FATIGUE OF PRESTRESSED CONCRETE MEMBERS

Thesis for the Degree of M. S.

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Vejubhai Gulababhai Patel

1961

This is to certify that the

thesis entitled

FATIGUE OF PRESTRESSED CONCRETE MEMBERS

presented by

Vejubhai Gulababhai Patel

has been accepted towards fulfillment of the requirements for

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O. E. Cutts

Major professor

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FATIGUE OF PRESTRESSED CONCRETE MEMBERS

By

Vejubhai Gulababhai Patel

AN ABSTRACT OF A THESIS

Submitted to
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ABSTRACT

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The object of this study is to determine the effect of repetitive flexural loading in pre-tensioned prestressed concrete members. The flexural loading effected reversal of stress in the test members, which were prestressed at the neutral axis of the members so that the prestress in the wires was not affected by the repetitive loading. Twenty-two specimens three inches by three inches in cross-section and fourteen and a half inches long were investigated at two levels of prestressing force. Flexural loadings were varied from thirty-six to fifty-two percent of the static ultimate flexural strength under third point loading. The beams were subjected to repetitive loading until fatigue failure occurred or if it did not occur, the loading was stopped at two and one-half million cycles. Fatigue failure occurred at approximately forty-five percent of the ultimate static strength of the members. Loss of prestress force did not occur in the wires.

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I. INTRODUCTION

During the past ten years, the use of prestressed concrete has increased progressively in all types of concrete structures. In utilizing this new technique, high strength steel and high strength concrete are required. It is natural that engineers should question the use of this method when confronted with the requirement of higher concrete stresses and the extremely high bond stresses. It is therefore necessary to determine the mechanical properties of the material, particularly with respect to fatigue loading.

In pre-tensioned prestressed concrete, the steel is tensioned before the concrete is placed and is released after the concrete has developed sufficient strength. The tension in the steel is transferred to the concrete entirely by bond.

Ultimate strength of concrete and steel subjected to repetitive loading may be less than static strength because of the phenomenon of fatigue. The significance of fatigue in prestressed concrete members has not been completely explored. Fatigue failure may occur in concrete, steel, splices, anchorages, or bond.

Information on the fatigue behavior of pre-tensioned prestressed concrete in flexure under reversal of loading is limited. This study seeks to determine the effect of reversal loading in pre-tensioned prestressed concrete members.

II. REVIEW OF PREVIOUS INVESTIGATIONS

General Information

When a material fails under a number of repeated loads, each smaller than the single static load which would cause failure, it is said to have failed in fatigue. Both concrete and the steel used in reinforced concrete or prestressed concrete possess the characteristic of failing by progressive or gradual fracture which becomes complete with load repetitions. Thus the factor of safety in a concrete member is related not only to the intensity of stress level but also to the number of cycles of loading.

Fatigue results are usually presented in the form of an S-N curve (stress versus log of number of cycles of load) as illustrated in Fig. 1a. If the curve becomes asymptotic to a line parallel to the horizontal axis, the bounding stress is called an endurance limit or fatigue limit. It does not appear that concrete has an endurance limit, thus, the curve continues to slope downward as shown in Fig. 1b. The fatigue strength is the strength for any predetermined number of cycles of load, usually the end point of the curve.

Plain Concrete

The fatigue of concrete in compression is particularly important because concrete is normally designed to carry compressive stress. The strength of plain concrete subjected to repeated compressive loads is approximately 50 to 55 percent of the static ultimate strength.

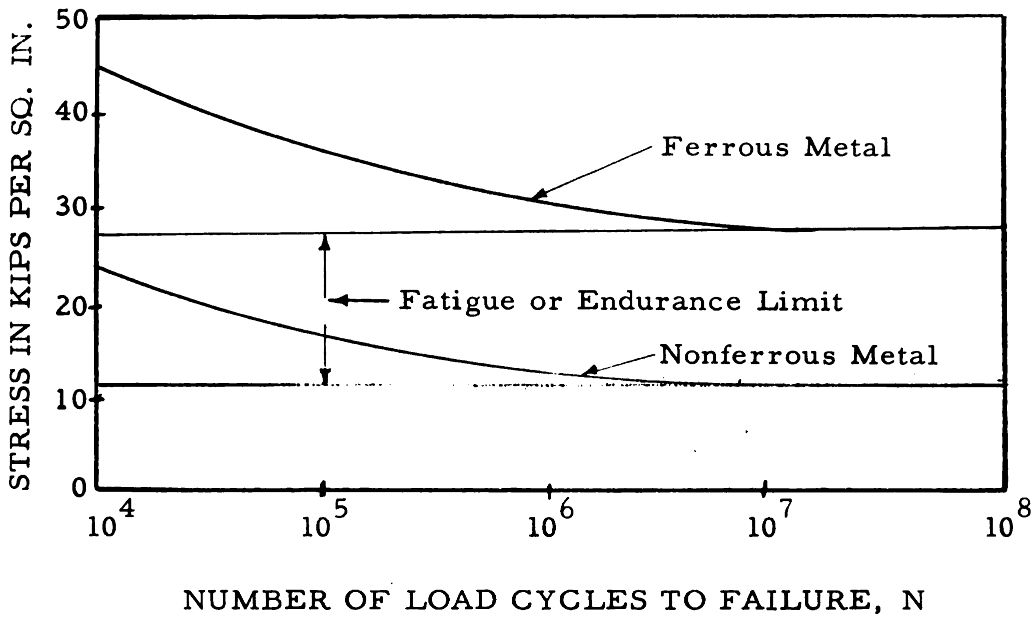


Fig. 1(a). Typical S-N diagram for fatigue in metals.

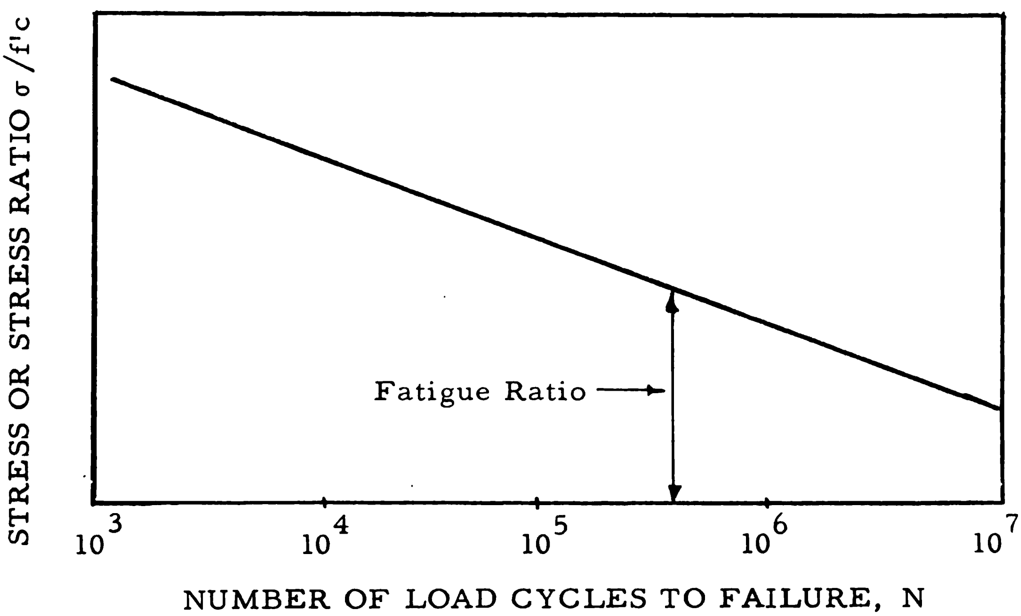


Fig. 1(b). Typical S-N diagram for plain concrete.

For plain concrete subjected to flexural stresses from zero to a maximum value, the fatigue strength is about 55 percent of the static ultimate strength.

