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CRACK-ARREST TECHNIQUES IN REINFORCED CONCRETE STRUCTURAL ELEMENTS. REPORT 1. LABORATORY TESTS

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In this study, tests were conducted to develop improved methods of arresting or preventing undesired flexural cracking within the tensile zones of conventionally reinforced concrete beams. Specimens representing several potential crack-arrest techniques including (a) fiber-reinforced concrete, (b) concrete with wire mesh, and (c) epoxy-resin concrete were tested under either short- or long-term static loading. The principal conclusions of the investigation

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20. ABSTRACT (Continued).

are as follows: (a) randomly mixed steel fibers incorporated in concrete can increase the precracking load over that of conventionally reinforced concrete; (b) 0.5-in.-long fibers will produce better results in certain mixtures than will 1-in.-long fibers; (c) steel fibers do not significantly increase the flexural capacity of conventionally reinforced beams; (d) properly constructed and positioned wire cages can increase the precracking loads of flexural members; (e) epoxy-resin concrete layers provided within the tensile zones of small conventionally reinforced composite beams can increase precracking loads by as much as 300 percent over those of conventionally reinforced concrete beams; (f) the size of cross section, length of member, etc., may influence the initial cracking load and resulting crack pattern as much as the size and spacing of the reinforcement; (g) although sustained loading does not appear to affect the ultimate loads of members using any of the tentatively recommended crack-arrest techniques, it does significantly affect crack patterns: (h) neglecting cost, placing steel fibers throughout the entire cross section appears to be the best technique but the desired results can also be obtained using epoxy-resin concrete or concrete with wire mesh; and (i) members designed according to the tentative recommendations outlined herein can be expected to have maximum crack widths smaller than those of conventionally reinforced members by factors of 1.4 to 2.5; however, it is emphasized that these designs are based on a limited number of tests and could be subject to change as additional test results become available.

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SUMMARY

In the design of conventionally reinforced concrete beams, it is assumed that all tensile forces are carried by the reinforcement. When such beams are subjected to their normal service loads, the tensile stresses within the concrete exceed the tensile strength of the concrete, and cracks occur in the tensile zones of the beams. These cracks are usually very fine and generally do not represent any immediate danger to the structure. However, they can provide a direct access to the surface of the conventional (steel) reinforcement for environmental factors that may initiate corrosion of the reinforcement and subsequent spalling of the concrete cover. As a result, a structure can become unsightly, and, unless remedial measures are taken, it may become unsafe. Therefore, a definite need exists for improved methods of arresting or preventing undesired flexural cracking within the tensile zones of conventionally reinforced concrete beams.

The available literature indicates that several potential crack-arrest techniques are currently being investigated. These include: (a) fiber-reinforced concrete, (b) wire mesh, and (c) epoxy-resin concrete. Unfortunately most of these techniques are or have been studied with only plain concrete mixtures and not in conjunction with reinforced concrete. Therefore, ten 4- by 9- by 72-in. and six 5- by 14- by 180-in. specimens (four 4- by 9- by 72-in. and two 5- by 14- by 180-in. specimens were used as control members), each using a crack-arrest technique that indicated promise either from the available literature or from previous testing, were tested under either short- or long-term static loading so that the information in the literature could be supplemented by results of tests of specimens that better simulate actual structural behavior of beams in service.

The principal conclusions of the investigation are as follows:

a. Randomly mixed steel fibers incorporated in concrete in the amount of 2.5 percent by volume can increase the precracking load over that of conventionally reinforced concrete beams by an amount as high as 275 percent in smaller members.

b. Probably due to the difficulty of obtaining a uniform mixture as fiber length versus cross-sectional area increases, 0.5-in.-long fibers will produce better results in certain mixtures than will 1-in.-long fibers.

c. Although some of the tests of the larger beams indicated otherwise, it is not believed that steel fibers significantly increase the flexural capacity of conventionally reinforced beams. In fact, as soon as cracking was initiated in the smaller beams, they tended to react as similar conventionally reinforced beams.

d. Properly constructed and positioned wire cages can increase the precracking loads of flexural members; however, it is believed that their greater value is in controlling or reducing the cracks once cracking is initiated.

e. Epoxy-resin concrete layers provided within the tensile zones of small conventionally reinforced composite beams can increase precracking loads by as much as 300 percent over those of conventionally reinforced concrete beams. This indicates that these special composite beams can provide a noncracking cross section up to or near failure. However, until more is known about epoxy-resin concrete mixtures (creep, shrinkage, thermal expansion, sensitivity to environmental factors, exothermic characteristics, etc.), composite construction cannot be recommended for practical use.

f. The size of cross section, length of member, etc. (possibly due partially to the initiation of minute, undetectable cracking during handling), may influence the initial cracking load and resulting crack pattern as much as the size and spacing of the reinforcement; therefore, it is believed that recommendations regarding crack-arrest techniques should be based on results of tests of specimens that are more nearly equal in size to concrete members found in actual structures and not on results of tests of small or essentially model specimens that have generally been used for investigating cracking within most conventionally reinforced concrete members.

g. Although sustained loading does not appear to affect the ultimate loads of members using any of the tentatively recommended crack-arrest techniques (steel fibers in the area of uniform tension only, steel fibers throughout the entire cross section, and properly positioned wire-mesh cages), it does significantly affect crack patterns. Therefore, conclusions regarding the effectiveness of any potential technique should be based on the results of sustained testing rather than on the results of standard short-term tests which are generally conducted on such members.

h. Neglecting cost, placing steel fibers throughout the entire cross section appears to be the best technique. However, the desired results may be obtained with either of the two remaining techniques. Therefore, economy as well

as effectiveness should be investigated before a technique is recommended for practical use.

i. The crack-arrest techniques examined in this investigation can be concluded to reduce maximum crack widths by 27 to 60 percent of those expected in conventionally reinforced members; however, it is emphasized that these conclusions are based on a limited number of tests.

PREFACE

This report presents the results of a literature study, the short-term static tests of fourteen 4- by 9- by 72-in. simply supported beams, ten of which used a potential crack-arrest technique in addition to their conventional reinforcement, and the short- and long-term static tests of eight similar but larger (5- by 14- by 180-in.) beams, six of which used a crack-arrest technique which had exhibited considerable potential during the small-beam investigation.

This investigation was authorized by letter from the Office, Chief of Engineers, U. S. Army, to the Director of the U. S. Army Engineer Waterways Experiment Station (WES) dated 10 April 1969, subject: Project Plan for Investigation of Crack-Arrest Techniques in Reinforced Concrete Structural Elements (ES Item 026.3). All work was performed at the Concrete Laboratory, WES, during the period May 1969 through August 1973 under the direction of Messrs. B. Mather, J. M. Polatty, J. E. McDonald, D. E. Harrison, and F. B. Cox. This report was prepared by Mr. Cox.

Directors of WES during the conduct of the investigation and preparation and publication of this report were COL L. A. Brown, CE, BG E. D. Peixotto, CE, and COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

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CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
square inches	6.4516	square centimetres
cubic feet	0.0283168	cubic metres
cubic yards	0.764555	cubic metres
pounds (mass)	0.45359237	kilograms
tons (2000 lb-mass)	907.185	kilograms
pounds (mass) per cubic foot	16.0185	kilograms per cubic metre
inch-pounds (force)	0.112985	newton-metres
pounds (force) per square inch	0.00689476	megapascals
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
revolutions per minute	0.0167	hertz

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

CRACK-ARREST TECHNIQUES IN REINFORCED CONCRETE STRUCTURAL ELEMENTS

LABORATORY TESTS

PART I: INTRODUCTION

Background

1. The tensile strength of concrete is approximately one-tenth of its compressive strength. The tensile strain at which concrete generally fails is well below the strain capacity of the tensile reinforcement that is provided in all conventionally reinforced concrete members. Thus, under normal service loads, it is reasonable to assume that the tensile zone of most conventionally reinforced concrete members will be cracked.

2. These cracks can provide direct access to the conventional (steel) reinforcement for environmental factors resulting in corrosion of the reinforcement and subsequent spalling of the concrete cover. The spalling of the concrete cover causes unsightly and possibly unsafe structures. Therefore, during the past several years, considerable emphasis has been placed on determining means of controlling or minimizing the tensile cracking of conventionally reinforced concrete members.

3. Most of the available literature indicates that laboratory investigations that have had varying degrees of success have been conducted on controlling or minimizing undesired tensile cracking by use of (a) fiber-reinforced concrete,[1,2] (b) various sizes and types of reinforcing bars,[3,4,5] (c) various percentages of tensile reinforcement,[3,4] and (d) epoxy or polyester resin concrete* in the tensile zones of composite concrete beams.[6] Unfortunately, however, most of the literature lacks detailed information on the feasibility of minimizing or

* The term "resin concrete" is applied to concretes in which resin instead of portland cement is used as a binder for the aggregate particles.

arresting tensile cracking by using combinations of any of these techniques (such as conventionally reinforcing a fiber-concrete mixture). Therefore, this investigation was concerned with new techniques that previous investigators have indicated are potentially worthwhile as well as with further evaluation of the more promising techniques previously used.

Objective and Scope

4. The objective of this investigation was to determine feasible techniques for limiting the size and spacing of reinforced concrete tensile (flexural) cracks to such magnitudes that the danger of corrosion of the steel reinforcement is minimized.

5. The work was conducted in the following phases:

   a. Phase I. A study of the literature was made, with special emphasis being placed on the various materials and methods that might be used to arrest cracking in reinforced concrete members.

   b. Phase II. Fourteen 4- by 9- by 72-in.* beams were fabricated, ten of which used a separate crack-arrest technique that was indicated effective by either the available literature or by previous testing.

   c. Phase III. Eight larger (5- by 14- by 180-in.) beams, including two control beams, were cast so that the most promising crack-arrest techniques indicated during Phase II could be further evaluated.

   d. Phase IV. The eight beams cast during Phase III were then divided into two similar groups, each consisting of a conventionally reinforced control beam, a conventionally reinforced beam with a 14-gage (2.03 mm) 1- by 1-in. wire-mesh cage positioned within its inner 84-in. section, a conventionally reinforced beam with 0.5-in.-long steel

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 8.

fibers (2.5 percent by volume) added throughout its entire cross section, and a conventionally reinforced beam with 0.5-in.-long steel fibers (2.5 percent by volume) added to its lower 3-in. tensile zone. Group 1 beams were tested under short-term static loading; group 2 beams under long-term static loading.

e. Phase V. Conclusions were then drawn regarding the effectiveness of each technique by comparing the results of all tests.

6. Since this is a continuing investigation, all conclusions and design approaches are preliminary and may be revised as the study continues and new results become available.

PART II: FABRICATION AND TESTING PROGRAM OF SMALL BEAMS

7. Fourteen (four control and ten using various crack-arrest techniques) 4- by 9- by 72-in. beams were cast and tested to investigate the most promising methods for arresting or minimizing flexural cracking of reinforced concrete structural elements.

Concrete Materials and Mixture Proportions

Plain concrete mixture

8. The materials used in the plain concrete mixture were type II portland cement (RC-622) manufactured in Alabama and crushed limestone (coarse and fine) aggregate from Tennessee (CRD-G-31(7), CRD-MS-17(3)).

9. A concrete mixture (table 1) was proportioned with a 3/8-in. maximum-size aggregate to have a slump of 2 ± 1/2 in. and a 28-day compressive strength of 3000 psi. The resulting compressive and tensile strengths of the various batches of concrete are presented in table 2, and the compressive stress-strain characteristics are given in plate 1.

Fiber-reinforced concrete mixture

10. The procedure for obtaining a uniform fiber-concrete mixture was as follows:

- a. The conventional concrete mixture described above was mixed in a 4-cu-ft-capacity rotary mixer by the normal procedure.
- b. The steel fibers (CRD-S-F-(1),(2)) were separated (fibers had previously settled into a tight ball due to shipping and previous handling) and placed into the regular concrete mixture in small quantities. This procedure ensured a uniformly distributed mixture with a minimum of additional mixing time.

11. The compressive stress-strain characteristics and the tensile and compressive strengths of the individual batches are shown in plate 1 and table 3, respectively. Structural properties of the steel fibers used are given in table 4.

Epoxy-resin concrete mixture

12. Due to its known qualities[7] and availability, a two-component polysulfide epoxy compound (EP-WES-B-10) meeting CRD-C 590-65 standards (Federal Specification MM-G-650a)[8] was selected as the binder for the aggregate used in fabricating the resin-concrete mixture used throughout this investigation.

13. Previous testing by Geymayer[6] indicated that a continuously graded, dry, clean, siliceous sand, fine aggregate (WES-1-S(4)47) (table 5) with a 14 percent resin content generally produced the most economical mixture, as well as the best strength properties. Therefore, this particular mixture was selected for use during this investigation. Each individual epoxy-resin concrete mixture was initially mixed in a 1-cu-ft-capacity vertical mixer and then given additional hand-mixing.

14. Following this particular design and mixing procedure resulted in the epoxy-resin concrete strengths and compressive stress-strain characteristics shown in table 6 and plate 1, respectively.

Fabrication and Curing of Concrete Specimens

Plain, plain plus fiber, and plain plus wire mesh specimens

15. Steel forms were used for casting all beams. The concrete for all beams and strips (layers) except the epoxy-resin concrete strips was consolidated in thin layers by a small electric vibrator with a frequency of 7000 rpm. In addition, the beams and strips were then placed on a vibrating table and vibrated briefly.

16. All beams, strips, and associated cylinders (except the epoxy-resin concrete cylinders and the composite beams consisting of strips or layers of epoxy resin and plain concrete within their tensile and compressive zones, respectively) were finished with a wooden float, covered with waterproof paper, stripped at a 48-hr age, and then moist-cured for an additional 12 days. After the moist-curing period, the specimens were allowed to cure under room conditions, approximately 73 ± 2 F and 50 to 60 percent relative humidity, until their test date.

Epoxy-resin concrete specimens

17. The plain concrete for the epoxy-resin concrete composite beams was cast in an inverted position, moist-cured in its forms for 14 days, and then cured under room conditions, approximately 73 ± 2 F and 50 to 60 percent relative humidity, for an additional 7 days. Preceding the application of the resin-concrete layers at the 21-day age, all concrete and reinforcement surfaces to be in contact with the epoxy-resin concrete were sandblasted and then painted with the epoxy. Finally, the epoxy-resin concrete was placed in thin layers and compacted with both mechanical tampers and tamping rods. The beams were then allowed to cure under the room conditions described above until their test date.

Beam Test Methods

18. All small beams were simply supported and tested to failure in an inverted position (fig. 1) by third-point static loading. Each

Fig. 1. Typical arrangement used for testing beams

beam was supported at its third-points by a half rocker system (load and support reactions were distributed to each beam by 1/8- by 1-in. pads placed between the concrete and steel rollers), with loads being placed by a combination of a hydraulic system and two 20-ton-capacity jacks that were positioned to provide a clear span of 6 ft for each test specimen. A control panel, a load cell, a displacement transducer, and an x-y recorder (fig. 1) were used to measure the loads and to obtain a continuous load versus end-span deflection plot for each beam. Midspan deflections, as well as verifications of end-span deflections, were made with three 1-in. travel length dial gages positioned as shown in fig. 1.

19. Total loads were applied in 500- to 1000-lb increments. After the application of each increment, readings of beam strains, deflections, and crack widths were made, and observations of other behavior were noted. The loads were completely removed at least twice during most of the tests so that inelastic deflections could be checked for each beam prior to continuation of loading.

20. Concrete strains were measured by (a) a 2-in. mechanical strain gage using Demec standard measuring disks located at previously designated points (fig. 2) and (b) by SR-4 electrical strain gages placed at the midspan of both the conventional tensile reinforcement and the compressive, top, face of the beam.

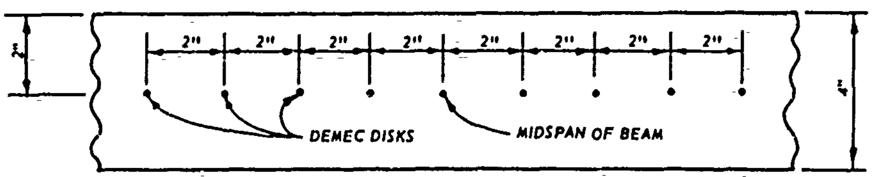

Fig. 2. Locations of Demec standard measuring disks along tensile face of beam

21. All cracks were marked and measured as they occurred. They were measured using a special electrically illuminated 40X microscope and the Demec disks.

Beam Test Results

22. The behavior of individual beams and the principal results

of individual tests are summarized in tables 7-20 and plates 2-13. The beams are described briefly as follows.

Group 1

23. Group 1 consisted of three beams, one of which was used as a control specimen. The control beam (beam 1) was reinforced for flexure to approximately 55 percent of a balanced design (P_b) according to the ACI Code[9] by using two No. 4 high-strength (grade 60) reinforcing bars as the tensile reinforcement (fig. 3). The shear requirement of the Code[9] was satisfied by using No. 2 vertical stirrups (fig. 4) on 4-in. centers throughout the sections of the beam requiring shear reinforcement.

24. The two remaining beams (2 and 3) of group 1 used the same plain concrete mixture and conventional reinforcement as described for the control beam; however, the concrete mixtures for beams 2 and 3 were additionally reinforced with 2.5 percent (by volume) steel fibers of 0.5- and 1.0-in. lengths, respectively. Information on properties of the fibers is given in table 4.

25. Test results for beam 1 are given in table 7, photo 1, and plates 2, 6, and 10 and are discussed briefly below:

a. A total load of 2000 lb resulted in the tensile zone of the concrete being strained to such a degree that a hairline crack appeared on the tensile face of the beam. The average tensile reinforcement and tensile face concrete strains were 215 and 220 millionths, respectively. However, a load of 2500 lb, resulting in average tensile reinforcement and concrete tensile strains of 260 and 300 millionths, respectively, was required before the first cracks, which had a maximum width of 0.0015 in. and a maximum depth of 1.15 in., were of such a depth that they could be marked and photographed. From this load level, the strains and resulting cracks continued to grow at an increasing rate until they reached the magnitudes of 1790 millionths, strain in tensile reinforcement; 2040 millionths, strain at tensile face of

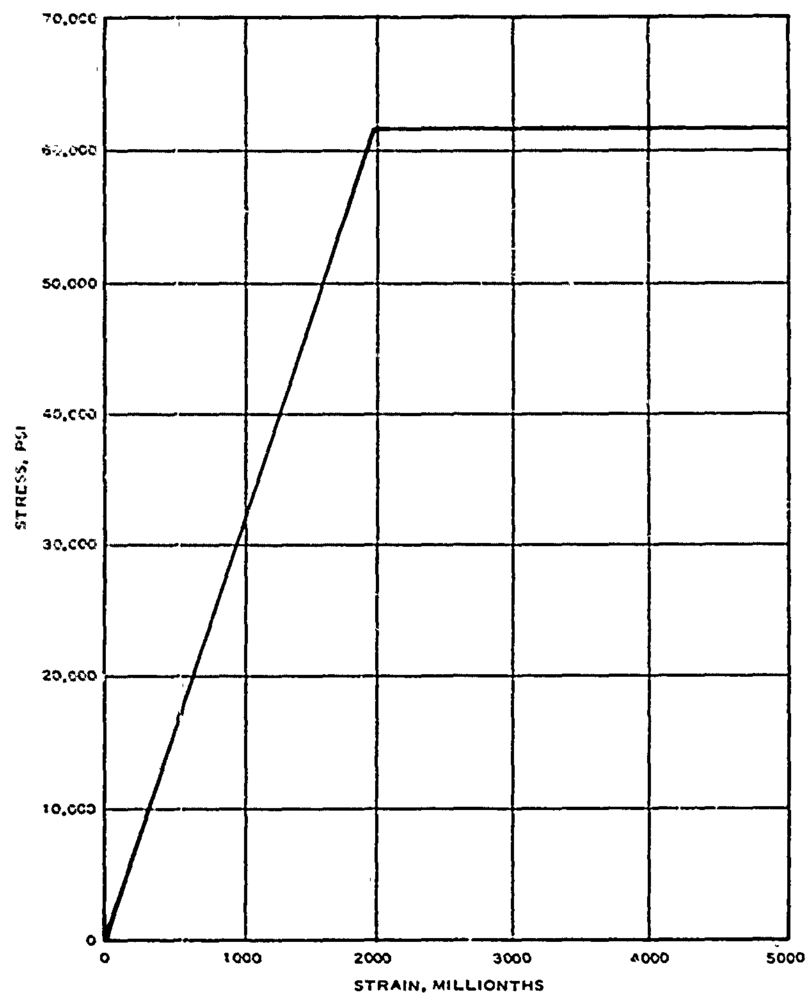

Fig. 3. Stress-strain curve for No. 4 bars used as flexural reinforcement

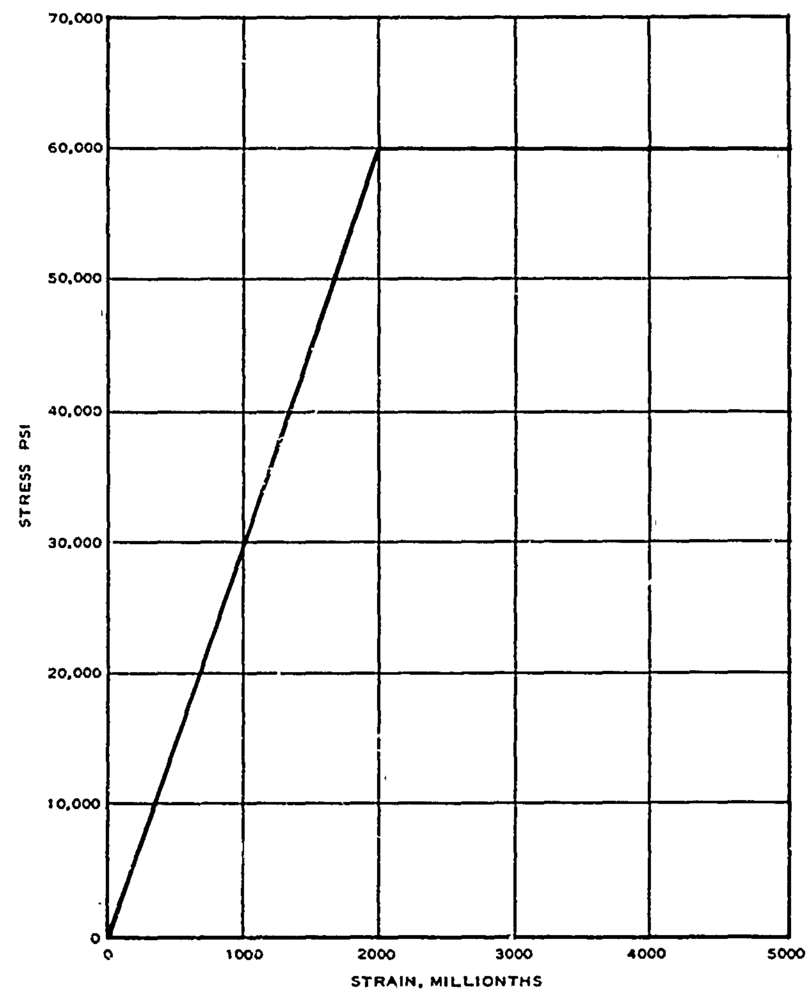

Fig. 4. Stress-strain curve for No. 2 stirrups used as shear reinforcement

concrete; 0.0040 to 0.0055 in., width of crack; and 5.20 in., depth of crack, at a load level of 12,500 lb. The load-carrying capabilities of the beam then began to change rather rapidly, with a flexural compressive failure occurring at a total load of 13,450 lb. The terminology is in accordance with Bjuggren.[10]

b. A midspan deflection of 0.589 in. was recorded just prior to failure. This represents a deflection ratio of approximately L/122.

c. The failure moment of 162,539 in.-lb, including weight of beam, was approximately 5.2 percent higher than the 154,495-in.-lb moment predicted by the ACI Code.[9] This shows, as suspected, a rather close agreement between the calculated and measured moments.

26. Test results for beam 2 are shown in table 8, photo 2, and plates 2, 6, and 10 and are discussed below:

a. The first tensile strains, 520 millionths for tensile reinforcement and 610 millionths for the tensile face of the beam, which resulted in hairline cracks, maximum width 0.0018 in., on the tensile face of the beam, occurred at a total load of 5500 lb. However, a load of 6000 lb was required before the strains, 710 and 610 millionths for the tensile face and reinforcement, respectively, reached such a magnitude that the depth, 0.34 in., of the resulting cracks, width 0.002 in., was sufficient for marking and photographing. Thereafter, the tensile strains and cracks continued to grow rather uniformly under the increasing test loads until their maximum measurable dimensions of (1) 2040 millionths for the tensile reinforcement, (2) 2270 millionths for the tensile face of beam, (3) 0.0045 to 0.0053 in. for width of major crack, and (4) 4.05 in. for the depth of the major crack, were reached under a total load of 13,000 lb. A flexural compressive failure then occurred under a

slightly higher load of 13,700 lb. Further examinations of the photographs taken immediately after the failures of beams 1 and 2 (photos 1j and 2i) revealed, as suspected, that more but generally smaller cracks occurred within the outer tensile zone of beam 2. However, the major crack width was essentially the same as for beam 1, and the crack depth was slightly less.

b. A total midspan deflection of 0.251 in. was measured just prior to failure. This represents a deflection ratio of approximately L/287, which was a considerable improvement over that for beam 1.

c. The failure moment for beam 2 was 165,539 in.-lb. This was only a slight increase over the 162,539-in.-lb moment required to fail beam 1.

27. Test results for beam 3 are shown in table 9, photo 3, and plates 2, 6, and 10 and are discussed below:

a. Initial hairline cracking, maximum width of 0.0015 in., depth was not measurable, was observed at a total load of 4000 lb, or approximately 2.00 and 0.73 times the loads required to initiate cracking in beams 1 and 2, respectively. At the 4500-lb load level, strains, 425 millionths for tensile reinforcement and 465 millionths for tensile face of beam, were sufficient to produce cracking, width of 0.0010 to 0.0019 in. and depth of 0.95 in., with depths of the magnitudes required for proper side marking and photographing. As in beam 2, there was a rather uniform growth of the strains and resulting cracks from this point until they reached their maximum measurable values of 2000 millionths for the strain of the tensile reinforcement, 2350 millionths for the strain on the tensile face of the beam, 0.0050 to 0.0088 in. for the maximum crack width, and 4.38 in. for the maximum crack depth, at a load of 14,000 lb, slightly below its ultimate load-carrying capacity of 14,520 lb. Again, as in

the previous tests, the photos (photo 3b) show that the beam failed in a flexural compressive mode, with the final crack pattern of its tensile face showing more and smaller cracks when compared with beam 1, but fewer and larger cracks when compared with beam 2. This, along with other test results,[1] indicates that a conventionally reinforced concrete beam with 1/2-in.-long steel fibers added to its prescribed mixture (table 1) may be superior to a like member using 1-in.-long steel fibers when only crack-arrest techniques for static loading are considered.

b. The failure moment for beam 3 was 175,379 in.-lb, which was slightly greater than the failure moments for beams 1 and 2.

c. A total midspan deflection of 0.308 in. was measured just prior to failure. This represents a deflection ratio of approximately L/234, which is in close agreement with the ratio found for beam 2.

Group 2

28. This group consisted of (a) beam 4, a conventionally reinforced control beam that was similar to beam 1; (b) beam 5, which was similar to beam 4 except that (1) the number of conventional stirrups was reduced, and (2) a 3- by 8- by 36-in. wire-mesh cage, fabricated from No. 14 gage (2.03 mm), 1- by 1-in. galvanized wire mesh, was used as a potential crack arrester throughout the inner 36-in. section of the beam (fig. 5); (c) beam 6, a beam maintaining reinforcement identical with that of beams 1 and 4, but its lower 1-1/2 in. consisted of epoxy-resin concrete; and (d) beam 7, a beam similar to beam 6 with the

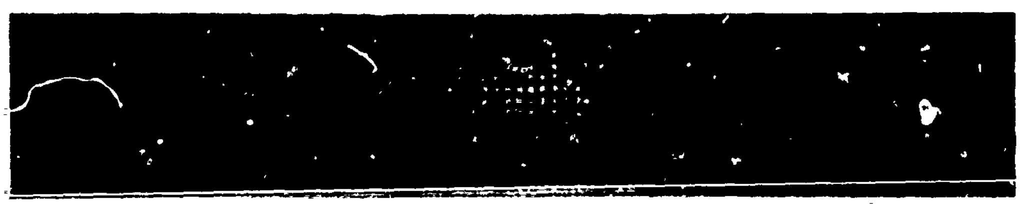

Fig. 5. Reinforcement for beam 5 (reinforcement is in as-cast position)

only difference being the depth of the epoxy-resin concrete, which was 3 in.

29. Test results for beam 4 are shown in table 10, photo 4, and plates 3, 7, and 11 and are discussed briefly below:

a. Average tensile reinforcement and tensile concrete strains at the tensile face of 305 and 310 millionths, respectively, produced the first noticeable cracking, 0.0015-in. maximum width, 2.76-in. maximum depth, at a total load of 2500 lb. When compared with control beam 1, this represented a slight increase in the initial cracking load, although table 2 shows a considerable decrease in the ultimate compressive strength of the concrete used in fabricating beam 4. This particular situation further indicates the inability of predicting the tensile strength of concrete. Also, as in most previous testing, there was a rather uniform growth in the initial strains and resulting crack patterns for the next several loading levels, with the largest changes of 1775 to 2190 millionths, strain of tensile reinforcement, 2220 to 2640 millionths, average strain on tensile face of beam 0.0065-0.0090 in. to 0.0180-0.0125 in., maximum crack widths; and 4.62 to 4.63 in. maximum crack depths occurring between the 11,000- and 12,000-lb levels. A flexural compressive failure then occurred at a slightly higher load of 12,700 lb.

b. A midspan deflection of 0.319 in. occurred just prior to failure. This represents a deflection ratio of approximately L/226, which is a considerable improvement over the deflection ratio of approximately L/122 found for comparable beam 1.

c. The failure moment of 153,539 in.-lb was approximately 5.5 percent higher than the 145,505-in.-lb moment predicted by the ACI Code.[9] As with beam 1, there was

rather close agreement between the predicted and tested failure moments.

30. As previously mentioned, a 3- by 8- by 36-in. galvanized wire-mesh cage, 1- by 1-in. squares (fig. 5) was placed in the inner 36-in. section of beam 5. This potential crack-arrest technique was selected because some of the literature,[3,4,5,11] indicated that the type, size, and spacing of the reinforcement can considerably alter the resulting cracking patterns of reinforced concrete structural members.

31. Test results for beam 5 are shown in table 11, photo 5, and plates 3, 7, and 11 and are discussed briefly below:

a. A load of 4000 lb was required to initiate the cracking of the beam; however, the strains, 335 millionths for tensile reinforcement and 440 millionths for average strain of tensile face of beam, were of such a magnitude that a resulting crack (photo 5b) opened to a maximum width of 0.001 to 0.0013 in. and penetrated to within 5 in. of the beam's compressive face. From this load level, the number of cracks began to increase rather rapidly, with most of the cracks forming and following a line just outside the vertical wires of the cage. Although most of the cracks did form as previously mentioned, just outside the vertical wires of the cage, strains of 1305 and 1720 millionths for the tensile reinforcement and face of the beam, respectively, produced a flexural shear crack[10] of significant magnitude, 0.002 to 0.0049 in. in maximum width and 4.74 in. in depth, at the 10,000-lb load level (photo 5g). This would indicate that the cage should not be designed and used as a replacement for conventional shear reinforcement; however, this, or the flexural shear cracks that formed during the later test stages, did not appear to affect the flexural compressive failure of the beam, which occurred at a total load of 14,800 lb. A study of tables 7 through 20 shows that the maximum widths, 0.003 to

0.0068 in., of beam 5's cracks were smaller than those of any other beam just prior to failure. This is also revealed by the final crack pattern of beam 5's tensile face (photo 5k).

b. A midspan deflection of 0.306 in. occurred just prior to failure. This is a deflection ratio of approximately L/235, which is approximately the same as that for other previously tested beams with nearly equal load-carrying capabilities.

c. The failure moment of 178,739 in.-lb was considerably higher than that of control beam 4 (153,539 in.-lb).

32. Geymayer[6] concluded that a composite beam consisting of conventional concrete and epoxy-resin concrete within its compressive and tensile zones, respectively, would considerably increase the beam's initial cracking load. Therefore, beam 6, which was similar to control beam 4 except that the bottom 1-1/2 in. was fabricated with an epoxy-resin concrete, was fabricated and tested to further evaluate this conclusion.

33. Test results for beam 6 are shown in table 12, photo 6, and plates 3, 7, and 11 and are discussed briefly below:

a. The first cracking occurred at a total load of 5000 lb. One crack was of such a magnitude, 0.002 in. wide and 2 in. deep, that the entire epoxy-resin layer was penetrated. From this point, the sizes of the strains and cracks grew rather rapidly as the loads were increased, with measurements of 1355 millionths, 1100 millionths, 0.010 to 0.0113 in., and 3.90 in. being noted, respectively, for the tensile reinforcement strain, average maximum tensile strain over the middle one-third section of the epoxy-resin concrete layer, maximum widths of the cracks, and maximum depths of the cracks, at the 8000-lb load level. Strains of 2320 millionths in the tensile reinforcement and 2570 millionths, average maximum tensile strain of epoxy-resin concrete, were noted at the

11,000-lb load level. This resulted in the larger cracks opening up to maximum widths and depths of 0.0433 to 0.045 and 4.60 in., respectively, which were greater than any crack widths found for any previously tested beam. A flexural compressive failure occurred at a slightly higher load of 12,000 lb.

b. A midspan deflection of 0.274 in. occurred just prior to failure. This represents a deflection ratio of L/262, which is about the same as that for control beam 4.

c. The failure moment, 145,139 in.-lb, was approximately 95 percent of the failure moment for the control beam, 153,539 in.-lb.

34. A comparison of the test results for beam 4, the control beam, and beam 6 indicated that the load required to initiate flexural cracking was increased from 2500 to 5000 lb by the 1-1/2-in. layer of epoxy-resin concrete. Therefore, beam 7 was fabricated with a 3-in. layer of epoxy-resin concrete within its tensile zone to further investigate this promising crack-arrest technique.

35. Test results for beam 7 are shown in table 13, photo 7, and plates 3, 7, and 11 and are discussed briefly as follows:

a. Initial cracking occurred at a total load of 6000 lb. As with beam 6, the cracks were of such a magnitude, 0.008-in. maximum width and 3.32-in. depth, that the entire layer of epoxy-resin concrete was penetrated. Although the strains changed considerably, 305 to 1820 millionths for the tensile reinforcement and 360 to 1960 millionths for the outer tensile face of the epoxy-resin concrete, from the 6,000- to the 11,000-lb load level, there was a somewhat lesser change, 0.008 to 0.017 in. for the maximum crack widths and 3.32 to 5.22 in. for the crack depths, in the corresponding crack pattern. Therefore, due to this particular situation, as well as the failure of the beam at the next loading increment, 12,000 lb, it is emphasized that the test results indicate that more

important than their influence on strength is the ability of resin-concrete layers to provide a noncracking moisture barrier or corrosion protection for beams under their normal service loads.

b. A midspan deflection of 0.216 occurred at the 11,000-lb load level, which was just prior to failure. This deflection ratio of L/333 shows a considerable improvement over the L/262 ratio found for beam 6 and tends to indicate that the thicker epoxy-resin concrete layers may provide higher precracking loads.

c. The ultimate tested moment of 145,139 in.-lb was the same as that found for beam 6 and approximately 95 percent of that found for the control beam.

Group 3

36. The beams of this group consisted of (a) beam 8, which was similar to beams 1 and 4 and was used as the control beam for group 3; (b) beam 9, which was similar to beam 8 except that it had a double layer of wire mesh attached to its outer surface of tensile reinforcement (fig. 6); and (c) beam 10, which was similar to beam 5 of the second group except that the vertical wires of the 1- by 1-in. mesh cage were clipped (fig. 7) throughout the middle one-third section of the beam, so that, in effect, they had been removed.

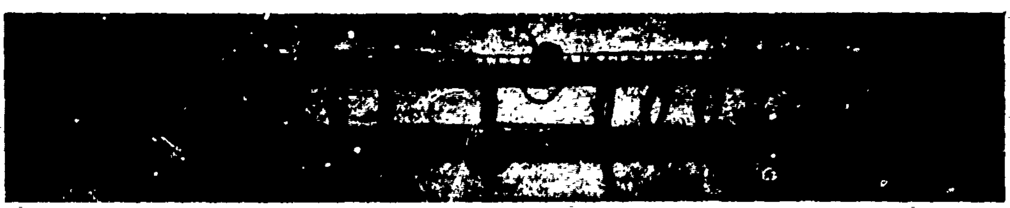

Fig. 6. Reinforcement for beam 9 (reinforcement is in as-tested position)

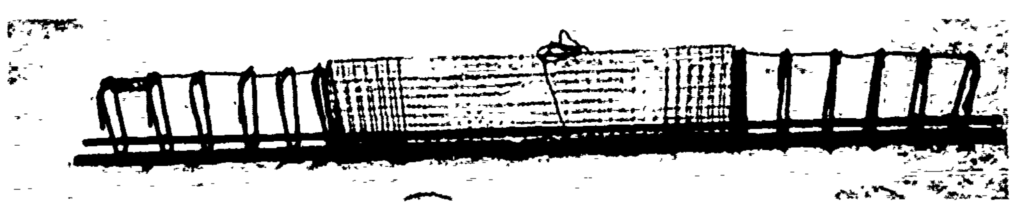

Fig. 7. Reinforcement for beam 10 (reinforcement is in as-cast position)

37. Test results for beam 8 are shown in table 14, photo 8, and plates 4, 8, and 12 and are discussed briefly as follows:

a. Initial cracking occurred at a total load of 2500 lb, which was the same load required to initiate the cracking of the control beam, beam 4, of group 2. However, table 14 shows that at this load, the strains, 160 millionths in the tensile reinforcement and 245 millionths in the concrete at tensile face of beam, and resulting cracks, 0.0005 to 0.0011 in. wide and 1.15 in. deep, were considerably less than those found for beam 4. At the 8000-lb load level, the strains of the tensile reinforcement and tensile face of the beam had increased to 1295 and 1565 millionths, respectively, which resulted in the cracks opening to a maximum width of 0.0045 to 0.0056 in. The crack depth was 4.6 in. at this particular load level and remained near this depth until just before the failure load of 11,000 lb was reached. Photo 8ℓ shows that the beam failed in a flexural compressive mode, which is the mode of failure that would be suspected for all similar underreinforced beams.

b. A midspan deflection of 0.269 in. occurred just prior to failure. This represents a deflection ratio of approximately L/268, which is a considerable change from the deflection ratios found for control beams 1 and 4 of groups 1 and 2. However, this can be partially explained by beam 8's rather low failure load of 11,000 lb.

c. The failure moment of 133,139 in.-lb is approximately 92 percent of the 144,950-in.-lb moment predicted using the ACI Code.[9] This, again, indicates that the beam failed at a moment (load) slightly less than would generally be expected for such members.

38. One technique previously used in attempting to control the cracking of reinforced concrete beams was to provide a wire-mesh cage (fig. 5) throughout the section of a beam (beam 5) where the

probability of cracking was the greatest; however, it was found that the flexural cracks generally followed a line just outside the vertical wires of the cage. Therefore, beam 9, fabricated with a double layer of the same 1- by 1-in. wire mesh attached to its conventional tensile reinforcement (fig. 6), was the first attempt to improve the technique used in beam 5.

39. Test results for beam 9 are shown in table 15, photo 9, and plates 4, 8, and 12 and are discussed briefly as follows:

a. Average tensile strains of 105 millionths for the conventional tensile reinforcement and 180 millionths for the tensile face of the beam resulted in the initial cracking of the beam at a total load of 2500 lb, which was the same load required to initiate the cracking of the control beam (beam 8). Also, table 15 reveals that these cracks were of about the same magnitude (0.0007 to 0.0010 in. wide and 1.67 in. deep), if not slightly greater, than those found for the control beam. A comparison of tables 14 and 15 indicates that at corresponding loads the dimensions of the strains and resulting cracks of beam 9 were nearly the same as those for beam 8. Although each beam failed in the same flexural compressive mode, a somewhat higher load of 13,000 lb was reached before beam 9 failed. This increased failure load was attributed to the double layer of wire mesh providing additional flexural reinforcement.

b. A midspan deflection of 0.294 in. was reached just as the load level reached 13,000 lb; however, this load resulted in the tensile reinforcement yielding, and a larger deflection of 0.408 in. was recorded at this same load level just prior to failure. The 0.294-in. deflection represents a deflection ratio of approximately L/245, which is about what should be expected for conventionally reinforced beams subjected to test loads within the 13,000-lb range.

c. The failure moment of 157,139 in.-lb represented a moment increase of 24,000 in.-lb over that of the control beam (beam 8). Again, this is attributed to the double layer of wire mesh providing additional flexural reinforcement.

40. Beam 10, using a 6- by 8- by 36-in. wire-mesh cage similar to that described for beam 5, except that the vertical wires were clipped (removed) throughout its inner 24-in. section (fig. 7), was the third attempt (beams 5 and 9 were the first and second, respectively) to employ wire mesh as an effective flexural crack arrester for reinforced concrete members.

41. Test results for beam 10 are shown in table 16, photo 10, and plates 4, 8, and 12 and are discussed briefly as follows:

a. Strains of 250 and 315 millionths for the conventional tensile reinforcement and the tensile face of the beam, respectively, resulted in the beam cracking (0.0005 to 0.0011 in. wide, 0.55 in. deep) at a total load of 3500 lb. This represents a decrease of 500 lb from the initial cracking load for beam 5 (a similar beam) and an increase of 1000 lb over the initial cracking load of beam 8 (control beam) and beam 9 (beam with double layer of wire mesh attached to the outer surface of its conventional tensile reinforcement). However, a comparison of tables 11, 14, 15, and 16 indicates, as suspected, somewhat higher degrees of differences between the corresponding strains of beams 5, 8, 9, and 10 under the initial cracking load levels. Table 16 shows a rather uniform growth in both the strains and cracks between beam 10's initial cracking load of 3500 lb and the 10,000-lb load level, and then an increasing growth (between load levels) in both strains and cracks until they reach magnitudes of (1) 1840 millionths for the strains of the conventional reinforcement, (2) 2485 millionths for the strains on the tensile face of beam, (3) 0.0025 to 0.0079 in. for the maximum crack widths, and (4) 4.90 in.

for the crack depths at the 14,000-lb load level. The beam then experienced a flexural compressive failure (photo 10o) at the 15,000-lb load level. However, table 16 indicates that the yielding of the reinforcement was of such a degree that the beam could not sustain the load nor produce any useful data (readings); therefore, for all practical purposes, the failure of the beam occurred before the 15,000-lb load level was reached.

b. A midspan deflection of 0.267 in. occurred just prior to failure. This represents a deflection ratio of approximately L/270, which is an improvement over the ratios found for the other beams of this group, although beams 8 and 9 failed under lower loads.

c. The failure moment of 181,139 in.-lb represents an increase over the failure moment of any of the comparable beams (beams 5, 8, and 9); however, it is again emphasized that for all practical purposes, beam 10 actually failed between 169,139 in.-lb and the recorded failure moment of 181,13? in.-lb.

