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EFFECT OF DRAPED REINFORCEMENT ON BEHAVIOR OF PRESTRESSED CONCRETE BEAMS

A Thesis
by
J. G. MacGregor

Issued as a Part
of the
SEVENTH PROGRESS REPORT
of the
INVESTIGATION OF PRESTRESSED CONCRETE
FOR HIGHWAY BRIDGES

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Issued as a Part of the Seventh Progress Report of the

INVESTIGATION OF PRESTRESSED CONCRETE

FOR HIGHWAY BRIDGES

Conducted by

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STATE OF ILLINOIS

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I. INTRODUCTION

1.1 Introduction

Tests of over a hundred beams with straight longitudinal reinforcement and no web reinforcement, carried out in the earlier phases of this investigation (1, 2, 3)*, showed that reinforcement should be provided to resist inclined tension stresses as well as horizontal tension stresses. Two different types of reinforcement are commonly used for this purpose: stirrups, and draped reinforcement. The term "draped reinforcement" refers to longitudinal reinforcement which is curved in elevation in the shear spans. At the supports the centroid of draped reinforcement is closer to the top of the beam than it is at midspan. Stirrups of various grades of steel and of various shapes are used solely to carry "shear". On the other hand, the longitudinal reinforcement may be draped, primarily to relieve tension stresses in the top flange or compression stresses in the bottom flange or to alleviate anchorage stresses. The slope of the tendons results in an upward component of the prestressing force which is customarily subtracted from the total downward shear acting on the section.

The investigation of the effect of vertical stirrups on the strength and behavior of prestressed concrete beams has been initiated by Mr. G. Hernandez whose findings are reported in a thesis published as a part of the Seventh Progress Report of the Investigation of Prestressed Concrete for Highway Bridges (4). The work described in this report has been undertaken to determine whether draped reinforcement improves the strength and behavior of prestressed concrete beams.

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* Numbers in parentheses refer to entries in the Bibliography.

1.2 Object and Scope

The object of this report is to describe and discuss the results of tests on sixteen simply supported pretensioned concrete beams with varying degrees of drape.

The overall cross-sectional dimensions for the beams were 6 by 12 in. There were 14 I-beams with 3-in. webs, one I-beam with a 1 3/4-in. web and one rectangular beam. All the beams were tested to failure with loads applied at the third-points of a 9-ft. span. The prestress and concrete strength were approximately 115,000 psi and 3,000 psi, respectively, in all beams. Vertical stirrups were used in two beams. The nominal dimensions are shown in Fig. 1, and the properties of the beams are listed in Table 1.

The principal variable in the test series was the profile of the prestressing steel. In all, nine different profiles were used as outlined in Table 2 and shown in Fig. 2. Those designated A through E had all the wires draped at the same angle in each shear span and those designated X through Z had only part of the wires draped. Profile S had only straight wires.

The drape profile used in an actual bridge member will depend on such factors as the method of prestressing, the method of draping, and the strength requirements of the finished structure. For this reason, several types of drape profiles, each quite different, are commonly used. In order to limit the number of variables, only the two basic types of drape profiles shown on Fig. 2 have been used in these tests. These were made up entirely of straight line segments, all the wires being level and parallel in the center third of the beam. In each case, the wires were bent at the load points.

Although it is not customary to drape all the reinforcement in a beam, this was done in ten of the test beams to isolate the effect of draped wires alone. Six different drape angles were used, varying from zero to ten degrees. The behavior of a beam with both straight and draped wires should lie somewhere between the extreme conditions of a beam with straight wires only and one with draped wires only. To confirm this idea, four beams with only some of the wires draped were tested.

1.3 Acknowledgments

The studies reported herein were made as a part of an investigation of prestressed reinforced concrete for highway bridges conducted by the Department of Civil Engineering of the University of Illinois in cooperation with the Division of Highways, State of Illinois, and the U. S. Department of Commerce, Bureau of Public Roads.

The program of the investigation has been guided by an advisory committee on which the following persons have served during the period covered by the work described in this report: E. L. Erickson and Harold Allen, representing the Bureau of Public Roads; W. E. Chastain, Sr.; W. J. Mackay, and C. E. Thunman, Jr., representing the Illinois Division of Highways; and C. P. Siess and N. Khachaturian representing the University of Illinois.

The project has been under the general direction of Dr. C. P. Siess and the immediate direction of Dr. M. A. Sozen.

This report was written as a thesis under the direction of Dr. Siess, and his assistance in planning the tests and his helpful comments are gratefully acknowledged. Credit for assistance in the development of equipment and analysis and critical study of the manuscript of this report is due Dr. M. A. Sozen. Appreciation is expressed to C. J. Fleming, N. M. Hawkins, Research Assistants in Civil Engineering and G. Tiezzi, Student, for their aid in conducting the tests, reducing data and preparing figures. Special thanks are given to Wyck McKenzie, Junior Laboratory Mechanic in Civil Engineering, for his aid in fabricating specimens and equipment.

The wire reinforcement was furnished without charge by the American Steel and Wire Division of the United States Steel Corporation.

1.4 Notation

The symbol M is used for moments on a beam and V denotes the shear in one shear span. These symbols may have the subscripts: c, referring to conditions at the formation of the inclined crack; d, referring to the upward force caused by draping the wires; f, referring to the computed flexural capacity of the beams; and u, referring to the conditions at ultimate loads. The symbol f is used for stress.

Cross-Sectional Constants and Beam Properties

A_c = gross area of cross section

A_s = total area of longitudinal reinforcement

A_v = area of one stirrup

b = top flange width

b' = web thickness

d = effective depth of the reinforcement in the flexure span

E_s = modulus of elasticity of steel

s = spacing of stirrups

φ = angle of drape = angle between the horizontal and the draped wires

Loads and Stresses

f'_c = compressive strength of concrete determined from 6- by 12-in. control cylinders

f_r = modulus of rupture of concrete determined from 6- by 6- by 24-in. control beams loaded at the third-points over an 18-in. span

f_{se} = effective prestress

f_t = assumed tensile strength of concrete

F_{se} = effective prestressing force

M_c = applied bending moment at inclined tension cracking

M_c' = computed bending moment at inclined tension cracking for a beam with straight longitudinal reinforcement

M_f = ultimate bending moment for flexural failure

M_u = ultimate moment measured in test

P_c = applied load at inclined tension cracking

P_f = applied load at flexural failure

P_u = applied load at ultimate load

V_c = applied shear at inclined tension cracking

V_d = upward component of prestressing force

V_n = net shear or effective shear at inclined cracking

Dimensionless Quantities

$p = A_s/bd$ = reinforcement ratio

$Q = E_s p/f_c'$

$r = A_v/b's$ = web reinforcement ratio

II. MATERIALS, FABRICATION, AND TEST SPECIMENS

2.1 Materials

(a) Cement. Marquette Type III Portland Cement was used for all the beams presented in this report. All the cement was purchased in paper bags from local dealers and stored under proper conditions.

(b) Aggregates. Wabash River sand and pea gravel were used in all the beams. Both aggregates have been previously used in the laboratory and have passed the usual specification tests. The maximum size of the gravel was 3/8 in. The absorption of both fine and coarse aggregate was about one percent by weight of the surface dry aggregate. Sieve analyses of the aggregates are given in Table 3.

The aggregates were from a glacial outwash of the Wisconsin glaciation. The major constituents of the gravel were limestone and dolomite with minor quantities of quartz, granite and gneiss. The sand consisted mainly of quartz.

(c) Concrete Mixes. Concrete mixes were designed by the trial batch method for a seven-day compressive strength of 3,000 psi and a slump of 3 in. Two batches were used in each beam, Batch I being in the lower half of the beam. Table 4 lists the proportions of the concrete batches used in each beam along with the slump, compressive strength, modulus of rupture, and age of test. Proportions are in terms of oven dry weights. The compressive strength, f'_c, is the average strength of a minimum of four 6- by 12-in. control specimens. Values reported for the modulus of rupture, f_r, are results of tests of 6- by 6- by 20-in. control beams loaded at the third-points of an 18-in. span. The concrete strength and modulus of rupture were determined immediately after testing each beam.

In Fig. 3, the moduli of rupture are compared with compressive strengths. Since a measure of the tensile strength of the concrete in each beam was necessary for the interpretation of the test results, and since the scatter in the data did not warrant the use of the results of individual control beams, the statistical expression used by Sozen (3) was selected to represent the test data. For concrete with small-size coarse aggregate this expression was:

$$f_r = \frac{3,000}{4 + \frac{12,000}{f'_c}} \tag{1}$$

The tensile strength of the concrete is assumed to be two-thirds of the modulus of rupture as expressed by Equation (1), since no tests were made to determine the tensile strength directly.

(d) Prestressing Wire. Steel designated as Lot XI and Lot XII were used as prestressing reinforcement for the beams. This was manufactured by the American Steel and Wire Division of the United States Steel Corporation and is designated by the manufacturer as "Hard Drawn Stress-Relieved Super-Tens Wire". The following steps were involved in its manufacture: hot rolling, lead patenting, cold drawing, and stress relieving. The wire was delivered in coils about 54 in. in diameter and weighing approximately 260 lb. The diameter of the wire itself was measured to be 0.196 in. The following heat analyses have been furnished by the manufacturer:

Lot No.	Carbon	Manganese	Phosphorus	Sulfur	Silicon
XI	0.85%	0.65%	0.010%	0.027%	0.18%
XII	0.88%	0.79%	0.024%	0.033%	0.25%

To improve the bond characteristics, all wires were first wiped with a cloth dipped in a hydrochloric acid solution and then placed in the moist room for a week to rust. This operation produced a slightly pitted surface. Insulating tape was wrapped around the wire at the intended locations of electric strain gages before rusting. All wires were cleaned with a wire brush just before use to remove loose rust.

Samples cut from different portions of the roll were tested in a 120,000-lb capacity Baldwin hydraulic testing machine. Strains were measured with an 8-in. extensometer employing a Baldwin "microformer" coil and were recorded by an automatic recording device. The extensometer was removed from the specimen at about 2.5 percent strain to prevent damage to the instrument. The average stress-strain relationships for Lot XI and Lot XII wires are shown in Figs. 4 and 5.

(e) Stirrup Wire. Beams M10 and M11 had No. 8 black annealed wire stirrups. The wires were rusted and samples were tested in the same way as indicated for the prestressing steel. The stress-strain relationship for this wire is shown in Fig. 6.

2.2 Description of Specimens

All the test beams were 10 ft long and nominally 6 by 12 in. in cross-section. Fourteen beams had 3-in. webs, one had a 1 3/4-in. web and one was rectangular in cross-section. The nominal dimensions are shown on Fig. 1. The end blocks were about 12 in. long. The beams without web reinforcement had prestressed external stirrups to prevent complete failure of the end block. Two of these stirrups were used in each beam, one at each end immediately on the outside of the reaction block.

Single wire reinforcing was used in all beams. The minimum vertical and horizontal spacing between these wires was maintained at 0.75 in. and

0.70 in. respectively. This spacing corresponds to that used by Sozen (3) and Hernandez (4). In all cases the wire profiles consisted entirely of straight line segments, the wires being bent only at the end plates and at the third-points of the span. In all beams the prestressing wires were parallel and horizontal in the flexure span. The center of gravity of the wires in the flexure span was at two inches above the bottom of the beam unless otherwise noted below. For the beams with eight wires, the lowest wires were thus 1 5/8 in. above the bottom of the forms while in the beams with six wires this distance was 1 3/4 in.

The prestressing wires were either all draped parallel to one another or some of the wires were draped and the rest of the wires straight. In all, nine drape profiles were used. These are described in Table 2 and Fig. 2 and will be referred to using the letter symbols given there. The center of gravity of the longitudinal steel over the support for drape profiles B, C and E passes through the lower kern point, the mid-height and the upper-kern point of the I-section, respectively. In the following sections these particular profiles are sometimes referred to in short as lower kern point drape, mid-height drape, and upper kern point drape.

Web reinforcement was used in beams M10 and M11. The spacing and dimensions of the stirrups are shown in Fig. 7. The details of the stirrups were chosen to be similar to those in beams G19 and G30 tested by Hernandez (4). The location of each stirrup was shifted 2 1/2 in. from the corresponding position in G19 and G30. The spacing was kept at 5 in., however.

In general, bridge members have a greater span to depth ratio than the beams in this test series. This, in turn, restricts the angle of drape possible in a bridge beam. The maximum angle of drape used in the Northern Illinois toll-road bridges was approximately 8.25 deg. with the horizontal for the draped wires, but only about 1.67 deg. for the center of gravity of all the steel. The cross-section of the test beams are similar to bridge beams except that the latter usually have cast-in-place decks. The ratios of web thickness to lower flange thickness of the AASHO standard sections range from 0.308 to 0.375 compared to 0.50 for the 3-in. I-beams of this test series and 0.29 for the 1 3/4-in. I-beam.

2.3 Prestressing and Draping

(a) End Details of Wires. A threaded end anchorage was used on the wires of all the beams during the prestressing operation. After release, no end anchorages were used. The threaded anchorages consisted of a special heat-treated nut screwed on the threaded end of a prestressing wire. Wires were threaded 24 threads to the inch for about three inches on each end in an automatic threading machine with specially heat-treated chasers. The threads on the wires were cut to provide a medium fit with the threads in the nuts.

The nuts were specially made in the laboratory machine shop. They were subdrilled with a No. 16 tap drill and tapped with a standard No. 12, 24 threads to the inch, tap. The thread cut on the wires to fit a No. 12 thread in the nut was sufficient to develop at least 190 ksi in the wire.

The nuts were 5/8-in. long and hexagonal in cross-section. They were made from hardened "Buster" alloy punch and chisel steel having the following composition limits: carbon, 0.56 to 0.60 percent; silicon, 0.60 to 0.80 percent; chromium, 1.10 to 1.30 percent; tungsten, 2.00 to 2.30 percent; and vanadium, 0.20 to 0.30 percent.

(b) Tensioning Apparatus. Since the beams in this series were all pre-tensioned, a prestressing frame was necessary to provide a reaction for the tensioning force. The frame consisted of two 11 ft 6 in. lengths of extra-heavy 3-in. diameter pipe and two bearing plates, 2 by 6 by 20 in. It was built to fit around the form for the beam. The bearing plates had six rows of 0.206-in. diameter holes to accomodate the various positions of the wires. The holes were 0.70 in. apart horizontally and 0.75 in. vertically.

A thirty-ton capacity Simplex center-hole hydraulic ram operated by a Blackhawk pump was used to tension the wires. A U-shaped jacking frame fitted between the pretensioning frame and the jack. To tension the wires the ram reacted against the frame and a 5/8-in. diameter rod. The thrust was transferred from the ram to the rod through washers and a nut. The rod in turn extended through the center hole in the ram and was directly connected to the wires by means of a heat-treated nut welded to the rod. After the wire was tensioned to the desired stress the nut on the wire was turned up against a shim and the jack was released allowing the nut and shim to bear on the pretensioning frame.

(c) Draping Apparatus. The reinforcing wires were tensioned in their uppermost position and then were pulled down to their final position by two draping saddles, one at each third-point. The draping saddles consisted of two long threaded 3/8-in. diameter rods with two 2 1/2-in. lengths of 1/2-in. diameter rod welded across them at one end (see Fig. 8). The cross-bars and the threaded vertical rods were dimensioned to allow the wires to pass between them in their normal spacing. The lower ends of the vertical rods passed through holes in the bottom of the form and the saddles were held in position by nuts bearing on the bottom of the form. No provision was made for longitudinal movements of the wires during draping since the saddles were flexible enough in that direction. This caused no trouble in draping the wires.

The form members rested on a stiffening beam built up from plates and two 15-in. channels. The form and beam are shown in Fig. 9. This beam was necessary to prevent the forms from warping when the prestressing wires were draped.

For low drapes it was possible to do all the draping by screwing nuts onto the threaded rods. A hydraulic jack was used to pull down the saddles for the higher drapes.

(d) Measurement of Prestressing Force. The tensioning force in each wire was determined by measuring the compressive strain in cylindrical aluminum dynamometers slipped on the wire between the nut and the bearing plate at the end opposite to which the tension was applied. The dynamometer consisted of a 2-in. length of 1/2-in. aluminum rod with a 0.2-in. diameter hole drilled through its center. Strains were measured by means of two Type A7 SR-4 electric strain gages mounted on opposite sides of each dynamometer and wired in series. This arrangement gave a strain reading which was the average of the strains in the two gages thereby compensating for small eccentricities of load that might occur. The dynamometers were calibrated using the 6000-lb range of a 120,000-lb capacity Baldwin hydraulic testing machine. The calibration constants of the dynamometers were very nearly the same; the strain increment necessary to measure a tensioning stress of 120 ksi in the wire was about 2000 millionths. This large increment of strain allowed a precise measurement of stress in the wires, since the strain indicator used had a sensitivity of two or three millionths.

Strains in the wires were measured by means of Type A-7 SR-4 electric strain gages mounted at various points along the beam. Measurements taken during draping showed that for all the beams instrumented in this way, the average stress at the jacking end and at the dynamometer end were 100 and 101 percent, respectively, of the stress in the flexure span. For this reason, beams M9 through M16 had electric strain gages at midspan only.

(e) Tensioning, Draping and Releasing Procedure. The longitudinal reinforcement was tensioned in the prestressing frame prior to draping and casting. The ends of the wires were slipped through the draping saddles, the end plates of the form, and through the bearing plates of the prestressing frame. The dynamometers were then slipped onto one end of the wires and the anchoring nuts were put on each end of the wire. The wires were tensioned individually to the desired stress less the increment of stress which would be added by draping the wires. An allowance was made for the elastic shortening of the prestressing frame. In drape profiles D and E, the draping process alone would have stressed the wires to more than 120 ksi and it was necessary to release some stress at intervals to maintain the desired prestress in the finished beam.

After initial tensioning, the prestressing frame and wires were transported to the form and the ends of the saddles were fitted through holes in the bottom of the form. The end plates were bolted to one side form to prevent being pulled out of line during the draping procedure. The other side form was not in place during draping to facilitate measurement of the height of the wires. The wires were then pulled down into position with saddles. Dynamometer and steel strain readings were taken before and after the draping operation. As noted in Section 2.3(d), there was no significant variation in stress from one part of the beam to another.

Two days before testing, the tension was released, ends first and drape last. Dynamometer and steel strain readings were taken before and after release.

Minor difficulties were encountered in prestressing the beams with only some of the wires draped. Since the wires to be draped and the straight wires were at opposite edges of the bearing plates, the bearing plates pivoted slightly against the ends of the pipes as the straight wires were prestressed.

This trouble was most noticeable with drape profile Z in which the draped wires gained most of their stress as they were deflected into position.

The force required to drape the upper-kern point beams was practically equal to the capacity of the threads of the draping saddles and in one case they were stripped.

2.4 Casting and Curing

All the beams in this test series were cast in metal forms. The side forms were made from 12-in. channels bolted securely to the stiffening beam described in 2.3(c). The form and stiffening beam are shown in Fig. 9. Removable metal inserts bolted to the side forms were used to form the webs of the beams tested.

All concrete was mixed from three to six minutes in a non-tilting drum type mixer of six cubic feet capacity, and was placed in the form with the aid of a high frequency internal vibrator. Each beam required two batches, the first batch filling the lower half of the beam. Six 6 by 12 in. control cylinders and one 6 by 6 by 20 in. modulus of rupture beam were cast from each batch.

Several hours after casting, the top surface of the beam was trowelled smooth and the cylinders were capped with neat cement paste. The forms of the beam and control specimens were removed after one day and the beam and control specimens were wrapped in wet burlap for several days. The burlap was removed three to five days before testing to allow the concrete surface to dry before electric strain gages were applied.

III. INSTRUMENTATION, LOADING APPARATUS, AND TEST PROCEDURE

3.1 Instrumentation

Strains in the wires were measured with Type A-7 SR-4 electric strain gages which have a nominal gage length of 1/4 in. and a minimum trim width of 3/16 in. They were chosen for their narrow width, short length, and flexibility. The gages were placed at midspan on one wire in each layer of steel. In beams M1 to M8 gages were also provided in the shear spans at three inches from the point of draping. The location of the steel strain gages is shown in Fig. 10.

The surface of the wire was prepared for gage application by using fine emery cloth and acetone. The gage was mounted with Duco cement. After several hours of air drying, heat lamps were used to speed the drying of the cement. Following this, while the wire was still warm, a layer of wax was applied on the gage to protect it from moisture. The gage was then wrapped with one layer of electric tape. Lead wires were soldered on and the whole assembly was wrapped again with electric tape. Because the beams were pretensioned it was necessary to waterproof the gages carefully since they would be in contact with wet concrete. Waterproofing was accomplished by pouring melted petrolastic into a preformed tin container 1 1/2 by 1/2 by 1/2 in. which was attached to the reinforcing wire.

Strains in the concrete at the top of the beam were measured with Type A-3 SR-4 electric strain gages. These gages had a nominal gage length of 3/4 in. and a width of 3/8 in. A portable grinder was used to smooth the top surface of the beam at the desired locations. A thin layer of Duco cement was applied to the smooth surface and allowed to dry for several minutes. A layer of Duco cement was then applied to the gage and it was mounted in place. Care was taken to remove all air bubbles from under the gage. The concrete gages were mounted two days before the test. No waterproofing or curing was used. The position of the top concrete gages is shown in Fig. 10.

Strains were read to the nearest 10 millionths with a Baldwin portable strain indicator. Dummy gages for temperature compensation were mounted on untressed steel blocks.

Deflections at midspan and at each third-point were measured with 0.001-in. dial indicators. The indicators were mounted on posts attached to a steel channel which spanned between the piers supporting the beam.

3.2 Loading Apparatus

All beams were tested in a specially constructed frame employing a 30-ton capacity Simplex hydraulic ram operated by a Blackhawk pump. Details of the frame are shown in Fig. 11. The hydraulic jack was used to apply deformation and a 50,000-lb capacity elastic ring dynamometer was used to measure the corresponding loads. The dynamometer was equipped with a dial indicator that was calibrated at 111 lb per division.

The beam rested on two 6 by 8 by 2-in. bearing blocks attached to the bottom of the beam with hydrocal. These in turn rested on a "half round" at one end and a roller at the other. The loading blocks were also 6 by 8 by 2-in. plates but these each rested on 4-in. squares of leather. Leather was used rather than hydrocal so that electric strain gages could be placed close to the point of loading.

3.3 Test Procedure

The failure load was usually reached in five to ten increments of load. Load and deflection readings were taken on the run during each increment of load. After a load increment, all deflection and strain measurements were taken and the cracks were marked. Load and mid-span deflection were measured again immediately before the resumption of loading since a drop-off in load and an increase in deflection occurred during the interval between load increments.

Usually there were two to three increments of load up to the flexural cracking load. After flexural cracking, small increments based on deflection were applied, the rate of loading depending on the development of the crack pattern. The beams were loaded until they ruptured completely or failed to develop increased resistance to large deformation. Control specimens were tested immediately after the beam test.

3.4 Measured and Derived Quantities

During prestressing, draping, and release, the prestress was measured by the dynamometers on the wires. The uniformity of the stresses in the three segments and the elastic losses were checked by the strain gages on the wires.