Investigations of plain concrete beams under repeated loads at low stress level indicate a beneficial effect under later static or repeated loading. Rest periods also appear to be beneficial in increasing fatigue limits. Frequency of loading varied between 70 and 440 cycles per minute appears to have no effect on fatigue. Inadequately cured concrete is less resistant to fatigue than properly cured concrete, and there is some evidence that leaner mixes are less resistant than richer mixes.

Most early papers referred to concrete as having an endurance limit similar to most metals. Recent studies by Kesler (13) indicate that plain concrete in flexure does not have an endurance limit, at least up to 10, 000, 000 cycles of load. In regard to bond strength, little can be said except that fatigue failures are possible at loads less than 55 percent of the ultimate static pull-out strengths.

In a recent paper, Murdock (14) has formulated a hypothesis of the fatigue failure of concrete. Hypothesis: The initiation of the fatigue failure may reasonably be attributed to the progressive deterioration of the bond between the coarse aggregate and the binding matrix together with an accompanying reduction of section of the specimen. He showed that the final fracture of the specimen occurs by the gradual deterioration of the paste-fine aggregate bond, if the remaining section is sufficient to withstand the applied load.

Reinforced Concrete

The characteristics of fatigue failure in reinforced concrete may be summarized by the following statements: Nordby (15).

a. Most failures of reinforced beams were due to failure of the reinforcing steel. Beams critical in longitudinal reinforcement seemed to have an endurance limit of 60-70 percent of static ultimate strengths for 1,000,000 cycles.

b. On occasion, beams failed in diagonal tension fatigue but the real cause of failure was obscured by bond and shear combination failures. Tests have been reported in which beams have failed in shear by repeated loads as low as 40 percent of the ultimate strength.

   In addition beams accumulate residual deflections under extensive fatigue loading much the same way as plain concrete specimens but recover somewhat during rest periods. Web reinforcement is shown to increase the fatigue limit.

Prestressed Concrete

From a fatigue standpoint the methods of failure are essentially the same for a prestressed concrete member as for a conventionally reinforced concrete beam, i.e., (a) fatigue of concrete in the compression zone, or in diagonal tension, (b) in the prestressing steel, (c) in bond, and (d) for post-tensioned beams by the fatigue failure of anchorages and splices. Unfortunately, both the early static and fatigue tests carried on for these purposes were inadequately instrumented, or a suitable criterion for performance was not established. Consequently, early conclusions were limited to the performance of a particular beam.

The first studies on prestressed concrete were carried out by Freyssinet (8) in 1934. He found the behavior of prestressed concrete from the fatigue point of view to be superior to conventional reinforced concrete.

Some fatigue studies made by Ros (19) with prestressed concrete I-beams having different prestressing forces were reported. The fatigue strength of the beams increased with the prestressing force. The fatigue strength was found to be about 0. 67 of the ultimate static strength of the member. The ratio of the fatigue cracking moment to static ultimate cracking moment was about 0. 80.

Thomas (21) carried out static and dynamic studies on prestressed concrete ties. He stressed the need of good bond stress in resisting dynamic loading.

Abeles (3-4) reported fatigue investigations on partially prestressed concrete members in collaboration with Campus at Liege. The beams consisted of two inverted T-beams, each prestressed with eighteen 0. 2 in. diameter wires per beam with an integral slab. The beams were 20 ft. long. It was concluded that cracking would not occur if fatigue loadings were within the prescribed ranges, and that cracks will close if an initially cracked beam was subjected to fatigue loadings within similar ranges. Fatigue loadings (within prescribed ranges) did not affect subsequent static loading to failure, even with several million repetitions. Within prescribed limits, fatigue loading did not affect prestressed and non-prestressed wires although if wires were unbonded the fatigue strength was reduced to the fatigue strength of the steel.

Abeles (5) also conducted studies on beams 8 in. x 12. 5 in. x 13 ft. 6 in. having 12 wires of 0. 276 in. diameter in two rows of which only the lower six wires were tensioned. Loads were applied at two points 1 ft. 9 in. on both sides of the center line. His conclusions were: (a) For a dynamic stress range of 750 psi, no cracks were formed; and (b) the secant modulus of elasticity was not appreciably reduced, even after several million cycles were applied.

Iomata Shunji (10) while reporting fatigue studies carried out on 24 prestressed concrete beams with pre-tensioned wires proved that when the resultant stress at the bottom fiber due to full working load did not exceed 35 kg. per sq. cm. ; (1) freedom from cracks was guaranteed, (2) the factor of safety against failure due to fatigue was greater than 1. 25. Ratio of failure loads in fatigue tests to that in static tests was 44-48 percent. It was also shown that if wire of less than 3 mm. diameter was used with concrete having minimum compressive strength of 450 kg. per sq. cm., sufficient bond was secured and there was sufficient resistance to slipping under dynamic load.

Hanson (9) studied ten beams (pre-tensioned) subjected to fatigue, vibrations, and repeated impact. The beams were of 3 in. x 5 in. section, 72 and 84 in. long. Reinforcement consisted of two 0. 208 in.

wires pre-tensioned to 150,000 psi. The 28-day concrete strength was 5500 psi. Both clean and rusted wires were used. Under repeated loading all beams failed in bond. For a given loading, resistance to failure appreciably increased in tests using rusted prestressing wires. With clean prestressing wires, beams showed marked decrease in fatigue resistance with an increase in the degree of loading. It was evident that the bond failure mechanism started at the center crack and moved toward the ends. Failure took place when a peak of flexural bond stress reached the prestress transfer region. It must be noted that the beams were 72 and 84 in. long. This short length may have had an effect on the bond stress distribution.

The behavior of prestressed concrete beams subjected to repetitive loading was studied by Ozell (17) and Ardaman in 1956. By applying excessive flexural loads tensile cracks are produced in concrete. These cracks presumably cause severe stress concentrations in the concrete and in the strands contiguous to cracks rendering them vulnerable to fatigue. Tests were conducted to determine which of these stress concentrations ultimately caused the fatigue failure of the beam.

Beam specimens subjected to repetitive loads were 6 in. x 8 in. in cross-section and 19 ft. center to center of end supports. Prestressing was accomplished by two 7/16 in. seven-wire strands 2 in. from the bottom of the beam. Also one unstressed #5 reinforcing bar 19 ft. long was placed 1 in. from the top of the beam to reduce the compressive stress in the concrete and to prevent compression failure due to fatigue.

Beams were subjected to a repetitive load of 0 to 3160 lb. approximately 2.3 times the design load. The design load was taken as 1390 lb. corresponding to zero tension of the bottom fiber assuming 18 percent stress loss in strands.

The fracture of the wires occurring at points bordering the tension cracks in the beam substantiated the belief that these cracks formed stress concentrations in the strands, especially as a result of overloads, rendering them vulnerable to fatigue and ultimately caused the cracking of wires. The following conclusions were made by the authors: (a) The flexural fatigue strength of prestressed beams tested was approximately 1.8 times the design load. Overloads above this ratio had a damaging effect on the fatigue life of such beams, i.e. a load 2 times the design load caused the strand failure at 940,000 cycles. (b) Fatigue failures were caused by the breaking of the wires in the strand and not as a result of bond failure. However, other loading conditions causing higher shear may induce bond failures.

Ekberg (7) presented a method to predict the fatigue strength of prestressed concrete based on the failure envelope of the materials involved. He proved that dynamic ultimate moment was always less than static ultimate moment and could vary over a large range. The ratio of the dynamic ultimate moment to static ultimate moment was increased by increasing the level of prestress, or by increasing the percentage of steel in beam. The author felt that the design of prestressed concrete members under severe fatigue loading should be based on the ultimate dynamic moment. In addition, the ultimate dynamic moment should be equal to or greater than a constant K times the sum of the dead load, live load, and impact moments, where K is a load factor.