Group 4

42. Group 4 consisted of four beams (beams 11, 12, 13, and 14), which are briefly described as follows:

a. Beam 11 (photo 11) was similar (used identical reinforcement) to beams 1, 4, and 8; therefore, it was used as the control beam for this group.

b. Beam 12 used reinforcement identical with that in beam 11; however, the lower 2-1/2 in. of its tensile zone was fabricated with epoxy-resin concrete (photo 12) in lieu of the conventional portland cement concrete mixture.

c. Beam 13 was similar to beam 12 except that 3 in. of epoxy-resin concrete (photo 13) was used in its fabrication instead of 2-1/2 in. as for beam 12.

d. The last beam (beam 14) was provided the normal reinforcement employed in all control beams (beams 1, 4, 8,

and 11) plus 1/2-in.-long steel fiber (2.5 percent by volume) reinforcement within the lower 3 in. of its tensile zone.

43. Test results for beam 11 are shown in table 17, photo 11, and plates 5, 9, and 13 and are briefly described as follows:

a. A load of 4000 lb was required to produce strains of such a magnitude (340 and 360 millionths for the reinforcement and tensile face of the beam, respectively) that cracking (0.0013 to 0.0020 in. wide, 4.74 in. deep) began within the tensile zone of the beam. A comparison of tables 7, 10, 14, and 17 shows that the load required to initiate the cracking of similar control beams (beams 1, 4, and 8) was as much as 50 percent lower. Therefore, this further indicates the difficulty of predicting the flexural (tensile) strength of conventionally reinforced concrete beams. The strain and resulting cracks continued to grow under increasing loads, with a significant change occurring in both at loads ranging from 7000 to 8000 lb (table 17). At the 12,000-lb load, the strains (1695 and 1990 millionths for the tensile reinforcement and face of the beam, respectively) and cracks (0.0060 to 0.0081 in. wide, 5.60 in. deep) were of such a magnitude that yielding of the tensile reinforcement was becoming evident. This resulted in a flexural compressive failure at the next higher loading level of 13,000 lb.

b. A midspan deflection of 0.223 in. occurred at the 12,000-lb load level. This represents a deflection ratio of approximately L/323, which is within the range indicated by the previously tested control (similar) beams.

c. The failure moment of 157,139 in.-lb was approximately 4.5 percent greater than the 150,290-in.-lb ultimate moment predicted using the ACI Code.[9] This, again, indicates the close agreement that is generally found between

the actual and predicted failure moments of conventionally reinforced concrete beams.

44. Beam 12 was intended to be identical with beam 6 in order that further evaluations could be made of beams using small depths of epoxy-resin concrete, in lieu of regular concrete, within the zones subjected to the maximum tensile stresses. However, a 2-1/2-in. depth of epoxy-resin concrete was unintentionally substituted for the intended 1-1/2-in. depth used in beam 6.

45. Test results for beam 12 are shown in table 18, photo 12, and plates 5, 9, and 13 and are discussed briefly as follows:

a. A total load of 12,000 lb was required to initiate the cracking of beam 12. However, the tensile strains were of such a magnitude (610 and 980 millionths for the tensile reinforcement and tensile face of the beam, respectively) that cracks opened up to a maximum width of 0.021 to 0.0250 in. and a maximum depth of 6.37 in. at this load level. Since the reinforcement of the beam started yielding (table 18) before the 13,000-lb load level was reached, the only useful data were collected up to and including the 12,000-lb load level. Therefore, for all practical purposes the beam actually failed during its initial stage of cracking, which further substantiates Geymayer's statement, "more important than their influence on strength is the ability of resin concrete layers to provide a noncracking moisture barrier or corrosion protection practically up to beam failure."[6] Although, as previously mentioned, for all practical purposes the beam actually failed near the 12,000-lb load level, it is emphasized that a load of 13,900 lb was required to completely collapse the beam.

b. A midspan deflection of 0.147 in. occurred at the 12,000-lb load level. This represents a deflection ratio of approximately L/490, which is a considerable improvement over that of most other beams under the same loading

levels. This increased deflection ratio further indicates the small amount of cracking that occurred prior to failure.

c. The collapse failure moment of 167,939 in.-lb is approximately the same as that reached by all the conventionally reinforced control beams (beams 1, 4, 8, and 11).

46. Beam 13, fabricated with a 3-in. layer of epoxy-resin concrete within its lower tensile zone, was similar to beam 7 of group 2. However, tests of this beam were deemed necessary in order that further evaluations could be made of the initial cracking loads of test specimens using various depths of epoxy-resin concrete within the zones subjected to the maximum tensile stresses.

47. Test results for beam 13 are shown in table 19, photo 13, and plates 5, 9, and 13 and are discussed briefly as follows:

a. Respective strains of 475 and 400 millionths by the conventional reinforcement and tensile face (average over middle one-third of beam) of the beam resulted in its initial cracking (0.0010 to 0.0034 in. wide, 3.20 in. deep) at a total load of 5000 lb. This represents (1) no change in the cracking load when compared with beam 6 with a 1-1/2-in. epoxy-resin concrete layer, (2) a slight decrease in the cracking load when compared with beam 7, and (3) a considerable decrease in the cracking load when compared with beam 12 with a 2-1/2-in. layer of epoxy-resin concrete. Therefore, these results indicate that there is not, as indicated initially, a corresponding increase in the initial cracking loads of like beams when the epoxy-resin concrete layer is varied between minimum and maximum depths of 1-1/2 and 3 in., respectively. The strains and resulting cracks grew at a rather uniform rate from their dimensions at the 5000-lb load level until they reached magnitudes of (1) 7500 millionths strain for the tensile reinforcement, (2) 4200 millionths average strain on the tensile face of the beam, (3) 0.0450 to

0.0523 in. for the maximum crack width, and (4) 6.30 in. for the maximum crack depth at a load of 12,000 lb. These magnitudes of the strains and cracks resulted in a flexural compressive failure just as the 12,700-lb load level was reached.

b. A midspan deflection of 0.328 was reached at the 12,000-lb load level, or just prior to failure. This represents a deflection ratio of approximately L/216, which is about what a majority of the other comparable beams would indicate for this load level.

c. The failure moment of 153,539 in.-lb represents an increase of approximately 5.8 percent over that found for beams 6 and 7 and a decrease of approximately 8.6 percent from that found for beam 12. This agrees with Geymayer's statement, "thicker (3 in.) layers* did not appear to more beneficially affect the moment capacity, possibly due to increasing internal stresses (shrinkage and temperature) in the thicker layers."[6]

48. Test results for beam 2 indicated that the initial cracking load of a small conventionally reinforced concrete beam would be increased considerably when 0.01- by 0.01- by 0.5-in. steel fibers (2.5 percent by volume) were added to the plain (normal) concrete mixture. Therefore, beam 14 was cast with a similar fiber/concrete mixture in its lower 3 in. and the normal concrete mixture throughout its remaining cross section in order that further evaluations could be made of this promising technique.

49. Test results for beam 14 are given in table 20, photo 14, and plates 5, 9, and 13 and are discussed briefly below.

a. Tensile strains of 560 millionths (average at midspan) for the tensile reinforcement and 510 millionths (average over middle one-third of beam) for the tensile face of the beam resulted in the initial cracking (0.0010 to

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* Geymayer means thicker epoxy-resin concrete layers.

0.0030 in. wide, 1.34 in. deep) of the test specimen at a load of 6000 lb. This is nearly the same as the load required to produce similar strains and initial cracks in beam 2, which indicates there is no particular advantage in providing the fiber/concrete mixture throughout the entire cross section of the beam. After the initial cracking occurred, the beam then reacted in a manner similar to that for the conventionally reinforced control beams until a flexural compressive failure occurred at a load of 14,000 lb.

b. A midspan deflection of 0.239 in. occurred at the 13,000-lb load level, or just prior to failure. This represents a deflection ratio of L/301, which is approximately the ratio that beam 2 and beam 11 (the control beam of this group) indicate as normal for a load level of this magnitude.

c. The failure moment of 169,139 in.-lb was approximately 2.2 percent greater than that of beam 2 and 7.6 percent greater than that of beam 11. Since this amount of variation can be expected for similar beams, it is believed that the addition of steel fibers (in the amount prescribed) to a plain concrete mixture will not significantly increase load-carrying capacity of beams cast from such a mixture over that of conventionally reinforced members.

PART III: SUMMARY OF TEST RESULTS FOR SMALL BEAMS

50. In this summary, the test results are compared with the available literature to provide a basis for using a particular crack-arrest technique with larger test specimens. The results are partially summarized in table 21 and plates 10, 11, 12, and 13 and are discussed briefly below.

Beams 2, 3, and 14 (Beams Using Steel Fibers as Crack Arresters)

51. Steel fibers increased the precracking loads from 1.50 to 2.75 times those of the conventionally reinforced concrete beams. However, their ultimate tensile straining (strain prior to cracking) capabilities were increased by factors of only 1.40 to 2.30. Therefore, steel fibers, as suspected, not only increase the tensile straining capabilities of a concrete mixture, but also the area under its tensile precracking load versus deflection curve, or, according to reference 12, its precracking toughness.

52. Probably due to the difficulty of obtaining a uniform mixture as the fiber length versus cross-sectional ratio increases, the shorter (0.5-in.-long) fibers produce the mixtures with the greatest tensile straining capabilities.

53. The addition of steel fibers did not significantly increase the ultimate flexural capacity of any beam. In fact, as shown in plates 10 and 13, immediately after cracking, the conventionally reinforced concrete beams using fiber mixtures reacted very similarly to conventionally reinforced beams using normal mixtures. However, it should be emphasized that the 0.5-in.-long fibers delayed initial cracking in the members until their maximum anticipated service loads were exceeded (Appendix A).

54. A comparison of beams 2 and 14 indicates that a fiber mixture placed only in the more critical tensile zones of a beam will probably be as effective in delaying the initial cracking as a fiber mixture placed throughout the entire cross section of a beam.

Beams 5, 9, and 10 (Beams Using Wire Meshes as Crack Arresters)

55. The use of a wire-mesh (fabric) cage with or without its vertical wires clipped and positioned in the critical zone (figs. 5 and 7) can increase both the tensile straining and the precracking load characteristics of small conventionally reinforced beams by as much as 1.40 to 1.60 times that of conventionally reinforced beams. A study of table 21 and plates 11 and 12 reveals that the cages not only increase the tensile strains and the precracking loads, but also aid in controlling (reducing) the sizes of the resulting cracks even at load levels greater than the flexural capacity of comparable conventionally reinforced (control) beams. However, double layers of wire mesh attached to the conventional reinforcement (fig. 6) cannot be expected to give similarly satisfactory results. Beam 9, due to its additional reinforcement (a double layer of wire mesh), reached higher flexural loads than those expected for similar conventionally reinforced members, but, unfortunately, the ability of the additional reinforcement to control the sizes of the cracks was somewhat limited.

56. The wire cage is believed to be the better of the two techniques using wire mesh due to its sizable area of reinforcement and to its greater height. Therefore, since the cage maintains both larger areas of reinforcement and heights than does the double layer of wire mesh attached to the conventional reinforcement, the wire mesh (fabric) of the cage distributes the tensile strains, stresses, and resulting cracks in a more satisfactory manner. Also, since the method of fabricating the cages apparently does not significantly affect its crack-arresting ability (especially up to maximum anticipated working loads), the method illustrated in fig. 5 (cages maintaining vertical wires) is believed to be the better because of its simplicity of fabrication.

Beams 6, 7, 12, and 13 (Beams Using Epoxy-Resin Concrete as Crack Arresters)

57. The routine testing conducted during this part of the investigation did not properly reflect the potential deficiencies of the

epoxy-resin concrete mixture with respect to shrinkage, thermal expansion, creep, sensitivity to environmental factors, exothermic characteristics, etc. Therefore, separate tests should be conducted to evaluate these properties before a mixture is selected for practical applications or before any final conclusions can be made based on these or similar test results.

58. Although, as mentioned, the test results are from routine test results only, the following indications resulting from this phase of the investigation are believed to be worthy of noting:

a. When compared with portland cement concrete, 1.50 to 3.00 in. of properly positioned epoxy-resin concrete generally increases the precracking loads of the beams from 125 to 240 percent. This represents loads that are from 1.04 to 1.44 times greater than maximum anticipated service loads. However, since only one beam (beam 12, table 21) delayed initial cracking beyond these levels, only partial agreement was found with Geymayer's statement, "more important than their influence on strength is the ability of resin concrete to provide a noncracking moisture barrier or corrosion protection up to beam failure."[6]

b. Probably because thicker layers (above 1.5 in.) are subject to more internal stresses (shrinkage, temperature, etc.), the 1.5-in. layers generally appear to provide nearly equal precracking protection as well as ultimate strength capabilities.

PART IV: CONCLUSIONS AND RECOMMENDATIONS DERIVED FROM SMALL-BEAM INVESTIGATION

Conclusions

59. A complete analysis of the small-beam data was needed prior to fabricating the larger test beams; therefore, the following conclusions were reached at this stage of the overall investigation. However, the conclusions were reached with the understanding that they are to be used as a guide only and are subject to change as additional test results become available.

Beams 2, 3, and 14 (beams using steel fibers as crack arresters)

60. Either the 0.5- or 1.0-in.-long fibers can be expected to increase the precracking loads of conventionally reinforced members; however, the shorter (0.5-in.-long) fibers will provide overall mixtures with the better tensile properties. This is believed to be due to the difficulty of obtaining a uniform mixture as fiber length versus cross-sectional ratio increases.

61. Once cracking is initiated, the fibers apparently do not significantly affect the size and spacing of the crack pattern. However, it is emphasized that the 0.5-in.-long fibers can be expected to prevent or delay cracking up to or near the maximum anticipated service loads (Appendix A) of beams similar to those tested.

Beams 5, 9, and 10 (beams using wire meshes as crack arresters)

62. Since layers of wire mesh attached to conventional reinforcement do not appear to increase the initial cracking loads of conventionally reinforced members, this method shows no promise as a potential crack-arrest technique. However, properly constructed and positioned wire-mesh cages can be expected to delay the initial cracking of similar beams until 85 to 95 percent of their maximum anticipated service loads are realized.

63. Of the specific techniques investigated, the wire-mesh cage is the only technique that can be recommended for helping to control the

crack growth (size, spacing, etc.) at loads greater than those required to initiate cracking.

64. There is no outstanding difference between wire-mesh cages with or without vertical wires. Therefore, due to their simplicity of construction, those containing vertical wires are recommended.

Beams 6, 7, 12, and 13
(beams using epoxy-resin
<u>concrete as crack arresters)</u>

65. Although there is only slight resistance to crack growth once cracking is initiated, small tensile layers of epoxy-resin concrete will delay the initial cracking of composite beams beyond the maximum anticipated service loads of similar conventionally reinforced concrete beams.

66. Until test results are available that will reflect the potential deficiencies (expansion, creep, sensitivity to environmental factors, exothermic characteristics, etc.) of the epoxy-resin concrete, a definite desirable thickness of the layer, or even epoxy-resin concrete itself, cannot be recommended for practical composite construction of this type.

Recommendations

67. Larger beams using 0.5-in.-long steel fibers and wire-mesh cages should be further investigated under both short- and long-term static loading. If resulting data substantiate current temporary conclusions, similar beams should then be investigated under various environmental conditions.

PART V: FABRICATION AND TESTING PROGRAM FOR LARGER BEAMS

68. Since the test results for the smaller (4- by 9- by 72-in.) beams (Parts III and IV) indicated that either randomly mixed steel fibers or properly positioned wire meshes provided suitable as well as the more economical means for arresting or minimizing the flexural cracking of conventionally reinforced concrete beams, eight larger (5- by 14- by 180-in.) beams, including two control specimens, were fabricated and then subjected to either short- or long-term static loading so that these two promising crack-arrest techniques could be further evaluated under conditions more realistic to those experienced by members of an actual structure.

Concrete Materials and Mixture Proportions

69. Since all test results were to be compared, it was desired that all test specimens possess like or very similar properties; therefore, except for the size of the mixer* and the variation in the casting techniques (paragraph 70), the larger beams were fabricated in a manner very similar to that described for the smaller beams (Part II) by using similar materials, mixture proportions, and curing procedure. The resulting compressive and tensile strengths of the various batches are presented in tables 22 and 23, and their compressive stress-strain characteristics are given in plate 14. Stress-strain curves for the reinforcement used in the larger beams are presented in plate 15.

Fabrication and Curing of Specimens

70. It was feared that some reinforcement sag was possible or probable due to the 180-in. unsupported length of the reinforcement; therefore, the beams were cast in an inverted position so that the

* A 16-cu-ft-capacity rotary mixer was used throughout the casting of the larger specimens.

desired depths could be noted and better controlled throughout their individual lengths. Since steel forms were not available in the indicated sizes, special plywood forms were constructed for use during this phase of the investigation.

71. All beams were vibrated with a small 7000-rpm electric vibrator, finished with a wooden float, covered with waterproof paper, stripped at a 48-hr age, and then moist-cured for an additional 12 days. After the moist-curing period, the beams were allowed to cure under room conditions (approximately 73 ± 2 F and 50 to 60 percent relative humidity) for an additional 14-day period. Static tests were then performed on the beams of group 1, and sustained testing was initiated on the group 2 beams.

Beam Test Methods

Short-term static tests

72. All beams subjected to short-term static testing were simply supported and tested to failure in an inverted position (fig. 8) by

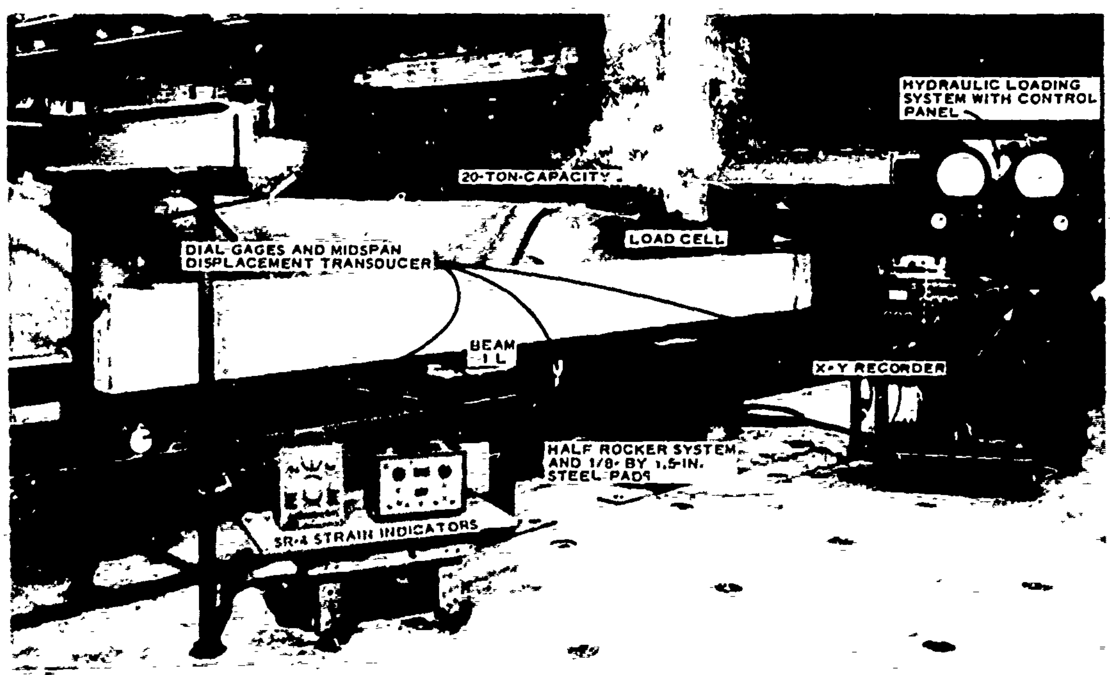

Fig. 8. Test arrangement for short-term static tests of larger beams

third-point loading. Each beam was supported at its third points by a half rocker system, with loads being placed by a combination of a hydraulic system and two 20-ton-capacity jacks that were positioned to provide a clear span of 15 ft for each test specimen. Load and support reactions were distributed to each beam by 1/8- by 1-1/2-in. steel pads placed between the concrete and the steel rollers or load transfers. A control panel, a load cell, a displacement transducer, and an x-y recorder (fig. 8) were used to measure the loads and to obtain a continuous load versus midspan deflection plot for each beam. Verifications of midspan deflections, as well as end-span deflections, were made with three 3-in. travel length dial gages positioned as shown in fig. 8.

73. Total loads were applied in 1000-lb increments, and readings of beam strains, deflections, crack widths, etc., were made after the application of each loading increment. The loads were completely removed at approximately 50 percent of each specimen's ultimate loading capacity so that inelastic deflections could be checked prior to continuation of loading.

74. Concrete strains were measured by an 8-in. mechanical strain gage using Demec standard measuring disks located at previously designated points (fig. 9) and by an SR-4 electrical strain gage placed at the midspan of the compressive (top) face of the beam. Reinforcement strains were also determined by an SR-4 electrical strain gage placed at the midspan of the conventional reinforcement.

75. All cracks were marked, measured, and photographed as they occurred. Crack depths and widths were measured with a steel scale graduated to 0.01 in. and a 40X microscope, respectively.

Long-term (sustained) static tests

76. Except for the duration of the loading increments, the

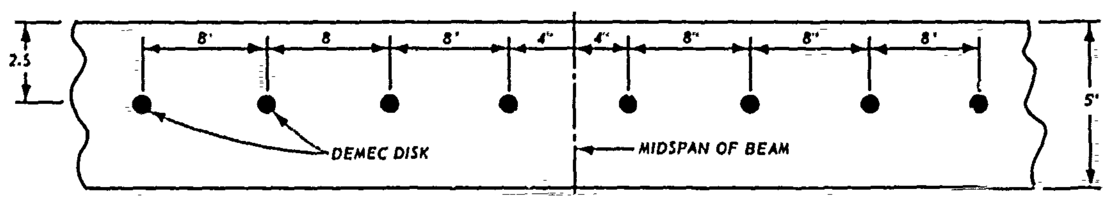

Fig. 9. Locations of standard measuring (Demec) disks along tensile face of beam

sustained tests were essentially the same as the short-term tests; however, some variations from the short-term static tests are described below:

a. Since all sustained testing was concurrent (fig. 10), applied loads were measured with calibrated hydraulic jacks instead of load cells. Although the hydraulic jacks were equipped with special cutoff valves in order that the desired loading could be applied and maintained, each loading increment was checked rather frequently, with pressure gages incremented to 10 psi.

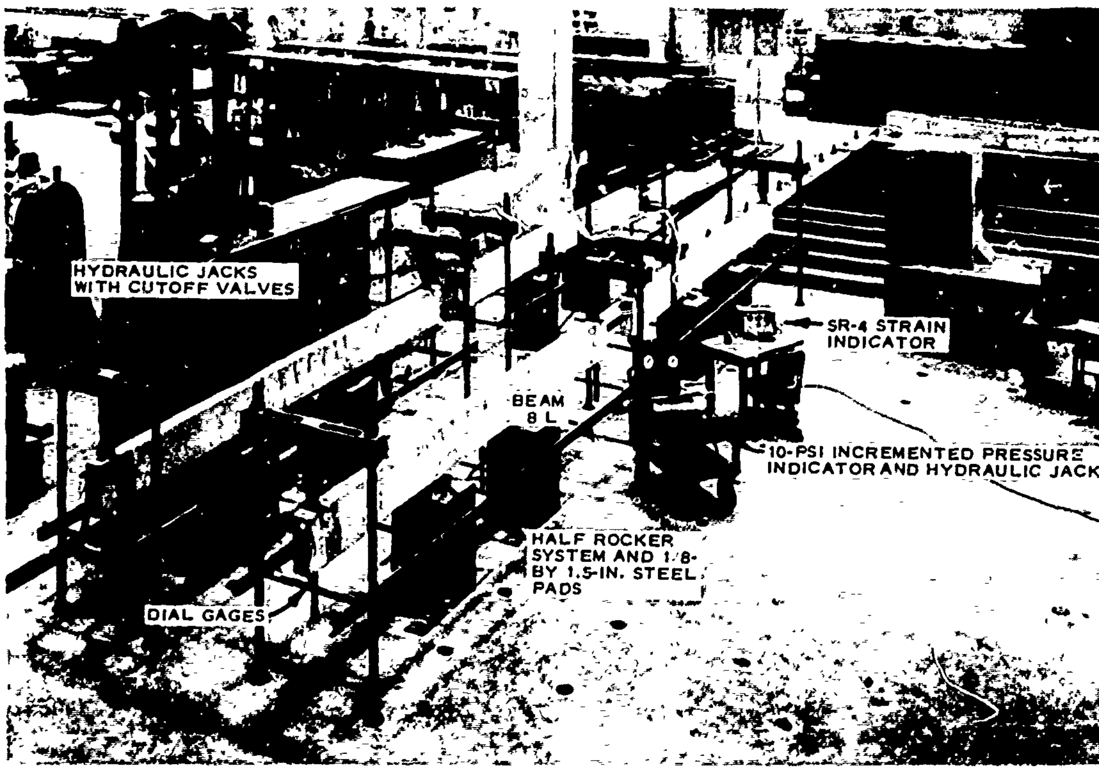

Fig. 10. Test arrangement for long-term static tests of larger beams

b. Generally, all loads were applied in 1000-lb increments; however, in a few cases, loading increments of up to 2000 lb were used.

c. All deflections were determined from either 1- or 3-in. travel length dial gages positioned at the midspans and end spans (fig. 10).

d. Inelastic deflections were not checked prior to increasing the loads.

77. Excluding extended observation periods at the 8000-lb loading levels, approximately 50 percent of each beam's ultimate load, all loading increments were usually maintained for a minimum of 2 weeks, with each loading increment being observed over a longer period of time when significant changes, strains, deflections, and crack patterns occurred between consecutive observations. As mentioned, the behavior of the individual beams was observed over a longer period of time, a minimum of 4 weeks, at the 8000-lb load level. These extended observations were made so the behavior of the individual beams could be more thoroughly investigated under loads near or slightly greater than any maximum anticipated service load.

Beam Test Results

78. The behavior of individual beams and the principal results of individual tests are summarized in tables 24-31 and plates 16-27.

Group 1L*

79. Group 1L consisted of four statically tested beams, one of which was used as the control specimen. The control beam (beam 1L) was reinforced for flexure to approximately 65 percent of a balanced design (P_b) according to the ACI Code[9] by using two No. 6 high-strength, grade 60, reinforcing bars as the tensile reinforcement (plate 15a). The shear requirements of the ACI Code[9] were satisfied by using No. 3 vertical stirrups (plate 15b) on 6-in. centers throughout the sections of the beam requiring shear reinforcement.

80. Beam 2L contained the same portland cement concrete mixture and conventional reinforcement as described for the control beam; however, its lower 3-3/8-in. tensile zone was additionally reinforced with 2.5 percent by volume of 0.5-in.-long steel fibers. The properties of

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* L denotes the 5- by 14- by 180-in. beams, or the larger beams, throughout this report.

the fibers used in this beam, as well as in all other large beams (4L, 6L, and 7L) which contained fibers, were identical with those reported for the small beams and are listed in table 4.

81. Beam 3L was fabricated with the same plain concrete mixture and conventional reinforcement as the control beam, except that the number of conventional stirrups was reduced and a 4- by 11- by 84-in. wire-mesh cage, fabricated from No. 14 gage (2.03 mm), 1- by 1-in. galvanized wire mesh, was used as a crack arrester throughout the inner 84-in. section of the beam (fig. 11). Also, extending each end of the wire-mesh cage 12 in. into the maximum shear zone of the beam allowed investigation of the cage's potential shear reinforcing capabilities as well as its potential as a crack arrester.

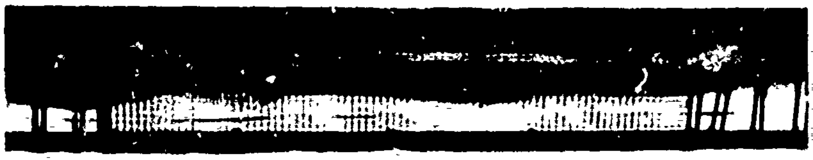

Fig. 11. Reinforcement for beam 3L (reinforcement was inverted prior to casting)

82. Beam 4L was similar to beam 2L except that the steel fibers were incorporated into the entire cross section of the beam so that further comparisons could be made of beams containing minimum depths of fibers with those possibly containing excessive fiber-reinforcement depths.

83. Test results for beam 1L are given in table 24, photo 15, and plates 16, 20, and 24 and are discussed briefly below:

a. A total load of 2000 lb resulted in the initial cracking of the beam (photo 15b). Table 24 indicates concrete tensile strains, averaged over middle ne-third of beams, and compressive strains of 150 and 165 millionths, respectively, at this load level, which resulted in maximum crack widths and depths of 0.001 and 0.90 in., respectively. Table 24 and photo 15 show that from this point the strains and resulting cracks grew somewhat uniformly

until the concrete tensile strain reached 1150 millionths, the concrete compressive strain reached 887 millionths, the maximum crack width reached 0.006 in., and the maximum crack depth reached 7.15 in. at the 7000-lb level (photo 15e). However, from this level there was a drastic change with cracks opening up to 0.014 in. in width at the 8000-lb load level (photo 15f). Since cracks this wide are considered excessive in most structural elements,[13,14] the beam was considered to have reached its crack-arrest potential somewhere between the 7000- and 8000-lb load levels. Although the beam reached its potential as a crack arrester as stated, a total load of 14,000 lb was required to completely fail the beam. Photos 15i and 15j indicate, as would be expected, that the beam experienced a flexural compressive failure shortly after its ultimate load of 14,000 lb was reached.

b. A midspan deflection of 1.473 in. was recorded just prior to failure. This represents a deflection ratio of approximately L/122, which is about what should be expected for such beams at this particular loading level.

c. The failure moment, including weight of beam, was 433,234 in.-lb. This is only about 86 percent of the ultimate moment of 502,016 in.-lb predicted by the ACI Code.[9] This difference is not to be expected and tends to indicate that the beam may have failed prematurely.

84. Test results for beam 2L are shown in table 25, photo 16, and plates 17, 21, and 25 and are briefly discussed below:

a. A concrete tensile strain of 463 millionths resulted in initial cracks of up to 0.002 in. in width and 1.06 in. in depth at the 5000-lb load level (photo 16b). The cracks increased from this magnitude to 0.004 in. in width and 6.50 in. in depth during the next 2000 lb of applied loading (to 7000-lb total load); however, there were no further changes observed in the crack pattern

until 10,000 lb of applied load produced concrete tensile strains of such a magnitude, 1434 millionths, that cracks opened up to maximum widths and depths of 0.009 and 7.25 in., respectively. An additional 4000 lb, 14,000-lb load level, produced a concrete tensile strain of 2166 millionths and a resulting major crack having a maximum width and depth of 0.015 and 8.25 in., respectively. Since this width is considered to be excessive for limiting moisture penetration,[13,14] the remaining test loads were applied with the realization that the beam provided no other crack-arrest potential. However, it is emphasized that the beam supported a total load of 17,000 lb before it experienced a flexural compressive failure.

b. A midspan deflection of 1.50 in. occurred just prior to failure. This represents a deflection ratio of L/120, or about the same ratio as that experienced by beam 1L. However, at a load of 14,000 lb (the failure load of beam 1L), the deflection ratio for beam 2L was L/201, which is a considerable improvement over that of the conventionally reinforced control beam.

c. The failure moment of 523,234 in.-lb was approximately 20 percent higher than that required to fail the conventionally reinforced control beam. This difference indicated not only that beam 1L failed prematurely but also that the fibers increased the load-carrying ability of beam 2L. However, the fibers did not appear to possess this potential during the small-beam investigation (Part III).

85. Test results for beam 3L are shown in table 26, photo 17, and plates 18, 22, and 26 and are briefly discussed below:

a. A concrete tensile strain of 314 millionths was sufficient to produce initial cracks of up to 0.001 in. in width and 0.68 in. in depth at the 3000-lb load level (photo 17b). From this point, there was a rather uniform

increase in the crack dimensions (table 26) until (1) the concrete strains reached 1464 millionths, (2) the maximum crack widths reached 0.006 in., and (3) the maximum crack depths reached 6.60 in. at the 8000-lb load level (photo 17d). Table 26 shows that there was no measurable difference in the maximum dimensions of the cracks between the 8,000- and 10,000-lb load levels; however, an additional load of 1,000 lb, total load of 11,000 lb, resulted in concrete tensile strains of 1931 millionths and corresponding cracks of up to 0.007 and 7.85 in. in width and depth, respectively (photo 17e). Maximum crack widths of 0.010 and 0.012 in. were noted at the 13,000- and 15,000-lb load levels, respectively. Since widths of these magnitudes, especially those of 0.012 in., are generally considered excessive for most structural elements,[13,14] the wire mesh was considered to have reached its maximum potential as a crack arrester somewhere between the 13,000- and 15,000-lb load levels. However, table 26 indicates that the beam sustained a total load of 18,000 lb before it actually failed, and photo 17h indicates that it failed, as had all previously tested specimens, in the desired flexural compressive mode.

b. A midspan deflection of 1.240 in. occurred just prior to failure. This represents a deflection ratio of approximately L/145. This is a slight improvement over the deflection ratio found for the control beam, although the failure load was increased approximately 38 percent.

c. The failure moment was 553,234 in.-lb. A significant amount of this increase, when compared with the failure moment of 433,234 in.-lb for beam 1L, was probably due to the wire mesh providing flexural reinforcement in addition to its primary function of arresting or minimizing cracking.

86. Test results for beam 4L are shown in table 27, photo 18, and

plates 19, 23, and 27 and are discussed briefly as follows:

a. A concrete tensile strain of 297 millionths was sufficient to initiate hairline cracking (0.0005-in. maximum width) at a load of 4000 lb (photo 18b); however, a 5000-lb load was required to produce cracks with widths (0.001 in. maximum) and depths (0.90 in. maximum) that were both measurable. From this point, table 27 and photos 18d-18h show that the strains and corresponding cracks increased somewhat uniformly until they reached magnitudes of 2099 millionths (concrete tensile strain), 0.010 in. (maximum width of cracks), and 8 in. (maximum depth of cracks), at the 13,000-lb load level, but no further measurable changes occurred in the crack pattern until the maximum cracks reached magnitudes of 0.05 in. in width and 8.65 in. in depth at the failure load of 16,000 lb. There was a drastic change between the 15,000- and 16,000-lb load levels, but photo 18j indicates that the beam failed in the desired flexural compressive mode.

b. A midspan deflection of 1.162 in. occurred just prior to failure. This represents a deflection ratio of approximately L/155, or about the same as that for beam 2L, which contained fibers in its lower 3 in. only.

c. The failure moment was 493,234 in.-lb, or about 95 percent of that for beam 2L. Since this is somewhat higher than the 433,234-in.-lb moment required to fail beam 1L, there is again some indication that fibers may increase the ultimate load-carrying capabilities of reinforced concrete members. However, since this was not indicated by the small-beam tests (Part III), and, since it is near the moment predicted by the ACI Code[9] for beam 1L, it is, again, some indication that the control beam (beam 1L) failed prematurely.

Group 2L

87. Since most structural elements are seldom limited to short-term static loading, the beams of group 2L were fabricated, cured as previously described, and then subjected to sustained loading in order that each specific crack-arrest technique could be investigated with test specimens that more nearly represented members of actual structures. And, since it was desired to obtain some idea of what effects sustained loading could have on any of the particular crack-arrest techniques, the beams of group 2L were fabricated so that they were identical, or as nearly identical as possible, with those of group 1L.

88. Test results for beam 5L, which was similar to beam 3L (contained a 4- by 11- by 84-in. wire-mesh cage in its inner 84-in. section), are given in table 28, photo 19, and plates 18 and 26 and are discussed briefly as follows:

a. Table 28 shows that there were no visible cracks immediately after the initial load of 2000 lb was applied; however, concrete tensile strains of 91 millionths and corresponding cracks of up to 0.002 in. in width and 1.06 in. in depth were noted after 24 hr of sustained loading at this particular level (photo 19b). From this point, the strains and cracks both grew rather uniformly until they reached a strain of 470 millionths, a crack width of 0.005-in., and a crack depth of 4.85 in. at the initial application of the 5000-lb load (photo 19c). Maximum crack widths of 0.009 and 0.010 in. were then noted at the completion of the 6000- and 7000-lb loading increments (photos 19d and 19e), respectively. Since widths of greater than 0.010 in. are generally considered excessive for many structural elements,[13,14] the beam was considered to have practically reached its crack-arrest potential at the 7000-lb loading level. Even though the beam reached its potential as a crack arrester at or near the 7000-lb level, an additional load of 10,900 lb, or a total load of 17,900 lb, was required to actually fail

the beam (table 28). Photo 19ℓ shows that the beam failed in the desired flexural compressive mode and indicates that properly designed and positioned wire meshes can provide sufficient shear reinforcement as well as perform the principal function of arresting flexural cracking.

b. A midspan deflection of 3.25 in. occurred at the failure load. This, when compared with the 1.20-in. deflection experienced by beam 3L, may indicate the effect that sustained loading will have on the deflections of members of this type.

c. The failure moment of 550,234 in.-lb was essentially the same as the 553,234 in.-lb for beam 3L. This indicates that sustained loading may not affect the ultimate loads of members of this type even though most of their other properties (strains, deflections, maximum crack widths and depths, etc.) may be affected significantly.

89. Test results for beam 6L, which was similar to beam 2L (2.5 percent by volume of 0.5-in.-long steel fibers were placed in the lower 3-3/8-in. tensile zone of the beam) are given in table 29, photo 20, and plates 17 and 25 and are discussed briefly as follows:

a. A concrete tensile strain of 284 millionths with corresponding 0.0005-in.-wide hairline cracks occurred 24 hr after a total load of 4000 lb had been applied (photo 20b); however, an additional 24 hr at this loading produced cracks of such magnitudes (0.001 in. wide and 0.50 in. deep) that their depths could be marked, measured, and photographed (photo 20c). From this point, the strains and corresponding cracks continued to grow at an increasing rate until they reached magnitudes of 1304 millionths (tensile strain of concrete), 0.010 in. (width of crack), and 6.95 in. (depth of crack) immediately after reaching the 9000-lb load level (photo 20f). The maximum crack width was then increased to 0.012 in. by an additional 2000 lb (11,000-lb total load) of loading

(photo 20g). Since widths of these magnitudes, especially those of 0.012 in., are probably excessive for most structural elements,[13,14] the beam was considered to have reached its maximum crack-arrest potential somewhere between the 9,000- and 11,000-lb loading levels. However, table 29 shows than an additional load of 6,400 lb (17,400 lb total) was required to completely fail the beam, and, as in all previous tests, the flexural compressive mode was evident at failure (photo 20k).

b. A midspan deflection of 1.85 in. occurred at the 17,400-lb failure load. This was only a modest increase over the 1.50 in. noted for beam 2L at the 17,000-lb failure load. However, plate 17 does indicate that the sustained loading produced deflections with percentage differences of or near this amount throughout their corresponding loading levels.

c. The failure moment of 535,223 in.-lb was essentially the same as the 523,223-in.-lb moment required to fail beam 2L. This again indicates that sustained loading may not affect the ultimate loads of members using either of the potential crack-arrest techniques even though most of their other properties (strains, deflections, maximum crack widths, etc.) may be affected significantly. Also, these failure moments, when compared with those predicted for the control beams, indicate that the steel fibers may have significantly increased the ultimate load-carrying ability of the beam; however, it is emphasized that this was not indicated by results of the small-beam tests (Part III).

90. Test results for beam 7L, which was similar to beam 4L (2.5 percent by volume of 0.5-in.-long steel fibers were used throughout its entire cross section), are given in table 30, photo 21, and plates 19 and 27 and are discussed briefly as follows:

a. Concrete tensile strains of 187 millionths resulted in

the initial cracking (0.001-in. maximum width, 0.81-in. maximum depth) of the beam after it had been subjected to a 3000-lb load for a period of 24 hr (photo 21b). From this point, there was a uniform crack width increase of 0.001 in. per 1,000 lb of additional loading until a total load of 12,000 lb was reached. This indicates that prolonged loadings of up to or slightly greater than ultimate working levels may have little or no effect on the crack widths of structural members of this type. The maximum crack width was increased by 0.002 in. (0.010 to 0.012 in.) by the next loading level (13,000 lb) (photo 21h). Since widths of these magnitudes are generally considered excessive for most flexural members,[13,14] the beam is considered to have reached its crack-arrest potential somewhere between the 12,000- and 13,000-lb loading levels. However, it is emphasized that an additional load of 4500 lb (17,500-lb total load) was required to completely fail the beam, and, as in all previous cases, the desired flexural compressive mode was evident at failure (photo 21k).

b. A midspan deflection of 2.65 in. occurred at its failure load of 17,500 lb (table 30). However, a deflection of only about 1.45 in. (table 30) occurred before yielding of the reinforcement became evident. Therefore, a substantial amount of the mentioned deflection actually occurred after the beam had, for all practical purposes, reached its ultimate load-carrying capability.

c. The failure moment of 538,234 in.-lb was approximately 9 percent greater than the 493,234 in.-lb required to fail beam 4L. This again indicates that sustained or prolonged loading will probably not affect the ultimate load-carrying capabilities of members using either of the recommended crack-arrest techniques. And, further, since it was considerably higher (up to nearly 25 percent) than

the failure moments of the control beams, there is again indication that steel fibers may significantly increase the load-carrying capabilities of conventionally reinforced concrete members even though this was not, as previously mentioned, indicated by results of the small-beam tests (Part III).