During loading, the applied load was measured by an elastic ring dynamometer, deflections by dial indicators, and concrete and steel strains by SR-4 strain gages. The cracks were marked at each load increment and photographs were taken at the end of the test. After failure, the web thickness and other dimensions were measured. After each beam was tested, the control cylinders and beams were tested for compressive strength, modulus of elasticity and modulus of rupture.

From the measured quantities the following information was derived: the prestress was determined from the dynamometers and the measured elastic losses and was checked using the flexural cracking load. The measured elastic losses were compared to computed elastic losses. The distribution of top concrete strains was determined. The moments and shears in the beam were computed.

IV. BEHAVIOR OF TEST BEAMS

4.1 Definition of Flexural and Shear Failures

A flexural failure is one due to bending stresses only. In the type of beam tested, such a failure would be expected to occur by crushing of the concrete over a crack within the constant moment section of the beam. Strains over the depth of the beam at the section of failure remain nearly linear throughout the life of a beam failing in flexure.

Shear failures result from a combination of bending and shear stresses. Such a failure is characterized by a major inclined crack in the shear span extending upward toward the point of loading. This crack disrupts beam action and results in a failure load smaller than the flexural capacity of the beam. The reduction in strength caused by the inclined crack depends on the properties of the beam. Inclined tension cracking and the different modes of shear failure are described in Section 4.3.

4.2 Flexural Failures

As a prestressed concrete beam is loaded to failure in flexure, its behavior goes through two distinct stages. In the first stage, before cracking, the beam behaves "elastically". The first flexural crack forms near midspan and is followed by similar cracks throughout the constant moment region. With the formation of these cracks, the stiffness of the beam decreases, and the beam enters the second stage of behavior. One or two flexural cracks may form in the shear spans as loading is continued. Further loading causes the neutral axis to rise higher in the beam until failure occurs by crushing of the concrete above one of the flexural cracks. If the steel percentage is small or moderate, the steel stress at failure will exceed the proportional limit and the failure will be gradual and gentle with ample warning. For very small steel percentages, fracture of the steel may precede crushing of the concrete.

Two beams of the series described in this report, beams M9 and M13, exhibited typical flexural failures. The ductile behavior of these beams under load is shown by the shape of their load-deflection curves in Fig. 12. Before flexural cracking, the curves are straight and steep. After cracking the stiffness of these beams decreased greatly as shown by the second portion of the curves.

4.3 Shear Failures

(a) Inclined Tension Cracks. The inclined cracks which develop in a region of combined shear and moment are called "inclined tension cracks". The initiation of inclined tension cracks is discussed in Section 4.3(b). These cracks cause a major redistribution of stresses and change the behavior of the member from beam action to a type of arch action in the region of the crack.

In a beam without stirrups, only the component of the inclined tension stresses in the direction of the longitudinal steel can be resisted by reinforcement. Therefore, an inclined tension crack develops more rapidly and extends higher than a flexural crack, decreasing the depth of the compression zone.

For a given angle change in a beam with an inclined crack, the compressive concrete strains are concentrated in a short length of compression zone above the highest part of the inclined crack, while the steel strains are distributed over a length at least equal to the horizontal projection of the crack. Thus, the strains at the crack are no longer linear over the depth of the beam. Sozen (3) has shown that the steel strain after inclined tension cracking may be from 30 percent to as low as one percent of that required for a linear distribution of strains at the cracked section. If the beam splits along the reinforcement in the vicinity of the crack, the steel strains are distributed over a still greater length of reinforcement and the strain incompatibility is more pronounced at failure.

The definition of inclined tension cracking used in this report is based on the phenomenon of non-linearity of strains. The method of determining the inclined tension cracking load is illustrated in Fig. 13 which compares the measured increase in steel strains to the measured concrete strains at various points along the top of beam M15. The steel strains are those occurring at the center of the flexural span. This location was chosen since it was not possible to measure the steel strains at the inclined tension crack. However, since the steel strains should be relatively constant throughout the flexure span, it was deemed feasible to compare the critical concrete strains with the steel strain at midspan. This assumption was further justified by the facts that trends rather than exact values were sought and the steel strain gage at midspan was almost always located at a cracked section.

Curves A, B and C in Fig. 13 represent conditions at the gage locations shown in the sketch of beam M15. In the two initial stages of behavior before inclined tension cracking, the relationship between concrete and steel strains is linear and approximately similar for all three locations. After flexural cracking, gage B, subjected to pure flexure, maintained a constant ratio of strains throughout the remainder of the test. As can be seen from the sketch of the beam, inclined tension cracks opened below gages A and C. This is reflected by the second major change in slope of lines A and C on Fig. 13. At the load corresponding to the break in the curve, the concrete strains at A started to increase more rapidly than the steel strains. This load has been defined as the inclined tension cracking load. An inclined crack also formed at end C. The ultimate failure of this beam was caused by crushing immediately under gage A.

In many cases the inclined tension cracking load was clearly evident since the crack opened suddenly to its full height. It was only necessary to use the above procedure to determine the inclined cracking load when the inclined crack developed gradually from an initiating flexural crack in the shear span.

The non-linearity of strains at the inclined tension crack obviously limits the steel stress which may be utilized to resist the applied moment. Therefore, a much larger rotation of the section at the inclined crack is required to preserve equilibrium for each increment of moment than would be required at a vertical flexural crack. Since crushing occurs when the compression strains reach some limiting value, failure at an inclined crack will occur at a lower load than at a flexural crack. This type of failure is called a "shear-compression" failure. A beam with draped wires is at a disadvantage in resisting moment at an inclined crack because the moment arm of the steel force has been reduced by raising the center of gravity of the steel.

Since a beam is primarily a flexural member, its dimensions are chosen on the assumption that the ultimate failure of the beam will be in flexure. If, however, an inclined tension crack and its subsequent strain concentrations develop before the flexural capacity is reached, a weaker beam will result. The failure will be more brittle than a flexural failure and there will be less warning of the impending failure. If the inclined tension crack opens at approximately the same load as the flexural failure load the beam will most likely fail in shear. In this case, however, there will be little difference in the deflections and ductility at ultimate load, whether the beam fails in flexure or shear. If there are no inclined tension cracks in the beam at the ultimate flexural load, the beam cannot fail in shear since an inclined tension crack is necessary for a shear failure. In general, beams with high ratios of longitudinal steel percentage to concrete strength, moderate effective prestress levels, thin webs, and short shear spans develop shear failures at loads lower than their flexural capacities. On the other hand, there is less difference between the ultimate loads for shear of flexural failures for beams with low ratios of longitudinal steel percentage to concrete strength, high effective prestress levels, relatively thick webs and long shear spans. For such beams, the flexural failure load may be less than the load necessary to cause inclined tension cracking. The beams most commonly used in present construction resemble the latter group.

(b) Development of Cracks. Until the formation of inclined tension cracks, the behavior of a beam failing in shear is essentially similar to that of a beam failing in flexure. Flexural cracks occur in the constant moment region and, as the load is increased, flexural cracks and/or inclined tension cracks develop in the shear spans.

Generally, the inclined tension crack may originate in three ways: 1) A flexural crack in the shear span may produce a state of stress sufficient to cause inclined tension cracking in the uncracked portion of the beam above or adjacent to the flexural crack. 2) The original inclined tension crack may originate in the web before flexural cracking, if the principal tensions in the web are large enough. 3) The initial crack may form at the juncture of the upper flange and web near the support, if very high prestresses at large eccentricities are used in combination with thin webs. The latter type of cracking has been designated "Secondary Inclined Tension Cracking" by Sozen (3).

The dimensions of most of the beams tested in this series were such that inclined tension cracks resulted most commonly from a flexural crack in the shear span. This crack usually occurred within a distance equal to the depth of the beam from the point of loading, and grew vertically as far as the longitudinal reinforcement. As the crack extended above this height, it bent over toward the load point. In some beams the initial crack and its subsequent development constituted the inclined tension crack (Fig. 14) and in others it merely acted as a stress raiser to cause the formation of a true inclined tension crack (Fig. 15). In further discussions the initial flexural crack in the shear span which appears to influence the formation of the inclined tension crack will be referred to as the "initiating crack" and the inclined tension crack causing the failure will be referred to as the "major inclined tension crack".

Of the sixteen beams tested in this series, eight developed an initiating crack before the major inclined tension crack. In two of these beams (M2 and M10; see Fig. 14) the inclined tension crack was an outgrowth of the original initiating crack. In six other beams, the initiating crack developed, crossed the tension steel, and bent over toward the load point. Immediately before the inclined crack formed, the initiating crack triggered splitting along the steel which in turn triggered the inclined tension crack. The steel between the two cracks, and often throughout the shear span, was unbonded by this splitting. Splitting at the level of the steel was most noticed with drape profile C and was somewhat retarded in drape profile B since the angle of drape was small in the latter case and the wires were in the lower flange at the location of the initiating crack. In one of the beams which developed the so-called initiating crack, there did not appear to be any relation between the initiating crack and the inclined tension crack.

In beam M7, the only beam with a small ratio of web to flange thickness, the first cracking occurred in the web as a result of principal tensions. (See Fig. 16).

"Secondary inclined tension cracking" did not occur in any of the beams tested.

(c) Observed Modes of Shear Failure. After inclined tension cracking the linear relationship between concrete and steel strains ceases to exist since the concrete strains are concentrated at the end of the crack while the steel strains are distributed over a length at least equal to the horizontal projection of the crack. Thus, a small increase in steel strain will be accompanied by a proportionately larger increase in concrete strain in the compression zone over the crack. When the concrete strains reach a limiting value, crushing occurs, destroying the beam. This type of failure is called a shear-compression failure and is discussed more fully by Sozen (3).

If the prestressing steel becomes unbonded in the neighborhood of the inclined tension crack, the incompatibility of strains is magnified even more since the length over which the steel strains are distributed is increased by the additional unbonded length. In the majority of the beams tested, cracks occurred along the steel in the vicinity of the initiating crack during the interval between initiating cracking and the formation of the major inclined tension crack. In the average beam in this test series a third to a half of the shear span was unbonded by these cracks when failure occurred.

In approximately half of the tests the beam failed when the major inclined crack formed. Although crushing was not usually present at failure, this was partly due to the drop-off in load resulting from the hydraulic loading system used.

4.4 Behavior of Beams With All Draped Wires

Ten beams with all the wires draped were tested (M1 to M6, M10, M12 to M14). Of these, one (M10) had web reinforcement and one (M1) was damaged in casting (see Fig. 2 and Table 2).

The behavior of these beams differed considerably depending on the drape profile. The beams with straight wires and upper kern point drapes represented the extremes in behavior. Except M13, which failed in flexure, and M6 and M14 which had very high drapes, all the beams with all wires draped developed initiating cracks. The principal difference in behavior for the different drapes was the rate at which the major inclined tension crack developed and its location. The number of wires (6 or 8) did not appear to affect the manner of crack development observed in these tests.

The different stages in the behavior of this series of beams are discussed below. Various drape profiles are considered and the behavior is compared to beams M15 and M16 which had straight wires.

(a) I-Beams. For all beams with either straight or draped reinforcement, the first flexural cracks occurred in the flexure span at loads which could be computed on the basis of an uncracked section analysis. Subsequent flexural cracks developed in the shear span. These cracks extended rapidly and vertically to the longitudinal steel. The position of the flexural cracks in the shear span differed from beam to beam; in beams with high drapes the initiating crack occurred farther from the load point than in beams with low drapes. In each location the load at cracking was lower than would be required to crack a similar beam with straight wires.

The behavior of the beam after formation of the initiating cracks was a function of the drape profile. Beams with straight wires had about a 23 percent increase in load in the interval between the formation of the initiating crack and the development of the major inclined tension crack. In these beams (M15 and M16) the major inclined crack crossed the reinforcing steel outside the load point at a distance approximately equal to the over-all depth of the beam from the load. Fig. 17 compares the locations of the initiating cracks and major inclined cracks and the drape angles for beams M2 through M16.

The beams with low drape profiles behaved very similarly to the beams with straight wires. There was a similar increase in load between the initiating crack and the major inclined tension crack, the average increase in load being approximately 25 percent. The major crack occurred at about the same place in these beams as in beams with straight wires. Beam M2 is shown in Fig. 14(b).

For the beams with high drape profiles the vertical crack and the major inclined tension crack appeared simultaneously. Although it was not possible to observe where the crack did start, there are indications that the crack started in the web. The major inclined tension crack crossed the steel at a location about 1.5 times the over-all depth outside the load point.

Five beams with mid-height drapes were tested, two failing in a manner similar to beams with high drapes and three failing in the same manner as beams with low drapes. In the latter, the load increased 6 percent in the interval between the appearance of the initiating crack and the formation of the major inclined tension crack. The major inclined tension cracks crossed the steel at a location about 1.5 times the over-all beam depth outside the load point.

The relative ductility of beams with all wires draped and various drape profiles is indicated by the load-deflection diagrams plotted in Figs. 18 and 19. Fig. 18 shows the behavior of all the test beams with eight wires, all draped. The load deflection curve for beam B4 tested by Billet (5) has been included in Fig. 18 to show the relative ductility of a similar beam failing in flexure. This beam had enough stirrups to ensure a flexural failure.

Fig. 19 presents the load-deflection curves for all the beams with six longitudinal wires including M13 which failed in flexure.

Splitting along the steel was a major factor in the behavior of beams with draped reinforcement. With only three exceptions, all the beams with all wires draped cracked along the tension steel. This was especially serious with the high drape profiles which had draped steel passing through the web of the beam. The rate of splitting depended on the height of drape. Beams with high drapes split faster than beams with low drapes.

According to Sozen (3), an I-beam with a 3-in. web, straight steel, and steel percentages equal to those used in this test series would carry about 10 percent more than the inclined tension cracking load before failing in shear compression. Beams M15 and M16, both of which had straight wires, failed at a load 5 percent greater than the inclined tension cracking load. Only one of the I-beams with all wires draped (M4) developed shear-compression failure. This beam had a lower kern point drape. The increase in load beyond inclined tension cracking for this beam was 8 percent. The average increase for all the beams tested was 5 percent. However, many of the beams failed at the inclined tension cracking load.

(b) Rectangular Beams. Since many of the I-beams initially cracked at the point where the prestressing wires entered the web there was some question as to whether the rapid development of these cracks was due to stress concentrations. For this reason a rectangular beam (M12) was tested. This beam had eight wires draped to mid-height over the support. In testing, one end broke very suddenly leaving the remainder of the beam undamaged. After removing the beam from the testing frame, the cracked portion was removed and the beam was retested on a 64-in. span with a single load at midspan. These beams are referred to as M12 and M12a, respectively. In beam M12a one shear span had draped wires and the other had straight wires. Beams M12 and M12a are shown in Fig. 20. The cracks in beams M12 and M12a are also shown in this figure. The numbers beside the cracks refer to the load increment which caused the crack to extend to the point marked.

Beam M12 developed a flexural crack in the shear span at a very low load. This crack grew very rapidly and caused some splitting. Further loading caused a major inclined tension crack, and failure. At failure there were only two minor cracks in the flexure span of the beam. The load deflection diagram for this beam is included in Fig. 18.

Beam M12a took more load than M12 before cracking. The first cracks were flexural cracks under the loading beam. The first crack in either shear span was a flexural crack in the draped span. This crack initially rose as high as the steel. With further loading the crack progressed toward the loading block, splitting occurred and a major inclined tension crack developed. Failure was by crushing of the compression zone under the load. At failure there were no shear cracks of any sort in the shear span with straight wires.

4.5 Behavior of Beams with Both Straight and Draped Wires

(a) I-Beams with 3-Inch Web. Beams M8, M9 and M11 had 3-in. thick webs and drape profiles X, Y and Z, respectively (see Table 2 and Fig. 2). M9 had a small steel percentage and failed in flexure. This beam has been discussed in Section 4.2. Beam M11 had web reinforcement.

The first cracks in both M8 and M11 were flexural cracks in the constant moment region. These were followed by initiating cracks in the shear spans.

Beam M8 failed suddenly due to a major inclined tension crack which occurred at a load about 12 percent greater than the load causing the initiating crack. The major crack may not have been affected by the initiating crack since it crossed the steel about a foot from the initiating crack. There was no splitting along the reinforcement in beam M8 prior to the formation of the major inclined tension crack. This crack did split along the steel, however. The load deflection curve for this beam is shown in Fig. 21.

The behavior of M11 was affected by the presence of the stirrups. The beam split a short distance along the steel during the interval between the formation of the initiating crack and the major inclined crack. A small amount of crushing occurred above one of the initiating cracks before the major inclined crack formed. After crushing started, more load was applied before the major inclined tension crack developed. Both the cracking and splitting may have been affected favorably by the web reinforcement. The load deflection curve for beam M11 is shown in Fig. 22.

(b) I-Beam with 1 3/4-Inch Web. Beam M7 was the only specimen with a 1 3/4-in. thick web. As such, it was the only beam likely to have the initial inclined tension crack develop in the web since this type of cracking only occurs in beams with small ratios of web to flange thickness.

The first crack in this beam was an inclined tension crack which originated at the bottom of the web, spread rapidly across the web toward the load point and also caused splitting along the steel toward the support (see Fig. 16). The location of this crack coincided approximately with the point where the draped prestressing wires entered the web. At inclined cracking, the tension stress at the bottom of the beam was less than the modulus of rupture and flexural cracking had not yet occurred anywhere in the beam. The load deflection diagram for beam M7 is shown in Fig. 21. The extremely brittle nature of this failure is clearly evident from the shape of the load deflection diagram.

4.6 Behavior of Beams with Web Reinforcement and Draped Wires

Two beams with stirrups and draped wires were tested. These beams, M10 and M11, were the same size and had the same percentage of longitudinal reinforcement as beams G19 and G30 tested by Hernandez (4). The size, shape, and spacing of the stirrups were the same in all four beams. The stirrups in M10 and M11 were offset 2 1/2 in. from their position in G19 and G30 to prevent interference with the draping saddles. The other properties are listed in the following table:

Mark	f'_c	Drape	Failure	Cracking Load		Ultimate
				Predicted	Test	Load
G19	2900	-	F	19.1	20.0	23.8
M11	4000	Z	S-C	20.8	20.4	24.6
M10	4200	C	S-C	19.1	17.4	18.3
G30	5500	-	F	20.9	25.2	27.8

Both beams with straight reinforcement failed in flexure, the compression zone crushing over a crack in the middle of the flexure span. In both cases, the deflections were quite large when the beam failed. Figure 22 shows the load-deflection curves for G19, G30, M10 and M11. As would be expected from its higher concrete strength, G30 carried considerably more load at failure than G19 and its deflections were greater also.

Since the concrete strengths of M10 and M11 fall half-way between those of G19 and G30, it could be expected that if flexural failures occurred in M10 and M11, the behavior of these beams would also be intermediate between those of G19 and G30. Beam M10, which had a mid-height drape (profile C), failed in shear-compression at a load less than that required to develop inclined tension cracks in either G19 and G30. Beam M11 cracked at a higher load than M10 but also failed in shear-compression.

The initiating crack in M10 crossed the steel about 8 in. outside the load point and subsequently developed into the inclined tension crack. In M11 the first crushing occurred over the initiating crack but, with additional loading, a major inclined crack formed. Splitting occurred at the level of the steel in both beams in the interval between the formation of the initiating crack and the first crushing.

V. DISCUSSION OF TEST RESULTS

5.1 Basis for Quantitative Comparison of Observed Inclined Tension Cracking Loads

The inclined tension cracking loads obtained in this test series have been compared on the basis of an empirical expression derived by Sozen (3) for the inclined cracking load of similar prestressed beams with straight wires. By using such a formula it was possible to eliminate the effect of different concrete strengths and different levels of prestress in comparing the effects of draped wires. The expression used was derived from the results of 99 tests of prestressed concrete beams failing in shear.

Sozen found the most simple and consistent relationship representing all the above-mentioned test data to be the following expression:

$$\frac{M'_c}{f_t b d^2 \sqrt{\frac{b'}{b}}} = 1 + \frac{F_{se}}{A_c f_t} \tag{2}$$

where:

M'_c = moment at inclined tension cracking defined as the product of the applied shear at the inclined tension cracking load and the length of the shear span.

f_t = assumed tensile strength of concrete = $\dfrac{1000}{2 + \dfrac{6000}{f'_c}}$ for small-sized coarse aggregates = two-thirds of the modulus of rupture from Eq. (1).

b = top flange width

b' = web thickness

d = effective depth

F_{se} = effective prestress force

A_c = gross area of cross section

The dominant variables in the beams reported by Sozen were assumed to be the geometry of the cross section, the ratio of the shear span to the effective depth of the beam, the compressive stress exerted by the prestressing force, and the tensile strength of the concrete. These factors are all included in the terms of Equation (2).

Equation (2) was originally derived from a consideration of the principal tensions in the web of a prestressed beam. It may also be reduced to a form resembling the expression for the flexural cracking load of a prestressed beam. An attempt to derive a more "rational" expression for the cracking load was made in the course of the investigation reported herein but it was not possible to express the complex development of an inclined tension crack in terms of a single rational formula. This was especially true when considering inclined tension cracks influenced in their development by an initiating crack [see Section 4.3(a)]. It was decided, therefore, that Eq. (2) would be used to compare the results of the beams discussed in this report.

The average ratio of measured to predicted inclined cracking load was 1.00 and the mean deviation was 0.074 for the 99 beams used to derive Eq. (2). For the six beams which had properties similar to those of beams M1 through M16, the average ratio of measured to predicted cracking load was 0.98 with maximum and minimum values of 1.07 and 0.91. The average value of this ratio for beams with straight wires and web reinforcement was 1.13 (4). The ratios of the cracking loads for beams M2 through M16 to the loads predicted by Eq. (2) are shown plotted against the angle of inclination of the draped wires in Fig. 23. The trend of these data suggests a ratio of measured to predicted cracking load somewhat greater than 1.0 for beams with straight longitudinal reinforcement. This was further borne out by the observed cracking loads for beams M15 and M16, both of which had straight wires. Inclined tension cracks developed in both shear spans of M15 at loads equal to 1.08 and 1.11 times the predicted cracking load. One crack developed in M16 at a load equal to 1.03 times the predicted load. These results lie well within the scatter of the original data used to derive Eq. (2), however. Since Eq. (2) is only used as a means of comparing the relative effects of draped wires on the cracking loads, the trends observed should be consistent.

5.2 Effect of Draped Reinforcement on Strength of Test Beams at Inclined Cracking

The main objective of this series of tests was to determine the effect of draped reinforcement on the shear strength of prestressed concrete beams. If web reinforcement is not provided in a beam failing in shear, the usable ultimate strength of the beam is the load causing inclined tension cracking. For this reason it is important to investigate the inclined tension cracking strength of beams with draped steel.

Figure 23 compares the externally applied load at inclined cracking with the predicted cracking load from Eq. (2) for various angles of drape. The measured and computed shears at cracking are listed in Table 5. For each beam, the angle of drape plotted in this figure is the angle between the center of gravity of the reinforcement and the horizontal. The scatter in the test results is especially noticeable with small drape angles. In the previous section, the scatter for beams with straight wires was mentioned. Four points have been plotted on the $\varphi = 0$ deg. ordinate, representing from bottom to top: The average ratio of measured over predicted inclined cracking load for six of Sozen's beams which fell within the range of variables considered in this test series, and the cracking load ratios for M16 and the two ends of M15.