Nordby (16) and Venuti made fatigue investigations at various load ranges on 27 beams cast from conventional and expanded shale aggregate concrete. The beams were subjected to a fatigue load ranging from 30 to 70 percent of the ultimate load for various numbers of cycles of load. The static strength of the fatigued beams was not impaired by one or two million cycles of the design load even when severely cracked. Steel fatigue failures occurred in three beams while other 24 beams performed satisfactorily under fatigue loading. These three beams were severely cracked during the repetitive loading and failure was attributed to stress concentrations and abration between the strands and the concrete. There was no difference in the fatigue performance of either concrete used. No bond failures were found due to fatigue; in fact, in beams statically preloaded so that slight slip of strands had occurred, additional fatigue cycles did not cause additional damage. He found that strand was superior to smooth wire because its spiral shape gave high mechanical bond.

Rowe (20) concluded that for loadings above the design load that the governing factor affecting the fatigue strength of the member was the fatigue strength of the high tensile steel. His tabulation of fatigue strengths for high tensile strength wires showed that for most types a mean value of 60-65 percent of the ultimate strength should be used.

Nordby (15) while reviewing the fatigue of prestressed concrete members summarized the work as follows:

a. In none of the tests concrete failed by fatigue.

b. Fatigue failure of prestressing wires or strands was the cause of all failures reported.

c. Bond failures were rare and were found only under unusual circumstances, i. e. short beams, short shear span.

d. The ultimate strength of prestressing beams for static loads was unaffected by repetitive loading if they did not fail by fatigue.

e. Safety factor seemed to be approximately 2 against fatigue failure.

f. Prestressed beams seemed superior to conventional beams for resisting fatigue loading.

Ozell and Diniz (18) made a continuation of the fatigue investigations previously conducted on 7/16 in. strands. Six beams pretensioned with two 1/2 in. strands each were tested. Beam dimensions were 8 in. x 10 in. x 19 ft. The study indicated that the use of 1/2 in. strands was feasible and that the flexural fatigue strength of the beams was about twice the design strength.

III. SCOPE OF INVESTIGATION

The previous investigations as cited in the review of current literature raise certain questions regarding the fatigue characteristics of pre-tensioned prestressed concrete members. These questions are as follows:

1. What is the fatigue strength of concrete under prestress if the prestressing load remains constant and stress in the prestressing wires is not altered by the dynamic loading?
2. Does slippage of the prestressing wires occur in short members during fabrication?
3. Does fatigue of concrete occur under a range of flexural compressive stress?
4. What is the ratio of fatigue strength of prestressed concrete members under reversal of stress to the ultimate static strength?

Answers to these questions require an experimental investigation of prestressed concrete members subjected to dynamic loading. Thirty-three specimens three inches by three inches in cross-section and fourteen and a half inches long were prestressed with two wires 0.148 inches in diameter. These wires were placed at the neutral axis of the members and tensioned to two different levels of prestress. Twenty-two members were subjected to dynamic loading and eleven were loaded to destruction by static means.

By prestressing the members at the neutral axis the prestressing force under dynamic loading will not be varied by the loading, thus, variation in the stress on a cross-section of the member will be confined to concrete stresses. By subjecting members to dynamic reversal of loading we can obtain the fatigue strength of the concrete.

Wire slippage can be obtained by fixing strain gages along the length of the member at the neutral axis and noting the changes in strain.

The fatigue effect of concrete under flexural compressive stress can be studied by confining the dynamic load to a range which will keep the entire section of the member in compression.

The members can be dynamically loaded at a certain percentage of the ultimate static strength. This will determine the critical ratio of fatigue strength of prestressed concrete members to the ultimate static strength under reversal of loading conditions.

IV. EXPERIMENTAL PROGRAM

Notations

The following notations are used throughout the presentation and analysis of results.

ϵ = compressive strain in concrete

Fi = pre-tension load

Ac = concrete area

As = steel area

Es = elastic modulus of wire

Ec = elastic modulus of concrete

fc = compressive stress in concrete

f'c = cylinder strength

Cc = creep coefficient in concrete

δs = unit shrinkage strain

σ = flexural (reversal) stress in specimen

N = number of cycles to failure

S = stress level ratio, ratio of maximum dynamic fiber stress to effective prestress

F = stress level ratio of maximum dynamic fiber stress to static ultimate flexural stress

r = correlation coefficient

Specimens

The test specimens were pre-tensioned prestressed concrete beams of size 3 in. x 3 in. x 14 1/2 in. long as shown in Fig. 2.

The prestressing steel consisted of two wires each 0.1483 in. in diameter placed at the center of the beam. Three beams were cast simultaneously from a single batch of concrete.

Materials

Prestressing steel. Three specimens of wire (10 in. gauge length) were tested in the testing machine. A-12 SR-4 strain gauges were attached to the specimen in order to calibrate the gauge readings at various loads.

Two levels of prestress = 128,000 psi; 151,000 psi

Nominal diameter = 0.1483 in.

Nominal steel area = 0.0172 $in.^2$

Minimum breaking strength on 10" gauge length = 4540 lb.

Minimum breaking strength = 263,300 psi.

Typical modulus of elasticity = 29,000,000 psi.

Concrete. Natural sand and gravel having a maximum size of 1/2 in. were used as the fine and coarse aggregates, respectively. The fineness modulus of sand was 2.72 and that of coarse aggregate was 5.90. Mix proportion was 1:2.68:2.72, the water-cement ratio was 0.439, and the cement content was 6.5 sacks per cu. yd. Type I cement was used. The concrete was mixed in a rotary drum-type mixer for a period of 2 1/2 minutes and placed in wooden molds in two layers; each layer was compacted by rodding. Wet curing was continued for 7 days and then moist room curing was continued up to the testing period. The average

