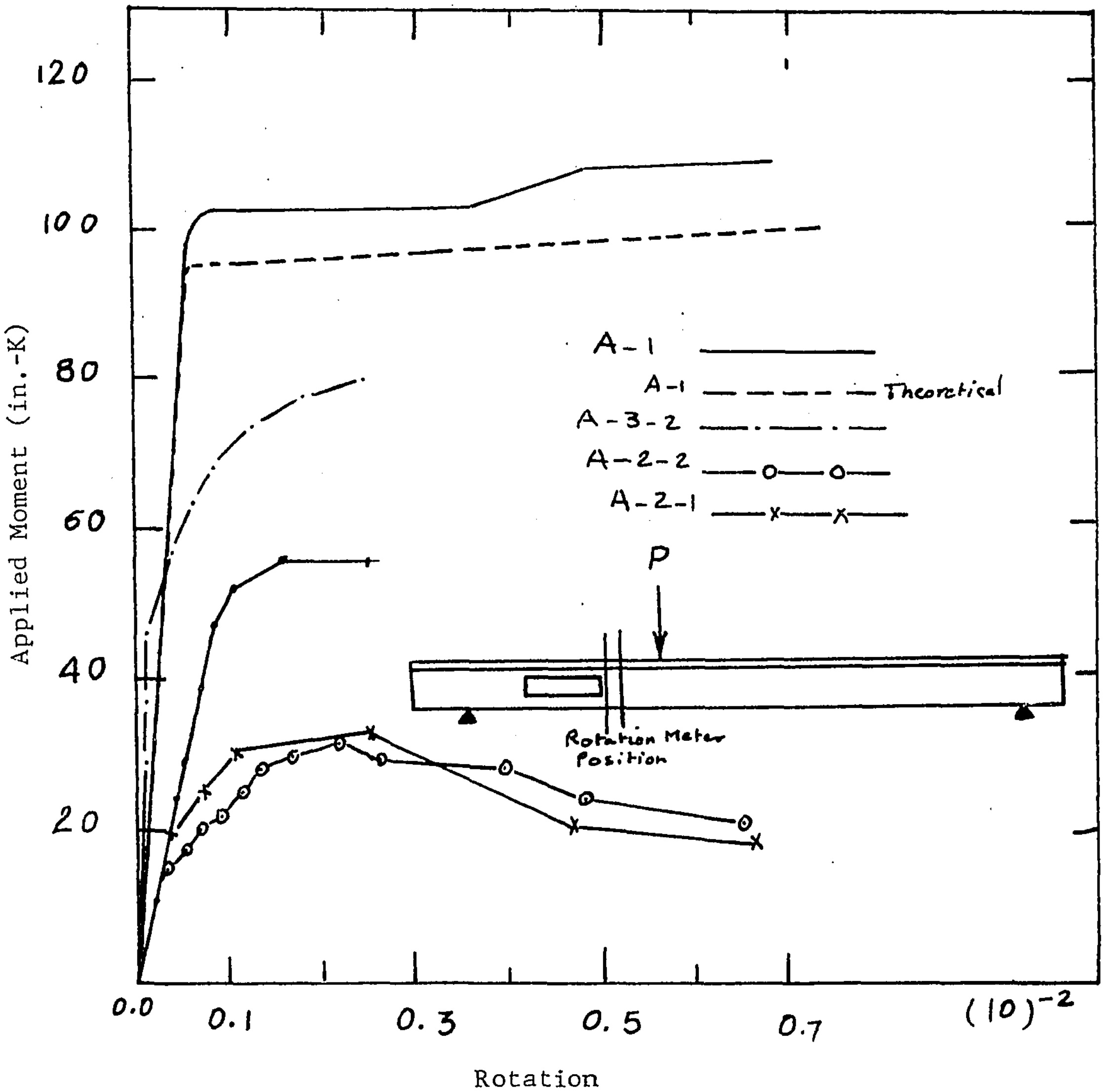


Figure 4.2 Applied Moment Versus Rotation

for Type A Beams.

but this is not the case when the cross section contains an opening. Therefore a rotation measuring device was mounted at one end of the opening as shown in Figure 4.8.

The results of the moment-rotation responses are shown in Figure 4.2.

Cracking and Collapse Mechanisms

The beams investigated exhibited widely differing cracking patterns. Flexural cracks were first visible in beam A-1 at about 30 percent of the beam ultimate load. These cracks widened and extended slowly toward the compression zone as the load was increased to the yield of the tension reinforcement. After yielding of the tension reinforcement, the cracks widened rapidly and also increased in number at the level of the tension reinforcement. As the load was further increased, surface cracking and spalling in the concrete compression zone occurred. The appearance of beam A-1 after the test is shown in Figure 4.8.

The cracks of beam A-2-1 and A-2-2 were first visible at the corners of the opening numbered (1), (2) and (3) in Figure 4.3 at roughly 13 percent of the beam ultimate load. As the load was increased, these cracks widened with crack (1) extending toward the corner (5) of the opening, while crack (2) extended toward the upper surface of the beam, and crack (3) extended toward the tension reinforcement. During this time the portion of the beam which contained the opening started to rotate in the direction of the loading around the center of the openings. The end of the opening nearest the support moved upward as shown in

Figures 4.11, 4.8 b, and 4.9 c. Further increase of the load resulted in corner cracks and failure.

The cracks of beam A-3-1 were first visible at the opposite corners (2) and (3) as shown in Figure 4.4 at roughly 25 percent of the ultimate load. As the load was further increased, inclined parallel cracks first appeared on the top chord with similar type cracks appearing later on the bottom chord. A diagonal tension failure occurred in the top and bottom chords. Rotation of the section containing the opening is shown in Figures 4.8 c, 4.12, and 4.9 b.

The cracks in beam A-3-2 first appeared at the corners (2) and (3) of the opening shown in Figure 4.6 at 30 percent of the ultimate loading. As the load increased, more cracks formed in the bottom chord as flexural cracks were being developed in the main section away from the opening. As the load increased further, the flexural cracks widened and extended toward the surface. Horizontal cracks appeared on the bottom chord and on the top chord between the slab and the web.

A bond failure occurred in both the top and bottom reinforcement. No rotation of the section of the beam that contained the opening occurred as shown in Figures 4.13, 4.8 d and 4.9 a.

4.2 Type B Beams

Two Type B beams were constructed, Beam B-1 did not have an opening and beam B-2 had an opening $2\frac{1}{2}$ x 9 inches at the center of its span. Both beams were subjected to third point loadings. Consequently the section that contained the opening was subjected to a pure bending moment.

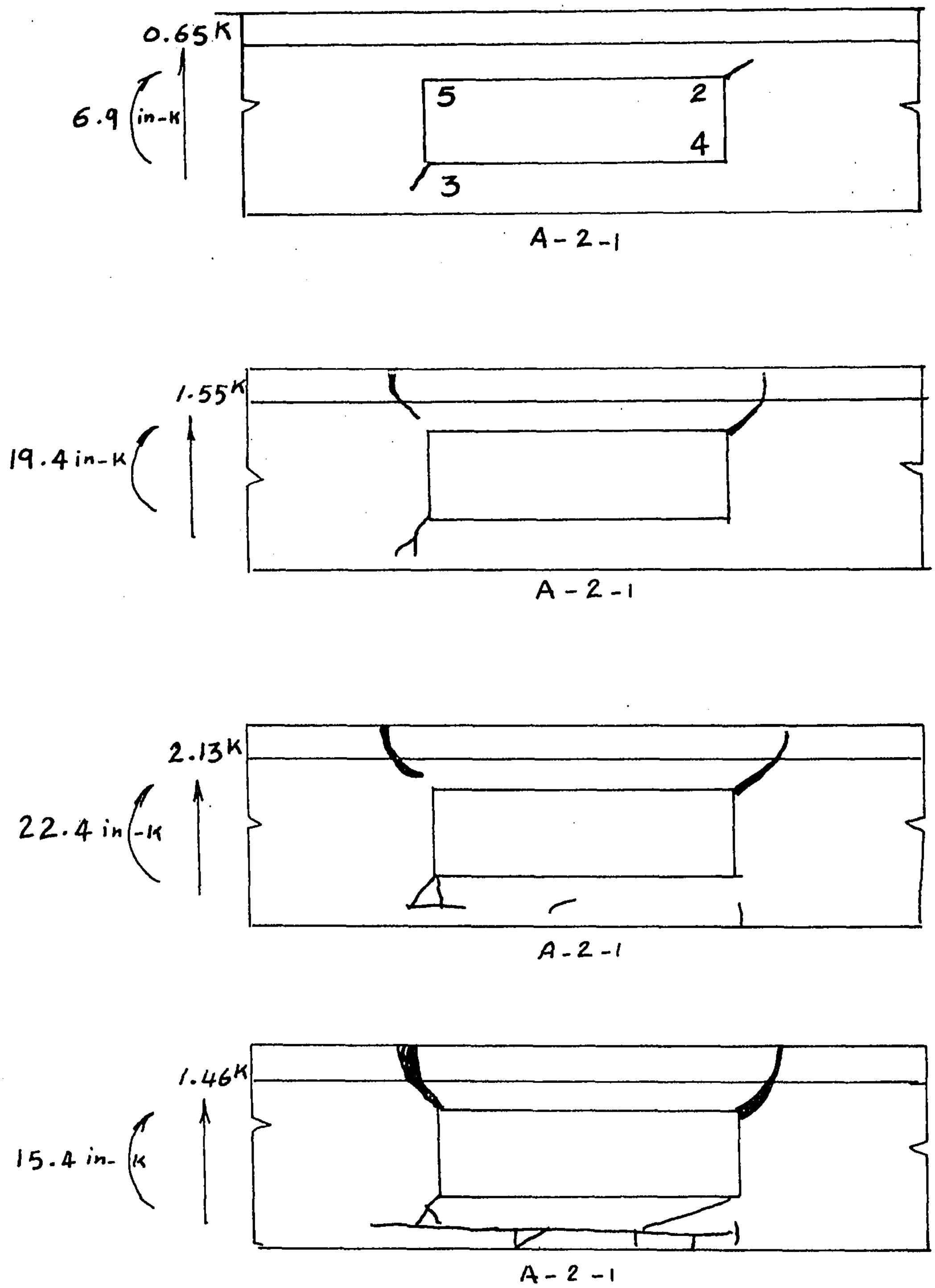


Figure 4.3 Cracking Versus Shearing Force and Bending Moment for Beam A-2-1.

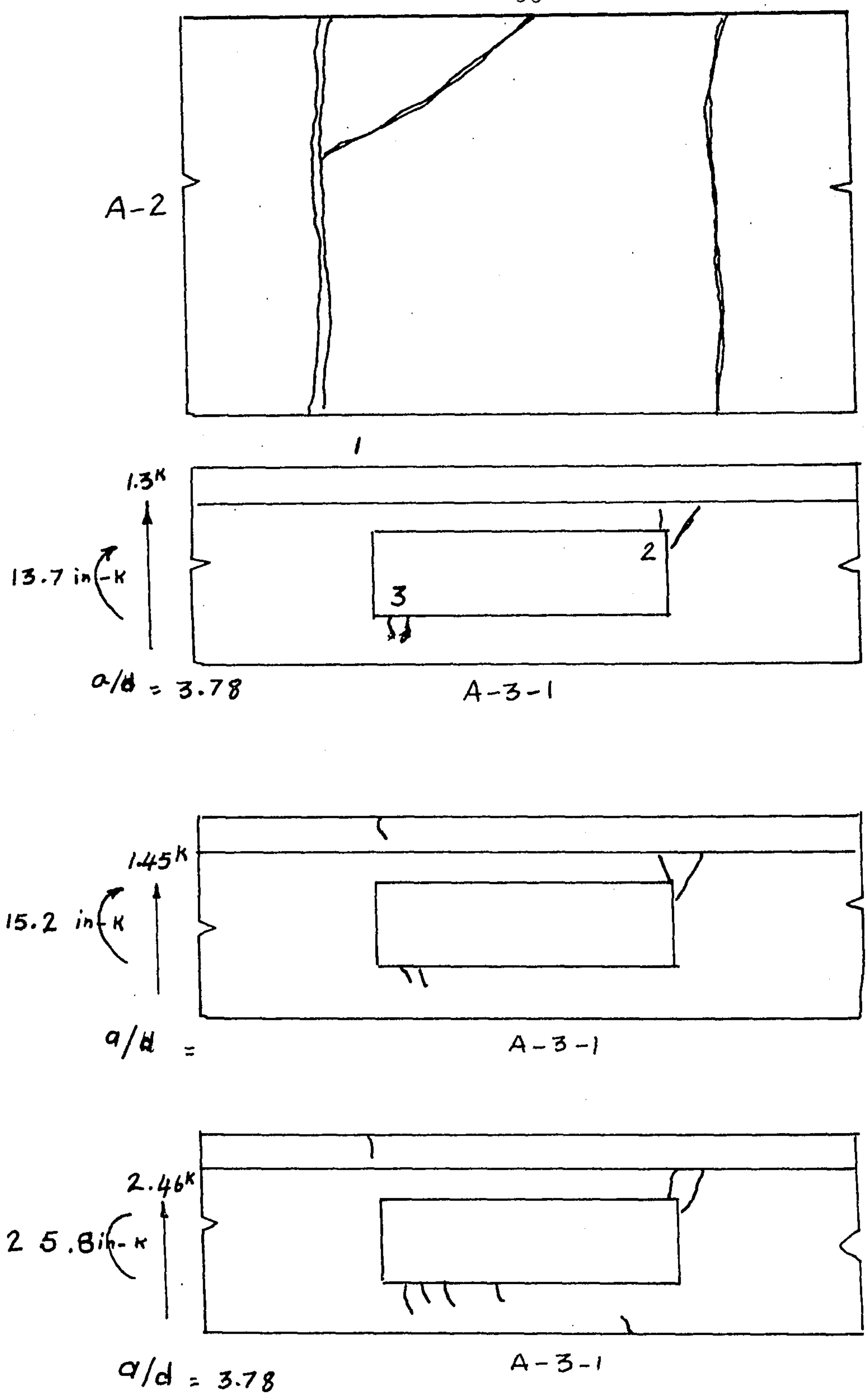


Figure 4.4 Cracking Versus Bending Moment

Around the Opening of Beam A-3-1.

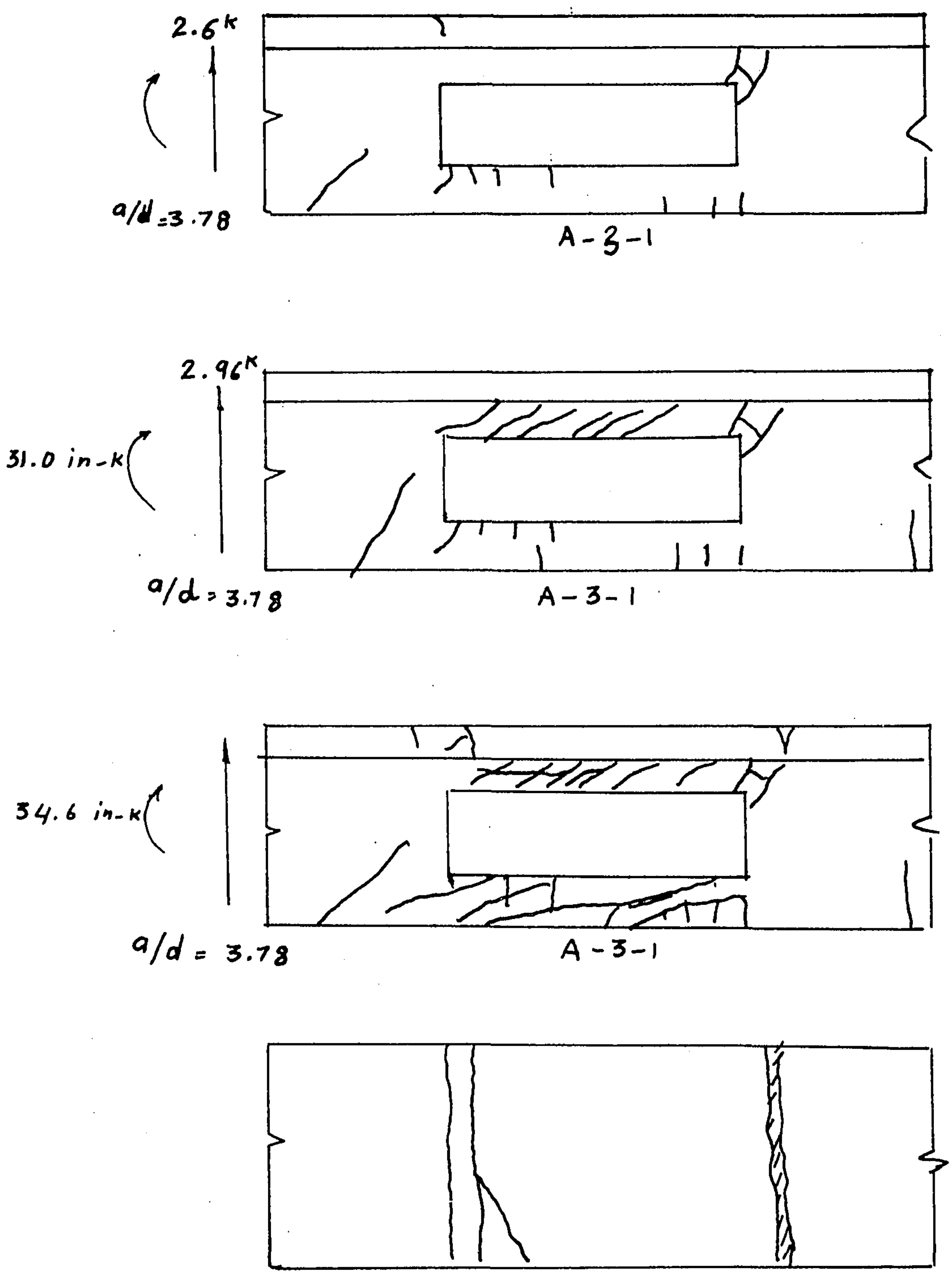


Figure 4.5 Cracking Versus Bending Moment and Shearing Force Around the Opening of Beam A-3-1.

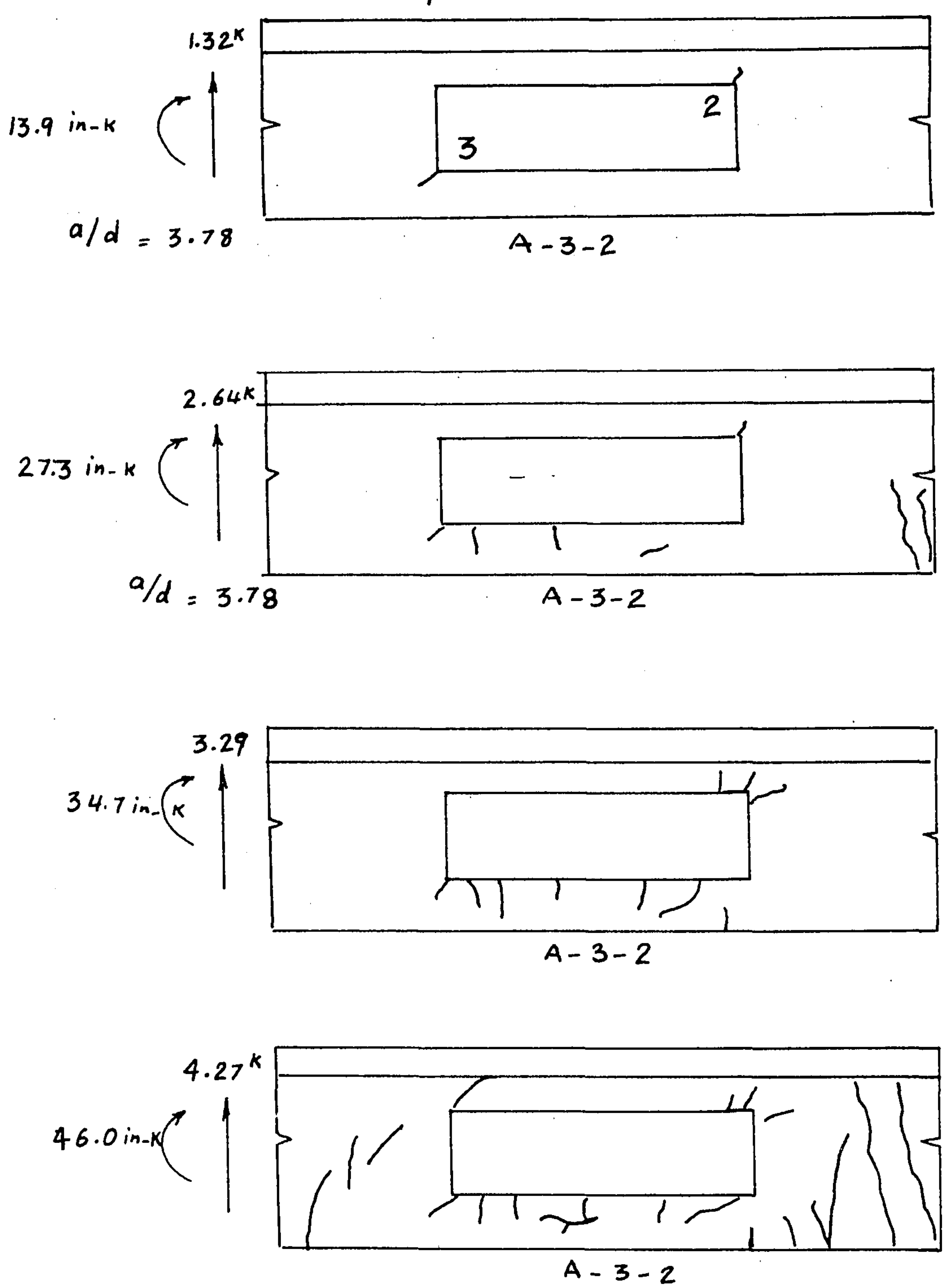
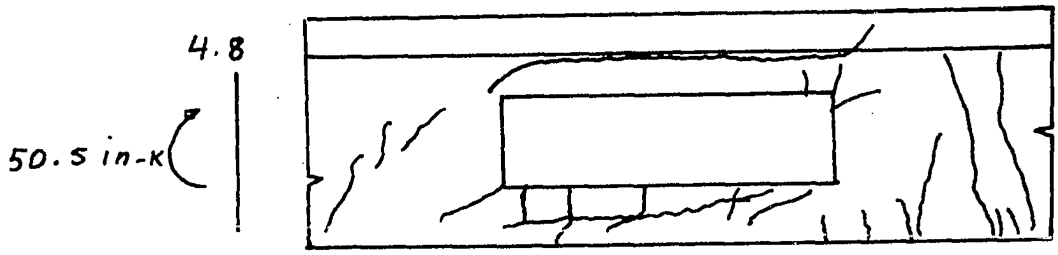
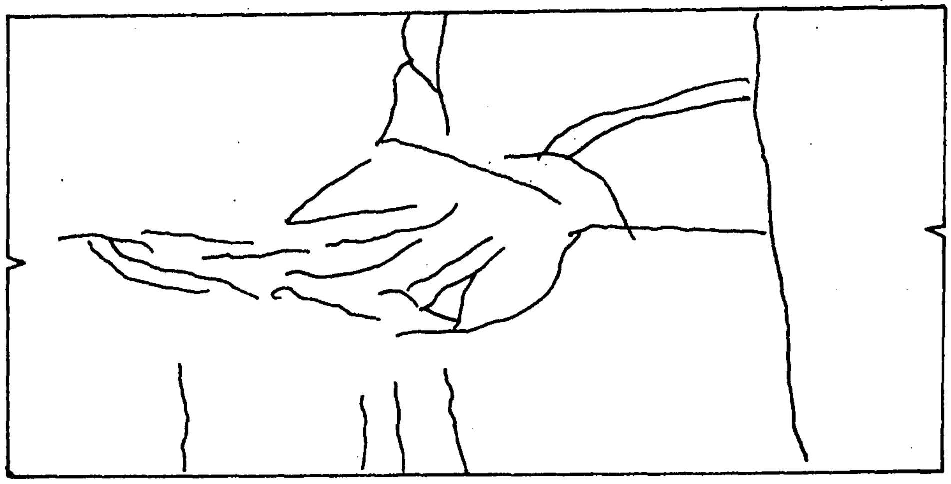


Figure 4.6 Cracking Versus Bending Moment and Shearing Force Around the Opening of Beam A-3-2.



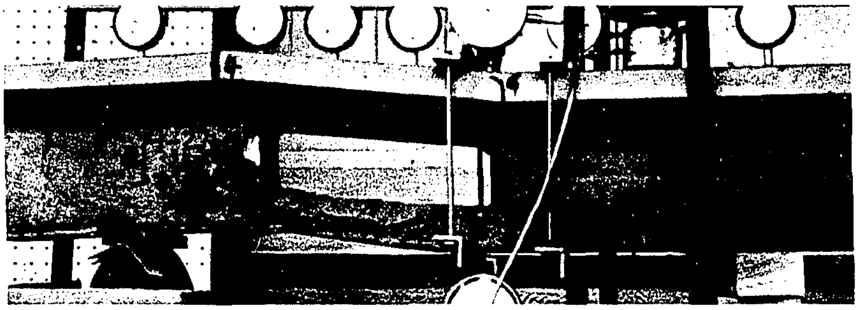
A-3-2



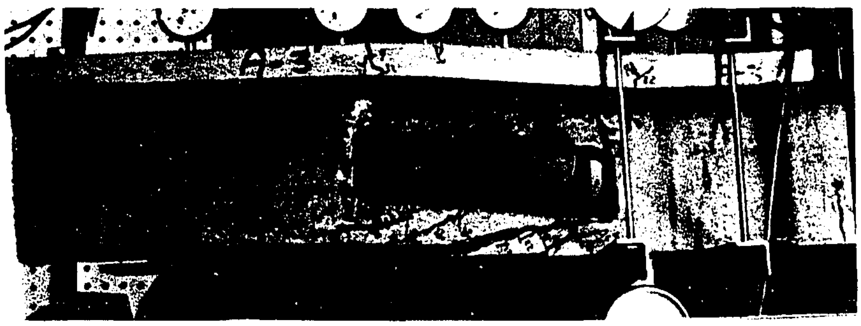
A-3-2

Figure 4.7 Cracking Versus Bending Moment and Shearing Around the Opening of Beam A-3-2.

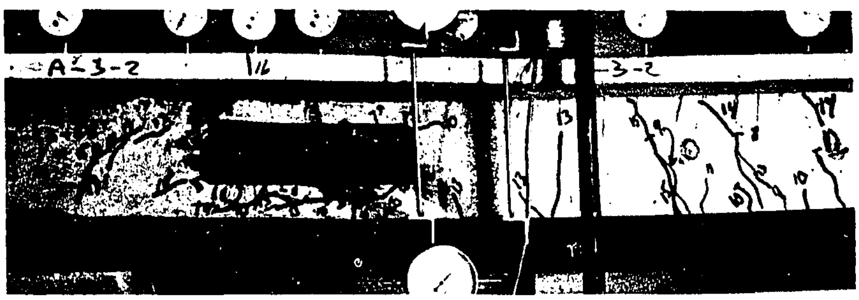
(a) Beam A-1



(b) Beam A-2-1



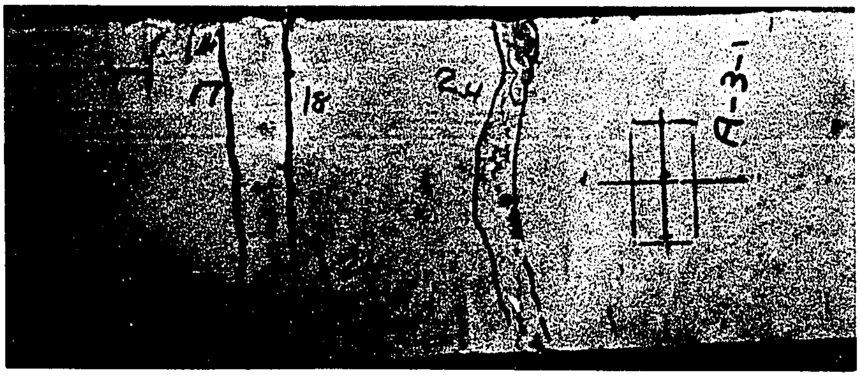
(c) Beam A-3-1



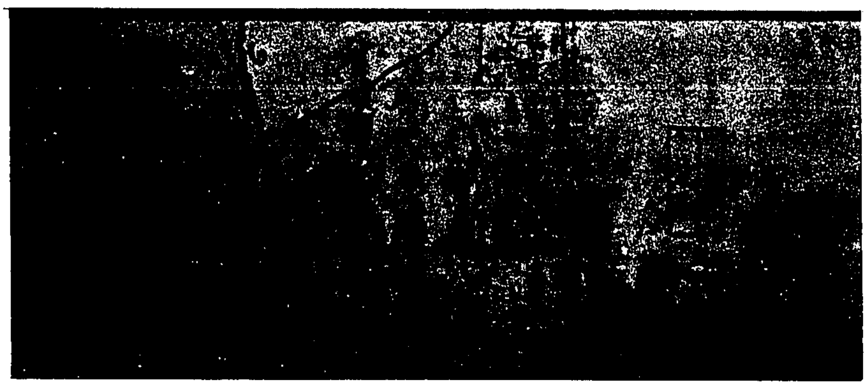
(d) Beam A-3-2

Figure 4.8 Cracking of Type A Beams at Failure .

(a) Beam A-3-2



(b) Beam A-3-1



(c) Beam A-2-1

Figure 4.9 Top Cracking of Type A Beams at Failure.

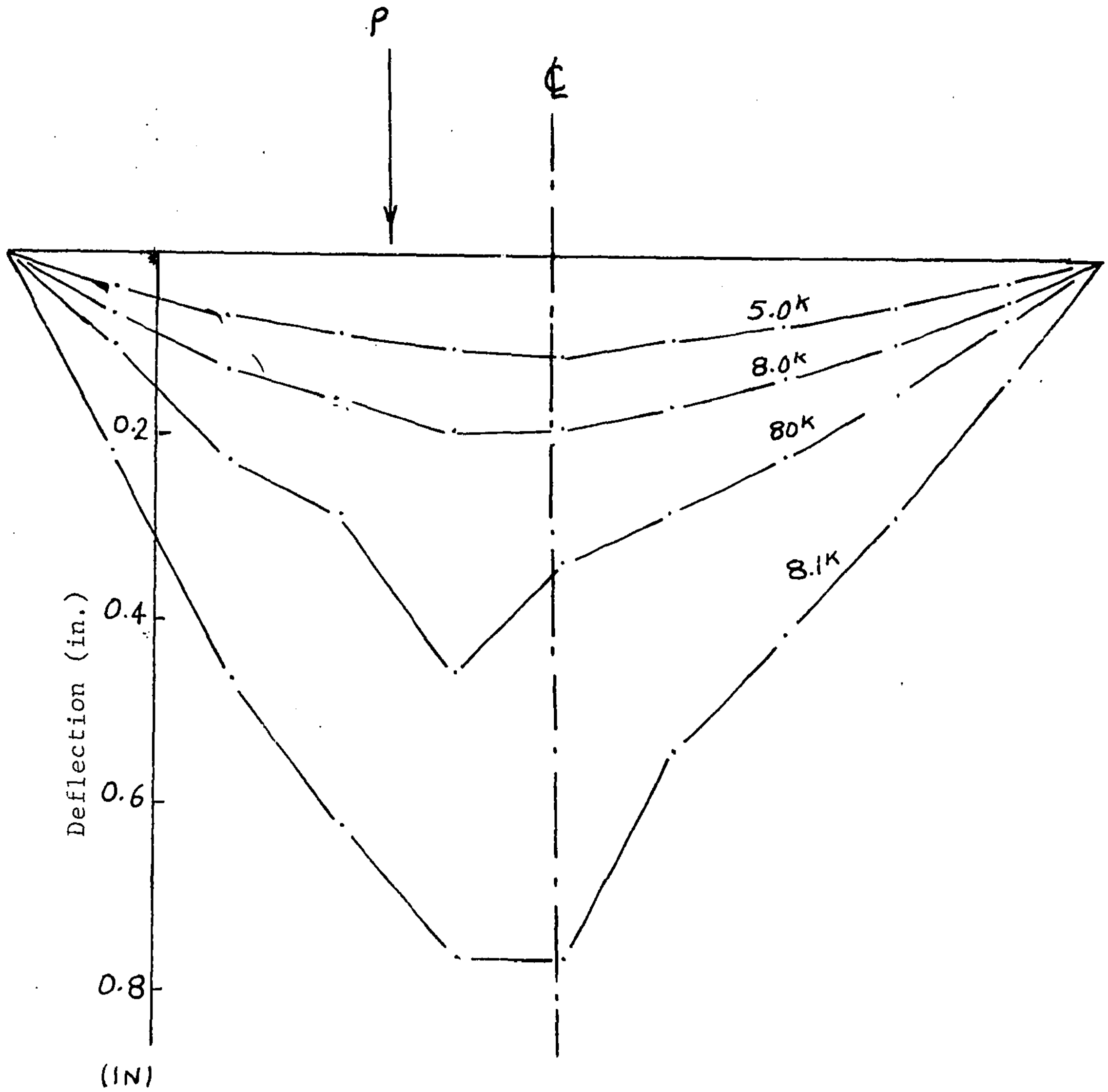


Figure 4.10 Span Deflections of Beam A-1

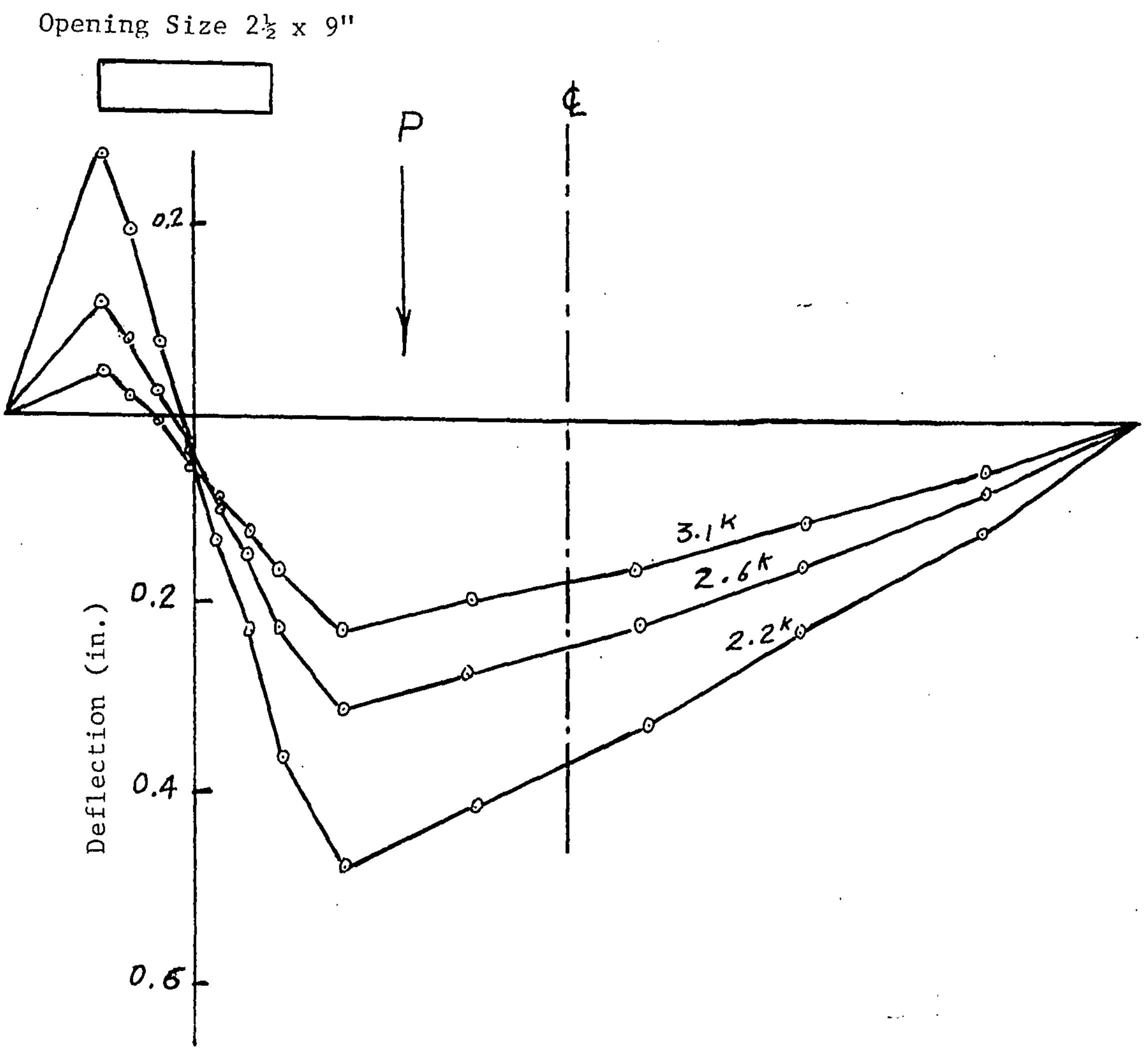


Figure 4.11 Span Deflections of Beam A-2-1.