91. Test results for beam 8L, which was similar to the conventionally reinforced control beam (beam 1L) of group 1L, are given in table 31, photo 22, and plates 16 and 24 and are discussed briefly as follows:

a. An average tensile strain of 61 millionths produced the first noticeable cracking (0.001-in. maximum width and 1.00-in. maximum depth) at the initial application of the 2000-lb load (photo 22b). The cracks increased to 0.002 in. in width and 2.38 in. in depth at the initial application of the 3000-lb load (photo 22c), and then further increased to 0.004 in. in width and 3.25 in. in depth after 3 days of sustained loading at this level (photo 22d). From this point, the concrete strains and resulting cracks continued to grow at an increasing rate until they reached magnitudes of 887 millionths, 0.010 in., and 6.15 in. for the strains, maximum crack widths, and maximum crack depths, respectively, at a 7-day sustain load of 7000 lb (photo 22f). Some cracks of up to 0.013 in. in width and 6.75 in. in depth were noted immediately after the application of the 8000-lb load (photo 22g). Although no measurable changes occurred during an additional 30 days of observation at this particular load level, the cracks were already of such a magnitude that the beam was considered to have reached or surpassed its crack-arrest potential[13,14] somewhere between the mentioned 7000- and 8000-lb levels. Even though the beam did reach its crack-arrest potential as stated, a total load of 15,900 lb was required to completely fail

the beam, and, as in all previous tests, the beam experienced a flexural compressive failure (photo 22ℓ).

b. A midspan deflection of 2.55 in. occurred at failure. This is considerably more than the 1.473 in. recorded for the failure load of beam 1L; however, plate 17 shows that this increase was due almost entirely to the additional load (1900 lb) required to fail beam 8L.

c. The failure moment was 490,234 in.-lb. This is within 1 to 2 percent of the failure moment of 495,016 predicted by the ACI Code[9] and indicates that beam 1L may have experienced premature failure, as mentioned, and that the sustained loads had no effect on the ultimate load-carrying capability of the beam.

PART VI: COMPARISON OF TEST RESULTS

Short-Term Static Tests of Larger Beams Versus Short-Term Static Tests of Smaller Beams

92. Most of the more widely known equations presently used for predicting crack widths and spacings appear to be based on actual test results.[13] However, it seems that some, if not a majority, of these equations are based primarily on the test results from rather small, essentially model specimens subjected to short-term static testing only. Therefore, results of the short-term static tests of the larger and smaller beams are compared so that the predictions from some of the more widely known equations can be compared with the test results obtained from members that were similar (except for the individual lengths and cross-sectional areas) and had nearly equal reinforcement ratios, were fabricated from like mixtures, used identical crack-arrest techniques, and were tested under third-point loading. This allows a determination, to some degree, of how size alone may affect cracking within reinforced concrete members that are subjected only to short-term static testing. A brief comparison of the test results for corresponding beams follows.

Beam 1L versus beam 1 (conventionally reinforced control beams)

93. Although there were four conventionally reinforced control beams tested during the small-beam phase, only beam 1 is used in this comparison. Beam 1 was selected for convenience and not for any particular statistical reason.

94. Table 32 shows that initial cracking occurred at maximum concrete tensile stresses of 360 and 370 psi for beams 1L and 1, respectively. From this point, the behavior of the two beams was very similar, with maximum crack widths closely following the predictions of the Base, Read, Beeby, and Taylor equation[5] of $W_{max} = 3.3\, d'f_s/E_s \left(\frac{h - kd}{d - kd}\right)$ until reinforcement stresses* of nearly 25,000 psi were reached

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* The method used for determining the reinforcement stresses at the various load levels is illustrated in Appendix C.

(plate 20). Although the previously mentioned equation[5] of $W_{max} = 3.3\ d'f_s/E_s \left(\frac{h - kd}{d - kd}\right)$ was valid (plate 20) in beam 1 to stress levels (45,000 psi) greatly exceeding those expected from normal service loads, the crack widths of beam 1L increased at a much more rapid rate (plate 20) and more nearly followed the predictions of the Chi and Kirstein equation[15] of $W_{max} = \frac{7.5FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$ at stresses greater than the previously mentioned 25,000-psi level.

Beam 2L versus beam 14
(beams containing 0.5-in.-
long fibers in lower tensile zones)

95. Maximum concrete tensile stresses of 800 and 1080 psi resulted in the initial cracking of beams 2L and 14, respectively (table 32). Plate 21 shows that the Chi and Kirstein equation[15] of $W_{max} = \frac{7.5FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$ predicts the crack widths of the respective beams (2L and 14) satisfactorily, especially up to the highest reinforcement stress levels expected in actual structural members, when the constant of 7.5 is changed to 2.5 and 2.0 for the larger (beam 2L) and smaller (beam 14) beams, respectively. This indicates, as did the control beams (1L and 1), that smaller beams or models are less preferable than larger beams when crack-arrest techniques are being investigated.

Beam 3L versus beam 5 (beams
containing wire-mesh cages in
more critical flexural zones)

96. Initial cracking occurred when the maximum concrete tensile stresses reached magnitudes of 510 and 730 psi in beams 3L and 5, respectively. Then, if the constant of the Chi and Kirstein equation[15] is adjusted from 7.5 to 2.7 for beam 3L and to 2.0 for beam 5, the maximum crack widths obtained by calculation will be in rather close agreement with those observed (plate 22) from this point (point of initial cracking) up to load levels which produced reinforcement stresses of 45,000 psi. This difference indicates, as do previous test differences, that small beams or models are less preferable than are large beams when potential crack-arrest techniques are being investigated.

Beam 4L versus beam 2 (beams containing 0.5-in.-long fibers throughout entire cross sections)

97. Table 32 shows that initial cracking occurred when the maximum concrete tensile stresses reached 650 and 990 psi for beams 4L and 2, respectively. From this point, the beams behaved quite similarly, with maximum crack widths closely following the adjusted Chi and Kirstein equation[15] of $W_{max} = \frac{2.0FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$ until reinforcement stresses of 30,000 psi were exceeded (plate 23). This close agreement of the behavior of the individual beams indicates that this particular crack-arrest technique could possibly be investigated successfully by the use of small beams or models when or if normal service loads are not exceeded; however, it is emphasized that this is the only technique investigated which indicated this possibility.

Short-Term Versus Long-Term Static Tests of Larger Beams

Background

98. Although the preceding comparison of the short-term static tests allowed determination, to some extent, of how flexural cracking may be affected by the length of members, size of cross sections, and other dimensional aspects of the structural member, it did not indicate the influence that sustained or long-term loading may have. Therefore, the test results for groups 1L and 2L were compared in order to obtain some idea of the influence that sustained loading, or loading which more nearly represents that found in structural members, may have on the individual crack-arrest techniques.

Beam 1L versus beam 8L (conventionally reinforced control beams)

99. Initial cracking occurred in both beams at a maximum concrete tensile stress of 360 psi or at a tensile stress near that indicated by the beams' corresponding tensile splitting specimens (table 32). From this point, the cracks of beam 8L (sustained static test) were, in general, slightly larger up to and including its maximum service stresses

(plate 24). However, there were no drastic differences in the maximum crack sizes noted for either beam because the same equations[5,15] $W_{max} = 3.3d'f_s/E_s \left(\frac{h - kd}{d - kd}\right)$ and $W_{max} = \frac{7.5FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$, can be used to approximate the maximum crack widths observed for each beam at reinforcement stress levels up to or near 25,000 psi and from 25,000 to over 40,000 psi, respectively.

Beam 2L versus beam 6L (conventionally reinforced beams with steel fibers incorporated into lower tensile zones)

100. Maximum tensile stresses of 800 and 650 psi resulted in the initial cracking of beams 2L and 6L, respectively. Plate 25 shows that, from this point, the maximum crack widths versus reinforcement stresses can be related to the Chi and Kirstein equation[15] when its constant of 7.5 is reduced to 2.5 for beam 2L and 4.4 for beam 6L. Since both of the conventionally reinforced control beams (1L and 8L) used the mentioned constant of 7.5 for this relation, especially at reinforcement stresses greater than 20,000 to 25,000 psi (plate 24), this indicates that this particular crack-arrest technique, i.e., steel fibers incorporated into lower tensile zone, may reduce or control normal crack widths by a factor of 1.70 to 3.00. However, as most structural members support sustained or prolonged loading, it is only logical to assume the smaller value, or the value produced by sustained loading, to be the more realistic.

Beam 3L versus beam 5L (conventionally reinforced beams with wire-mesh cages positioned within inner 84-in. sections)

101. Initial cracking occurred at maximum concrete tensile stresses of 510 psi for beam 3L and 360 psi for beam 5L (table 32). This represents an approximate increase of 25 percent over the tensile strength indicated by the tensile splitting specimens* corresponding to

* Tensile splitting specimens were fabricated from plain concrete mixture only and contained no wire mesh.

beam 3L; however, there was no change in the indicated and tested tensile strengths of the concrete for beam 5L (table 32). Therefore, it appears from these limited test results that wire mesh may not significantly affect, as would be suspected, the precracking characteristics of similar structural members, especially those subjected to sustained loading. Plate 26 shows that adjusting the constant of the Chi and Kirstein equation[15] from 7.5 to 2.7 for beam 3L and to 5.5 for beam 5L allows a rather close approximation of the maximum crack widths actually observed from the point of initial cracking up to reinforcement stresses of 45,000 psi. This again indicates the effect that sustained loading, or loading that more nearly represents that of most structural members, may have on test specimens. However, it is emphasized that due to the limited testing of this investigation only indications and not completely valid conclusions are presented.

Beam 4L versus beam 7L
(conventionally reinforced
beams with steel fibers incor-
porated into entire cross sections)

102. Maximum tensile stresses of 650 and 510 psi resulted in the initial cracking of beams 4L and 7L, respectively (table 32). From this point, adjusting the constant of the Chi and Kirstein equation[15] from 7.5 to 2.0 for beam 4L and to 3.0 for beam 7L allows a rather close approximation of the observed crack widths, especially those corresponding to reinforcement stresses up to and slightly greater than the actual stresses that structural members normally encounter (plate 27). These differences, along with those previously described for beam 2L versus beam 6L and beam 3L versus beam 5L, indicate that sustained loading may have a significant effect on the performance of each potential crack-arrest technique investigated. Since most structural members are subjected to sustained or prolonged loading, it appears that all equations used for predicting crack widths should be correlated with results obtained from sustained testing and not from the customary static testing procedures.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

103. The following conclusions have been reached based on the test results discussed. Due to the limited scope of this investigation, these conclusions should be regarded as tentative.

Small versus larger beam tests

104. A comparison of the maximum crack widths predicted through use of some of the better known equations[13] (CEB simplified, Thomas, Odman, etc.) with the to-date test results of this investigation shows that the predictions more nearly follow the test results obtained from tests of the small beams. This indicates:

- a. That most of the better known equations may have resulted from the tests or observations of small beams, or essentially models, subjected to short-term static loading only.
- b. That either a sustained load or increased cross section, and probably both, may have more effects on the resulting crack widths than most present equations[13] may indicate. Since most structural members are subject to sustained or long-term loading, it is concluded that future investigations concerning cracks or crack-arrest techniques of reinforced concrete members should be concentrated on members that more nearly represent those found in actual structures.

Short-term versus long-term static tests of larger beam

105. Since (a) a beam of each specific type (conventionally reinforced control, conventional reinforcement plus wire-mesh cages, conventional reinforcement plus steel fibers in lower tensile zone of cross section, and conventional reinforcement plus steel fibers throughout entire cross section) was tested under both short- and long-term static loading; (b) since the test results of the control beams generally followed,

especially at or near service loads, the crack width predictions of the Chi and Kirstein equation[15] of $W_{max} = \frac{7.5FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$; and (c) since the crack widths of each technique (plates 25, 26, and 27) can be correlated with the same equation by simply adjusting the constant of 7.5, the effectiveness of each technique is determined by comparisons of the initial and adjusted constants.

106. The conclusions reached based on tests of beams using each technique are as follows:

<u>a</u>. <u>Beams 2L and 6L (conventionally reinforced beams with 0.5-in.-long steel fibers (2.5 percent by volume) incorporated into lower 3-3/8-in. tensile zones).</u> Plate 25 shows that a close relationship can be obtained between the predicted and observed values of the maximum crack widths when the constant of the Chi and Kirstein equation[15] is adjusted from 7.5 to 2.5 for beam 2L and to 4.4 for beam 6L. This represents crack-width reduction percentages of 67 and 41 percent for the respective beams. However, since it is only logical to assume that most structural members will be subjected to sustained loading, the equation $W_{max} = \frac{4.4\ FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$ is the only one recommended in conjunction with this particular technique. Also, since the effective area in uniform tension (A_e) is <u>now</u> <u>generally</u> considered to be equal to $2(h - d)b$ for members similar to these,[13] it is recommended that this be the minimum area in which steel fibers are to be incorporated.

<u>b</u>. <u>Beams 3L and 5L; (conventionally reinforced beams with wire-mesh cages positioned within inner 84-in. sections).</u> Although wire-mesh cages have potential as crack arresters, the limited testing of this investigation indicates that use of wire-mesh cages is possibly the least desirable of the three techniques used. This conclusion is reached because plate 26 shows that the maximum cracks observed for beam 5L maintained widths

closely related to those predicted using the Chi and Kirstein equation[15] with an adjusted constant of 5.5, which represents a crack-width reduction percentage of 27 percent. It is emphasized that short-term statically tested beam 3L indicated a much better crack-arrest potential; however, for reasons previously stated, the equation $W_{max} = \frac{5.5FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$ is the only one that can be recommended for evaluating this particular crack-arrest technique. Also, since all beams tested used wire-mesh cages which were extended into their maximum shear zones, and since all beams failed in a desired mode, it can tentatively be concluded that properly designed* and positioned wire-mesh cages could be used for shear reinforcement as well as for the principal objective of providing techniques for arresting cracks.

c. Beams 4L and 7L (conventionally reinforced beams with 0.5-in.-long steel fibers (2.5 percent by volume) incorporated into entire cross sections). Since these beams indicated the best crack-arrest potential under both short- and long-term static testing (plate 27), one could conclude initially that it is the best method used. However, the cost of fabrication must be considered before even a tentative conclusion is reached. But it is emphasized that the maximum crack widths observed under the most severe testing conditions (long-term static loading) did closely follow an adjusted Chi and Kirstein equation[15] of $W_{max} = \frac{3.0FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$, or crack-width reductions of 60 percent when compared with conventionally reinforced beams. This also represents crack-width reductions of 19 to 33 percent from those experienced by the beams using the two previously described crack-arrest techniques.

* This investigation indicates that the total area of all horizontal wires, which are the principal crack arresters, should not be less than 0.25 to 0.30 percent of the net area (b times d) of concrete.

Summary of Conclusions

107. Based on a correlation of the conclusions reached concerning each crack-arrest technique, the following design procedure was developed.

a. Select a mixture* similar to the one described for the test specimens and then design the members according to the design procedures given in ACI Code 318-71.[9]

b. Estimate the maximum crack widths using the Chi and Kirstein equation $W_{max} = \frac{7.5FD}{E_s}\left(\frac{h - jd}{d - jd}\right)\left(f_s - \frac{2500}{FD}\right)$.

c. Determine the maximum size of cracks allowable using reference 13 as a guide for specific environmental conditions. Then use the most economical of the three described techniques with the expectation that under normal service loads (a) 0.5-in.-long steel fibers (2.5 percent by volume) incorporated into the zone of uniform tension will reduce the maximum anticipated crack widths by approximately 41 percent; (b) properly designed and positioned wire meshes will reduce the maximum anticipated crack widths by approximately 27 percent; and (c) 0.5-in.-long steel fibers incorporated into the entire cross section will reduce the maximum anticipated crack widths by approximately 60 percent.

Future Research Recommended

108. The previously cited, tentative conclusions were based on laboratory tests only; therefore, it is recommended that a series of beams be tested in an outdoor climate such as that found at the Treat Island Exposure Station, Maine.

109. Each test specimen would be subjected to its normal working

* The mixture selected could be very important because the bond between the fibers and mixtures could be affected by the size of aggregates selected.

load for a prolonged period of time from which valuable information could be obtained on the possible effects that a changing environment could have on the crack-arrest techniques under present consideration.

110. This information combined with the laboratory tests outlined in this report would then allow one to make more lasting conclusions on the actual practical capabilities of each crack-arrest technique showing promise during laboratory testing.

REFERENCES

1. Williamson, G. R., "Fibrous Reinforcements for Portland Cement Concrete," Technical Report No. 2-40, May 1965, U. S. Army Engineer Division, Ohio River, CE, Cincinnati, Ohio.

2. Romualdi, J. P. and Ramey, M. R., "Effects of Impulsive Loads on Fiber-Reinforced Concrete Beams," Final Report, Contract No. OCD-PS-64-21, Carnegie Institute of Technology, Pittsburgh, Pa.

3. Base, G. D. et al., "An Investigation of the Crack Control Characteristics of Various Types of Bar in Reinforced Concrete Beams," Research Report 18, Part 1, Dec 1966, Cement and Concrete Association, London, England.

4. Beeby, A. W., "An Investigation of Cracking in Slabs Spanning One Way," Technical Report TRA 433, Apr 1970, Cement and Concrete Association, London, England.

5. Base, G. D. et al., "An Investigation of Crack Control Characteristics of Various Types of Bar in Reinforced Concrete Beams," Research Report 18, Part 2, Dec 1966, Cement and Concrete Association, London, England.

6. Geymayer, H. G., "Use of Epoxy or Polyester Resin Concrete in Tensile Zone of Composite Concrete Beams," Technical Report C-69-4, Mar 1969, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

7. Cox, F. B., "A Study of the Feasibility of Methods for Increasing the Load-Carrying Capacities of Existing Concrete Beams," Technical Report C-70-3, May 1970, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

8. U. S. Army Engineer Waterways Experiment Station, CE, "Handbook for Concrete and Cement," Aug 1949 (with quarterly supplements), Vicksburg, Miss.

9. ACI Committee 318, "ACI Standard Building Code Requirements for Reinforced Concrete," ACI 318-71, 1971, American Concrete Institute, Detroit, Mich.

10. Bjuggren, U., "Nomenclature for Phenomena of Failure in Reinforced Concrete Beams," Proceedings, American Concrete Institute, Vol 64, No. 10, Oct 1967, pp 625-633.

11. McDonald, J. E., "The Effect of Confining Reinforcement on the Ductility of Reinforced Concrete Beams," Technical Report C-69-5, Mar 1969, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

12. Shah, S. P. and Rangan, B. V., "Fiber Reinforced Concrete Properties," Proceedings, American Concrete Institute, Vol 68, No. 2, Feb 1971, pp 126-135.

13. Reis, E. E. et al., "Causes and Control of Cracking in Concrete Reinforced with High-Strength Steel Bars--A Review of Research," Engineering Experiment Station Bulletin 479, 1965, College of Engineering, University of Illinois, Urbana, Ill.

14. Parkin, J. W., "On the Calculation of Crack Width in Beams," Technical Note, Nov 1971, Cement and Concrete Association of Australia.

15. Chi, M. and Kirstein, A. F., "Flexural Cracks in Reinforced Concrete Beams," Proceedings, American Concrete Institute, Vol 54, Apr 1958, pp 865-878.

Table 1

Concrete Mixture for Reinforced Concrete Members

Mixture for 94-lb Batch*

Mixture	Volume cu ft	Weight lb
Type II cement (RC-622)	0.479	94.0
Limestone fine aggregate (CRD-MS-17(3))	2.154	358.3
Limestone coarse aggregate (CRD-G-31(7))	2.069	349.4
Water	1.297	80.8

* Water-cement ratio by weight, 0.86; slump, 2 ± 1/2 in.; cement content, 423 lb/cu yd.

Table 2

Compressive and Tensile Strengths of Individual Batches of Plain Concrete for Small Beams

Batch No.	Age of Plain Concrete when Tested days	Beams Made from Indicated Batch	Compressive Strength psi	Splitting Tensile Strength psi
1	28	1-3	3510	Not recorded
2	35	4-7	2510	295
3	28	8-10	2470	400
4	28	11-14	2950	430

Table 3

Compressive and Tensile Strengths of Individual Batches of Steel-Fiber-Reinforced Plain Concrete Mixture for Small Beams

Latch No.	Length of Fiber in.	Age of Fiber-Reinforced Concrete When Tested days	Representing Beam No.	Compressive Strength psi	Splitting Tensile Strength psi
1	0.5	28	2	Not recorded	490
2	1.0	28	3	Not recorded	545
3	0.5	28	14	3410	520

Table 4

Properties of Steel Fibers Used

Serial No.	Length of Fiber in.*	Cross-Sectional Dimension of Fiber in.*	Ultimate Tensile Strength psi*	Density pcf
CRD-S-F-1	0.5	0.01 by 0.01	55,000	485.19
CRD-S-F-2	1.0	0.01 by 0.01	55,000	483.70

* As given by manufacturer.

Table 5

Aggregate Grading for Epoxy-Resin Concrete

Passing Sieve	Retained on Sieve	Percentage by Weight
3/8 in.	No. 4	25.2
No. 4	No. 8	17.3
No. 8	No. 16	13.9
No. 16	No. 30	10.6
No. 30	No. 50	8.0
No. 50	No. 100	6.5
No. 100		18.5

Table 6

Compressive and Tensile Strengths of Individual Batches of Epoxy-Resin Concrete

Batch No.	Age of Epoxy-Resin Concrete When Tested days	Representing Beam No.	Compressive Strength psi	Splitting Tensile Strength psi
1	7	6-7	11,290	1225
2	14	12-13	11,220	1290

Table 7

Summary of Test Results for Beam 1*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Width, in. D†	Depth in.
0	0	0	0	0	0	0	0	0
5	500	0.004	45	40	30	0	0	0
8	1,000	0.010	95	90	100	0	0	0
13	1,500	0.016	140	130	160	0	0	0
25	2,000	0.025	215	175	220	0.0010	0	0
34	2,500	0.031	260	215	300	0.0015	0.0008	1.15
45	3,000	0.039	340	280	390	0.0015	0.0010	3.22
55	3,500	0.046	415	330	480	0.0020	0.0013	4.38
67	4,000	0.054	480	370	560	0.0020	0.0015	4.38
75	0	0.013	-40	30	-10	0.0003	0.0001	--
92	4,000	0.053	450	380	550	0.0020	0.0015	4.38
95	4,500	0.062	540	435	610	0.0020	0.0017	4.38
99	5,000	0.069	600	475	690	0.0022	0.0019	4.38
103	5,500	0.080	690	545	790	0.0023	0.0021	4.75
112	6,000	0.085	730	575	840	0.0023	0.0022	4.75
117	6,500	0.094	805	635	940	0.0023	0.0024	4.75
121	7,000	0.103	885	690	1000	0.0023	0.0026	4.75
125	7,500	0.112	940	740	1080	0.0025	0.0028	4.75
130	8,000	0.120	1030	810	1180	0.0030	0.0030	4.95
141	0	0.014	-30	100	25	0.0008	0.0002	--
166	8,000	0.126	1035	840	1155	0.0032	0.0031	5.18
170	8,500	0.133	1100	890	1265	0.0032	0.0033	5.18
173	9,000	0.142	1165	945	1330	0.0032	0.0035	5.18
175	9,500	0.150	1240	1000	1420	0.0032	0.0037	5.18
179	10,000	0.162	1325	1080	1520	0.0035	0.0039	5.20
182	10,500	0.173	1400	1160	1610	0.0040	0.0042	5.20
187	11,000	0.187	1480	1240	1700	0.0040	0.0044	5.20
189	11,500	0.200	1565	1315	1795	0.0040	0.0047	5.20
195	12,000	0.215	1655	1385	1900	0.0040	0.0050	5.20
201	12,500	0.232	1790	1560	2040	0.0040	0.0055	5.20
209	13,000	0.589	4530	3830	9400	0.0500	0.0407	6.31
211	13,450††	--	--	--	--	--	--	--

* Beam 1 was conventionally reinforced and used as a control beam for group 1.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Could not sustain load at this level. Flexural compressive failure occurred.

Table 8

Summary of Test Results for Beam 2*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
4	500	0.003	20	25	20	0	0	0
8	1,000	0.006	40	55	50	0	0	0
12	1,500	0.010	75	95	80	0	0	0
17	2,000	0.013	115	125	120	0	0	0
20	2,500	0.016	155	170	160	0	0	0
24	3,000	0.022	195	215	220	0	0	0
28	3,500	0.027	245	260	270	0	0	0
33	4,000	0.035	295	310	340	0	0	0
37	0	0.019	65	45	90	0	0	0
53	4,000	0.038	310	330	370	0	0	0
56	4,500	0.046	370	380	440	0	0	0
61	5,000	0.053	440	435	510	0	0	0
88	5,500	0.066	520	505	610	0.0008	0.0018	--
104	6,000	0.077	610	575	710	0.0015	0.0020	0.34
118	6,500	0.087	695	635	800	0.0018	0.0023	1.36
129	7,000	0.098	805	710	910	0.0020	0.0025	1.36
139	8,000	0.119	995	835	1120	0.0030	0.0029	1.96
153	0	0.033	235	185	275	0.0005	0.0008	--
160	8,000	0.127	1070	890	1205	0.0030	0.0031	1.96
165	9,000	0.145	1225	1025	1385	0.0035	0.0037	3.62
178	10,000	0.172	1440	1190	1615	0.0038	0.0041	4.05
193	11,000	0.200	1610	1355	1835	0.0040	0.0046	4.05
203	12,000	0.223	1800	1540	2055	0.0040	0.0050	4.05
212	13,000	0.251	2040	1750	2270	0.0045	0.0053	4.05
--	13,700††	--	--	--	--	--	--	--

* Beam 2 was similar to beam 1 (control beam) except 1/2-in.-long steel fibers were added to its concrete mixture.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Could not sustain load at this level.

Table 9

Summary of Test Results for Beam 3*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
10	500	0.001	20	30	60	0	0	0
13	1,000	0.002	45	60	75	0	0	0
15	1,500	0.007	80	100	105	0	0	0
19	2,000	0.012	105	140	150	0	0	0
20	2,500	0.016	155	185	200	0	0	0
22	3,000	0.023	210	235	245	0	0	0
27	3,500	0.029	270	205	310	0	0	0
32	4,000	0.038	350	350	385	0.0004	0.0015	--
40	0	0.021	100	75	125	--	0.0003	--
45	4,000	0.040	375	365	415	0.0010	0.0017	--
53	4,500	0.046	425	415	465	0.0010	0.0019	0.95
73	5,000	0.056	490	475	550	0.0020	0.0022	1.55
90	6,000	0.073	640	595	720	0.0020	0.0027	2.65
103	7,000	0.089	810	735	900	0.0020	0.0034	4.17
119	8,000	0.110	960	865	1095	0.0025	0.0040	4.25
121	0	0.024	215	185	270	0.0010	0.0011	--
122	8,000	0.109	1000	895	1145	0.0030	0.0042	4.25
126	9,000	0.129	1140	1020	1310	0.0030	0.0048	4.35
142	10,000	0.155	1315	1190	1505	0.0035	0.0056	4.35
153	11,000	0.184	1490	1375	1695	0.0035	0.0063	4.35
166	12,000	0.205	1675	1615	1910	0.0040	0.0072	4.38
180	13,000	0.231	[illegible]	[illegible]	[illegible]	0.0050	0.0080	4.38
194	14,000	0.26[illegible]	2000	2100	2350	0.0050	0.0088	4.38
207	14,000	0.308	2100	2295	2625	--	--	--
--	14,520††	--	--	--	--	--	--	--

* Beam 3 was similar to the control beam except that 1-in.-long steel fibers were added to its concrete mixture.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Could not sustain load at this level.

Table 10

Summary of Test Results for Beam 4*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain Compressive millionths	Concrete Strain Average** Tensile millionths	Maximum Dimension of Crack Width, in. M†	Maximum Dimension of Crack Width, in. D†	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0	0
5	500	0.002	25	25	20	0	0	0
8	1,000	0.006	50	60	60	0	0	0
15	1,500	0.012	105	100	120	0	0	0
19	2,000	0.019	195	155	210	0	0	0
25	2,500	0.029	305	200	310	0.0015	0.0015	2.76
34	3,000	0.038	415	260	470	0.0020	0.0020	3.65
48	3,500	0.046	520	315	570	0.0020	0.0025	3.74
60	4,000	0.059	610	375	660	0.0020	0.0029	3.96
72	0	0.031	175	50	170	0.0020	0.0011	3.96
75	4,000	0.060	620	390	710	0.0025	0.0031	4.24
82	5,000	0.078	775	475	890	0.0030	0.0038	4.28
100	6,000	0.114	970	640	1120	0.0040	0.0048	4.52
112	7,000	0.126	1120	700	1370	0.0040	0.0056	4.59
120	8,000	0.147	1215	795	1530	--	0.0063	4.59
125	9,000	0.180	1445	960	1790	0.0050	0.0074	4.62
132	10,000	0.205	1585	1040	1990	0.0060	0.0081	4.62
140	11,000	0.238	1775	1205	2220	0.0065	0.0090	4.62
152	12,000	0.319	2190	1610	2640	0.0180	0.0125	4.63
--	12,700††	--	--	--	--	--	--	--

* Beam 4 was conventionally reinforced and used as a control beam for group 2.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level.

Table 11

Summary of Test Results for Beam 5*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain Compressive millionths	Concrete Strain Average** Tensile millionths	Maximum Dimension of Crack Width, in. M†	Maximum Dimension of Crack Width, in. D†	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0	0
5	500	0.001	30	25	10	0	0	0
9	1,000	0.005	50	70	60	0	0	0
12	1,500	0.008	65	85	60	0	0	0
17	2,000	0.013	90	130	80	0	0	0
20	2,500	0.018	130	155	200	0	0	0
24	3,000	0.028	180	210	240	0	0	0
29	3,500	0.034	265	250	320	0	0	0
35	4,000	0.045	335	315	440	0.001	0.0013	4.0
65	0	0.012	110	60	110	--	0.0004	--
76	4,000	0.051	380	325	510	0.001	0.0014	4.0
84	5,000	0.077	505	420	670	0.002	0.0018	4.22
100	6,000	0.085	645	515	860	0.002	0.0025	4.70
130	7,000	0.107	790	635	1090	0.002	0.0031	4.70
135	8,000	0.128	950	735	1310	0.002	0.0040	4.70
156	0	0.024	205	155	250	--	0.0006	4.70
168	8,000	0.137	1015	785	1360	0.002	0.0041	4.71
190	9,000	0.157	1155	895	1520	0.002	0.0044	4.74
211	10,000	0.180	1305	1030	1720	0.002	0.0049	4.74
228	11,000	0.210	1455	1185	1920	0.003	0.0054	4.75
239	12,000	0.236	1625	1360	2160	0.003	0.0060	4.75
245	13,000	0.262	1770	1510	2340	0.003	0.0065	4.75
252	14,000	0.306	2015	1750	2750	0.003	0.0068	4.75
--	14,800††	--	--	--	--	--	--	--

* Beam 5 was similar to the control beam (beam 4) except the middle one-third section was reinforced with a 3- by 8- by 36-in. wire-mesh cage.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level.

Table 12

Summary of Test Results for Beam 6*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
8	500	0.003	45	35	30	0	0	0
12	1,000	0.009	80	70	120	0	0	0
15	1,500	0.013	105	110	140	0	0	0
17	2,000	0.017	125	140	170	0	0	0
21	2,500	0.022	155	185	200	0	0	0
26	3,000	0.029	180	235	235	0	0	0
28	3,500	0.033	220	275	270	0	0	0
33	4,000	0.039	225	320	340	0	0	0
38	0	0.007	60	30	50	0	0	0
51	4,000	0.039	260	320	330	0	0	0
66	5,000	0.050	365	410	430	0.0020	0.0005	2.0
79	6,000	0.071	770	610	680	0.0040	0.0046	2.0
99	7,000	0.095	980	755	890	0.0045	0.0075	3.1
121	8,000	0.131	1355	970	1100	0.0100	0.0113	3.9
141	0	0.038	295	100	130	0.0030	0.0036	--
168	8,000	0.134	1375	965	1160	0.0070	0.0144	4.6
195	9,000	0.165	1675	1105	1370	0.0180	0.0202	4.6
223	10,000	0.202	2044	1280	1780	0.0340	0.0292	4.6
238	11,000	0.274	2320	1670	2570	0.0450	0.0433	4.6
--	12,000††	--	--	--	--	--	--	--

* Beam 6 was similar to beam 4 (control beam) except the bottom 1-1/2 in. was fabricated with epoxy-resin concrete.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level.

Table 13

Summary of Test Results for Beam 7*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
5	500	0.006	70	15	40	0	0	0
10	1,000	0.011	95	75	80	0	0	0
13	1,500	0.014	105	100	100	0	0	0
20	2,000	0.017	135	130	140	0	0	0
26	2,500	0.021	155	165	180	0	0	0
31	3,000	0.025	175	200	210	0	0	0
35	3,500	0.030	200	240	240	0	0	0
40	4,000	0.034	220	280	280	0	0	0
50	0	0.005	60	20	40	0	0	0
55	4,000	0.033	215	285	280	0	0	0
57	5,000	0.041	260	370	350	0	0	0
60	6,000	0.064	305	460	360	0.008	0.0010	3.32
73	7,000	0.084	715	725	500	0.007	0.0009	3.70
90	8,000	0.116	950	900	670	0.008	0.0070	4.16
102	0	0.049	325	220	220	0.002	0.0019	4.18
112	8,000	0.131	1075	975	730	0.006	0.0074	4.18
120	9,000	0.153	1235	1115	1190	0.008	0.0086	4.42
130	10,000	0.179	1540	1280	1400	0.009	0.0100	4.58
135	11,000	0.216	1820	1615	1960	0.017	0.0124	5.22
155	12,000††	0.624	--	--	--	--	--	--

* Beam 7 was similar to control beam 4 except that the bottom 3 in. was fabricated with epoxy-resin concrete.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level.

Table 14

Summary of Test Results for Beam 8*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
5	500	0.002	30	35	50	0	0	0
8	1,000	0.004	50	70	65	0	0	0
13	1,500	0.011	75	150	105	0	0	0
18	2,000	0.019	110	195	170	0	0	0
34	2,500	0.028	160	245	245	0.0005	0.0011	1.15
39	3,000	0.040	335	330	370	0.0010	0.0014	2.20
54	3,500	0.052	455	400	445	0.0010	0.0018	3.1
73	4,000	0.064	550	475	620	0.0013	0.0022	3.1
84	0	0.025	190	150	180	--	0.0008	--
92	4,000	0.069	590	490	660	--	0.0025	--
94	5,000	0.089	750	615	855	0.0025	0.0031	3.98
109	6,000	0.113	915	755	1085	0.0032	0.0039	3.98
122	7,000	0.143	1115	945	1315	0.0040	0.0047	4.18
134	8,000	0.169	1295	1125	1565	0.0045	0.0056	4.6
141	0	0.046	250	285	320	--	0.0012	--
204	8,000	0.178	1290	1180	1595	0.0050	0.0055	4.6
217	9,000	0.208	1455	1370	1845	0.0050	0.0063	4.6
221	10,000	0.237	1625	1560	2070	0.0050	0.0069	4.6
227††	11,000	0.269	1895	1820	2360	0.0075	0.0080	4.6

* This beam was used as control beam for group 3.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam failed in a flexural compressive mode.

Table 15

Summary of Test Results for Beam 9*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
15	500	0.004	10	15	20	0	0	0
19	1,000	0.008	25	60	40	0	0	0
25	1,500	0.010	50	105	90	0	0	0
29	2,000	0.014	70	145	140	0	0	0
35	2,500	0.018	105	200	180	0.001	0.0007	1.67
55	3,000	0.022	145	240	220	0.001	0.0008	2.10
115	0	0.004	35	75	60	--	0.0003	--
130	3,000	0.032	190	310	390	--	0.0011	--
132	3,500	0.039	270	365	390	0.0015	0.0013	3.50
140	4,000	0.051	355	460	520	0.0020	0.0018	3.80
155	0	0.016	85	145	150	--	0.0005	--
160	4,000	0.054	375	465	560	--	0.0018	--
164	5,000	0.071	520	575	740	0.0025	0.0023	4.3
173	6,000	0.095	660	730	980	0.0030	0.0028	4.3
182	7,000	0.118	825	885	1190	0.0035	0.0033	4.75
217	0	0.031	145	280	260	--	0.0007	--
227	7,000	0.117	835	975	1210	--	0.0033	--
241	8,000	0.144	940	1085	1360	0.0035	0.0038	4.75
250	9,000	0.165	1055	1235	1570	0.0035	0.0042	4.75
259	10,000	0.195	1215	1430	1760	0.0045	0.0047	4.95
280	11,000	0.224	1330	1645	1920	0.0050	0.0054	4.95
284	12,000	0.257	1500	1880	2230	0.0055	0.0062	4.95
291	13,000	0.294	[illegible]	[illegible]	2640	0.0030	0.0095	5.6
293††	13,000	0.402	--	5570	5570	--	0.0434	--

* Beam 9 was conventionally reinforced and had a double layer of wire mesh attached to its regular layer of tensile reinforcement.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam failed in a flexural compressive mode.

Table 16

Summary of Test Results for Beam 10*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
15	500	0.003	15	30	30	0	0	0
18	1,000	0.006	30	65	70	0	0	0
20	1,500	0.010	55	90	90	0	0	0
23	2,000	0.014	85	125	120	0	0	0
27	2,500	0.019	115	165	170	0	0	0
31	3,000	0.029	165	200	230	0	0	0
39	3,500	0.033	250	250	315	0.0005	0.0011	0.55
57	4,000	0.043	345	320	445	0.0008	0.0012	2.45
71	0	0.015	105	50	135	--	--	--
76	4,000	0.046	325	325	475	--	0.0014	--
93	5,000	0.063	545	445	710	0.0010	0.0022	2.93
105	6,000	0.079	670	540	860	0.0010	0.0027	3.72
116	7,000	0.101	820	645	1100	0.0012	0.0034	4.15
124	8,000	0.120	950	770	1255	0.0015	0.0040	4.30
131	0	0.026	185	150	240	--	0.0008	--
184	8,000	0.125	950	790	1250	0.0015	0.0040	4.30
199	9,000	0.144	1080	945	1440	0.0018	0.0047	4.58
217	10,000	0.165	1205	1070	1610	0.0020	0.0052	4.9
239	11,000	0.189	1320	1235	1795	0.0024	0.0059	4.9
245	12,000	0.213	1490	1410	2025	0.0025	0.0065	4.9
267	13,000	0.239	1625	1590	2234	0.0025	0.0071	4.9
279	14,000	0.267	1840	1800	2485	0.0025	0.0079	4.9
294	15,000††	--	--	--	--	--	--	--

* Beam 10 used a 3- by 8- by 36-in. wire-mesh cage with its vertical wires clipped throughout its inner 24-in. section.
** Average tensile strain was average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level.

Table 17

Summary of Test Results for Beam 11*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
14	500	0.003	15	30	30	0	0	0
16	1,000	0.004	35	60	40	0	0	0
1[illegible]	1,500	0.008	55	85	70	0	0	0
[illegible]	[illegible],000	0.012	75	120	100	0	0	0
[illegible]	[illegible]	0.018	100	160	140	0	0	0
[illegible]1	3,000	0.024	140	200	190	0	0	0
[illegible]	[illegible],500	0.03[illegible]	[illegible]15	250	2[illegible]0	0	0	0
[illegible]	4,000	0.042	340	[illegible]05	3[illegible]0	0.00[illegible]0	0.0013	4.7[illegible]
71	0	0.007	[illegible]0	50	110	--	0.00[illegible]	--
[illegible]	4,000	0.044	400	325	4[illegible]0	0.00[illegible]	0.001[illegible]	4.74
[illegible]	5,000	0.062	57[illegible]	425	[illegible]10	0.00[illegible]	0.00[illegible]	4.76
107	6,000	0.083	7[illegible]	530	[illegible]10	0.0025	0.0032	4.76
1[illegible]1	7,000	0.104	[illegible]	640	1000	0.00[illegible]	0.0040	[illegible]
1[illegible]	8,000	0.124	1070	765	1250	0.0040	0.00[illegible]	[illegible].59
140	9,000	0.149	1[illegible]	[illegible]	1400	0.00[illegible]	0.00[illegible]6	5.60
1[illegible]4	10,000	0.17[illegible]	13[illegible]	10[illegible]	1[illegible]	0.00[illegible]	0.00[illegible]4	[illegible]
[illegible]	11,[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
164	1[illegible],000	0.[illegible]3	1[illegible]	1305	1900	0.00[illegible]	0.0091	[illegible].60
17[illegible]	1[illegible],000††	--	--	--	--	--	--	--

* This beam was used as control beam for group 4.
** Average tensile strain is average over middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level and failed in a flexural compressive mode.

Table 18

Summary of Test Results for Beam 12*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
6	500	0.005	20	25	30	0	0	0
9	1,000	0.009	35	60	60	0	0	0
12	1,500	0.014	55	85	100	0	0	0
15	2,000	0.019	75	115	140	0	0	0
18	2,500	0.024	95	150	170	0	0	0
22	3,000	0.029	120	185	210	0	0	0
26	3,500	0.033	140	215	230	0	0	0
28	4,000	0.037	160	250	270	0	0	0
34	0	0.004	10	5	30	0	0	0
39	4,000	0.038	160	245	260	0	0	0
44	5,000	0.048	200	315	340	0	0	0
46	6,000	0.056	245	365	420	0	0	0
51	7,000	0.065	290	460	490	0	0	0
56	8,000	0.078	340	540	580	0	0	0
63	0	0.009	25	50	50	0	0	0
67	8,000	0.079	350	560	590	0	0	0
72	9,000	0.089	400	640	670	0	0	0
77	10,000	0.102	465	735	780	0	0	0
82	11,000	0.116	530	850	900	0	0	0
86	12,000	0.147	610	975	980	0.025	0.021	6.37
--	13,000††	--	--	--	--	--	--	--
--	13,900‡	--	--	--	--	--	--	--

* In the lower 2-1/2 in. of beam 12, epoxy-resin concrete was used in lieu of normal concrete.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† The beam began yielding with no additional load.
‡ Beam experienced a flexural compressive failure at this load.

Table 19

Summary of Test Results for Beam 13*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
4	500	0.002	15	30	60	0	0	0
6	1,000	0.005	35	60	60	0	0	0
9	1,500	0.009	55	85	80	0	0	0
13	2,000	0.013	70	115	100	0	0	0
15	2,500	0.017	90	145	140	0	0	0
19	3,000	0.021	110	180	160	0	0	0
22	3,500	0.025	130	210	200	0	0	0
26	4,000	0.029	155	240	220	0	0	0
28	0	0	10	0	10	0	0	0
32	4,000	0.029	150	240	220	0	0	0
38	5,000	0.038	475	390	400	0.0010	0.0034	3.20
60	6,000	0.058	670	500	500	0.0025	0.0049	3.20
68	7,000	0.076	835	610	620	0.0040	0.0064	3.20
80	8,000	0.094	1020	735	800	0.0030	0.0063	3.80
91	0	0.019	190	130	180	--	0.0019	--
189	8,000	0.108	1040	760	1020	0.0030	0.0086	4.15
205	9,000	0.125	1185	890	1180	0.0030	0.0101	4.15
214	10,000	0.144	1375	1020	1390	0.0040	0.0113	4.15
222	11,000	0.169	1575	1215	1640	0.0050	0.0131	5.04
231	12,000	0.328	7500	2490	4200	0.0450	0.0523	6.30
--	12,700††	--	--	--	--	--	--	--

* Beam 13 used a 3-in. layer of epoxy-resin concrete in its lower 3-in. zone.
** Average tensile strain is average for middle one-third of beam.
† M - measured using microscope; D - measured using Demec disk.
†† Beam could not sustain load at this level.