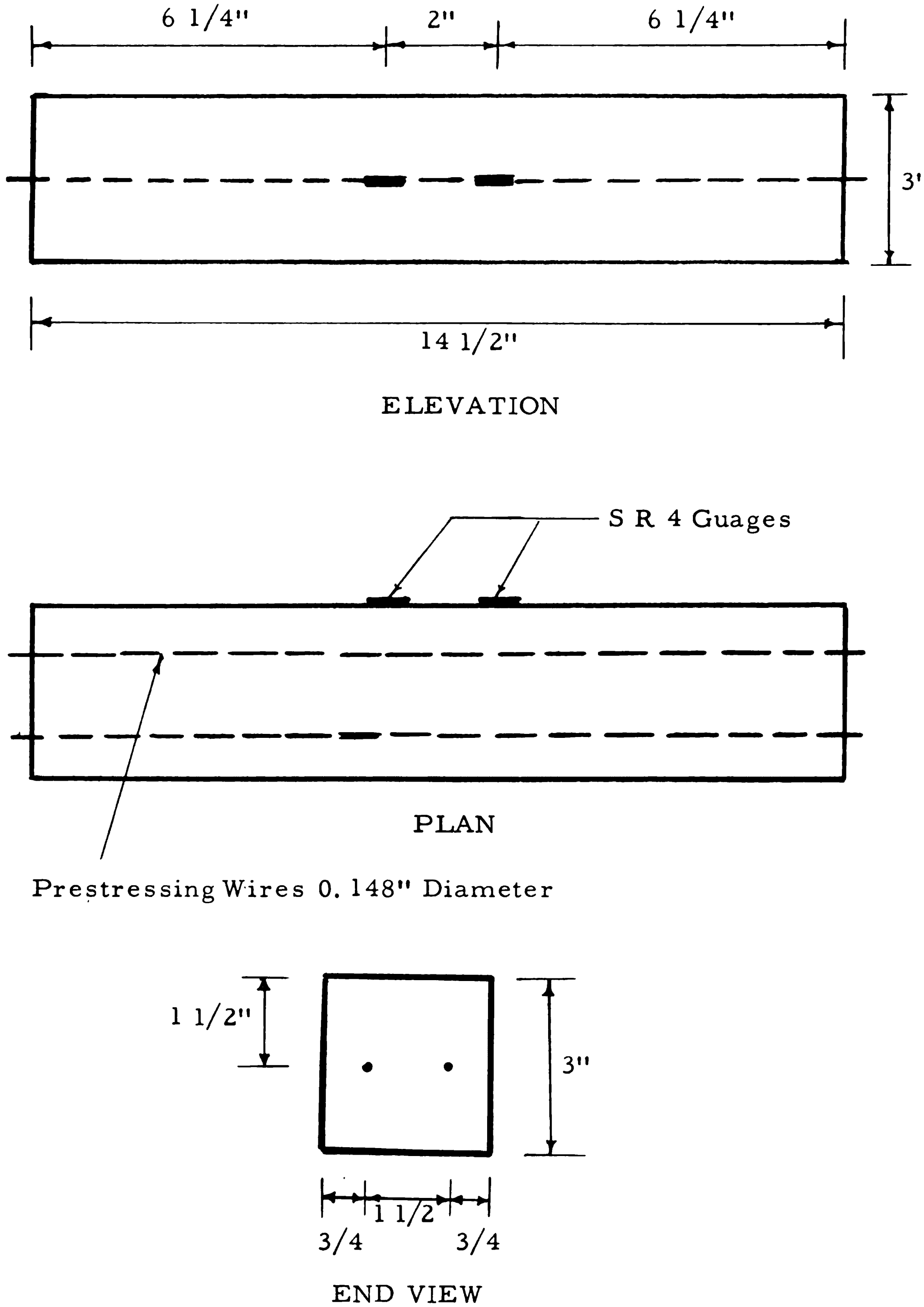


Fig. 2. Sketch of specimen.

strength of concrete cylinders (8" x 4" diameter) was 4200 psi at release of wires; and 5600 psi at 28 days.

Prestress Losses

a. Loss due to elastic shortening.

Ec = 1, 800, 000 + 500 times the cylinder strength at the age considered, as recommended by ACI-ASCE joint committee 323 (2).

$= 1,800,000 + 500\ (4200)$

$= 3.9 \times 10^{6}$ psi.

The average value of sonic modulus at 7 days was 3.85×10^{6} psi.

Es = modulus of elasticity of steel

$= 29 \times 10^{6}$ psi

$$\eta = \frac{Es}{Ec} = \frac{29 \times 10^{6}}{3.9 \times 10^{6}} = 7.44$$

fi = initial prestress = 151, 000 psi.

Using elastic theory,

$$fc = \frac{Fi}{Ac + \eta \cdot As} = \frac{Fi}{Ag + (\eta - 1)As}$$

$$= \frac{151,000 \times 2 \times 0.0172}{9 + (7.44 - 1)\ 0.0344}$$

$= 564$ psi.

Loss of prestress in steel $= \eta \times fc$

$= 4176$ psi

$$\%\ \text{loss} = \frac{4176}{151,000} = 2.72.$$

b. <u>Shrinkage loss</u>. Average value of unit shrinkage strain, as recommended by ACI-ASCE joint committee 323 (2) and Lin (12), is 0.0003.

Loss of prestress in steel = $\delta s \cdot Es$

$$= 0.0003 \times 29,000,000$$

$$= 8700 \text{ psi}$$

For an initial steel stress of 151,000 psi, % loss = $\frac{8700}{151,000}$ = 5.77.

c. <u>Creep loss</u>. (1) Concrete creep: A maximum prestress of 1000 psi in the concrete under loading condition was assumed.

$$Ec = 1,800,000 + 500 f'c$$

$$Ec = 1,800,000 + 500 (5600)$$

$$= 4,600,000 \text{ psi}$$

The average value of sonic modulus at 28 days was 5,000,000 psi.

Cc = creep coefficient

= 2.5 (adopted value) — ACI-ASCE Committee 323 (2).

Δfs = loss of prestress in steel due to creep in concrete

$$= (Cc-1)fc \cdot \frac{Es}{Ec}$$

$$= (2.5 - 1) \times 1000 \times \frac{29 \times 10^6}{4.6 \times 10^6}$$

$$= 9450 \text{ psi}$$

$$\% \text{ loss} = \frac{9450}{151,000} = 6.25.$$

(2) Creep in steel: Three percent loss of prestress in steel due to creep in steel was assumed. Thus, total losses were as follows:

Loss due to elastic shortening	=	2.72%
Loss due to creep of concrete	=	6.25%
Loss due to creep of steel	=	3.00%
Loss due to shrinkage of concrete	=	5.77%
Total		17.74%

Total prestress losses were assumed to be equal to 18 percent.

Instrumentation

The general set up is shown in Fig. 3. The prestressing force was applied by measuring elongation and checking jack pressure on a calibrated gauge. A-12 SR-4 gauges were attached to each wire to check the same stress induced in each wire. Small adjustment, if needed, was accomplished by adjusting bolts provided for this purpose.

Two SR-4 strain gauges were placed along the side of each specimen (Figs. 2 and 4) after moist curing was completed. Just before the pre-tension was released, readings were taken on all gauges on the concrete surface. The prestressing jack was released , and again complete strain readings were taken. These readings established the pre-tension in steel just prior to release, and the tension retained in the steel at the center of the specimen after release. At any point the total concrete compression must be equal to steel tension at that point. The procedure was believed to be reliable except within a very small region near the ends of the specimen. If there was no slip between the wire and the surrounding concrete, the reduction in steel strain from its initial pre-tensioned value should be the same as the increase in concrete strain resulting from the release of stress, and the corresponding

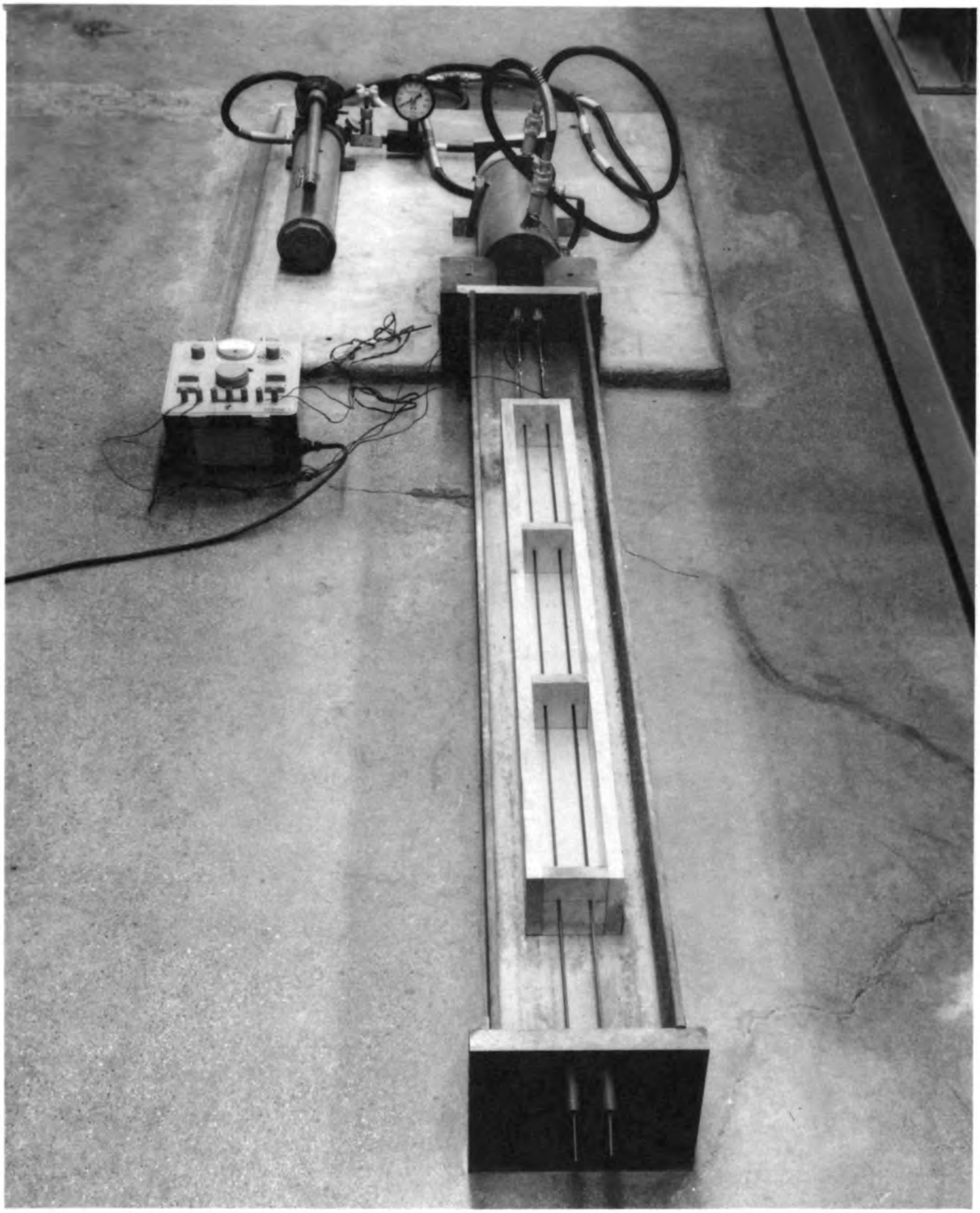


Fig. 3. General arrangement for **prestressing** the wires.