The test results plotted in Fig. 23 indicate that there is a reduction in the inclined-tension cracking load of a prestressed beam as the drape angle of the longitudinal reinforcement is increased. At low drape angles there is no definite reduction within the scatter band. However, in no case can it be said conclusively that draping the reinforcement has increased the inclined tension cracking load.

The solid line has been fitted to the test results by observation. The two dotted lines represent plus and minus five percent change in load from the solid line. The majority of the test results fall within this band. Two of the points plotted in Fig. 23 fall significantly outside this range, however. The first of these, corresponding to a drape angle of 1.9 deg., represents beam M7 which developed its first inclined crack in the web. This crack was probably caused by stress concentrations in the web of the beam at the point where the prestressing wires entered the web. In the other beams plotted on this curve, initiating cracks formed prior to the formation of the major inclined tension crack. Since the cracking mechanism in M7 differed from that in all other beams, it is reasonable to expect that the effect of draped wires on the strength of M7 would be different from the effect of such wires on the development of inclined cracks starting from initiating cracks. The other low point in Fig. 23, plotted at 6.45 deg., represents the first test on beam M12. It is likely that there was low strength concrete in the region of this beam where the inclined crack started. The other end of this beam was subsequently tested and it developed an inclined crack at a load comparable to other beams with this drape profile.

Draping the wires at angles less than about 2 deg. appeared to have little effect on the shear strength of the beams tested. For drape angles less than about 2 deg., the draped wires enter the web only in the outer ends of the shear span, if at all. Since splitting along the reinforcement is less likely to occur if all the wires are in the lower flange at the point where the initiating crack crosses the longitudinal steel, the initiating crack probably has less effect on subsequent cracking for drape angles less than 2 deg.

So far, the effect of draped reinforcement has been considered only in relation to the total applied shear at a section. Since the reinforcement itself provides an upward component of force, shear stresses in the concrete do not carry the entire applied shear. Before loading, a beam with draped wires has an upward or negative shear acting in each shear span as a result of the draped reinforcement. The shear stresses in the shear span become zero after an increment of shear equal to the upward component of the prestress is applied to the beam. Further loading will cause positive shear stresses in the shear span. Therefore, the effective or "net" shear force acting in each shear span at inclined cracking may be stated as:

$$V_n = V_c - V_d \tag{3}$$

where:

V_n = net shear or effective shear at inclined cracking,

V_c = total applied load shear at inclined cracking,

V_d = upward component of prestressing force.

In computing shear stresses and principal stresses, only the net shear need be considered. If the net shear strength of a beam is independent of the position of the prestressing force, it may be concluded that draping the reinforcement will add to the shear strength of a prestressed beam. It is worthwhile therefore, to investigate the variation in net shear strength as the angle of drape is varied.

Figure 24 compares the net shear at inclined tension cracking with the cracking shear predicted by Eq. (2) for various drape angles. Such a plot isolates the effects of the steel position on the cracking strength of the section. The points plotted on this curve emphasize the marked reduction in the net shear strength for beams with draped reinforcement. When the prestressing wires in the test beams were draped through the upper kern point at the support (φ = 10 deg.), the resulting net shear strength was only 30 percent of that for a beam with straight wires. This trend would indicate that the portion of the shear strength provided by the beam itself, over and above the strength provided by the upward components of the prestress force would be negligible for beams with the center of gravity of the steel raised at such unlikely angles as 12 to 14 deg. Such a reduction in strength is reasonable however, because of the reduction in flexural cracking load in such a beam.

As the wires in a prestressed beam are draped, the eccentricity of the reinforcement is decreased at every section in the shear span and the prestressing force is less effective in resisting flexural tensions at the bottom of the beam. Therefore, initiating cracks should form at lower loads in beams with draped wires than in beams with straight wires. In the beams tested, however, the location of the initiating cracks also varied, with the result that when the drape was increased there was actually an increase in the loads causing the initiating crack. As shown in Fig. 17, there was an increase in the distance between the load point and the point where the initiating crack crossed the reinforcement as the angle of drape was increased.

Figure 25 illustrates the relationship between the angle of drape and the different stages in the behavior of a test beam. The curves on this figure are based on a concrete strength of 3000 psi, a longitudinal reinforcement ratio of 0.003, and a prestress level of 113,000 psi. These values were chosen to represent the majority of the beams tested. Curve A representing the inclined cracking load is taken directly from Fig. 23. Curve B representing the ultimate load is taken from the test results. The location of the initiating crack used in computing the load causing this crack has been based on the lower curve on Fig. 17. As explained above, the change in position of the initiating crack causes an increase in the cracking load with increase in drape angle. This trend is illustrated by the Curve C on Fig. 25. As the angle of drape increases, however, the effect of the decrease in eccentricity becomes more important, causing a decrease in the load corresponding to the initiating crack.

For low drape angles there is a considerable increase in load between the formation of the initiating crack (C) and the inclined cracking load (A). After inclined cracking, there is a further increase in load before the ultimate load (B) is reached. Fig. 25 indicates that the inclined crack will occur before the initiating crack for drape angles larger than about seven degrees. For such beams the inclined crack will start in the web. For high drape angles, inclined cracking causes immediate failure of the beam.

The two beams with very high drapes, M6 and M14, developed what appeared to be initiating cracks at 90 and 91 percent of the expected load. These cracks developed so rapidly, however, that it was not possible to observe whether they started in the bottom flange or were a downward extension of an inclined crack originating in the web. The cracks in M6 and M14 crossed the reinforcement at about the same distance from the load point in both beams. The location of these cracks did not agree with the linear trend for location of the initiating crack observed in beams with smaller drape angles and shown in Fig. 17. On the other hand, the cracks approximately followed the principal stress trajectories computed for the cracking load using an elastic analysis. Both these beams had drape angles large enough to develop the initial inclined crack before an initiating crack developed (see Fig. 25). It appears therefore, that in the beams with high drape angles, such as beams M6 and M14, the inclined tension crack will start in the web, probably in the vicinity of the prestressing wires, before initiating cracks can form.

For drape angles less than seven degrees, the increment of load carried by the beam between the formation of the initiating crack and inclined tension cracking decreases as the angle of drape is increased. A part of this decrease may be traced to the effect of the position of the wires on the speed at which the initiating crack develops.

After the initiating crack forms, the tension force which was carried by the concrete before cracking is transferred to the reinforcing steel. To carry this force, the longitudinal wires must develop a certain strain which in turn causes an angle change at the cracked section. The magnitude of the steel strain and the depth to the steel govern the height of crack which develops. Also, the higher the position of the steel in the beam, the larger the steel stress must be to preserve moment equilibrium at the crack. Thus the initiating crack grows rapidly to a greater height in a beam with draped reinforcement than in a beam with straight reinforcement since the centroid of the draped steel is higher in the beam. The higher the initiating crack extends, the more effect it has on the formation of an inclined tension crack. Therefore, when the reinforcement is draped, the increment of load between the formation of the initiating crack and the development of the inclined tension crack is reduced.

The rate at which the beam splits at the level of the steel is also a function of the height of drape. Before cracking, the tensile strains in the reinforcement are smaller if the reinforcement is close to the neutral axis of the beam than if the reinforcement is in its normal position. After cracking, the stress required for moment equilibrium varies inversely as the depth of the steel below the top of the beam. Thus, in a beam with draped wires there is a greater difference in steel stress before and after cracking than in a comparable beam with straight wires. This increased stress difference between the cracked and the uncracked part of the beam results in a larger bond stress adjacent to the crack. In addition, if the wires pass through the web there is a definite plane of weakness along which splitting can develop.

In beams with small ratios of web to flange thickness the first inclined tension cracking occurs in the web as a result of principal tensions. Only one beam in the test series (M7) had a web thin enough to have this type of cracking.

If the principal tensions in the web of a prestressed I-beam are computed by the normal methods of uncracked sections, it is seen that draping the reinforcement reduces the tension stresses in the web. The computed principal tensions acting at points along the juncture of the web and lower flange are shown in Fig. 26 for beam M7 and for a similar beam with straight wires. These values were computed for the inclined cracking load of M7 and include an allowance for the effect of local vertical stresses at the loads, reactions, and draping saddles. The lower curve on Fig. 26 represents the principal tensions for beam M7. The upper curve represents the same stresses for a similar beam with straight wires. The computed tensile strength of the concrete is about 235 psi for beam M7. These curves indicate that draped wires reduce the maximum principal tensions in the web of a prestressed beam. In beams with web to flange thickness ratios small enough to have the initial crack start in the web, the cracking load will be increased by draping the reinforcement unless the draped reinforcing passes through the web of the beam.

If the draped wires enter the web in the shear span, the stress-concentrations introduced by shear transfer at the level of the steel or by the reduction of the concrete section may be sufficient to raise the principal tensions at that location to a critical value. In M7 the inclined crack originated approximately at the point where the reinforcing wires entered the web (see Fig. 16). The load at inclined cracking was 92 percent of that predicted by Equation (2). Figure 26 indicates that the computed principal tensions at the point where the inclined crack started are only 60 percent of the maximum principal tensions in a comparable beam with straight wires.

A beam with straight wires may fail in shear by "secondary inclined tension cracking" if the prestress is high and the web is relatively thin. Secondary inclined tension cracking occurs at the juncture of the web and upper flange near the support, causing a very sudden failure. This type of failure should not occur in beams with draped steel since the maximum effect of draped wires comes near the end of the beam where this type of cracking occurs. Figure 27 compares the maximum principal tension forces at the top of the web near the support for beam M7 and for a similar beam with straight wires. In this case the maximum tensile stress is reduced 26 percent by draping. However, although none of the test beams with draped reinforcement exhibited this type of failure, similar beams with straight reinforcement would not be expected to fail by secondary inclined tension cracking either.

To summarize, there was a reduction in the inclined cracking load when the angle of inclination of the prestressing steel was increased. On the basis of net shear, this reduction was severe. The decrease in cracking load appears to be caused primarily by the change in the eccentricity and depth of the longitudinal force when the wires are draped. In beams with draped wires, the cracks must extend higher into the beam before equilibrium is restored than would be necessary with straight wires. This affects the amount of load which can be applied between formation of the initiating crack and formation of the inclined tension crack, and between inclined tension cracking and failure. The bond stresses are also increased and may cause splitting. When the major crack forms, it generally follows a stress trajectory which intersects the top of the web under the load. If the web of a beam is thin with respect to the flange thickness, inclined tension cracking may originate in the web as a result of principal tensions. If the steel is draped in such a beam, the principal tensions in the web are lowered unless the draped wires enter the web, in which case the stress concentrations introduced may be sufficient to cause premature cracking in the web. Secondary inclined tension cracks should not occur in beams with draped wires since the tensile stresses at the top of the beam at the support are reduced considerably by draping the reinforcement.

5.3 Effect of Draped Reinforcement on Ultimate Strength of a Prestressed Beam

The increment of load that can be applied to a prestressed beam between inclined tension cracking and failure decreases as the wires are draped since the rotations necessary to preserve moment equilibrium are greater if the reinforcement is draped and since splitting at the level of the steel is more likely. The observed increase in load after inclined tension cracking has been plotted for each beam in Fig. 28. The observed and computed ultimate moments are listed in Table 6. There is a decrease in the load increment after cracking as the angle of inclination of the wires is increased. The combined effect of the decrease in cracking load and the decrease in load capacity after inclined cracking is indicated in Fig. 25.

When an inclined tension crack develops in a prestressed beam with draped wires, the upward component of the prestress carries some of the shear at the cracked section. This affects the development and appearance of the major inclined crack. After inclined cracking, the shear force in the compression zone of the beam at the cracked section is only the net shear. When the load on the beam is increased, only the increase in the vertical component of the steel stress can be assumed to carry a portion of the increased load.

In most cases the beams tested split along the prestressing wires at the time the major inclined crack formed. Since splitting at the level of the steel tends to minimize the increase in steel stress demanded by a shear-compression failure as discussed in Section 5.2, the stress in the draped wires increases very little and they can carry very little additional shear after cracking occurs.

A shear-compression failure is caused by excessive rotations at the section of the inclined crack. Nearly all the changes in behavior caused by draping the reinforcement in a beam tend to increase the angle changes at the cracked section and therefore tend to decrease the shear-compression strength. Raising the centroid of the steel and the unbonding that is caused by splitting at the level of the steel both have this effect. The premature development of inclined cracking observed in these tests merely lowers the load at which the concentration of strain is initiated.

When web reinforcement is added to a beam with draped reinforcement, the inclined cracking load is not noticeably affected. After cracking, however, the web reinforcement restrains the development of splitting, thus decreasing the length of the tension reinforcement over which the steel strains at the crack are distributed. At the same time it provides a means of transferring shear force across the crack and tends to restrain rotation at the cracked section.

According to Hernandez (4), the amount of web reinforcement required to prevent shear failure in a prestressed beam with straight wires is proportional to the difference between the inclined cracking load and the flexural failure load. The theoretical and actual web reinforcement ratios for the beams G19, G30, M10 and M11 and the test results are summarized in the following table (see also Section 4.6):

Mark	$r = A_v/b's$ Actual	Balanced	$\frac{\text{Actual}}{\text{Balanced}}$	$\frac{M_{test}}{M_{flexure}}$	$\frac{M_u - M_c}{M_f - M_c}$	Failure
G19	0.280	0.203	1.38	0.93	0.97	F
M11	0.280	0.258	1.08	0.89	0.65	S
M10	0.280	0.329	0.85	0.67	0.13	S
G30	0.280	0.306	0.95	0.98	0.84	F

The values tabulated as "balanced" web reinforcement ratios were computed using Equation (2) to predict the inclined cracking loads. If the actual cracking loads were used to compute balanced web reinforcement ratios, beams M10 and M11 need 0.406 and 0.274 percent, respectively. These values represent increases of 23 and 6 percent, respectively, over the requirements computed using the cracking load predicted by Equation (2).

The values tabulated for $M_u - M_c/M_f - M_c$ compare the actual increase in load after cracking to that which would occur if the beam failed in flexure. Since this comparison involves the differences of large similar numbers, the normal experimental scatter is greatly magnified. This is especially true of G30. The values tabulated have been computed for the actual cracking loads observed. A comparison such as this does not give a true indication of the efficiency of the web reinforcement since the balanced web reinforcement ratio depends on the cracking load. This type of comparison shows the relative effects of web reinforcement on the failure loads of beams of varying properties.

The tests of M10 and M11 showed that the beams with draped wires required more web reinforcement than similar beams with straight wires. Indeed, although M11 had more web reinforcement than the amount required, computed on the basis of the actual cracking load, it only developed 58 percent as much load after cracking as was theoretically possible. It is evident that the web reinforcement was not as effective in these beams as in beams with straight wires. This may be attributed to the increased amount of rotation which occurs at the cracked section in a beam with draped wires. The increased rotations result from the position of the reinforcement and the splitting at the level of the steel as explained in Section 5.2. Thus, the relationships derived by Hernandez (4) would not apply to beams with draped reinforcement even if it were possible to predict the cracking load for such beams.

To summarize, the increment of load that can be applied to a prestressed beam between inclined tension cracking and failure decreases as the wires are draped. Since the wires are higher in the beam, and since splitting at the level of the steel is more likely, more steel strain and thus more concrete strain is necessary to preserve moment equilibrium after an inclined crack forms. For this reason, web reinforcement which would have been enough to ensure a flexural failure in a beam with straight wires is not adequate to restrain the rotations occurring at an inclined crack in a beam with draped wires. More web reinforcement is necessary to prevent shear failures in a beam with draped wires than in a beam with straight wires.

VI. SUMMARY

The object of the tests described in this report was to determine the effect of draped reinforcement on the shear strength of prestressed concrete beams.

Sixteen pretensioned beams with varying degrees of drape were tested. These beams were 6 by 12 in. in cross-section with an overall length of 10 ft. Fifteen of the specimens were I-beams, fourteen with 3-in. webs, and one with a 1 3/4-in. web. One beam was rectangular in cross-section. The concrete strengths varied from 2600 to 4200 psi, the longitudinal steel ratios varied from 0.197 to 0.403 percent and the effective prestress from 84,200 psi to 129,500 psi, averaging about 110,000 psi. Two beams were provided with sufficient web reinforcement to ensure flexural failures in similar beams with straight wires.

The principal variable in the test series was the profile of the prestressing steel. Nine different profiles were used. Five profiles had all the wires draped at the same angle in both shear spans, three had some of the wires straight and some draped, and the remaining profile had straight wires only. In every case, the wires were bent at the load points, the profiles being made up of straight line segments. Six different drape angles were used.

The beams were tested to failure with loads applied at the third-points of a 9-ft span. Records of load, deflection, concrete strain at the top of the beam, steel strain, and crack pattern were obtained throughout all stages of loading.

Thirteen of the test beams failed in shear, two failed in flexure, and one was damaged in casting. The flexural failures were typical of moderately under-reinforced beams; the concrete crushed in the compression zone above a vertical crack in the constant moment region of the beam and the steel strain was well into the inelastic range.

All the beams which failed in shear developed inclined tension cracks which caused the shear failure. Most commonly, the inclined tension cracks developed adjacent to, or as an extension of flexural cracks in the shear spans. In three of the beams, the inclined tension crack originated in the web of the beam, before initiating flexural cracks had formed. In one beam the inclined tension crack in the web was the first crack in the beam. In general, the type of inclined cracking observed depended on the geometry of the beam and the location of the wires in the section. The beams that initially cracked in the web had either a thin web or a high drape profile.

As the angle of inclination of the prestressing steel was increased, there was a reduction in the inclined cracking load. At low drape angles, there was no definite reduction. However, in no case could it be said conclusively that draping the reinforcement increased the inclined cracking load.

In the majority of the beams with draped wires, splitting occurred parallel to the steel immediately before the inclined crack developed, destroying the bond over at least a third of the length of the shear span. These cracks started from the initiating crack and extended along the steel toward the end of the beam. This splitting was especially serious when the wires passed through the web. Splitting also occurred in the rectangular beam.

Generally, the beams with high drape angles failed suddenly at the inclined cracking load. The other beams failing in shear carried only from one to nineteen percent more load after the inclined tension crack developed. Only two of these beams carried enough load to develop a shear-compression failure. The two beams with web reinforcement also failed in shear-compression and were more ductile than similar beams without web reinforcement. The trend of the test results showed a reduction in the load increment after cracking as the drape angle increased.

Web reinforcement did not affect the inclined cracking load in the two beams with web reinforcement, and did not prevent shear failures. These tests indicated that web reinforcement was less effective as the angle of inclination of the wires was increased.

In general, it was concluded that draping the longitudinal wires did not increase either the inclined cracking load or the ultimate shear strength of the prestressed concrete beams tested. Instead, the trend of the test results indicated a reduction in both the inclined cracking load and the ultimate strength of the beams with draped wires.

VII. BIBLIOGRAPHY

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TABLE 1

PROPERTIES OF BEAMS

Mark	Concrete Strength f'_c psi	Flange Width b in.	Web Thickness b' in.	Effective Depth* d in.	Area of Steel A_s sq. in.	Longit. Reinf. p %	Effective Prestress f_{se} ksi	Drape Profile**	Wire Lot
M1	3180	6.05	2.95	10.05	0.242	0.398	106	B	XI
M2	2980	6.00	2.90	10.10	0.242	0.400	109	B	XI
M3	3090	6.05	3.00	10.10	0.242	0.395	84	C	XI
M4	3850	6.00	3.00	10.10	0.181	0.298	113	B	XI
M5	2610	6.05	2.95	10.10	0.181	0.297	110	C	XI
M6	3160	6.00	3.00	10.10	0.181	0.298	120	E	XI
M7	2660	6.00	1.75	10.22	0.181	0.296	109	X	XI
M8	2720	6.05	3.00	10.22	0.181	0.293	114	X	XI
M9	3440	6.05	2.90	10.10	0.121	0.197	112	Y	XI
M10	4000	5.95	2.95	10.10	0.242	0.403	113	C	XI
M11	4200	5.95	2.95	10.10	0.242	0.403	129	Z	XI
M12	2700	6.00	6.00	10.15	0.242	0.398	106	C	XI
M13	4230	6.00	3.00	10.10	0.181	0.300	122	A	XI
M14	4210	6.00	3.00	10.10	0.181	0.300	109	D	XI
M15	3090	6.05	3.10	10.30	0.181	0.290	115	S	XII
M16	3000	6.05	3.00	10.00	0.242	0.399	114	S	XII

* Measured in flexure span.

** Drape profiles are described in Table 2.

TABLE 2

PROPERTIES OF DRAPE PROFILES

Drape Profile*	Mark	No. of Wires	No. of Draped Wires	Height of Center of Gravity of Steel: At Support, Draped Wires in.	Height of Center of Gravity of Steel: At Support, All Wires in.	Height of Center of Gravity of Steel: At Midspan, All Wires in.	Draped Wires deg.	φ** All Wires deg.
A	M13	6	6	2.96	2.96	2.0	1.53	1.53
B	M2	8	8	3.5	3.5	2.0	2.38	2.38
(Lower Kern)	M4	6	6	3.4	3.4	2.0	2.22	2.22
C	M3	8	8	6.07	6.07	2.0	6.45	6.45
(Mid-Height)	M5	6	6	5.96	5.96	2.0	6.28	6.28
	M10	8	8	6.07	6.07	2.0	6.45	6.45
	M12	8	8	6.07	6.07	2.0	6.45	6.45
D	M14	6	6	7.8	7.8	2.0	9.13	9.13
E (Upper Kern)	M6	6	6	8.33	8.33	2.0	9.95	9.95
X	M7	6	4	4.14	3.04	1.87	3.42	1.88
	M8	6	4	4.14	3.04	1.87	3.42	1.88
Y	M9	4	2	4.61	3.07	2.0	3.92	1.70
Z	M11	8	4	6.07	4.04	2.0	6.75	3.25
S	M15	6	-	----	1.87	1.87	----	----
(Straight)	M16	8	-	----	2.0	2.0	----	----

* Profiles A, B, C, D and E have only draped wires. Profiles X, Y and Z comprise both straight and draped wires. Profile S designates straight reinforcement only.