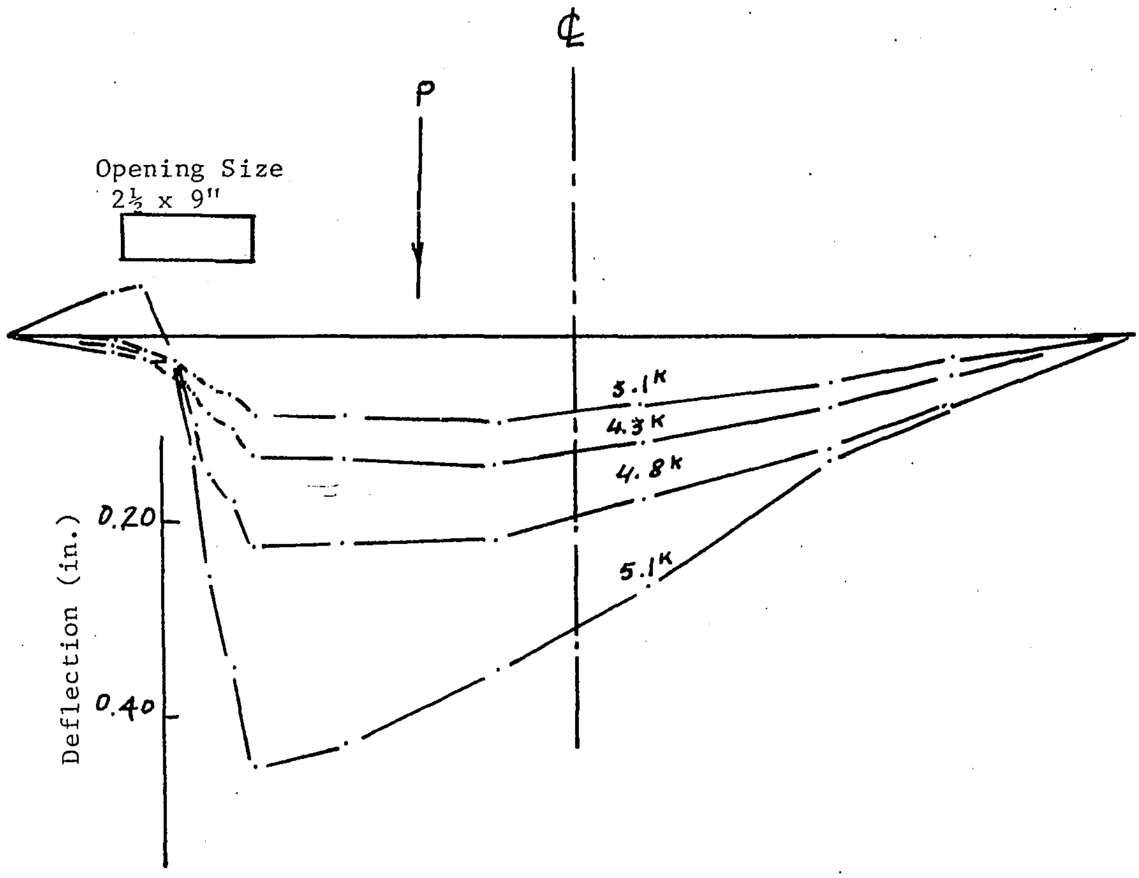


Figure 4.12 Span Deflections of Beam A-3-1.

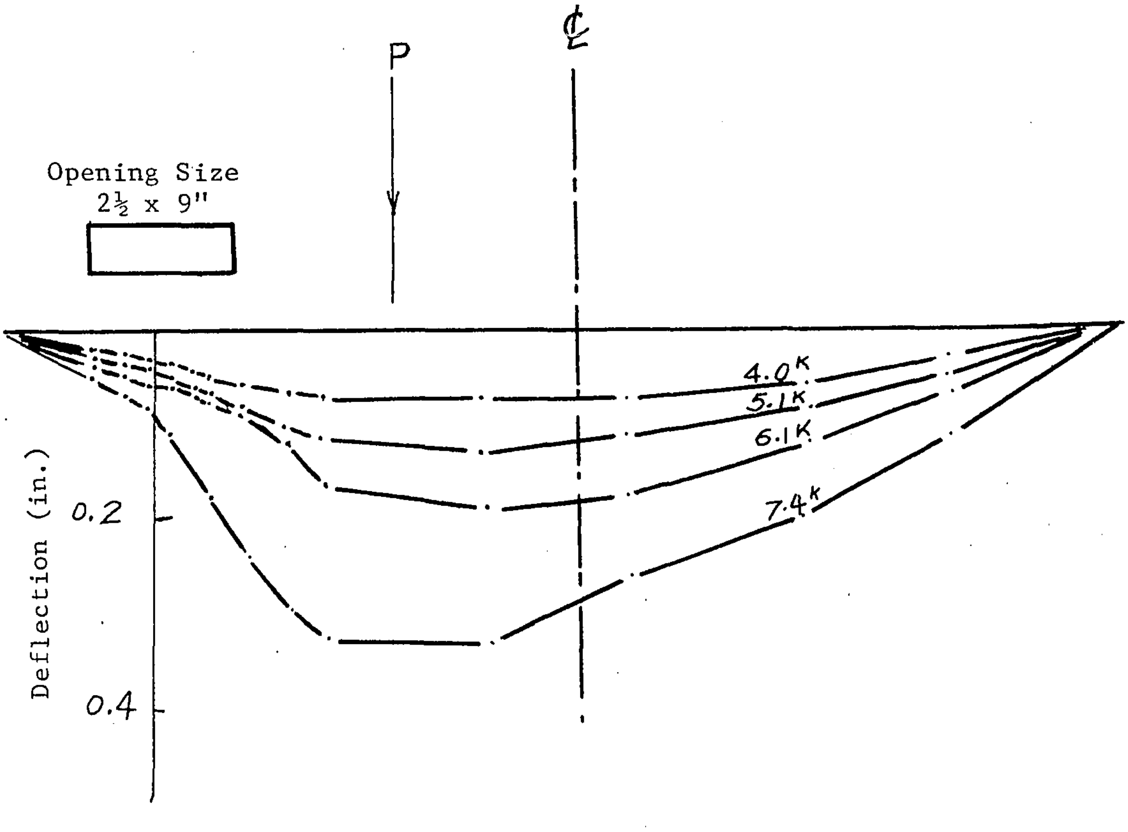


Figure 4.13 Span Deflections of Beam A-3-2.

The purpose for the design of the Type B beam was to study the behavior of a beam having an opening and subjected to pure bending.

Load-deflection responses and moment-rotation responses are shown in Figures 4.14 and 4.15; the results indicate that both beams behaved identically except that the yield load of beam B-2 was approximately 14 percent lower than that of the control beam B-1.

The propagation of cracks in both beams was similar; flexural cracks were first visible at 30 percent of the ultimate load. These cracks slowly widened and extended toward the compression zone as the load was increased. After the yield of the tension reinforcement, the cracks widened rapidly and also increased in number at the level of the tension reinforcement. As the load was further increased, surface cracking and spalling in the concrete compression zone occurred.

Flexural compression type of failure occurred in both beams as shown in Figures 4.16 through 4.18. Span deflection curves are shown in Figures 4.19 and 4.20.

4.4 Type C Beams

The purpose for the design of the Type C beams was to provide adequate reinforcement for openings having the dimensions $2\frac{1}{2}$ x 3 and $2\frac{1}{2}$ x 6 inches. These beams were compared with beam A-1. The Type C beams were loaded in a manner similar to that used for the Type A beams.

Beam C-3-1 (opening $2\frac{1}{2}$ x 3 with no special reinforcement) was tested first. The load-deflection response for this beam is shown in Figure 4.21. The slope of this curve shows a 17% increase in the deflection when compared with the slope of the curve for beam A-1. It

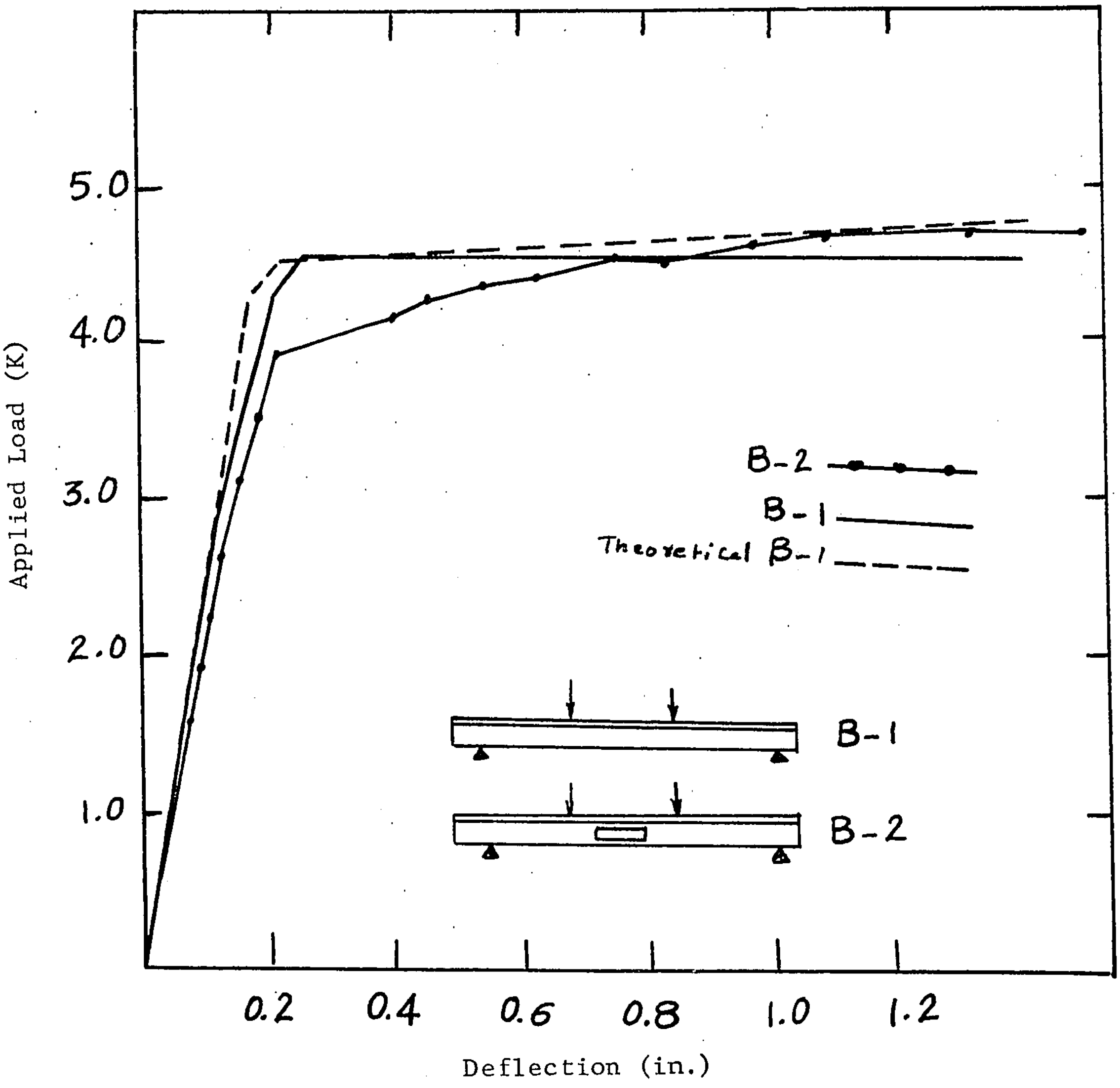


Figure 4.14 Applied Load Versus Vertical Deflection for Type B Beams.

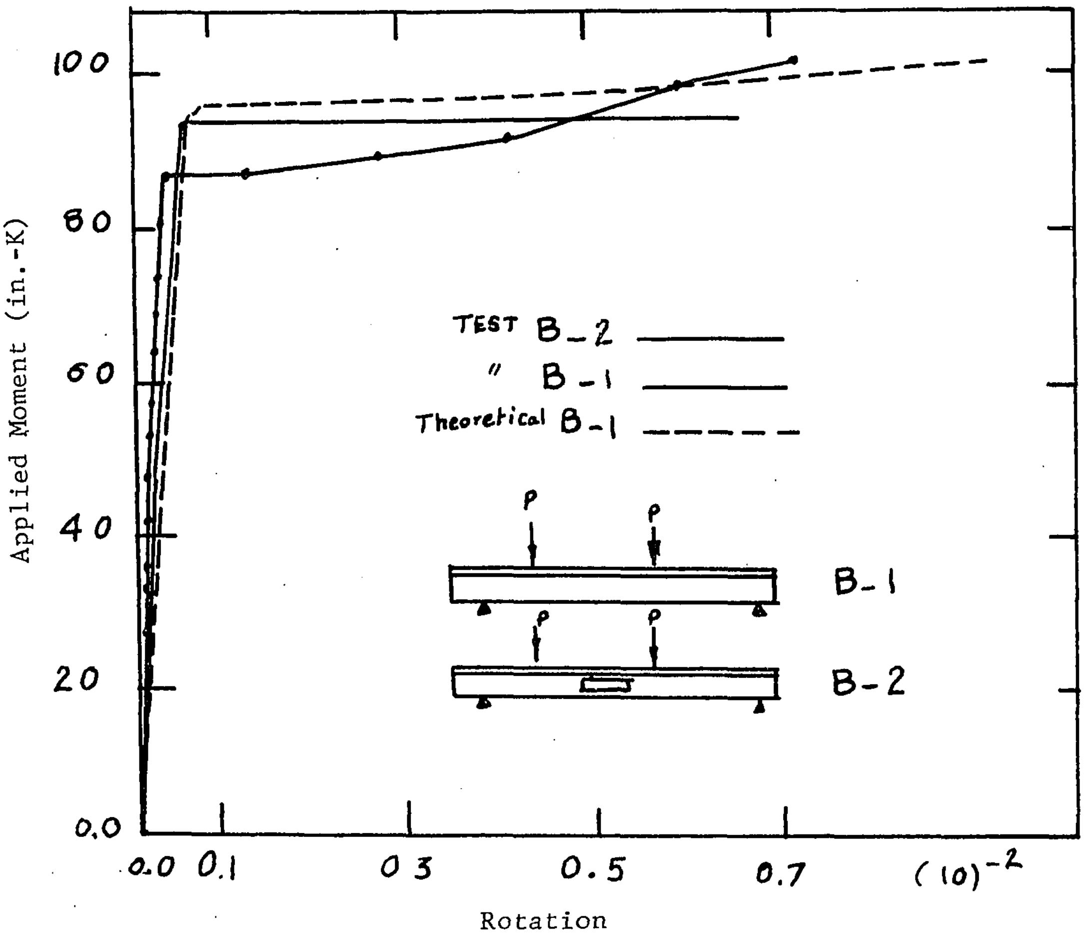


Figure 4.15 Applied Moment Versus Rotation

at Mid-span for Type B Beams.

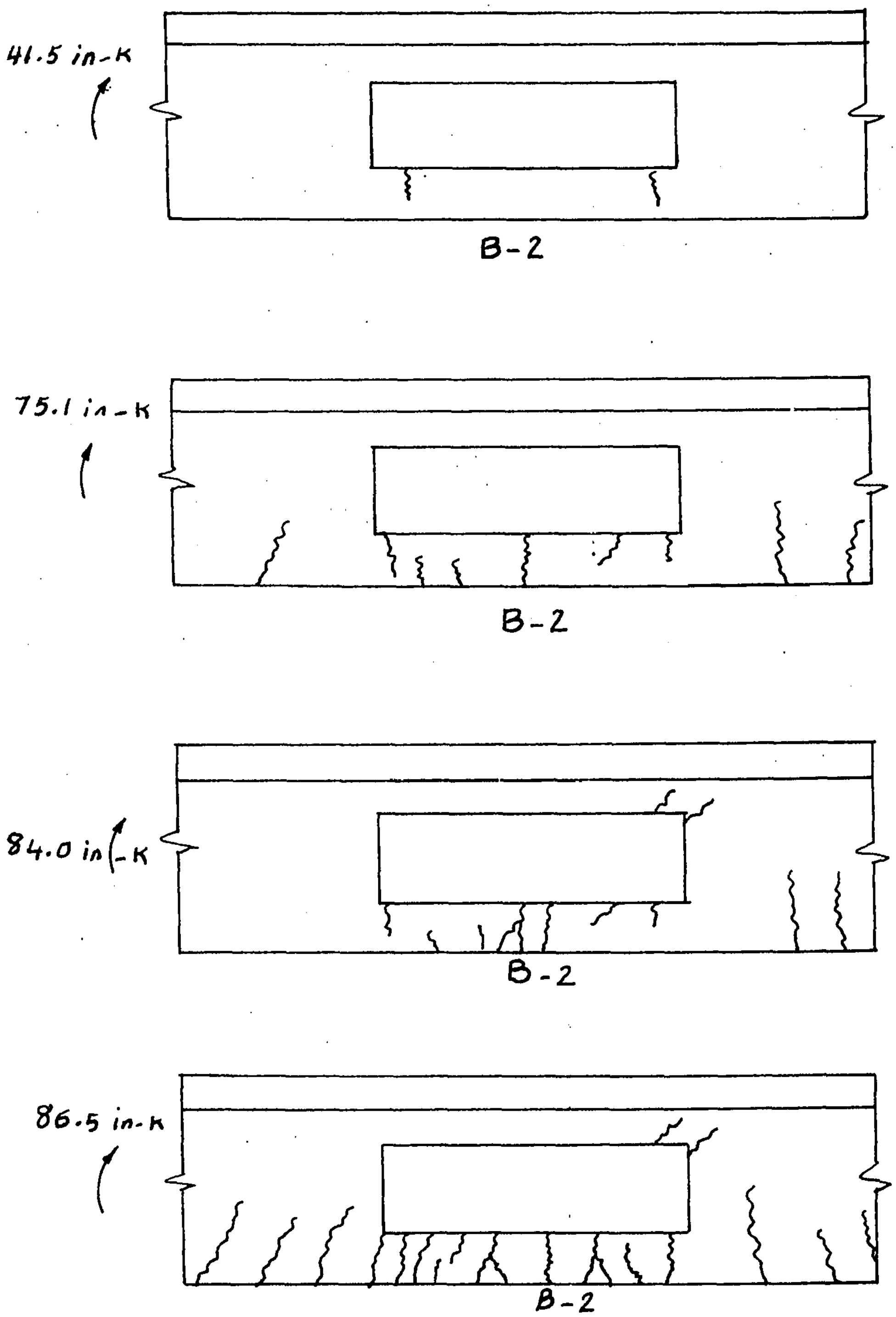


Figure 4.16 Cracking Versus Shearing Force and Bending Moment Around the Opening of Beam B-2.

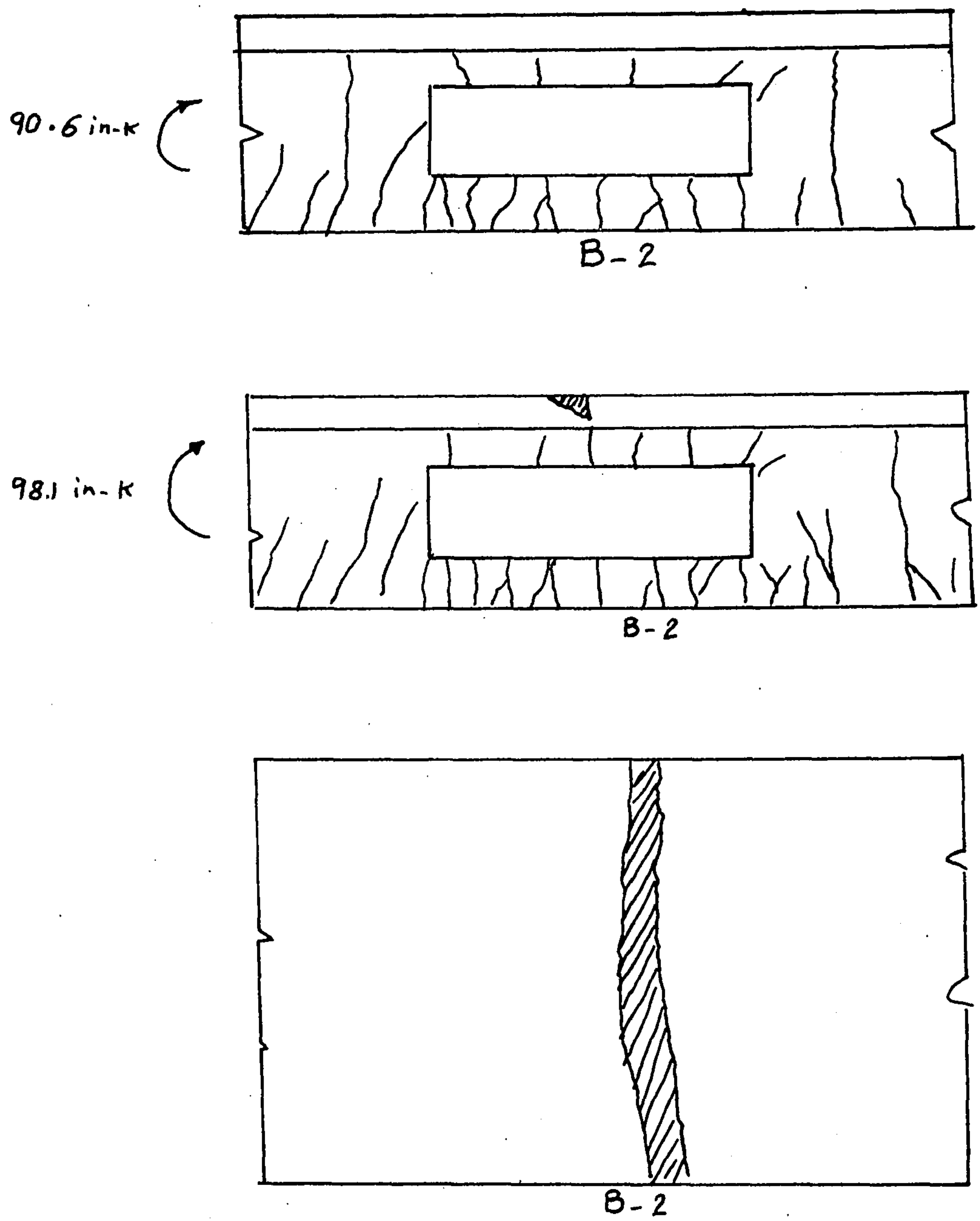
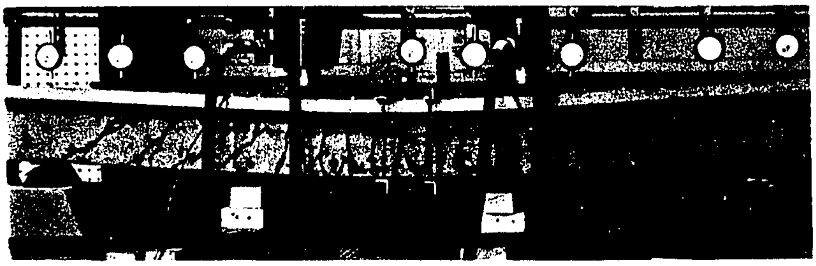


Figure 4.17 Cracking Versus Bending Moment and Shearing Force Around the Opening of Beam B-2.



(a) Beam B-1

(b) Beam B-2

(c) Beam D-1

(d) Beam D-2

Figure 4.18 Cracking of Beams Type

B and D at Failure.

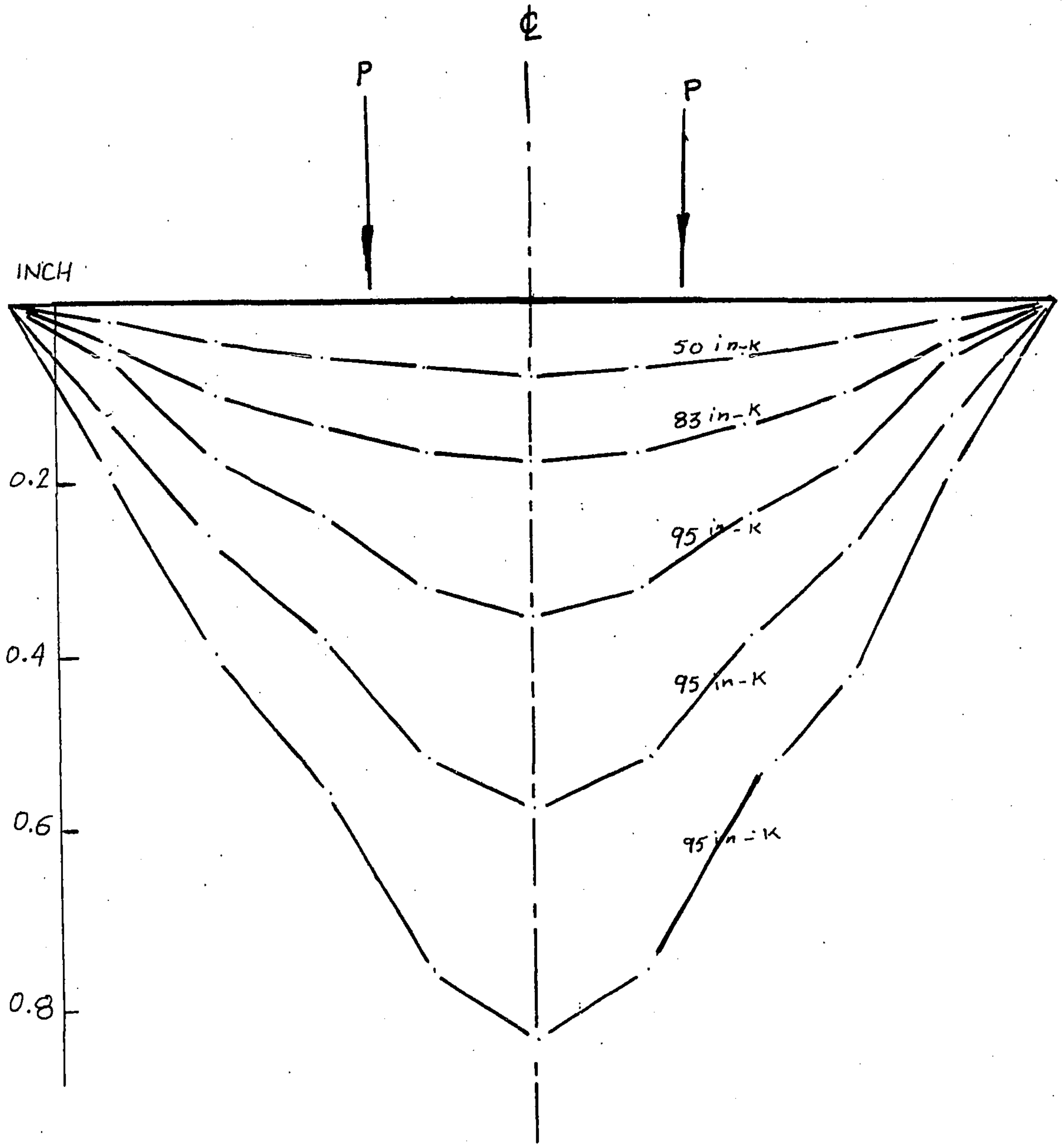


Figure 4.19 Span Deflections
of the Control Beam B-1.

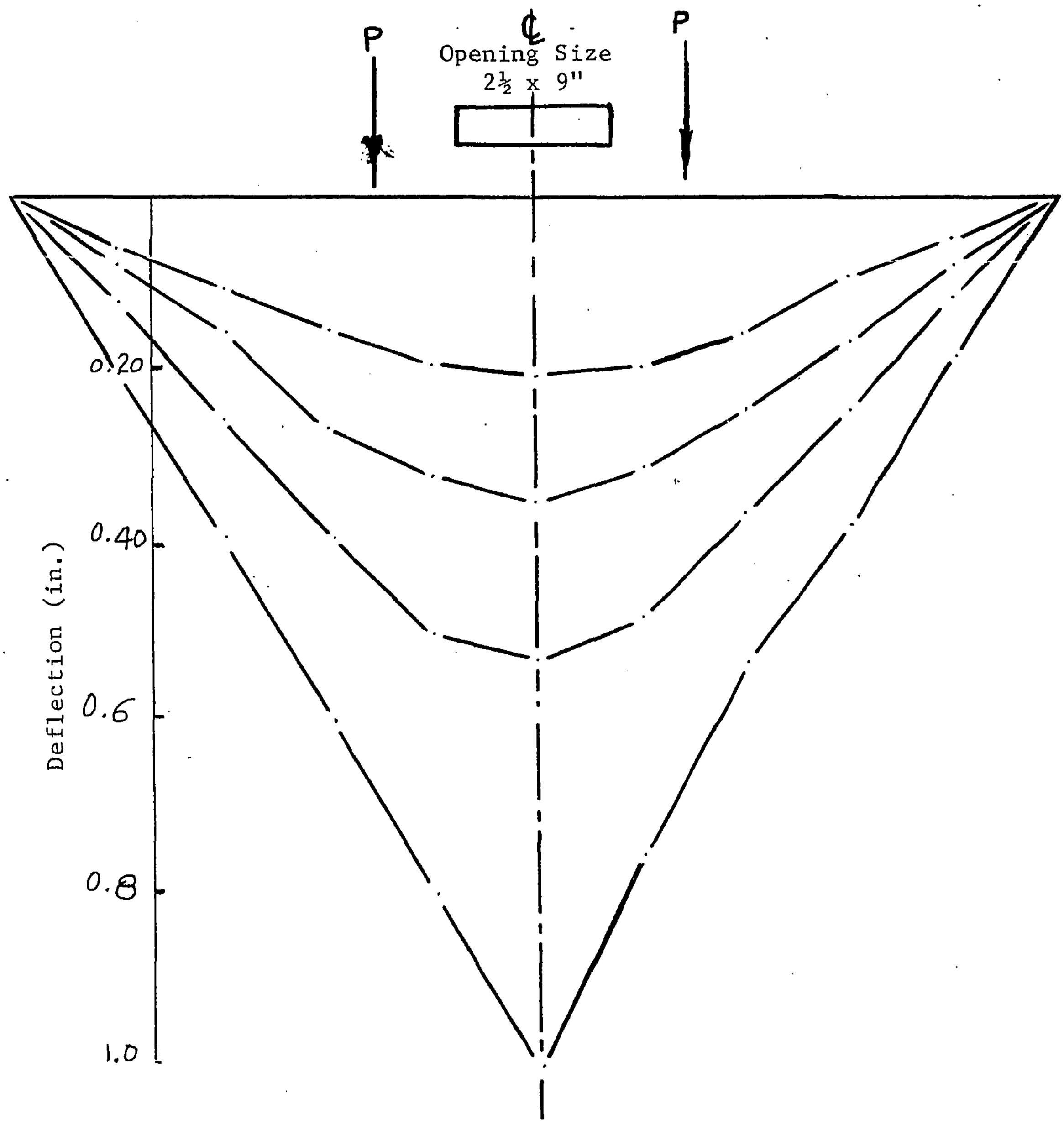


Figure 4.20 Span Deflections of Beam B-2.

should further be noted that the beam (C-3-1) carried a load equal to the yield load of beam A-1. However, it failed suddenly due to the diagonal tension cracks through the openings. Therefore, another beam was constructed identical to beam C-3-1 except that it contained reinforcement around its opening. This beam exhibited behavior similar to that of beam A-1 as shown in Figure 4.21.