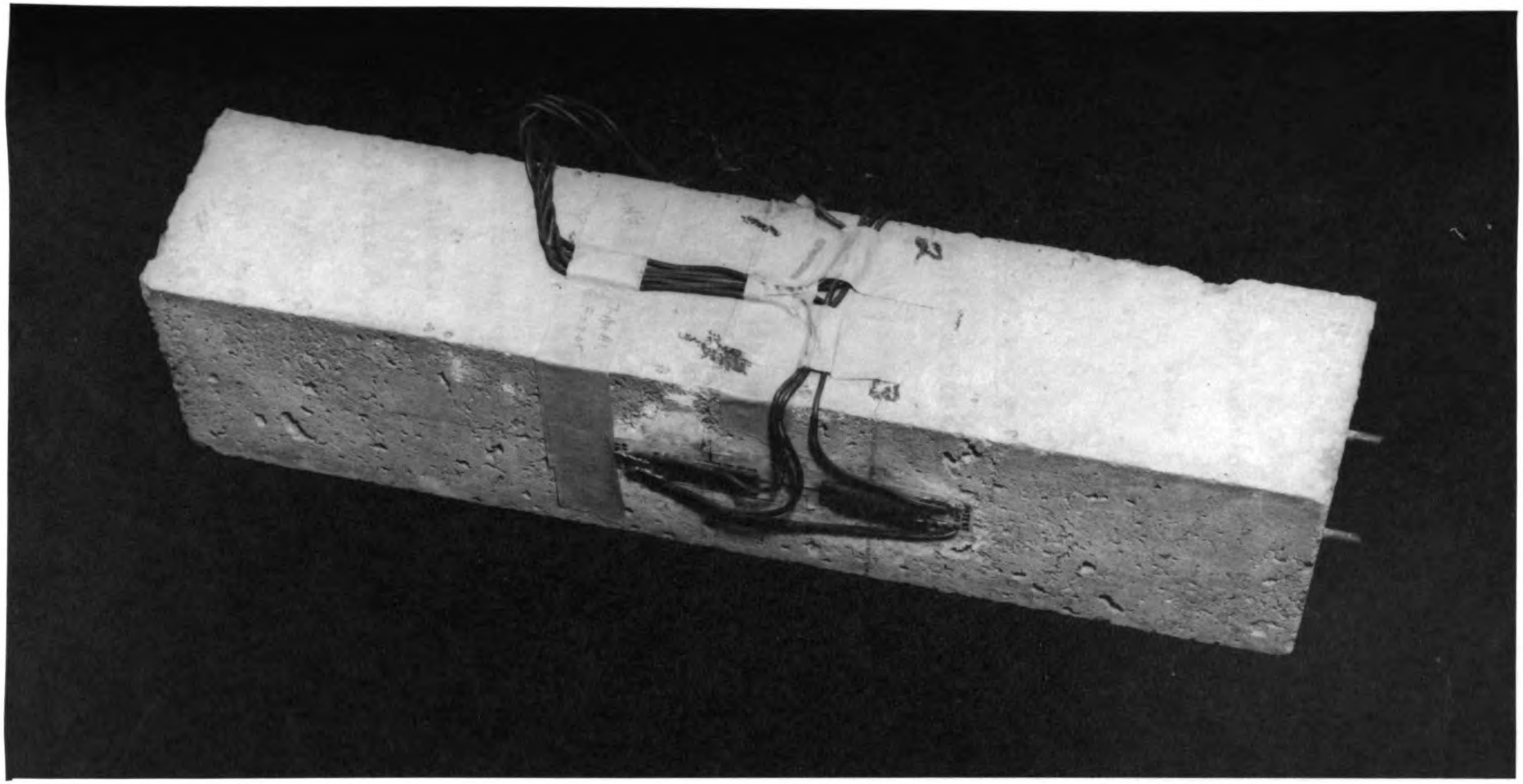


Fig. 4. Typical specimen with S. R. 4 gages to measure concrete strain at the release of prestressing force.

reduction in the steel stress would be equal to ϵEs. Compressive strain in concrete resulting from full transfer of prestress was

$$\epsilon = \frac{Fi}{As \cdot Es + Ac \cdot Ec}$$

Comparison between the assumed loss of prestress at transfer and the actual loss is shown in Table 1.

Equipment

Bending fixtures (Fig. 5). The specimen A was fastened in a horizontal position between the two pairs of grips N-O. Grips O were pivoted on two parallel axes, on needle bearing pivots held in arms, P, Q, R, S; arms P, S, being held by the stationary platen and arms Q, R, being held on vibrating platen. Arms P, Q, S, were pivoted at two points each to provide freedom in the direction of the specimen axis, thus minimizing the tensile stresses in the specimen. Arms R had only one pivot. Arms P had rubber blocks under them which prevented the grip from falling when the specimen separated. Arms R, Q, transmitted the vibratory force from the reciprocating platen F to grips, and arms P, S, resisted this force. The bending moment thus produced in the specimen was the product of the force on arms "Q" (or R), and the leverage which was the distance between pivots of Q, and P (or R and S).

Fig. 6 shows a vertical view of the internal mechanism of this machine. The function of this machine was to apply a vertical vibratory force to any specimen or structure fastened between the top