** Angle of inclination of the prestressing force in the shear span.

TABLE 3

SIEVE ANALYSIS OF AGGREGATES

Percentages Retained

Aggregate Lot		I	II	III	IV	V
	Sieve					
Gravel	1/2"	3.1	0.7	0.7	2.0	0
	3/8"	24.5	12.5	12.5	23.2	31.4
	No. 4	97.8	97.2	97.2	93.4	88.0
	No. 8	98.6	98.5	98.5	97.4	96.6
	No. 16	----	98.7	98.7	98.0	98.6
Sand	No. 4	2.7	0.3	0.2	0.2	0.3
	No. 8	19.7	13.7	13.2	13.2	14.1
	No. 16	41.5	38.4	36.2	36.2	37.1
	No. 30	70.5	70.2	64.4	64.4	66.2
	No. 50	94.9	92.5	90.4	90.4	90.2
	No. 100	98.6	98.2	98.3	98.3	98.2
	Fineness Modulus	3.28	3.13	3.03	3.03	3.06

TABLE 4

PROPERTIES OF CONCRETE MIXES

Mark	Cement:Sand:Gravel by weight	Water/Cement by weight		Slump in.		Compressive Strength f'_c psi		Modulus of Rupture f_r psi		Age at Test Days	Aggregate Lot
Batch	1 and 2	1	2	1	2	1	2	1	2		
M1	1:4.20:4.56	.91	.88	5	2	3180	3290	516	558	16	I
M2	1:4.20:4.51	1.00	1.00	3	2	2980	2870	491	491	8	I
M3	1:4.27:4.60	.81	.81	2	2	3090	3800	416	375	8	I
M4	1:4.27:4.60	.79	.79	1.5	2	3850	3400	442	416	9	II
M5	1:4.17:4.55	.92	.92	1.0	1.0	2610	2610	375	400	8	II
M6	1:4.24:4.60	.84	.84	1.5	1	3160	3460	383	392	8	II
M7	1:3.83:4.25	.91	.94	7	3	2660	2560	417	420	6	III
M8	1:4.25:4.56	.79	.79	3	2.5	2720	2700	404	350	8	III
M9	1:3.87:4.12	.87	.87	1.5	---	3440	3960	450	458	7	IV
M10	1:3.92:4.20	.74	.76	1	1.5	4000	4210	412	446	7	IV
M11	1:3.93:4.17	.82	.82	2	3	4200	4020	346	350	20	IV
M12	1:4.21:4.58	.91	.91	1.5	2	2700	3260	300	282	12	IV
M13	1:4.01:4.36	.77	.78	2	2	4230	3320	457	367	8	V
M14	1:4.05:4.30	.78	.78	2	1.5	4210	3870	337	310	15	V
M15	1:4.07:4.42	.79	.79	1	1	3090	2640	340	275	8	V
M16	1:4.07:4.42	.79	.79	1	1	3000	2890	358	358	7	V

TABLE 5

SHEAR AT INCLINED CRACKING LOAD

Mark	Computed Cracking Shear* V'_c kips	Observed Cracking Shear V_c kips	$\frac{\text{Observed } V_c}{\text{Computed } V'_c}$	Drape Shear $A_s f_{se} \sin\varphi$ V_d kips	Net Shear V_n kips	$\frac{V_n}{V'_c}$	Drape Profile
M2	8.85	9.95	1.12	1.06	8.89	1.01	B
M3	7.65	7.45	0.93	2.28	5.17	0.675	C
M4	8.00	8.95	1.12	0.80	8.15	1.02	B
M5	7.30	6.49	0.89	2.18	4.31	0.59	C
M6	8.25	6.38	0.77	3.75	2.63	0.32	E
M7	6.05	5.45	0.90	0.65	4.80	0.79	X
M8	7.60	7.90	1.04	0.68	7.22	0.95	X
M9**	6.52	----	----	----	----	----	Y
M10	9.55	8.70	0.91	3.08	5.62	0.59	C
M11	10.40	10.20	0.98	1.78	8.42	0.81	Z
M12	10.20	6.55	0.64	2.88	3.67	0.36	C
M12a	11.50	9.45	0.82	2.88	6.57	0.57	C
M13**	8.50	----	1.18+	0.59	----	1.10+	A
M14	7.95	5.60	0.71	3.13	2.47	0.31	D
M15	7.88	8.49	1.08	----	8.49	1.08	S
M16	8.95	9.21	1.03	----	9.21	1.03	S

* From Equation (2) for a beam with straight reinforcement.

** Flexural Failure.

TABLE 6

MEASURED AND COMPUTED MOMENTS

Mark	Observed Cracking Moment M_c kip-in.	Observed Ultimate Moment M_u kip-in.	Flexural Moment Capacity M_f kip-in.	$\frac{M_u}{M_c}$	$\frac{M_u}{M_f}$	Mode of Failure	Drape Profile
M2	358	358	485	1.00	0.74	S	B
M3	268	320	483	1.19	0.66	S	C
M4	322	348	371	1.08	0.94	S	B
M5	234	234	380	1.00	0.62	S	C
M6	230	230	385	1.00	0.60	S	E
M7	196	196	380	1.00	0.52	S	X
M8	284	318	382	1.12	0.83	S	X
M9	---	230	274	---	0.84	F	Y
M10	313	330	502	1.05	0.66	S	C
M11	368	443	497	1.21	0.74	S	Z
M12	235	235	481	1.00	0.49	S	C
M12a	303	306	481	1.01	0.64	S	C
M13	---	360	389	----	0.93	F	A
M14	202	202	400	1.00	0.50	S	D
M15	305	322	376	1.06	0.81	S	S
M16	332	348	475	1.05	0.68	S	S