Table 20

Summary of Test Results for Beam 14*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain millionths	Concrete Strain: Compressive millionths	Concrete Strain: Average** Tensile millionths	Maximum Dimension of Crack: Width, in. M†	Maximum Dimension of Crack: Width, in. D†	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0	0
9	500	0.003	10	0	5	0	0	0
12	1,000	0.006	31	20	35	0	0	0
16	1,500	0.009	55	45	65	0	0	0
20	2,000	0.014	80	80	75	0	0	0
26	2,500	0.019	120	135	110	0	0	0
30	3,000	0.026	170	175	150	0	0	0
36	3,500	0.031	210	215	180	0	0	0
40	4,000	0.035	255	245	230	0	0	0
45	0	0.004	38	15	10	0	0	0
52	4,000	0.037	265	245	225	0	0	0
55	5,000	0.048	390	330	330	0	0	0
67	6,000	0.065	560	460	510	0.0010	0.0030	1.34
92	7,000	0.089	755	610	730	0.0015	0.0045	1.79
108	8,000	0.106	870	720	885	0.0020	0.0055	1.79
126	0	0.023	130	130	180	0.0010	0.0015	1.79
135	8,000	0.115	915	765	960	0.0020	0.0060	2.58
147	9,000	0.131	1040	875	1105	0.0030	0.0070	2.59
161	10,000	0.154	1185	1010	1310	0.0045	0.0082	3.68
167	11,000	0.178	1350	1155	1497	0.0045	0.0092	3.68
178	12,000	0.207	1500	1335	1680	0.0045	0.0102	3.70
188	13,000	0.239	1525	1475	1945	0.0060	0.0122	3.70
195	14,000††	--	--	--	--	--	--	-

* Beam 14 was conventionally reinforced with its lower 3 in. consisting of a 0.5-in.-long fiber/regular concrete mixture.

** Average tensile strain is average for middle one-third of beam.

† M - measured using microscope; D - measured using Demec disk.

†† No readings could be obtained at this load level.

Table 21

Comparison of Initial Cracking and Maximum Anticipated Service Loads for the Smaller Beams

Group No.	Beam No.	Type of Special Crack-Arrest Technique Used	Initial Cracking Load (P_c), lb	Maximum Anticipated Live Service Load (P_{1sl}), lb*	P_c/P_{1sl} %	P_c of Beams Using Special Reinforcing Techniques/P_c of Control Beams
1	1	None; conventionally reinforced control beam of group 1	2,000	5370	37	--
1	2	0.5-in.-long steel fibers added throughout beam's cross section	5,500	5370	102	2.75
1	3	1-in.-long steel fibers added throughout beam's cross section	4,000	5370	75	2.00
2	4	None; conventionally reinforced control beam of group 2	2,500	4114	61	--
2	5	Inner 36-in. section of beam reinforced with 3- by 8-in. wire-mesh cage	4,000	4114	97	1.60
2	6	Lower (bottom) 1-1/2 in. fabricated with epoxy-resin concrete	5,000	4114	122	2.00
2	7	Lower (bottom) 3 in. fabricated with epoxy-resin concrete	6,000	4114	146	2.40
3	8	None; conventionally reinforced control beam of group 3	2,500	4055	62	--
3	9	Double layer of wire mesh attached to lower edge of tensile reinforcement	2,500	4055	62	1.00
3	10	Inner 36-in. section of beam reinforced with 3- by 8-in. wire-mesh cage	3,500	4055	86	1.40
4	11	None; conventionally reinforced control beam of group 4	4,000	4712	85	--
4	12	Lower 2-1/2 in. fabricated with epoxy-resin concrete	12,000	4712	255	3.00
4	13	Lower 3 in. fabricated with epoxy-resin concrete	5,000	4712	106	1.25
4	14	Lower 3 in. fabricated with 0.5-in.-long fiber/concrete mixture	6,000	4712	127	1.50

Note: All beams contained the same amount and type of conventional reinforcement for flexure.

* Maximum anticipated live service loads were assumed to be the loads that, when combined with their dead loads, produced concrete compressive stresses of 0.45 f'_c (Appendix A).

Table 22

Compress . and Tensile Strengths of Individual Batches of Plain Concrete for Larger Beams

Batch No.	Age of Concrete When Tested days	Representing Beam No.	Compressive Strength psi	Splitting Tensile Strength psi
1	28	1-2	3000	375
2	28	3-4	3120	405
3	28	5	3050	350
4	28	6-7	2920	360
5	28	8	2800	325

Table 23

Compressive and Tensile Strengths of Individual Batches of Steel-Fiber-Reinforced Concrete Mixture for Larger Beams

Batch No.	Age of Fiber-Reinforced Concrete When Tested, days	Representing Beam No.	Compressive Strength psi	Splitting Tensile Strength psi
1	28	2	3910	675
2	28	4	3940	620
3	28	6-7	3930	560

Table 24

Summary of Test Results for Beam 1L*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain Compressive Millionths	Concrete Strain Average** Tensile Millionths	Maximum Dimension of Crack Width in.	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0
14	1,000	0.024	30	68	59	0	0
19	2,000	0.067	88	165	150	0.001	0.90
37	2,000	0.073	88	184	167	0.001	0.90
40	3,000	0.127	167	280	306	0.003	3.60
50	3,000	0.140	189	300	324	0.003	3.60
54	4,000	0.207	321	411	517	0.003	3.60
61	4,000	0.223	323	430	539	0.003	3.60
64	5,000	0.289	424	536	733	0.005	6.60
71	5,000	0.313	450	584	766	0.005	6.60
75	6,000	0.379	586	750	780	0.006	7.15
85	6,000	0.401	594	820	974	0.006	7.15
90	7,000	0.468	725	847	1131	0.006	7.15
95	7,000	0.487	737	887	1150	0.006	7.15
98	8,000	0.547	830	978	1306	0.014	8.15
107	8,000	0.581	845	1046	1350	0.014	8.15
112	0	0.132	78	264	265	--	--
130	0	0.125	76	242	253	--	--
136	4,000	0.337	367	654	771	--	--
138	8,000	0.579	811	1074	1359	0.014	8.15
142	9,000	0.654	947	1208	1540	0.014	8.15
148	9,000	0.689	977	1281	1581	0.014	8.15
150	10,000	0.756	1105	1401	1611	0.025	8.35
157	10,000	0.793	1135	1509	1809	0.025	8.35
160	11,000	0.868	1258	1648	1963	0.025	8.35
168	11,000	0.902	1279	1730	1996	0.025	8.35
172	12,000	0.966	1390	1871	2173	0.030	8.75
179	12,000	1.018	1429	2001	2234	0.030	8.75
182	13,000	1.092	1554	2138	2423	0.030	8.75
187	13,000	1.141	1596	2285	2479	0.030	8.75
190	14,000	1.473	2534	--	6973	0.060	9.50
--	12,200†	2.90††					
--	0†	2.70††					

* Beam 1L was conventionally reinforced and used as a control beam for the short-term static tests of group 1L.

** Average tensile strain is average for middle one-third of beam.

† Loads after failure.

†† String-line measurements.

Table 25

Summary of Test Results for Beam 2L*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain Compressive Millionths	Concrete Strain Average** Tensile Millionths	Maximum Dimension of Crack Width in.	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0
8	1,000	0.021	25	49	50	0	0
20	2,000	0.048	59	104	104	0	0
23	3,000	0.082	104	178	203	0	0
30	4,000	0.125	161	253	289	0	0
34	5,000	0.188	255	343	463	0.002	1.06
58	6,000	0.264	407	454	637	0.003	5.50
75	6,000	0.276	438	472	671	0.003	5.50
78	7,000	0.327	551	556	796	0.004	6.50
90	7,000	0.352	577	573	841	0.004	6.50
94	0	0.087	170	113	210	--	--
143	0	0.082	163	104	197	--	--
148	4,000	0.223	383	355	397	--	--
152	7,000	0.346	591	570	876	0.004	6.50
153	8,000	0.398	684	638	974	0.004	6.50
165	8,000	0.419	711	660	1013	0.004	6.50
169	9,000	0.466	826	739	1183	0.004	6.50
175	9,000	0.487	849	769	1223	0.004	6.50
180	10,000	0.538	954	849	1434	0.009	7.25
185	10,000	0.544	983	887	1493	0.009	7.25
190	11,000	0.616	1084	972	1664	0.009	7.25
195	11,000	0.634	1109	996	1683	0.009	7.25
199	12,000	0.690	1212	1085	1813	0.010	7.85
211	12,000	0.724	1249	1147	1866	0.010	7.85
215	13,000	0.779	1352	1244	1976	0.010	7.85
219	13,000	0.791	1372	1274	2029	0.010	7.85
223	14,000	0.855	1474	1363	2166	0.015	8.25
235	14,000	0.895	1517	1449	2199	0.015	8.25
240	15,000	0.953	1607	1546	2320	0.015	8.25
245	15,000	0.975	1633	1596	2391	0.015	8.25
250	16,000	1.042	1742	1710	2530	0.035	8.70
260	16,000	1.275	1766	2232	2637	0.035	8.70
--	17,000	1.500	--	--	--	--	--
--	14,800†	3.20††	--	--	--	0.20	14.00

* Beam 2L was conventionally reinforced, had an additional 2.5 percent by volume of 0.5-in.-long steel fibers added to its lower 3-3/8-in. tensile zone, and was subjected to the short-term static tests outlined for the members of group 1L.

** Average tensile strain is average for middle one-third of beam.

† Load after failure; beam collapsed at this load.

†† String-line measurement.

Table 26

Summary of Test Results for Beam 3L*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain: Compressive Millionths	Concrete Strain: Average** Tensile Millionths	Maximum Dimension of Crack: Width in.	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0
15	1,000	0.021	50	20	44	0	0
20	2,000	0.045	99	99	137	0	0
25	3,000	0.093	174	170	314	0.001	0.68
40	4,000	0.129	304	261	477	0.002	2.05
50	4,000	0.146	321	275	556	0.002	2.05
56	5,000	0.193	426	349	657	0.004	4.40
64	5,000	0.216	448	361	729	0.004	4.40
68	6,000	0.263	552	427	916	0.005	5.40
78	6,000	0.279	572	444	924	0.005	5.40
83	7,000	0.327	676	512	1146	0.006	6.60
95	7,000	0.351	704	538	1187	0.006	6.60
98	8,000	0.396	791	597	1364	0.006	6.60
105	8,000	0.416	813	621	1373	0.006	6.60
112	0	0.099	156	89	271	--	--
152	0	0.095	151	83	263	--	--
160	4,000	0.231	464	336	686	--	--
166	8,000	0.405	814	611	1464	0.006	6.60
170	9,000	0.461	911	677	1609	0.006	7.80
178	9,000	0.487	944	715	1631	0.006	7.80
183	10,000	0.533	1034	778	1763	0.006	7.80
186	10,000	0.550	1053	800	1784	0.006	7.80
190	11,000	0.599	1146	871	1931	0.007	7.85
200	11,000	0.628	1177	918	1973	0.007	7.85
203	12,000	0.674	1265	973	2113	0.007	7.85
210	12,000	0.697	1288	1015	2136	0.007	7.85
213	13,000	0.747	1398	1104	2271	0.010	7.90
223	13,000	0.776	1418	1145	2310	0.010	7.90
225	14,000	0.831	1520	1233	2464	0.010	7.90
233	14,000	0.852	1543	1266	2489	0.010	7.90
235	15,000	0.906	1638	1336	2643	0.012	7.90
242	15,000	0.939	1676	1403	2684	0.012	7.90
246	16,000	1.004	1789	1507	2853	0.013	8.10
255	16,000	1.034	1816	1563	2894	0.013	8.10
258	17,000	1.102	1951	1684	3107	0.013	8.10
265	17,000	1.162	1963	1900	3307	0.013	8.10
--	18,000	1.240	--	--	--	0.080	9.90
--	15,000†	4.05††	--	--	--	0.150	--
--	0†	3.40††	--	--	--	--	--

* Beam 3L used a 4- by 11- by 84-in. wire-mesh cage throughout its inner 84-in. section and was subjected to the short-term static tests outlined for members of group 1L.

** Average tensile strain is average for middle one-third of beam.

† Load after failure.

†† String-line measurements.

Table 27

Summary of Test Results for Beam 4L*

Time min	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain: Compressive Millionths	Concrete Strain: Average** Tensile Millionths	Maximum Dimension of Crack: Width in.	Maximum Dimension of Crack: Depth in.
0	0	0	0	0	0	0	0
4	1,000	0.020	40	48	49	0	0
7	2,000	0.045	102	107	106	0	0
12	3,000	0.076	172	179	174	0	0
17	4,000	0.116	214	264	297	0.0005	Hairline
24	5,000	0.173	318	355	460	0.001	0.90
37	5,000	0.199	352	388	499	0.001	0.90
39	6,000	0.243	420	485	636	0.003	1.85
49	6,000	0.266	466	510	684	0.003	1.85
52	7,000	0.315	526	590	851	0.003	1.85
57	7,000	0.337	587	620	886	0.003	1.85
60	8.000	0.393	662	718	1041	0.005	4.70
72	8,000	0.429	888	762	1097	0.005	4.70
77	0	0.099	236	151	254	--	--
102	0	0.092	232	142	240	--	--
107	4,000	0.241	586	422	627	--	--
109	8,000	0.420	1090	751	1111	0.005	4.70
112	9,000	0.477	1182	850	1250	0.005	4.70
122	9,000	0.502	1186	878	1274	0.005	4.70
125	10,000	0.559	1298	977	1439	0.007	6.30
130	10,000	0.591	1318	1029	1481	0.007	6.30
133	11,000	0.647	1352	1114	1621	0.008	7.30
142	11,000	0.679	1408	1182	1683	0.008	7.30
146	12,000	0.730	1496	1265	1809	0.008	7.30
152	12,000	0.760	1520	1324	1874	0.008	7.30
155	13,000	0.816	1608	1414	2011	0.010	8.00
164	13,000	0.875	1676	1473	2099	0.010	8.00
167	14,000	0.921	1768	1628	2241	0.010	8.00
175	14,000	0.966	1846	1700	2309	0.010	8.00
177	15,000	1.030	1968	1819	2484	0.010	8.00
185	15,000	1.089	2042	1917	2584	0.010	8.00
190	16,000	1.162	2090	2668	--	0.050	8.65
--	6,000†	5.80††	--	--	--	0.30	--
--	0†	4.90††	--	--	--	--	--

* Beam 4L was conventionally reinforced, had an additional 2.5 percent by volume of 0.5-in.-long steel fibers added throughout its entire cross section, and was subjected to the short-term static tests outlined for the members of group 1L.

** Average tensile strain is average for middle one-third of beam.

† Load after failure.

†† String-line measurements.

Table 28

Summary of Test Results for Beam 5L*

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain Compressive Millionths	Concrete Strain Average** Tensile Millionths	Maximum Dimension of Crack Width in.	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0
†	2,000	0.041	43	70	67	0	0
1	2,000	0.061	72	71	91	0.002	1.06
2	2,000	0.068	100	70	107	0.002	1.06
3	2,000	0.073	102	70	109	0.002	1.06
6	2,000	0.082	104	82	109	0.002	1.06
7	2,000	0.089	106	89	116	0.002	1.06
8	2,000	0.095	106	109	116	0.002	1.06
8	3,000	0.114	118	141	164	0.002	3.20
13	3,000	0.134	168	142	190	0.002	3.20
13	3,000	0.139	179	150	213	0.004	3.95
15	3,000	0.146	184	160	213	0.004	3.95
16	3,000	0.152	203	165	213	0.004	3.95
17	3,000	0.157	211	185	213	0.004	3.95
20	3,000	0.166	212	207	234	0.004	3.95
22	3,000	0.171	223	218	246	0.004	3.95
27	3,000	0.184	252	223	246	0.004	4.15
27	5,000	0.275	332	348	470	0.005	4.85
31	5,000	0.325	414	418	559	0.006	5.35
34	5,000	0.339	444	416	579	0.006	6.05
38	5,000	0.357	472	454	601	0.006	6.05
43	5,000	0.370	510	478	623	0.006	6.05
49	5,000	0.381	537	493	629	0.006	6.05
49	6,000	0.417	561	549	713	0.008	6.15
55	6,000	0.444	594	608	754	0.009	6.65
63	6,000	0.476	631	641	786	0.009	6.65
63	7,000	0.523	663	718	917	0.010	7.20
79	7,000	0.575	695	853	976	0.010	7.20
84	7,000	0.585	705	879	1003	0.010	7.20
84	8,000	0.621	773	951	1094	0.011	7.44

(Continued)

* Beam 5L used a 4- by 11- by 84-in. wire-mesh cage throughout its inner 84-in. section and was subjected to the long-term static loading outlined for members for group 2L.

** Average tensile strain is average for middle one-third of beam.

† Immediately after loading.

Table 28 (Concluded)

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain: Compressive Millionths	Concrete Strain: Average Tensile Millionths	Maximum Dimension of Crack: Width in.	Maximum Dimension of Crack: Depth in.
91	8,000	0.674	836	1029	1160	0.011	7.44
99	8,000	0.693	832	1029	1176	0.011	7.44
106	8,000	0.701	885	1077	1181	0.011	7.44
112	8,000	0.715	891	1135	1217	0.011	7.44
112	9,000	0.753	928	1184	1311	0.012	7.50
120	9,000	0.776	937	1276	1354	0.012	7.50
128	9,000	0.798	945	1317	1374	0.012	7.50
128	10,000	0.841	1019	1389	1487	0.014	7.80
135	10,000	0.856	1046	1425	1523	0.014	7.80
139	10,000	0.872	1088	1453	1541	0.014	7.80
139	11,000	0.919	1178	1526	1677	0.014	7.80
148	11,000	0.951	1231	1581	1730	0.014	7.80
153	11,000	0.961	1231	1611	1753	0.014	7.80
153	12,000	1.058	1281	1690	1874	0.017	8.00
170	12,000	1.081	1286	1780	1931	0.017	8.00
177	12,000	1.097	1291	1776	1953	0.017	8.00
177	13,000	1.148	1377	1853	2080	0.018	8.00
184	13,000	1.171	1417	1923	2126	0.018	8.00
191	13,000	1.183	1426	1957	2139	0.018	8.00
191	14,000	1.235	1521	2032	2271	0.020	8.05
198	14,000	1.268	1540	2088	2344	0.020	8.05
206	14,000	1.293	1542	2110	2396	0.020	8.05
206	15,000	1.335	1605	2175	2501	0.023	8.10
212	15,000	1.359	1651	2225	2550	0.023	8.10
219	15,000	1.379	1666	2281	2580	0.023	8.10
221	15,000	1.383	1681	2296	2596	0.023	8.10
221	16,000	1.439	1764	2384	2733	0.030	8.10
228	16,000	1.470	1814	2399	2800	0.030	8.10
235	16,000	1.480	1839	2544	2849	0.030	8.10
235	17,000	1.532	1936	2614	2980	0.040	8.25
241	17,000	1.620	1948	2699	3129	0.085	8.65
249	17,000	1.667	1959	2753	3283	0.085	8.65
249	17,500	2.55††	--	3268	--	0.125	9.50
249	17,900	3.25††	--	--	--	0.150	14.00
249	0‡	2.45††	--	--	--	0.125	14.00

†† String-line measurements.
‡ Load after failure.

Table 29

Summary of Test Results for Beam 6L*

				Concrete Strain		Maximum Dimension of Crack	
Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Compressive Millionths	Average** Tensile Millionths	Width in.	Depth in.
0	0	0	0	0	0	0	0
†	2,000	0.040	60	50	80	0	0
0.10	2,000	0.042	60	53	86	0	0
1	2,000	0.045	61	64	86	0	0
2	2,000	0.059	74	74	87	0	0
3	2,000	0.060	87	74	90	0	0
6	2,000	0.064	94	75	97	0	0
7	2,000	0.065	96	75	100	0	0
7	4,000	0.116	225	151	244	0	0
8	4,000	0.130	264	186	284	0.0005	Hairline
9	4,000	0.157	280	193	296	0.001	0.50
13	4,000	0.176	301	205	311	0.001	1.10
17	4,000	0.185	319	260	321	0.001	1.55
20	4,000	0.203	330	277	336	0.001	1.65
20	5,000	0.240	429	362	431	0.002	1.65
22	5,000	0.263	435	402	487	0.002	1.65
24	5,000	0.268	445	405	489	0.003	3.55
29	5,000	0.299	461	424	521	0.003	3.55
35	5,000	0.314	485	481	540	0.003	3.55
41	5,000	0.338	526	548	569	0.003	4.95
41	6,000	0.370	594	601	664	0.004	5.35
52	6,000	0.376	598	605	660	0.005	6.05
62	6,000	0.452	695	708	776	0.006	6.10
62	7,000	0.498	766	817	903	0.006	6.10
66	7,000	0.504	766	890	911	0.007	6.45
72	7,000	0.530	769	897	953	0.007	6.45
79	7,000	0.552	794	900	967	0.007	6.45
84	7,000	0.558	849	913	980	0.007	6.45
84	8,000	0.613	986	1012	1120	0.007	6.45
92	8,000	0.626	1004	1022	1133	0.007	6.45

(Continued)

* Beam 6L was conventionally reinforced, had an additional 2.5 percent by volume of 0.5-in.-long steel fibers added to its lower 3-3/8-in. tensile zone, and was subjected to the long-term static tests outlined for members of group 2L.

** Average tensile strain is average for middle one-third of beam.

† Immediately after loading.

Table 29 (Concluded)

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain: Compressive Millionths	Concrete Strain: Average Tensile Millionths	Maximum Dimension of Crack: Width in.	Maximum Dimension of Crack: Depth in.
99	8,000	0.635	1012	1033	1147	0.007	6.45
106	8,000	0.645	1019	1048	1163	0.007	6.45
114	8,000	0.677	1021	1078	1209	0.007	6.45
114	9,000	0.713	1090	1160	1304	0.010	6.95
121	9,000	0.720	1099	1180	1337	0.010	6.95
127	9,000	0.737	1154	1191	1349	0.010	6.95
127	10,000	0.789	1215	1241	1486	0.011	7.05
134	10,000	0.802	1231	1249	1481	0.011	7.05
141	10,000	0.832	1301	1263	1569	0.011	7.05
141	11,000	0.869	1412	1362	1644	0.012	7.10
148	11,000	0.879	1454	1417	1677	0.012	7.10
155	11,000	0.907	1463	1485	1733	0.012	7.10
155	12,000	0.958	1471	1514	1863	0.014	7.20
162	12,000	0.971	1476	1519	1877	0.014	7.20
169	12,000	1.010	1545	1547	1931	0.014	7.20
169	13,000	1.041	1645	1658	2067	0.015	7.55
176	13,000	1.054	1646	1818	2086	0.015	7.55
183	13,000	1.080	1648	1827	2136	0.015	7.55
183	14,000	1.137	1665	1918	2281	0.017	7.75
190	14,000	1.150	1715	1956	2289	0.017	7.75
197	14,000	1.185	1735	1995	2366	0.017	7.75
197	15,000	1.232	1800	2084	2486	0.019	7.88
204	15,000	1.246	1815	2119	2501	0.019	7.88
211	15,000	1.286	1835	2142	2573	0.019	7.88
211	16,000	1.337	1900	2238	2704	0.035	8.00
217	16,000	1.353	1905	2248	2733	0.035	8.00
225	16,000	1.387	1910	2260	2790	0.035	8.00
225	17,000	1.445	1965	2393	2931	0.050	8.25
232	17,000	1.486	2000	2609	3041	0.050	8.25
239	17,000	1.500	2025	2613	3051	0.050	8.25
239	17,400	1.851	2145	--	3880	0.065	14.00
239	13,500††	3.25‡	--	--	--	--	--
239	0††	2.75‡	--	--	--	--	--

†† Load after failure.
‡ String-line measurements.

Table 30

Summary of Test Results for Beam 7L*

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain Compressive Millionths	Concrete Strain Average** Tensile Millionths	Maximum Dimension of Crack Width in.	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0
†	2,000	0.055	58	20	94	0	0
0.17	2,000	0.059	82	25	96	0	0
1	2,000	0.065	100	37	100	0	0
2	2,000	0.076	104	43	104	0	0
5	2,000	0.080	122	58	123	0	0
7	2,000	0.087	123	75	129	0	0
7	3,000	0.112	147	175	187	0	0
8	3,000	0.121	178	175	187	0.001	0.81
12	3,000	0.139	193	188	201	0.001	1.55
16	3,000	0.156	210	202	216	0.001	1.55
20	3,000	0.171	223	215	229	0.001	1.55
20	4,000	0.201	270	265	287	0.002	1.55
23	4,000	0.217	282	295	317	0.002	1.55
28	4,000	0.238	315	319	339	0.002	1.55
34	4,000	0.253	317	320	350	0.003	2.16
35	4,000	0.257	316	349	359	0.003	2.16
35	5,000	0.287	421	410	430	0.003	2.16
40	5,000	0.318	481	467	489	0.004	2.60
49	5,000	0.360	483	501	546	0.004	2.60
49	6,000	0.384	595	549	599	0.004	2.60
51	6,000	0.397	639	592	641	0.004	2.77
61	6,000	0.420	649	622	653	0.004	3.70
63	6,000	0.432	664	653	669	0.004	3.70
63	7,000	0.462	733	733	739	0.005	4.72
70	7,000	0.491	815	798	817	0.005	4.72
77	7,000	0.513	824	797	830	0.005	5.00
84	7,000	0.531	827	824	844	0.005	5.02
89	7,000	0.542	834	863	866	0.005	5.02
89	8,000	0.569	886	911	923	0.006	5.13

(Continued)

* Beam 7L was conventionally reinforced, had an additional 2.5 percent by volume of steel fibers added throughout its entire cross section, and was subjected to the long-term static tests outlined for members of group 2L.

** Average tensile strain is average for middle one-third of beam.

† Immediately after loading.

Table 30 (Concluded)

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain: Compressive Millionths	Concrete Strain: Average Tensile Millionths	Maximum Dimension of Crack: Width in.	Maximum Dimension of Crack: Depth in.
98	8,000	0.592	909	972	973	0.006	5.13
105	8,000	0.600	910	1008	1011	0.006	5.13
113	8,000	0.620	909	1010	1011	0.006	5.13
117	8,000	0.632	908	1020	1026	0.006	5.13
117	9,000	0.656	951	1129	1129	0.007	5.25
120	9,000	0.667	986	1130	1133	0.007	5.25
126	9,000	0.684	1004	1141	1163	0.007	5.25
131	9,000	0.693	1020	1144	1170	0.007	5.25
131	10,000	0.729	1078	1168	1249	0.008	5.94
138	10,000	0.764	1080	1238	1340	0.008	5.94
145	10,000	0.775	1084	1323	1366	0.008	5.94
145	11,000	0.808	1157	1353	1474	0.009	6.00
152	11,000	0.837	1241	1377	1503	0.009	6.00
159	11,000	0.849	1242	1378	1541	0.009	6.00
159	12,000	0.886	1308	1562	1644	0.010	6.00
166	12,000	0.915	1351	1612	1693	0.010	6.00
173	12,000	0.937	1372	1648	1740	0.010	6.00
173	13,000	0.965	1412	1702	1819	0.012	6.10
181	13,000	0.992	1457	1758	1879	0.012	6.10
187	13,000	1.004	1461	1778	1896	0.012	6.10
187	14,000	1.044	1567	1848	2001	0.015	6.15
194	14,000	1.071	1572	1888	2050	0.015	6.15
201	14,000	1.085	1574	1923	2087	0.015	6.15
201	15,000	1.121	1634	1988	2179	0.018	6.25
208	15,000	1.156	1693	2043	2251	0.018	6.25
215	15,000	1.176	1718	2082	2284	0.018	6.25
215	16,000	1.219	1793	2103	2394	0.020	6.35
222	16,000	1.271	1862	2246	2497	0.020	6.35
229	16,000	1.286	1863	2282	2521	0.020	6.35
229	17,000	1.453	1975	2355	2693	0.065	9.55
236	17,000	1.75††	2012	2396	2774	0.080	9.60
243	17,000	1.80††	2112	2450	2860	0.080	9.60
243	17,500	2.65††	--	--	--	0.125	14.00
243	15,500†‡	3.75††	--	--	--	0.250	14.00
243	7,200†‡	3.50††	--	--	--	--	--
243	0†‡	3.00††	--	--	--	0.225	14.00

†† String-line measurements.
‡ Load after failure.

Table 31

Summary of Test Results for Beam 8L*

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain Compressive Millionths	Concrete Strain Average** Tensile Millionths	Maximum Dimension of Crack Width in.	Maximum Dimension of Crack Depth in.
0	0	0	0	0	0	0	0
†	1,500	0.041	9	9	51	0	0
3	1,500	0.050	23	10	50	0	0
7	1,500	0.056	25	10	50	0	0
12	1,500	0.060	43	9	50	0	0
18	1,500	0.070	43	10	50	0	0
18	2,000	0.087	50	28	61	0.001	1.00
24	2,000	0.099	70	29	76	0.001	1.00
31	2,000	0.113	80	28	81	0.001	1.08
32	2,000	0.118	81	29	84	0.001	1.08
32	3,000	0.161	145	89	154	0.002	2.38
35	3,000	0.173	190	118	204	0.004	3.25
46	3,000	0.207	192	196	216	0.004	3.25
46	4,000	0.243	282	246	297	0.004	3.70
54	4,000	0.278	331	322	341	0.004	3.70
61	4,000	0.297	367	350	383	0.004	3.70
61	5,000	0.327	387	408	447	0.005	4.50
68	5,000	0.364	458	430	510	0.005	4.50
75	5,000	0.382	485	485	536	0.005	4.50
75	6,000	0.418	529	542	627	0.007	5.55
82	6,000	0.441	530	626	687	0.007	5.55
89	6,000	0.457	559	697	707	0.007	5.55
89	7,000	0.498	588	774	823	0.009	5.85
96	7,000	0.540	645	838	887	0.010	6.15
103	7,000	0.550	675	845	894	0.010	6.15
103	8,000	0.591	703	918	1006	0.013	6.75
110	8,000	0.613	745	942	1039	0.013	6.75
117	8,000	0.634	781	967	1061	0.013	6.75
124	8,000	0.634	797	1025	1087	0.013	6.75
131	8,000	0.645	826	1068	1094	0.013	6.75

(Continued)

* Beam 8 was conventionally reinforced and used as control beam for the long-term static tests of group 2L.

** Average tensile strain is average for middle one-third of beam.

† Immediately after loading.

Table 31 (Concluded)

Time days	Total Load lb	Midspan Deflection in.	Reinforcement Strain Millionths	Concrete Strain Compressive Millionths	Concrete Strain Average** Tensile Millionths	Maximum Dimension of Crack Width in.	Maximum Dimension of Crack Depth in.
133	8,000	0.649	830	1069	1100	0.013	6.75
133	9,000	0.687	856	1132	1187	0.015	7.10
140	9,000	0.707	896	1140	1231	0.015	7.10
147	9,000	0.715	919	1156	1249	0.015	7.10
147	10,000	0.763	948	1226	1369	0.018	7.20
154	10,000	0.794	949	1255	1423	0.018	7.20
161	10,000	0.813	998	1310	1461	0.018	7.20
161	11,000	0.852	1022	1377	1571	0.020	7.25
168	11,000	0.873	1076	1386	1601	0.020	7.25
175	11,000	0.878	1123	1400	1606	0.020	7.25
175	12,000	0.928	1227	1470	1736	0.022	7.25
182	12,000	0.947	1300	1517	1763	0.022	7.25
189	12,000	0.949	1332	1584	1770	0.022	7.25
189	13,000	1.000	1416	1589	1896	0.025	7.35
196	13,000	1.018	1496	1611	1907	0.025	7.35
203	13,000	1.033	1496	1679	1944	0.025	7.35
203	14,000	1.083	1629	1720	2075	0.038	7.95
210	14,000	1.113	1713	1764	2126	0.038	7.95
217	14,000	1.123	1728	1774	2146	0.038	7.95
220	14,000	1.128	1761	1780	2150	0.038	7.95
220	15,000	1.181	1861	1864	2287	0.041	8.00
227	15,000	1.218	1865	1909	2330	0.041	8.00
234	15,000	1.228	1971	1918	2353	0.041	8.00
234	15,900	2.55††	--	--	--	0.375	14.00
234	12,500‡	2.50††	--	--	--	--	--
234	5,800‡	2.25††	--	--	--	--	--
234	0‡	1.95††	--	--	--	--	--

†† String-line measurements.
‡ Load after failure.

Table 32

Maximum Concrete Tensile Stresses Within Corresponding Beams at Initial Cracking Loads

Beam No.	Type of Crack-Arrest Technique	Overall Cross-Sectional Dimensions (b × h) of Beam in.	Type of Loading	Load at Which Cracking Was Initiated, lb	Initial Cracking Moment* in.-lb	Maximum Tensile Stress of Concrete at Initial Cracking,** psi	Tensile Strength of Concrete According to Splitting Tensile Test, psi
1	Conventional reinforcement	4 × 9	Short-term static	2000	25,139	370	Not recorded
1L	Conventional reinforcement	5 × 14	Short-term static	2000	73,234	360	375
8L	Conventional reinforcement	5 × 14	Long-term static	2000	73,234	360	325
14	Conventional reinforcement plus 0.5-in.-long steel fibers in lower tensile zone	4 × 9	Short-term static	6000	73,139	1080	520
2L	Conventional reinforcement plus 0.5-in.-long steel fibers in lower tensile zone	5 × 14	Short-term static	5000	163,234	800	675
6L	Conventional reinforcement plus 0.5-in.-long steel fibers in lower tensile zone	5 × 14	Long-term static	4000	133,234	650	560
2	Conventional reinforcement plus 0.5-in.-long steel fibers throughout entire cross section	4 × 9	Short-term static	5500	67,139	990	490

(Continued)

* Moment includes weight of beams.
** Appendix B illustrates method used for calculating the tensile stresses at cracking.

Table 32 (Concluded)

Beam No.	Type of Crack-Arrest Technique	Overall Cross-Sectional Dimensions (b × h) of Beam in.	Type of Loading	Load at Which Cracking Was Initiated, lb	Initial Cracking Moment* in.-lb	Maximum Tensile Stress of Concrete at Initial Cracking,** psi	Tensile Strength of Concrete According to Splitting Tensile Test, psi
4L	Conventional reinforcement plus 0.5-in.-long steel fibers throughout entire cross section	5 × 14	Short-term static	4000	133,234	650	620
7L	Conventional reinforcement plus 0.5-in.-long steel fibers throughout entire cross section	5 × 14	Long-term static	3000	103,234	510	560
5	Conventional reinforcement plus wire-mesh cage	4 × 9	Short-term static	4000	49,139	730	295†
3L	Conventional reinforcement plus wire-mesh cage	5 × 14	Short-term static	3000	103,234	510	405†
5L	Conventional reinforcement plus wire-mesh cage	5 × 14	Long-term static	2000	73,234	360	360†

† Splitting tensile strength of plain concrete mixture used in fabricating beams.

a. 0 load (prior to testing)

b. 2500-lb load

c. 3000-lb load

d. 3500-lb load

e. 5500-lb load

f. 8000-lb load

g. 9500-lb load

h. 12,500-lb load

i. 4650-lb load (after failure)

j. 0 load (tensile face after failure)

Photo 1. Crack pattern in beam 1 at various load levels

a. 0 load (prior to testing)

b. 6000-lb load

c. 6500-lb load

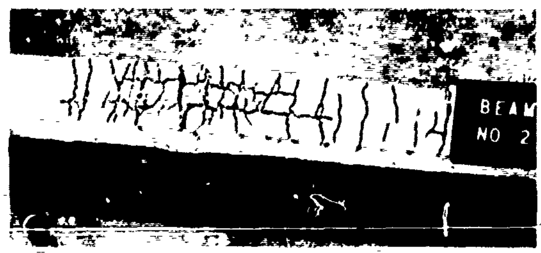
d. 8000-lb load

e. 9000-lb load

f. 10,000-lb load

g. 12,000-lb load

h. 13,700-lb load



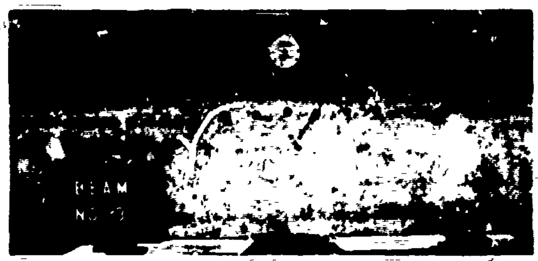
i. 0 load (tensile face
after failure)

Photo 2. Crack pattern in beam 2 at various load levels

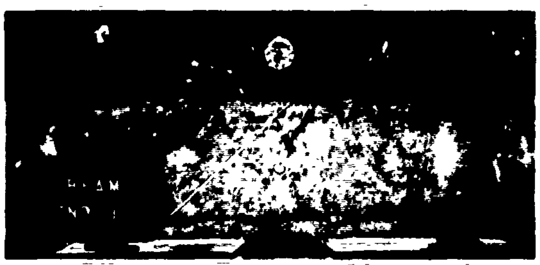
a. 0 load (prior to testing)

b. 4500-lb load

c. 5000-lb load

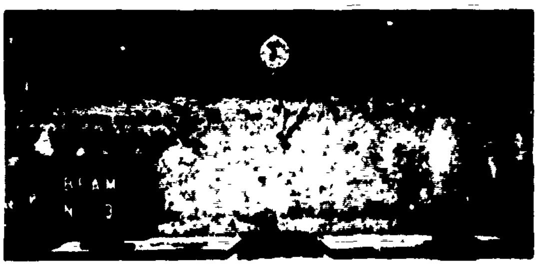


d. 7000-lb load

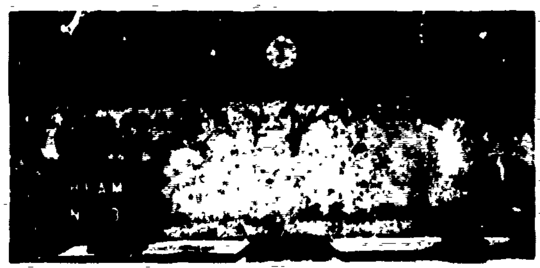


e. 8000-lb load

f. 9000-lb load



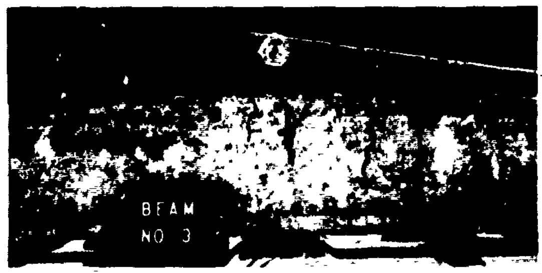
g. 10,000-lb load



h. 12,000-lb load

Photo 3. Crack pattern in beam 3 at various load levels (sheet 1 of 2)

i. 14,000-lb load



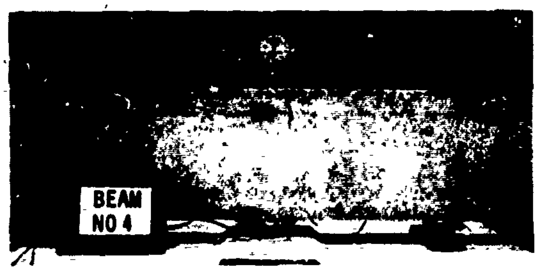
j. 14,000-lb load (yielding of reinforcement)

k. 14,520-lb load (failure)

l. 0 load (tensile face after failure)

Photo 3 (sheet 2 of 2)

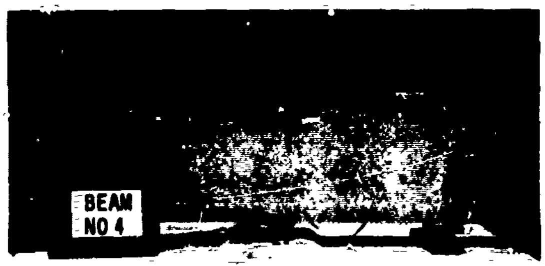
a. 0 load (prior to testing)



b. 2500-lb load

c. 3000-lb load

d. 4000-lb load (first loading)



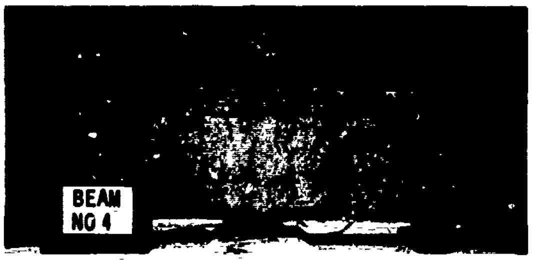
e. 4000-lb load (second loading)

f. 5000-lb load

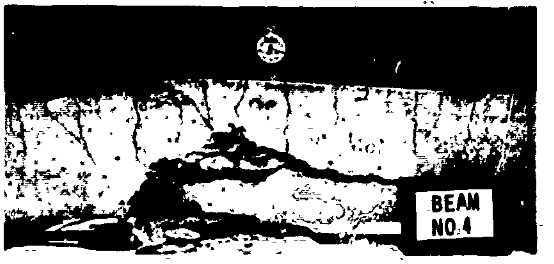
g. 6000-lb load

h. 8000-lb load

Photo 4. Crack pattern in beam 4 at various load levels (sheet 1 of 2)



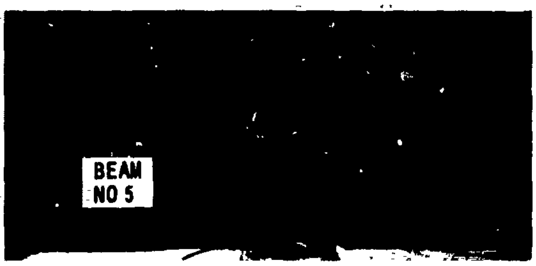
i. 11,000-lb load



j. 0 load (after failure)

k. 0 load (tensile face
after failure)

Photo 4 (sheet 2 of 2)