Beam C-2 had a $2\frac{1}{2}$ x 6 inch opening with special web reinforcement around the opening. The response of this beam was similar to that of beam A-1, but the load versus deflection response indicates a decrease by approximately 20% in the deflection when compared with beam A-1. The progression of crack versus shearing force and moment at the center of the opening is shown in Figures 4.22, 4.23 and 4.25. The span deflection responses are shown in Figures 4.26 through 4.28.

4.5 Type D Beams

Two Type D beams were constructed. Beam D-1 had two openings and beam D-2 had three openings as shown in Figure 4.18. The two beams were loaded in the same manner as beam B-1 (two-point loading). The two openings near the supports were reinforced in a way identical to the reinforcement of beam A-3-2. The resulting load-deflection responses are shown in Figure 4.29. A comparison of the load-deflection responses of these two beams (type D) with those of beam B-1 shows that Beam D-1 yielded at the same load as beam B-1, but the yield load for beam D-2 load was roughly 20 percent less than that for beam B-1.

The moment-rotation responses (Figure 4.30) for the midspan indicate an agreement with the results given by the load-deflection re-

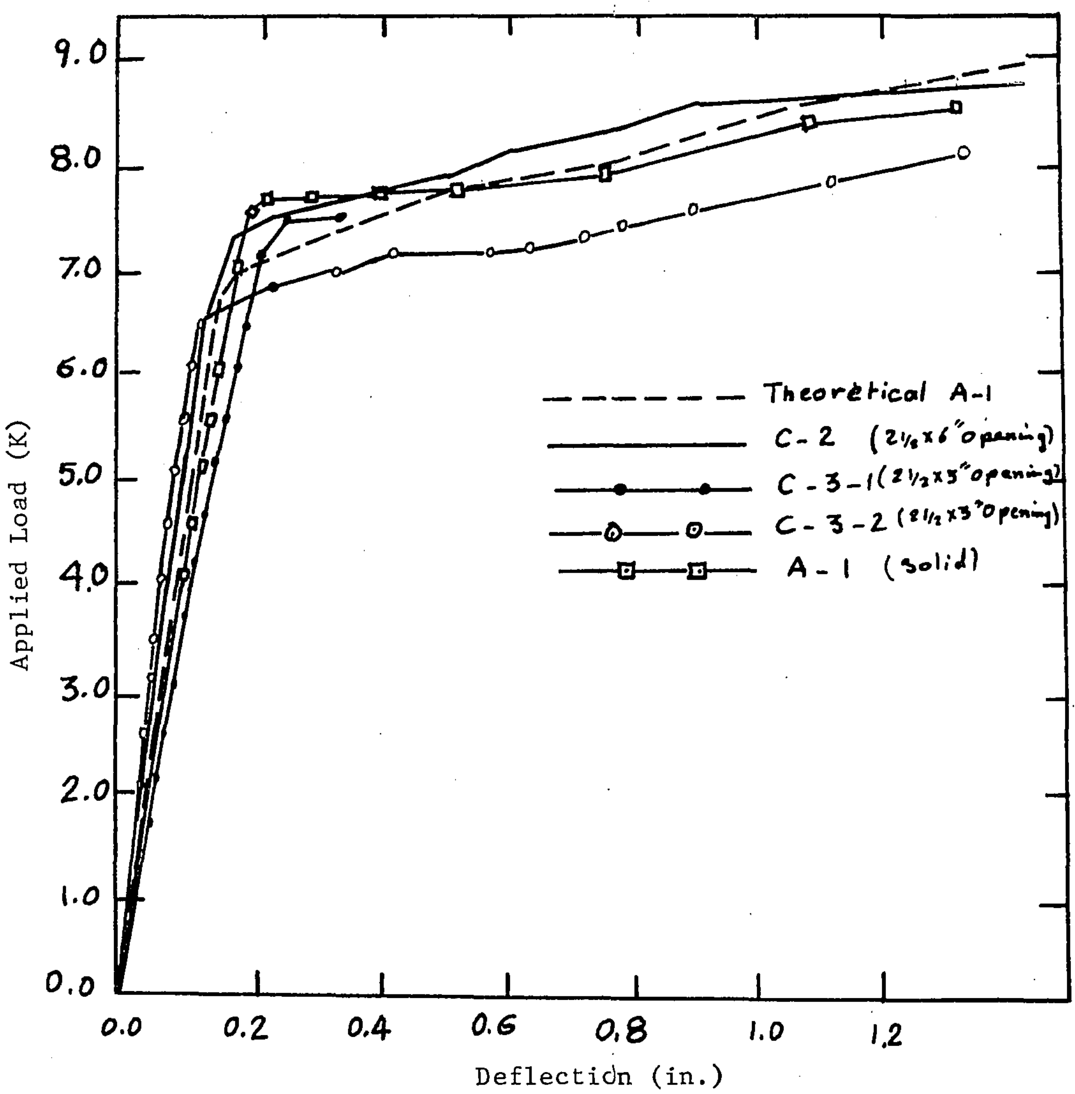


Figure 4.21 Applied Load Versus Vertical Deflection for Type C Beams.

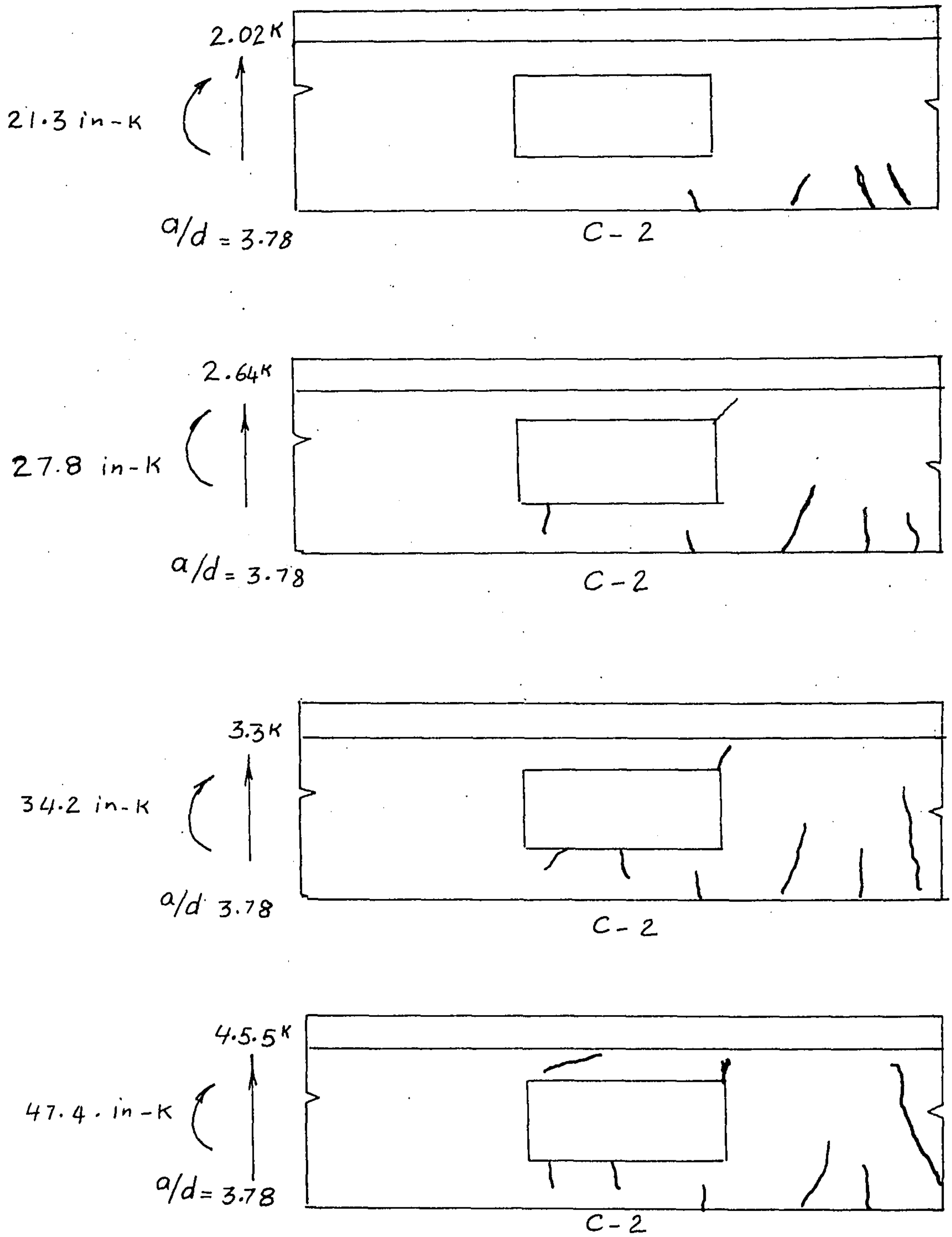


Figure 4.22 Cracking Versus Bending Moment and Shearing Force for Beam C-2.

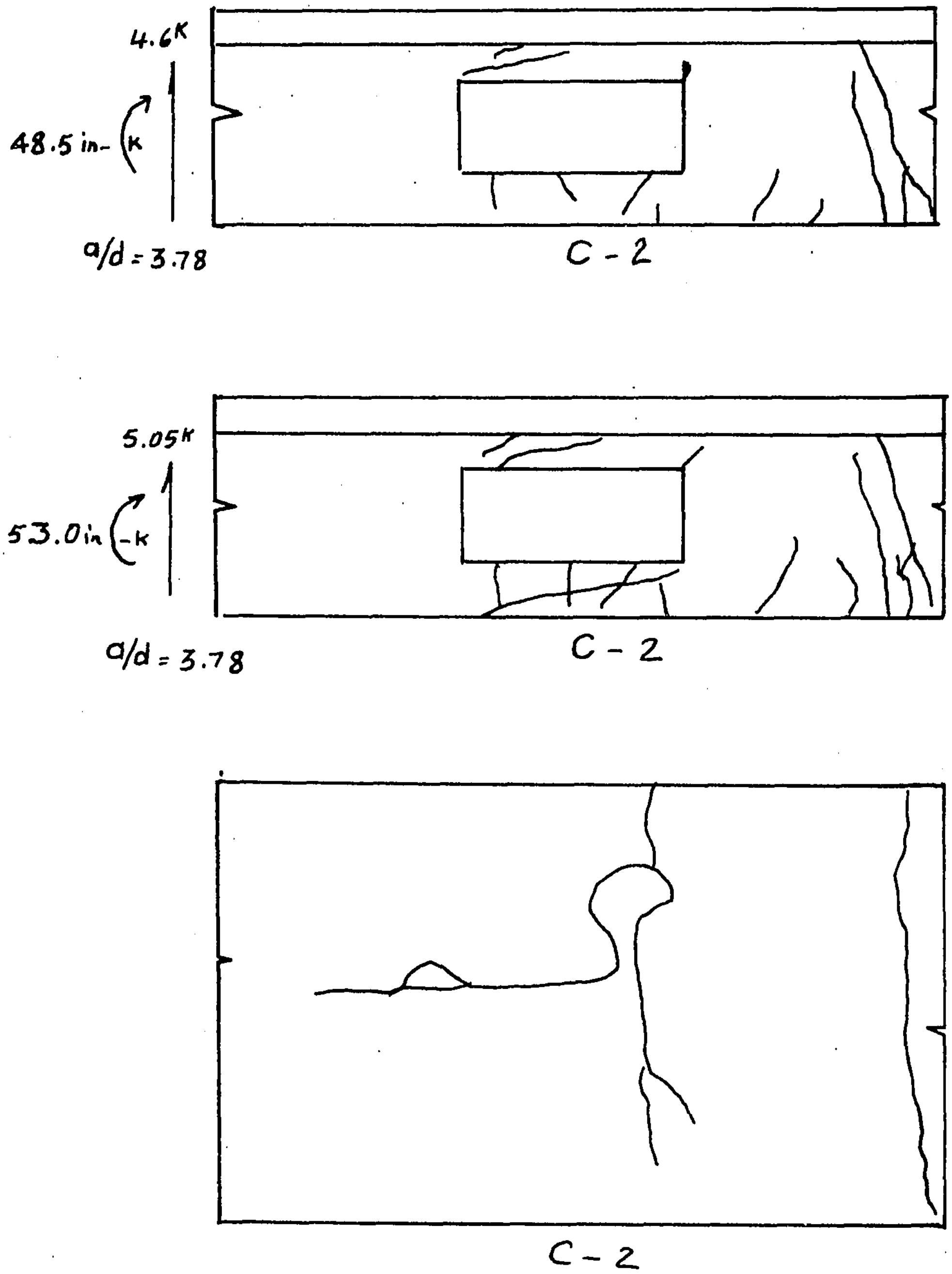


Figure 4.23 Cracking Versus Shearing Force and Bending Moment for Beam C-2.

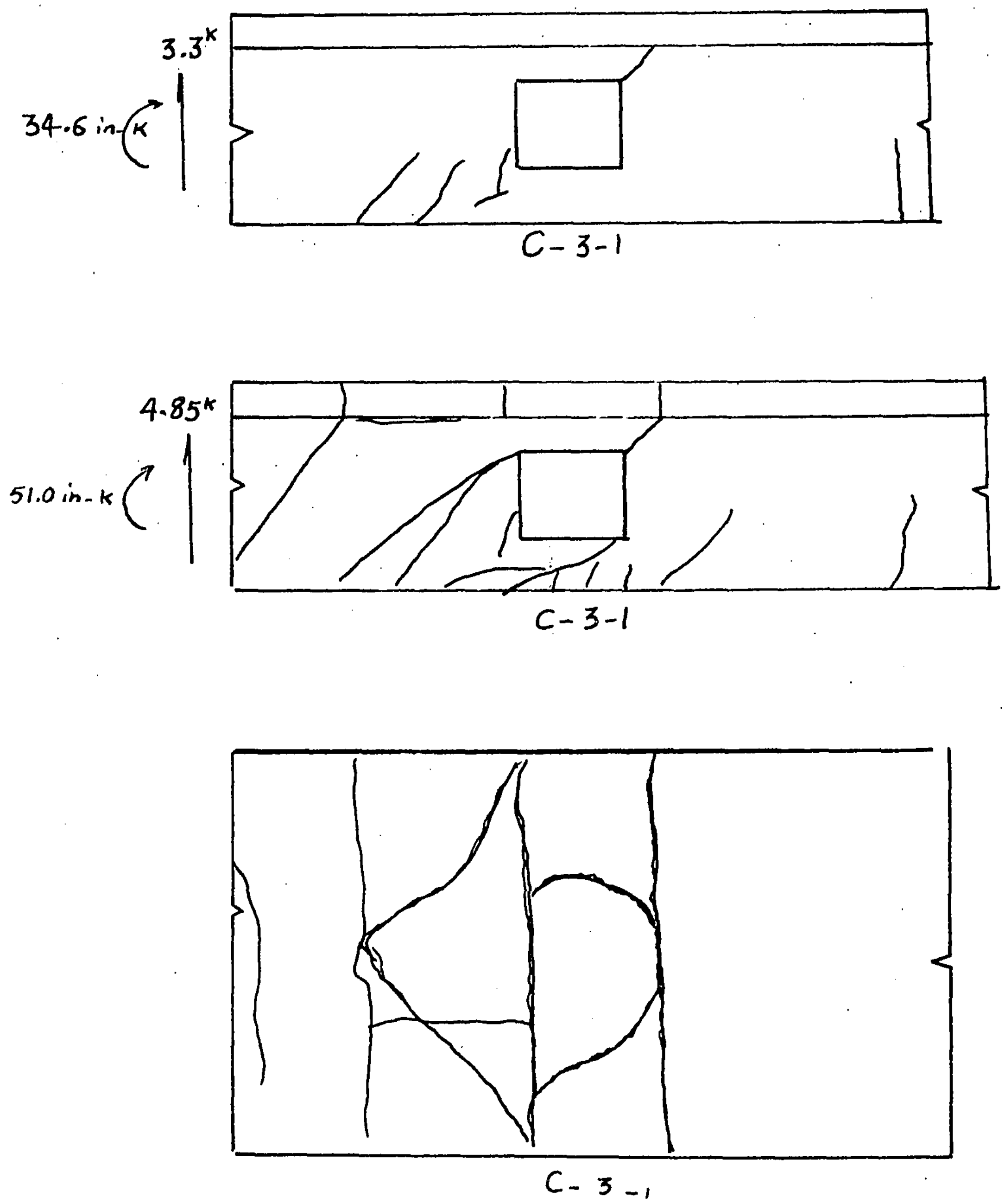
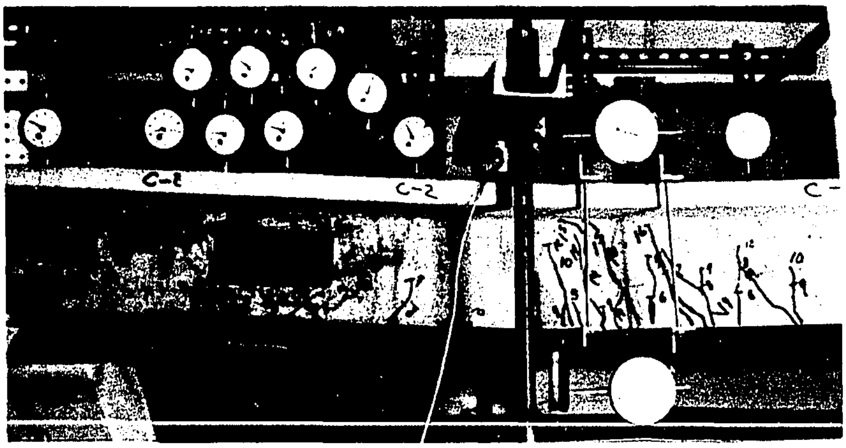
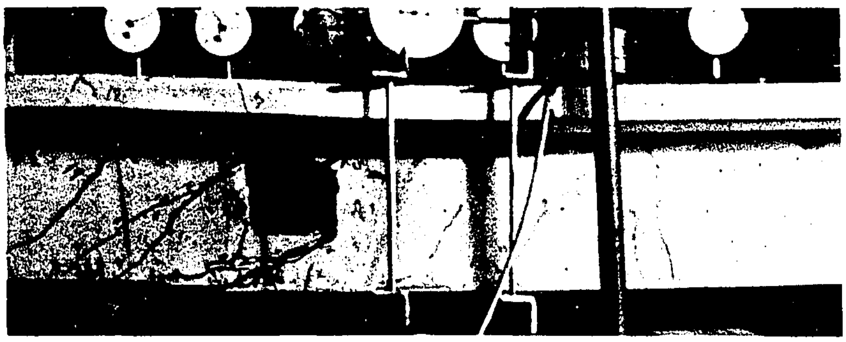


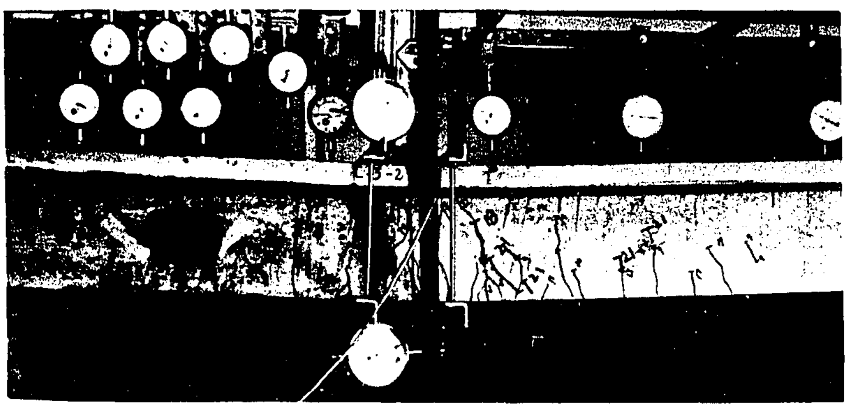
Figure 4.24 Cracking Versus Shear Force, Bending Moment Around the Opening of Beam C-3-1.



(a) Beam C-2



(b) Beam C-3-1



(c) Beam C-3-2

Figure 4.25 Cracking of Beams Type C at Failure.

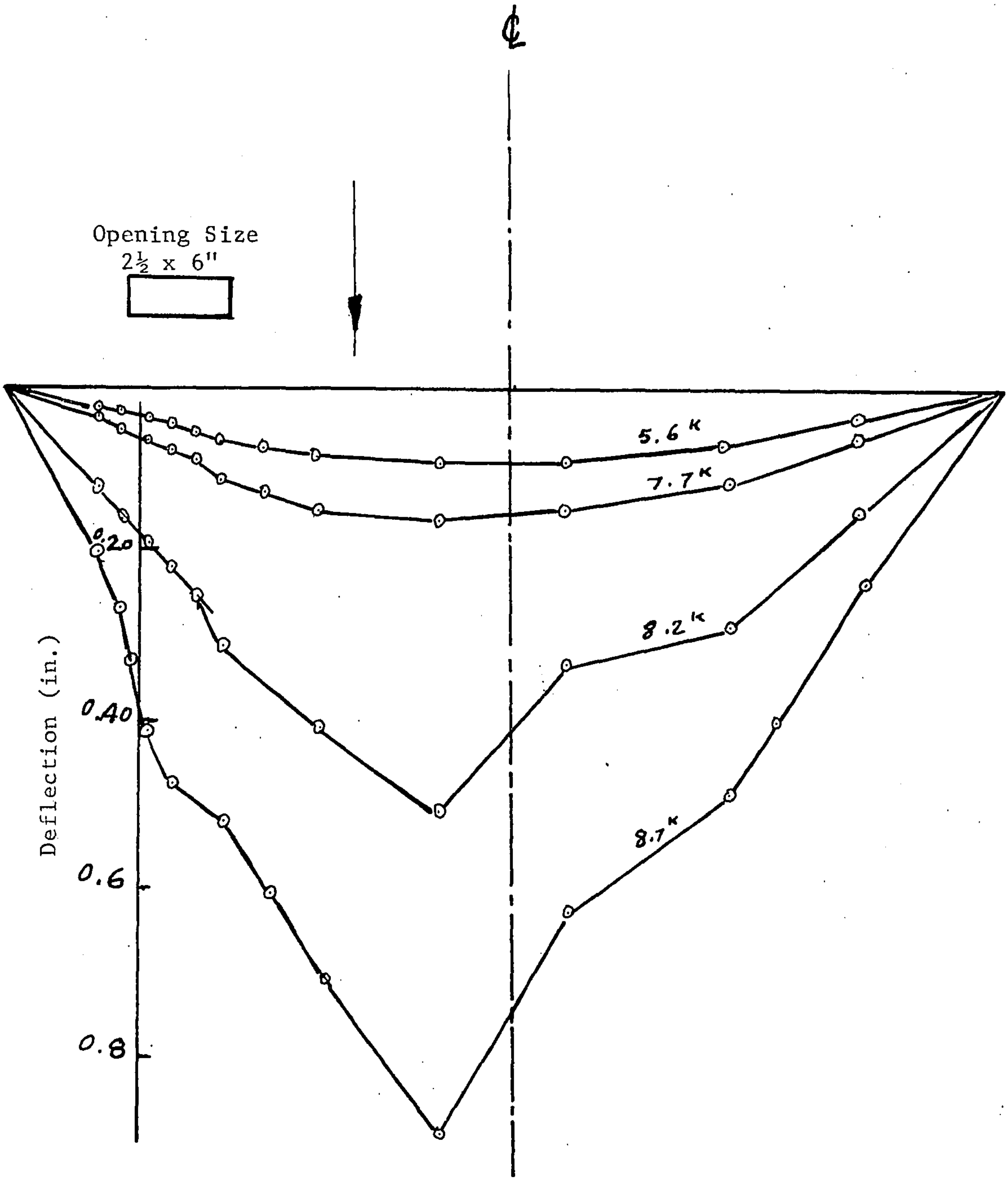


Figure 4.26 Span Deflections of Beam C-2.

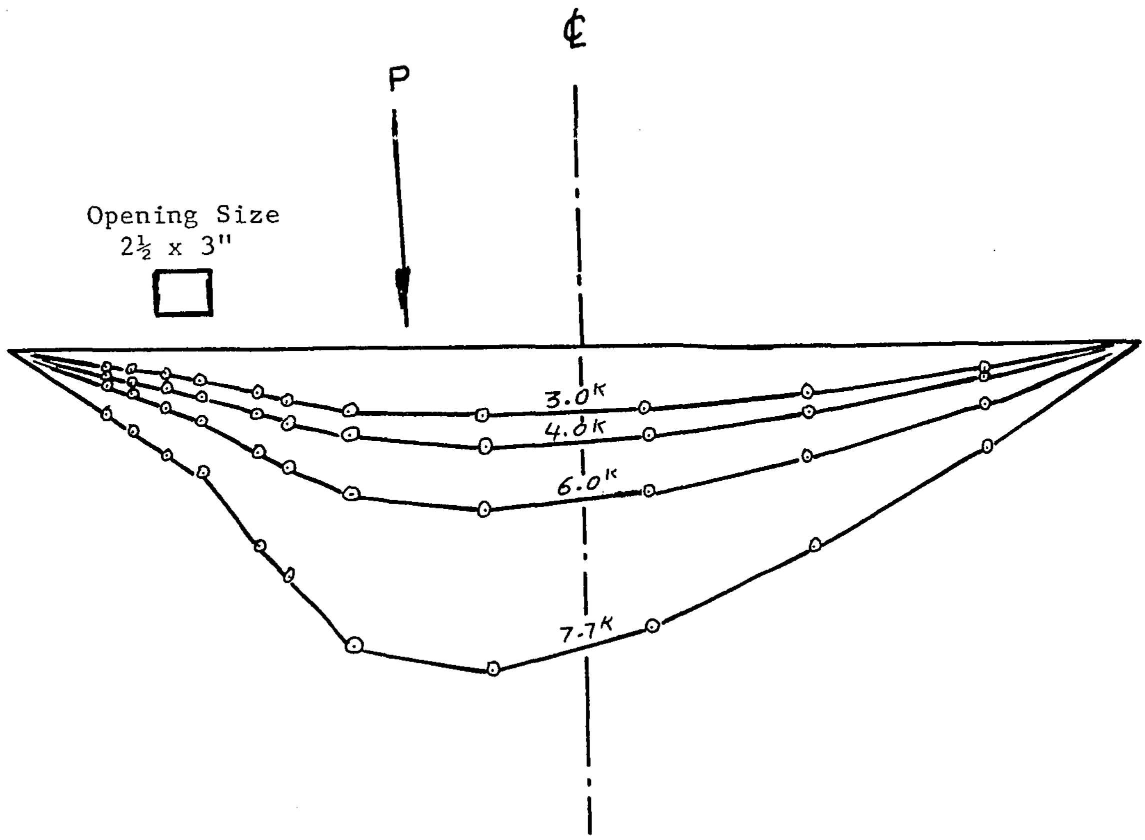


Figure 4.27 Span Deflections

of Beam C-3-2.

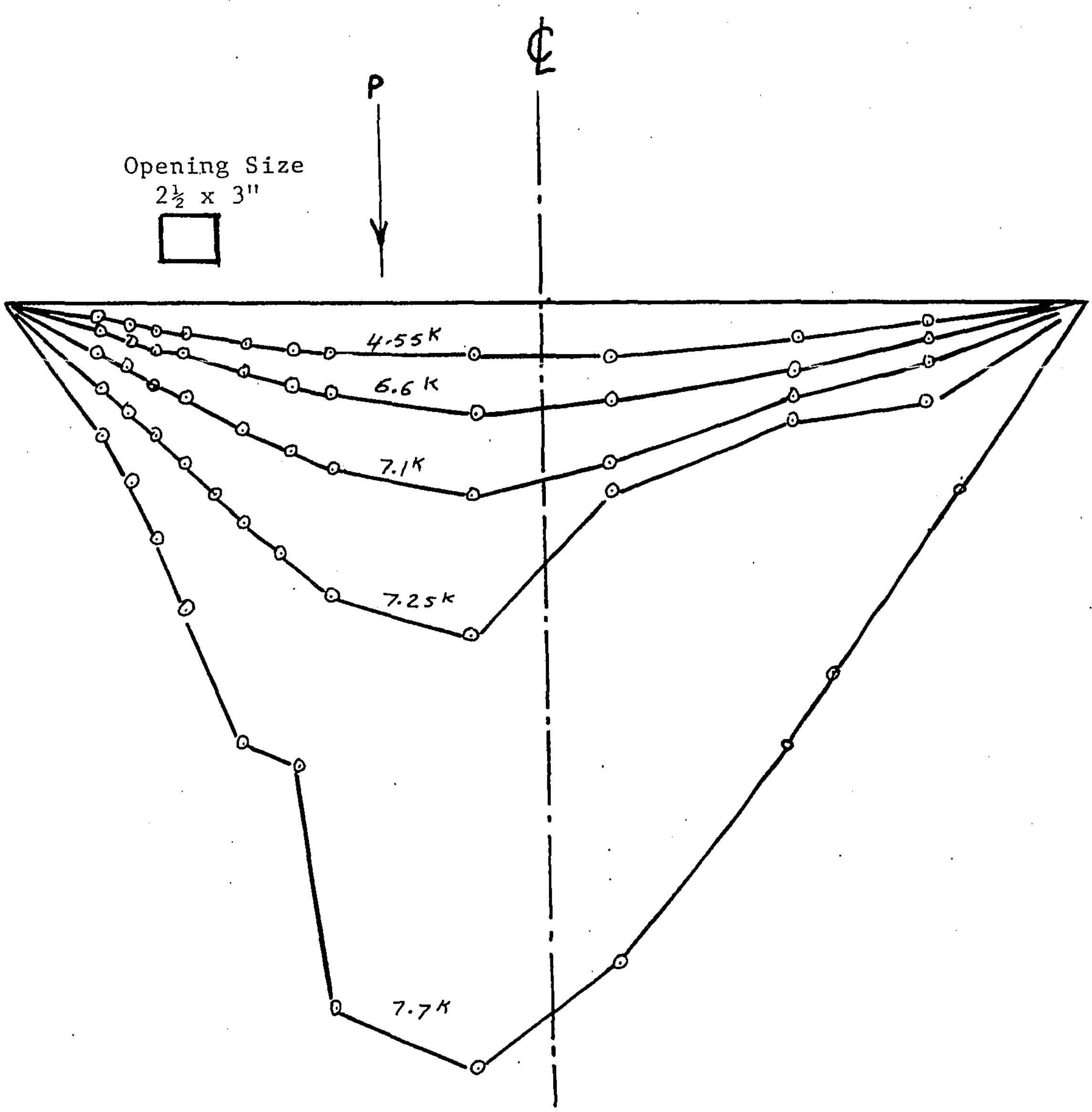


Figure 4.28 Span Deflections of Beam C-3-1.

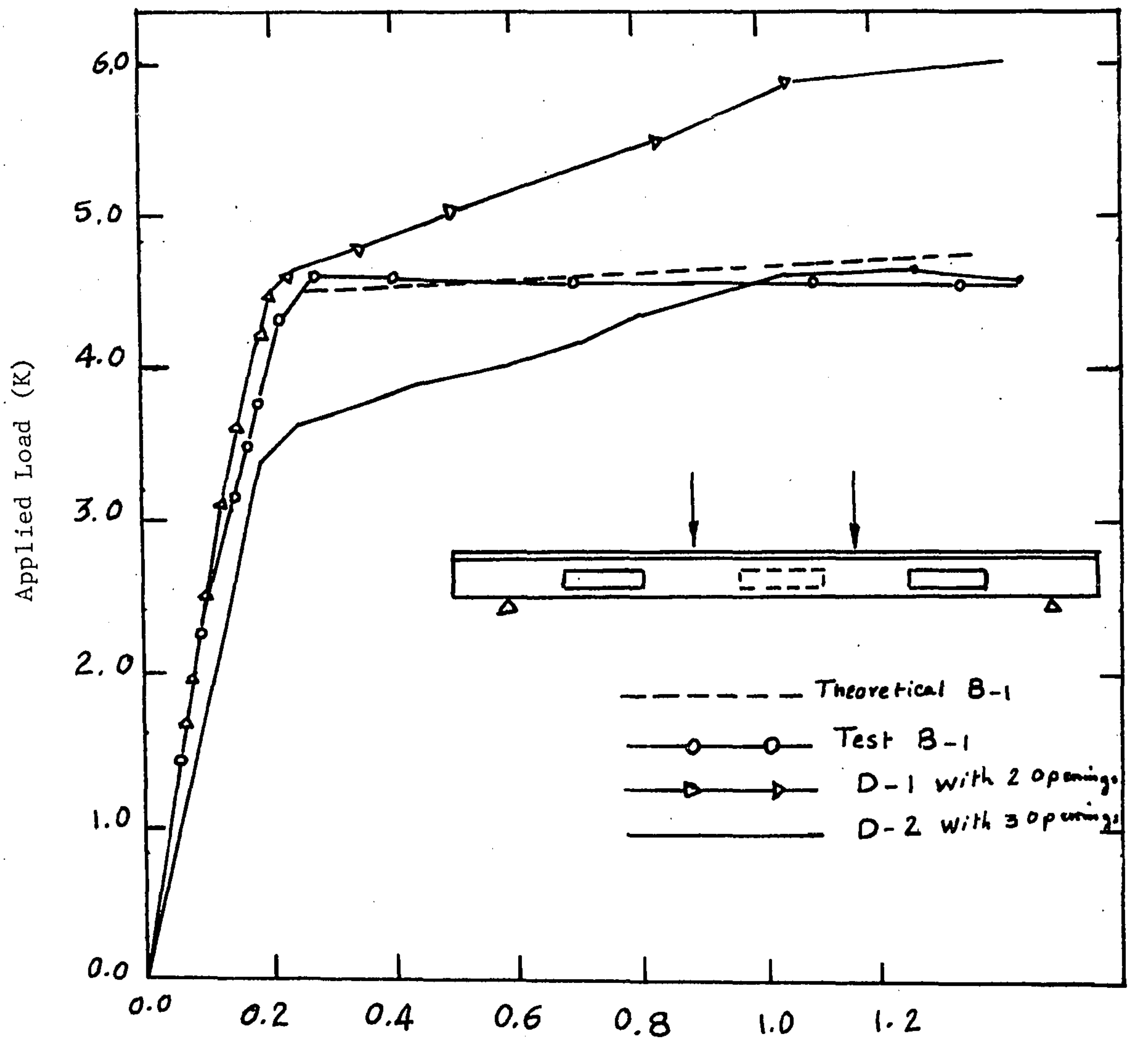


Figure 4.29 Applied Load Versus Vertical

Deflection for Type D Beams.