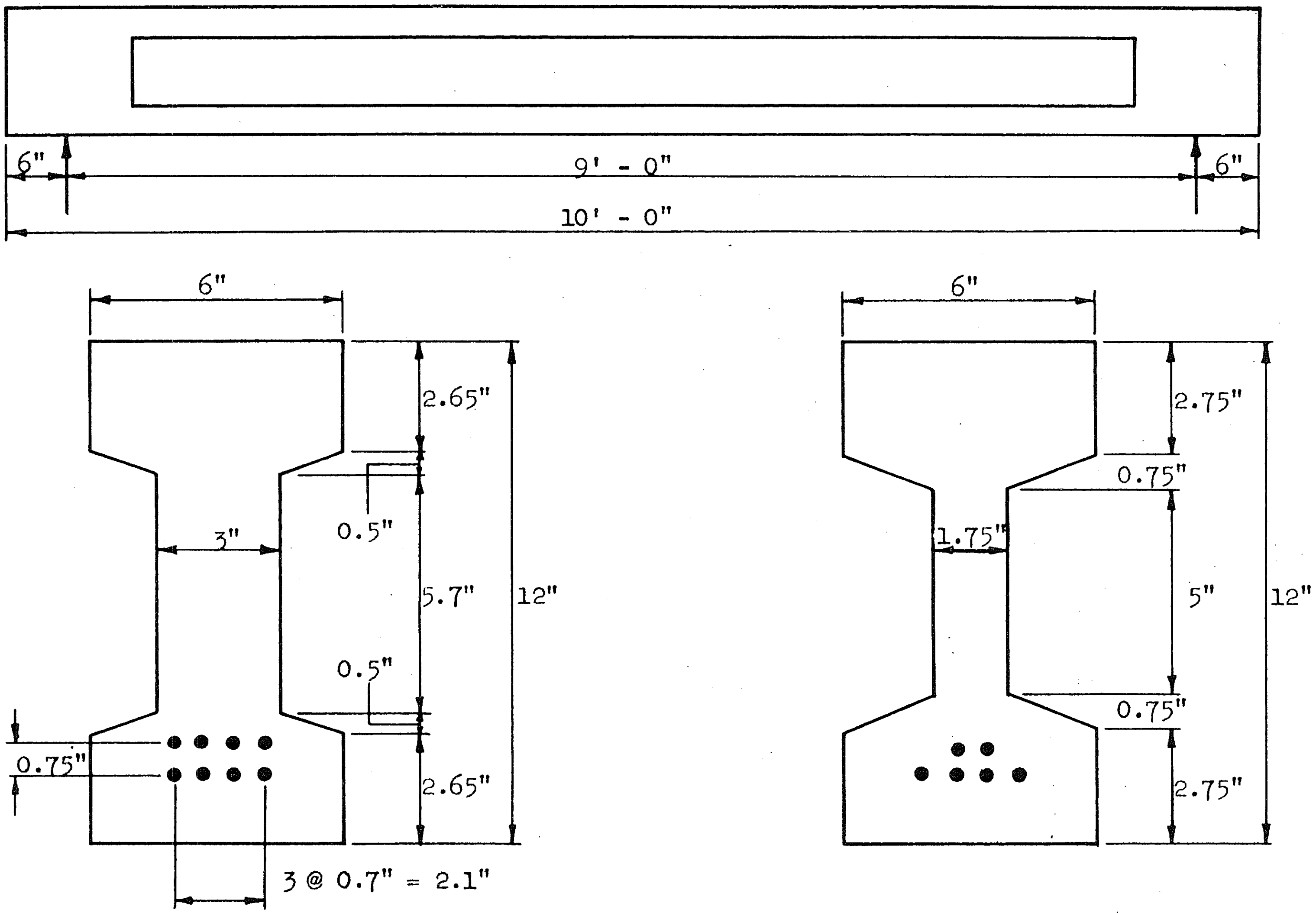


FIG. 1 - NOMINAL DIMENSIONS OF I-BEAMS

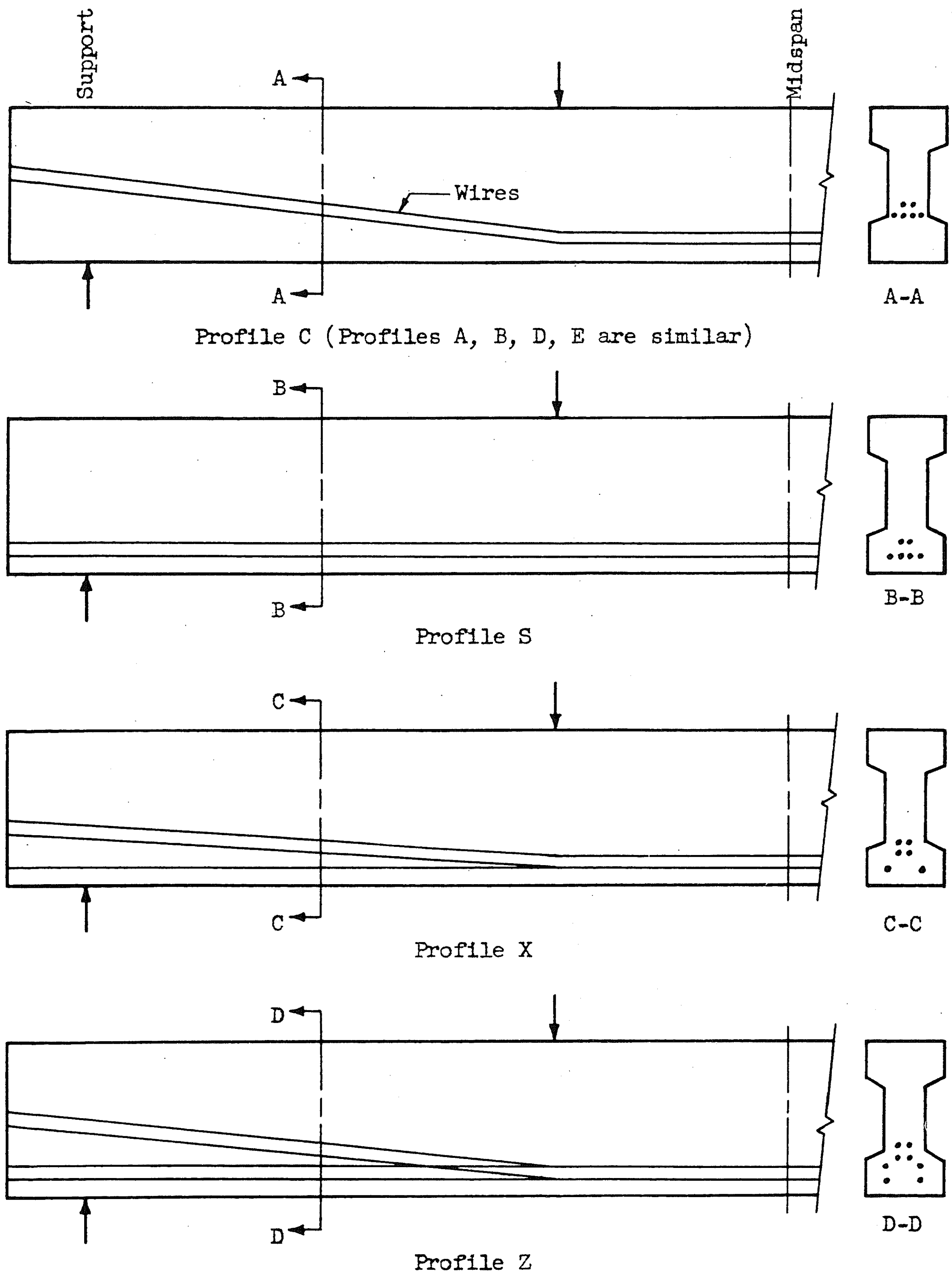


FIG. 2 - DRAPE PROFILES

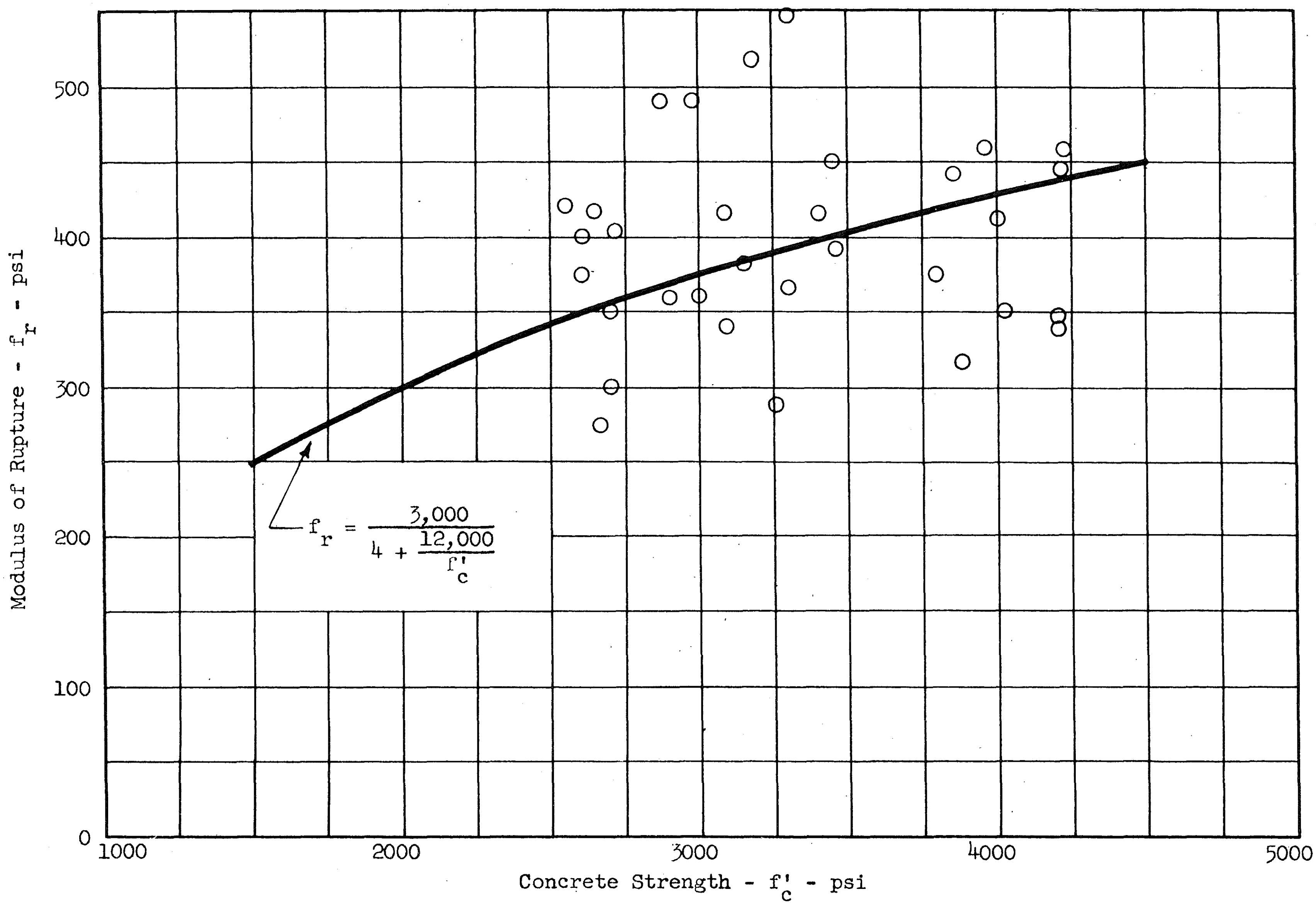


FIG. 3 - COMPARISON OF MODULUS OF RUPTURE WITH CONCRETE STRENGTH

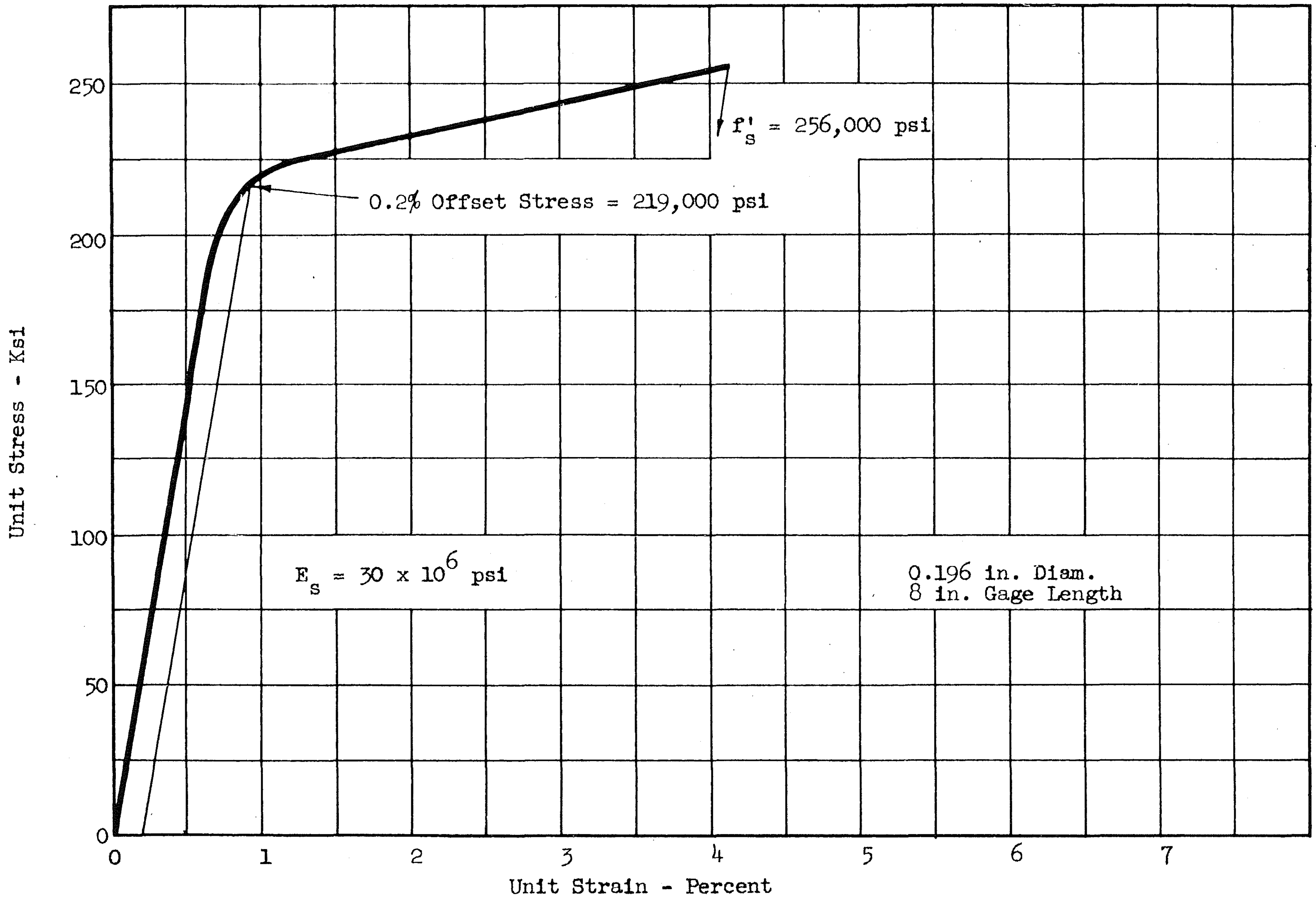


FIG. 4 - STRESS-STRAIN RELATIONSHIP FOR LOT XI WIRE

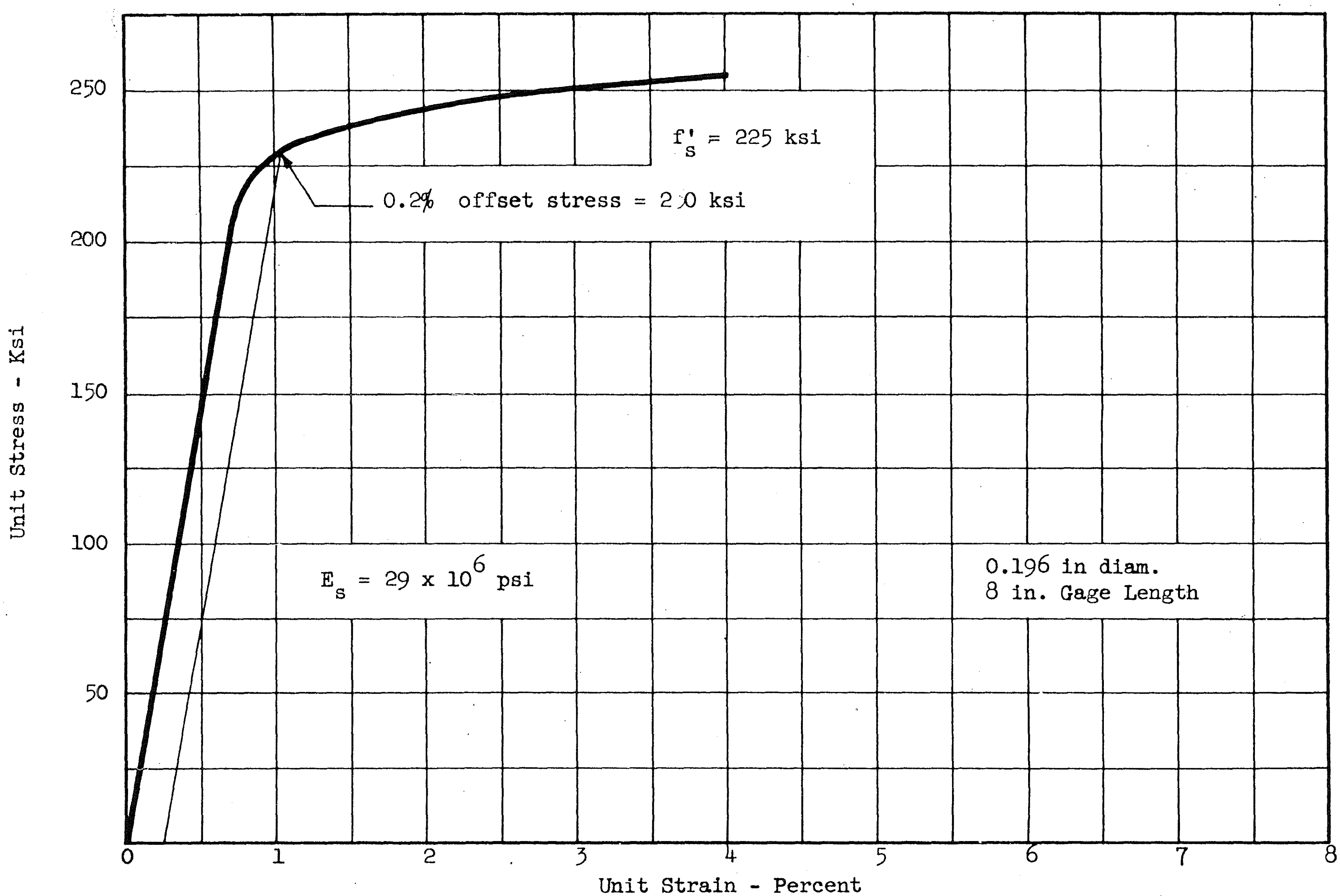


FIG. 5 - STRESS-STRAIN RELATIONSHIP -- LOT XII WIRE

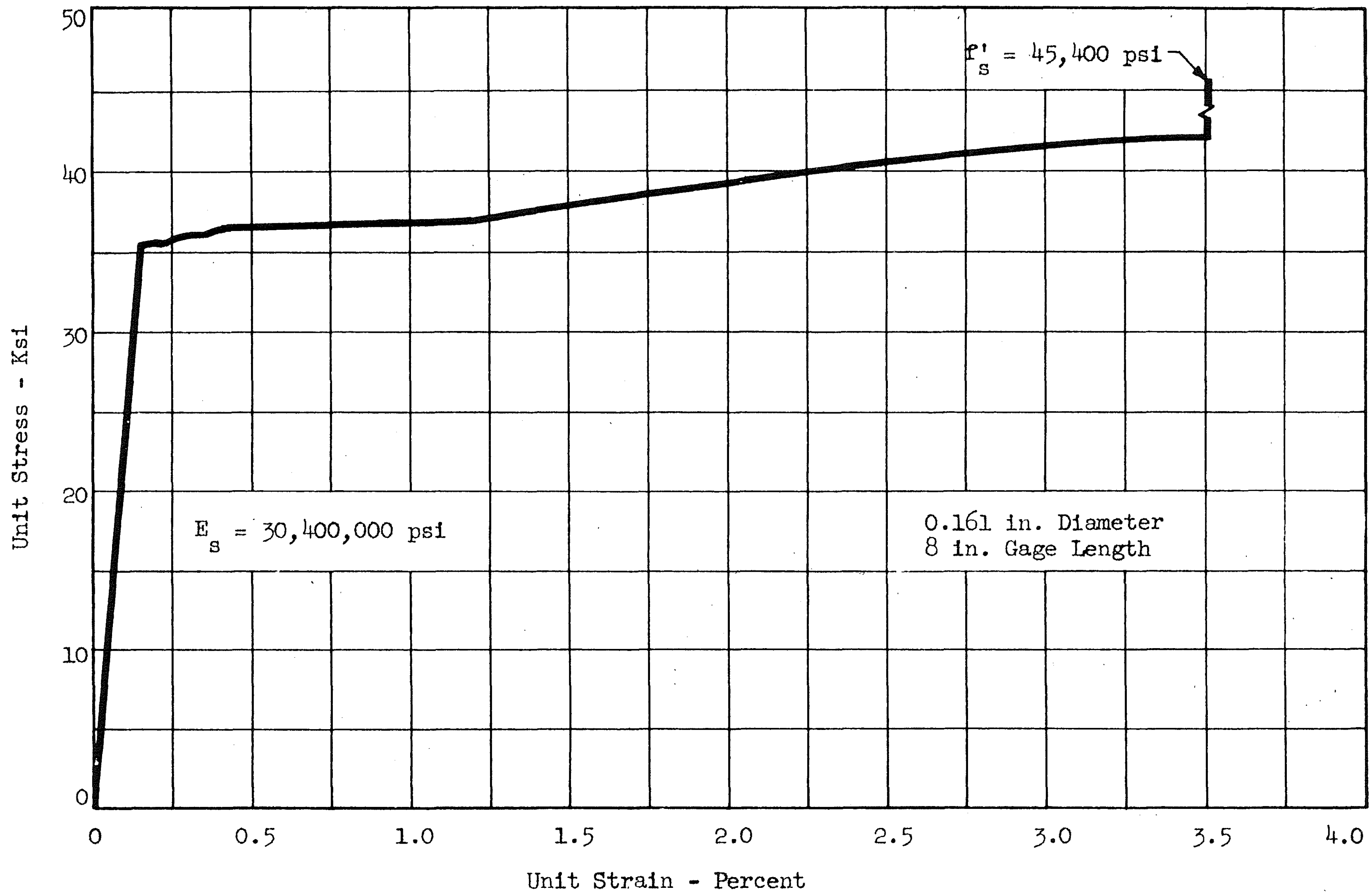


FIG. 6 - STRESS-STRAIN RELATIONSHIP FOR BLACK ANNEALED WIRE

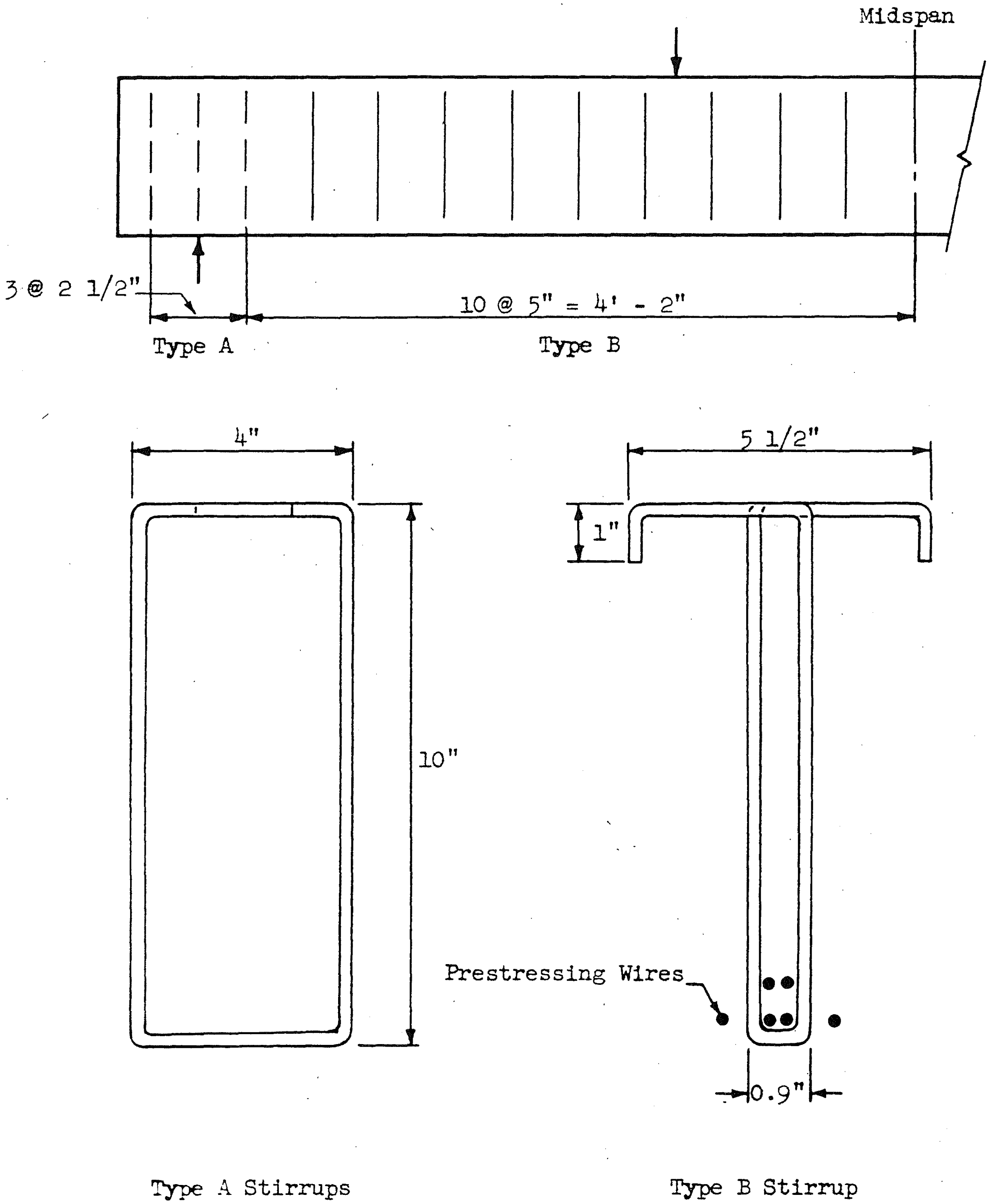
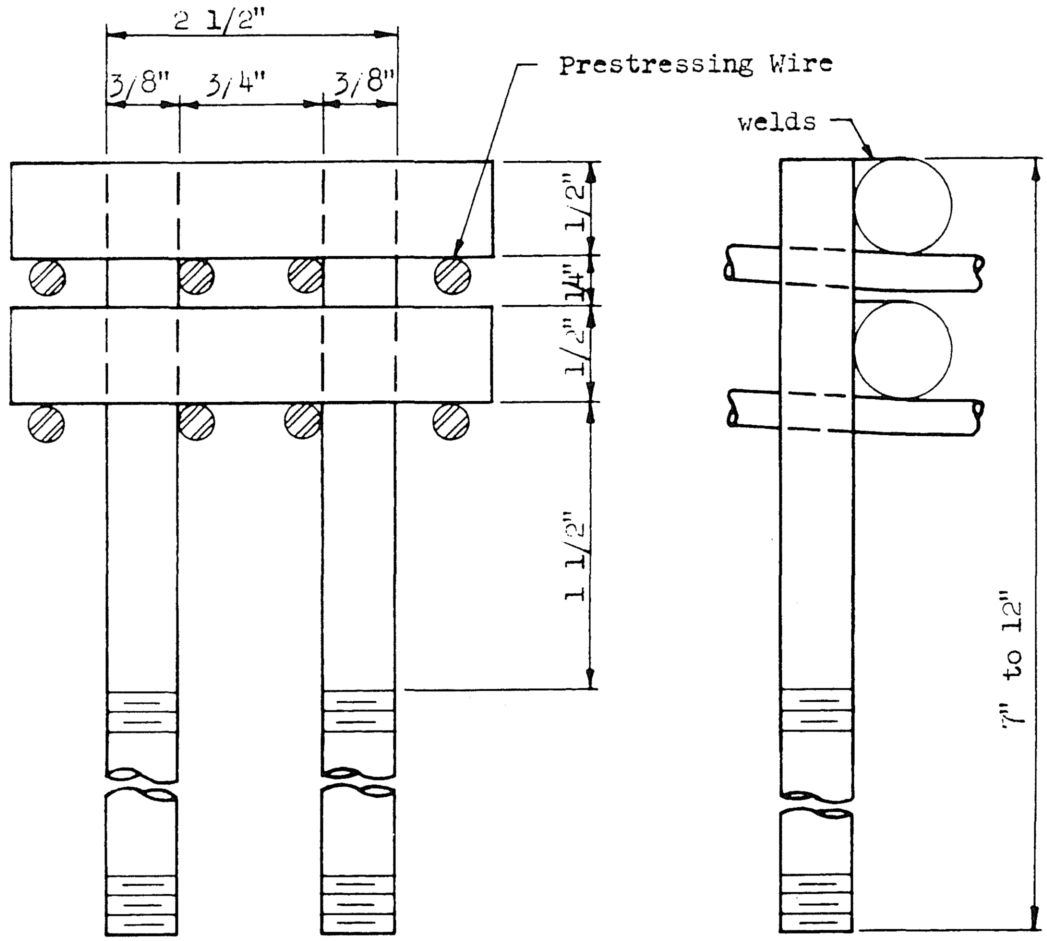
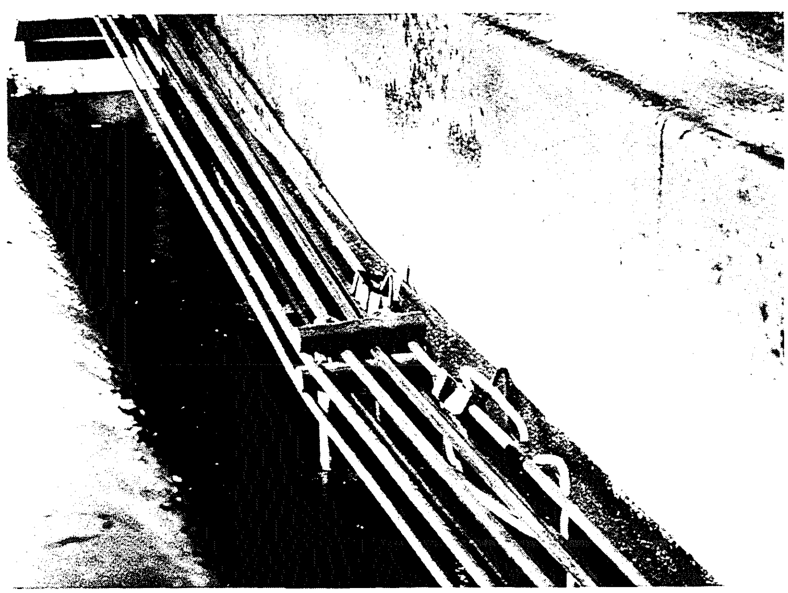


FIG. 7 - STIRRUP DETAILS

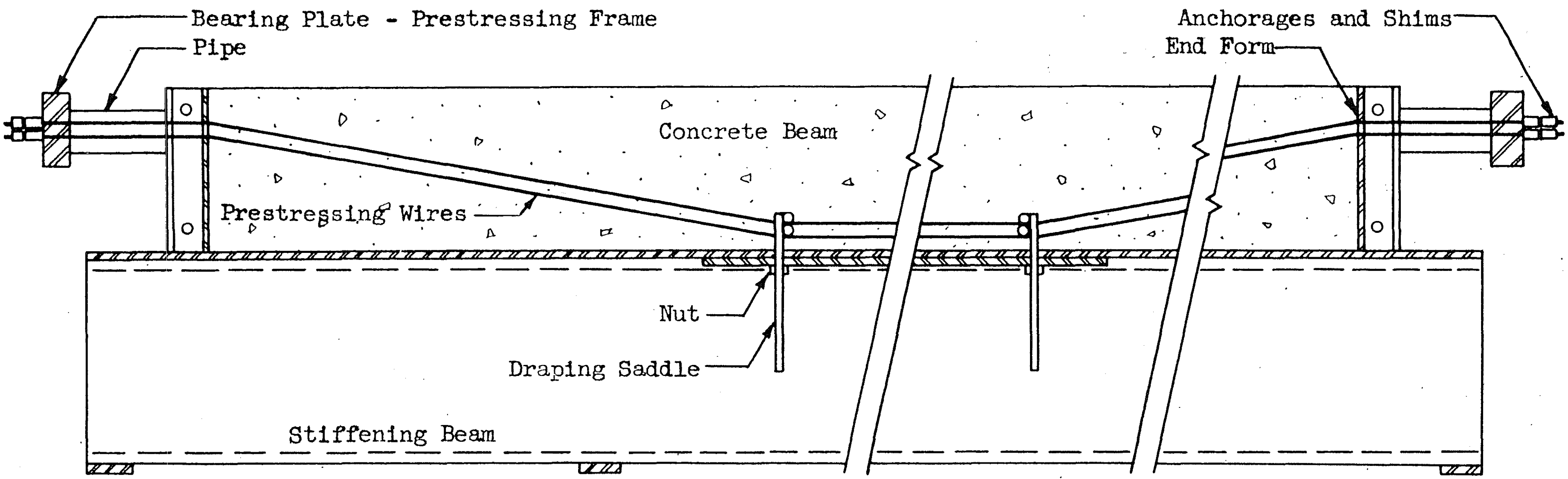


Detail of Draping Saddle

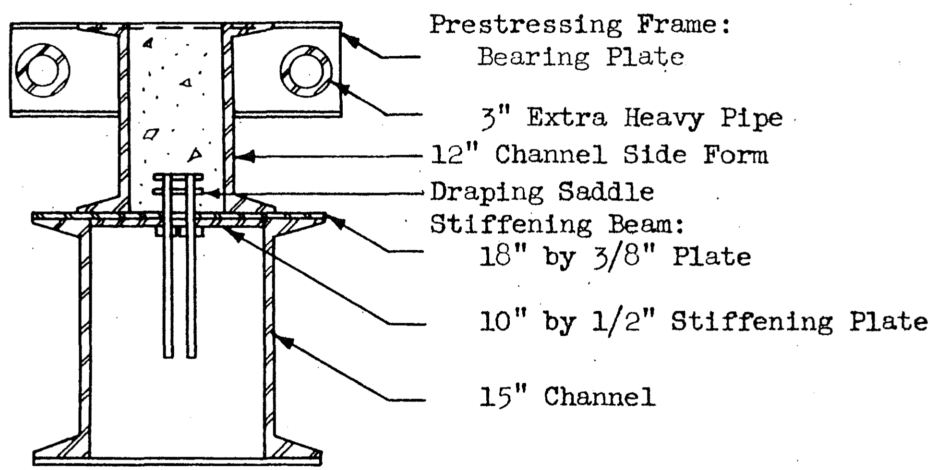


A Draping Saddle in Beam M2

FIG. 8 - DRAPING SADDLES

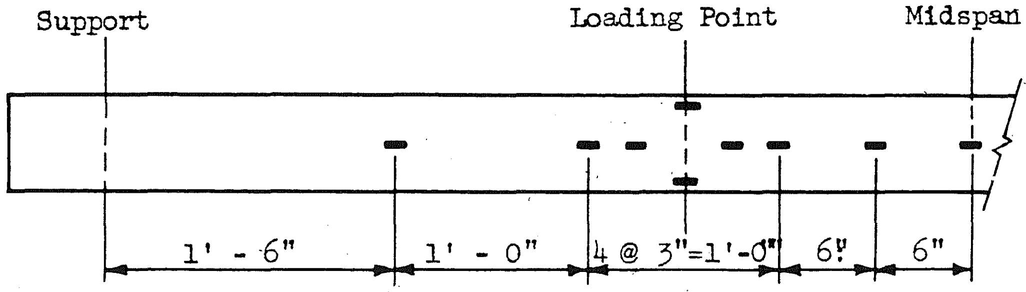


Longitudinal Section

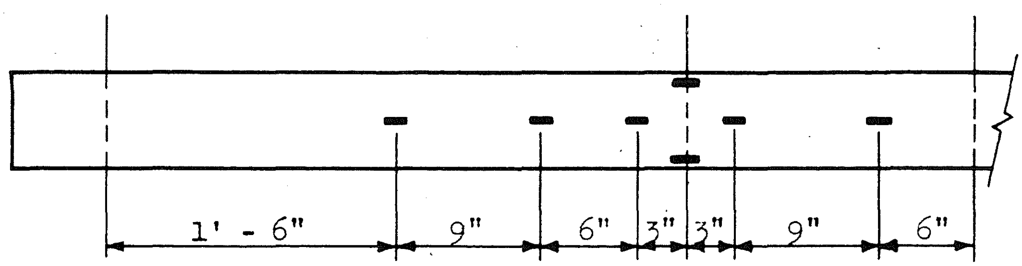


Transverse Section

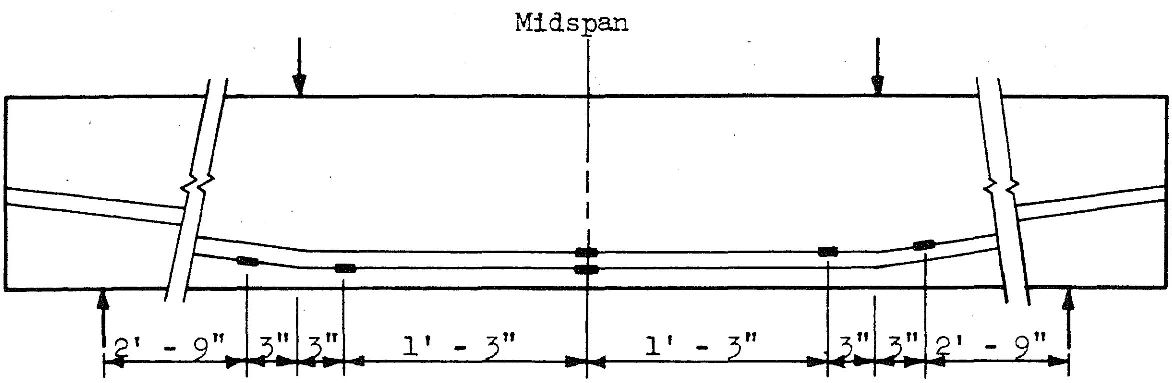
FIG. 9 - CROSS-SECTIONS OF BEAM AND FORM



Strain Gages on Top of Beams M1 through M3

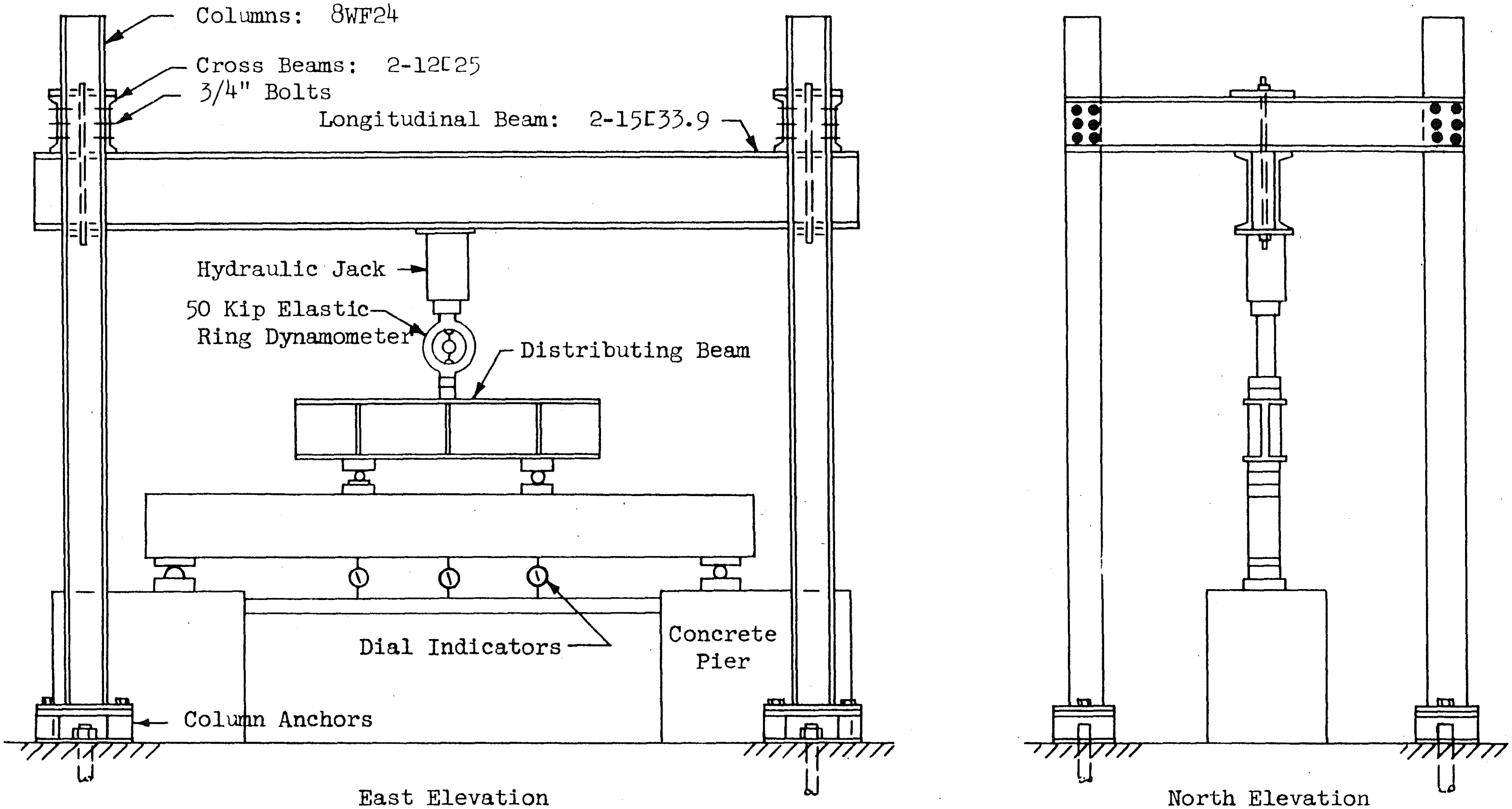


Strain Gages on Top of Beams M4 through M16



Strain Gages on Steel-Beams M1 through M8
(Beams M9 through M16 had gages at midspan only)