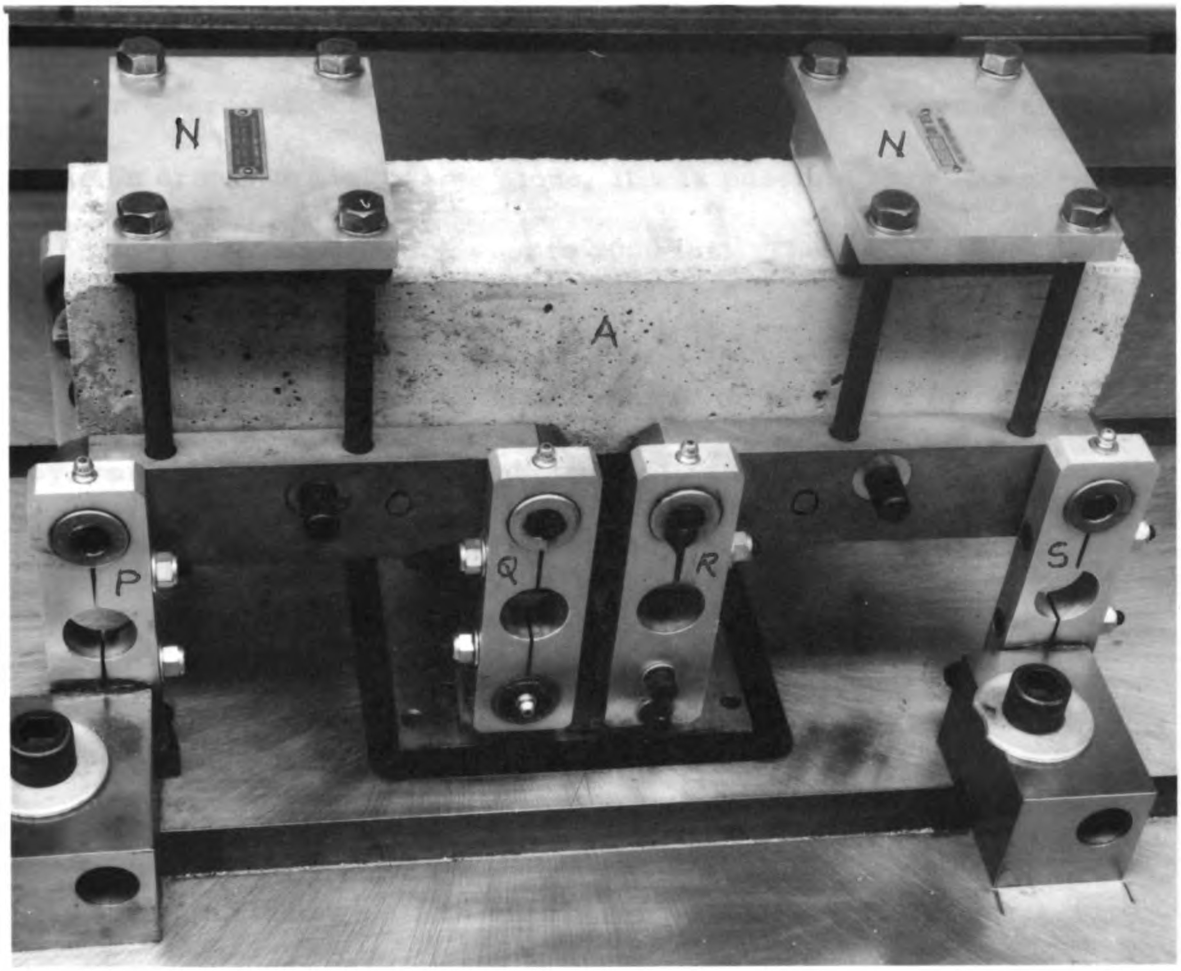


Fig. 5. Bending fixture for Sonntag SF-1-U Fatigue Testing Machine

A	Specimen
N-O	Pair of grips
P-S	Arms held on stationary platen
Q-R	Arms held on vibrating platen

plate and the vibrating cage (F). This force in the specimen could have any static component from zero to 1000 lbs. in tension or compression and an alternating component from zero to 1000 lbs. when operating in tension or compression side alone, it was possible to have a maximum vibratory fluctuating from zero to 2000 lbs. The force could also be increased by the use of amplifying fixture.

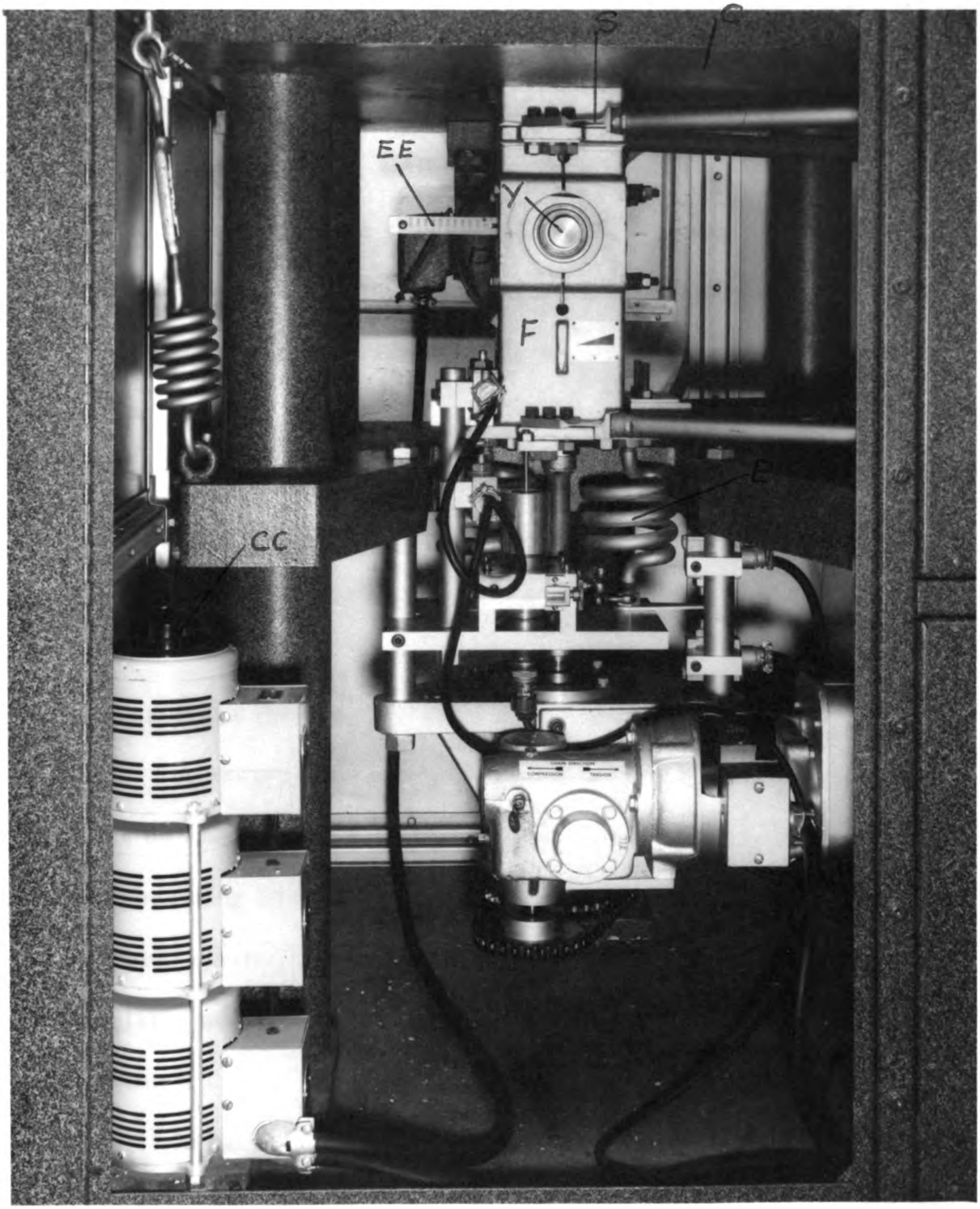
Dynamic force. The dynamic or vibratory force generated by a mechanical oscillator and applied to an elastic specimen or structure, was completely reversed and sinusoidal. It was produced by rotating an unbalanced mass (D). The shaft, through which the eccentric was threated, rotated in the oscillar housing in two ball bearings.

The oscillator shaft was driven by a synchronous motor at 1800 rpm. through a flexible shaft assembly. The eccentric was threated at one end to enable adjustment of its unbalance. Scale EE mounted on the oscillator read directly in pounds.

The vertical component of dynamic force was the only component transmitted to the specimen. The horizontal component was absorbed by four Flexplates (S) which guided the oscillator assembly in the vertical direction. The fixed ends of the flexplates were held to the heavy welded frame which was suspended from the cabinet by soft springs to prevent transmission of vibrations to the floor. Springs (E) which were fastened between the lower end of the oscillator and the frame were designed to absorb all inertia forced produced by the

Fig. 6. A vertical view of the internal mechanism of the SF-1-U Fatigue Testing Machine.

C	the stationary top plate
F	the vibrating cage
D	the eccentric
Y	knob to adjust the position of eccentric
EE	scale to indicate dynamic force
S	flexplates to absorb horizontal component of dynamic load
E	springs to absorb all inertia forces
CC	a variable transformer


S
C
EE
Y
F
E
CC

vertical vibration of the oscillator housing and all other masses attached to and vibrating with it. Thus, dynamic force induced in the specimen was equal to the eccentric setting, and remained so, irrespective of the rigidity of the specimen or the amplitude of vibration. If the rigidity of the specimen changed during the test, then the amplitude of vibration would change too, thereby maintaining a constant repeated force in the specimen.