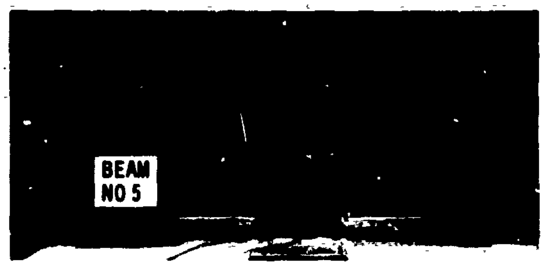
a. 0 load (prior to testing)

b. 4000-lb load

c. 5000-lb load

d. 6000-lb load

e. 8000-lb load



f. 9000-lb load

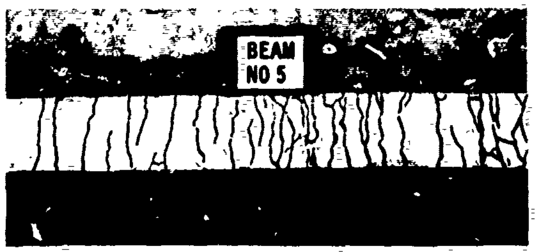
g. 10,000-lb load

h. 13,000-lb load

Photo 5. Crack pattern in beam 5 at various load levels (sheet 1 of 2)

i. 14,000-lb load

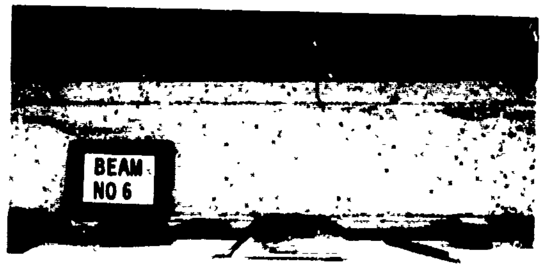
j. 8500-lb load (after failure)



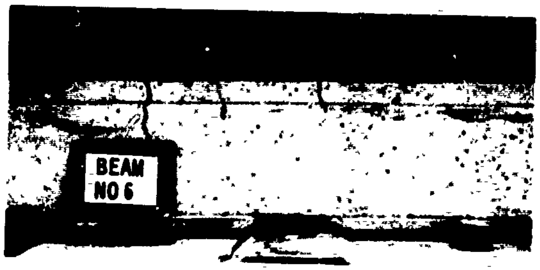
k. 0 load (tensile face
after failure)

Photo 5 (sheet 2 of 2)

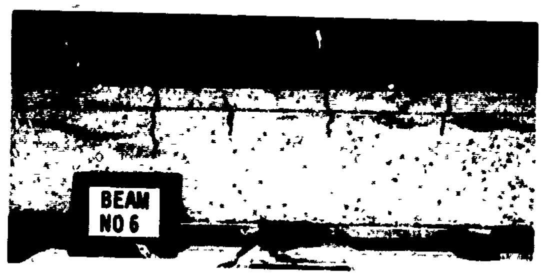
a. 0 load (prior to testing)



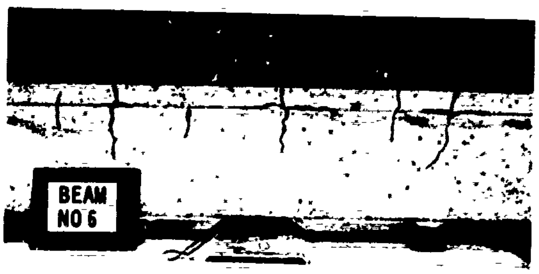
b. 5000-lb load



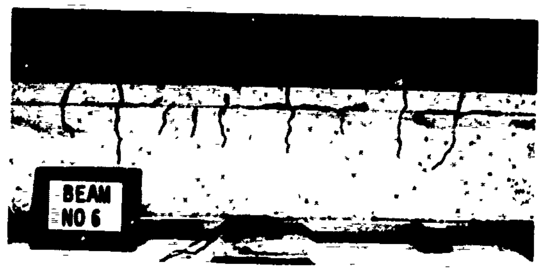
c. 6000-lb load



d. 7000-lb load

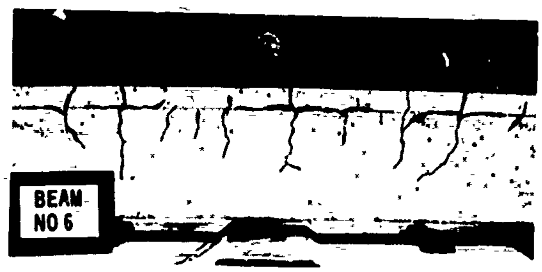


e. 8000-lb load



f. 10,000-lb load

g. 11,000-lb load



h. 12,000-lb load (just prior to failure)

i. 12,000-lb load (failure)

j. 0 load (tensile face after failure)

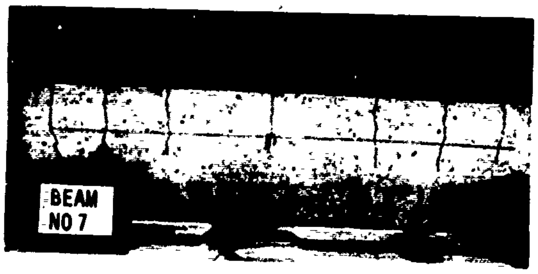
Photo 6. Crack pattern in beam 6 at various load levels

a. 0 load (prior to testing)

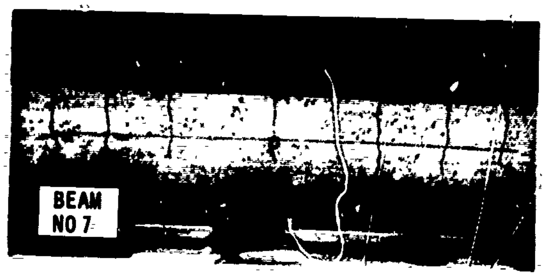
b. 6000-lb load

c. 7000-lb load

d. 8000-lb load



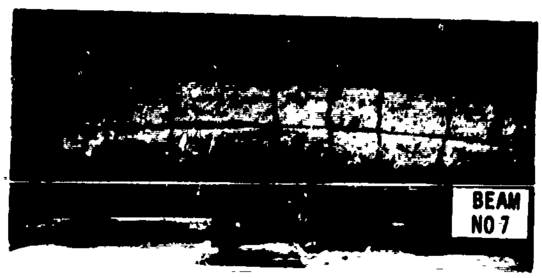
e. 9000-lb load



f. 10,000-lb load

g. 11,000-lb load

h. 12,000-lb load (just prior to failure)



i. 12,000-lb load (failure)

j. 0 load (tensile face after failure)

Photo 7. Crack pattern in beam 7 at various load levels

a. 0 load (prior to testing)

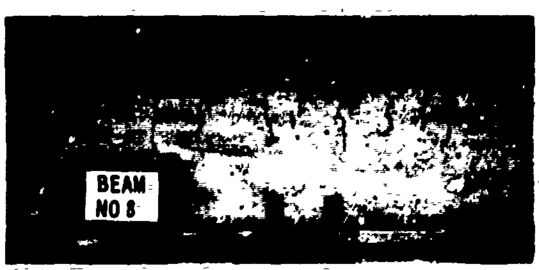
b. 2500-lb load

c. 3000-lb load

d. 3500-lb load

e. 4000-lb load

f. 5000-lb load

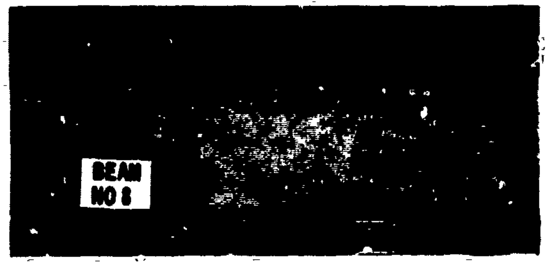


g. 6000-lb load

h. 7000-lb load

Photo 8. Crack pattern in beam 8 at various load levels (sheet 1 of 2)

i. 8000-lb load



j. 9000-lb load

k. 11,000-lb load
(just prior to failure)

l. 11,000-lb load (failure)

m. 0 load (tensile face
after failure)

Photo 8 (sheet 2 of 2)

a. 0 load (prior to testing)

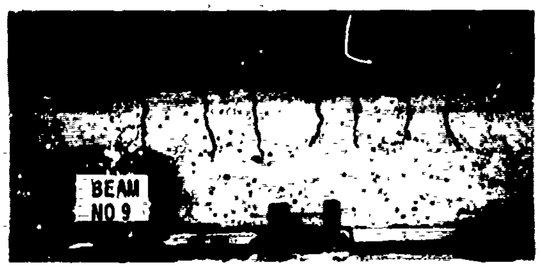
b. 2500-lb load

c. 3000-lb load

d. 3500-lb load

e. 4000-lb load

f. 5000-lb load



g. 6000-lb load

h. 7000-lb load

Photo 9. Crack pattern in beam 9 at various load levels (sheet 1 of 2)

i. 8000-lb load

j. 10,000-lb load

k. 11,000-lb load

l. 12,000-lb load

m. 13,000-lb load (just prior to failure)

n. 13,000-lb load (failure)

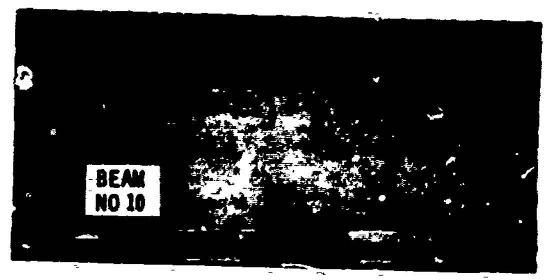
o. 0 load (tensile face after failure)

Photo 9 (sheet 2 of 2)

a. 0 load (prior to testing)

b. 3500-lb load

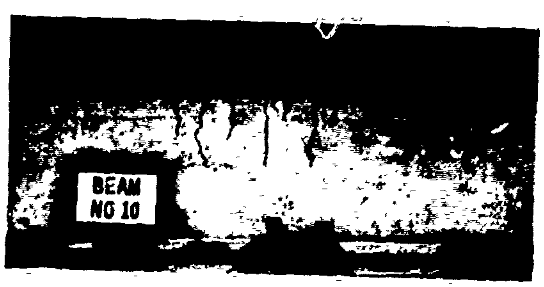
c. 4000-lb load



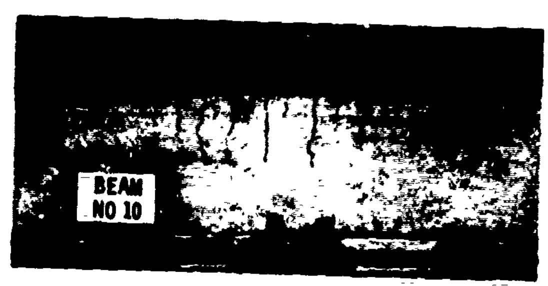
d. 5000-lb load

e. 6000-lb load

f. 7000-lb load

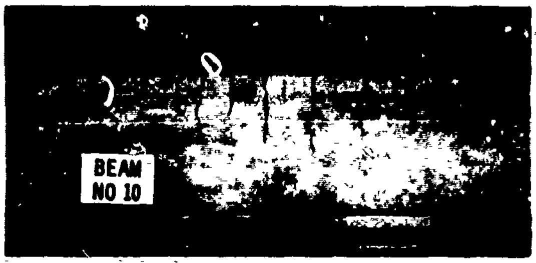


g. 8000-lb load



h. 9000-lb load

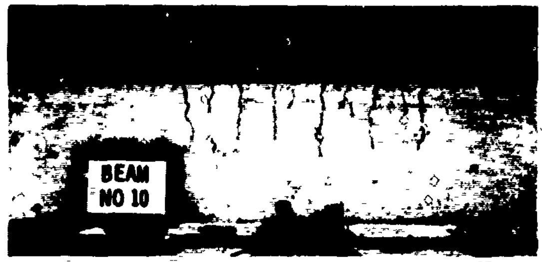
Photo 10. Crack pattern in beam 10 at various load levels (sheet 1 of 2)



i. 10,000-lb load

j. 11,000-lb load

k. 12,000-lb load

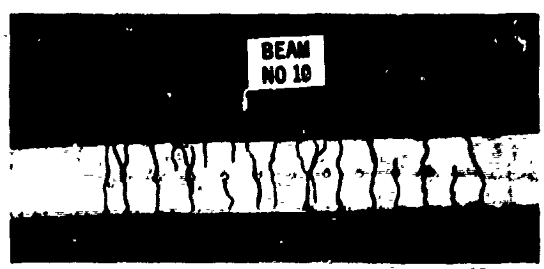


ℓ. 13,000-lb load

m. 14,000-lb load

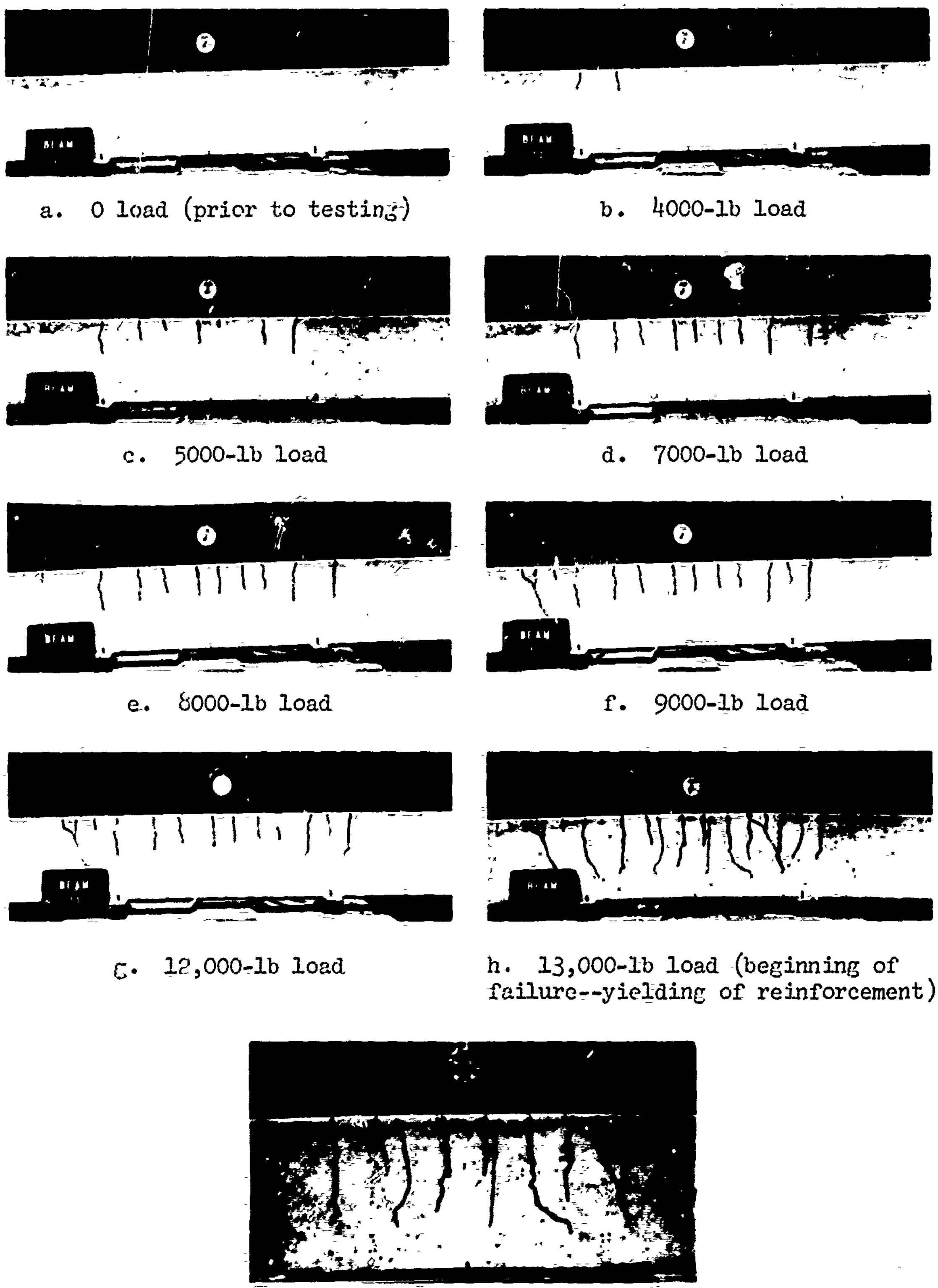
n. 15,000-lb load (failure)

o. 0 load (after failure)



p. 0 load (tensile face after failure)

Photo 10 (sheet 2 of 2)



a. 0 load (prior to testing)

b. 4000-lb load

c. 5000-lb load

d. 7000-lb load

e. 8000-lb load

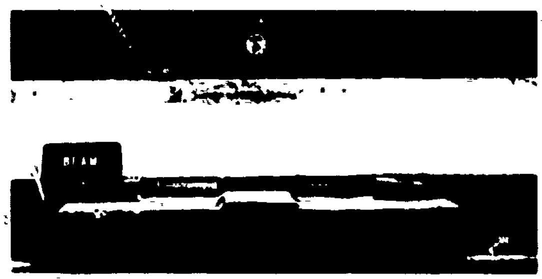
f. 9000-lb load

g. 12,000-lb load

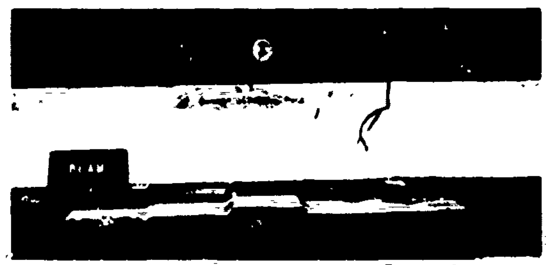
h. 13,000-lb load (beginning of failure--yielding of reinforcement)

i. 0 load (after failure)

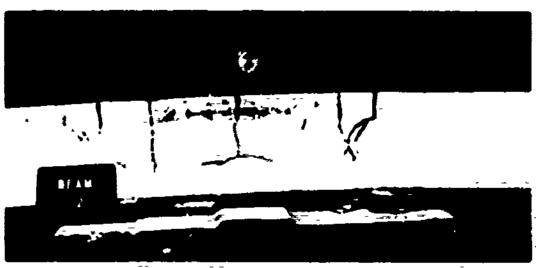
Photo 11. Crack pattern in beam 11 at various load levels



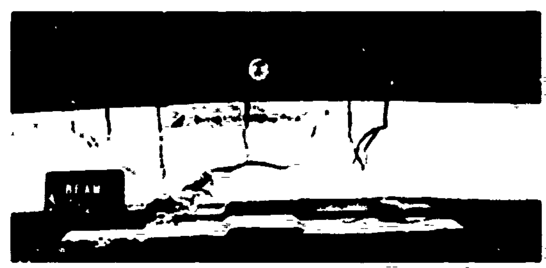
a. 0 load (prior to testing)



b. 12,000-lb load



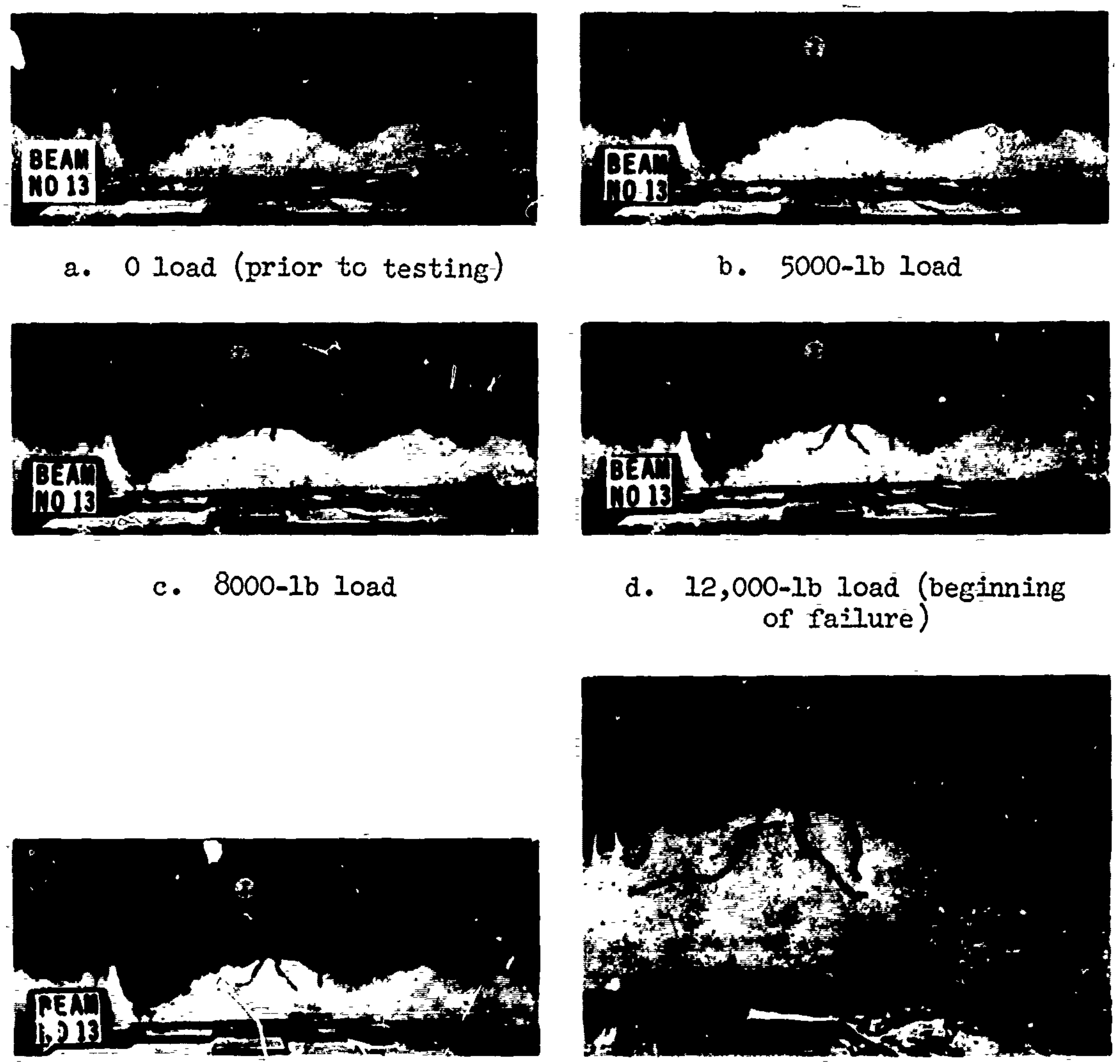
c. 13,000-lb load (beginning of failure)



d. 13,900-lb load (failure)

e. 0 load (after failure)

Photo 12. Crack pattern in beam 12 at various load levels



a. 0 load (prior to testing)

b. 5000-lb load

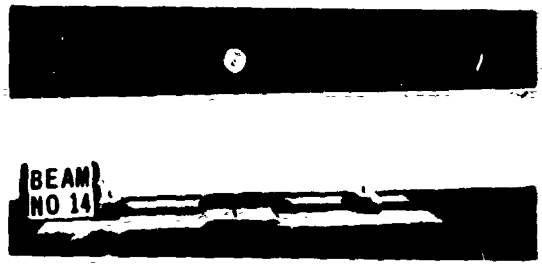
c. 8000-lb load

d. 12,000-lb load (beginning of failure)

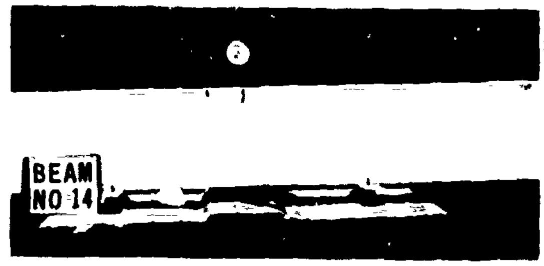
e. 12,700-lb load (failure)

f. Close-up after failure

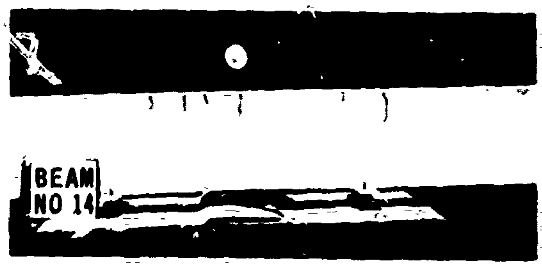
Photo 13. Crack pattern in beam 13 at various load levels



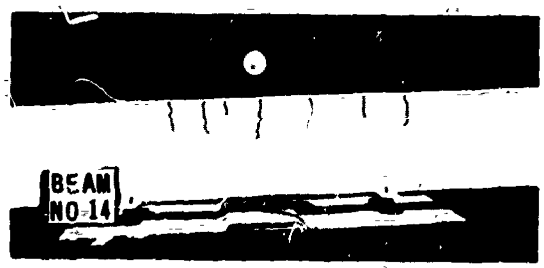
a. 0 load (prior to testing)



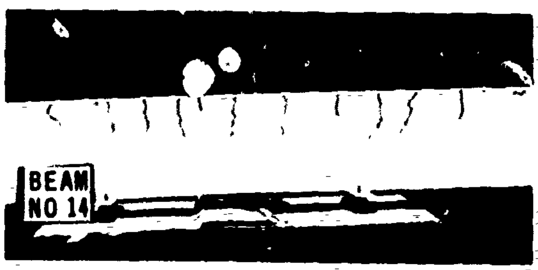
b. 6000-lb load



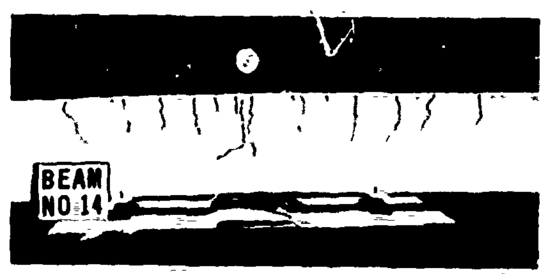
c. 8000-lb load



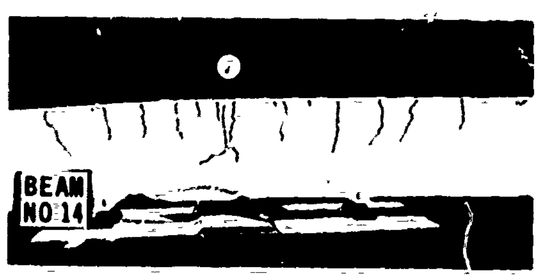
d. 10,000-lb load



e. 12,000-lb load



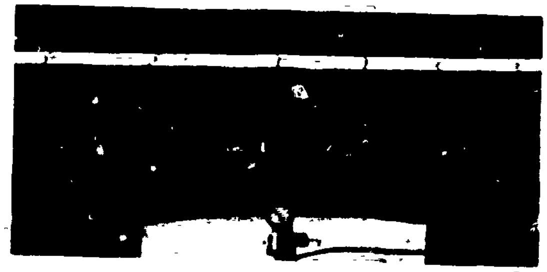
f. 14,000-lb load (beginning of failure)



g. 14,000-lb load (failure)

Photo 14. Crack pattern in beam 14 at various load levels

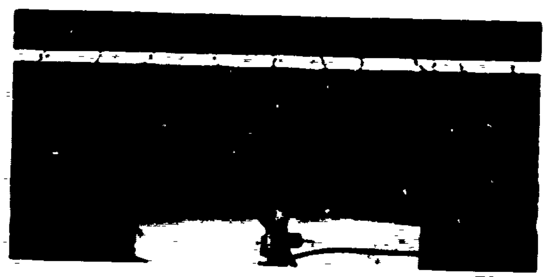
a. 0 load (prior to testing)



b. 2000-lb load

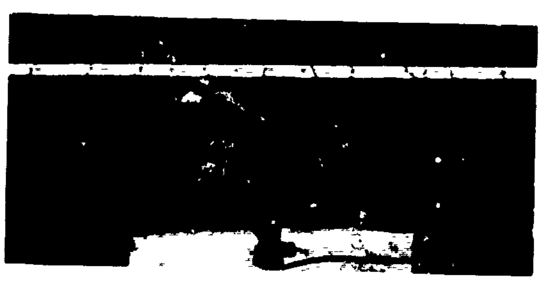
c. 3000-lb load

d. 5000-lb load

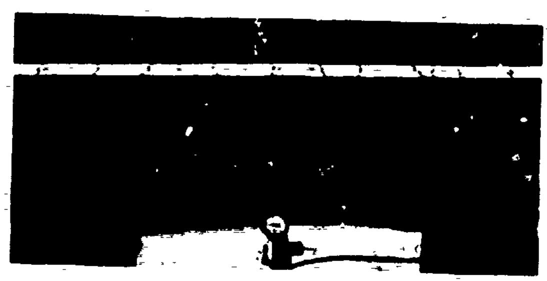


e. 7000-lb load

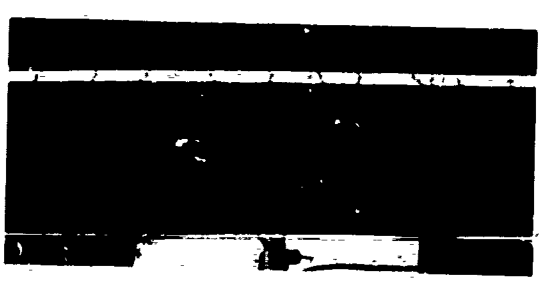
f. 8000-lb load



g. 10,000-lb load



h. 12,000-lb load

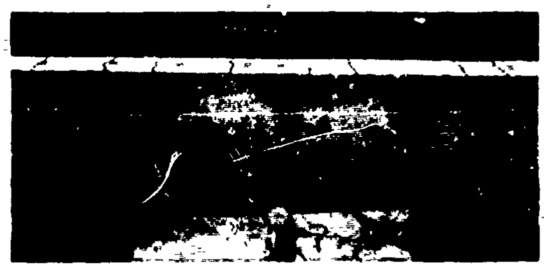


i. 14,000-lb load (yielding of reinforcement)

j. 14,000-lb load (failure)

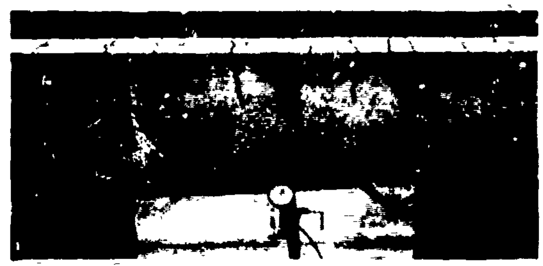
Photo 15. Crack pattern in beam 1L at various load levels

a. 0 load (prior to testing)



b. 5000-lb load

c. 6000-lb load

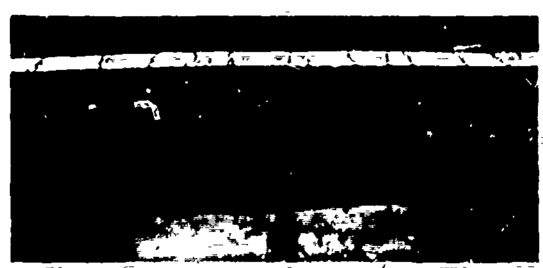


d. 7000-lb load

e. 10,000-lb load

f. 14,000-lb load

g. 16,000-lb load

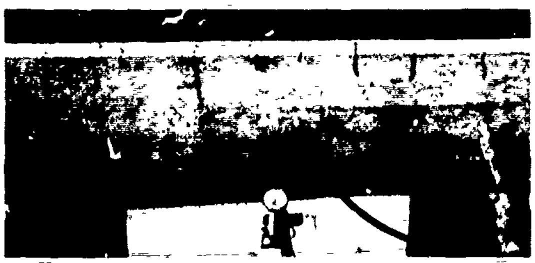


h. 17,000-lb load (failure)

Photo 16. Crack pattern in beam 2L at various load levels

a. 0 load (prior to testing)

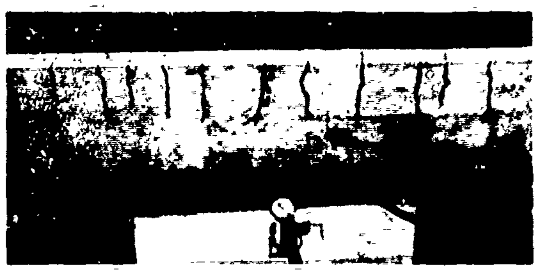
b. 3000-lb load



c. 5000-lb load

d. 8000-lb load

e. 11,000-lb load



f. 13,000-lb load

g. 16,000-lb load

h. 19,300-lb load (failure)

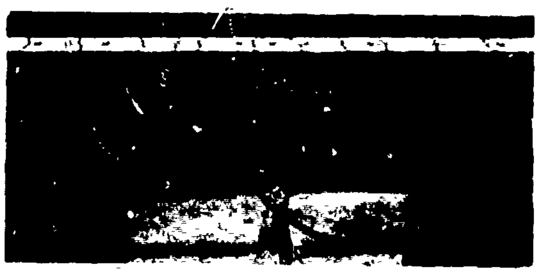
Photo 17. Crack pattern in beam 3L at various load levels

a. 0 load (prior to loading)

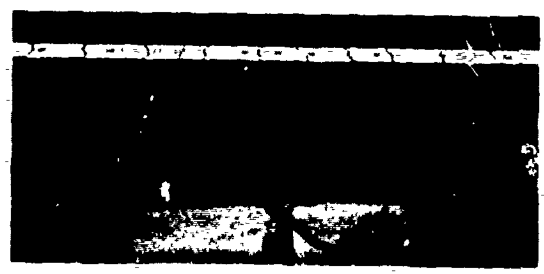
b. 4000-lb load

c. 5000-lb load

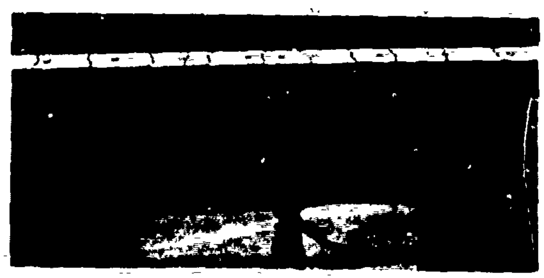
d. 6000-lb load



e. 8000-lb load



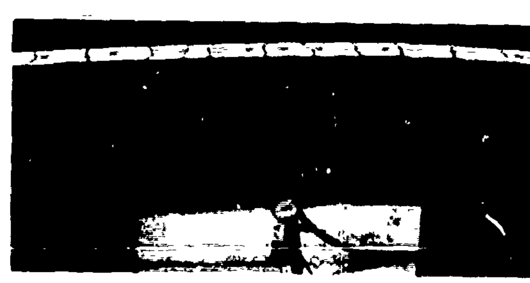
f. 10,000-lb load



g. 11,000-lb load

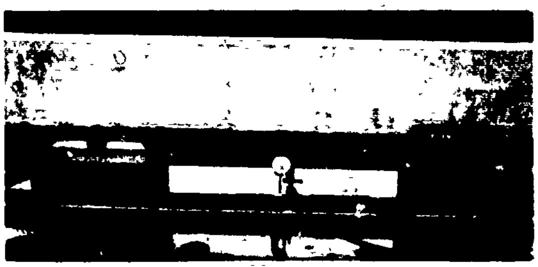
h. 13,000-lb load

i. 16,000-lb load (reinforcement beginning to yield)

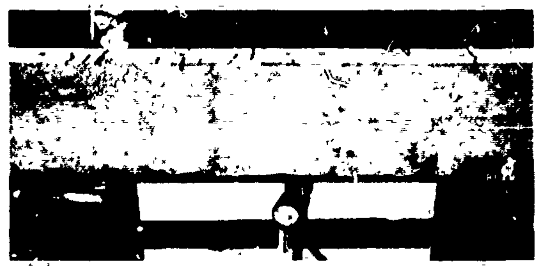


j. 6000-lb load (after failure)

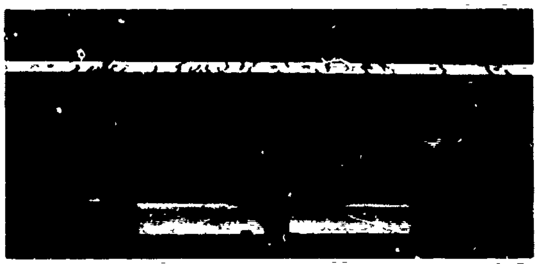
Photo 18. Crack pattern in beam 4L at various load levels



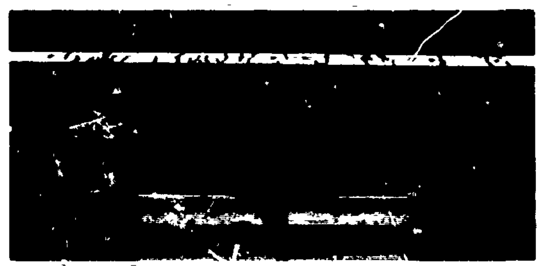
a. 0 load (prior to loading)



b. 2000-lb load for 24 hr



c. 5000-lb load (immediately after loading to this level)

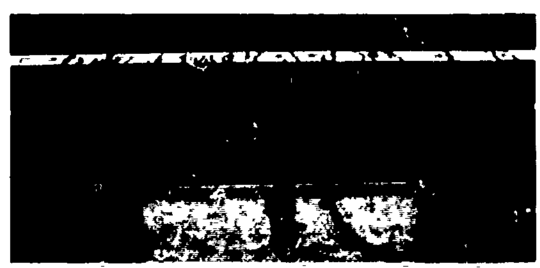


d. 6000-lb load for 14 days

e. 7000-lb load for 21 days

f. 9000-lb load for 16 days

g. 11,000-lb load for 14 days



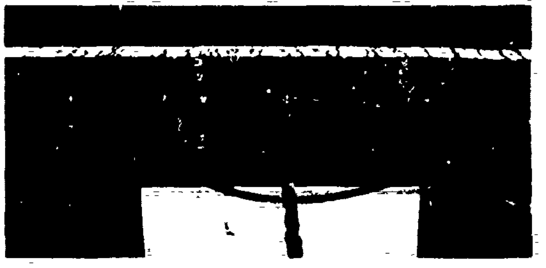
h. 13,000-lb load for 14 days

Photo 19. Crack pattern for beam 5L at various load and time levels (sheet 1 of 2)

i. 16,000-lb load (immediately after loading)

j. 17,000-lb load (immediately after loading)

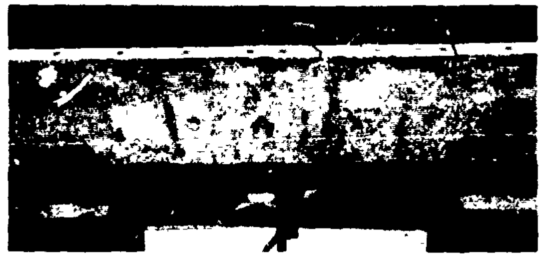
k. 17,500-lb load (just prior to failure)



ℓ. 0 load (after failure)

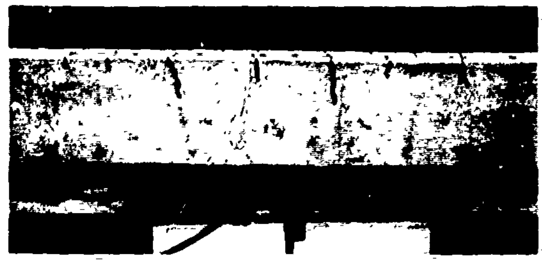
Photo 19 (sheet 2 of 2)

a. 0 load (prior to loading)



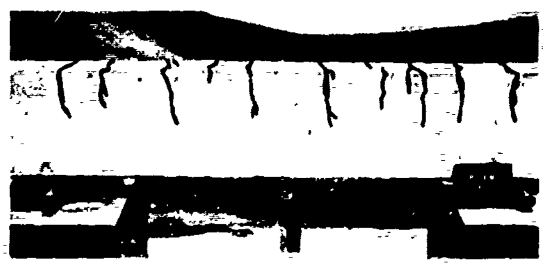
b. 4000-lb load for 24 hr

c. 4000-lb load for 48 hr



d. 6000-lb load (immediately after loading)

e. 7000-lb load for 12 days

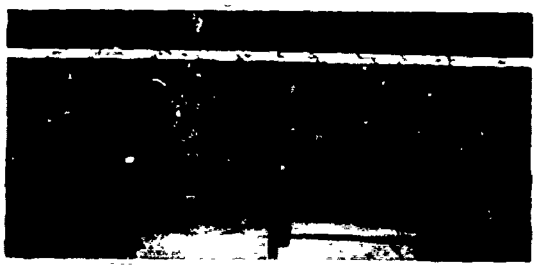


f. 9000-lb load (immediately after loading)

g. 11,000-lb load (immediately after loading)

h. 13,000-lb load (immediately after loading)

Photo 20. Crack pattern for beam 6L at various load and time levels (sheet 1 of 2)



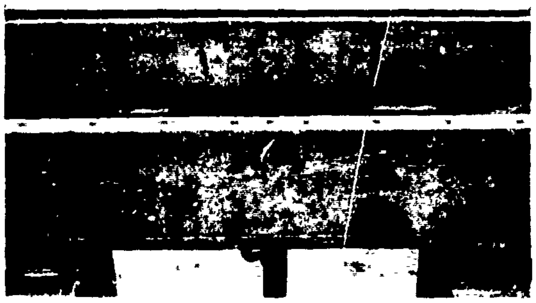
i. 15,000-lb load (immediately after loading)

j. 17,000-lb load (immediately after loading)

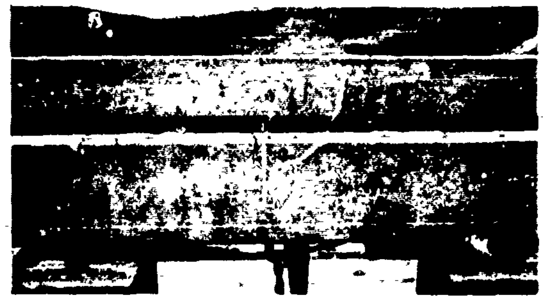
k. 17,400-lb load (failure)

ℓ. 13,500-lb load (after failure)

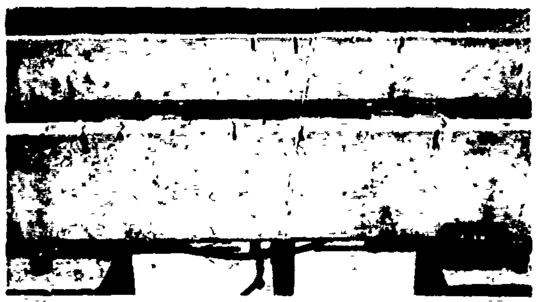
Photo 20 (sheet 2 of 2)



a. 0 load (prior to testing)



b. 3000-lb load for 24 hr

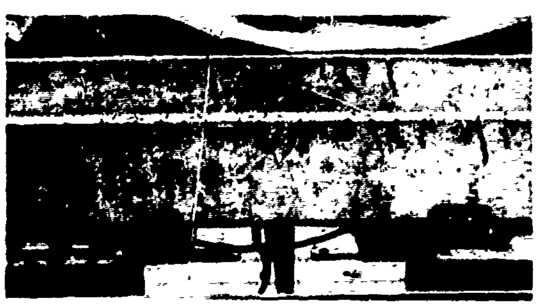


c. 5000-lb load (immediately after loading)