FIG. 10 - LOCATION OF STRAIN GAGES



Scale 3/8 in. = 1 ft

FIG. 11 - DETAILS OF TESTING FRAME

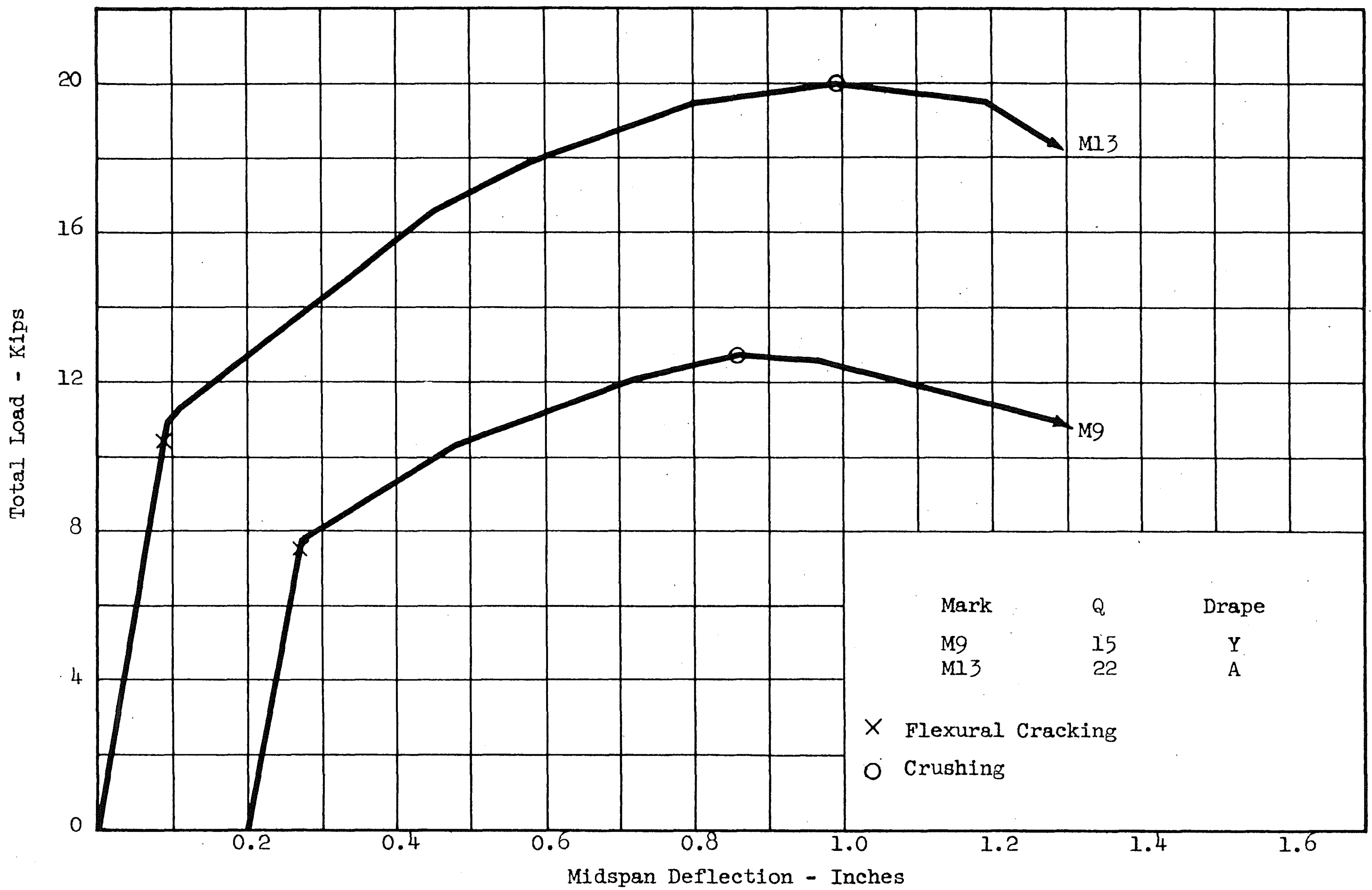


FIG. 12 - LOAD DEFLECTION CURVES FOR BEAMS FAILING IN FLEXURE

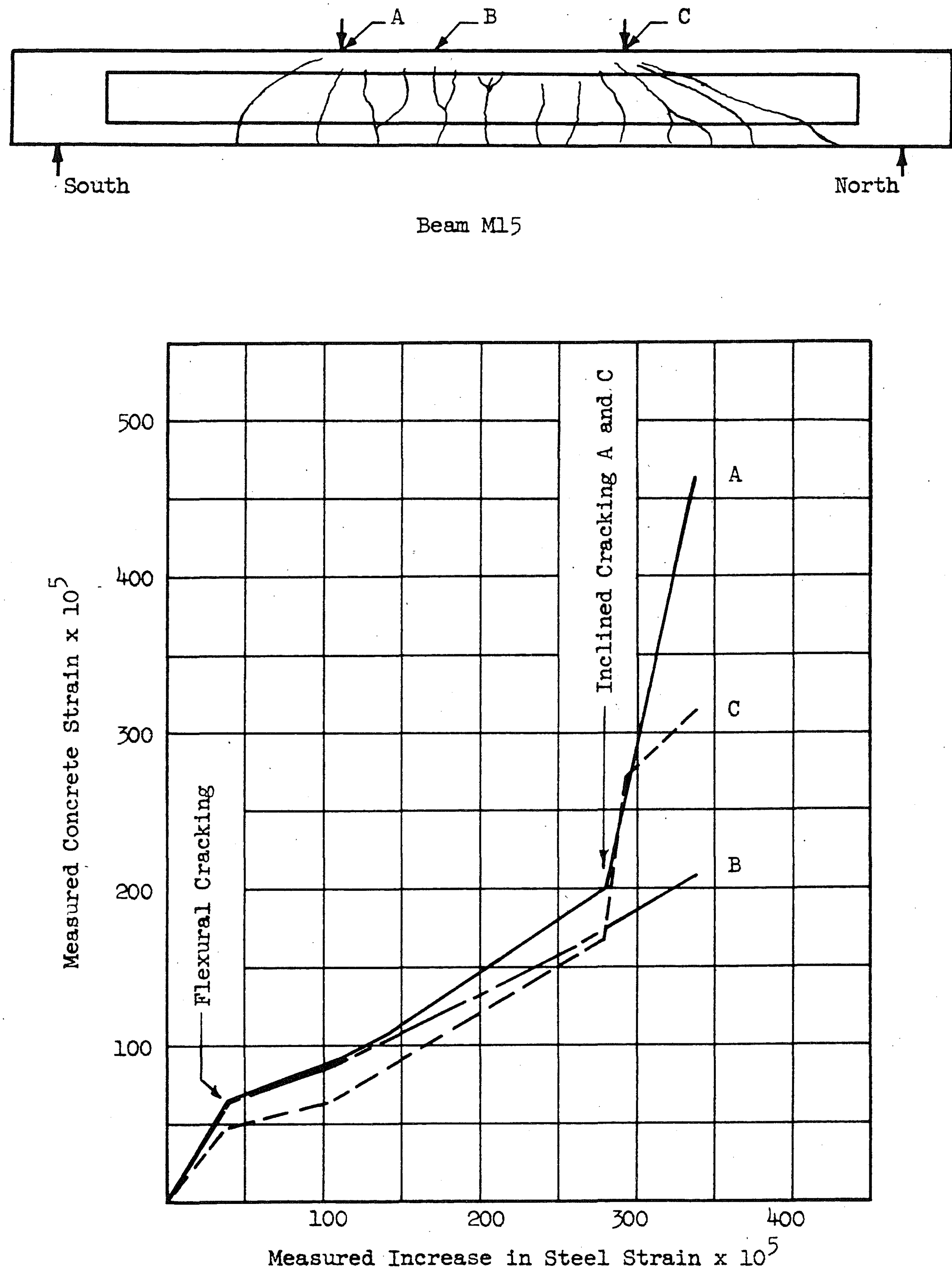
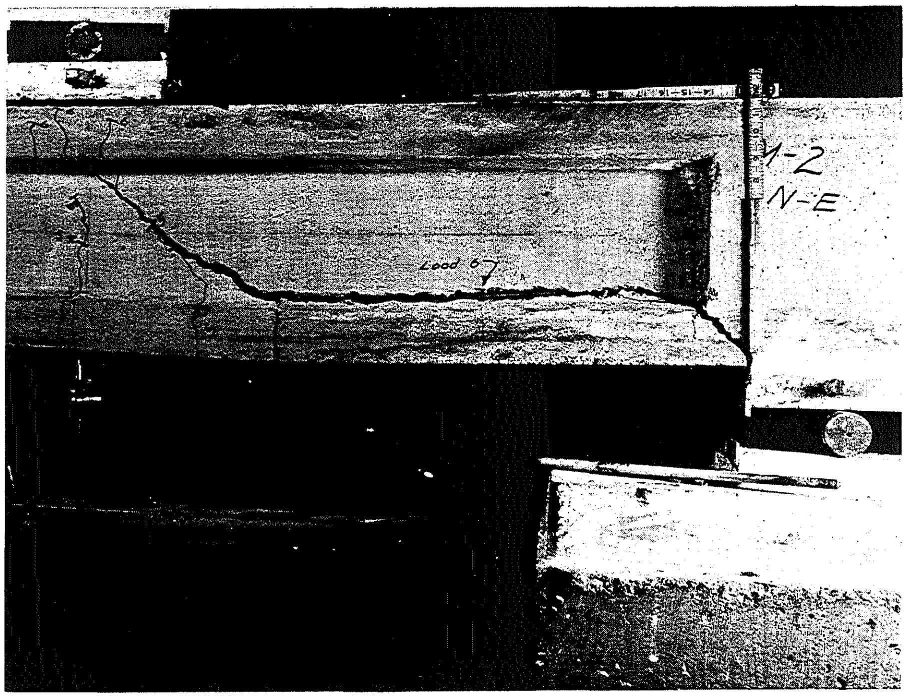
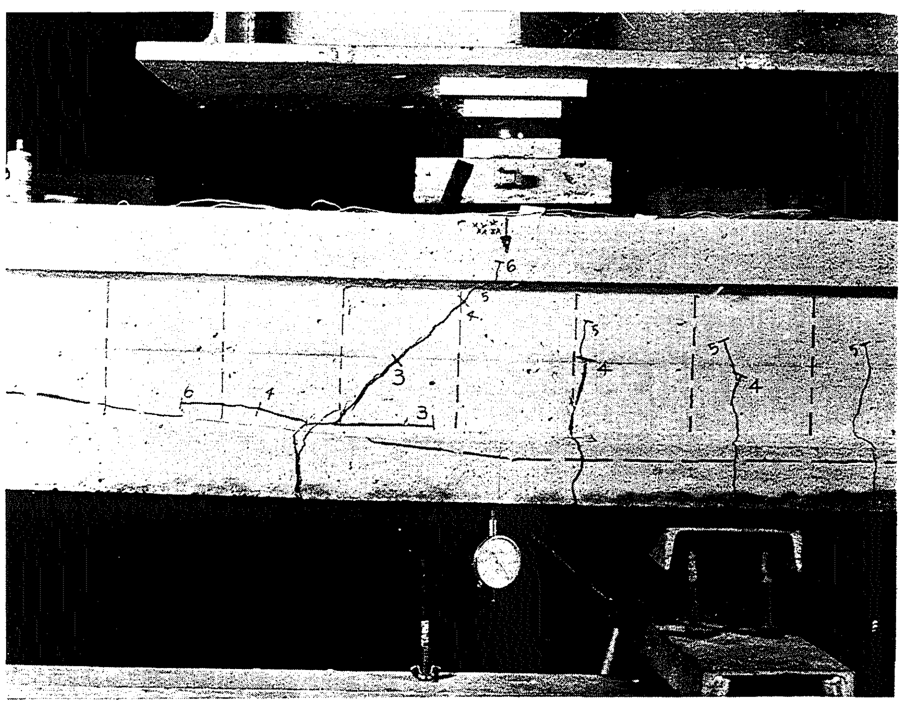