The main motor was a synchronous motor, 3/4 h. p., 1800 rpm, 220/440 volts, 60 cycles, 3 phase. A variable transformer (C-C) was supplied to enable the operator to control the accelerating period of the main motor. Uncontrolled acceleration of the motor would overstress the specimen during starting and possibly ruin the test.

A re-set type counter located on the control panel, registered the number of repeated load cycles applied to the specimen, one unit representing 1000 cycles. It was driven by small synchronous motor which started and stopped automatically with the main motor, thus registering the total number of load cycles applied at the failure of the specimen.

In all tests, the machine was stopped automatically whenever a specimen failed, and the load cycles were recorded by the counter.

Dynamic Load Setting

Six inch lever arm fixture was used for fatigue tests. Specimen was 3 in. x 3 in. x 14 1/2 in. long.

$$\sigma = \frac{MC}{I}$$

$$= \frac{P}{2} \times 6 \times \frac{3}{2} \times \frac{12}{3 \times 27}$$

$$= \frac{6P}{9} = \frac{2}{3}P$$

Therefore, $P = \frac{3}{2}\sigma$ lb. vibratory force required to produce reversal of stress σ psi in the specimen. The eccentric weight is adjusted for P lbs.

V. EXPERIMENTAL RESULTS

Series 1

All specimens utilized in this series had aged a minimum of 28 days; any gain in strength attained during the course of the fatigue tests is therefore assumed to be negligible.

The average flexural stress for beams tested under static loadings was 827 psi as indicated in Table IV(a). For all beams of this series uniform prestress over the entire section was 400 psi, assuming 18 percent stress loss in wires. Six beams were fatigue tested under reversal loading and the results obtained are shown in Table 2.

As per specification of prestress concrete, design load will be that load which produces zero tension at the bottom of the beam. It was observed that under the design load (reversal), the beam withstood 460, 000 cycles before failure. When the ratio S was 1. 03, it failed at 150,000 cycles. Thus increase in the ratio S showed appreciable decrease in the number of cycles to failure. When the ratio S ranged between 0. 75 to 0. 915, the beams did not fail even at 2.5 million cycles.

Run-out for all beams was set at 2. 5 million cycles; tests in which beams sustained this number of repetitions of load without failure were discontinued.

The data from this tèst series are plotted in Figs. 7 and 8; ordinates to the curve are the applied stress levels, while abscissas are the logarithms of the cycles of stress sustained by the beam prior to failure. The regression line of S upon log N or of F upon log N was not determined for this series of tests because the number of tests did not justify it.

Series II

All specimens of this series of tests had aged a minimum of 28 days; it is therefore assumed that no significant gain in strength occurred during the course of the tests.

The average static flexural stress for beams tested under third point loading was 978 psi as shown in Table 4(b). Uniform pre-stress over the entire section was 473 psi for all beams fatigue tested. Sixteen beams were tested to fatigue under reversal loadings and the data obtained from the tests of the beams in Series II are given in Table 3.

For the beam no. 9 the dynamic loading was increased in two stages; and for beams no. 10 and 11, the loadings were increased in three stages. A remarkable increase over the full loading applied at a time was observed. When the ratio S was 0.942 and below, the beams did not fail even at 2.5 million cycles.

Run-out was again taken at 2.5 million repetitions of load and the tests of beams attaining run-out without failure were discontinued.

The data from this series of tests are plotted in Figs. 9 and 10 in a manner identical to that of Figs. 7 and 8.

Linearity of the data was statistically verified by determining the level of significance of the correlation coefficient r. From these data r was significant between the two and three percent levels of significance, an adequate assurance of linearity.

The regression line of S upon log N was computed by method of least squares as shown by Dixon (6). The equation of the regression line in Fig. 10 is as follows:

$$S = 1.2929 - 0.0534 \log N$$

The flexural fatigue strength, at 2.5 million repetitions of stress, is 94.2 percent of the effective prestress of the specimens as determined from the regression line of Fig. 10.

The equation of the regression line of F upon log N as determined by a similar manner is as follows:

$$F = 0.6257 - 0.0263 \log N$$

The flexural fatigue strength, at 2.5 million repetitions of stress, is 45.5 percent of the static ultimate flexural strength of the specimens as determined from the regression line of Fig. 9.

Specimens which attained run-out and were then broken statically, as well as specimens which failed after few repetitions of load, are excluded from the computations which determine the regression line. Both exclusions are noted in the tables.

TABLE I

SPECIMENS FOR SERIES II

Comparison of Theoretical and Measured Strains at Center Section of a Beam, after Release of Wires

$Ec = 3.9 \times 10^6$ psi

$Es = 29 \times 10^6$ psi

Initial pre-tension = 151, 000 psi

Specimen number	Concrete strains in millionths		Reduction in steel stress in psi	
	Theoretical	Measured	Theoretical	Calculated by assumed relationship
1	144	160	4176	4640
2	144	155	4176	4500
3	144	140	4176	4060
4	144	158	4176	4580
5	144	152	4176	4410
6	144	150	4176	4350
7	144	163	4176	4740
8	144	139	4176	4030
9	144	135	4176	3920
	Average	150	Average	4360

TABLE 2

SPECIMENS FOR SERIES I

Initial prestress = 128,000 psi

Losses in prestress = 23,040 psi

Effective prestress = 104,960 psi

Uniform prestress over entire section = 400 psi

Specimens cast = 9

Fatigue Tests Results

Number	Uniform prestress in psi	Dynamic stress in psi	Stress ratio S	Stress ratio F	Cycles to failure in thousands
1	-400	±413	1.030	0.500	150
2[1]	-400	±300	0.750	0.363	2500
3[2]	-400	±314	0.786	0.380	2600
4[3]	-400	±366	0.915	0.443	2500
5	-400	±400	1.000	0.484	460
6	-400	±430	1.075	0.520	66

[1, 2, 3] Broken statically upon attainment of run-out at 1862, 1870, 1878 lbs.

TABLE 3

SPECIMENS FOR SERIES II

Initial prestress = 151,000 psi
Losses in prestress = 27,180 psi
Effective prestress = 123,820 psi
Uniform prestress over the entire section = 473 psi
Specimens cast = 24

Fatigue Tests Results

Number	Uniform prestress in psi	Dynamic stress in psi	Stress ratio S	Stress ratio F	Cycles to failure in thousands
1[1]	-473	±510	1.08	0.522	3
2	-473	±500	1.06	0.512	40
3	-473	±490	1.037	0.502	60
4	-473	±480	1.016	0.491	115
5	-473	±480	1.016	0.491	100
6	-473	±480	1.016	0.491	108
7	-473	±473	1.00	0.483	400
8	-473	±473	1.00	0.483	425
9[2]	-473	±473	1.00	0.483	550
10[3]	-473	±473	1.00	0.483	650
11[3]	-473	±473	1.00	0.483	680
12	-473	±460	0.975	0.471	1000
13	-473	±460	0.975	0.471	975
14	-473	±450	0.954	0.460	1800
15[4]	-473	±445	0.942	0.455	2500
16[4]	-473	±435	0.921	0.445	3500

[1] Failed after few repetitions of stress and so it did not fail in fatigue. Not included in computations.

[2] Dynamic loading was increased in two stages. Not included in computations nor shown in Figs. 9 and 10.

[3] Dynamic loading was increased in three stages. Not included in computations nor shown in Figs. 9 and 10.

[4] Broken statically at values of 2210, 2215 lbs upon attainment of run-out. Not included in computations.

TABLE 4

STATIC TESTS UNDER THIRD POINT LOADINGS
ON A SPAN OF 12 IN.

a. Specimen for Series I

Number	P in lbs.	Static flexural stress in psi
1	1850	823
2	1870	831
3	1860	827
Average	1860	827

b. Specimen for Series II

Number	P in lbs.	Static flexural stress in psi
1	2210	982
2	2200	978
3	2190	974
4	2195	976
5	2200	978
6	2208	981
7	2190	974
8	2200	978
Average	2200	978

P = total load on specimen as indicated by testing machine dial.