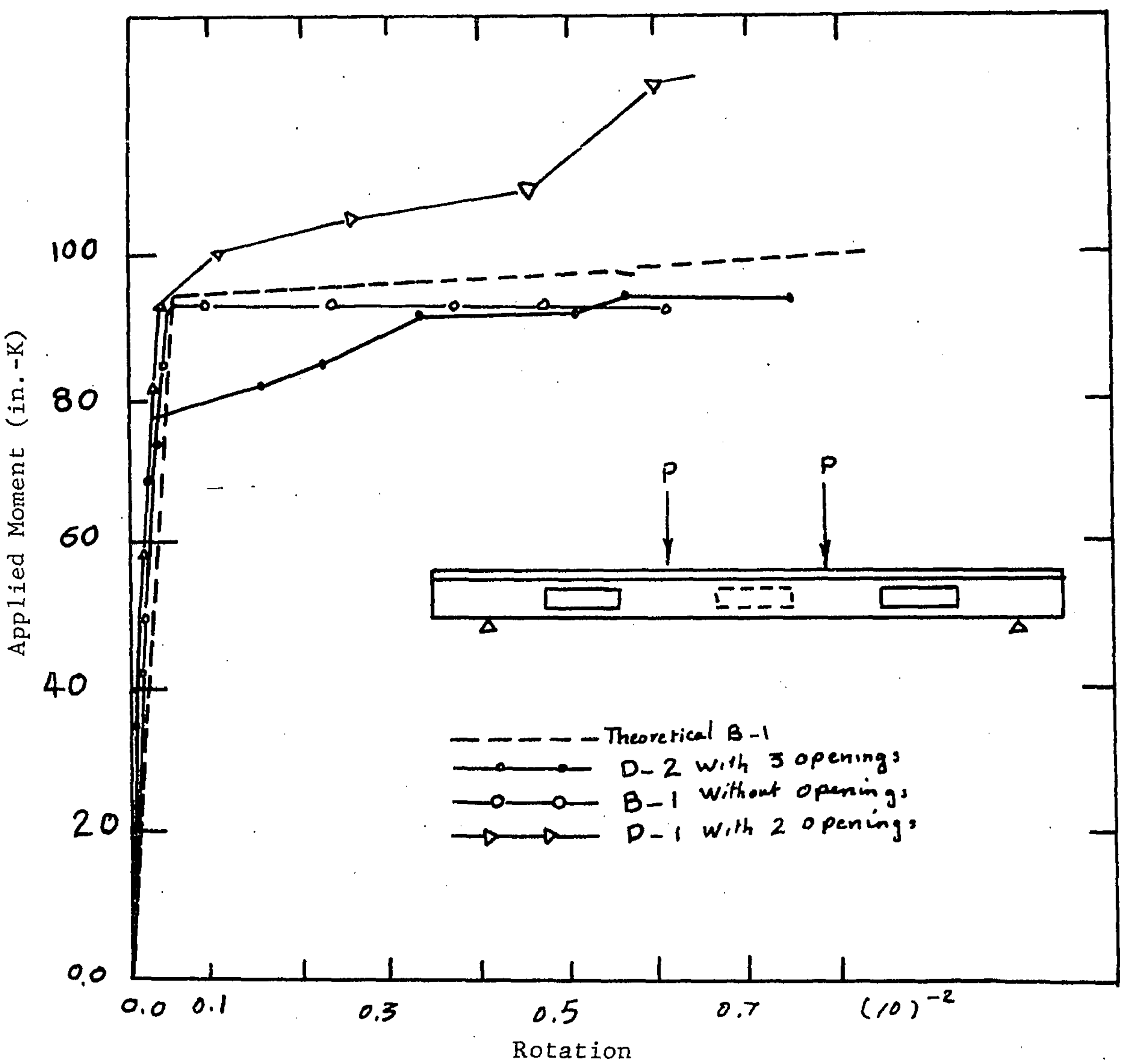


Figure 4.30 Applied Moment Versus Rotation at Mid-span for Type D Beams.

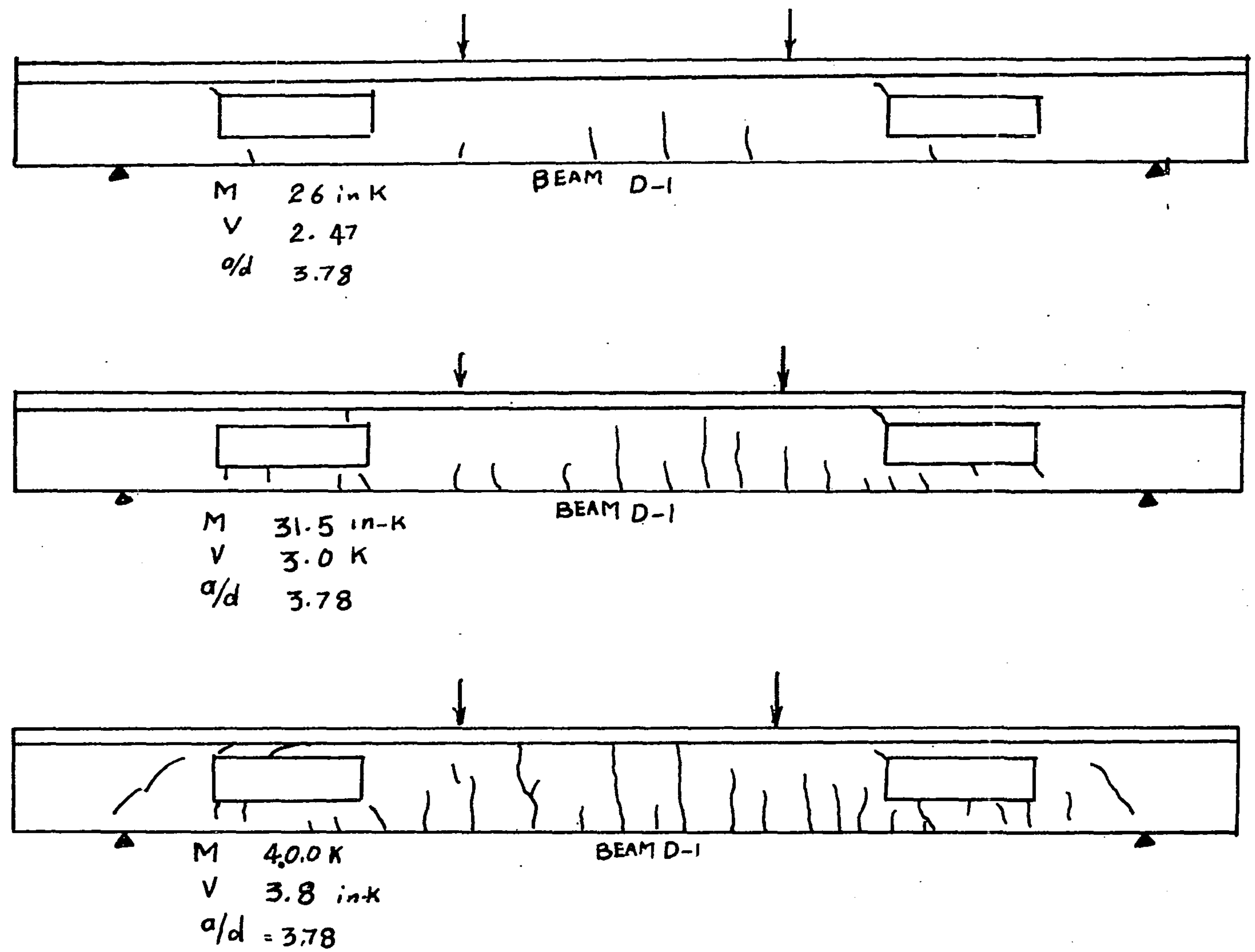


Figure 4.31 Cracking Versus Shearing Force and Bending Moment for Beam D-1.

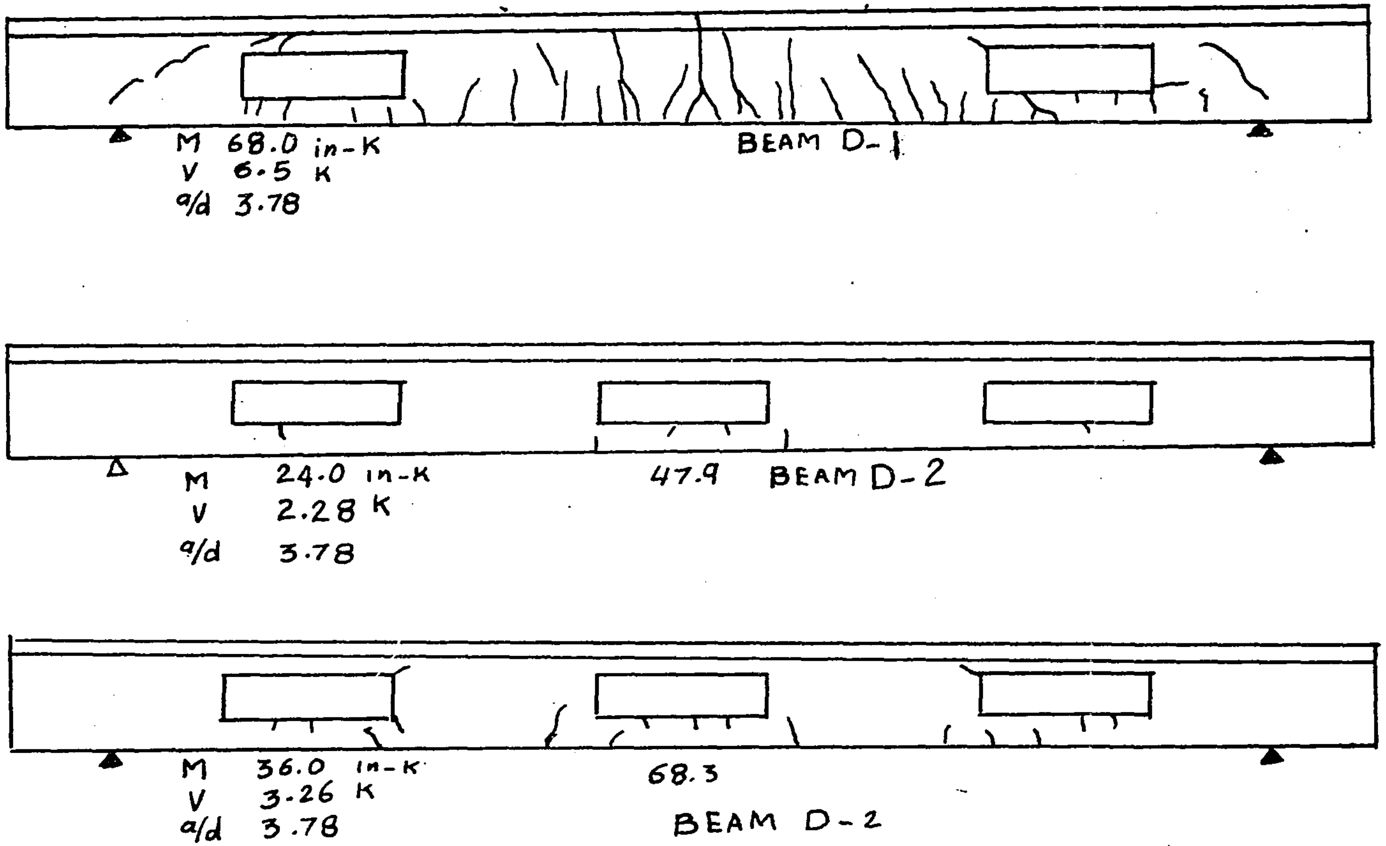


Figure 4.32 Cracking Versus Bending Moment and Shearing Force for Beams D-1 and D-2.

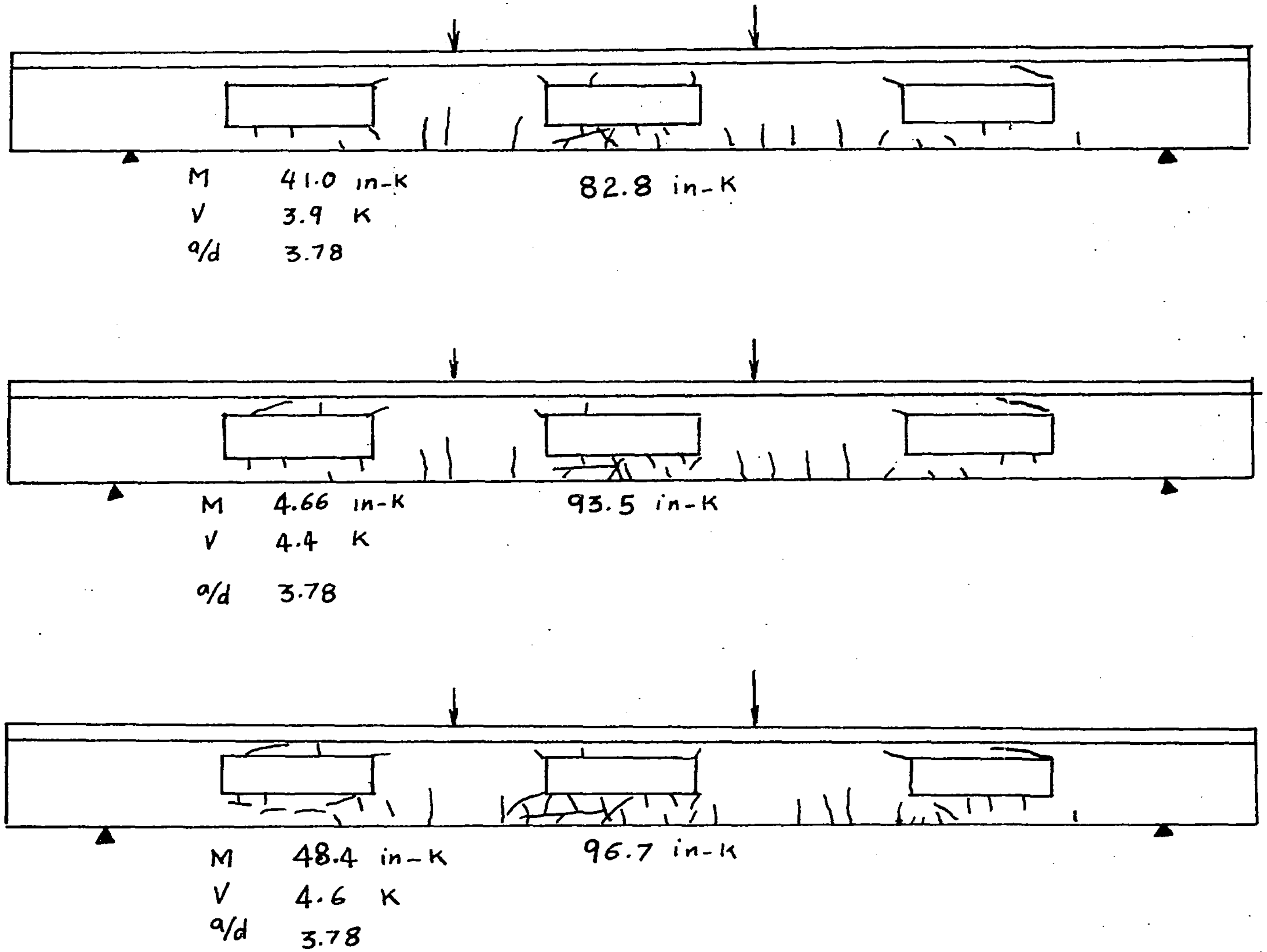


Figure 4.33 Cracking Versus Shearing Force and Bending Moment for Beam D-2.

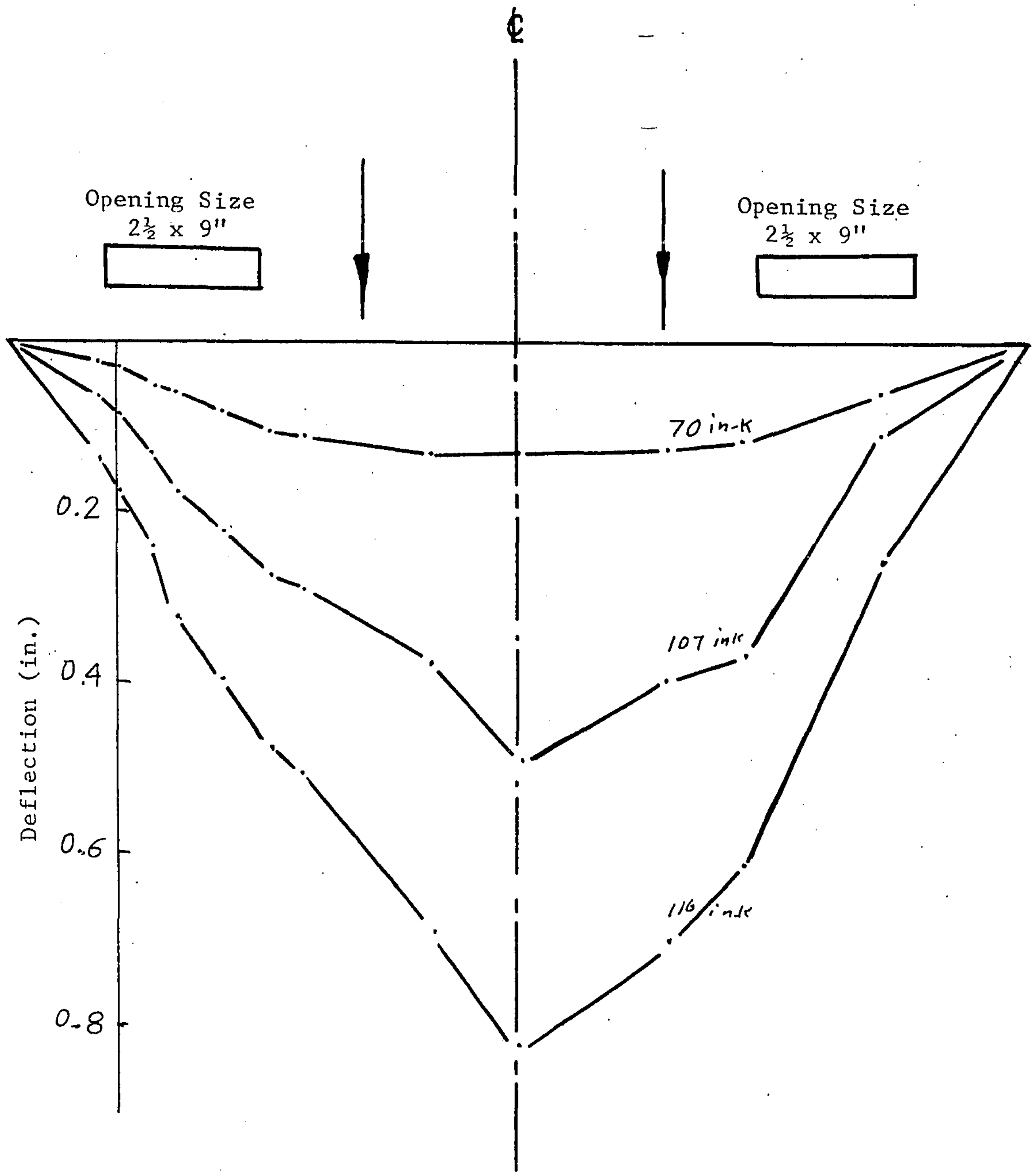


Figure 4.34 Span Deflection of Beam D-1.

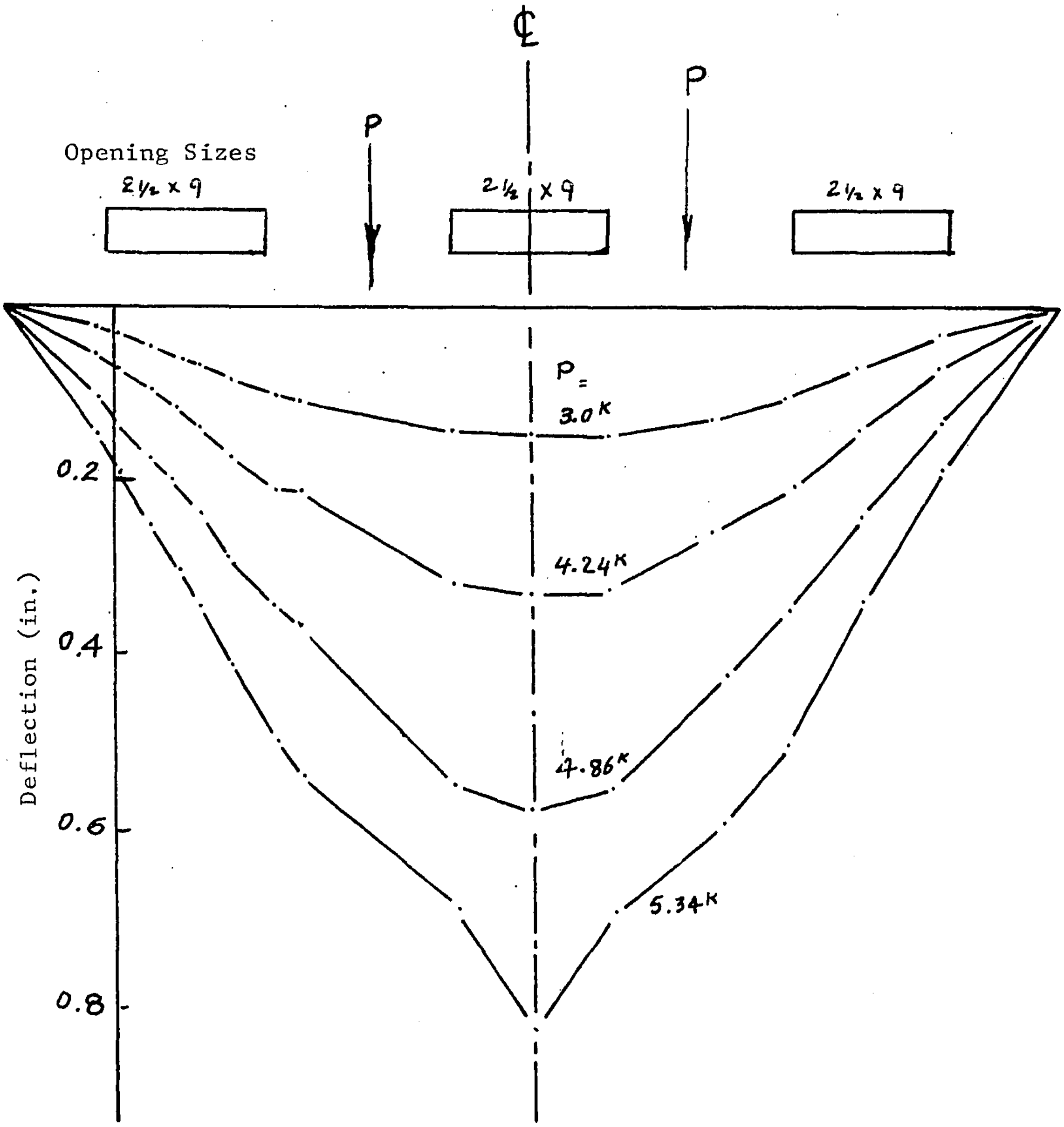


Figure 4.35 Span Deflection of Beam D-2.

sponse. Shearing forces and bending moment (acting at the center of the opening) versus cracking are shown in Figures 4.31 through 4.33. The failure of beam D-1 was similar to the failure of beam B-1 and the failure of beam D-2 was similar to the failure of beam B-2 as shown in Figure 4.18.

4.6 Type E Beams

Three Type E beams were constructed and tested to study the effect of shear-span on the behavior of beams each having an opening 2½ x 9 inches with special web reinforcement. The propagation of cracks for all beams of type E was similar; flexural cracks were first visible at roughly 30 percent of the ultimate load. These cracks slowly widened and extended toward the compression zone as the load was increased. After the yield of the tension reinforcement, the cracks widened rapidly and also increased in number at the level of the tension reinforcement. As the load was further increased, surface cracking and spalling occurred in the concrete compression zone.

Flexural compression type of failure occurred in the three beams. The behavior of type E beam was virtually identical to the behavior of the control beam A-1 (without opening) in all respects even the the cracks in the vicinity of the opening formed in a pattern similar to that of the control beam A-1 ignoring the existence of the opening. The load-deflection responses for the Type E beams are shown in Figure 3.36; shear and bending moment versus cracking are shown in Figures 4.37 through 4.39; and load-span deflections are shown in Figures 4.40 through 4.42.

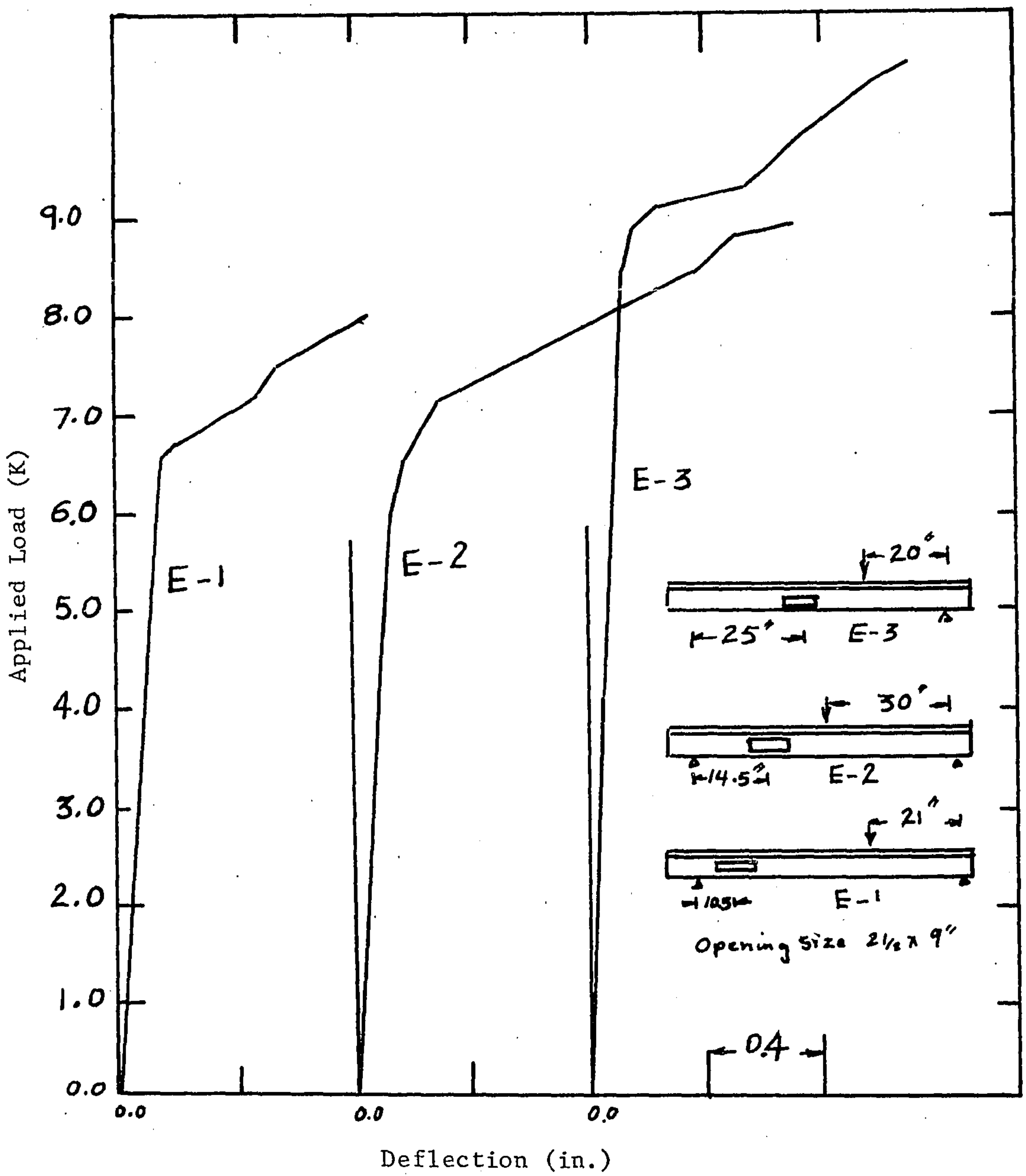


Figure 4.36 Applied Load Versus Vertical Deflection for Type E Beams.

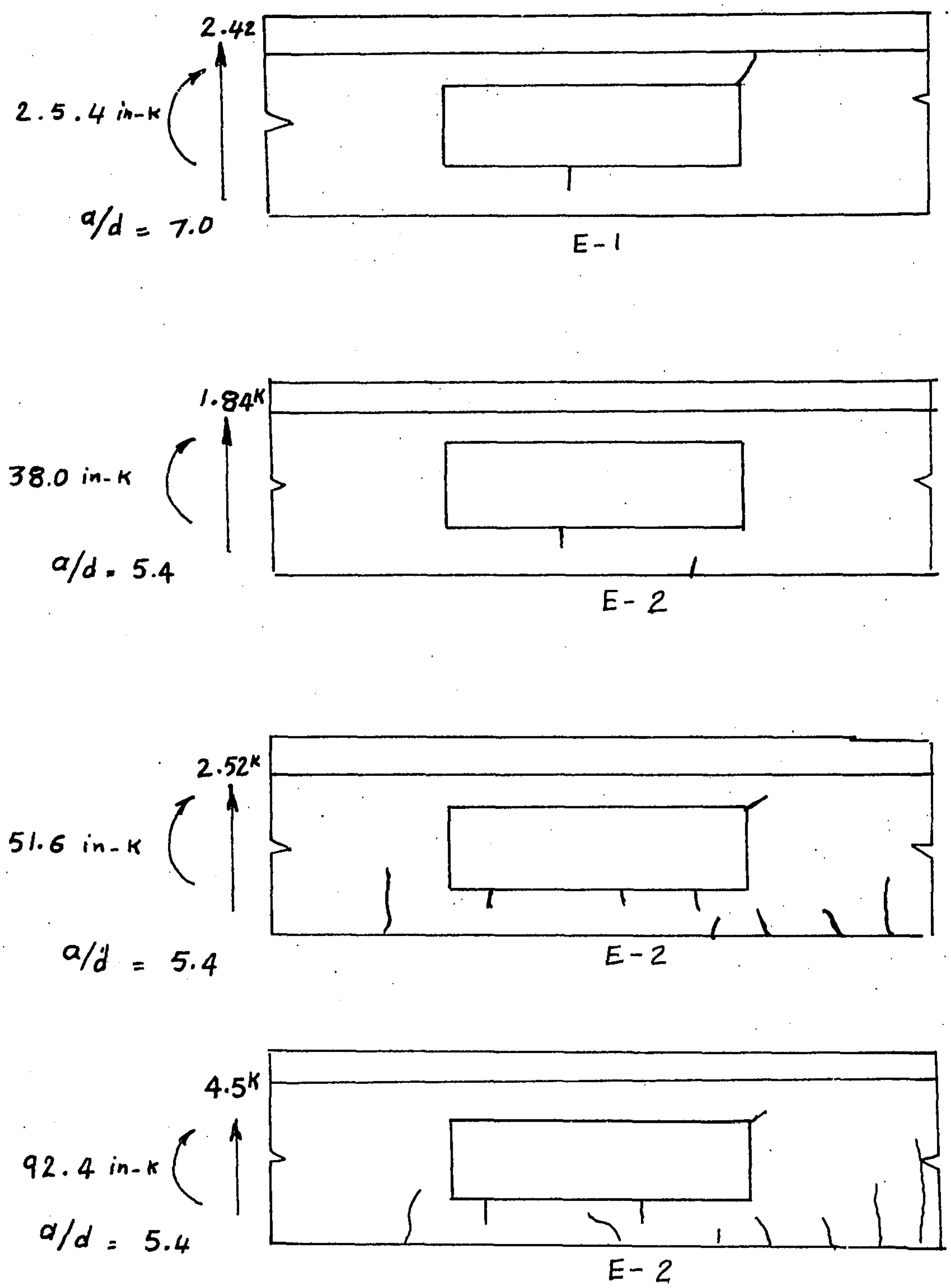


Figure 4.37 Cracking Versus Shear Force and Bending Moment Around the Opening of Beams E-1 and E-2.