FIG. 13 - RELATION BETWEEN CRITICAL CONCRETE AND STEEL STRAINS

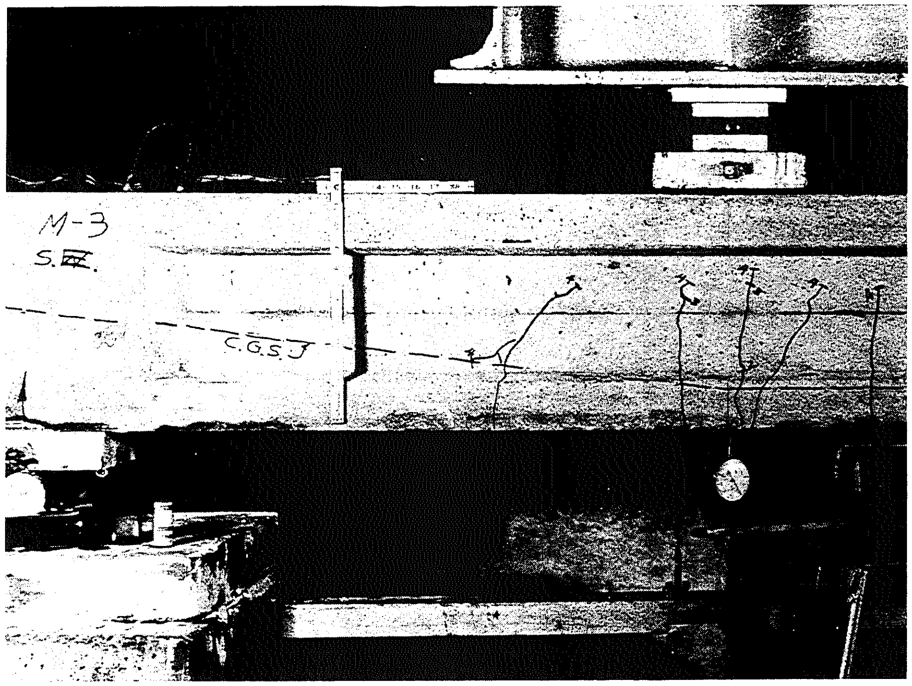


(a) Beam M2 at Failure

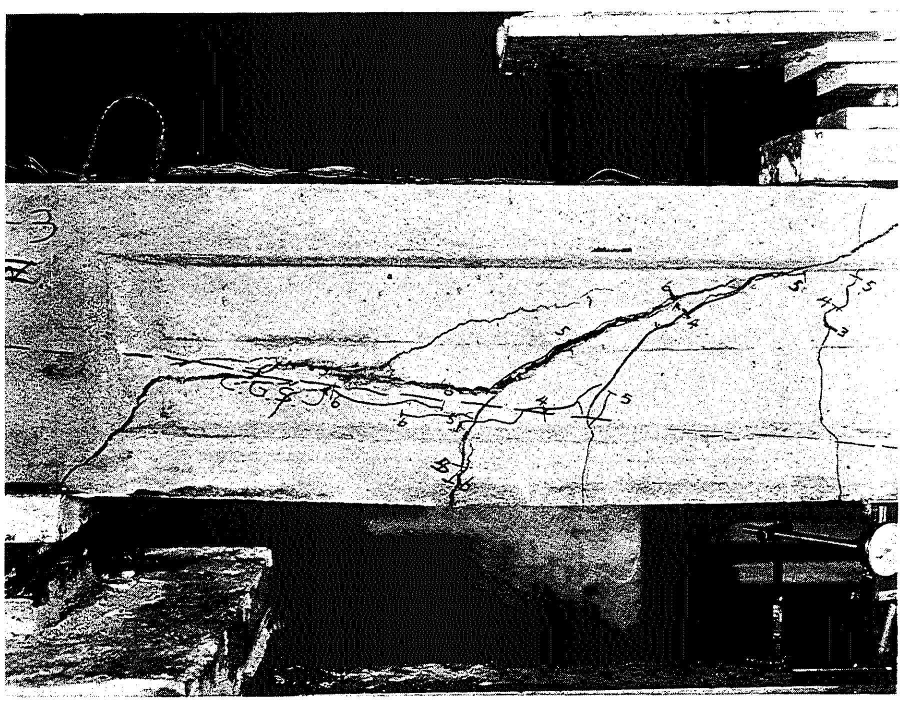


(b) Beam M10 at Failure

FIG. 14 - INITIATING CRACK DEVELOPING INTO A MAJOR INCLINED CRACK

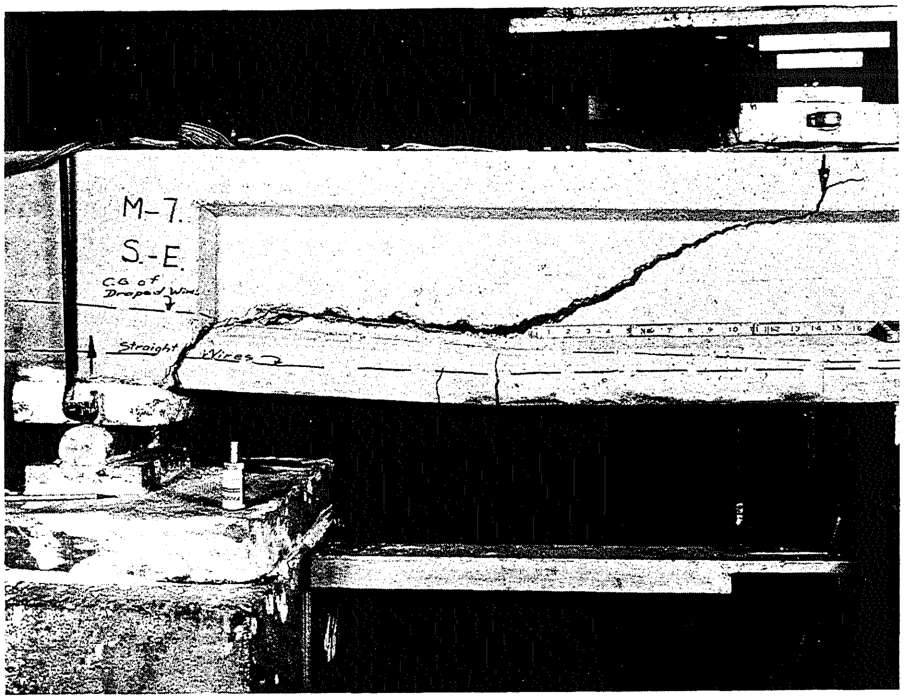


(a) Initiating Crack in Beam M3



(b) Beam M3 at Failure

FIG. 15 - INITIATING CRACK FOLLOWED BY A MAJOR INCLINED CRACK



Beam M7 at Failure

FIG. 16 - MAJOR INCLINED CRACK ORIGINATING IN WEB

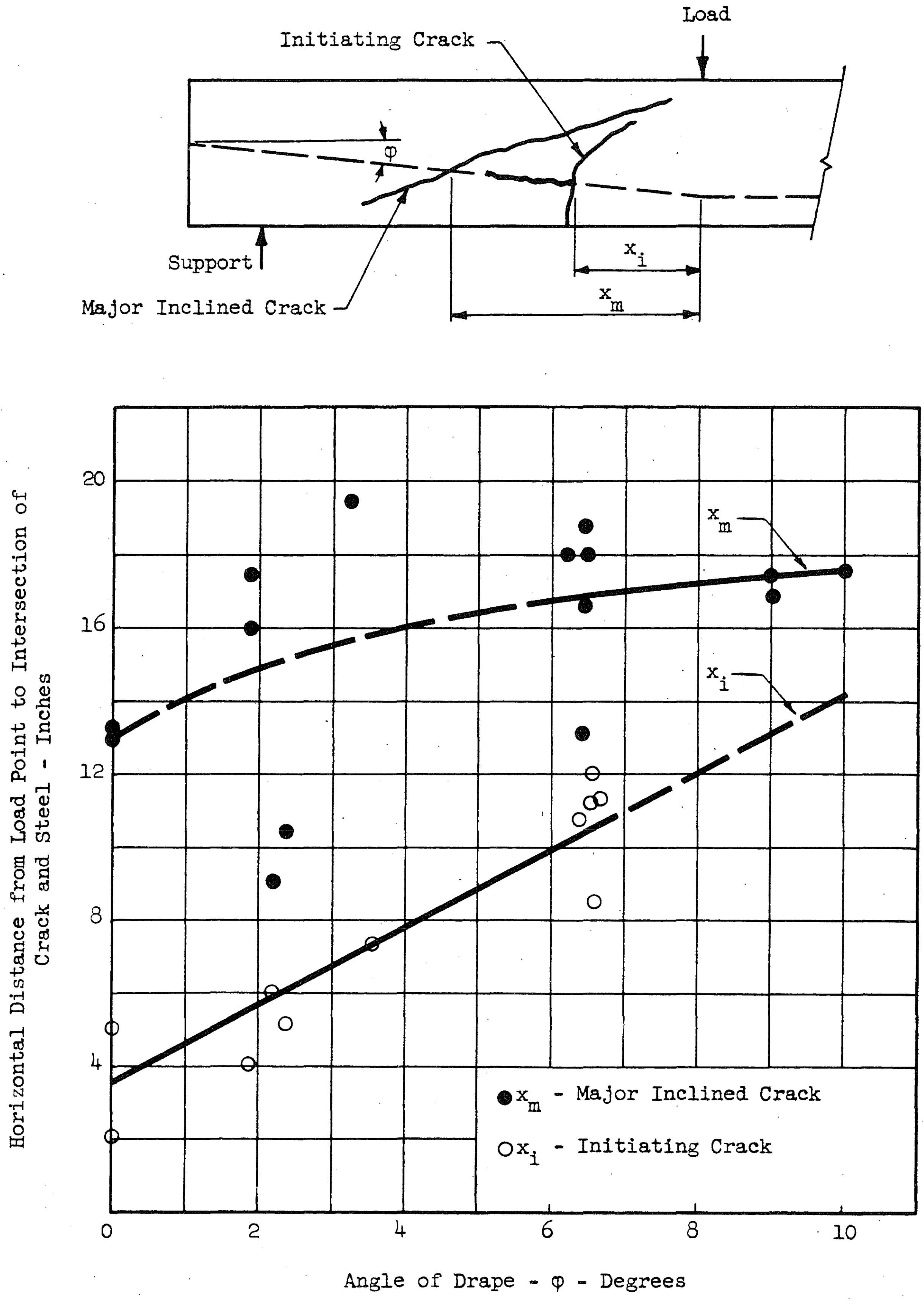


FIG. 17 - LOCATION OF INITIATING CRACKS AND MAJOR INCLINED TENSION CRACKS

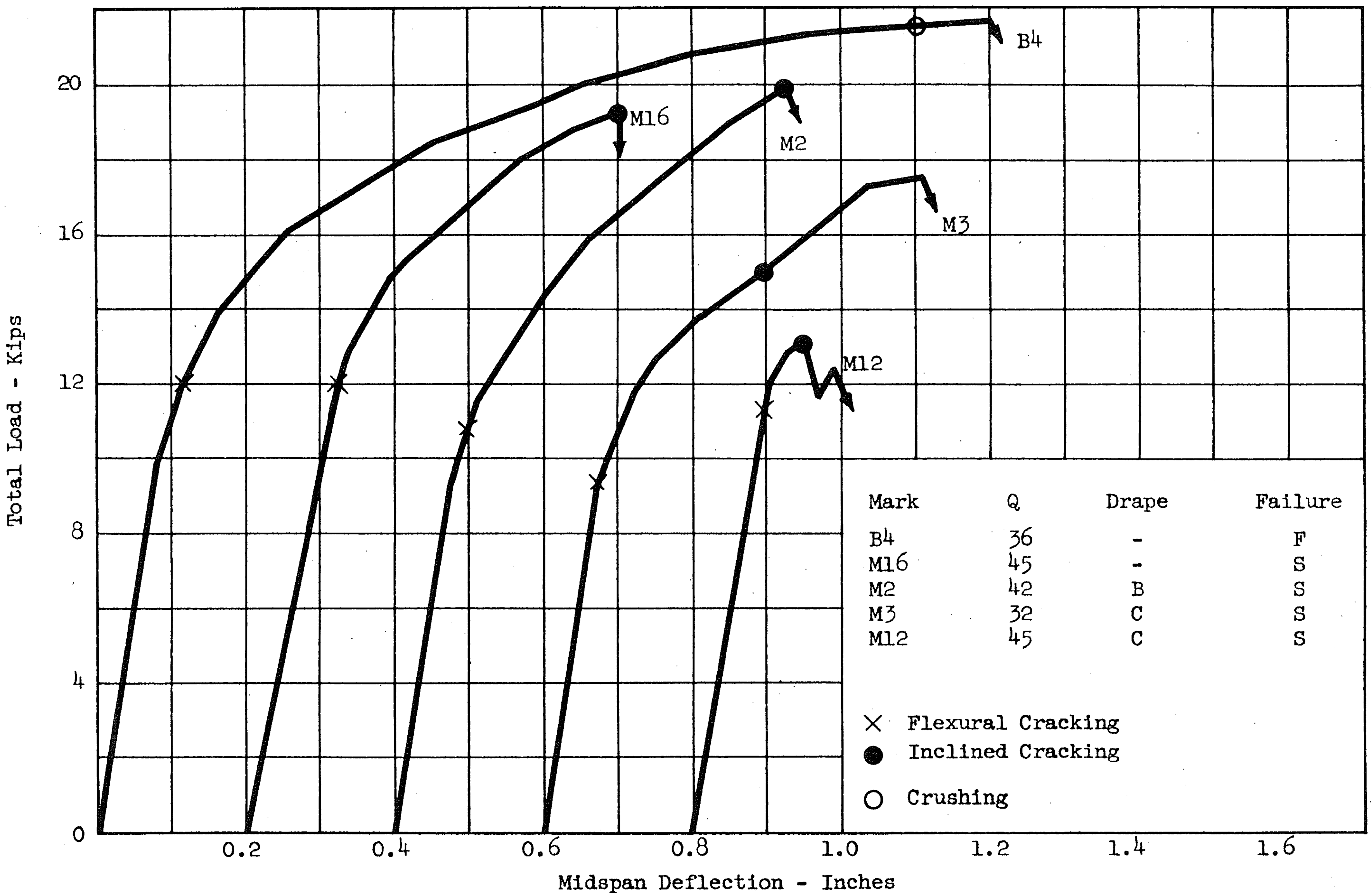


FIG. 18 - LOAD DEFLECTION CURVES FOR BEAMS WITH ALL WIRES DRAPED
BEAMS WITH EIGHT WIRES

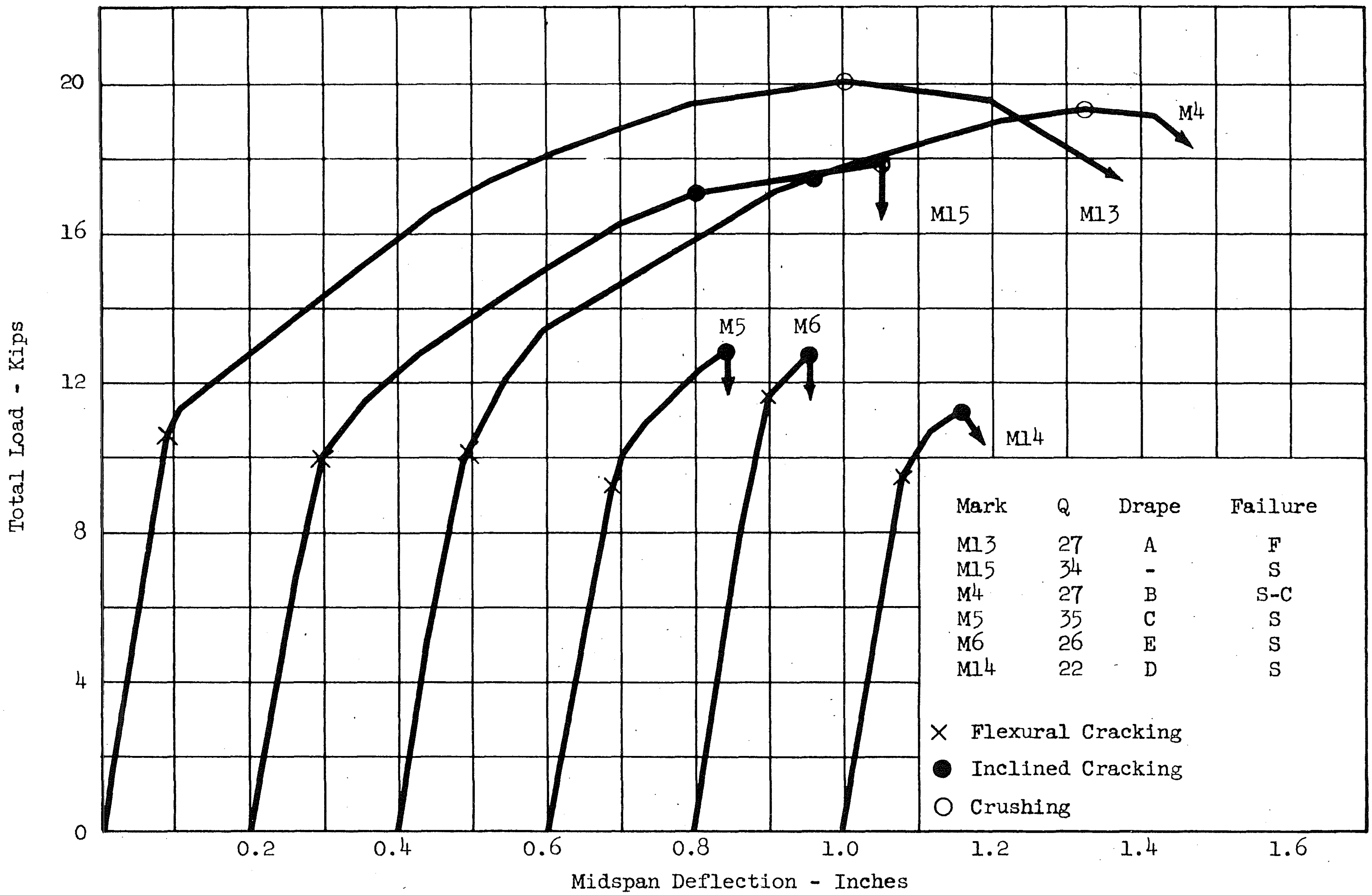
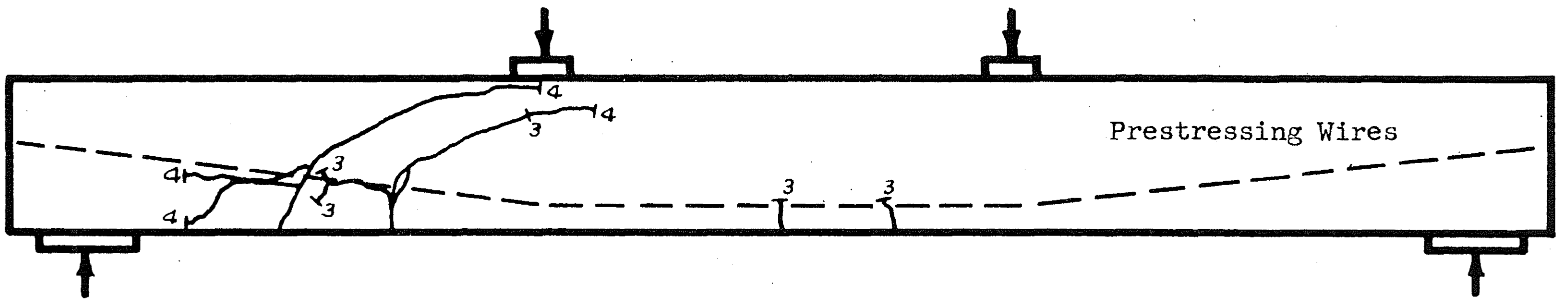
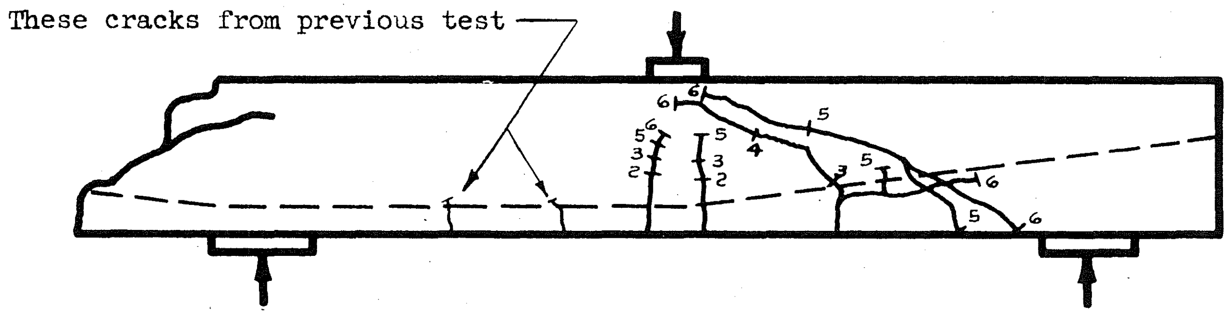


FIG. 19 - LOAD DEFLECTION CURVES FOR BEAMS WITH ALL WIRES DRAPED

BEAMS WITH SIX WIRES



(a) Beam M12 at Failure



(b) Beam M12a at Failure

(Numbers show Extent of Crack at Load Increments 2, 3, 4, 5 and 6)