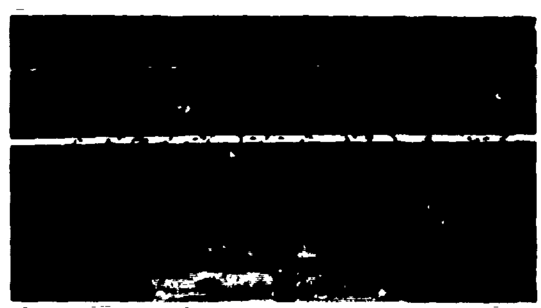
d. 7000-lb load (immediately after loading)

e. 9000-lb load (immediately after loading)

f. 12,000-lb load (immediately after loading)



g. 12,000-lb load for 7 days

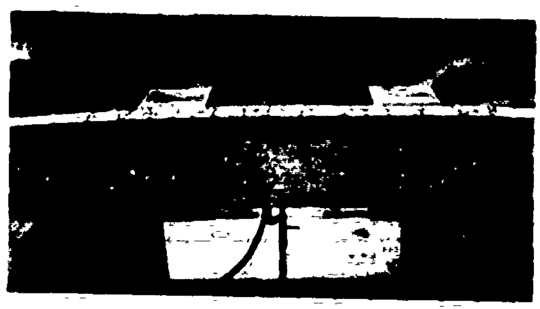


h. 13,000-lb load (immediately after loading)

Photo 21. Crack pattern for beam 7L at various load and time levels (sheet 1 of 2)

i. 15,000-lb load for 14 days

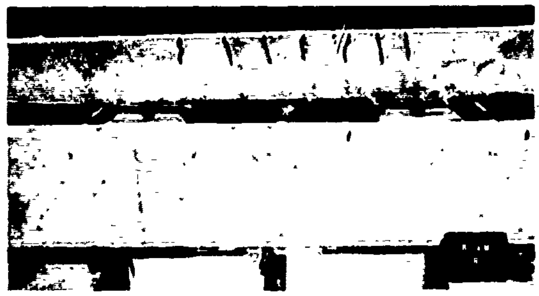
j. 17,000-lb load (yielding of reinforcement is evident)



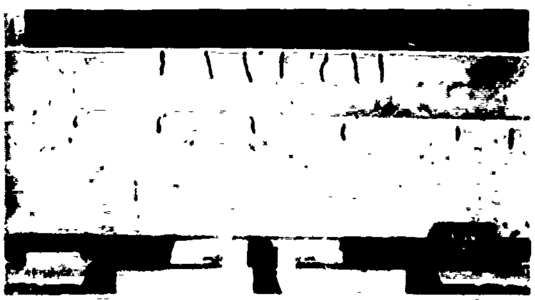
k. 17,500-lb load (failure)

Photo 21 (sheet 2 of 2)

a. 0 load (prior to testing)



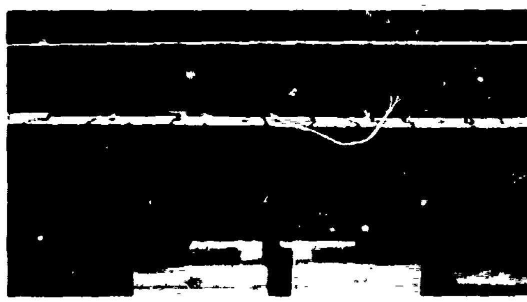
b. 2000-lb load (immediately after loading)



c. 3000-lb load (immediately after loading)

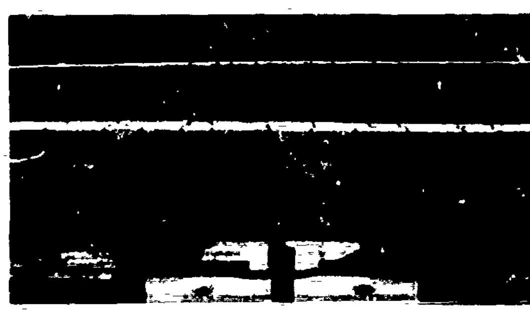
d. 3000-lb load for 3 days

e. 5000-lb load (immediately after loading)



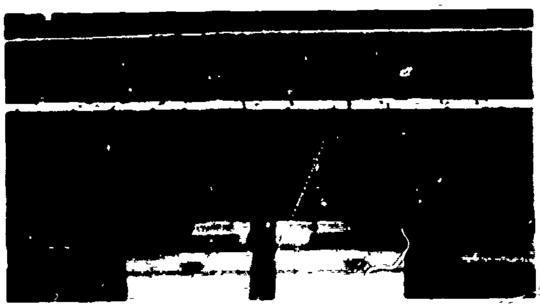
f. 7000-lb load for 7 days

g. 8000-lb load (immediately after loading)

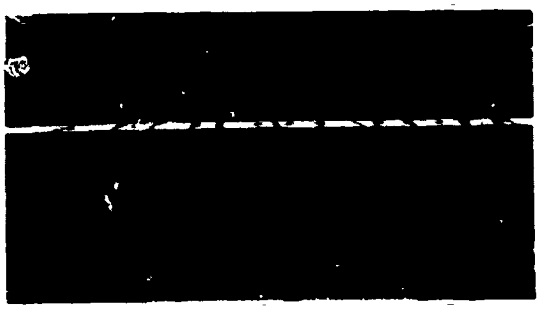


h. 10,000-lb load (immediately after loading)

Photo 22. Crack pattern for beam 8L at various load and time levels (sheet 1 of 2)



i. 12,000-lb load (immediately after loading)

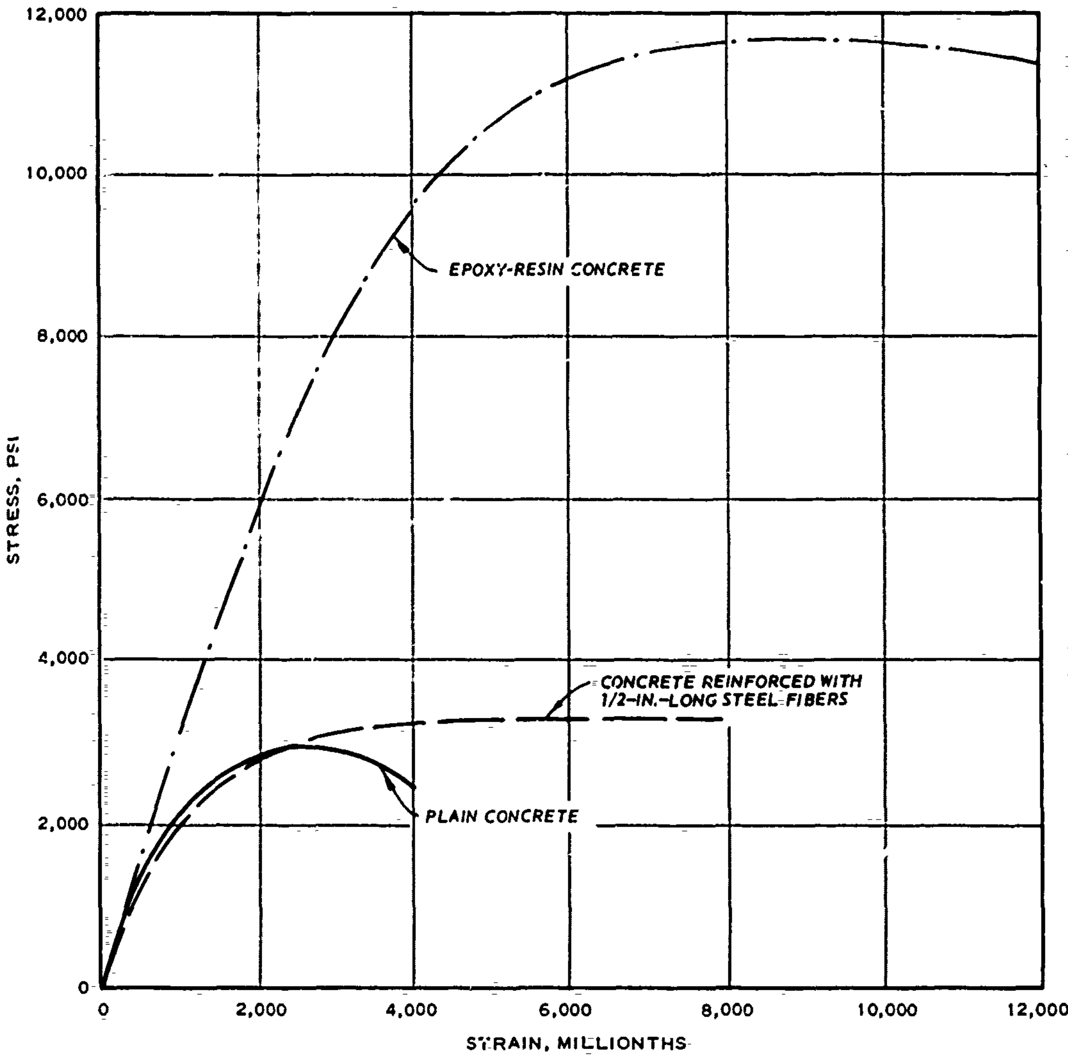


j. 14,000-lb load (immediately after loading)

k. 15,000-lb load (immediately after loading)

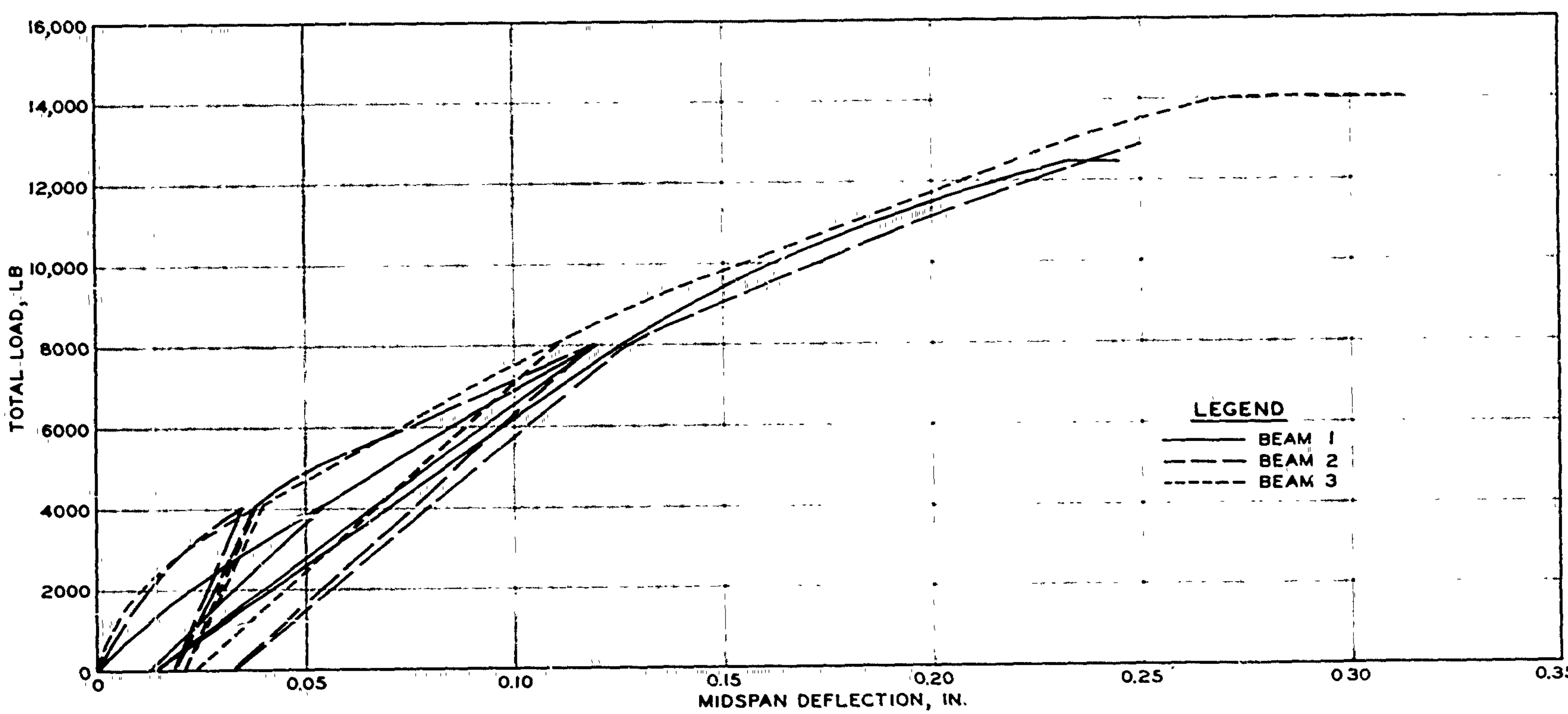
ℓ. 15,900-lb load (failure)

Photo 22 (sheet 2 of 2)



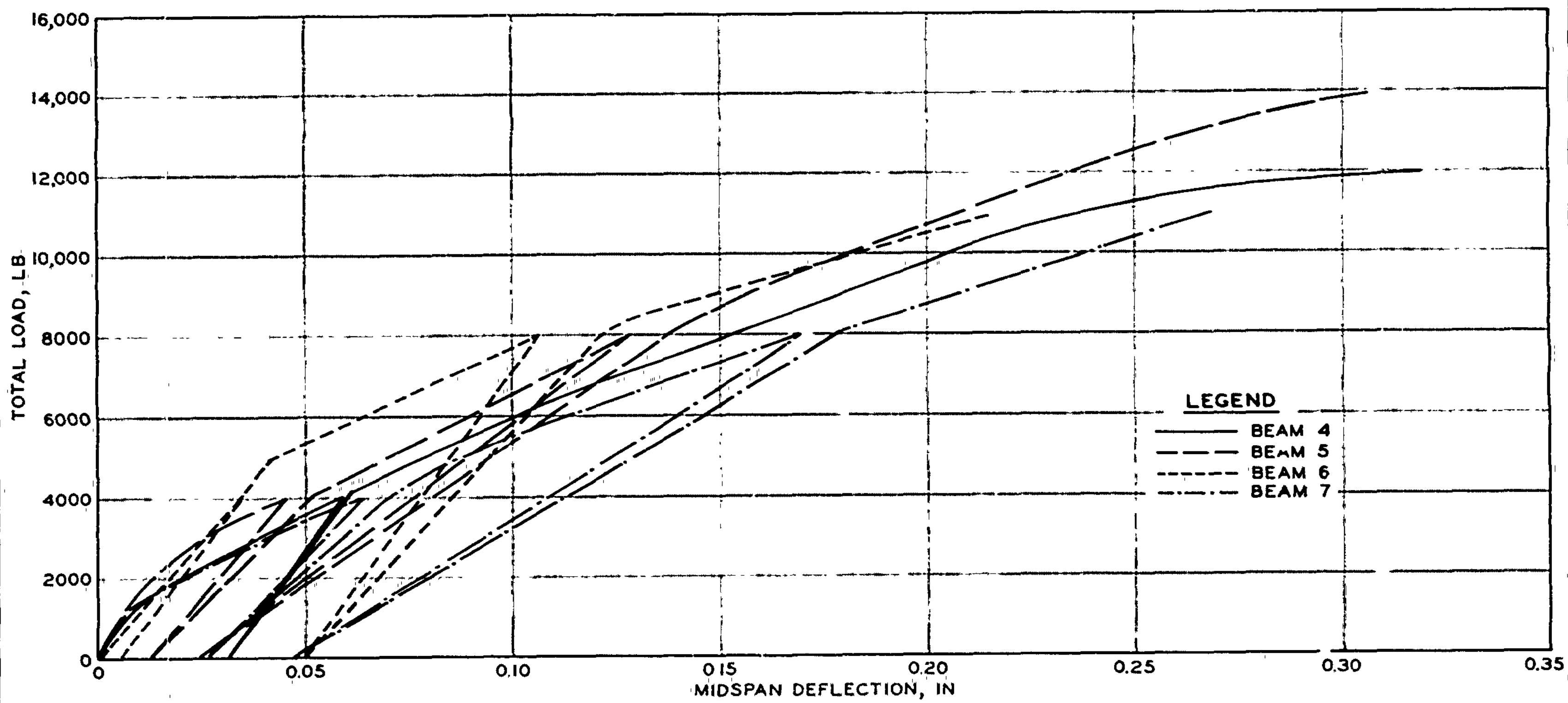
TYPICAL STRESS-STRAIN CURVES FOR PLAIN CONCRETE, CONCRETE REINFORCED WITH 1/2-IN.-LONG STEEL FIBERS, AND EPOXY RESIN CONCRETE USED IN FABRICATING SMALL BEAMS (BEAMS 1-14)

PLATE 1



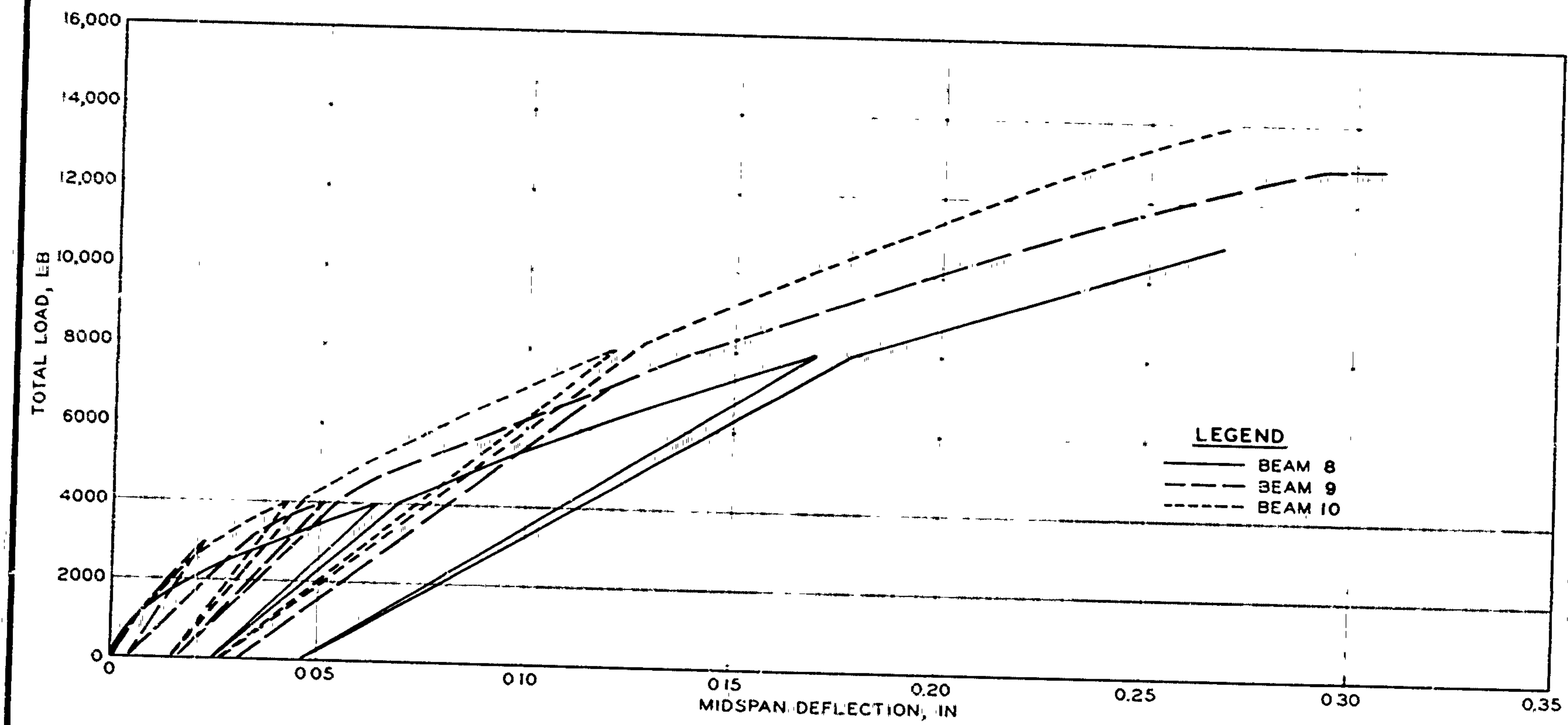
LOAD VERSUS
MIDSPAN DEFLECTION FOR
GROUP 1 BEAMS

PLATE 2

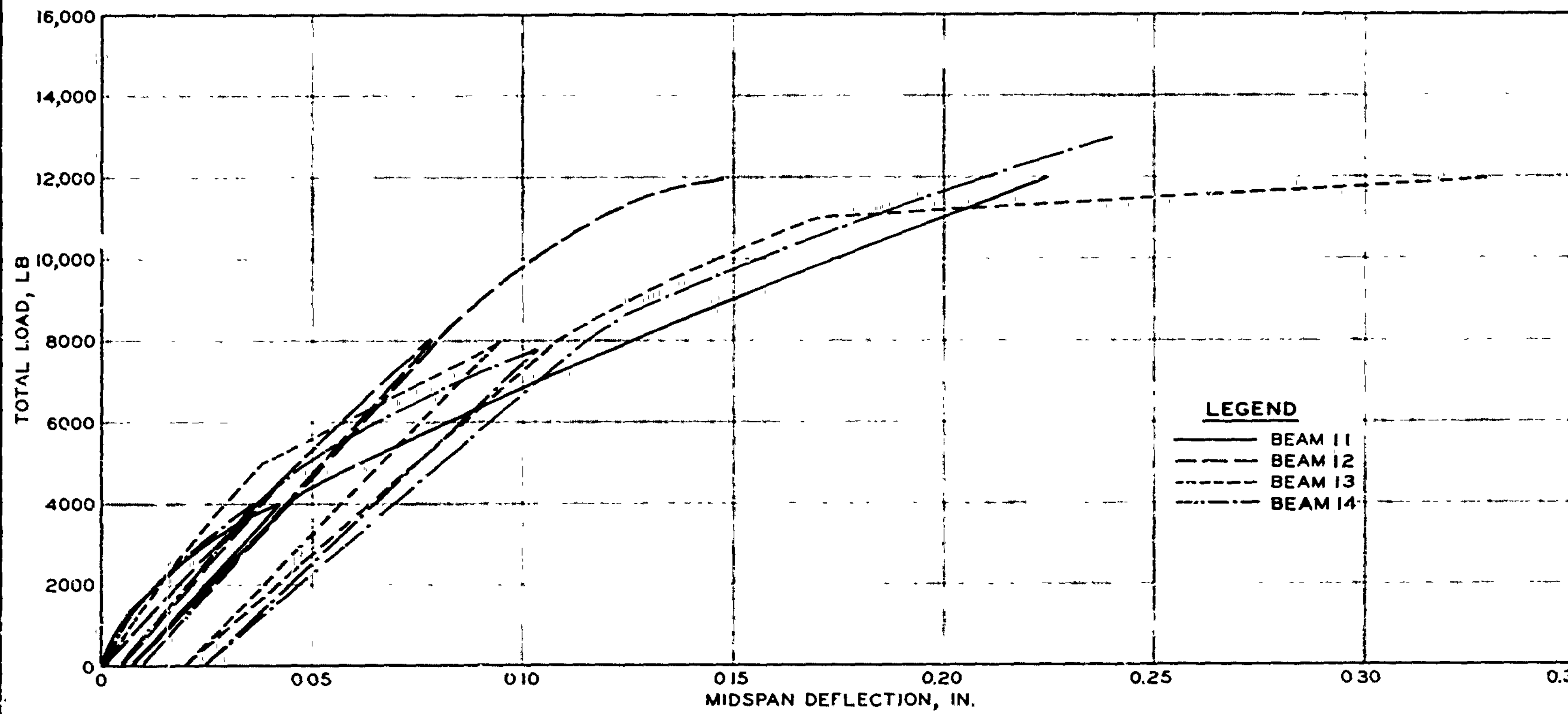


LOAD VERSUS
MIDSPAN DEFLECTION FOR
GROUP 2 BEAMS

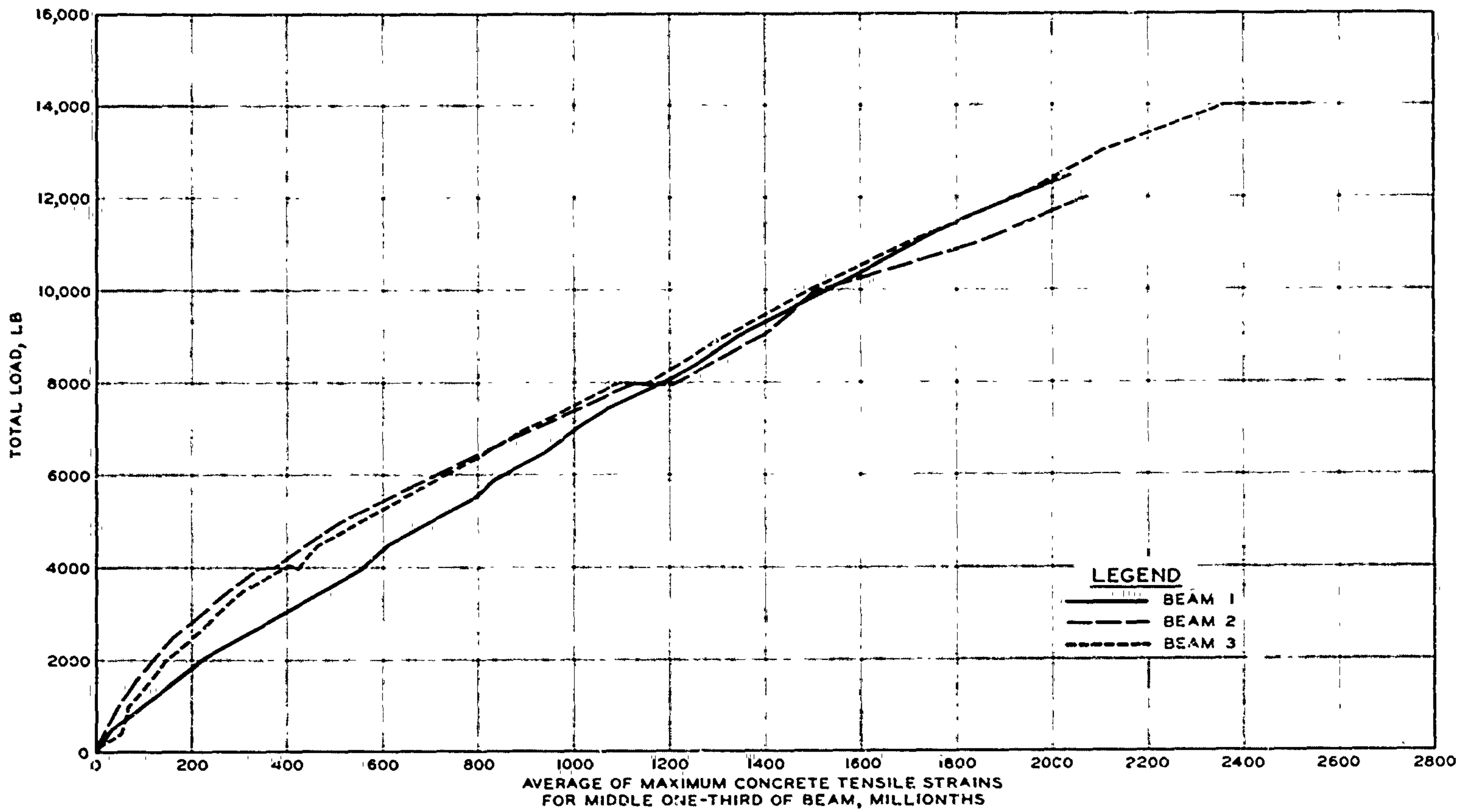
PLATE 3



LOAD VERSUS
MIDSPAN DEFLECTION FOR
GROUP 3 BEAMS

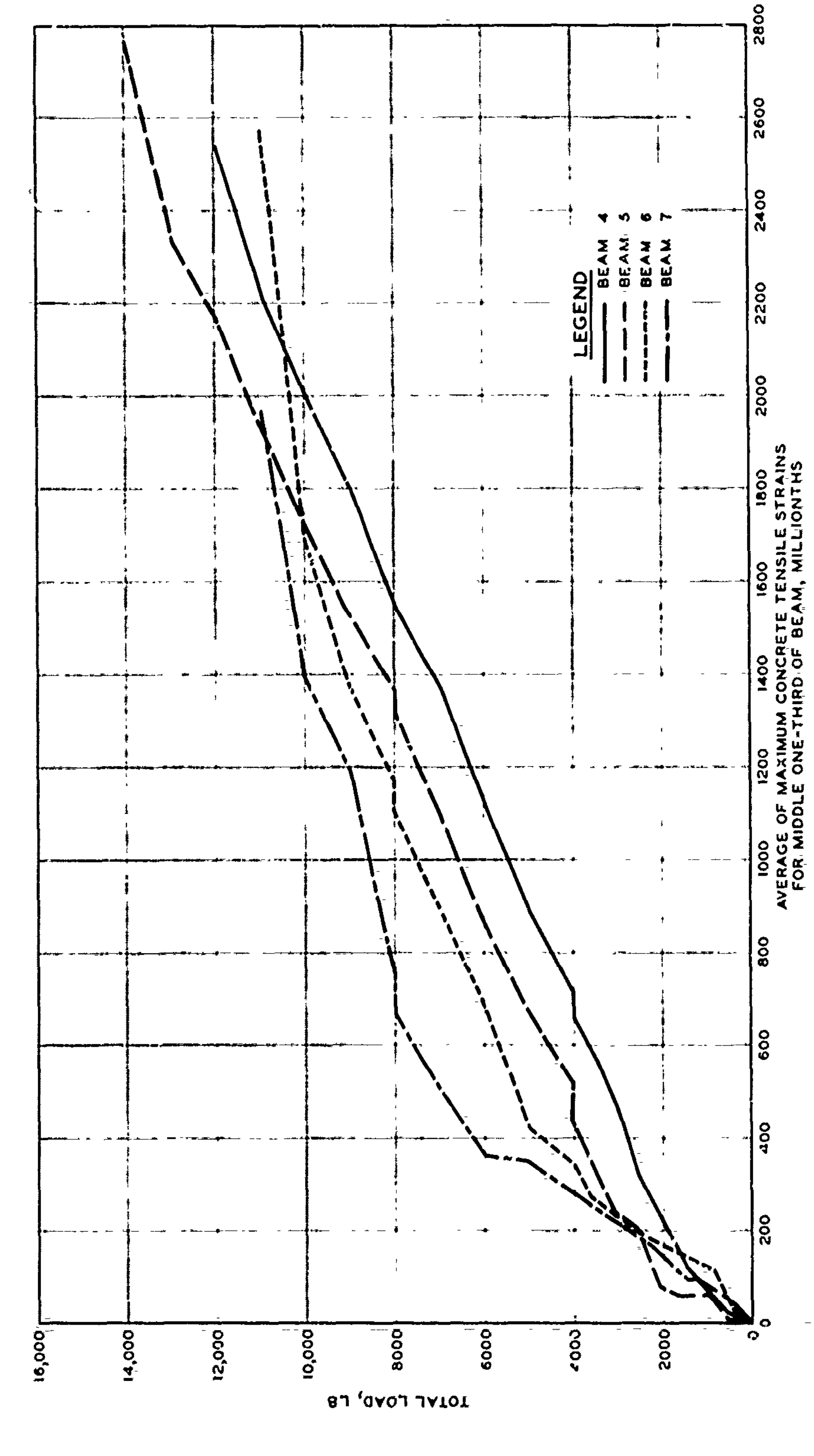


LOAD VERSUS
MIDSPAN DEFLECTION FOR
GROUP 4 BEAMS

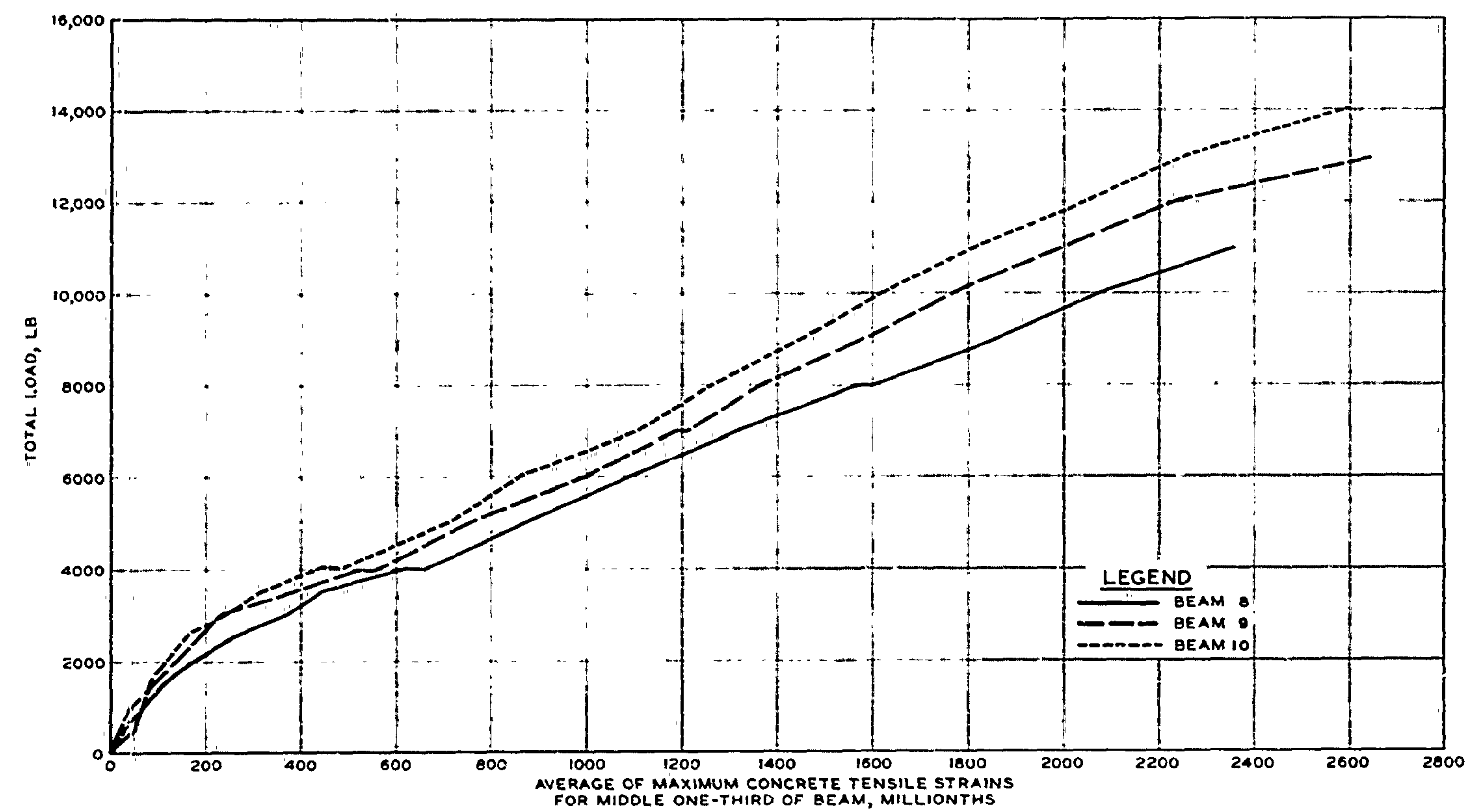


LOAD VERSUS
TENSILE STRAIN FOR
GROUP I BEAMS

PLATE 6

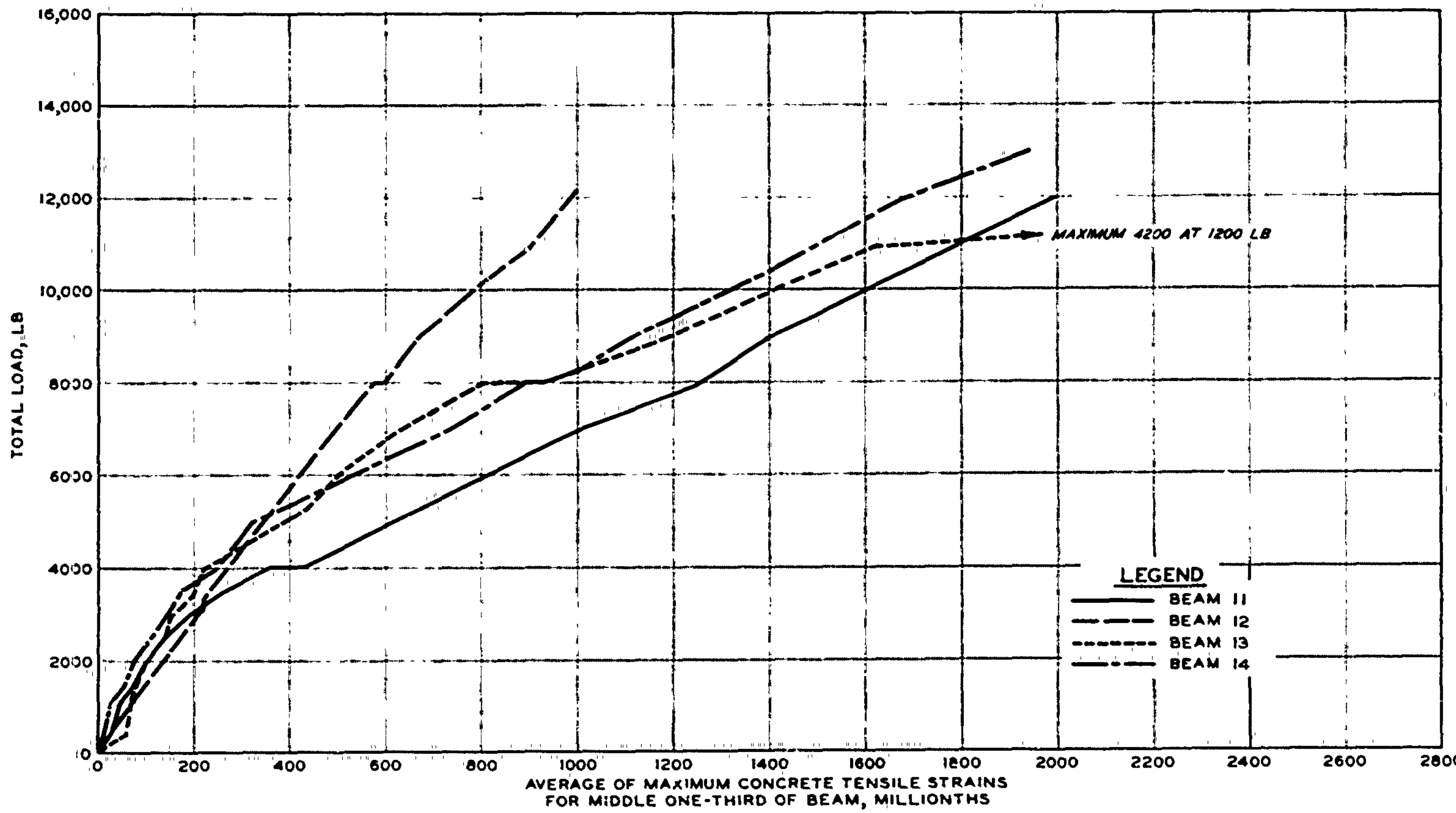


LOAD VERSUS
TENSILE STRAIN FOR
GROUP 2 BEAMS



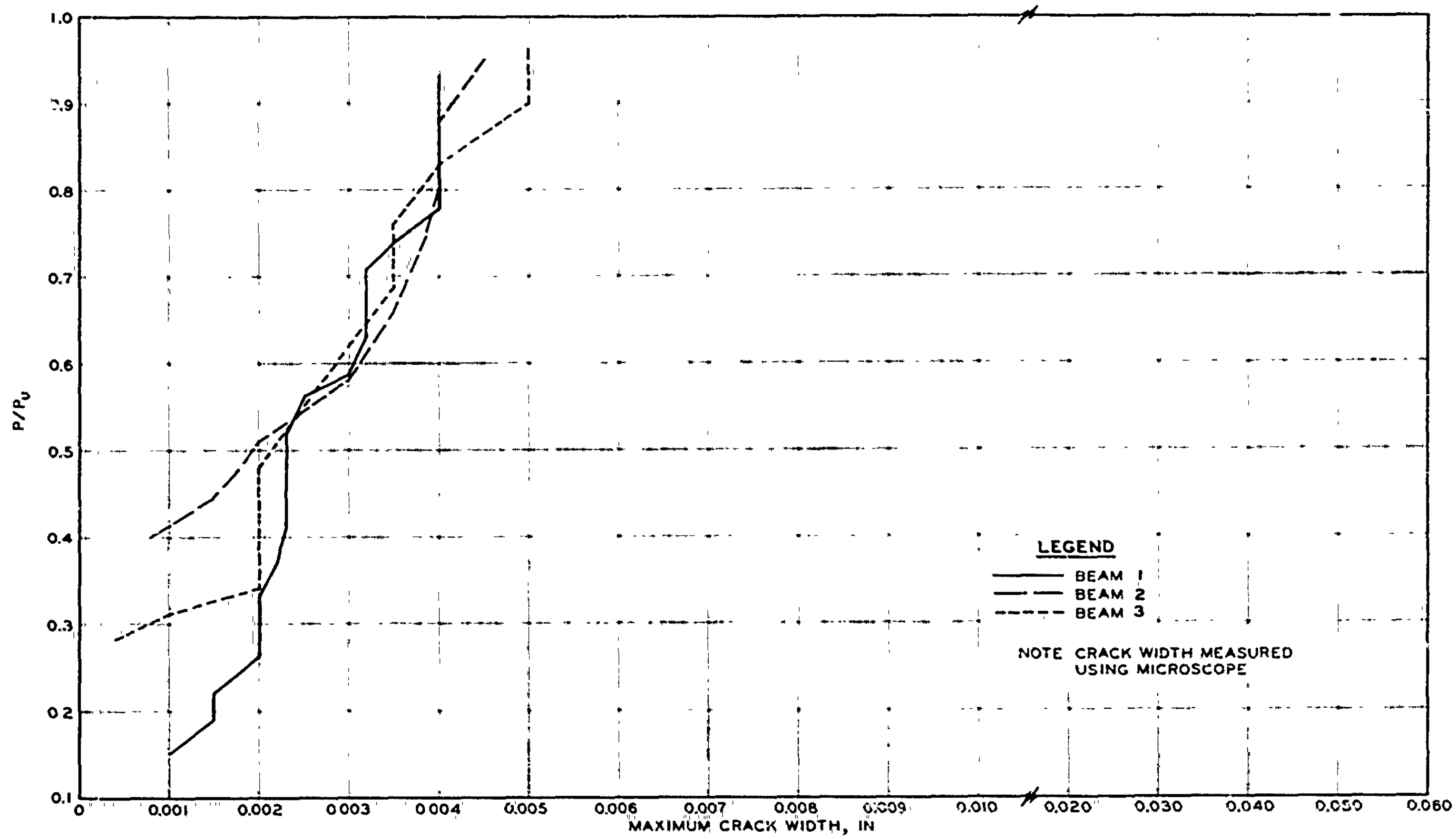
LOAD VERSUS
TENSILE STRAIN FOR
GROUP 3 BEAMS