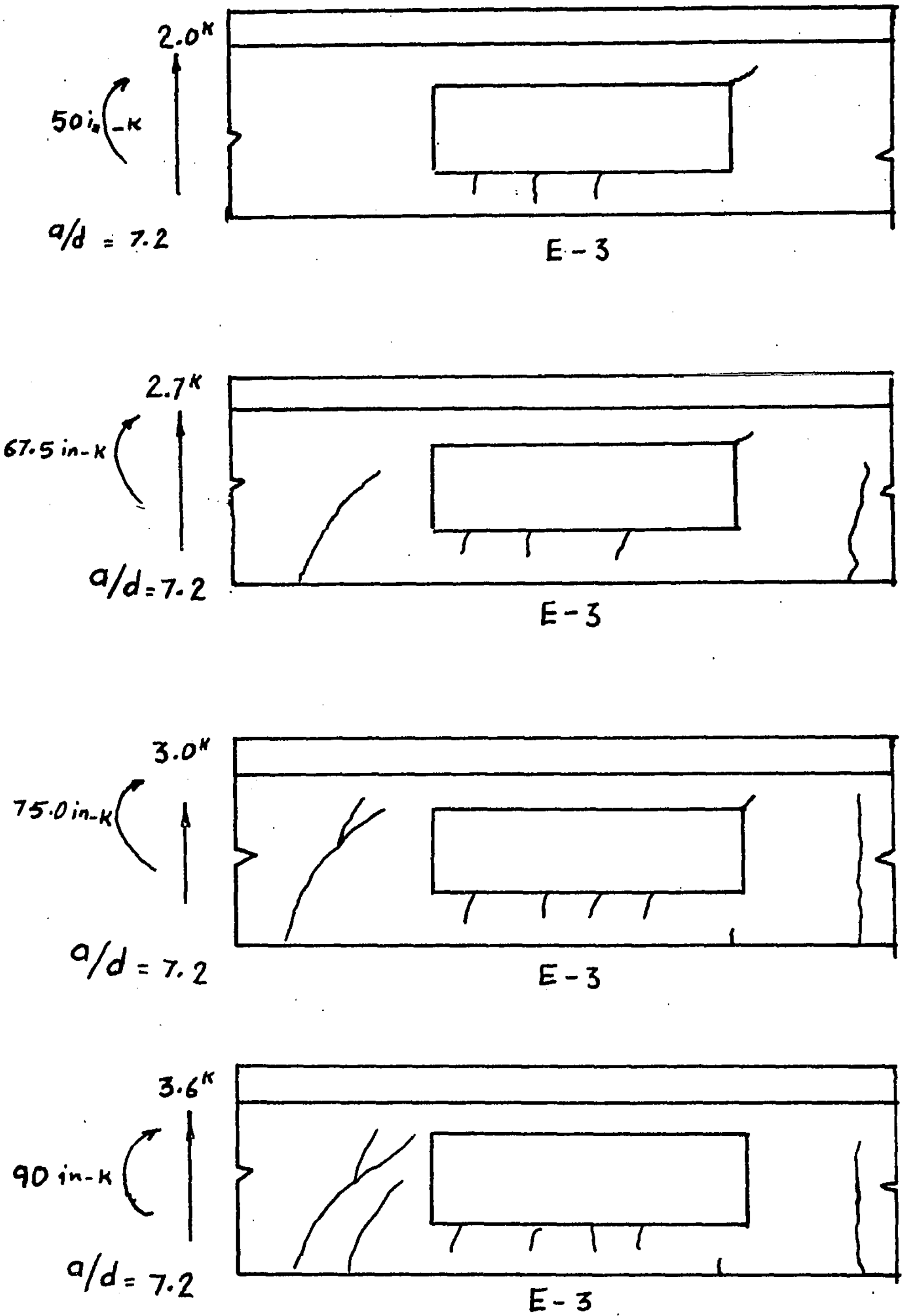


Figure 4.38 Cracking Versus Shearing Force, and Bending Moment Around the Opening of Beam E-3.

(a) Beam E-1

(b) Beam E-2

(c) Beam E-3

Figure 4.39 Cracking of Type E Beams at Failure.

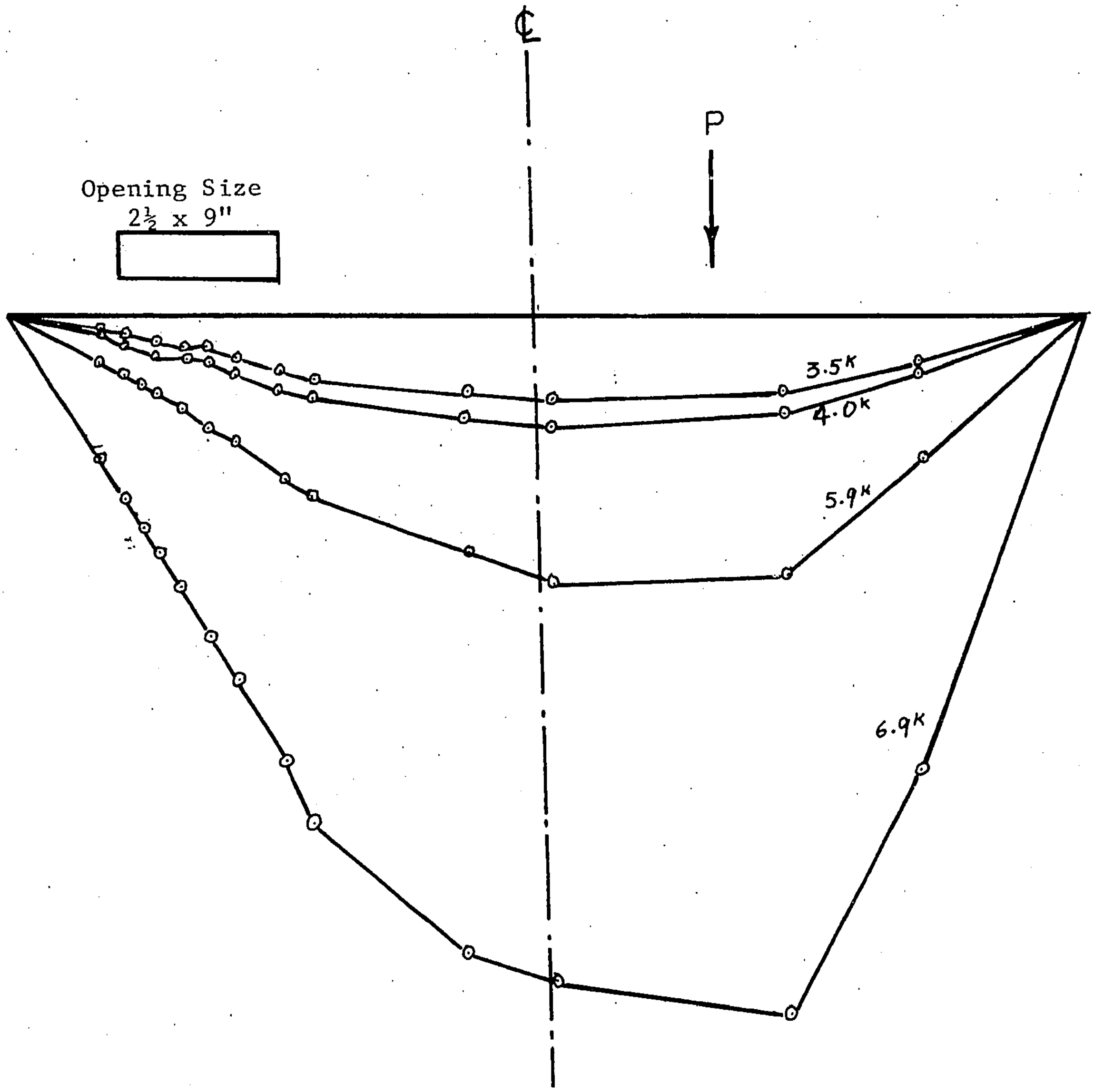


Figure 4.40 Span Deflections of Beam E-1.

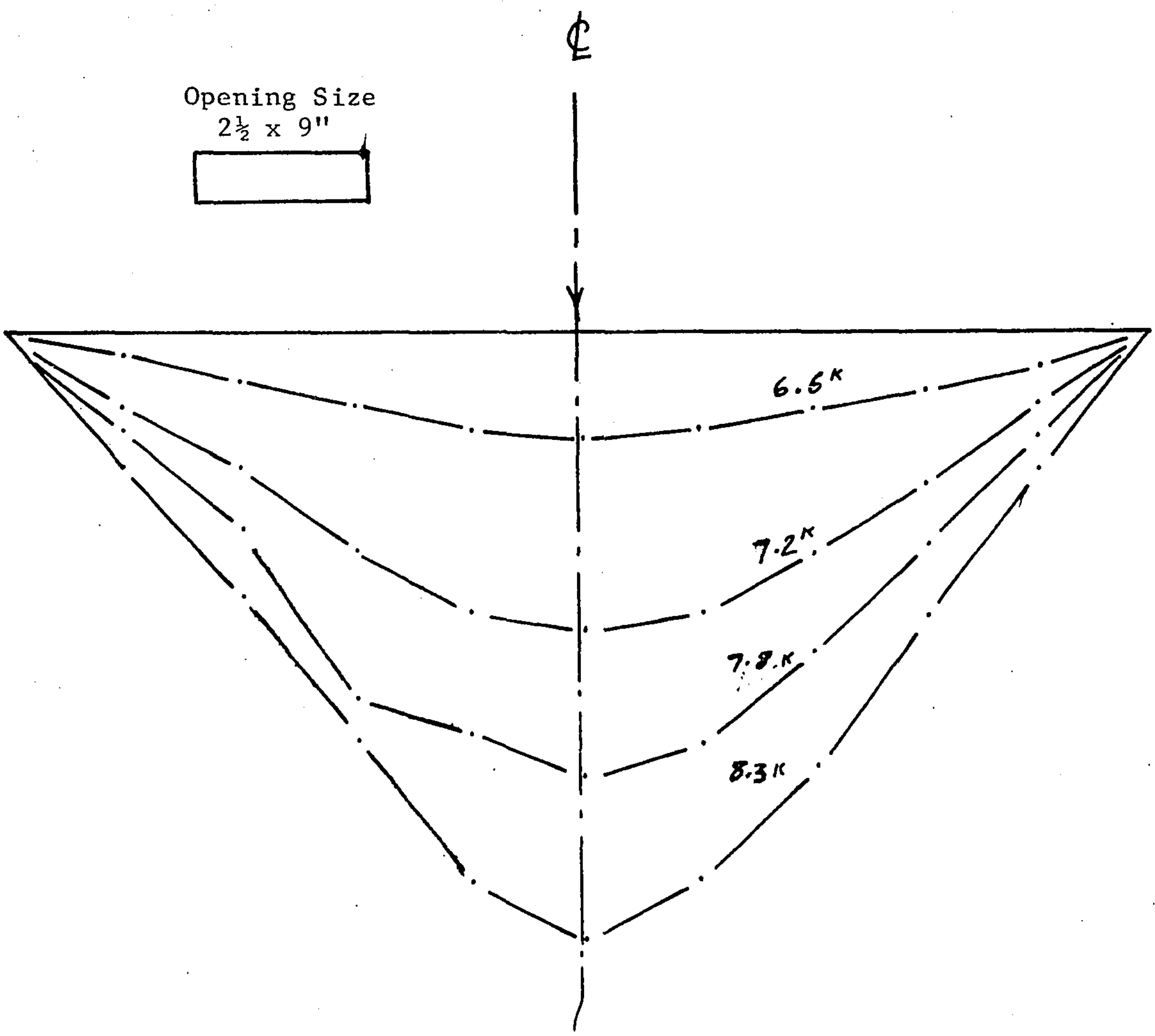


Figure 4.41 Span Deflection of Beam E-2.

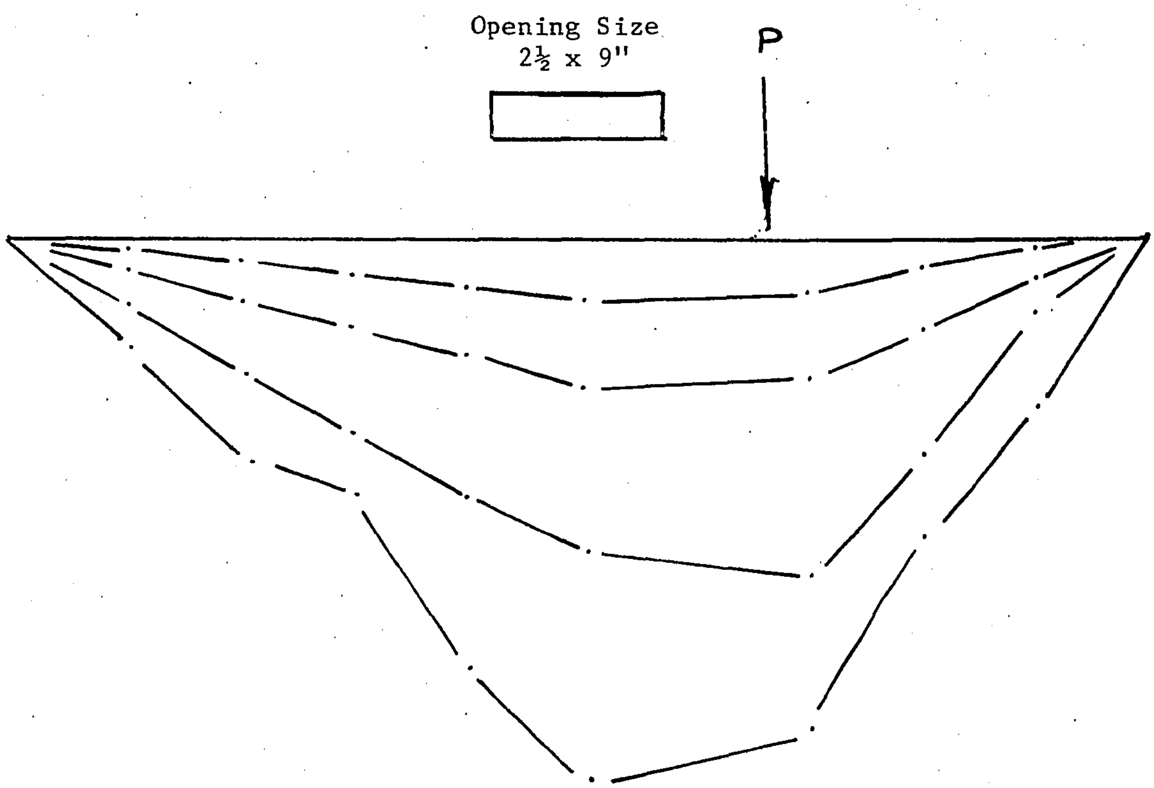


Figure 4.42 Span Deflection of Beam E-3.

CHAPTER V

DISCUSSION OF EXPERIMENTAL RESULTS

5.1 Effect of Geometrical Parameters

In this study, the behavior of reinforced concrete T-Beams with large openings in the webs was investigated with respect to the following parameters:

(1) Type of loading (one-point and two-point loadings),

(2) Special web reinforcement for the openings,

(3) Multiple openings, and

(4) Shear-span ratio.

The results were compared with those of the two control beams A-1 and B-1. Both beams (A-1 and B-1) were without openings and were subjected to one-point and two-point loadings, respectively. The test results obtained in the form of the load-deflection and moment-rotation responses for the beams were shown to be in good agreement with the results of a theoretical solution (the computer program LDDFN, Appendix A).

5.1.1 The Effect of Loading

Two Symmetrical Point Loading

Beam B-2 with an opening (size 2½ x 9 without special web reinforcement) was loaded symmetrically by two-point loading so that the opening was subjected to bending moment only. The test results of this beam in the form of load-deflection response, moment-rotation response, and cracking and collapse mechanism, were compared with the results of the control beam B-1. The following were observed:

(1) The yield load deflection and the yield moment rotation at midspan of beam B-2 were 14 percent less than those of the solid beam B-1 (Figures 5.1 and 5.2).

(2) The deflection of beam B-2 for all loading levels within the elastic range showed virtually no increase to that of the solid beam B-2 (Figure 5.1).

(3) The load deflection response of beam B-2 after yield and at a deflection approximately equal to 0.6 inches became virtually the same as that of the solid beam B-1 (Figure 5.1).

Since the two beams (B-1 and B-2) were geometrically identical except for the presence of the opening in beam B-2, the changes in the yield load-deflection and yield moment-rotation responses (14%) can be attributed only to the presence of the opening in beam B-2. Baker (3) reported similar results for rectangular beams.

Lorentsen (7) also reported similar results for T-Beams with openings which were reinforced at the chords. Lorentsen's study of the behavior of the beams was based on strain determinations at the top surface and at the level of the main reinforcement.

The Effect of One-Point Loading on Openings Without Special Web Reinforcement

Beam A-2-1 and its companion beam A-2-2 each with an opening $2\frac{1}{2}$ x 9 inches were subjected to bending moment and shearing forces.

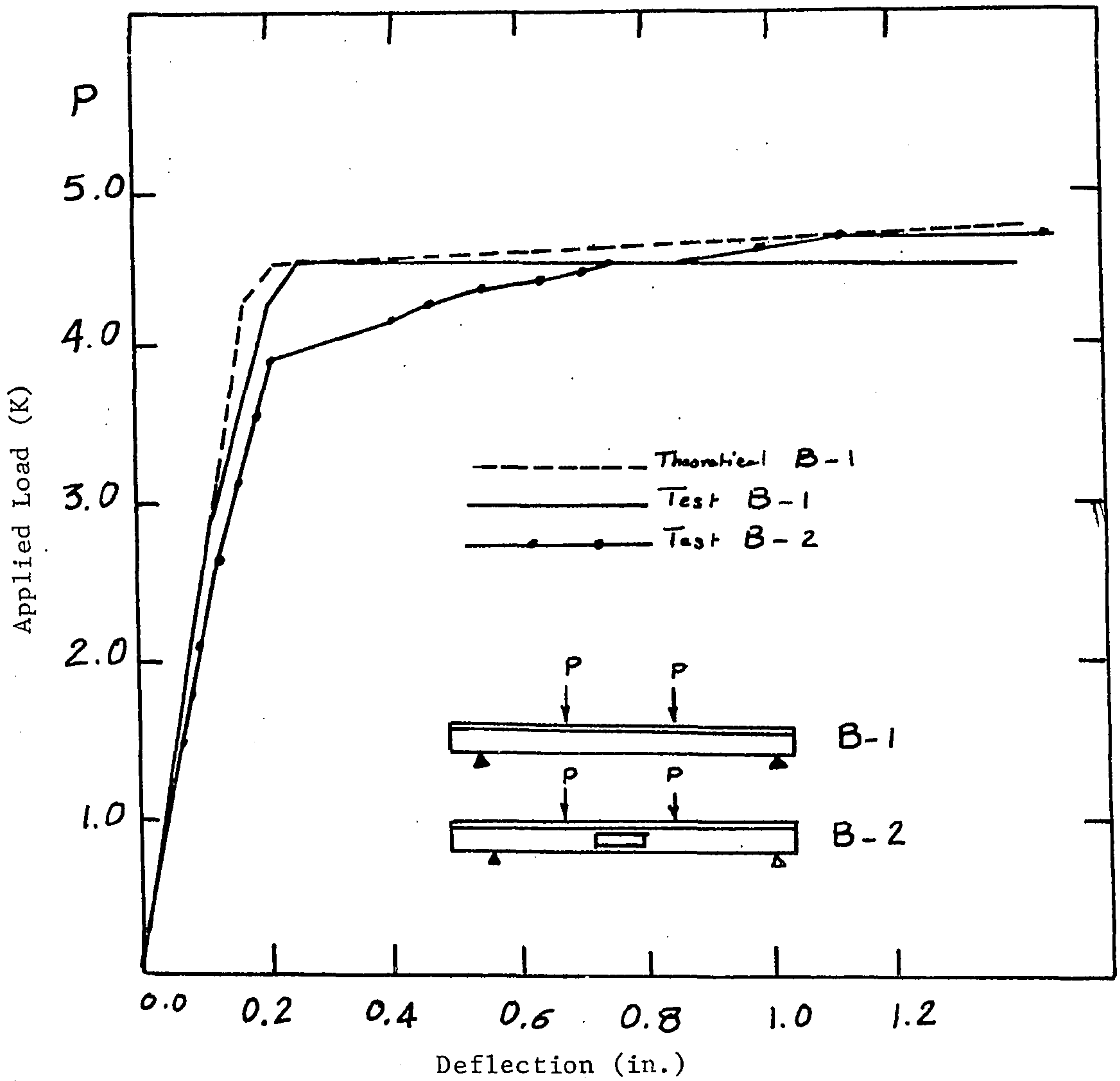


Figure 5.1 Applied Load Versus Vertical

Deflection for Type B Beams.

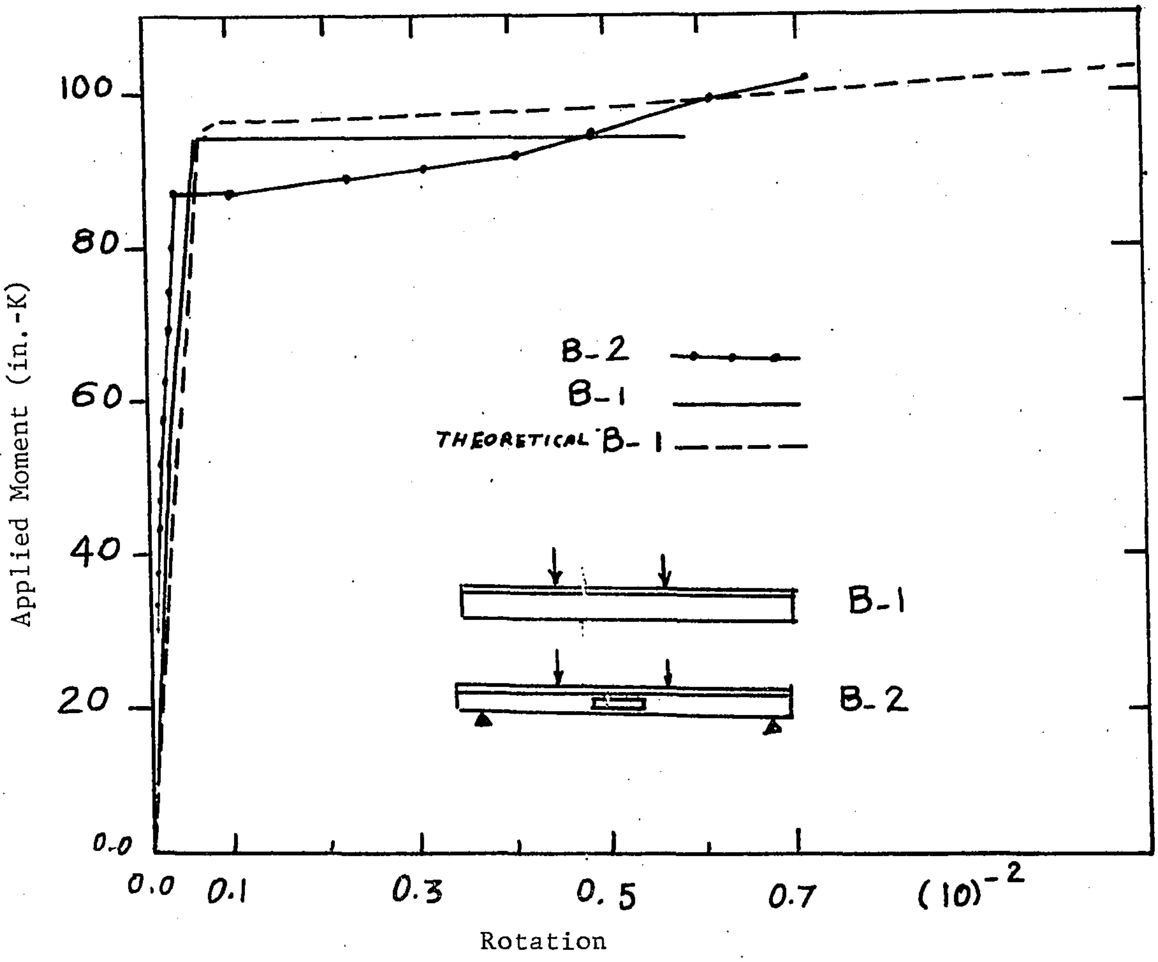


Figure 5.2 Applied Moment Versus Rotation at Mid-span for Type B Beams.

The behavior of both beams when subjected to loading compared with behavior of the control beam A-1 illustrates the following:

(1) The ultimate load for both beams (A-2-1 and A-2-2) was approximately 60 percent less than that of the solid beam (A-1) (Figure 5.3).

(2) The moment-rotation responses of beams A-2-1 and A-2-2 at the point of intersection of the chords of the opening with the main section of the beam (Section nearest midspan) indicated that the rotation was several times larger than the rotation of the main section (Figure 5.4).

(3) The load-deflection responses of beams A-2-1 and A-2-2 showed an average increase of approximately 63 percent in deflection over that of the solid beam (Figure 5.3).

(4) The curvature of the top chord of beam A-2-1 and A-2-2 rotated in a clockwise direction as shown in Figure 5.5.

The behavior of beams A-2-1 and A-2-2 can be explained as follows:

The rotation of the section BC (Figure 5.5) as measured by the rotation meter was several times greater than that of the solid section of the beam. Consequently the strains at the section BC were several times larger than those of the solid section of the beam. These excessive strains caused the development of cracks at the corners of the opening, especially at points E and D of the top chord (Figure 5.5). As the loading increased, these cracks increased and widened; consequently the top chord rotated about points A and B. This type of rotation caused an increase of length in the top chord equal to the difference in length between BD and the diagonal AB. (This is explained by the fact that the beam originally rested on DB and after the cracks had formed the beam rested on AB). This net increase in length of the top chord was larger than the increase in the length of the bottom chord (The increase in length of the bottom chord is due to the strain of the main reinforcement.) This net increase in length of the top chord caused an upward vertical deflection at point D as shown in Figure 5.5.

Neglecting minor contributions due to dowel action in the compression reinforcement and the contribution of small forces in the amount of concrete that was in contact (virtually points of contacts), it could be assumed that the bulk of the shearing force was carried by the bottom chord.

This assumption of the shear force being carried by the bottom chord will be used in a later section in order to facilitate the discussion regarding the contribution of the special web reinforcement.

5.1.2 Web Reinforcement Effect On Beams With Openings

The test results of beams A-2-1 and A-2-2 clearly indicated that the assumption can be made that the top chord was not carrying any significant part of the shearing forces or bending moments. Therefore, in order to make the top chord stiff enough to carry shearing force and transfer bending moment, beam A-3-1 was constructed and its top chord was reinforced in the manner shown in Figure 3.2, Section c-c. This reinforcement was similar to the type used by Lorentsen (5) and Nasser, et al. (7).

The behavior of Beam A-3-1 when subjected to loading compared to that of the control beam A-1 can be stated as follows:

(1) Deflections of beam A-3-1 increased approximately 40 percent in the elastic range and the yield load was roughly 60 percent of that of beam A-1 (Figure 5.3).

(2) The rotation of the section containing the opening in beam A-3-1 with respect to the moment at that section increased approximately 400 percent over that of the solid section (beam A-1)(Figure 5.4).

(3) The top chord span rotated in a clockwise direction as shown in Figure 4.8 c. The end of the top chord nearer to the support cracked and deflected vertically upward and the end nearer to the center of the span of the beam cracked and deflected vertically downward. This is shown in Figures 4.8 c and 5.6.

(4) Beam A-3-1 failed due to the diagonal tension in the top and bottom chords.

The behavior of beam A-3-1 was similar to the behavior of beams without reinforcement except that when the top chord of beam A-3-1 was reinforced, its carrying capacity increased from 40 percent to 60 percent of the solid beam.

The rotations measured by the rotation meter at any location furnished a measure of existing strain and stresses at that location. The readings of the rotation meter when placed at one end of the opening showed that the stresses in the section containing the opening were four times larger than those in the solid section (Figure 5.4). Similar high stresses were also present in the beams studied by Lorentsen.

The purpose of constructing and testing beam A-3-1 was to replicate the previous works of Nasser, et al. (7) and Lorentsen(5) in order to check the validity of their claims. Although beam A-3-1 behaved in a manner similar to that predicted by Nasser, et al. and Lorentsen, the presence of the high concentration of stresses renders such beams highly impractical as well as economically undesirable. It could be further emphasized that since the purpose of having openings in beams is a problem of economy, the designer faced with the kind of beams suggested by Lorentsen and Nasser et al. finds himself in a dilemma: high stress concentrations, large deflections, uneconomical and possibly an unsafe structure. Obviously to achieve a practical solution to this problem the concentration of high stresses must be relieved and deflections must be reduced at minimum cost. The test results of this study indicated that in order to solve the problem a new method of reinforcing the top chord of the opening was necessary.

It should be further noted that in order for the reinforcement method to be successfully effective, two objectives must be accomplished:

(1) Reduce the diagonal tensile stresses in the concrete.

(2) Reduce the bond stresses at the top chord of the opening.

These aims were met by special web reinforcements as described in the following section.

5.1.3 <u>The Effects of Special Web Reinforcements On Beams With Openings</u>

Beam A-3-2 had additional special web reinforcement as shown in Figure 5.7. The test results of this beam when compared with the results of the control beam A-1 indicate the following:

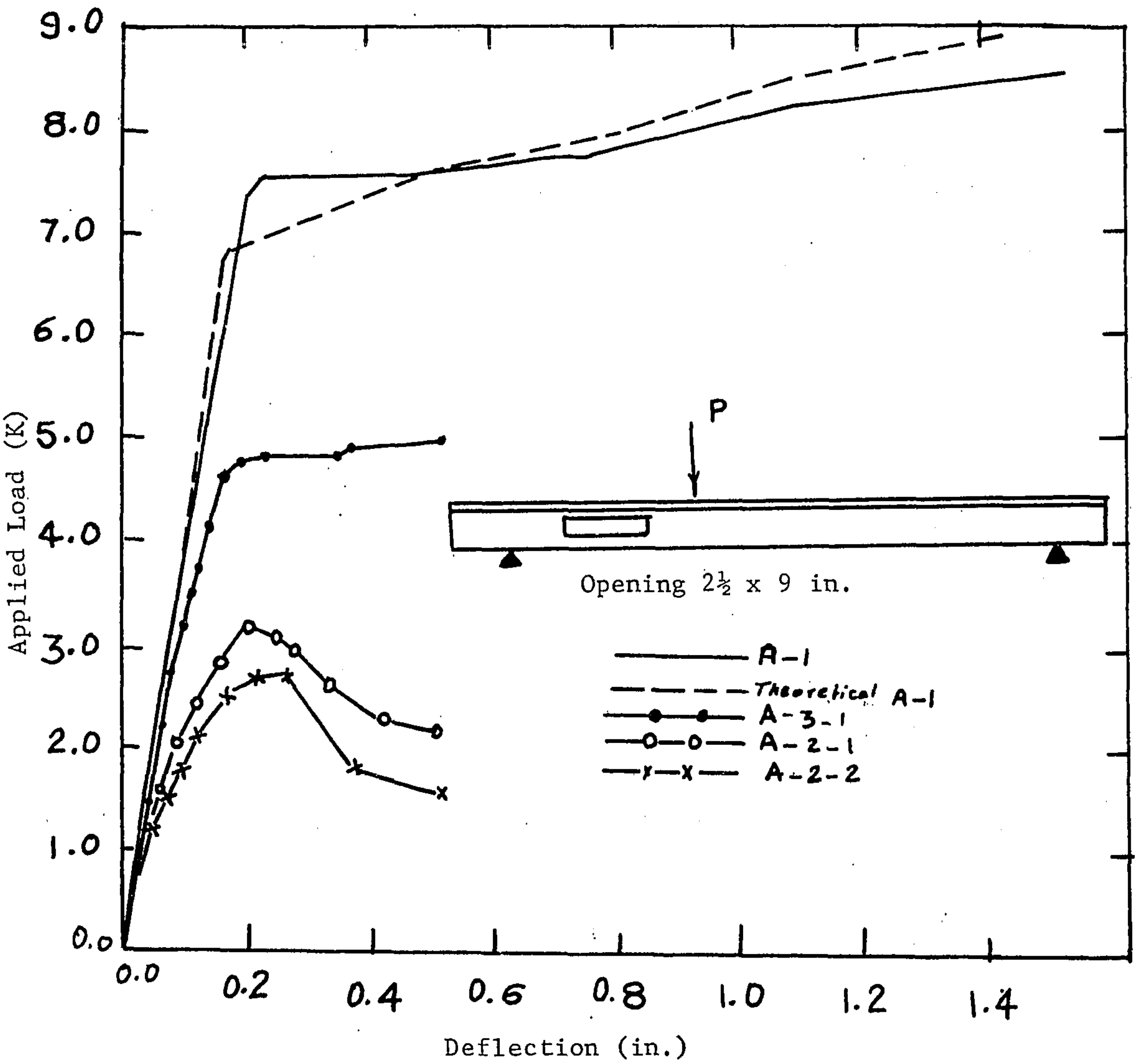


Figure 5.3 Applied Load Versus Vertical Deflection for Type A Beams.