FIG. 20 - TESTING PROCEDURE -- BEAMS M12 AND M12a

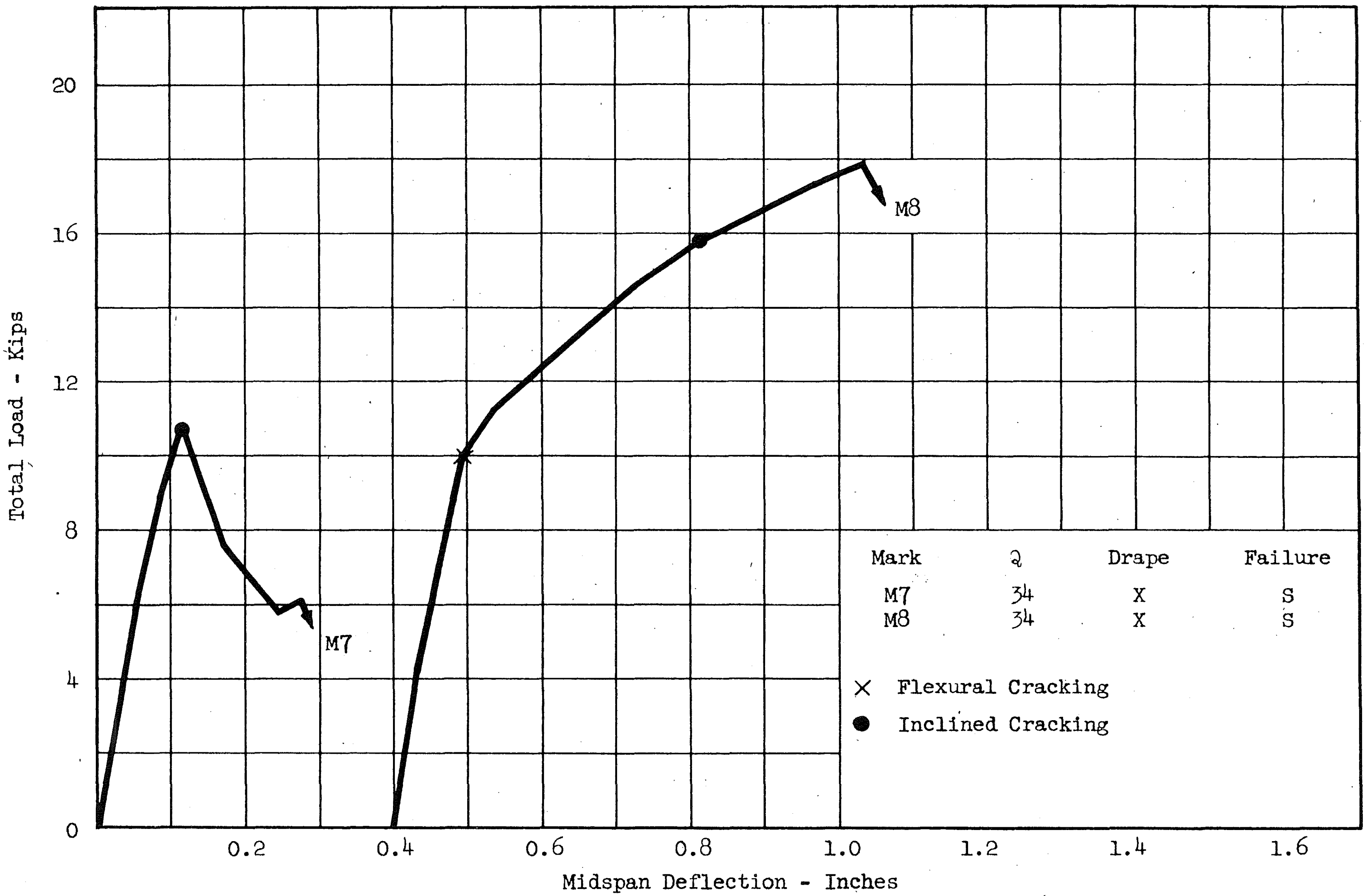


FIG. 21 - LOAD DEFLECTION CURVES FOR BEAMS WITH BOTH STRAIGHT AND DRAPED WIRES

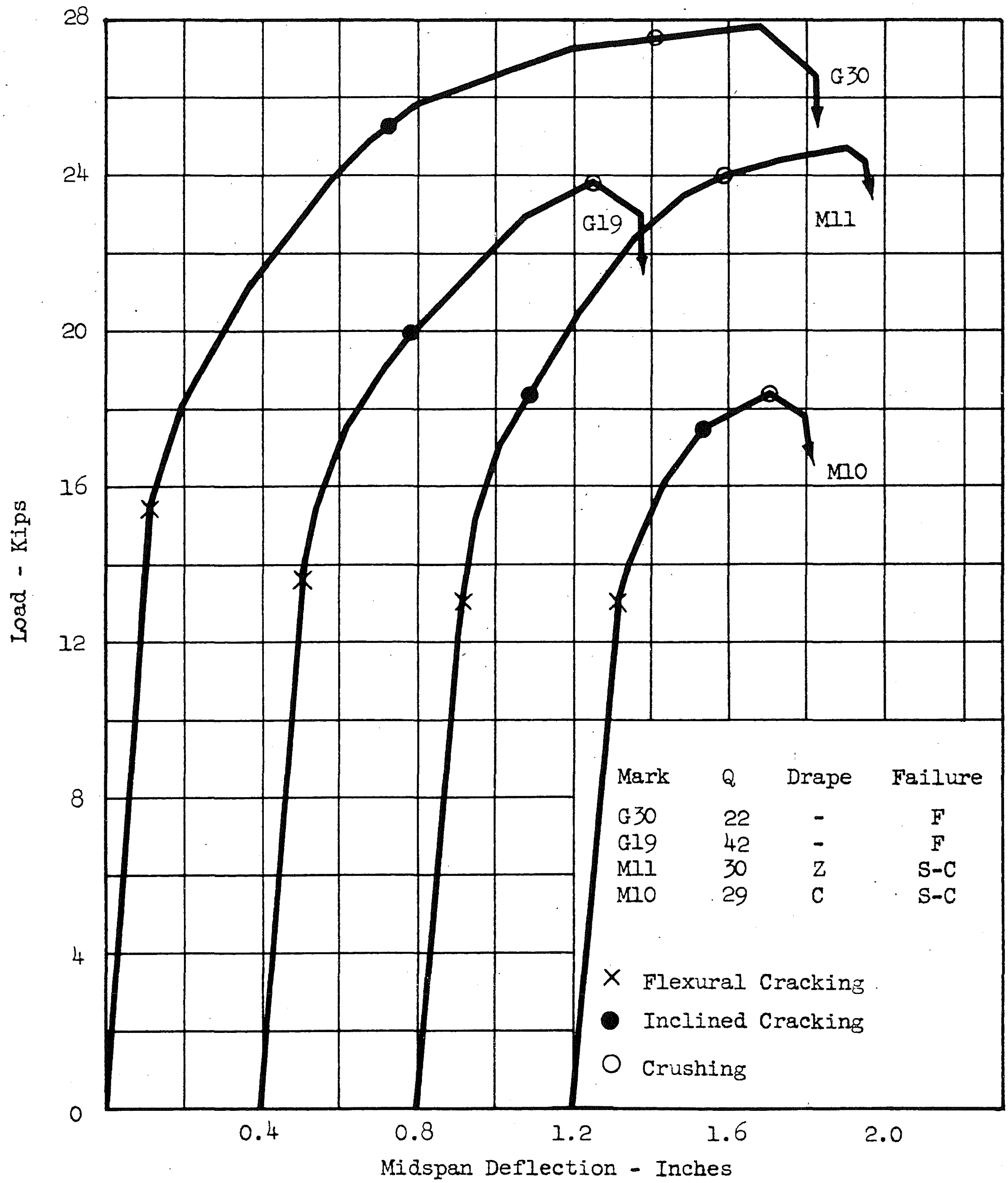


FIG. 22 - LOAD DEFLECTION CURVES FOR BEAMS WITH WEB REINFORCEMENT

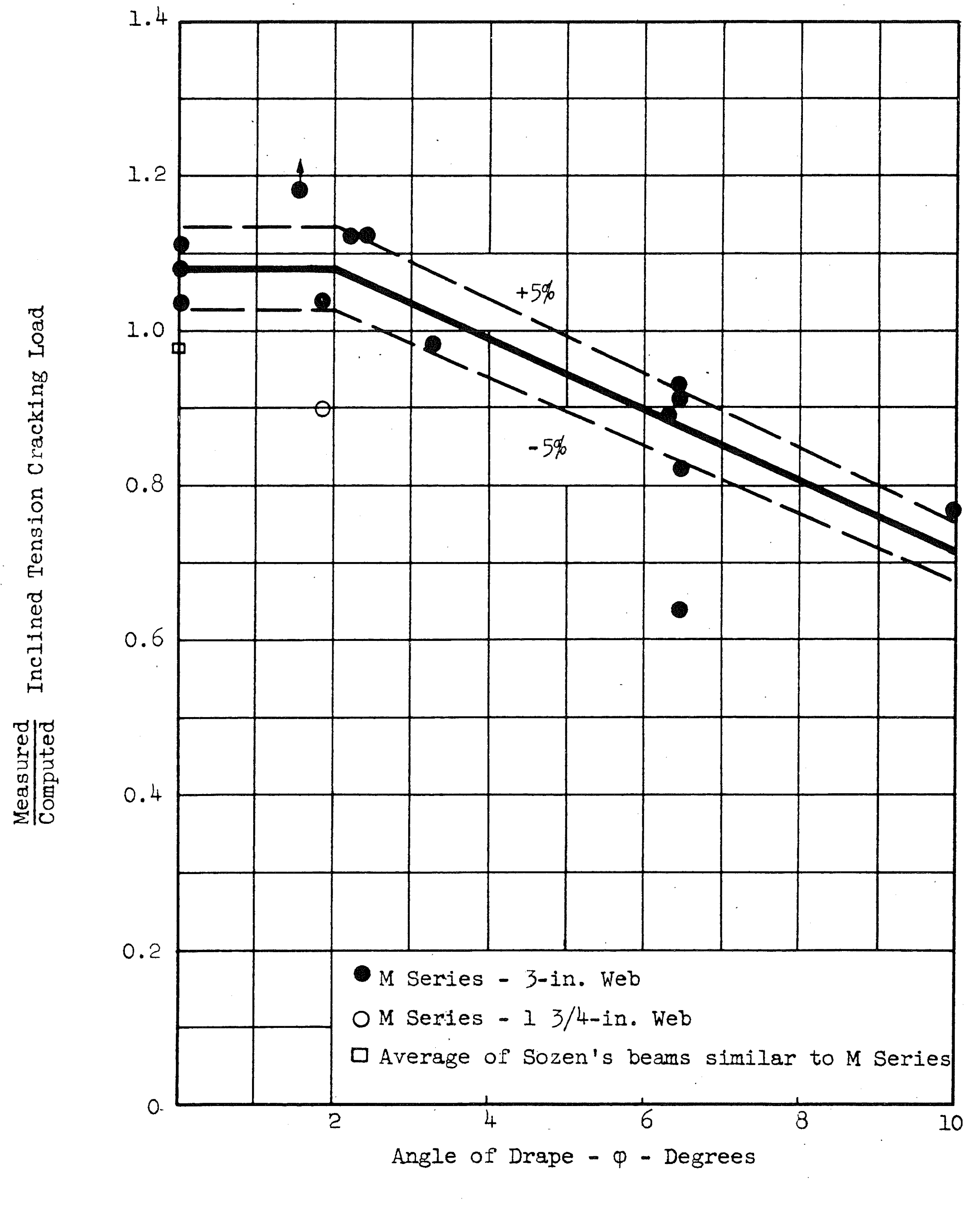


FIG. 23 - EFFECT OF ANGLE OF DRAPE ON INCLINED TENSION CRACKING LOAD

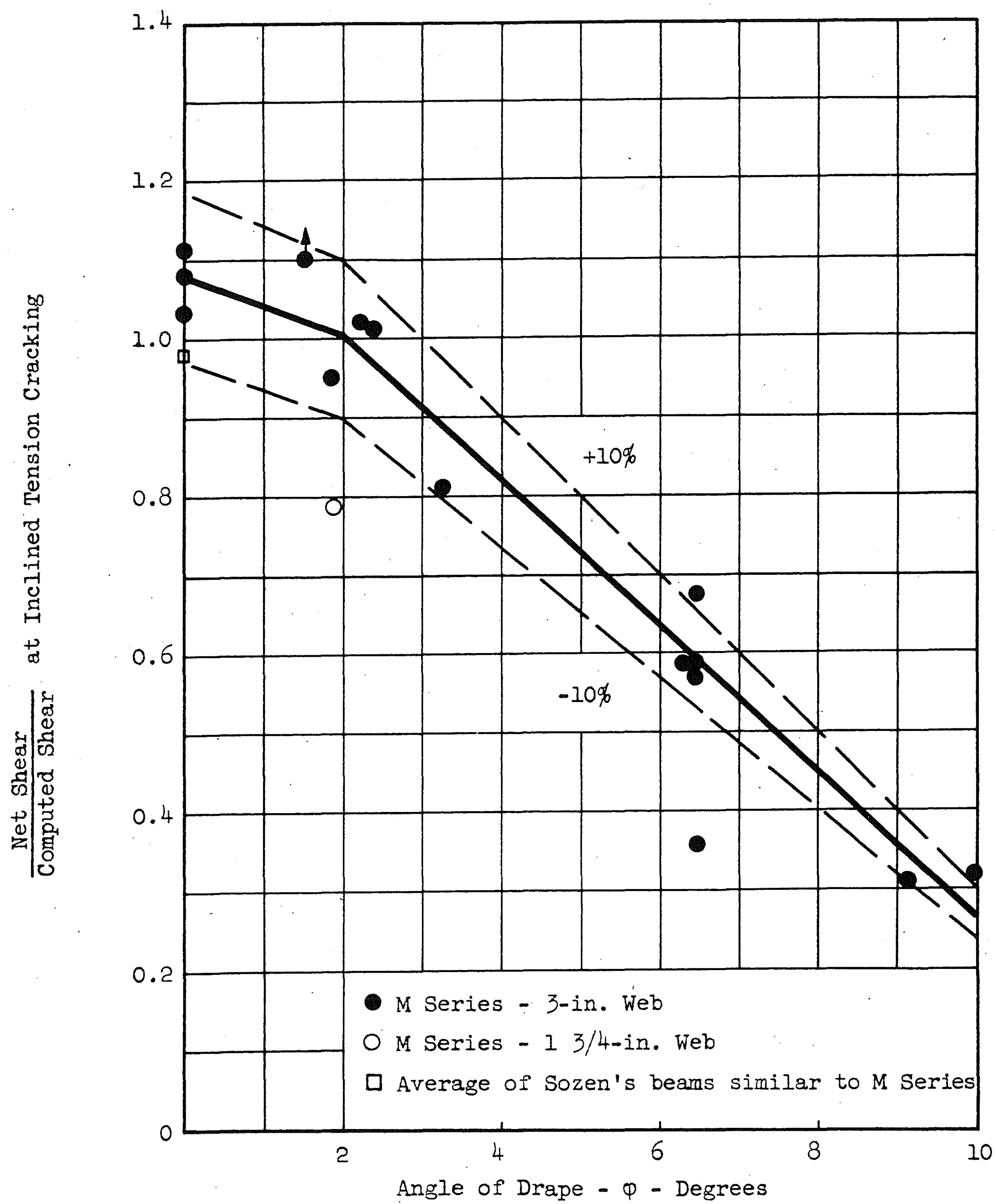


FIG. 24 - EFFECT OF ANGLE OF DRAPE ON NET SHEAR AT INCLINED TENSION CRACKING

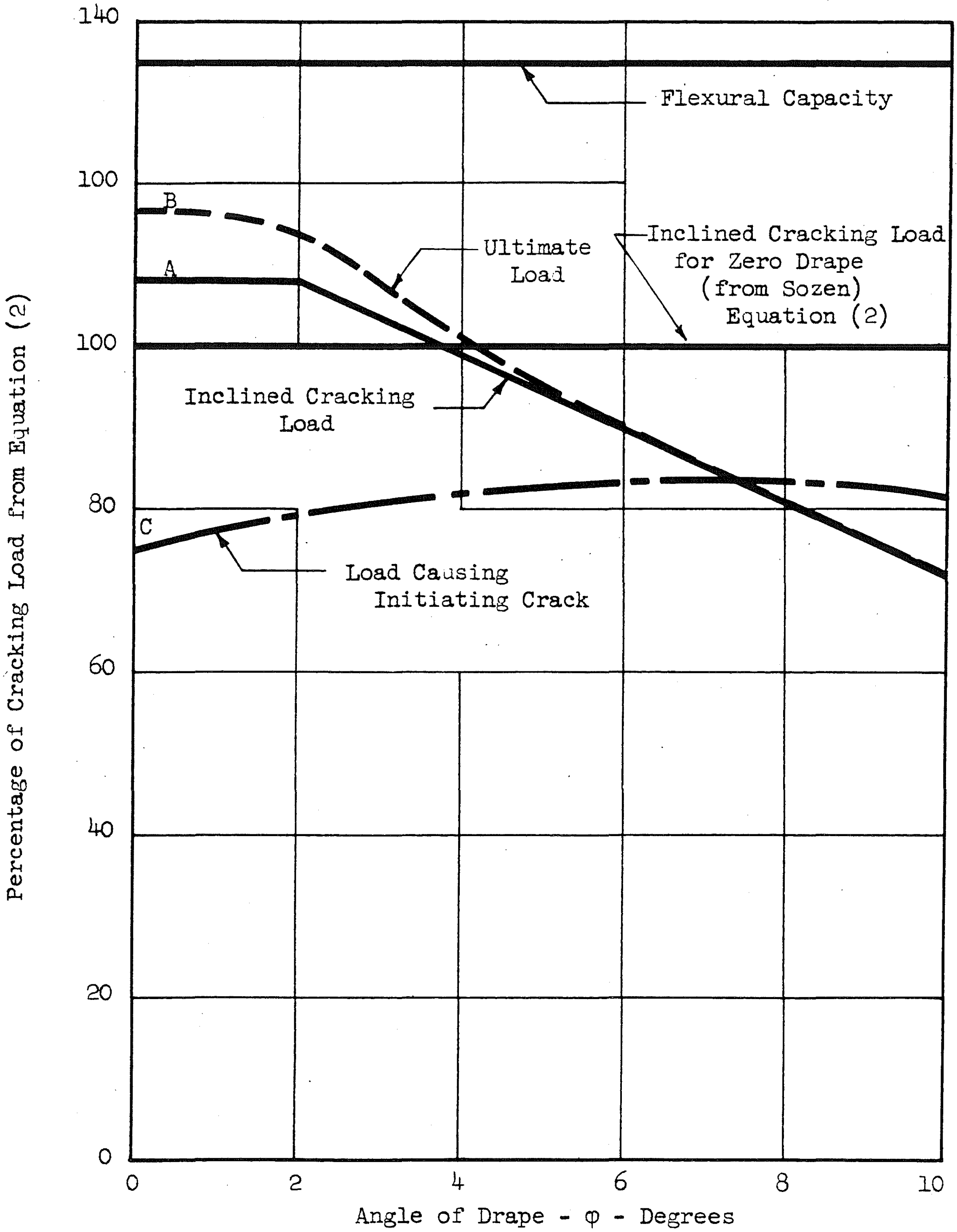
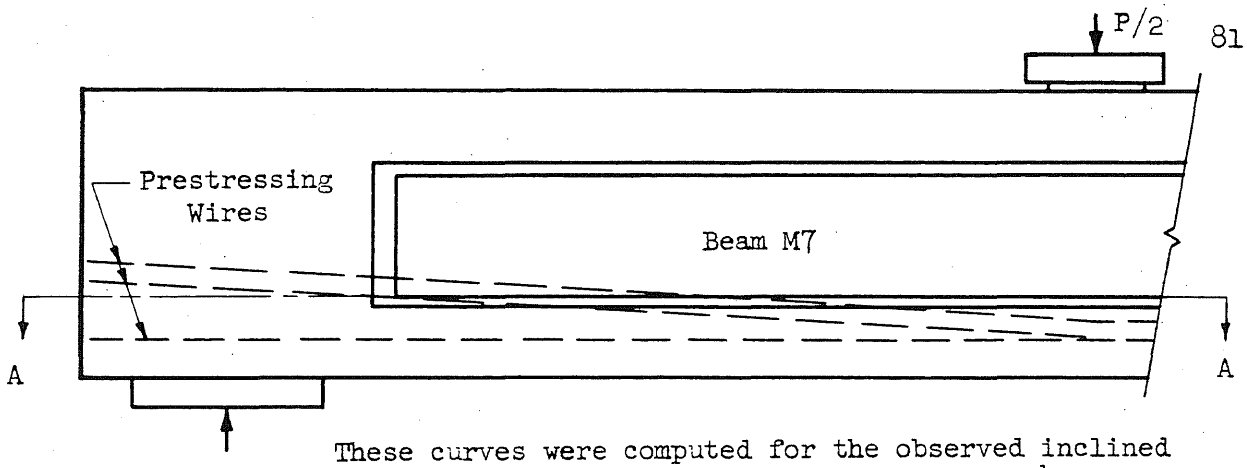


FIG. 25 - COMPARISON OF CRACKING LOADS AND ULTIMATE LOADS FOR VARIOUS DRAPE ANGLES



These curves were computed for the observed inclined tension cracking load for Beam M7 ($P_c = 10.9^k$).

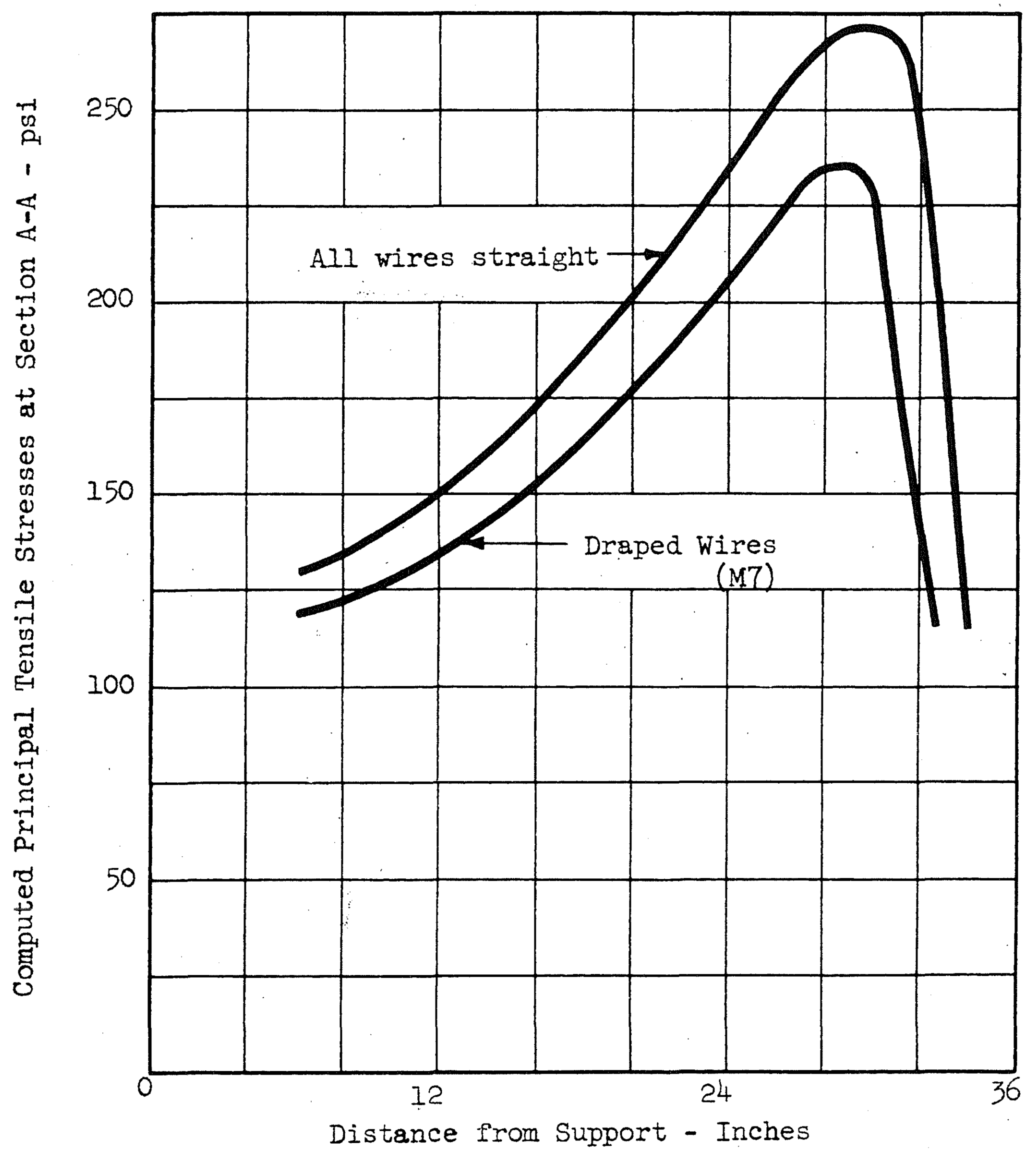
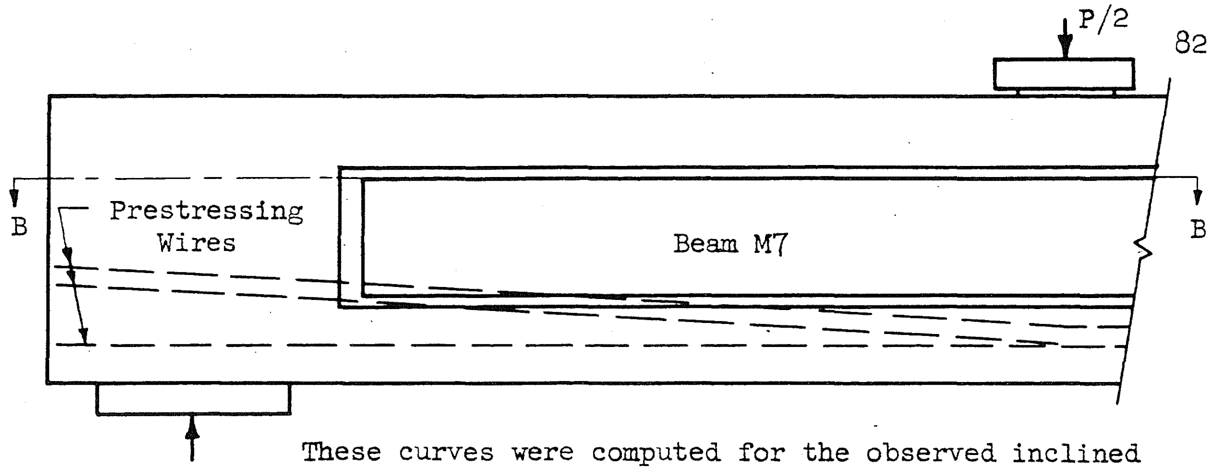


FIG. 26 - EFFECT OF DRAPING ON PRINCIPAL TENSILE STRESSES AT BOTTOM OF WEB



These curves were computed for the observed inclined tension cracking load for Beam M7 ($P_c = 10.9^k$).

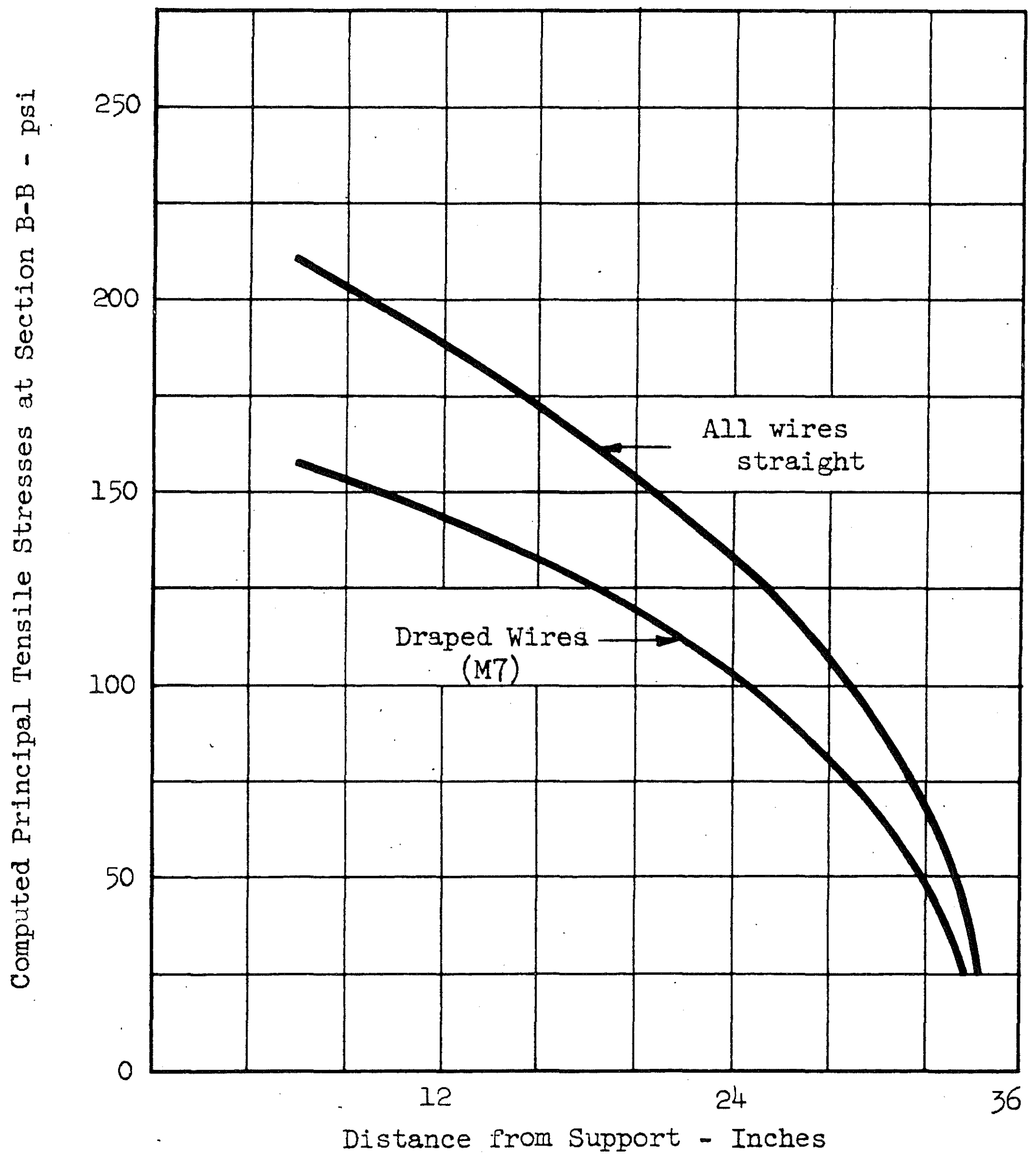


FIG. 27 - EFFECT OF DRAPING ON PRINCIPAL TENSILE STRESSES AT TOP OF WEB

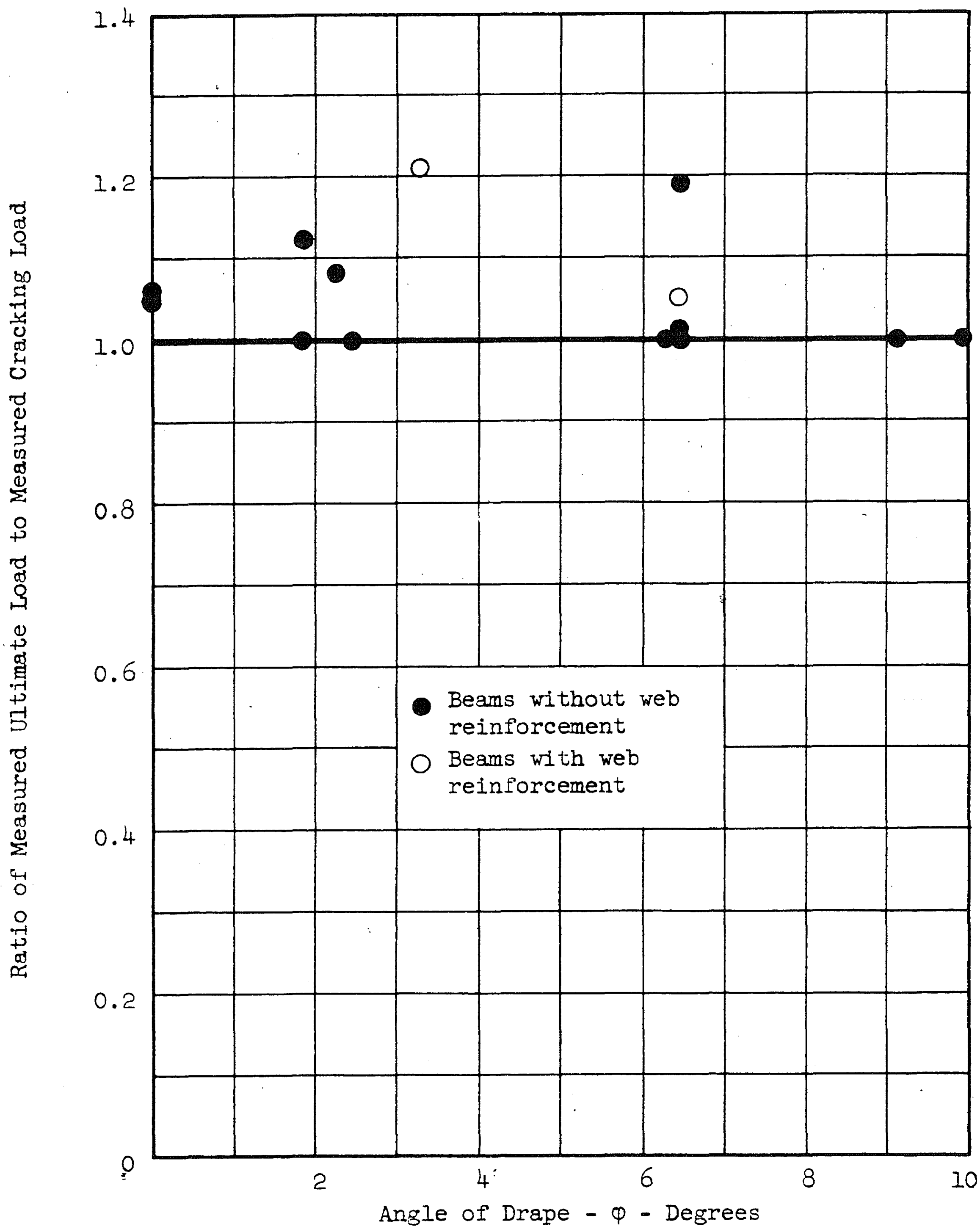


FIG. 28 - INCREASE IN LOAD AFTER INCLINED CRACKING