PLATE 8



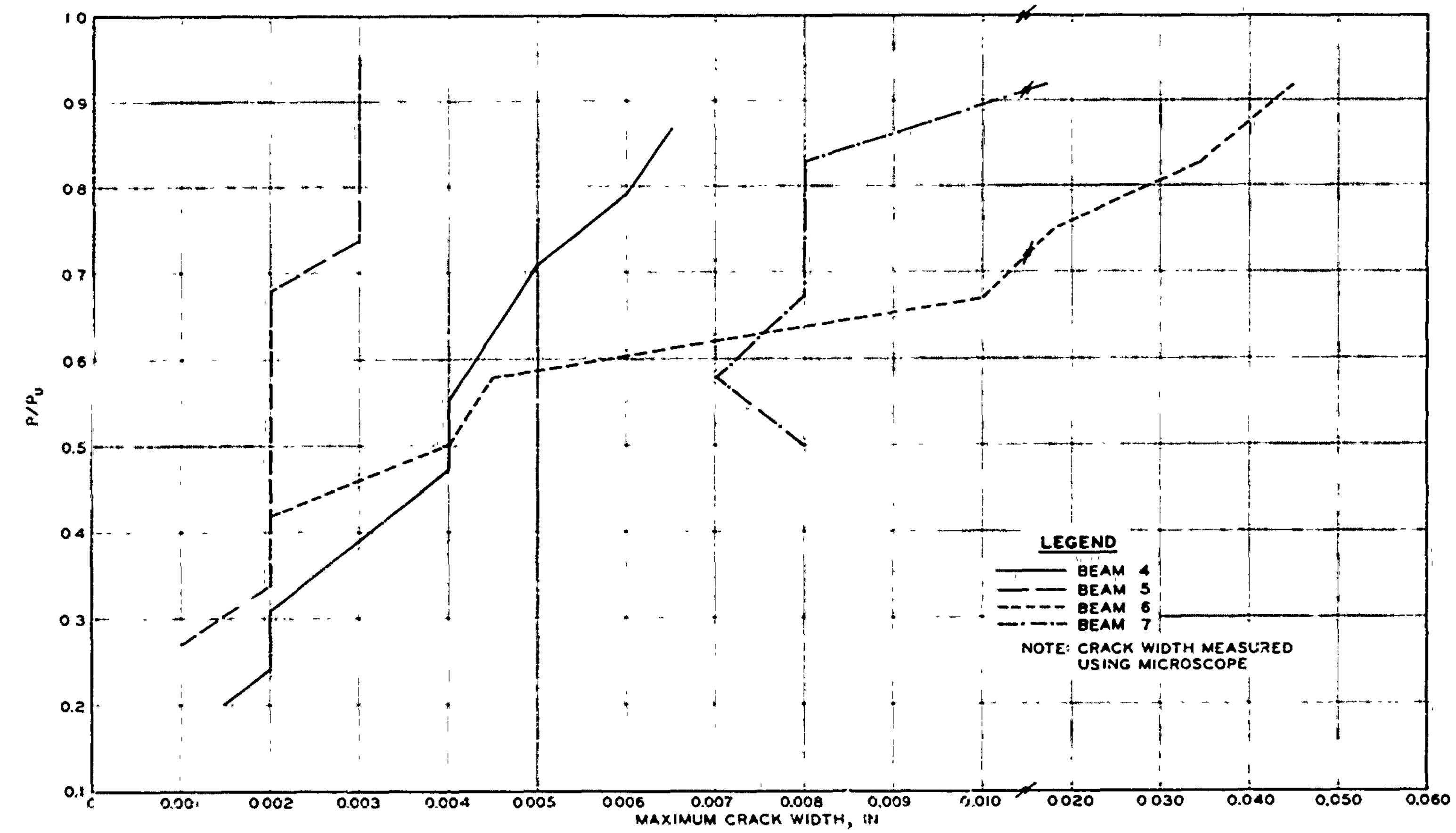
LOAD VERSUS
TENSILE STRAIN FOR
GROUP 4 BEAMS

PLATE 9

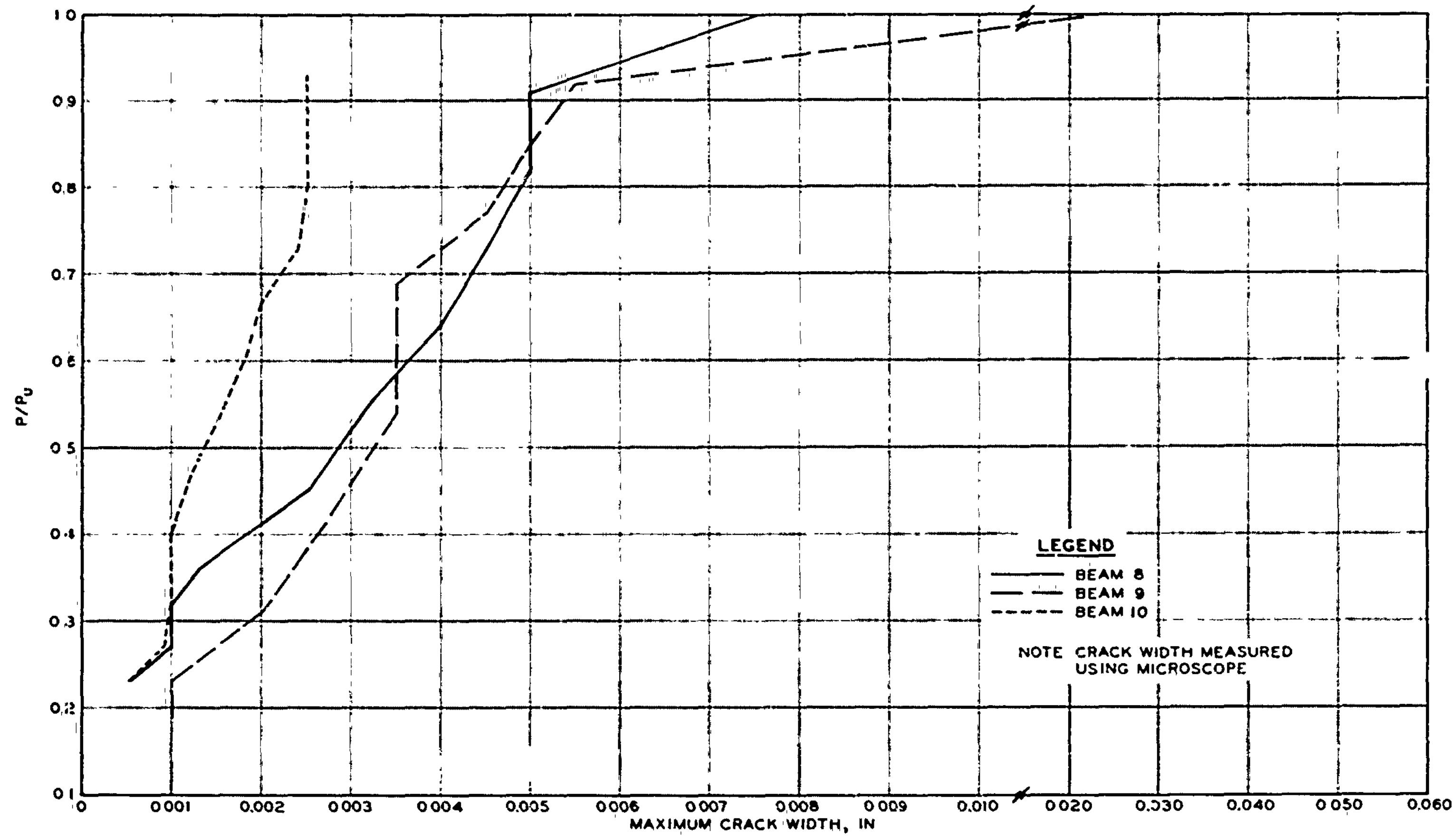


APPLIED LOAD (P)/ULTIMATE (FAILURE) LOAD (P_u) VERSUS MAXIMUM CRACK WIDTH FOR GROUP I BEAMS

PLATE 10

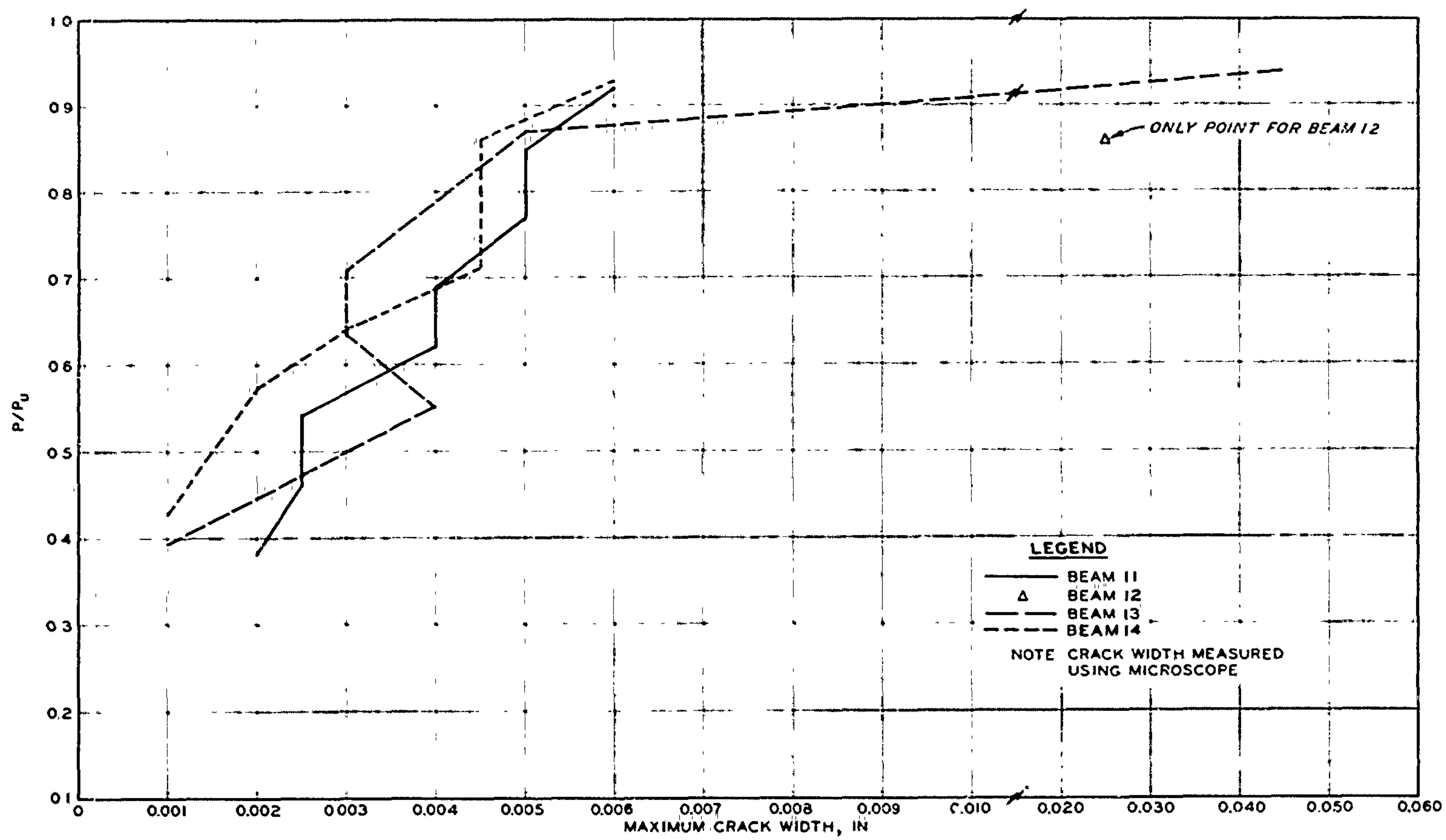


APPLIED LOAD (P)/ULTIMATE (FAILURE) LOAD (P_U) VERSUS MAXIMUM CRACK WIDTH FOR GROUP 2 BEAMS



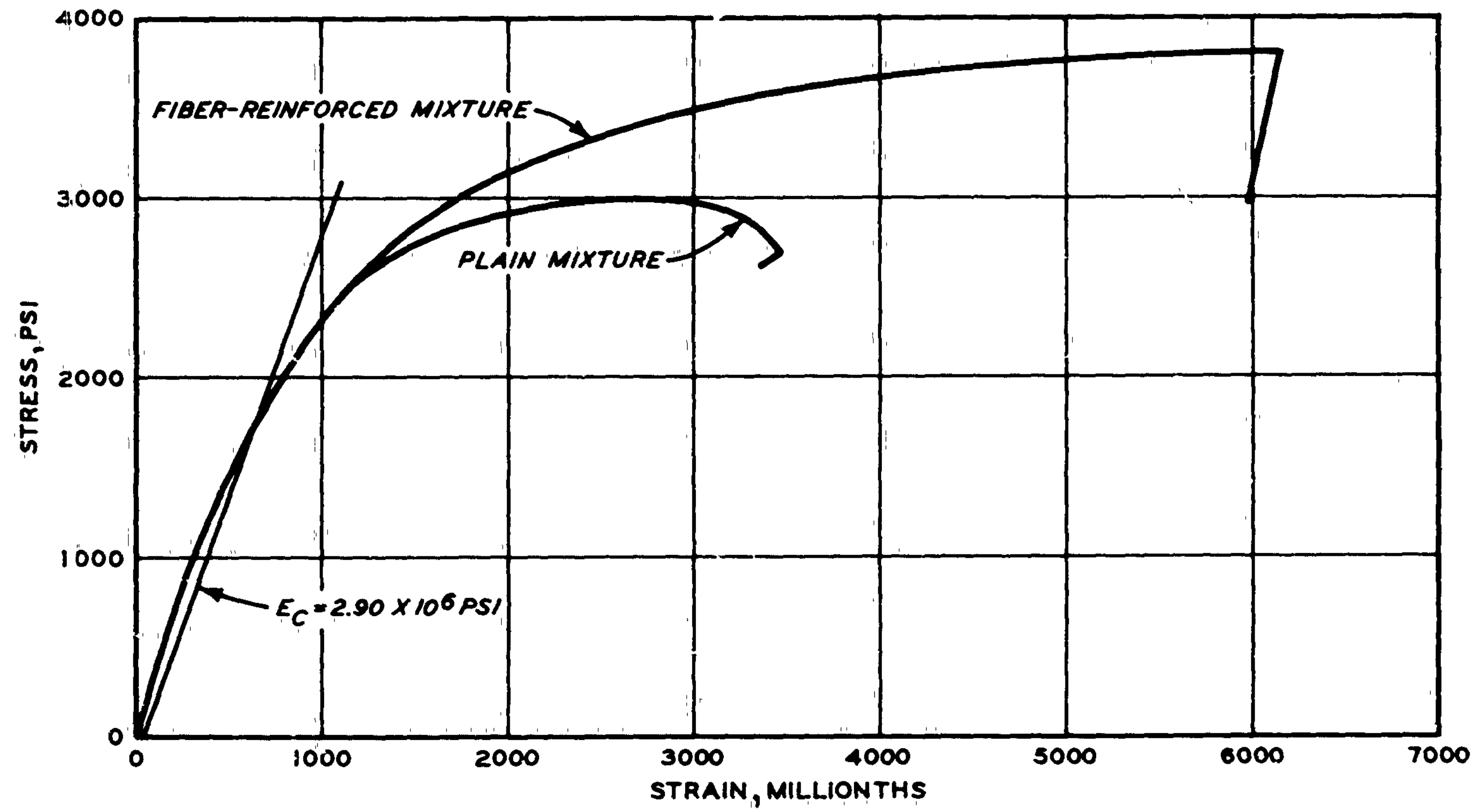
APPLIED LOAD (P)/ULTIMATE (FAILURE) LOAD (P_U) VERSUS MAXIMUM CRACK WIDTH FOR GROUP 3 BEAMS

PLATE 12

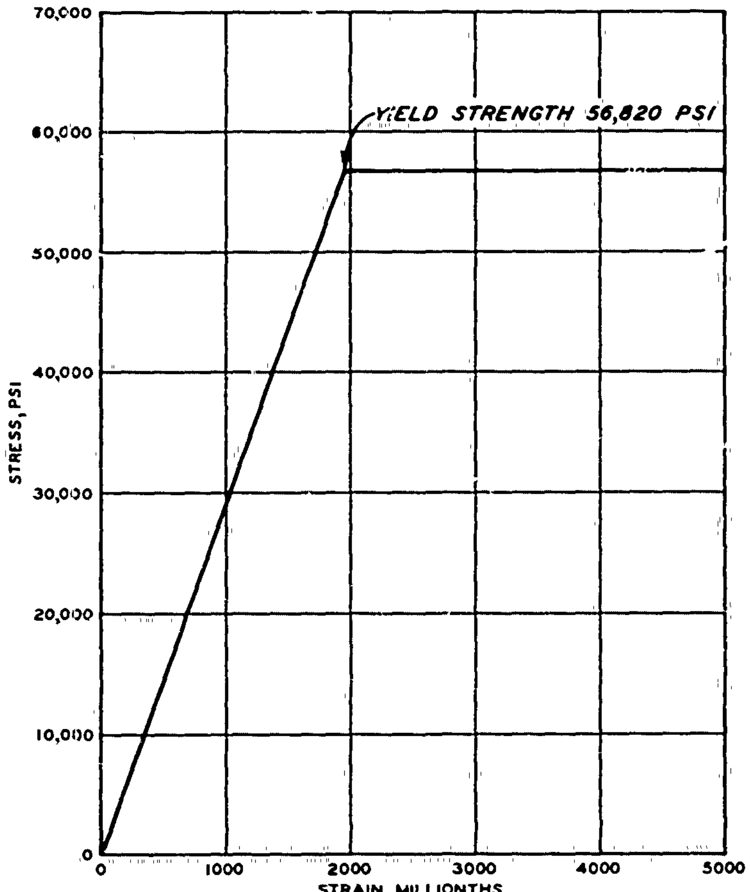


APPLIED LOAD (P)/ULTIMATE (FAILURE) LOAD (P_u) VERSUS MAXIMUM CRACK WIDTH FOR GROUP 4 BEAMS

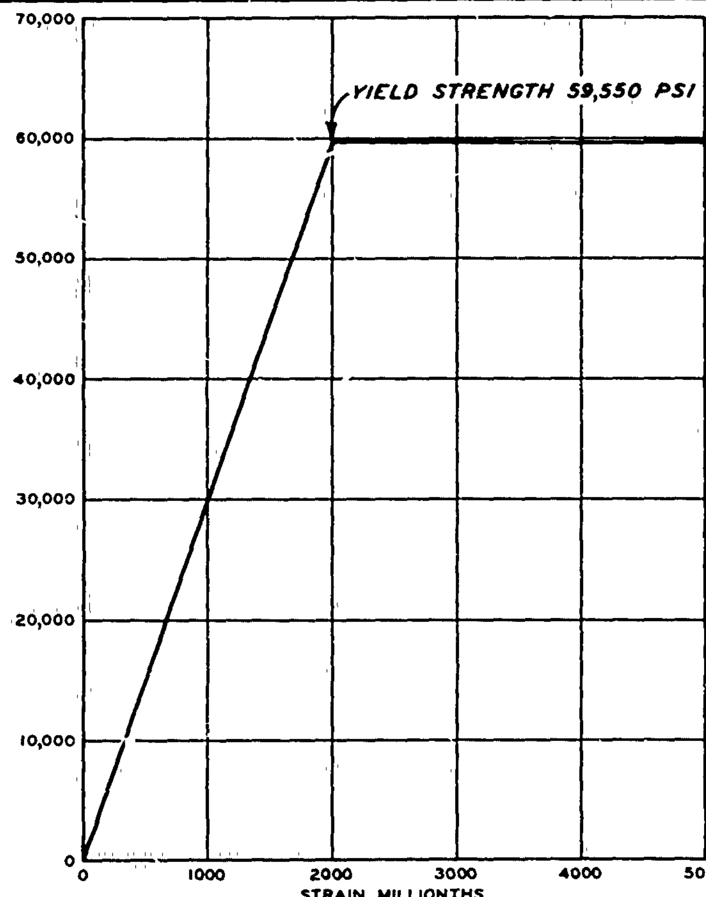
PLATE 13



TYPICAL STRESS-STRAIN CURVES FOR PLAIN AND FIBER-REINFORCED CONCRETE MIXTURES USED IN FABRICATING LAGER BEAMS (BEAMS 1L-8L)

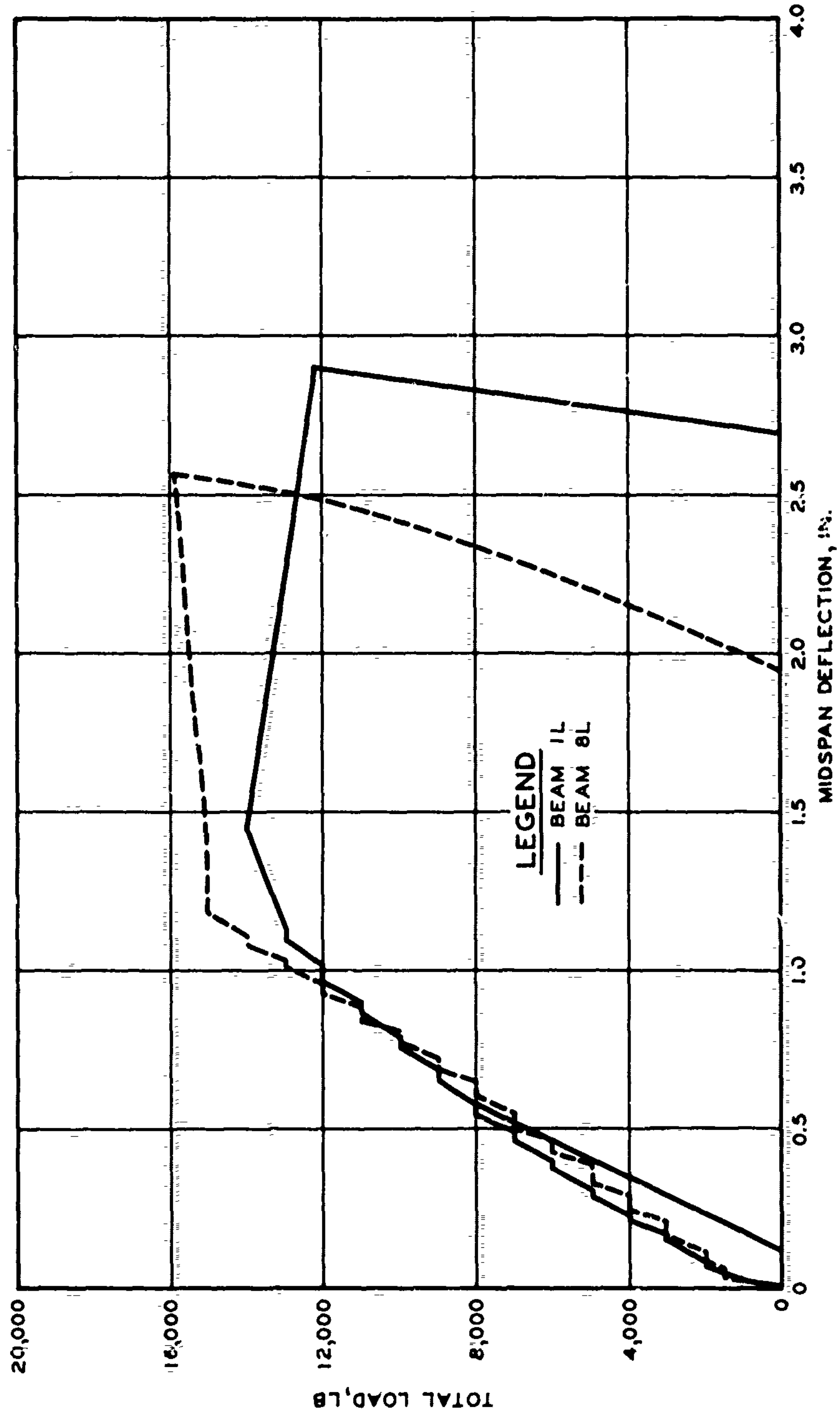


a. CURVE FOR NO. 6 BARS (FLEXURAL REINFORCEMENT)

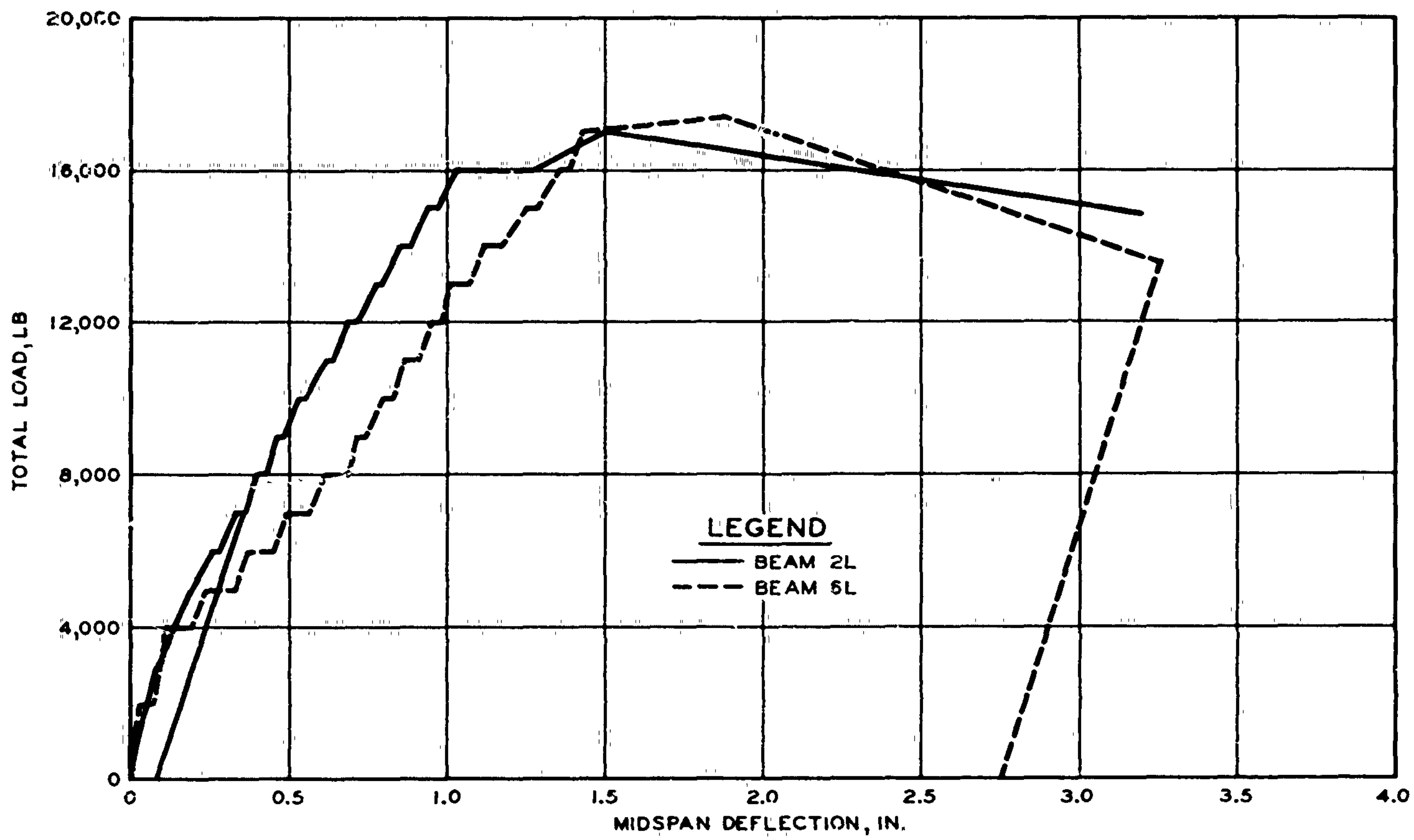


b. CURVE FOR NO. 3 BARS (CONVENTIONAL SHEAR REINFORCEMENT)

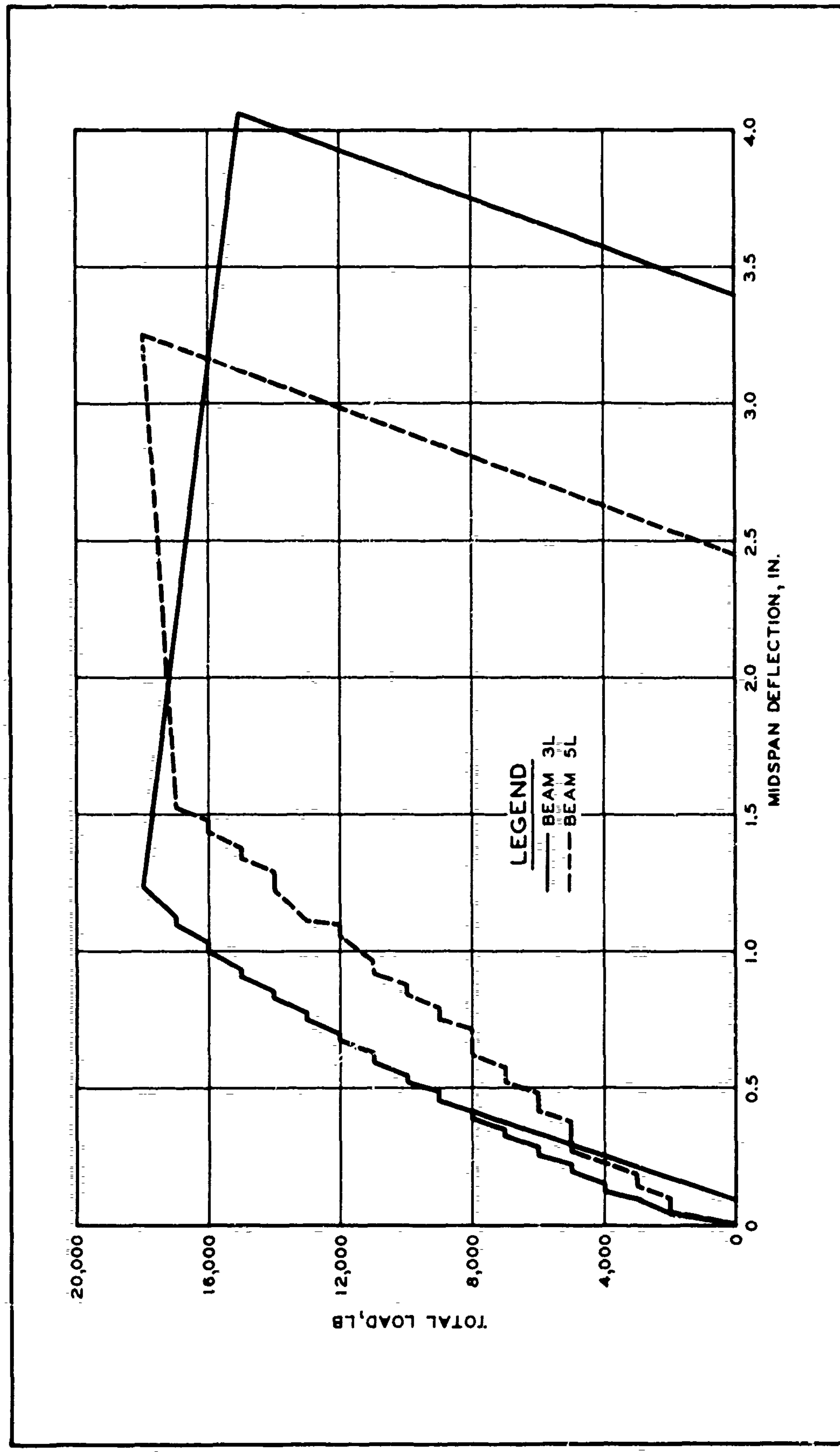
STRESS – STRAIN CURVES FOR
NO. 6 AND NO. 3 REINFORCING BARS
USED IN LARGER BEAMS



LOAD VERSUS MIDSPAN DEFLECTION FOR BEAMS 1L AND 8L

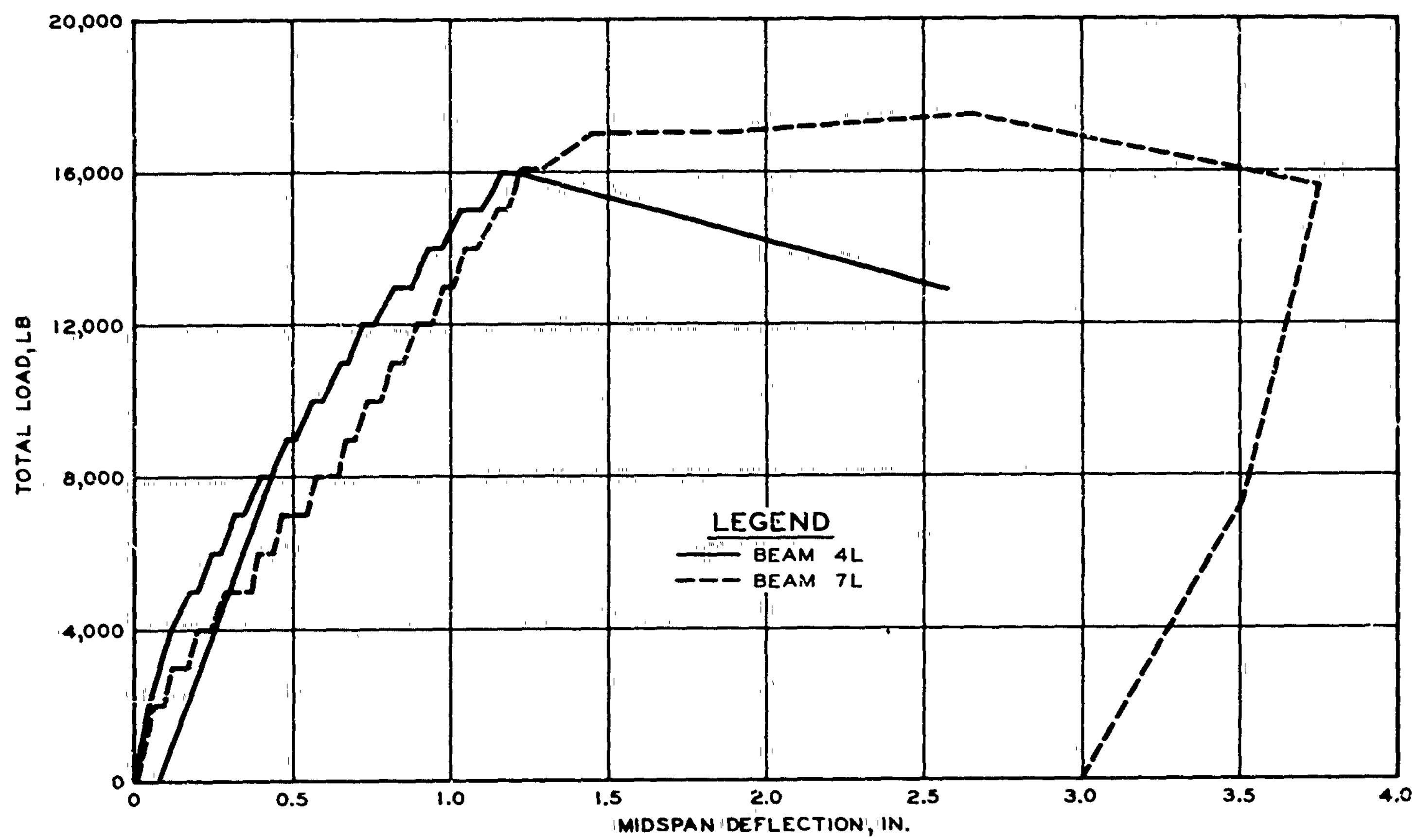


LOAD VERSUS MIDSPAN
DEFLECTION FOR BEAMS 2L AND 6L

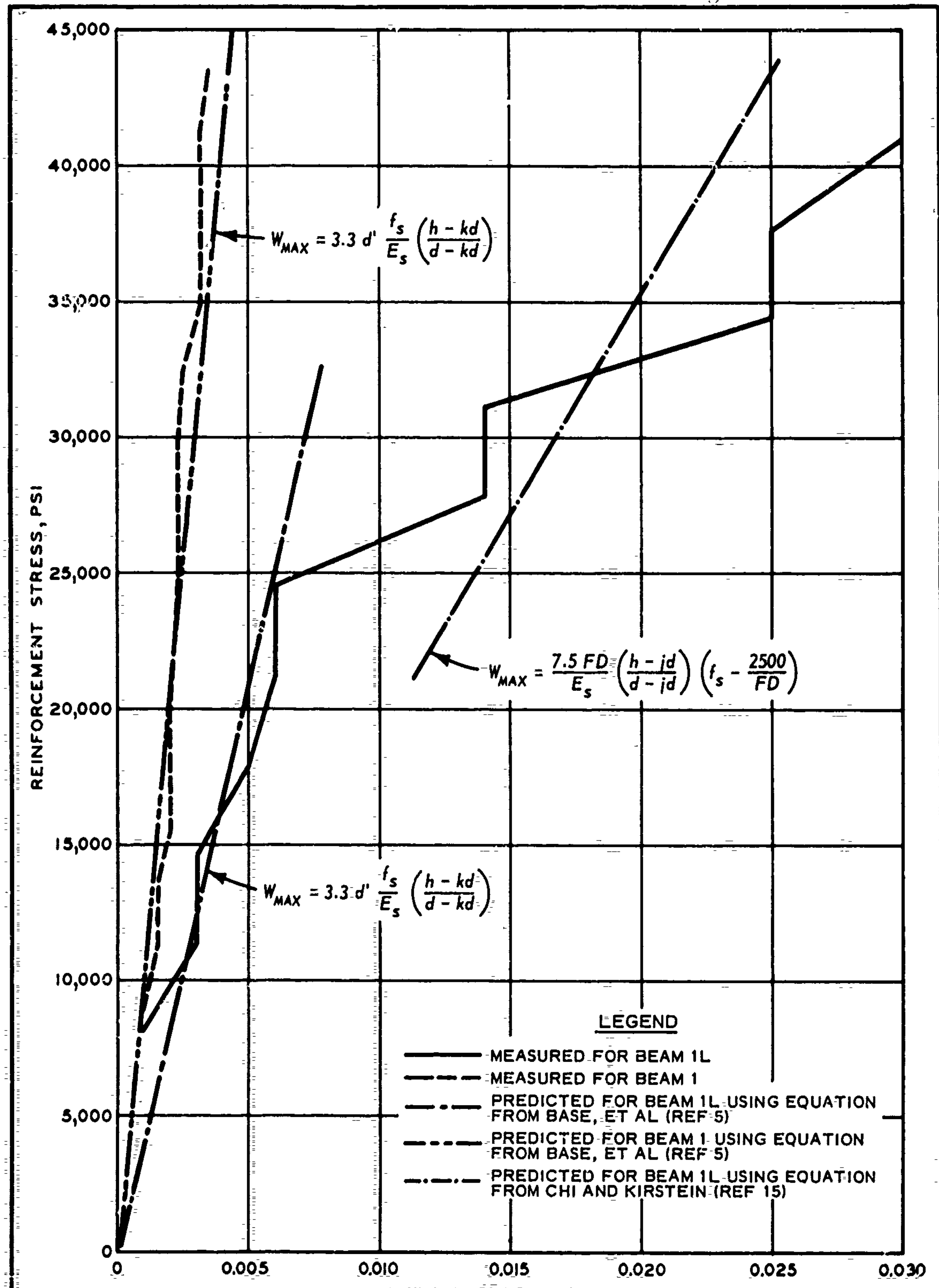


LOAD VERSUS MIDSPAN
DEFLECTION FOR BEAMS 3L AND 5L

PLATE 18

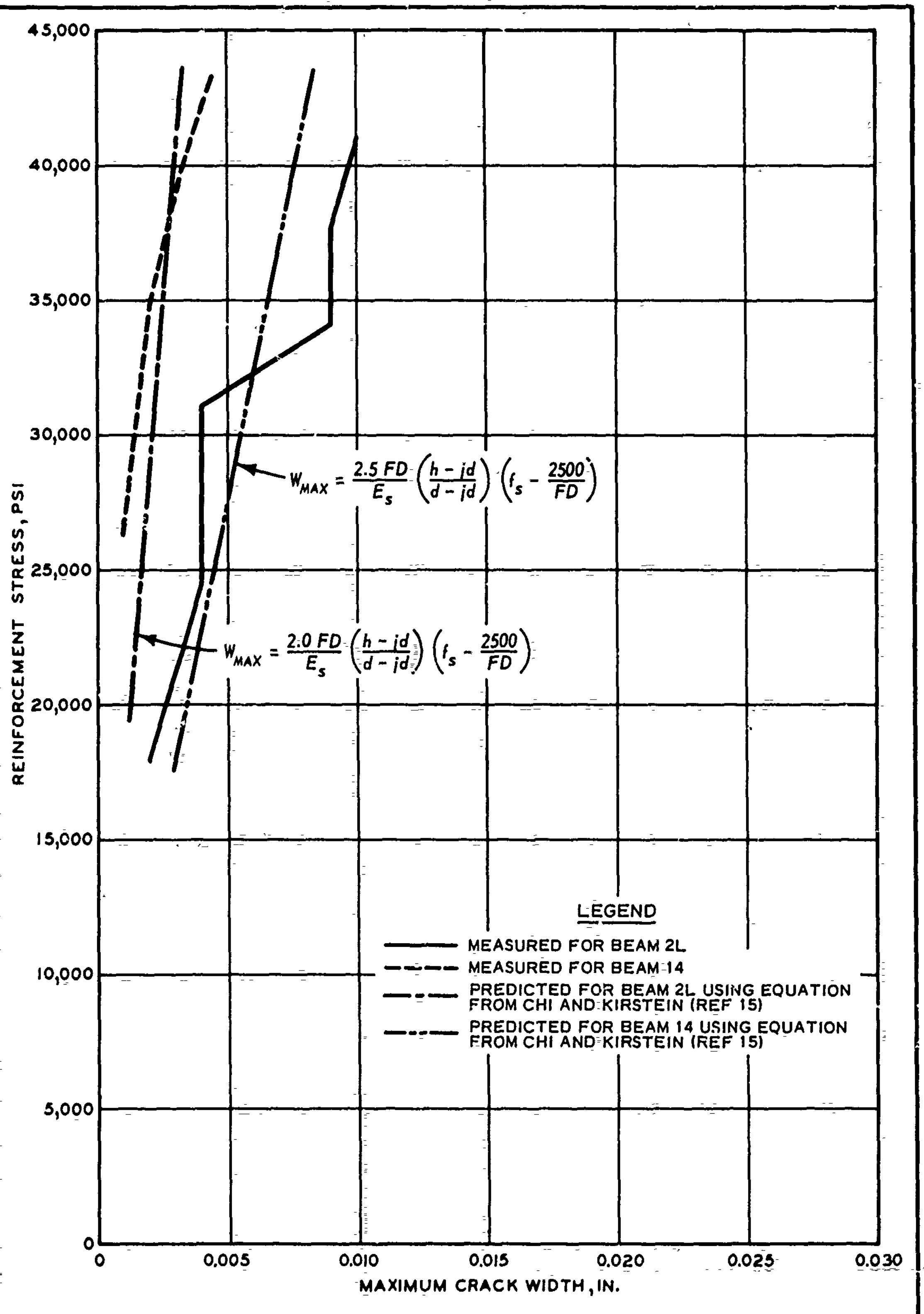


LOAD VERSUS MIDSPAN
DEFLECTION FOR BEAMS 4L AND 7L



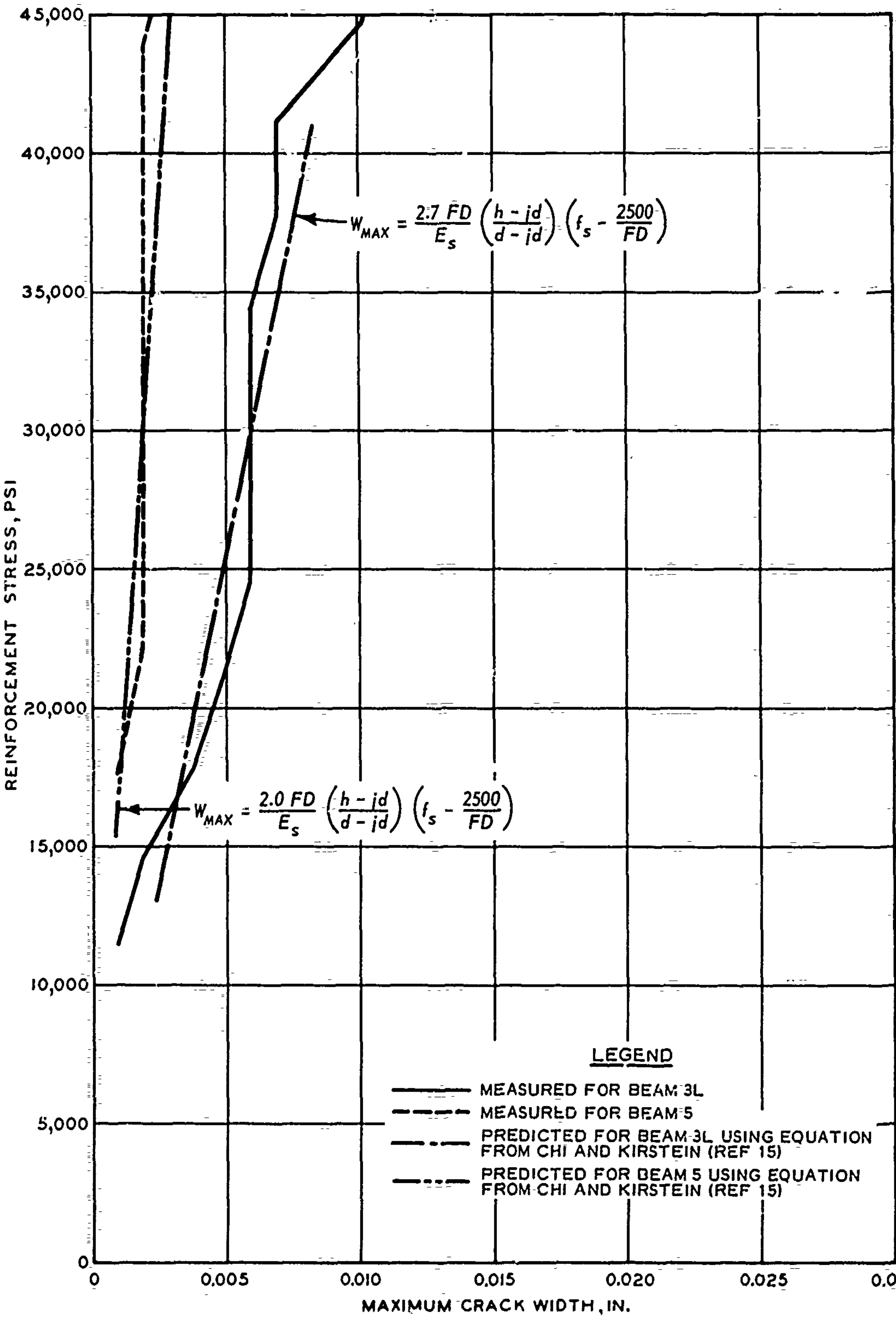
MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 1L AND 1