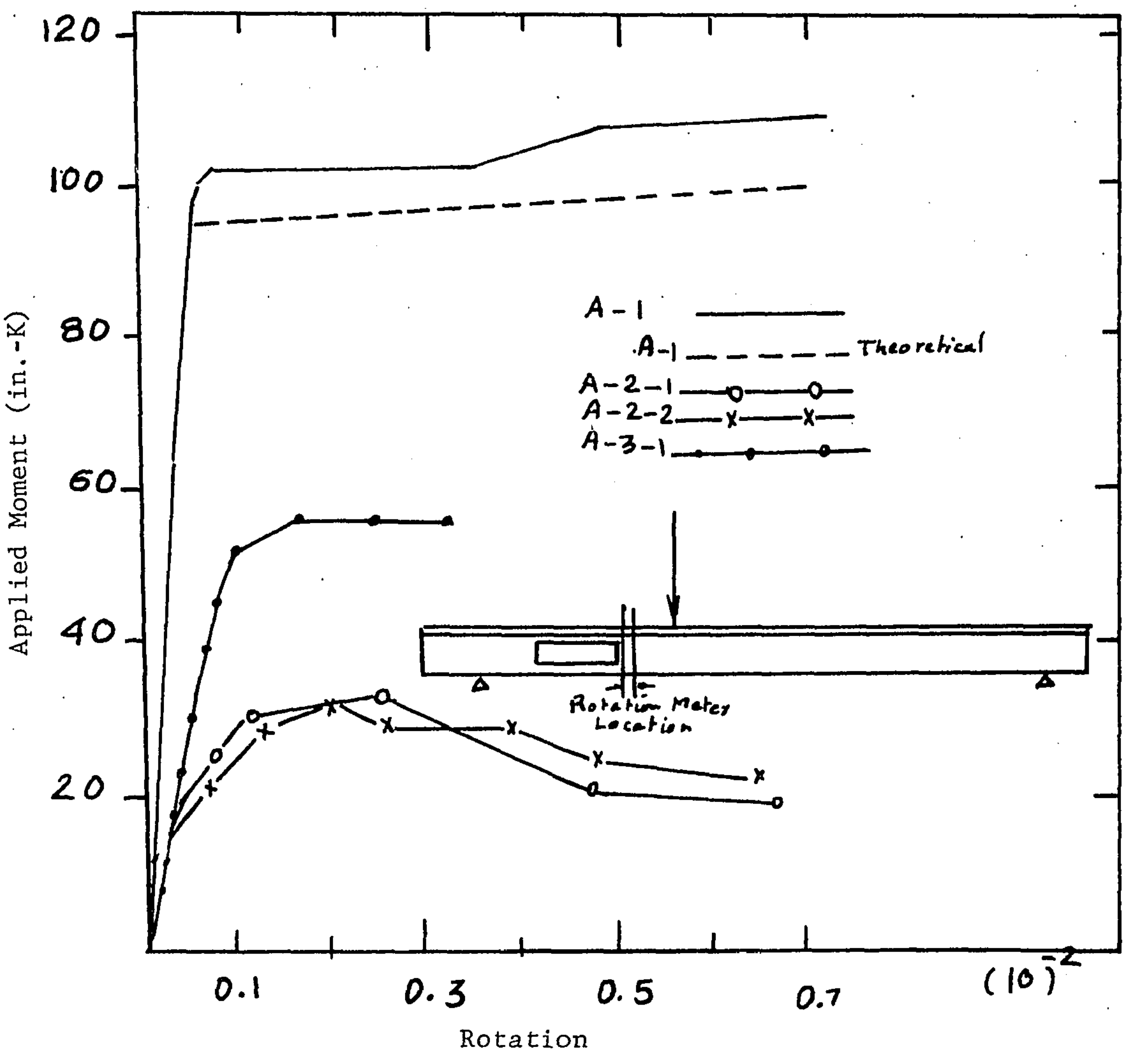


Figure 5.4 Applied Moment Versus

Rotation for Type A Beams.

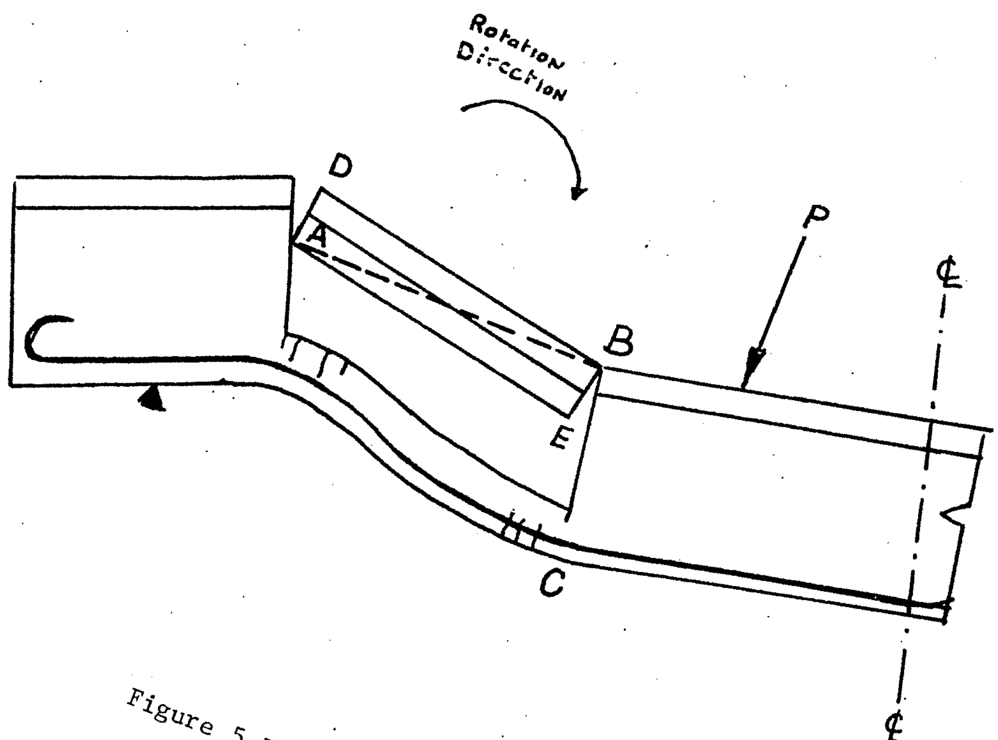


Figure 5.5 Failure Mechanism of Opening Without Reinforcement.

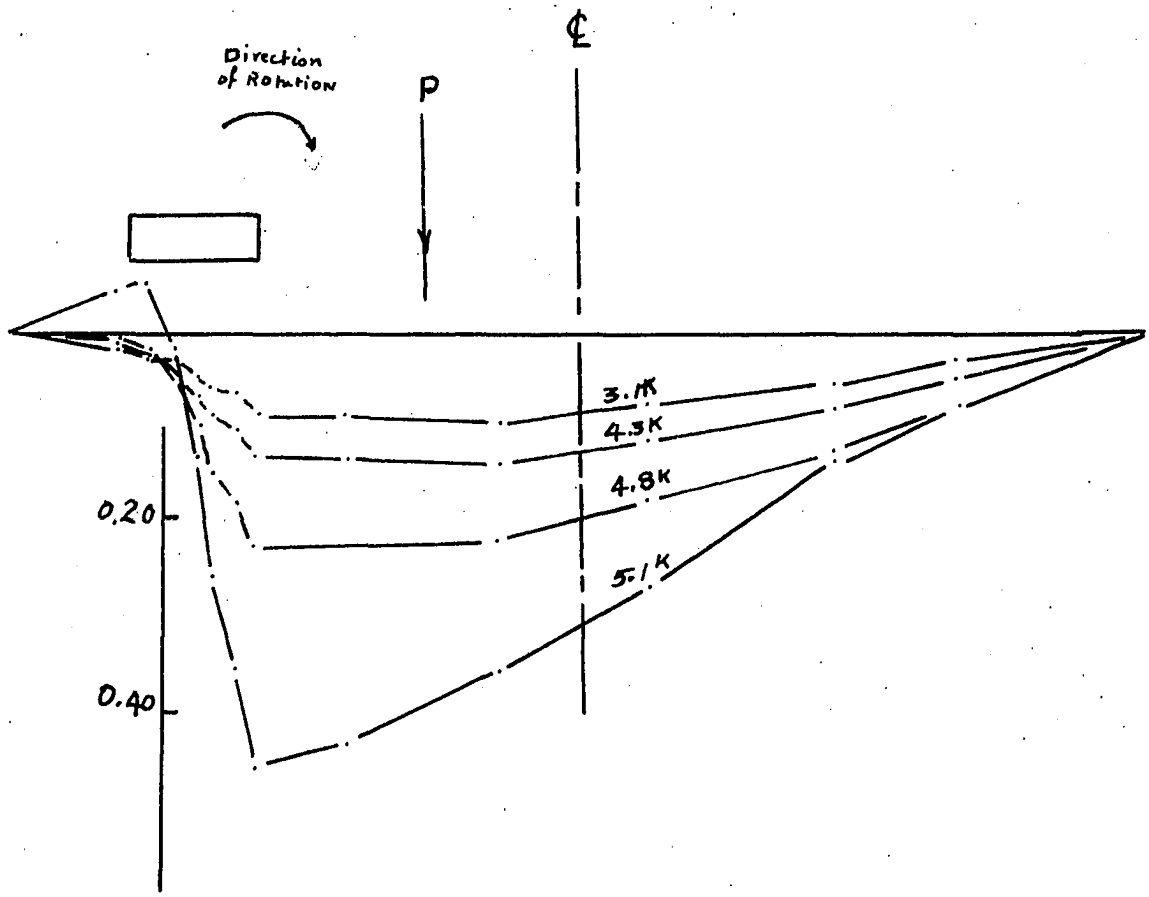
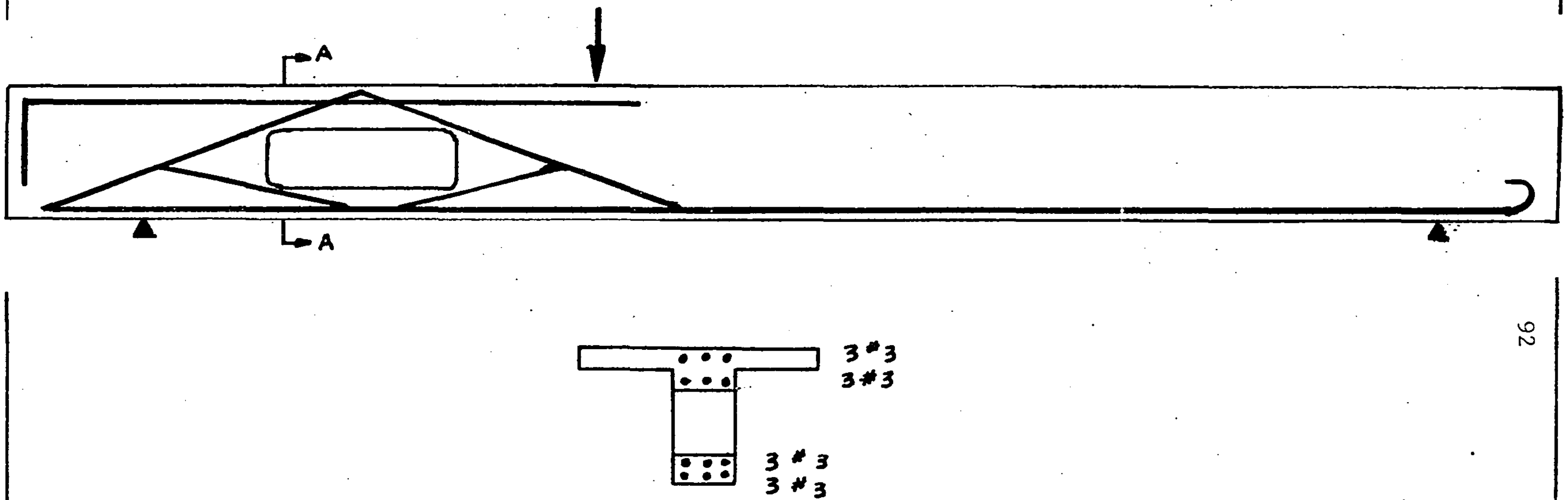


Figure 5.6 Span Deflection of Beam A-3-1.



Section A-A

Figure 5.7 Special Web Reinforcement of Beam A-3-2 Arc-Welded at Points of Intersection.

(1) The load-deflection responses were almost identical to that of the control beam to a point equal to about 65 percent of the yield load of the control beam. Increasing the load to a point equal to about 85 percent of the control beam caused an approximate increase of 20 percent in the deflection response curve of beam A-3-2. Further increase of load caused continuous gradual increase in the deflection response curve until failure occurred at about 95 percent of the yield load of beam A-1 (Figure 4.1).

(2) The moment-rotation response of beam A-3-2 and the control beam A-1 are shown superimposed in Figure 5.9. It can be easily seen that the rotations of the section containing the opening in beam A-3-2 decreased approximately 50 percent from those of the solid beam with respect to the applied moment to a value where M is equal to approximately 45 percent of the applied moment. Increasing the applied moment above 45 percent caused a continuous gradual increase in rotation until the failure occurred at an applied moment value equal to about 78 percent of the yield moment of the control beam A-1. This test was conducted with the rotation meter placed at the end of the opening nearer to the center of the beam.

(3) At approximately 95 percent of the yield load of the control beam A-1, bond failure occurred in both the top and bottom chords of beam A-3-2.

(4) The span deflection of both beams A-1 and A-3-2 were similar (Figure 5.10).

As indicated by test results it can be assumed that beam sections subjected to bending moment only in general behave similarly (with-

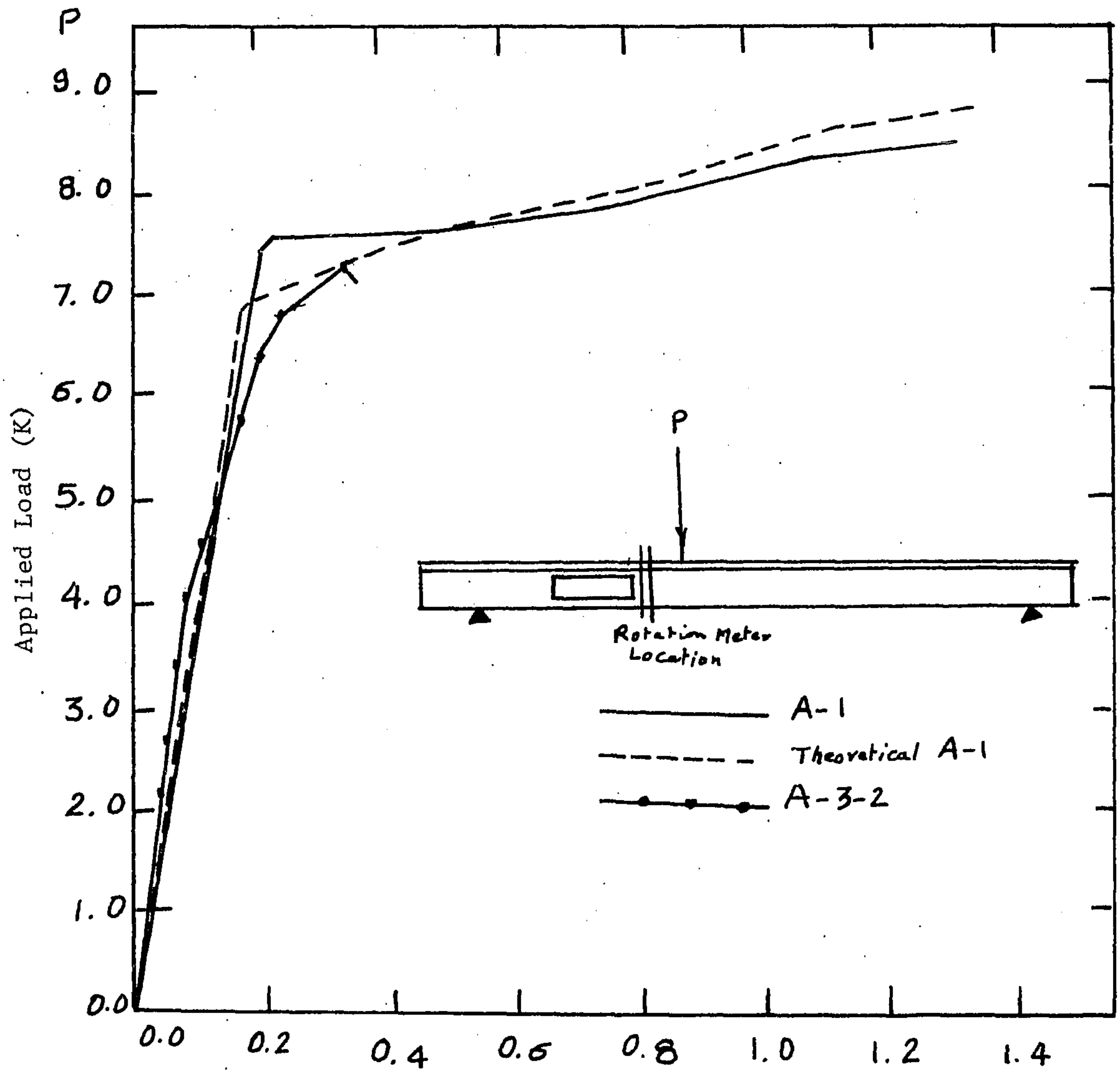


Figure 5.8 Applied Load Versus Vertical

Deflection for Beams A-1 and A-3-2,

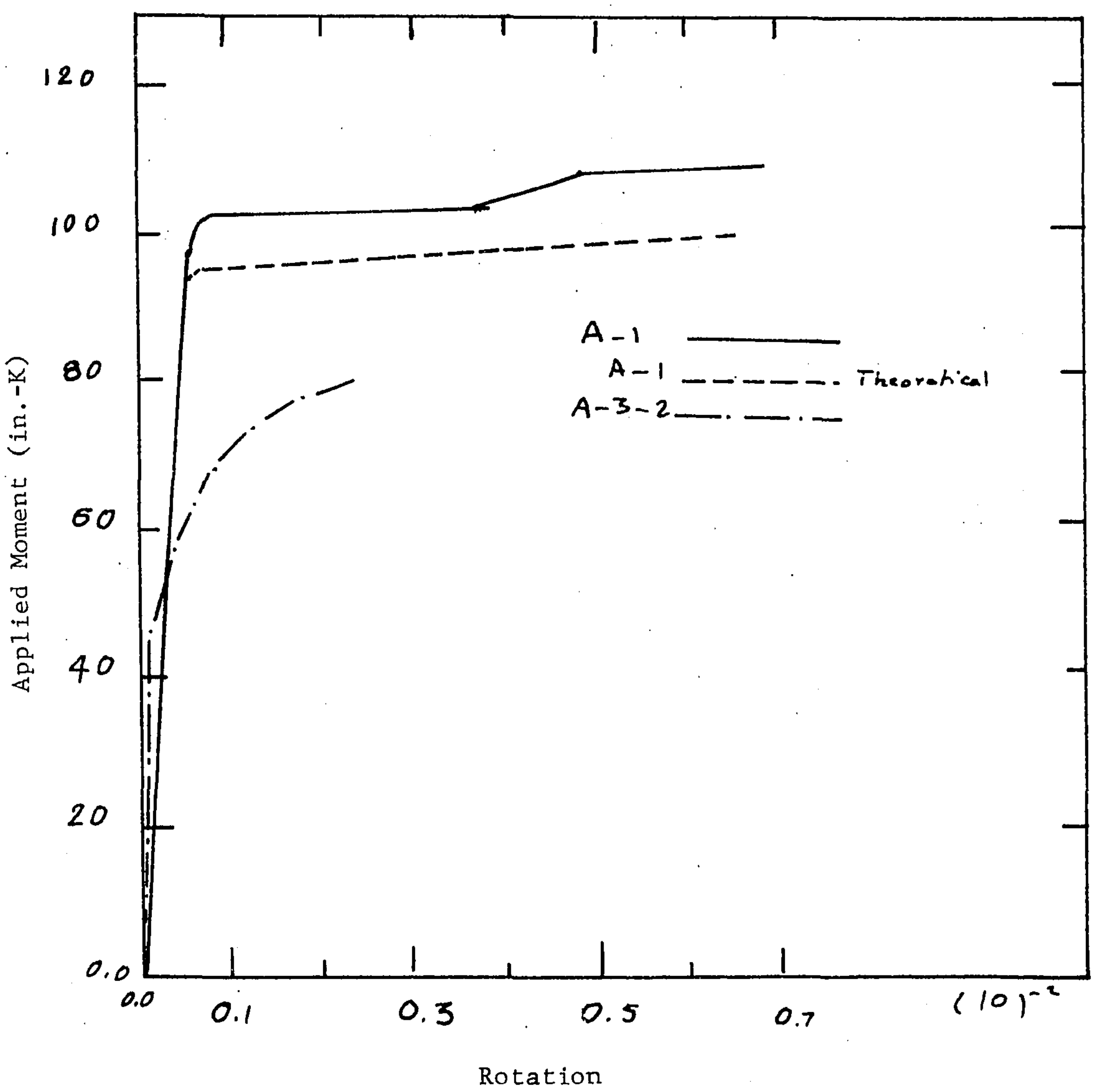


Figure 5.9 Applied Moment Versus Rotation

For Beams A-1 and A-3-2.

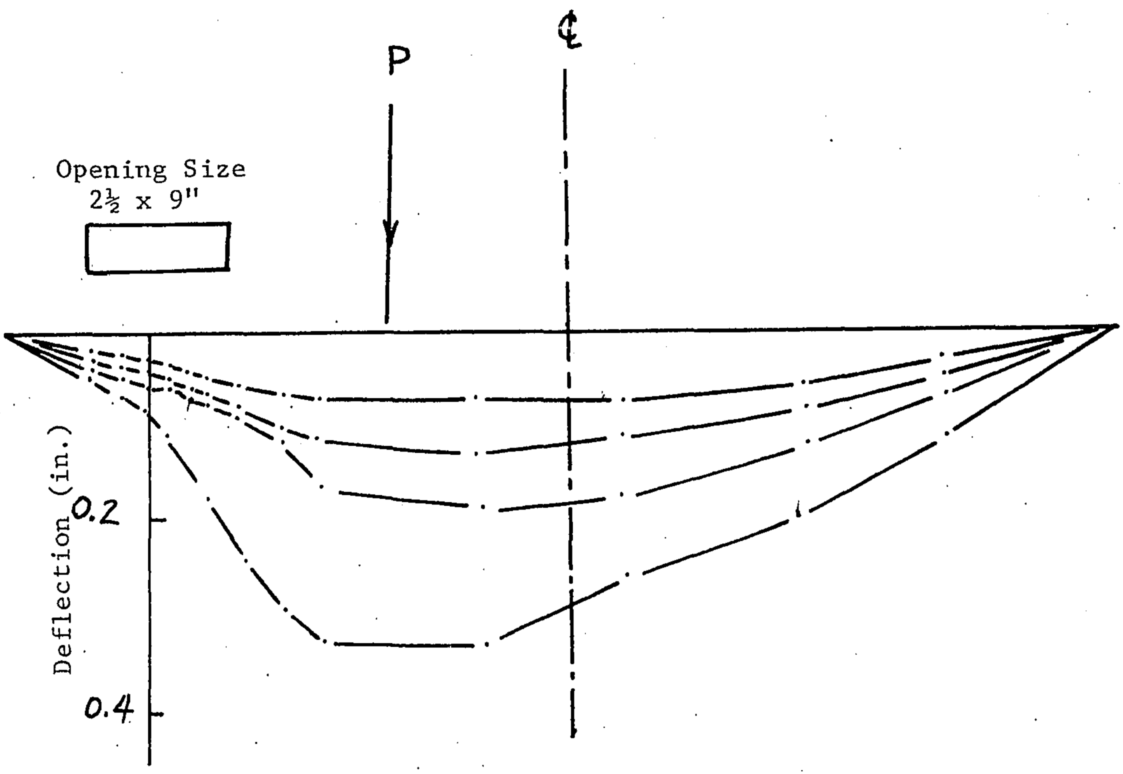


Figure 5.10 Span Deflections in Beam A-3-2.

in reasonable test scatter) with or without an opening. The sections of the beam subjected to combined bending moments and shearing forces behaved differently depending on the presence or absence of the opening. Therefore, it seems reasonable to assume that the major element effecting the behavior of beams containing openings is the presence of shearing forces. The presence of the shearing forces caused a significant increase in the rotation and deflection responses of beams containing openings. Since the relation between shearing force and rotation is obvious, a shear-rotation response can be obtained. Such curves are shown in Figure 5.11. It is important to note that the net area between the response curves of beams A-3-2 and A-2 represents the contibution of the web reinforcement in carrying the shearing force at any loading value. Furthermore, those curves also indicate the shearing force carried by web reinforcing bars. In the case of beam A-3-2 the maximum value of the shearing force carried by web reinforcement was equal to 2.7 Kips (Figure 5.11).

In order to further correlate the experimental results, it is indicated that the maximum shearing force carried by web reinforcing bars when calculated from the deflection responses curves shown in Figure 5.12 showed agreement with the values obtained previously.

In order to find the stresses in the web reinforcement, a section through the center of the opening was taken as shown in Figure 5.13, Section a-b. The calculated value of the shear force carried by the web reinforcement (2.7 Kips) was assumed to act in the direction and at the location shown in Figure 5.13. This section passes through the arc-welded intersection of the two bars in the top chord and thus

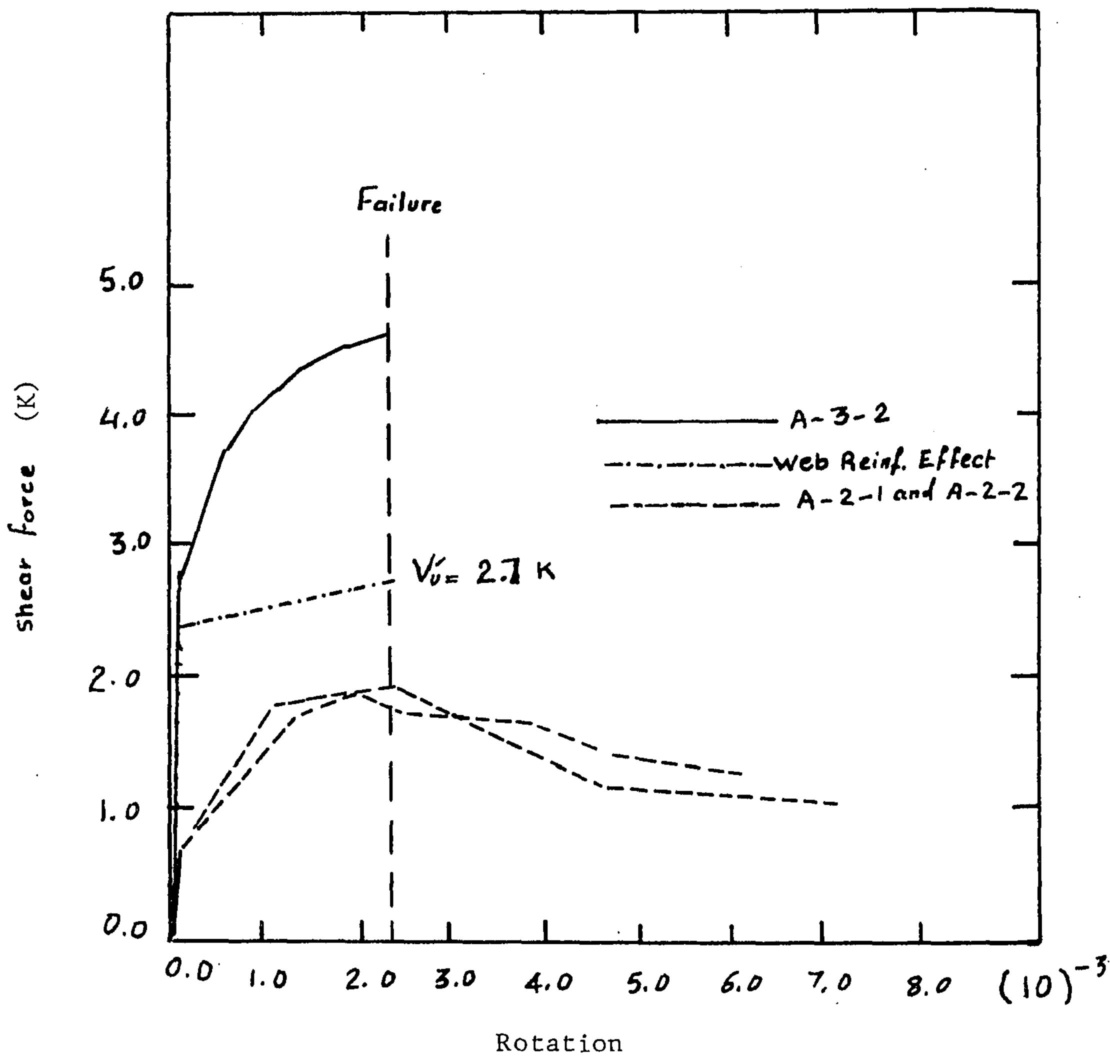


Figure 5.11 Effect of Special Web Reinforcement.

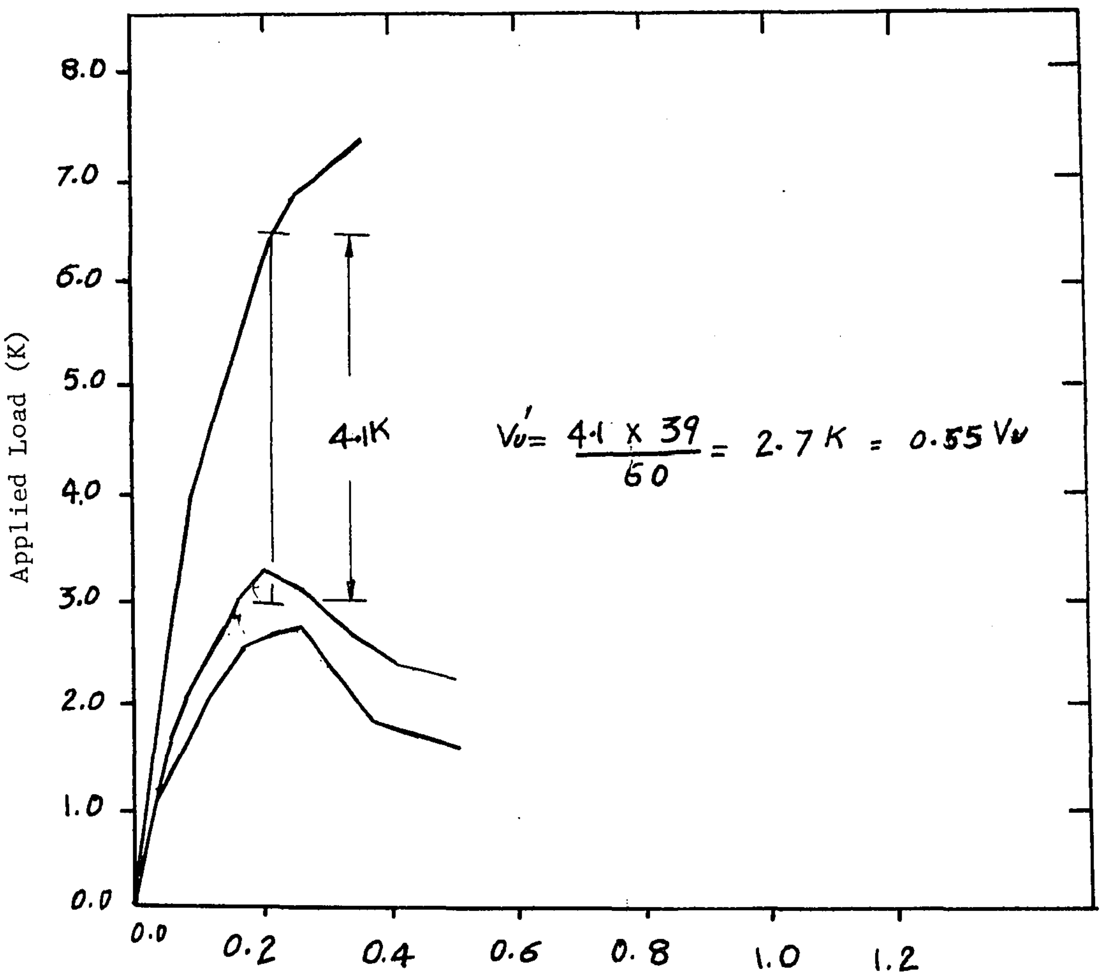


Figure 5.12 Ultimate Shear Carried by the Special Web Reinforcement for Opening Size 2½ x 9".

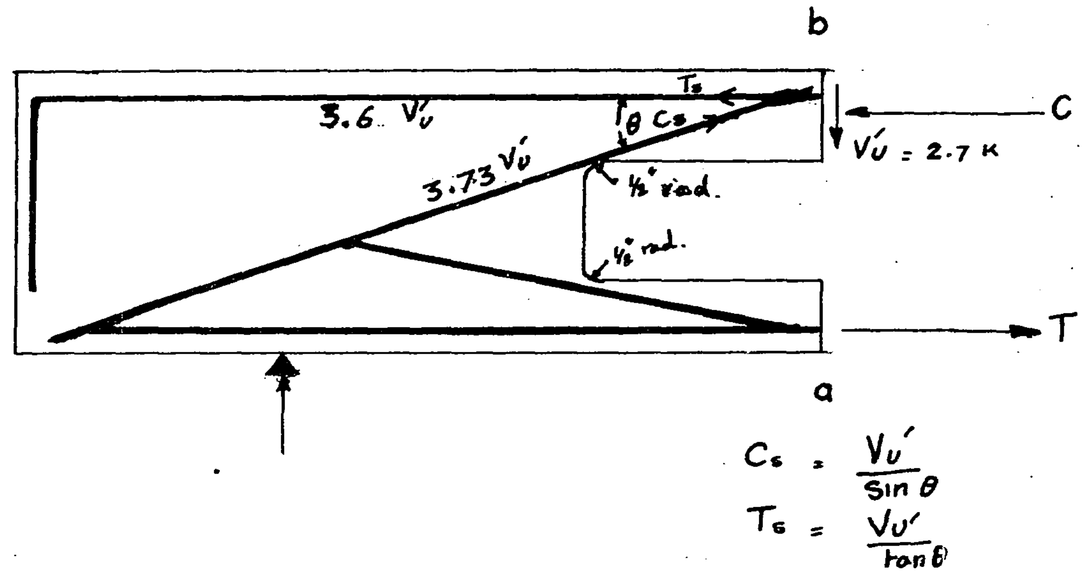


Figure 5.13 The Special Web Reinforcement Resisted the Stresses in a Manner Similar to Truss Action.

only minor dowel action could exist in the reinforcement at this point. Therefore, it seems reasonable to assume that the special web reinforcement resisted the stress in a manner similar to truss action. Accordingly the maximum stress in the steel was found to be 30,5 kips ($0.60f_y$).