PLATE 20



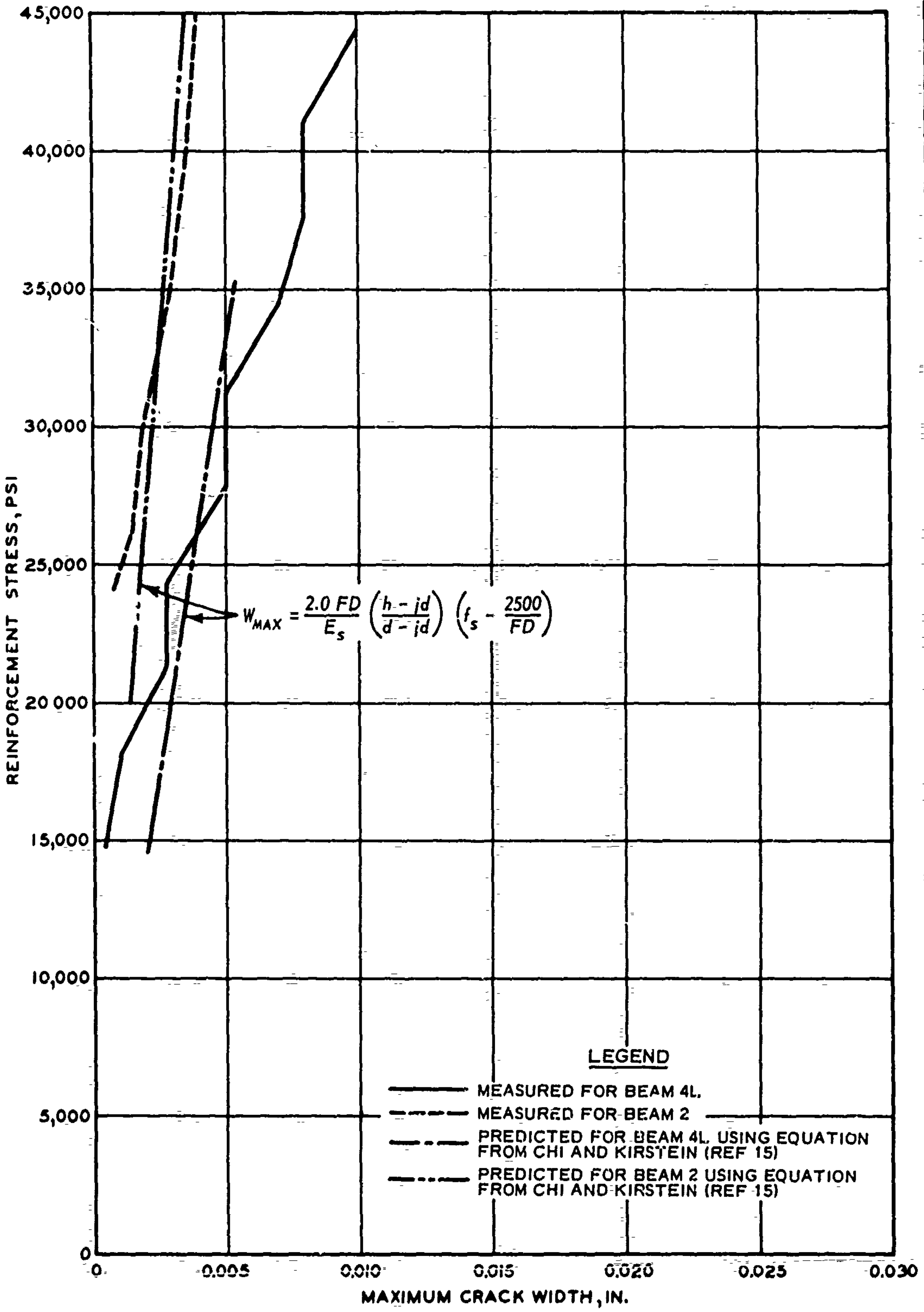
MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 2L AND 14

PLATE 21

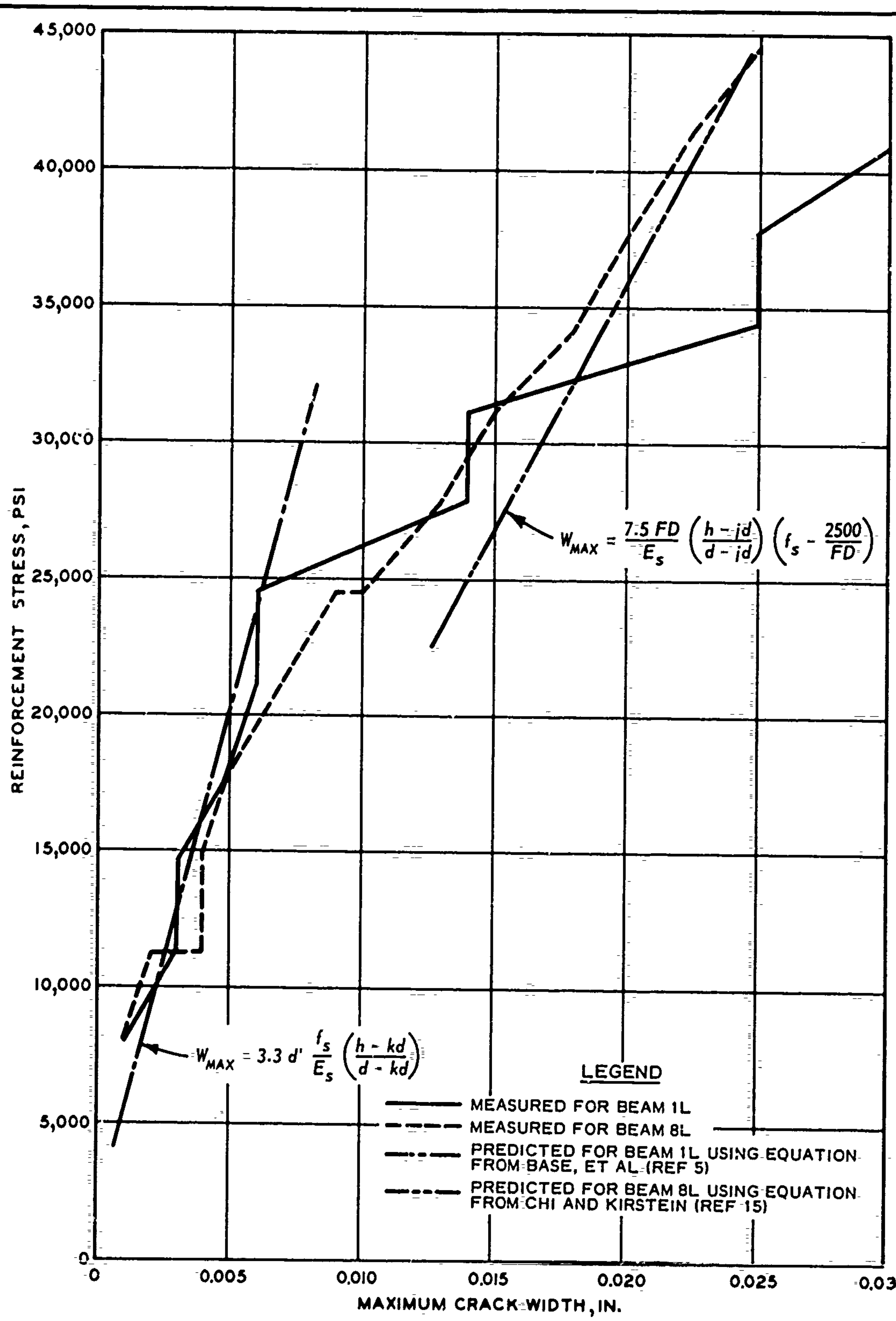


MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 3L AND 5

PLATE 22

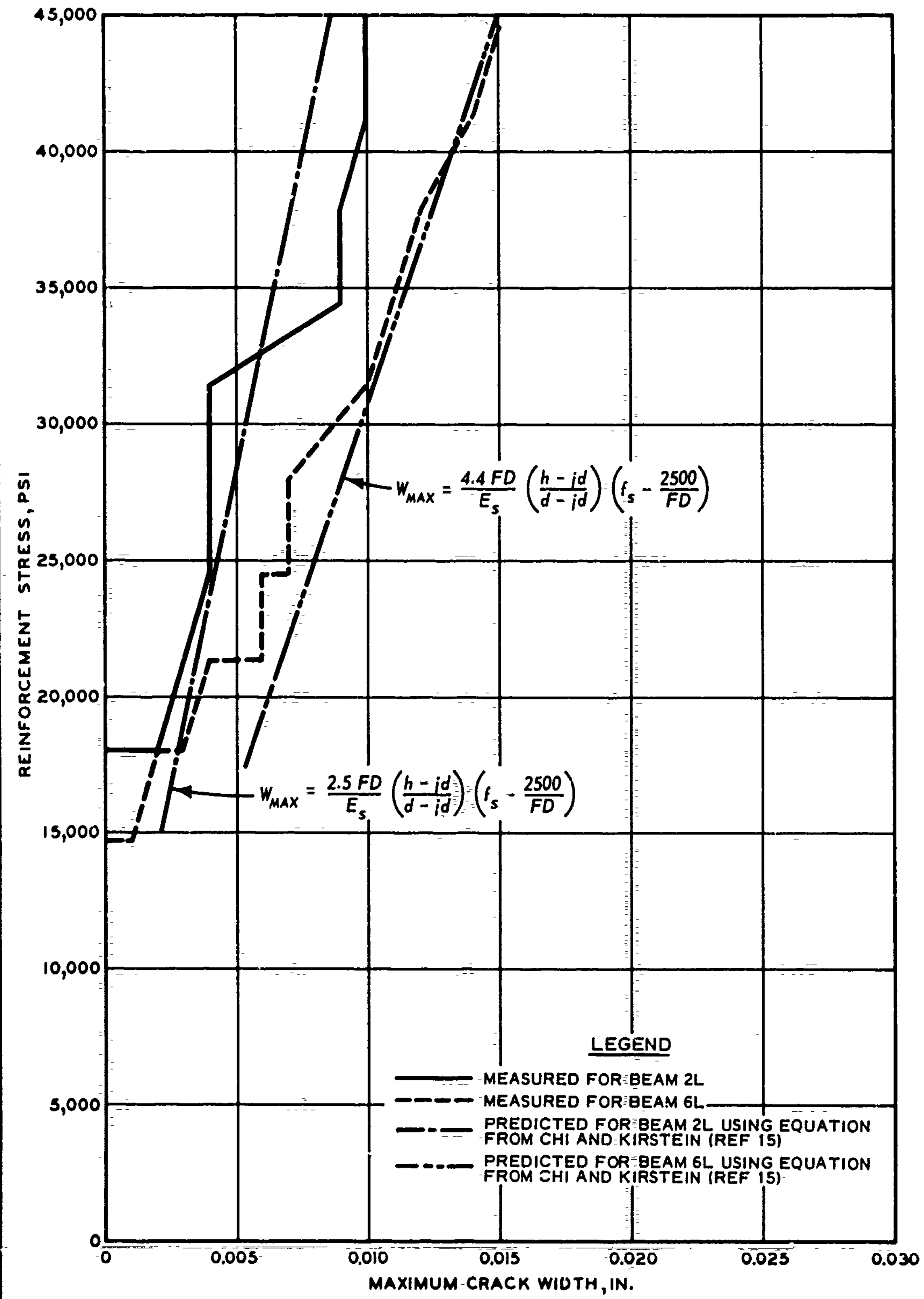


MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 4L AND 2



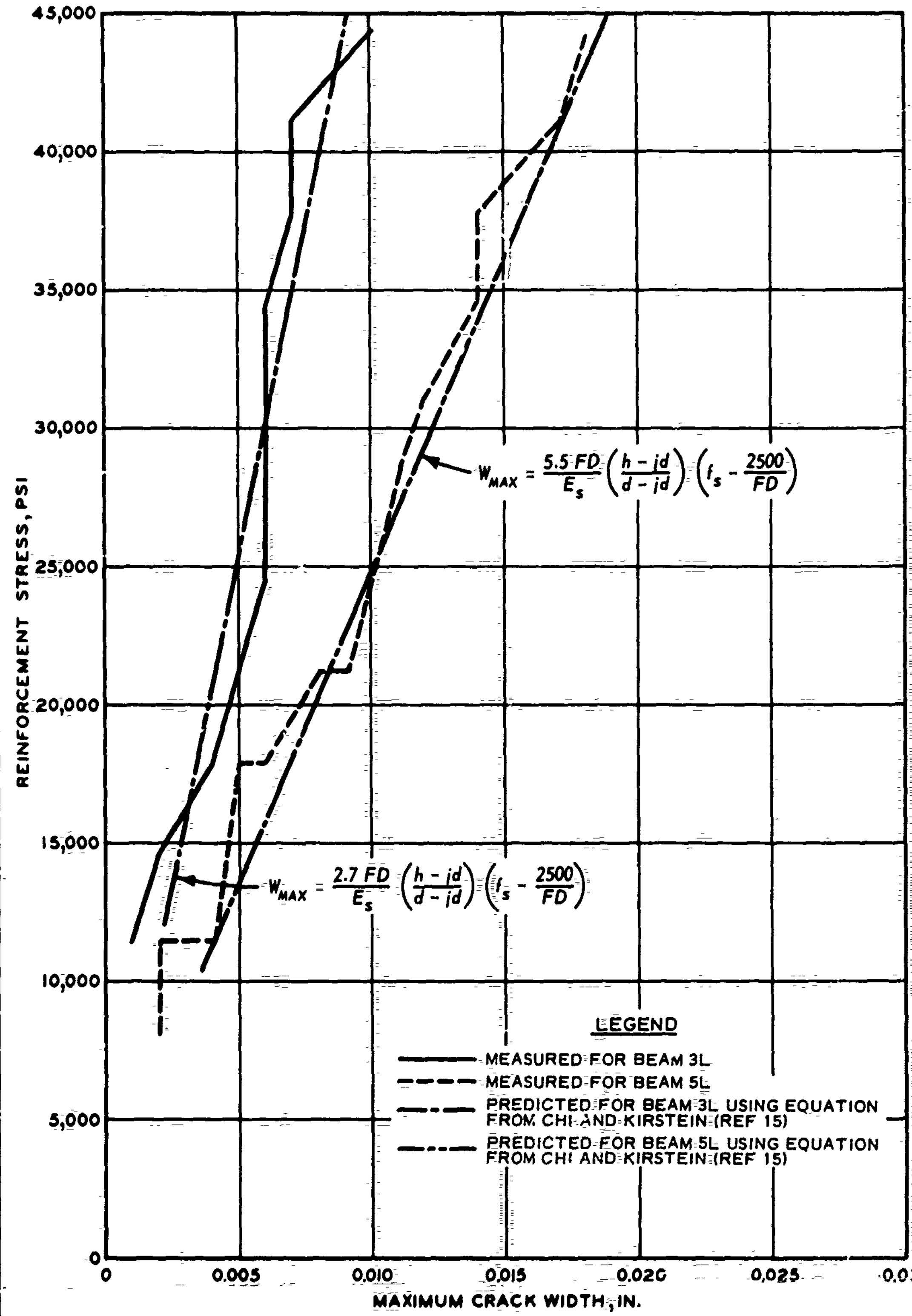
MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 1L AND 8L

PLATE 24



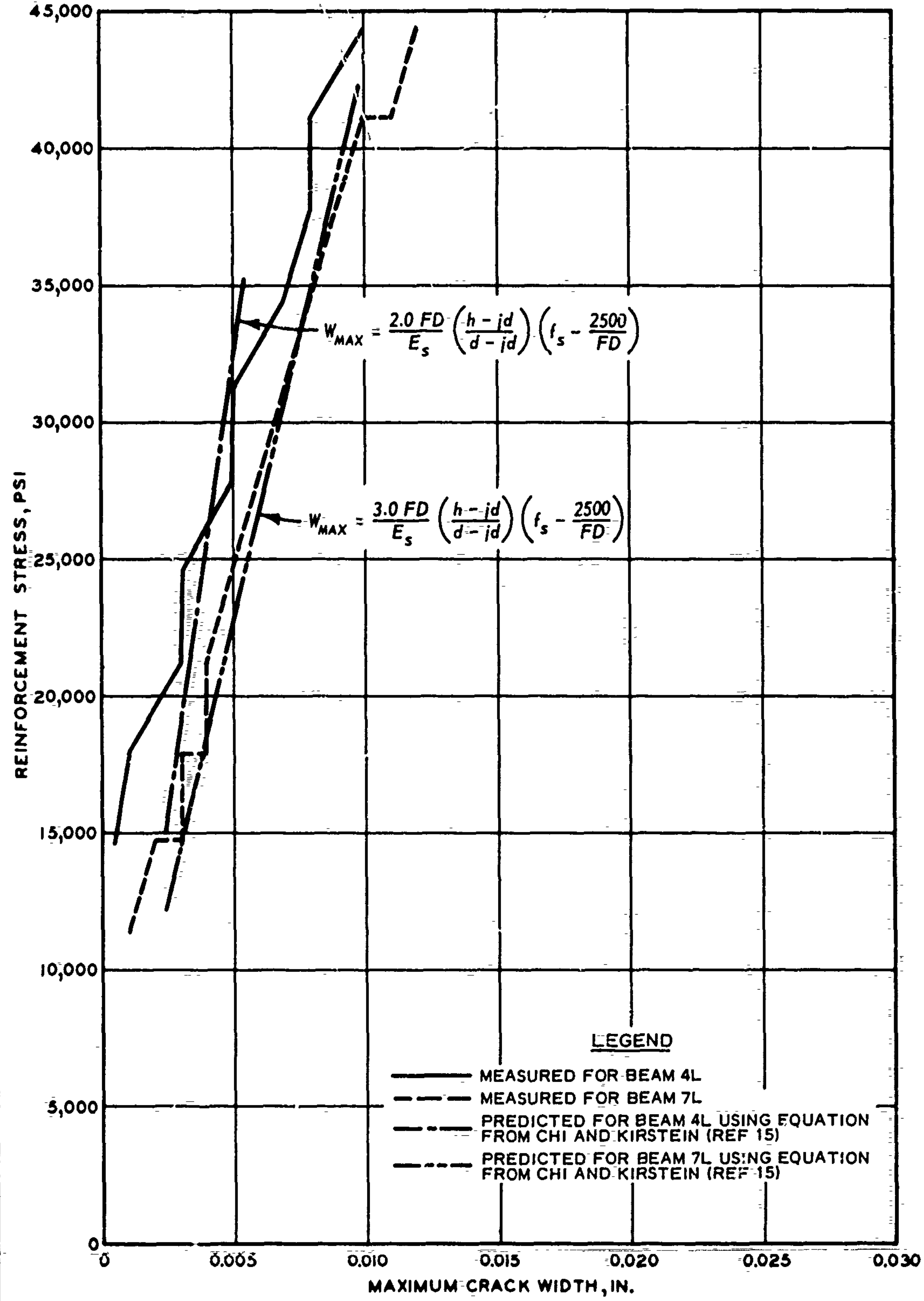
MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 2L AND 6L

PLATE 25



MAXIMUM CRACK WIDTH VS REINFORCEMENT STRESS FOR BEAMS 3L AND 5L

PLATE 26



MAXIMUM CRACK WIDTH
VS REINFORCEMENT STRESS
FOR BEAMS 4L AND 7L

PLATE 27

APPENDIX A: TYPICAL EXAMPLE ILLUSTRATING METHODS USED FOR DETERMINING MAXIMUM ANTICIPATED SERVICE LOADS

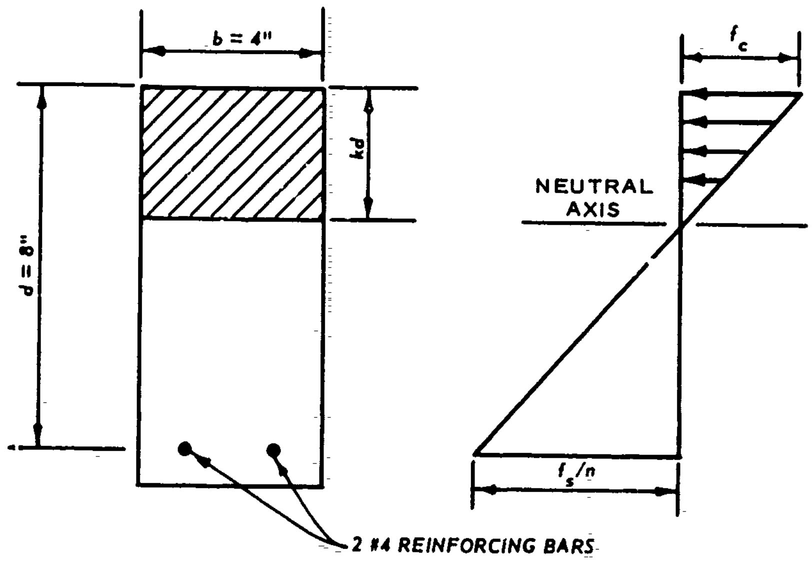


Fig. A1. Flexural reinforcement and assumed stress distribution of beam 1

1. For maximum anticipated service load of beam 1:

$$f_y = 62{,}000 \text{ psi}$$

$$f_c = 3510 \text{ psi}$$

$$E_c = W^{1.5} 33 \sqrt{f'_c} = (140)^{1.5}(33)(3510)^{0.5} = 3.24 \times 10^6 \text{ psi}$$

$$E_s = 30 \times 10^6 \text{ psi}$$

$$n = E_s/E_c = \frac{30 \times 10^6 \text{ psi}}{3.24 \times 10^6 \text{ psi}} = 9.26 \text{ ; } \underline{\text{use } 9}$$

$$p = A_s/bd = \frac{0.40 \text{ in.}^2}{(4 \text{ in.})(8 \text{ in.})} = 0.0125$$

$$k = \sqrt{(pn)^2 + 2pn} - pn$$

$$k = \sqrt{[(0.0125)(9)]^2 + 2(0.0125)(9)} - (0.0125)(9)$$

$$k = 0.4874 - 0.1125 = 0.3749 \text{ ; } \underline{\text{use } 0.37}$$

2. Since working-stress design is based on the maximum concrete stress being $0.45f'_c$, the maximum anticipated service load can be safely assumed to be at this level. The moment (M_{sl}) due to this service load is determined as follows:

$$f_c = 0.45f'_c = (0.45)(3510 \text{ psi}) = 1579.5 \text{ psi}$$

$$C_c = \frac{1}{2} f'_c kbd$$

$$C_c = \frac{1}{2} (1579.5 \text{ psi})(0.37)(4 \text{ in.})(8 \text{ in.}) = 9350.6 \text{ lb}$$

$$M_{int} = C_c Jd$$

where

$$J = 1 - 0.333k = 1 - (0.333)(0.37) = 1 - 0.1233 = 0.8767$$

$$M_{int} = (9350.6 \text{ lb})(0.8767)(8 \text{ in.}) = 65{,}581 \text{ in.-lb}$$

M_{sl} is defined as being the moment producing maximum concrete compressive stresses of $0.45f'_c$; therefore, the calculated M_{int} must be, by definition, equal to M_{sl} .

3. Then, since $M_{int} = M_{ext}$ (fig. A2), M_{sl} must be equal to M_{ext} which, at this point, is composed of the moment due to the beam's dead (M_{dl}) and live service (M_{lsl}) loads, both of which can be determined from the support and loading conditions of the individual beams. Therefore, M_{dl} and M_{lsl} were determined as follows:

$$M_{dl} = \frac{(4 \text{ in.})}{(12 \text{ in./ft})} \frac{(9 \text{ in.})}{(12 \text{ in./ft})} (2.25 \text{ ft})(150 \text{ lb/ft}^3) \frac{27 \text{ in.}}{2}$$

$$M_{dl} = 1139 \text{ in.-lb}$$

then

$$M_{lsl} = M_{sl} - M_{dl} = 65{,}581 \text{ in.-lb} - 1139 \text{ in.-lb}$$

$$M_{lsl} = 64{,}442 \text{ in.-lb}$$

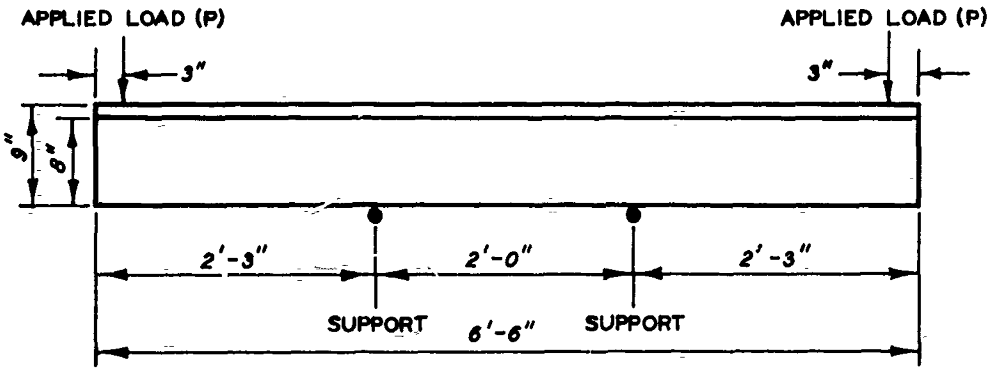


Fig. A2. Position of beam at testing

4. Now that M_{1sl} is known, the maximum anticipated live service load (P_{1sl}) can be determined from the loading conditions of the beams by using the following procedure:

$$M_{1sl} = (0.5\, P_{1sl})(24 \text{ in.})$$

$P_{1sl} = \dfrac{64{,}442 \text{ in.-lb}}{(0.5)(24 \text{ in.})} = 5370$ lb, which by definition is the largest load (excluding weight of beam) anticipated for the beam.

5. It is emphasized that $0.45f'_c$ was used in these calculations. For hydraulic structures, which this investigation is directed to primarily, the allowable stresses for normal service loads are $0.35f'_c$ for the concrete and 20,000 psi for the steel because $0.45f'_c$ is the maximum allowable concrete stress for a vast majority of all remaining types of reinforced concrete structures as well as for individual structural elements.

APPENDIX B: TYPICAL EXAMPLE ILLUSTRATING METHOD USED FOR DETERMINING MAXIMUM CONCRETE TENSILE STRESSES AT INITIAL CRACKING LOADS OF INDIVIDUAL MEMBERS

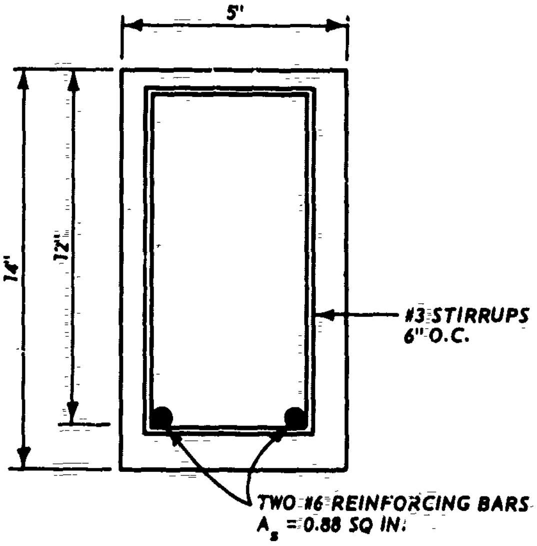


Fig. B1. Typical cross section of beam 1L

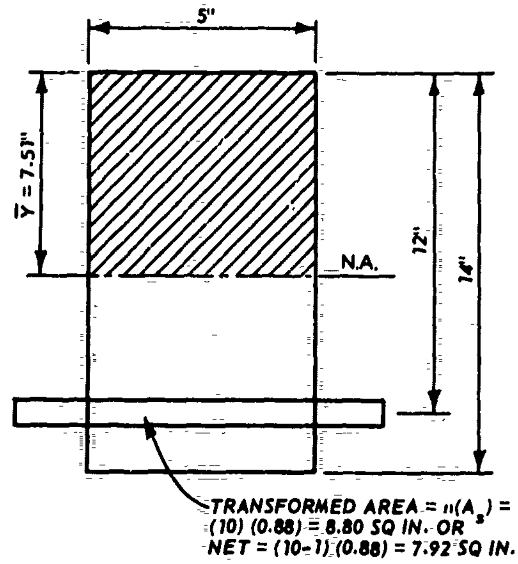


Fig. B2. Transformed cross section of beam 1L

1. For neutral axis (NA) of beam:

$$E_c \text{ (plate 14)} = 2.90 \times 10^6 \text{psi}$$

$$E_s \text{ (plate 15)} = 2.90 \times 10^6 \text{ psi}$$

$$n = E_s/E_c = \frac{29.0 \times 10^6 \text{ psi}}{2.90 \times 10^6 \text{ psi}} = 10$$

$$\overline{Y} = \frac{(5 \text{ in.})(14 \text{ in.})(7 \text{ in.}) + (7.92 \text{ in.}^2)(12 \text{ in.})}{(5 \text{ in.})(14 \text{ in.}) + 7.92 \text{ in.}^2} = 7.51 \text{ in.}$$

2. For moment of inertia of beam:

$$I_c = \frac{bh^3}{12} + A_c(\overline{Y} - 7)^2 = \frac{(5 \text{ in.})(14 \text{ in.})^3}{12} + (5 \text{ in.})(14 \text{ in.})(0.51 \text{ in.})^2$$

$$= 1161.5 \text{ in.}^4$$

$$I_{st} = (7.92 \text{ in.}^2)(12 \text{ in.} - 7.51 \text{ in.})^2 = (7.92 \text{ in.}^2)(4.49 \text{ in.})^2$$

$$= 159.7 \text{ in.}^4$$

$$I_t = I_c + I_{st} = 1161.5 \text{ in.}^4 + 159.7 \text{ in.}^4 = 1321.2 \text{ in.}^4$$

3. For moment of beam:

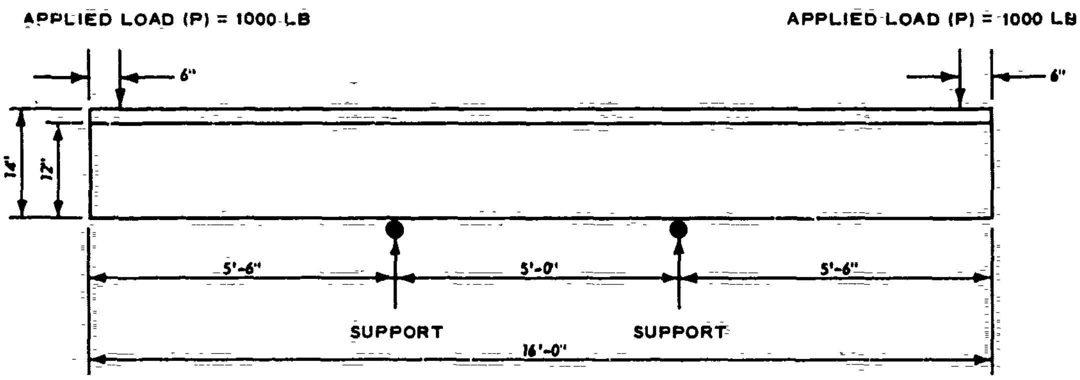


Fig. B3. Position of beam at testing

Total load required to initiate cracking is 2000 lb (table 24)

$$M_{ll} = (1000 \text{ lb})(60 \text{ in.}) = 60{,}000 \text{ in.-lb}$$

$$M_{dl} = \frac{(5 \text{ in.})}{(12 \text{ in./ft})} \frac{(14 \text{ in.})}{(12 \text{ in./ft})} (5.5 \text{ ft})(150 \text{ lb/ft}^3)(33 \text{ in.})$$

$$= 13{,}234 \text{ in.-lb}$$

$$M_t = M_{ll} + M_{dl} = 60{,}000 \text{ in.-lb} + 13{,}234 \text{ in.-lb} = 73{,}234 \text{ in.-lb}$$

4. Stress of concrete when cracking initiated: equate external to internal moments, then

$$M_t = M_{int}$$

$$f_{tc} = \frac{(M_{int})(14 - \bar{y})}{I} = \frac{(73{,}234 \text{ in.-lb})(14 \text{ in.} - 7.51 \text{ in.})}{1321.2 \text{ in.}^4}$$

$$f_{tc} = 359.7 \text{ or } 360 \text{ psi}$$

APPENDIX C: TYPICAL EXAMPLE ILLUSTRATING METHOD USED FOR DETERMINING REINFORCEMENT STRESSES RESULTING FROM VARIOUS LEVELS OF LOADING

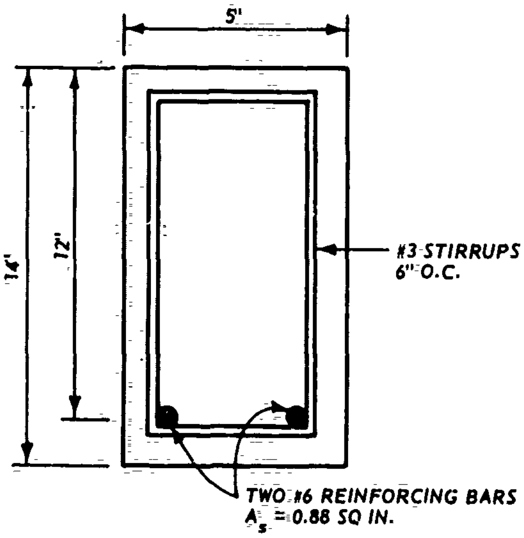


Fig. C1. Typical cross section of beam 1L

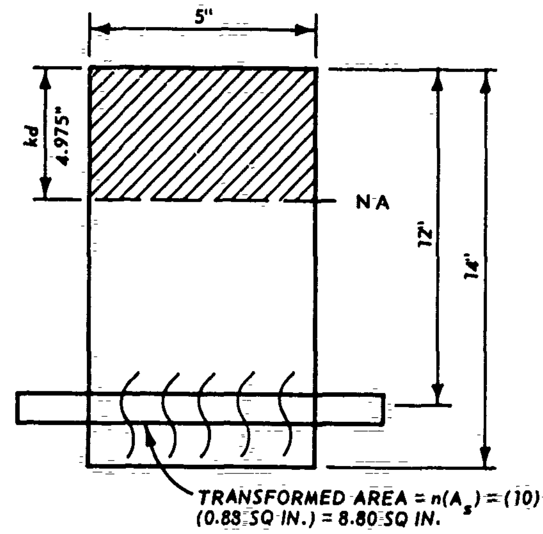


Fig. C2. Transformed cross section of beam 1L

1. For neutral axis (NA) of beam (assuming all tensile stresses are transferred to reinforcement once cracking is initiated).

Moments about NA give:

$$(5 \text{ in.})(kd)(kd/2) = (8.80 \text{ in.}^2)(12 \text{ in.} - kd)$$

$$(2.5 \text{ in.})(kd)^2 = 105.60 \text{ in.}^3 - 8.80 \text{ in.}^2\, kd$$

$$kd^2 + 3.5\, kd - 42.24 = 0$$

$$kd = \frac{-3.52 \pm \sqrt{(3.52)^2 - (4)(1)(-42.24)}}{2(1)}$$

$$kd = \frac{-3.52 + 13.47}{2} = 4.975 \text{ in.}$$

2. After locating neutral axis as shown and then equating

external to internal moments, the reinforcement stresses are determined as follows:

$$M_t = M_{int} = A_s f_s jd$$

$$jd = d - \frac{1}{3}\,(kd) = 12 \text{ in. } \frac{4.975 \text{ in.}}{3} = 10.34 \text{ in.}$$

Using a 6000-lb load and the procedures outlined in Appendix B,

$$M_t = 193{,}234 \text{ in.-lb}$$

$$f_s = \frac{(M_{int})}{(A_s)(jd)} = \frac{193{,}234 \text{ in.-lb}}{(0.88 \text{ in.}^2)(10.34 \text{ in.})} = 21{,}236 \text{ or } 21{,}240 \text{ psi}$$

APPENDIX D: NOTATION

A_c Cross-sectional area of concrete = $(b \times h)$

A_e Effective concrete area in uniform tension, which is assumed to be $2(h - d)b$

A_s Area of conventional (tensile) reinforcement

b Width of beam

C_c Concrete compressive force

d Distance from center of gravity of conventional reinforcement to the top fiber of concrete

d' Distance from the point of measurement of a crack to the surface of the nearest reinforcing bar

D Diameter of individual reinforcing bar

E_c Modulus of elasticity of conventional concrete

E_s Modulus of elasticity of conventional reinforcement

f_c Concrete compressive (unit) stress in the uppermost fibers of the beam

f'_c Compressive strength of concrete

f_j Reinforcement field strength

f_s Reinforcement stress

f_{tc} Concrete tensile stress at which cracking occurs in lowermost fibers of beams

F Term of a crack width prediction equation (considered to be equal to $1/m^2P_e$)

h Height of beam

I_c Moment of inertia of the gross concrete cross section of beam

I_{st} Moment of inertia of the net transformed area of reinforcement

I_t Total moment of inertia or $I_c + I_{st}$

J Constant = $1 - k/3$

Jd Moment arm or distance between the compressive and tensile forces

k Constant depending on properties of conventional concrete and reinforcement

kd Distance from neutral axis to the top fiber of concrete when tensile strength of the concrete is neglected

L Denotes larger beams

m Constant which is considered to be equal to 4

M_{dl}	Moment due to the dead load
M_{ext}	External moment of beam
M_{int}	Internal moment of beam
M_{ll}	Moment due to the live load
M_{lsl}	Live service load moment, or the live load moment which when combined with the dead load moment (M_{dl}) produces the maximum anticipated moment of the service loads (M_{sl})
M_{sl}	Maximum anticipated moment of the service loads, or the moment required to produce concrete compressive stresses of $0.45\ f'_c$
M_t	Total external moment or $M_{dl} + M_{ll}$
n	E_s/E_c
p	Reinforcement ratio = A_s/bd
P	Any load applied to beam
P_b	Balanced design
P_c	Initial cracking load of beam
P_e	Ratio of area of conventional reinforcement to the effective concrete area subject to uniform tension or A_s/A_e
P_{lsl}	Live service load
P_u	Ultimate failure load of beam
W	Weight (mass) of concrete
W_{max}	Maximum crack width
$\bar{Y}$	Distance from neutral axis to the top fiber of concrete when the tensile strength of the concrete is considered

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Cox, Frank B
Crack-arrest techniques in reinforced concrete structural elements; Report 1: Laboratory tests, by Frank B. Cox. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1974.
1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Technical report C-74-7, Report 1)
Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C.
Includes bibliography.

1. Concrete beams. 2. Concrete cracking. 3. Crack arresters. 4. Reinforced concrete. I. U. S. Army. Corps of Engineers. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Technical report C-74-7, Report 1)
TA7.W34 no.C-74-7 Report 1