The effect of the special web reinforcement can also be noticed from the load-deflection response curves of beam C-3-1 and C-3-2 each of which had an opening 2½ x 3 as shown in Figure 5.14. Beam C-3-2 had 1-# 3 bar of special web reinforcement and beam C-2-1 had no special web reinforcement. The deflection of beam C-3-1 increased by approximately 17 percent over that of the solid beam, and the deflection of beam C-3-2 decreased by approximately 30 percent from that of the solid beam. The maximum load difference for the same deflection was found to be 2.1 kips (Figure 2.1). This means that a shear force equal to 1.37 kips was carried by the web reinforcement. The maximum stress in the special web reinforcement was calculated as 19.4 ksi by using the method described above.

Based on the test results it can be stated that: (1) special web reinforcement reduces the rotation of the section containing the opening, and (2) consequently reduces the maximum beam deflections, and (3) increases the load carrying capacity of T-Beams with large openings in the webs.

5.1.4 The Effect of Size of Opening

Beams Without Special Web Reinforcement

The carrying capacity of beams with an opening size of 2½ x 9 (beams A-2-1 and A-2-2) had a yield of only 45 percent of an identical

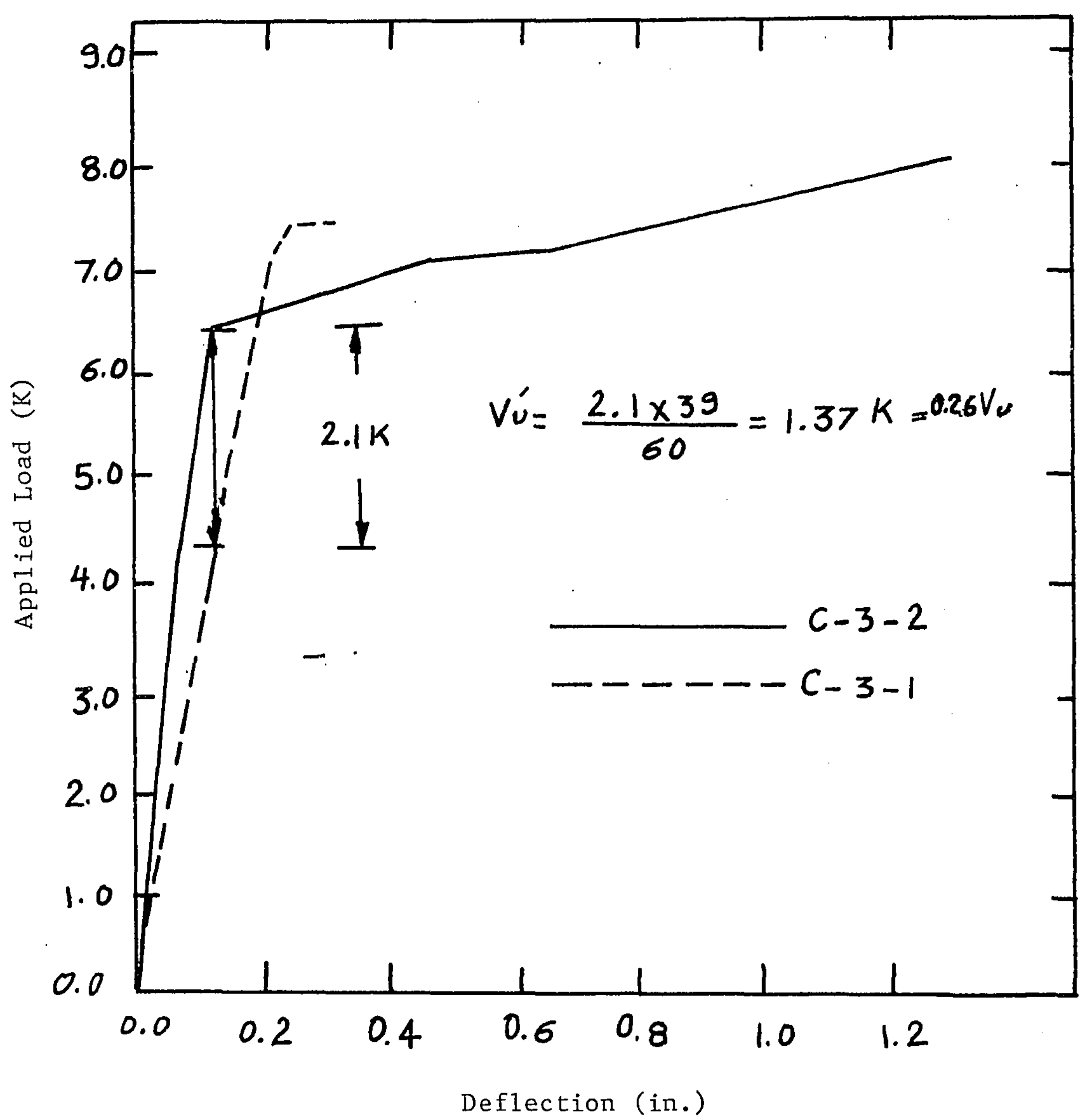


Figure 5.14 Shear Carried by Special Web

Reinforcement for 2½ x 3" Openings.

beam without opening, while a beam with an opening size 2½ x 3 (beam C-3-1) had a carrying capacity of approximately 98 percent of the yield load of the identical beam without opening. Therefore, the loading capacity increases with a decrease of the opening size.

The relation between the ultimate shear carried by the special web reinforcement (V_u') and the size of the openings are given in Table 5.1.

Beam	V_u'	Size of Opening Inches	Length of Opening/L α	
A-3-2	0.55 V_u	2½ x 9	0.15	(Figure 5.12)
C-2	0.43 V_u	2½ x 6	0.10	(Test results of beam C-2)
C-3-2	0.26 V_u	2½ x 3	0.05	(Figure 5.14)

Table 5-1. Summary of Shear Carried by the Web Reinforcement.

Since the relation between the ultimate shear carried by the special web reinforcement and the size of the openings is linear it (relation) can be expressed by the following equation

$$V_u' = V_u\ (0.18 + 2.5\ \alpha) \quad . \ . \ . \ . \ . \ . \ 5.1$$

Where

α = Length of the opening/Beam span length

V_u' = Ultimate shear carried by the special web reinforcement

V_u = Total ultimate shear

V = Total shear force

Beams With The Special Web Reinforcement

From test results of 8 beams with openings 2½ x 9, 2½ x 6 and 2½ x 3 inches it can be stated that beams with openings reinforced with special web reinforcement behave identically to beams without openings when the correct amounts of special web reinforcement is used around the openings.

5.1.5 The Effect of Multiple Openings in the Webs of Beams

Beam D-1 had two openings (sizes 2½ x 9 inches) located near the supports and was provided with special web reinforcement. Beam D-2 had three openings; two near the supports. The beams were reinforced with special web reinforcement. The behavior (load-deflection and moment-rotation responses) of these beams were similar to the control beam B-1. The only difference was that the yield load of beam D-2 was approximately 20 percent less than that of beam B-1 (Figures 5.15 and 5.16).

Therefore, beams with two or three openings behave similarly to beams without openings when the sections containing the openings are adequately reinforced with special web reinforcement. Obviously, in the limiting case of a very large number of finite sized openings, the behavior of the beams would be affected.

5.16 The Effect of Shear Span Ratio on Beams With Openings

Three values of shear ratios were investigated (3.78, 5.4 and 7.2) on beams with openings size 2½ x 9 inches. The sections containing the opening were reinforced with special web reinforcement. The amounts of reinforcement were proportional to the shearing force acting

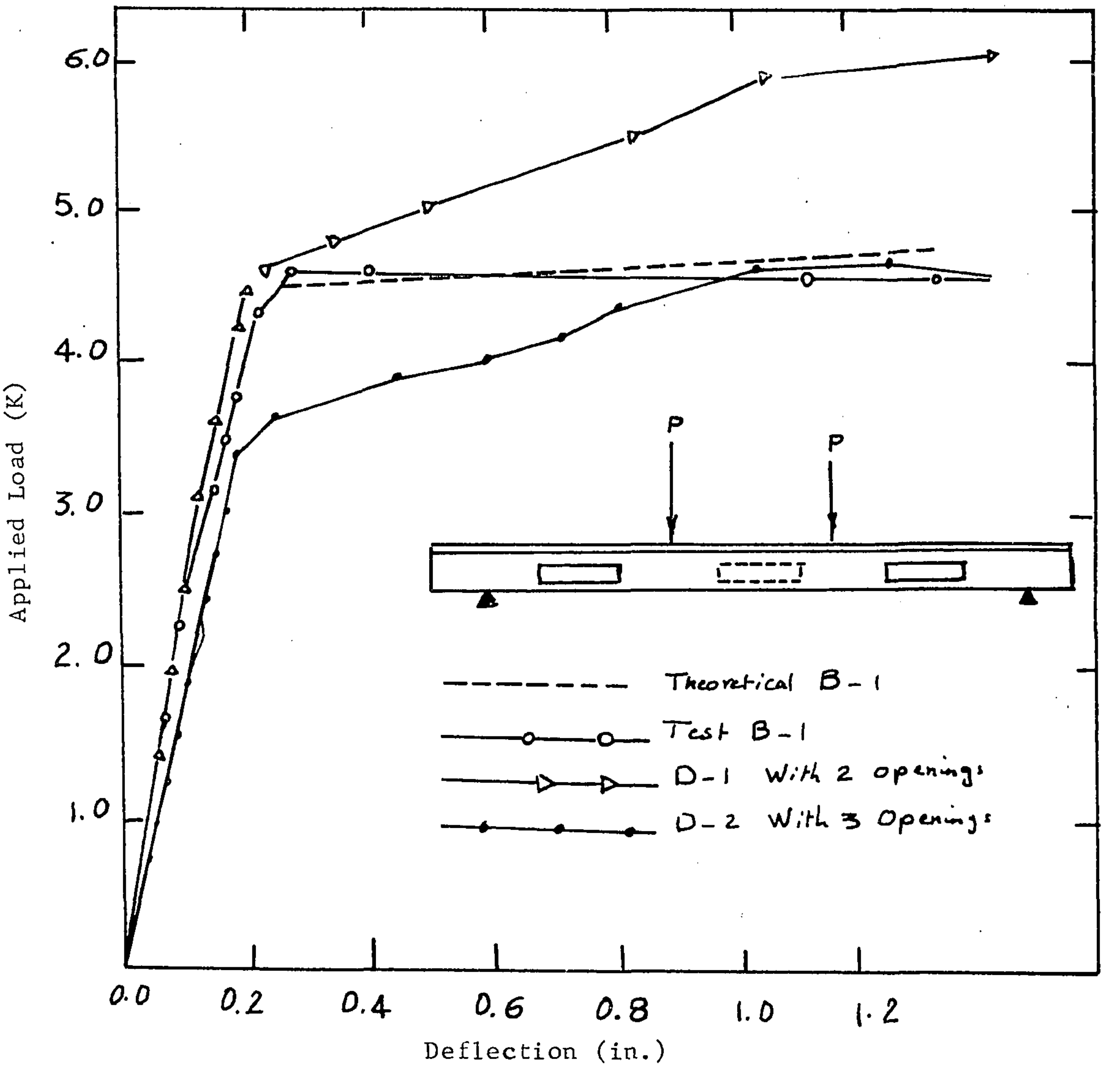


Figure 5.15 Applied Load Versus Vertical

Deflection for Type D Beams,

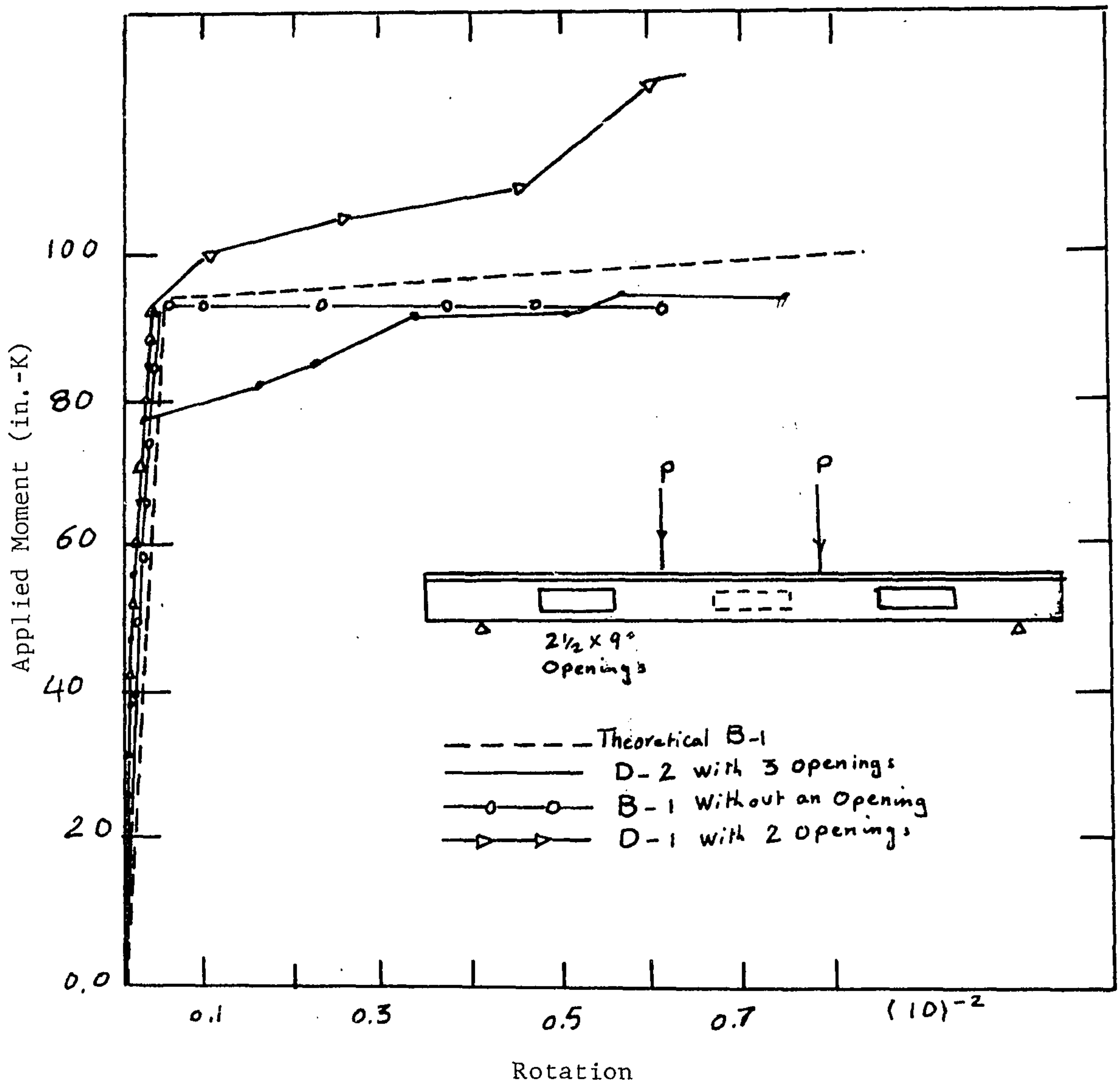


Figure 5.16 Applied Moment Versus Rotation

at Mid-span for Type D Beams.

at the center of the opening. All beams behaved similar to the control beam A-1. The only noticeable difference between the beams was the progression of cracks around the openings. To find the effect of the shear span ratio, similar cracking for all beams was compared with shear span ratio, shear force, bending moment and amount of web reinforcement as shown in Figures 5.21 and 5.22. The results of this comparison are shown in Figures 5.17 and 5.18, also a summary of the results are given in Table 5.1.

The investigation of the cracks and the corresponding shearing forces indicated certain definite trends. For example, the shear strength increased with the increase of the shear span ratio. When the shear span ratio increased from 3.78 to 5.5, the shear strength of the beam increased approximately 33 percent. However, the further increase of the shear span ratio did not increase the strength as shown in Figure 5.19.

			Moment (in-k)		Shear Force (K)	
Beam	Shear Span Ratio	Web Reinforcement	Bending Cracking Level 1 Fig. 5.17	Cracking Level 2 Fig. 5.18	Cracking Level 1 Fig. 5.17	Cracking Level 2 Fig. 5.18
A-3-2	3.78	3 # 3	13.9	27.8	1.32	2.64
E-2	5.4	2 # 3	27.6	60.0	1.84	4.0
E-1	7.0	1 # 3	25.4		2.42	
E-3	7.2	1 # 3	40.0	72.0	2.0	3.6

Table 5.2 Summary of the results of the effect of shear span ratio.

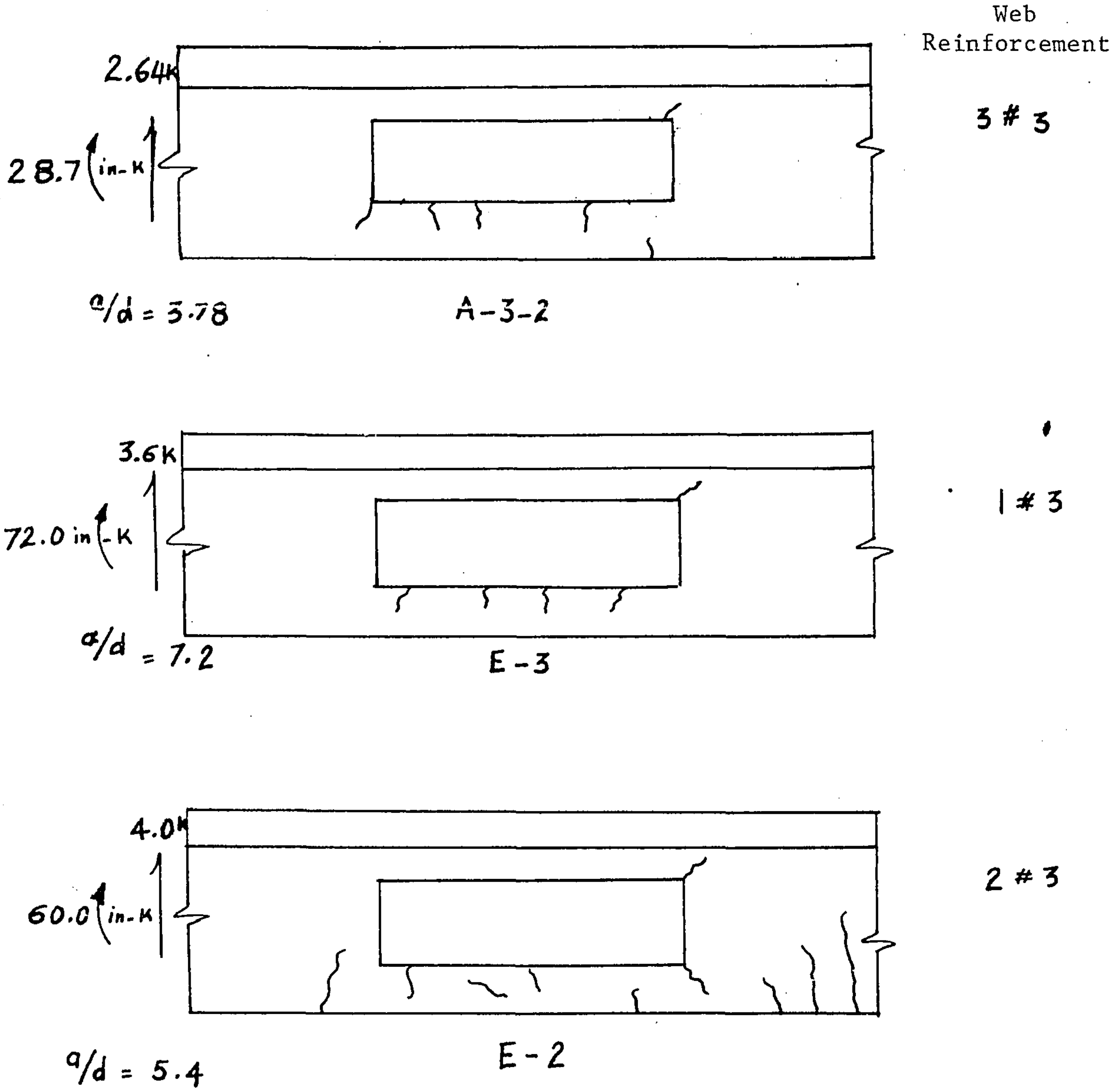


Figure 5.18 Cracking Level 2 Versus Bending Moment and Shear Force.

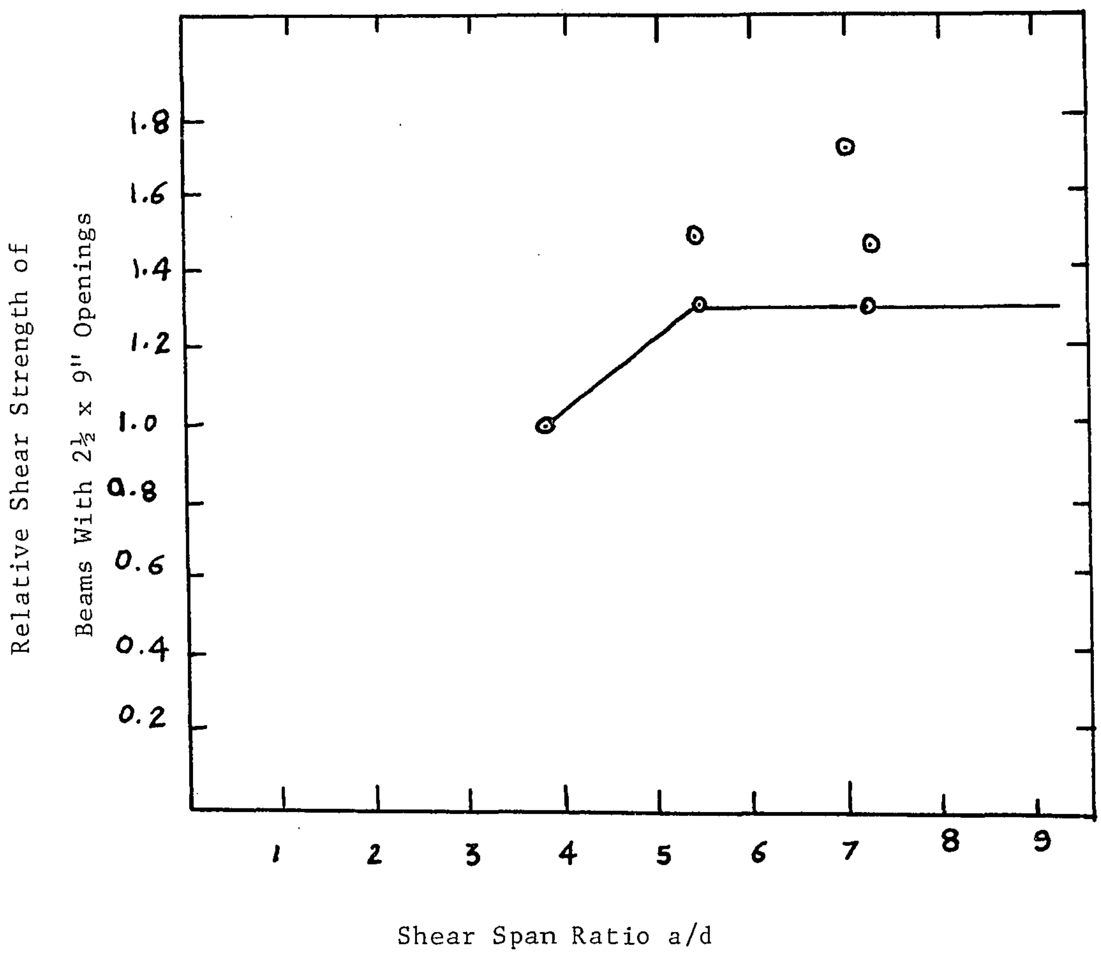


Figure 5.19 Beams Relative Shear Strength

Versus Shear Span Ratio a/d,

5.2 Proposed Design Criteria

No special web reinforcement is required for beams with a section containing an opening if this section will be subjected to bending moment only.

Special web reinforcement is required when the section containing an opening is subjected to bending moment and shearing force.

Design of the area of special web reinforcement required:

Let, α = Length of the opening/length of the beam span

B = Height of the opening/total depth of the beam

a/d = Shear span ratio

f = Top chord thickness/total depth of the beam

t = Total depth of beam

θ = Angle of inclination of the web reinforcement with the horizontal (Figure 5.15)

Q = Yield load/failure load = 0.85

Then,

$$A_v = \frac{1}{Q}\left[\frac{V_u}{\text{Sin }\theta} \frac{(0.18 + 2.5\,\alpha)}{\text{x } 0.60\, f_y}\right]\left[1 - \frac{a/d - 3.78}{5.16}\right] \qquad 5.1$$

Restrictions

(1) The beam is simply supported

(2) $a/d \geq 3.78$ and $\frac{a/d - 3.78}{5.16} \leq 0.33$

(3) $t/L = 0.1$

(4) Slab thickness $\geq 0.17t$

(5) $B \leq 0.45$

(6) $\alpha \leq 0.15$

(7) $f = 0.33$

This design formula is based on the following:

(1) Shear strength decreases as the length of the opening increases (i.e. = $V_u' = V_u(0.18 + 2.5\alpha)$ (Section 5.1.4)

(2) Shear strength increases 33 percent by the increase of shear span increases from 3.78 to 5.5

(3) The value of stress in the web reinforcement was reduced (i.e., $f_v = 0.60 f_y$) to avoid escessive strains

(4) The special web reinforcement resists the stresses in a manner similar to truss action (i.e., $c = \dfrac{V_u'}{\sin\theta}$ (Fig. 5.13) .

Note: Arc-welding is essential at points of intersection of all web reinforcement.

The area of special web reinforcement (A_v) for seven beams tested and A_v calculated from equation 5-2 are listed in Table 5-3.

Beam	V_u Calculated (K)	a/d	Sin θ	f_y Ksi	[illegible]	A_v Calc.	A_v test
A-3-2	4.4	3.78	0.267	51.0	0.85	0.35	0.33
C-2	4.4	3.78	0.320	51.0	0.85	0.22	0.22
C-3-2	4.4	3.78	0.640	51.0	0.85	0.08	0.11 *
E-1	2.4	7.0	0.267	51.0	0.85	0.127	0.11
E-2	2.35	5.4	0.267	51.0	0.85	0.125	0.11
E-3	4.4	7.2	0.267	51.0	0.85	0.23	0.22
D-1	4.4	3.78	0.267	51.0	0.85	0.35	0.33

Table 5-3 The observed value A_v(test), and the calculated value A_v(Calc.)

*The difference between the calculated value (by the use of the

Footnote continued.

empirical equation derived above) A_V (Cal.) and the experimentally observed value A_V(test) is due to the pressure of more reinforcing steel in beam C-3-2 than actually needed.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 General

As stated earlier the primary objectives of this study of large openings in the webs of reinforced concrete T-Beams were as follows:

(a) To determine the most effective type of reinforcement to be used around the openings.

(b) To determine the optimum amount of reinforcement with respect to the size of the openings.

(c) To determine some of the effects of loading type (i.e., one-point loading and two-point loading).

(d) To determine the effect of variations in the shear span (a/d).

(e) To study the effect of multiple openings.

(f) To develop an equation that could be used to conservatively design the steel area required for the special web reinforcement when large openings are present.

A total of fifteen reinforced microconcrete simple supported T-Beams having large openings were constructed and loaded monotonically to collapse with concentrated loadings. The load-deflection response, moment-rotation responses, load versus cracking and load-span deflections were compared to those of two beams without openings. The theoretical load-deflection responses and moment-rotation responses of these two beams without openings were also compared with the experimental results.

6.2 Conclusions

The following conclusions are justified by the test results:

1. Beams with an opening 0.42d x 1.5d or smaller without special web reinforcement and subject to bending moments only behave similar to beams without openings.
2. Beams with an opening without special web reinforcement and subject to shearing force and bending moment will fail due to diagonal tension by corner cracking of the opening at a low load level.
3. Beams with openings subject to shearing forces and bending moments, reinforced by a type of reinforcement based on progression of cracking (This type of reinforcement was used by Lorentsen (5) and Nasser, et al. (7)) showed high rotation and deflection with respect to the loading and concentration of stresses at the corners of the openings. Since beams reinforced with this type of reinforcement will be extremely highly stressed, the reinforcement requirements render these members impractical and uneconomical. Thus the previous theoretical studies concerned with the design of this reinforcement with these methods are of no practical value.
4. Beams with their openings reinforced by the method presented in this study (This type of web reinforcement was built specially to restrain rotation of the section containing

the opening and to resist shearing forces) behaved similar (load-deflection and moment rotation responses) to beams without opening.

5. The strength of beams with openings and without reinforcement is greatly reduced by the increase of the length of openings.

6. The amount of special web reinforcement necessary to compensate for the shear strength lost is a function of the length of opening.

7. The shear strength of beams with openings increased by approximately 33 percent as shear span ratio increased from 3.78 to 5.5. A further increase in the shear span ratio did not increase the shearing strength.

8. Beams with multiple openings (maximum of three) reinforced with special web reinforcement behaved in a manner similar to the beams without openings.

9. An empirical formula based on the results of this study for the calculation of reinforcement requirements near large openings in the web of T-Beams is given below:

$$A_v = \frac{1}{Q}\left[\frac{V\upsilon}{\text{Sin}\theta} \frac{(0.18 + 2.5\alpha)}{0.60f_y}\right]\left[1 - \frac{a/d - 3.78}{5.16}\right]$$

Where,

A_v = Area of special web reinforcement

V = Total shear force

θ = Angle between the top inclined web reinforcing bars and the horizontal

α = Length of opening/length of beam span

a/d = Shear span ratio

Q = Yield load/failure load = 0.85

Refer to Section 5.2 for special restrictions which must be observed when using the equation.

6.3 Recommendations

Web reinforcement suggested by Nasser, et al. (7) and Lorentsen (5) did not relieve the high concentration of stresses in the beams, especially at the corners of the openings. The type of web reinforcement proposed and used in this study proved to be adequate.

The suggested design empirical formula presented herein can be used for practical purposes provided that none of the restrictions are violated.

6.4 Suggestions For Future Research

More tests are needed to extend the suggested design empirical formula to include all variables that may influence the strength of the beam. For example, such studies could include the effects of (a) further variations in geometric parameters, (b) the time dependent parameters, and (c) load history.

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APPENDIX A

FORTRAN COMPUTER PROGRAM

LDDFN

```
      DIMENSION ACONC%176<,HST%176<
      DIMENSION AST%176<,FCONC%176<
      DIMENSION FST%176<,BEND%300<
      DIMENSION PHE%300<,ESTI%10<
      DIMENSION TRIM%82<,ROT%82<
      DIMENSION TY%82<,FSBI%10<
      DIMENSION ESBI%10<,FSTI%10<                                        LDF40080
      COMMON NSI,EP3,PHI,EPSIO,FPPC,FCONC,FST,ACONC,PCALC,AST,HST,FPC
      COMMON EPSMAX,FSBI,FSTI,ESBI,ESTI
C     PROGRAM LDDFN                                                      LDF70010
  999 FORMAT %1H1<                                                       LDF70060
    1 FORMAT % / 30H      TABLE 1.  CONTROL DATA       //
     1 40H           NUM SECTION INCS          #     I5, /
     2 40H           MAX ALLOWABLE COMP STRAIN #      1E10.3, /
     3 48H           DELTA PHI                        VARIES, /
     5 40H           SPAN                      #      1E10.3, /
     4 40H           DEPTH                     #      1E10.3, /
     5 40H           WIDTH                     #      1E10.3, /
     4 40H           A                         #      1E10.3, /
     4 40H           B                         #      1E10.3, /
     5 22H           BEAM NUMBER ,I5,/<
    2 FORMAT%//  42H      TABLE 2.  PROPERTIES OF THE MATERIALS   //
     1 35H           CONC CYLINDER STRENGTH  #      1E10.3, /
     2 35H           K FOR FPPC # K * FPC    #      1E10.3, /
     3 35H           STEEL YIELD POINT %BOT< #      1E10.3, /
     4 35H           STEEL YIELD POINT %TOP< #      1E10.3, /
     5 35H           STEEL MOD OF ELASTICITY #      1E10.3, /  <
 1441 FORMAT%//17X,5H SPAN,7X,5H XINC,6X,5H    P ,10X,2HWU,9X,3H A ,9X,3H
     1 B ,8X,4HM/MY/<
 1443 FORMAT%/ 4X,8H ROT%41<,6X,4HTRH1,7X,5H TRHM,6X,9H TRIM%41<,4X,6H T
     1Y%9<,6X,7H TY%17<,5X,7H TY%22<,5X,7H TY%33<,5X,7H TY%41<  //<
 1921 FORMAT%/ 4X,7H TY%49<,5X,7H TY%57<,5X,7H TY%65<,5X,7H TY%75<,5X,7H
     1TY%81<  //<
     13<
    9 FORMAT %2I5,5F10.0,2F5.0<
```

```
   11 FORMAT %7F10.0<
   12 FORMAT %1X,E9.3,6E10.3<                                          LDF70170
   14 FORMAT %4E10.3<                                                  LDF70180
 4010 READ %1,9<NSTB,NSI,SH,SPAN,FPC,EPSMAX,WIDE,A,B
      IF %NSTB<5,99,5
    5 READ %1,14<%FSBI%I<,ESBI%I<,FSTI%I<,ESTI%I<,I#1,10<
      READ %1,11<%AST%I<,I#1,NSI<
      READ%1,11<%ACONC%I<,I#1,NSI<
      ZNSI#NSI                                                         LDF70460
      H#SH*ZNSI
      ZK3#.85
      DELPHI#.000025
      ES#FSBI%2</ESBI%2<
      WRITE %3,1<NSI,EPSMAX,SPAN,H,WIDE,A,B,NSTB
      WRITE %3,2<FPC,ZK3,FSBI%4<,FSTI%4<,ES
      WRITE %3,31<                                                     LDF70340
   31 FORMAT %//23H  PLACEMENT OF CONCRETE//<                          LDF70350
      WRITE %3,12<%ACONC%I<,I#1,NSI<                                   LDF70360
      HST%1<#SH/2.                                                     LDF70370
      DO 4005 I#2,NSI                                                  LDF70380
      J#I-1                                                            LDF70390
 4005 HST%I<#HST%J<&SH                                                 LDF70400
      WRITE %3,999<
      WRITE %3,32<                                                     LDF70410
   32 FORMAT %///20H  PLACEMENT OF STEEL//<                            LDF70420
      WRITE %3,12<%AST%I<,I#1,NSI<                                     LDF70430
      WRITE%3,19<
   19 FORMAT %//27H  STEEL STRESS-STRAIN CURVE//<
      WRITE %3,14<%FSBI%I<,ESBI%I<,FSTI%I<,ESTI%I<,I#1,10<             LDF70440
      FPPC#ZK3*FPC                                                     LDF70470
      EC#60000.0*ABS%FPC<**0.5                                         LDF70480
      EPSIO#2.0*FPPC/EC                                                LDF70490
      EPSCON#EPSMAX*.8
      PHI#0.0                                                          LDF70520
      HCL#H/2.0                                                        LDF70760
```

```
      L#NSI/2
      NN#0
      HSTL#0.0
      DO 301 I#L,NSI
      IF %AST%I<< 302,301,302
  302 HSTL#HSTL&HST%I<
      NN#NN&1
  301 CONTINUE
      DIV#NN
      DEPTH#HSTL/DIV
      STLHT#%H-HSTL/DIV</H
      M#0                                                               LDF70510
      WRITE %3,999<
      WRITE %3,33<                                                      LDF70550
   33 FORMAT %12X,5H ZMOM,15X,5H  PHI,15X,5H EPST,15X,5H EPSB<          LDF70560
  104 EP1#EPSMAX-DELPHI                                                 LDF70580
      K#0                                                               LDF70590
      EP2#0.00150                                                       LDF70600
  107 EP3#%EP1&EP2</2.0                                                 LDF70610
      K#K&1                                                             LDF70620
      CALL AXLD
      IF %ABS%PCALC<-5.0<108,108,109                                    LDF70650
  109 IF %K-99<110,110,111
  110 IF %PCALC<113,108,112                                             LDF70670
  112 EP2#EP3                                                           LDF70680
      GO TO 107                                                         LDF70690
  113 EP1#EP3                                                           LDF70700
      GO TO 107                                                         LDF70710
  111 WRITE %3,114<PCALC                                                LDF70720
  114 FORMAT %5X,30HPCALC DID NOT CONVERGE ON P    ,1E10.3<             LDF70730
  108 EPST#EP3                                                          LDF70740
      EPSB#EP3&H*PHI                                                    LDF70750
      ZMOM#0.0                                                          LDF70770
      DO 115 J#1,NSI                                                    LDF70790
  115 ZMOM#ZMOM&%HST%J<-HCL<*%FCONC%J<&FST%J<<
```

```
      WRITE %3,24<ZMOM,PHI,EPST,EPSM                                LDF70340
   24 FORMAT %1X,E20.5,F20.4,E20.5,F  .5<
      M#M&1                                                         LDF70360
  122 BEND%M<#ZMOM                                                  LDF70370
  123 PHI%M<#PHI
      EYLD#EPSB-%EPSB-EPST<*STLHT
      IF%EYLD-ESBI%4<<170,170,169
  170 YLDMOM#ZMOM
  169 CONTINUE
      IF %M-299< 1002,1103,1103
 1002 IF %EPST-EPSMAX<1103,118,118
  118 IF %EYLD*1.1-ESBI%4<<8868,8878,8878
 8868 PHI#PHI&DELPHI
      GO TO 104
 8878 IF %EYLD-ESBI%4<<8858,8848,8848
 8858 DELPHI#.000001
      PHI#PHI&DELPHI
      GO TO 104
 8848 IF %EPST-EPSCON<8828,8838,8838
 8838 DELPHI#.0015/DEPTH
      PHI#PHI&DELPHI
      GO TO 104
 8828 DELPHI#.00075/DEPTH
      PHI#PHI&DELPHI
      GO TO 104
 1103 WRITE %3,1003<M                                               LDF71010
 1003 FORMAT %1X,28H NUMBER OF POINTS ON CURVE #1I4/<               LDF71020
      BEND%1<#0.0
      BENDCK#0.0
      DO 444 I#2,M
      IF %BEND%I<-BENDCK<444,443,443
  443 BENDCK#BEND%I<
      KKK#I
  444 CONTINUE
      WRITE%3,445<BENDCK,KKK,YLDMOM
```

```
 445 FORMAT %13X,23HMAX RESISTING MOMENT # ,F12.5,14H  AT POINT NO.,I4,
    111H  ON CURVE.//13X,15HYIELD MOMENT # ,E12.5<
     WRITE %3,999<
     WRITE%3,497<
 497 FORMAT %12X,5H ZMOM,15X,5HRATIO,15X,4H PHI<
     DO 499 I#1,M
     RATIO#BEND%I</YLDMOM
     WRITE %3,24<BEND%I<,RATIO,PHE%I<
 499 CONTINUE
     WRITE %3,999<
     MM#M&1                                                             LDF71100
     DO 333 I#MM,300
  41 PHE%I<#0.0                                                         LDF70030
 333 BEND%I<#0.0                                                        LDF71120
     J#80
     M#J&1
     AM1#J
     XINC#SPAN/AM1                                                      LDF70310
     P#%ZMOM/SPAN</2.0
     W#ZMOM/%SPAN*SPAN<
     IF %A< 500,501,500
 501 P#0.0
     GO TO 502
 500 W#0.0
 502 W1#W/25.0
     W2#W/100.0
     ZMOM2#ZMOM*.85
     PC1#P/5.0
     PC2#P/50.0
     WRITE %3,1441<                                                     LDF71180
     WRITE %3,1443<                                                     LDF71210
 550 CALL CMOM%P,SPAN,TRIM,XINC,A,B,W<
3064 DO 3080 K#1,M                                                      LDF71290
     IF %TRIM%K<<3065,3066,3067                                         LDF71300
3065 ZZ#-1.0                                                            LDF71310
```

```
      TRM#-1.0*TRIM%K<                                                  LDF71320
      GO TO 7067                                                        LDF71330
 3066 ROT%K<#0.0                                                        LDF71340
      GO TO 3080                                                        LDF71350
 3067 ZZ#1.0                                                            LDF71360
      TRM#TRIM%K<                                                       LDF71370
 7067 DO 3068 L#1,MM
      IF %TRM-BEND%L<<3070,3069,3068                                    LDF71390
 3068 CONTINUE                                                          LDF71400
      GO TO 6000                                                        LDF71410
 6000 WRITE %3,6001<J,K                                                 LDF71420
 6001 FORMAT %1X,11H  FAILURE  ,2I10//<                                 LDF71430
      WRITE %3,999<                                                     LDF70050
      GO TO 4010
 3069 ROT%K<#PHE%L<                                                     LDF71450
      IF %ZZ<7075,7076,3080                                             LDF71460
 7075 ROT%K<#-1.0*ROT%K<                                                LDF71470
      GO TO 3080                                                        LDF71480
 7076 GO TO 4010                                                        LDF71490
 3070 YA#TRM                                                            LDF71500
      YO#BEND%L-1<                                                      LDF71510
      Y1#BEND%L<                                                        LDF71520
      XO#PHE%L-1<                                                       LDF71540
      X1#PHE%L<                                                         LDF71550
    7 X#X1&%%YA-Y1</%Y1-YO<<*%X1-XO<
      IF %ZZ<3075,3076,3077                                             LDF71580
 3075 ROT%K<#-1.0*X                                                     LDF71590
      GO TO 3080                                                        LDF71600
 3076 GO TO 4010                                                        LDF71610
 3077 ROT%K<#X                                                          LDF71620
 3080 CONTINUE                                                          LDF71630
 3082 CALL  SHAPE%ROT,XINC,TRH1,TRHM,TY,M<                              LDF71640
      RATUO#TRIM%41</YLDMOM
      WRITE %3,1442<SPAN,XINC,P,W,A,B,RATUO
 1442 FORMAT %13X,8E12.4<
```

```
      WRITE%3,1444<ROT%41<,TRH1,TRHM,TRIM%41< ,TY%9<,TY%17<,TY%22< ,TY%3
     13<
      WRITE%3,1444< TY%41<,TY%49<,TY%57<,TY%65<,TY%73<,TY%81<
     1,TY%81<
 1444 FORMAT %1X,9E12.4<
  560 IF %TRIM%41<-ZMOM2<571,572,572
  571 P#P&PC1                                                          LDF71720
      W#W&W1
      GO TO 550                                                        LDF71730
  572 P#P&PC2                                                          LDF71740
      W#W&W2
      GO TO 550                                                        LDF71750
   99 STOP
      END                                                              LDF71780
      SUBROUTINE AXLD
      DIMENSION ACONC%176<,HST%176<
      DIMENSION AST%176<,FCONC%176<
      DIMENSION FST%176<,FSBI%10<
      DIMENSION ESBI%10<,FSTI%10<                                         40040
      DIMENSION ESTI%10<                                                  40050
      COMMON NSI,EP3,PHI,EPSIO,FPPC,FCONC,FST,ACONC,PCALC,AST,HST,FPC
      COMMON EPSMAX,FSBI,FSTI,ESBI,ESTI
      PCALC#0.0                                                           70020
      DO 100 J#1,NSI                                                      70030
      EPS#EP3&HST%J<*PHI
      IF %EPS<10,10,20                                                    70050
   10 IF %ABS%EPS<&EPSIO<11,12,12                                         70060
   11 FC#FPPC*%2.0*%EPS/EPSIO<-ABS%EPS/EPSIO<**2.0<                       70070
      GO TO 50                                                            70080
   12 IF %ABS%EPS<&EPSMAX<14,14,42                                        70090
   14 FC#FPPC-%%EPSIO-EPS</%EPSIO-EPSMAX<<*%0.15*FPPC<                    70100
      GO TO 50                                                            70110
   20 FPPT#-0.08*FPC                                                      70120
      EPT#&0.0001                                                         70130
      IF %EPS-EPT<30,30,40                                                70140
```

```
 30 FC#2.0*FPPT*%%EPS/EPT</%1.0&%EPS/EPT<**2.0<<                       70150
    GO TO 50                                                           70160
 40 IF %EPS-0.0004<41,41,42                                            70170
 41 FC#FPPT-%%EPS-EPT</%0.0003<<*%-0.03*FPC<                           70180
    GO TO 50                                                           70190
 42 FC#0.0                                                             70200
 50 FCONC%J<#ACONC%J<*FC                                               70210
100 PCALC#PCALC&FCONC%J<                                               70230
    DO 99 J#1,NSI
300 EPS#EP3&HST%J<*PHI                                                 70250
    IF %AST%J<*EPS<102,150,202
102 DO 110 I#1,10                                                      70310
    IF %EPS-ESTI%I<<110,106,107
110 CONTINUE                                                           70340
106 FS#FSTI%I<                                                         70350
    GO TO 150                                                          70360
107 FS#FSTI%I-1<&%EPS-ESTI%I-1<<*%FSTI%I<-FSTI%I-1<</%ESTI%I<-ESTI%I-1 70370
   1<<                                                                 70380
    GO TO 150                                                          70390
202 DO 210 I#1,10                                                      70480
    IF %EPS-ESBI%I<<207,206,210
210 CONTINUE                                                           70510
206 FS#FSBI%I<                                                         70520
    GO TO 150                                                          70530
207 FS#FSBI%I-1<&%EPS-ESBI%I-1<<*%FSBI%I<-FSBI%I-1<</%ESBI%I<-ESBI%I-1 70540
   1<<                                                                 70550
150 FST%J<#FS*AST%J<                                                   70590
200 PCALC#PCALC&FST%J<                                                 70600
 99 CONTINUE                                                           70640
    RETURN                                                             70650
    END                                                                70660
    SUBROUTINE SHAPE%DT,G,TH1,THM,YD,M<
    DIMENSION YD%82<
    DIMENSION R%82<,DT%82<
    DIMENSION TA%82<,CONJ%82<
```

```
      R%1<#%G/24.0<*%7.0*DT%1<&6.0*DT%2<-DT%3<<                        SHP70040
      MIN#M-1                                                           SHP70050
      DO 2 K#2,MIN                                                      SHP70060
 0002 R%K<#%G/12.0<*%DT%K-1<&10.0*DT%K<&DT%K&1<<                        SHP70070
      R%M<#%G/24.0<*%7.0*DT%M<&6.0*DT%M-1<-DT%M-2<<                     SHP70080
      TA%1<#0.0                                                         SHP70090
      MP1#M&1                                                           SHP70100
      DO 3 K#2,MP1                                                      SHP70110
 0003 TA%K<#TA%K-1<&R%K-1<                                              SHP70120
      CONJ%1<#0.0                                                       SHP70130
      DO 4 K#2,M                                                        SHP70140
 0004 CONJ%K<#CONJ%K-1<&%G*TA%K<<                                       SHP70150
      AM1#MIN
      CORR#CONJ%M</AM1
      DO 5 K#1,M                                                        SHP70170
      AK1#K-1                                                           SHP70180
 0005 YD%K<#AK1*CORR-CONJ%K<
      TH1#CORR/G
      THM#TH1-TA%M&1<                                                   SHP70220
      RETURN                                                            SHP70230
      END                                                               SHP70240
      SUBROUTINE CMOM%P,SPAN,TRIM,XINC,A,B,W<
      DIMENSION TRIM%82<
      I#1
      X#0.0
      IF %A<1,2,1
    1 IF %B<7,8,7
    8 RLEFT#P*%SPAN-A</SPAN
    9 IF%X-A<10,10,11
   10 TRIM%I<#RLEFT*X
      X#X&XINC
      I#I&1
      GO TO 9
   11 IF %X-SPAN< 13,13,14
   13 TRIM%I<#RLEFT*X-P*%X-A<
```

```
   X#X&XINC
   I#I&1
   GO TO 11
 7 RLEFT#P*%SPAN-A&B</SPAN
20 IF%X-A<21,21,22
21 TRIM%I<#RLEFT*X
   X#X&XINC
   I#I&1
   GO TO 20
22 IF %X-%SPAN-B<<23,23,24
23 TRIM%I<#RLEFT*X-P*%X-A<
   X#X&XINC
   I#I&1
   GO TO 22
24 IF %X-SPAN< 25,25,14
25 TRIM%I<#RLEFT*X-P*%X-A<-P*%B&X-SPAN<
   X#X&XINC
   I#I&1
   GO TO 24
 2 RLEFT#W*SPAN/2.0
 3 IF %X-SPAN<4,4,14
 4 TRIM%I<#W*X*%SPAN-X</2.0
   X#X&XINC
   I#I&1
   GO TO 3
14 RETURN
   END
```

The special characters % = (, < =), # = = and & = +

APPENDIX B

MODEL BEAMS AVERAGE DIMENSIONS

T-Beam	Flange Width Inches b	Web Width Inches b'	Slab Thickness Inches	Total Depth Inches t	Depth Inches d
A-1	12.0	3.0	1.06	6.06	5.59
A-2-1	12.0	3.0	1.10	6.10	5,63
A-2-2	12.0	3.0	1.05	6.05	5.58
A-3-1	12.0	3.0	1.05	6.05	5.58
A-3-2	12.0	3.0	1.05	6.05	5.58
B-1	12.0	3.0	1.05	6.05	5.58
B-2	12.0	3.0	1.03	6.03	5.58
C-2	12.0	3.0	1.08	6.08	5.61
C-3-1	12.0	3.0	1.05	6.05	5.58
C-3-2	12.0	3.0	1.05	6.05	5.58
D-1	12.0	3.0	1.20	6.20	5.73
D-2	12.0	3.0	1.05	6.05	5.58
E-1	12.0	3.0	1.08	6.08	5.61
E-2	12.0	3.0	1.05	6.05	5.58
E-2	12.0	3.0	1.25	6.25	5.78

Table B-1 Summary of Model Beams Dimensions

APPENDIX C

MATERIAL PROPERTIES

Model	Cylinder Size (in)	Age at Time of Test	f'c	Compressive Strength Rate of Loading psi/sec	No. of Cyls.
A-1	3.35 x 6.3	28	2960	25	4
A-2-1	3.35 x 6.7	41	2820	23	4
A-2-2	3.35 x 6.6	6	2200	18	5
A-3-1	3.35 x 6.5	14	2830	22	7
A-3-2	3.35 x 6.6	8	2820	22	4
B-1	3.35 x 6.7	32	3100	26	4
B-2	3.35 x 6.4	26	4140	34	4
C-2	3.35 x 6.6	4	3120	26	4
C-3-1	3.35 x 6.5	35	2480	21	4
C-3-2	3.35 x 6.5	5	3150	25	4
D-1	3.35 x 6.5	12	3240	26	4
D-2	3.35 x 6.5	11	4230	27	4
E-1	3.35 x 6.5	7	3280	27	4
E-2	4.15 x 8.1	7	5000	28	2
E-3	4.15 x 8.0	10	4600	28	3

Table c-1. Summary of Models Beams Quality Control Cylinder Tests.

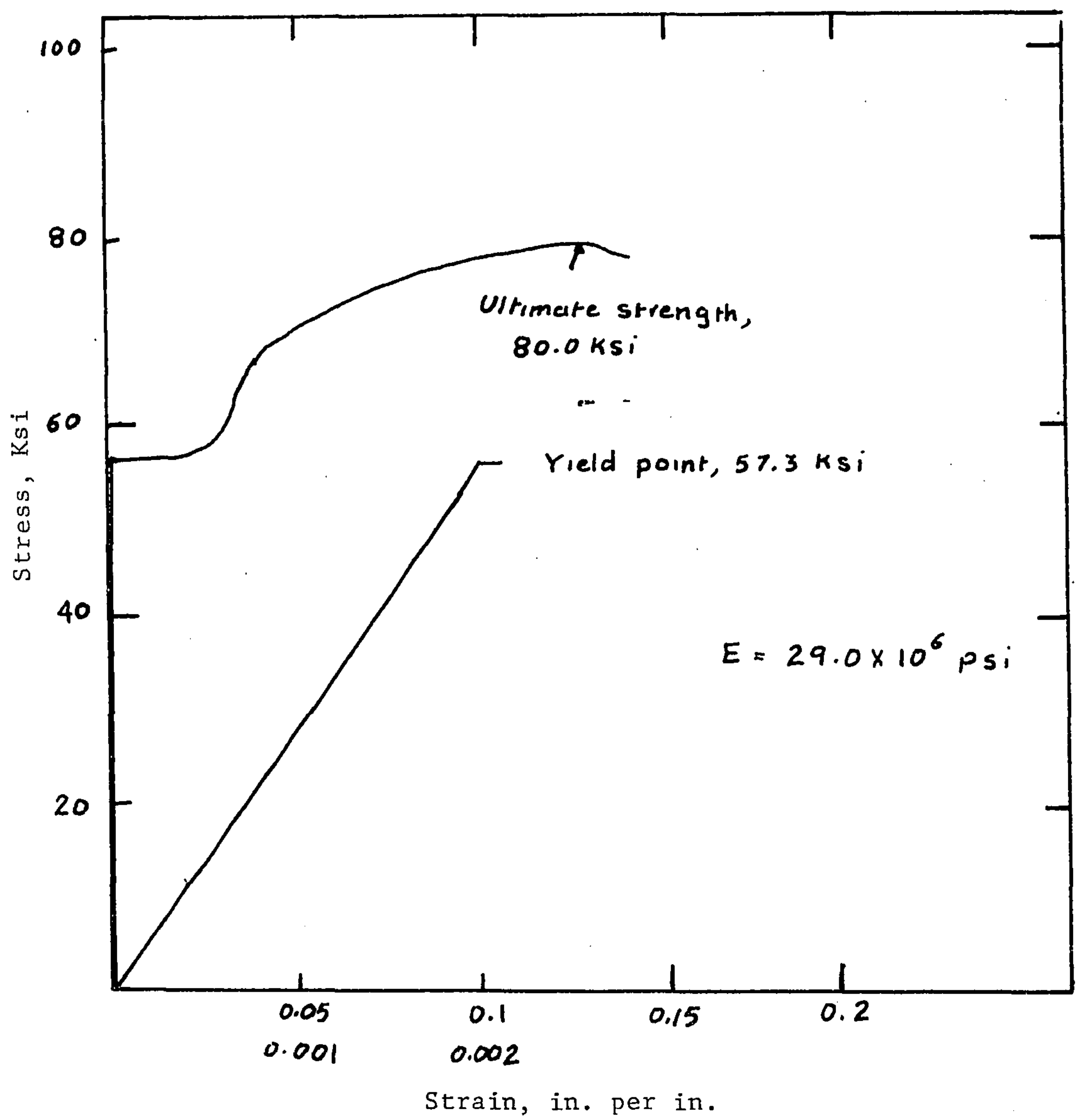

Figure C.1 Stress-Strain Diagram for # 3 Bars

(Main Reinforcing Bars),

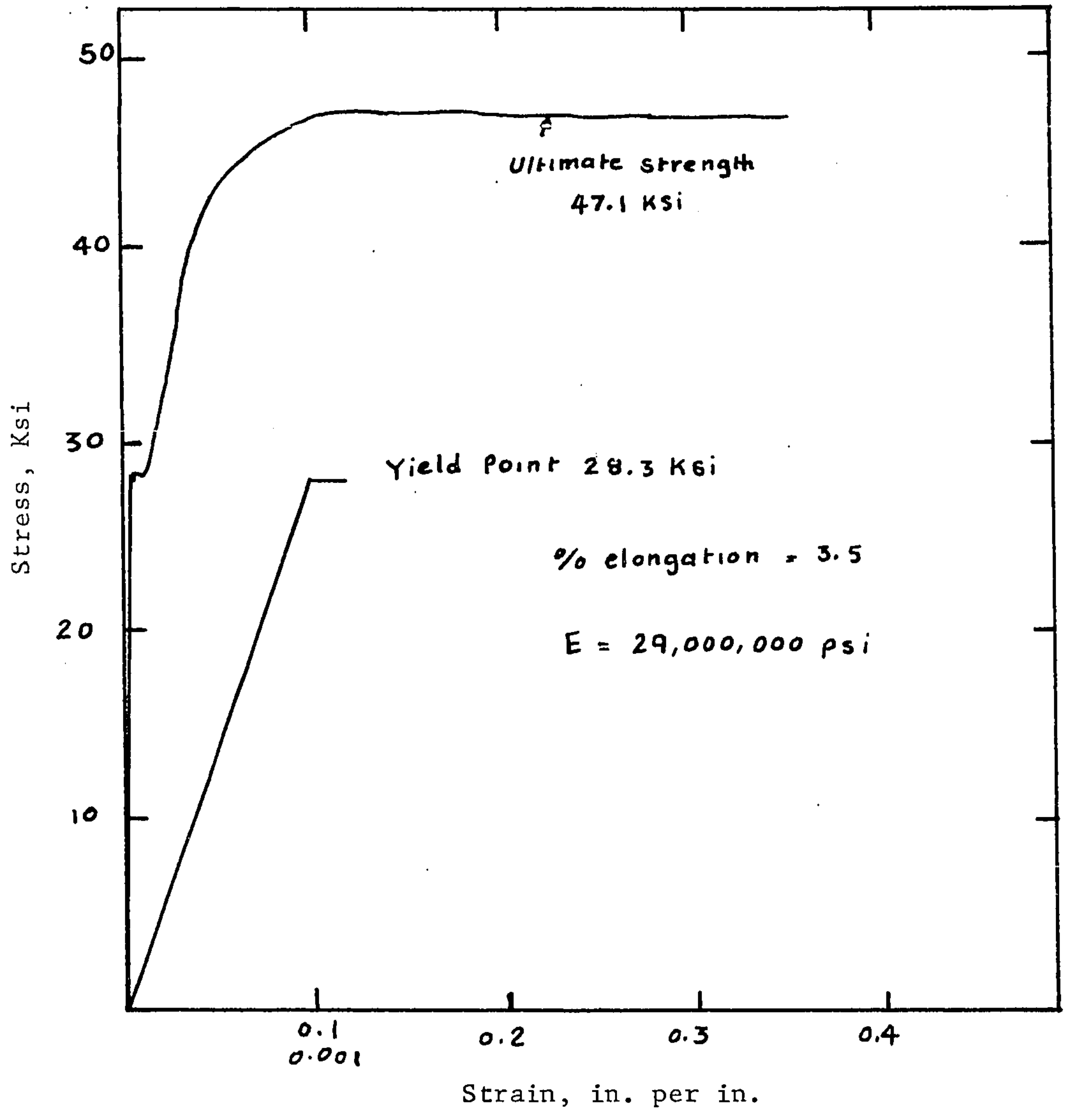

Figure C.2 Stress-Strain Diagram for SWG # 13.

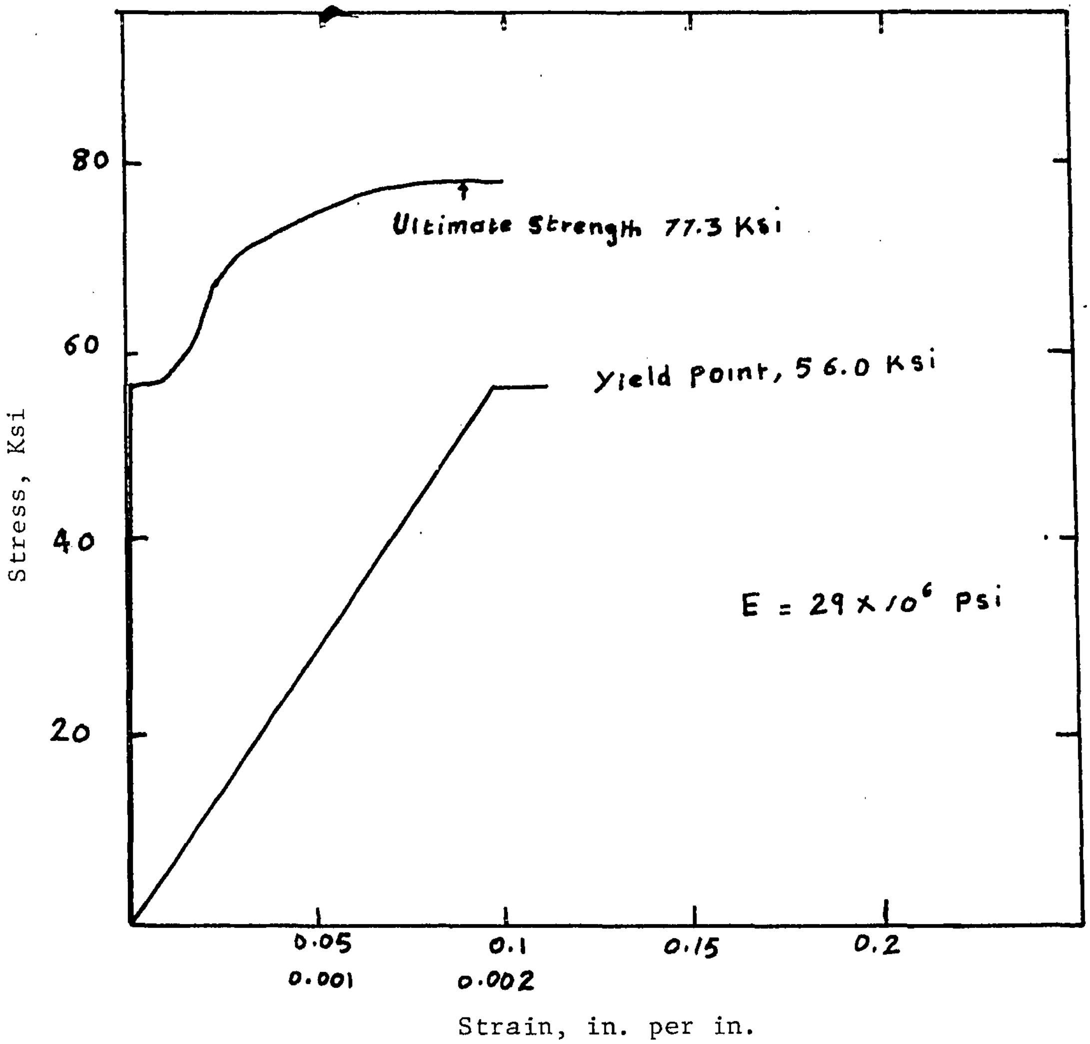

Figure C.3 Stress Diagram for # 2 Bars.

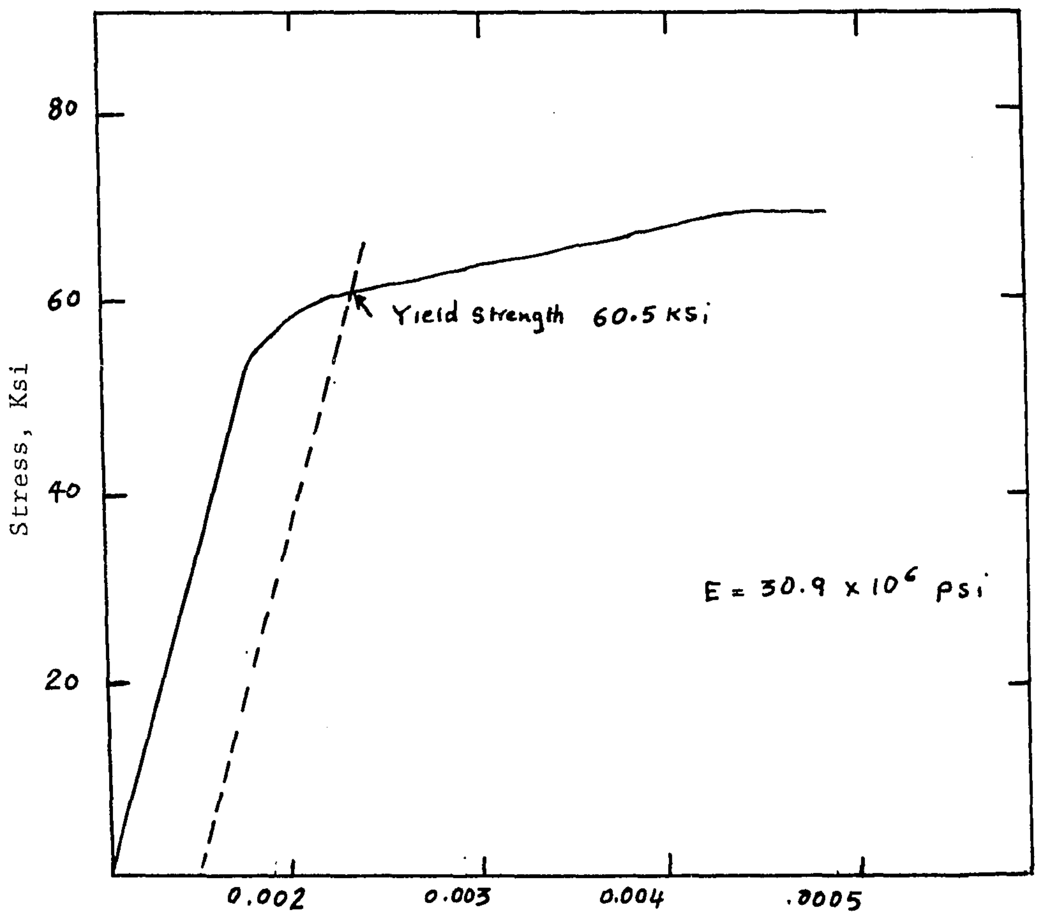

Figure C.4 Stress-Strain Diagram for 3/16" Dia. Steel Bars.

VITA

Michael S. Dallal was born in Jaffa, Palestine, on December 26, 1926, the son of Salim Dallal and Emilia Dallal. After graduation from Amiria High School, Jaffa, Palestine, in 1946 he was employed by the Palestine Education Administration as a teacher. In September of 1950, he enrolled at Cairo University, Cairo, U.A.R., and was awarded the degree of Bachelor of Science in Civil Engineering in July 1955. In September 1955, he was employed as a design and construction engineer by Othman Ahmad Othman Corporation. He served in their main office in Cairo, U.A.R. and their office in Riyad, Saudi Arabia. In September 1959, he was employed by the Highway Department, Riyad, Saudi Arabia, as design and inspector engineer, at the same time he was a consultant for Riyad Bank, Saudi Arabia. In September, 1962, he came to U.S.A. and enrolled at the University of Oklahoma, Norman, Oklahoma. He was awarded the degree of Master of Science in Civil Engineering in August 1963. In September, 1965, he was employed by Old Dominion College, Norfolk, Virginia, as Chief Instructor and Head of Civil Engineering Technology. In August 1967, he returned to the University of Oklahoma.