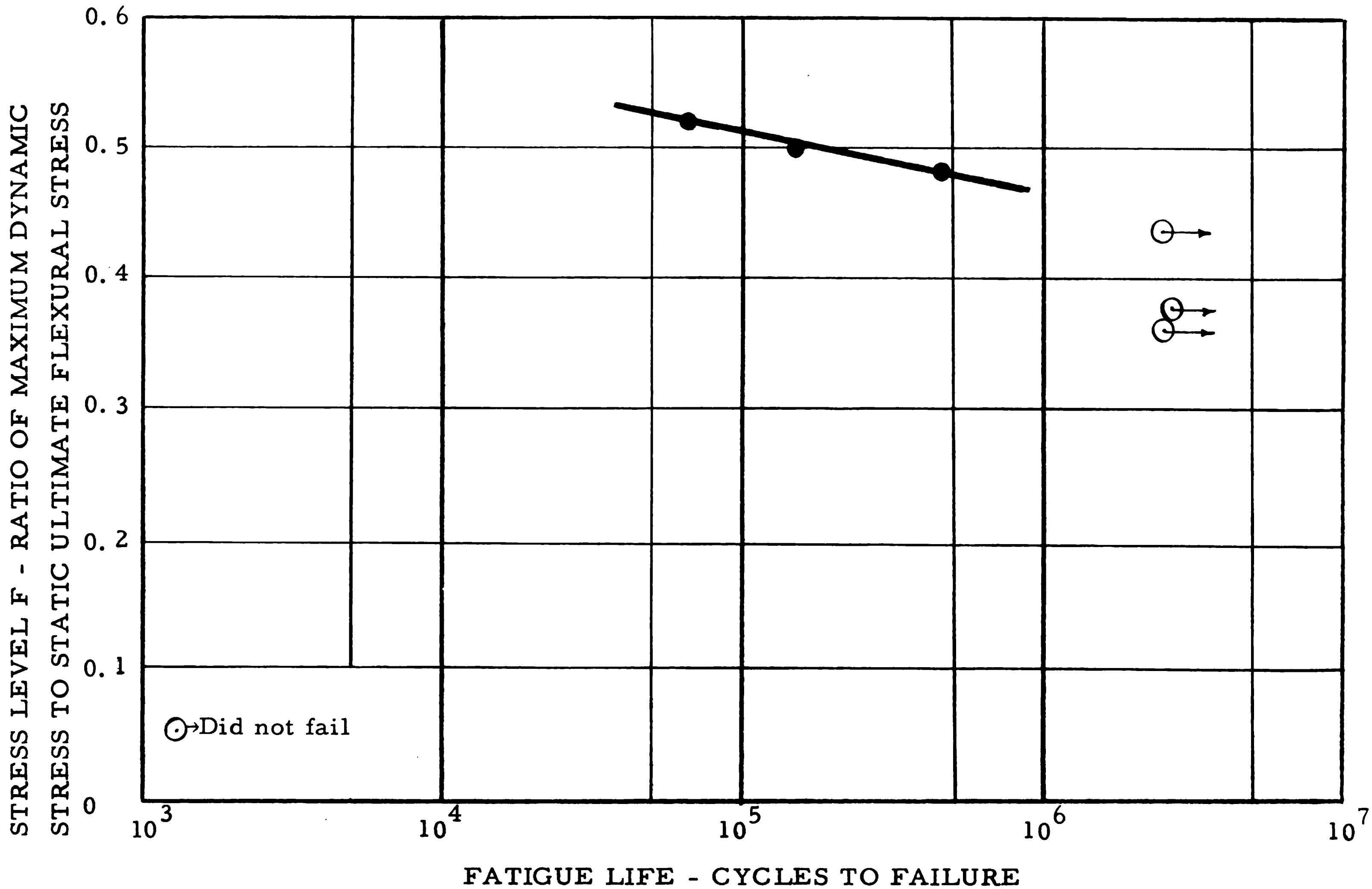


Fig. 7. Behavior of specimens under fatigue loading in Series I

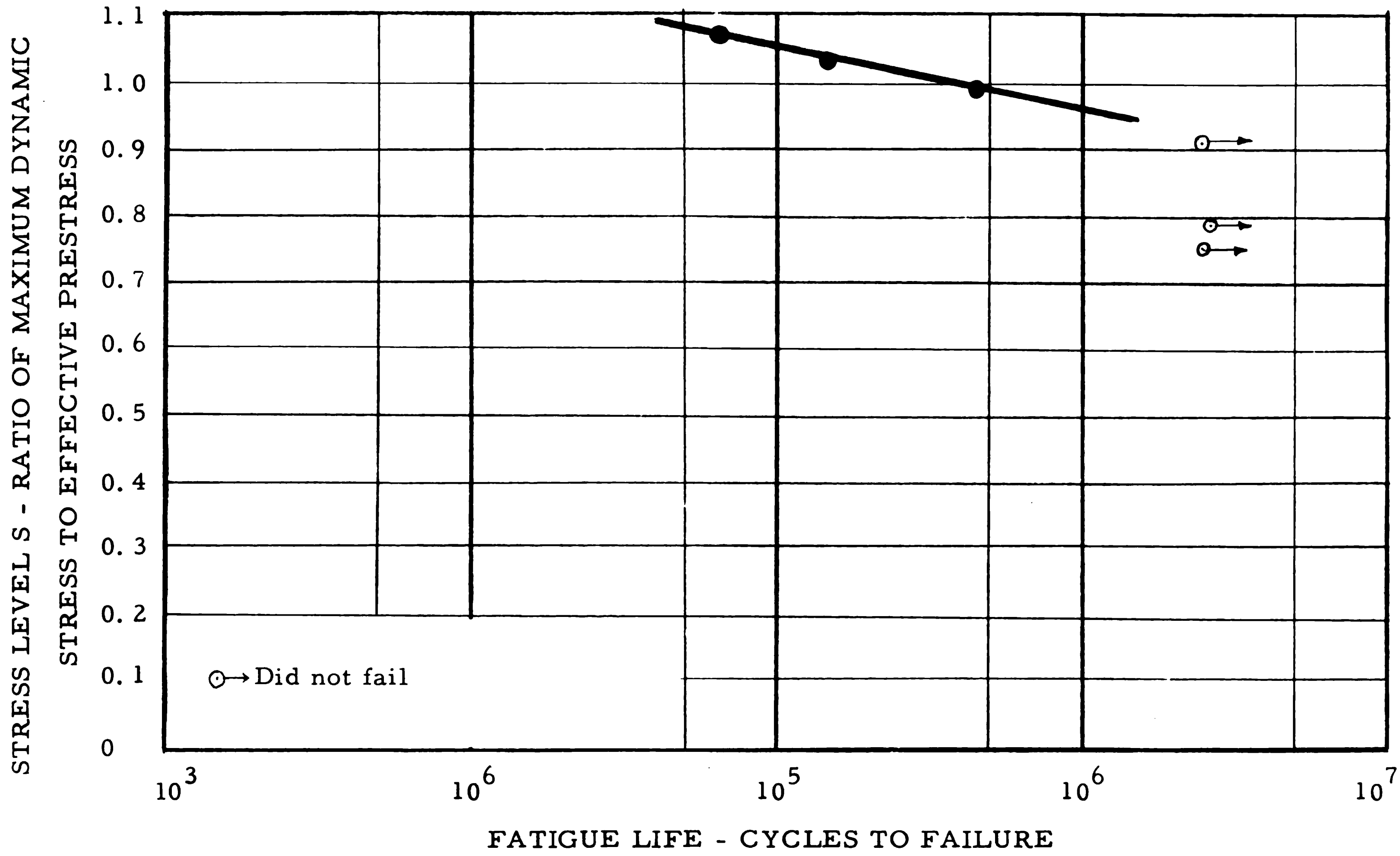


Fig. 8. Behavior of specimens under fatigue loading in Series I.

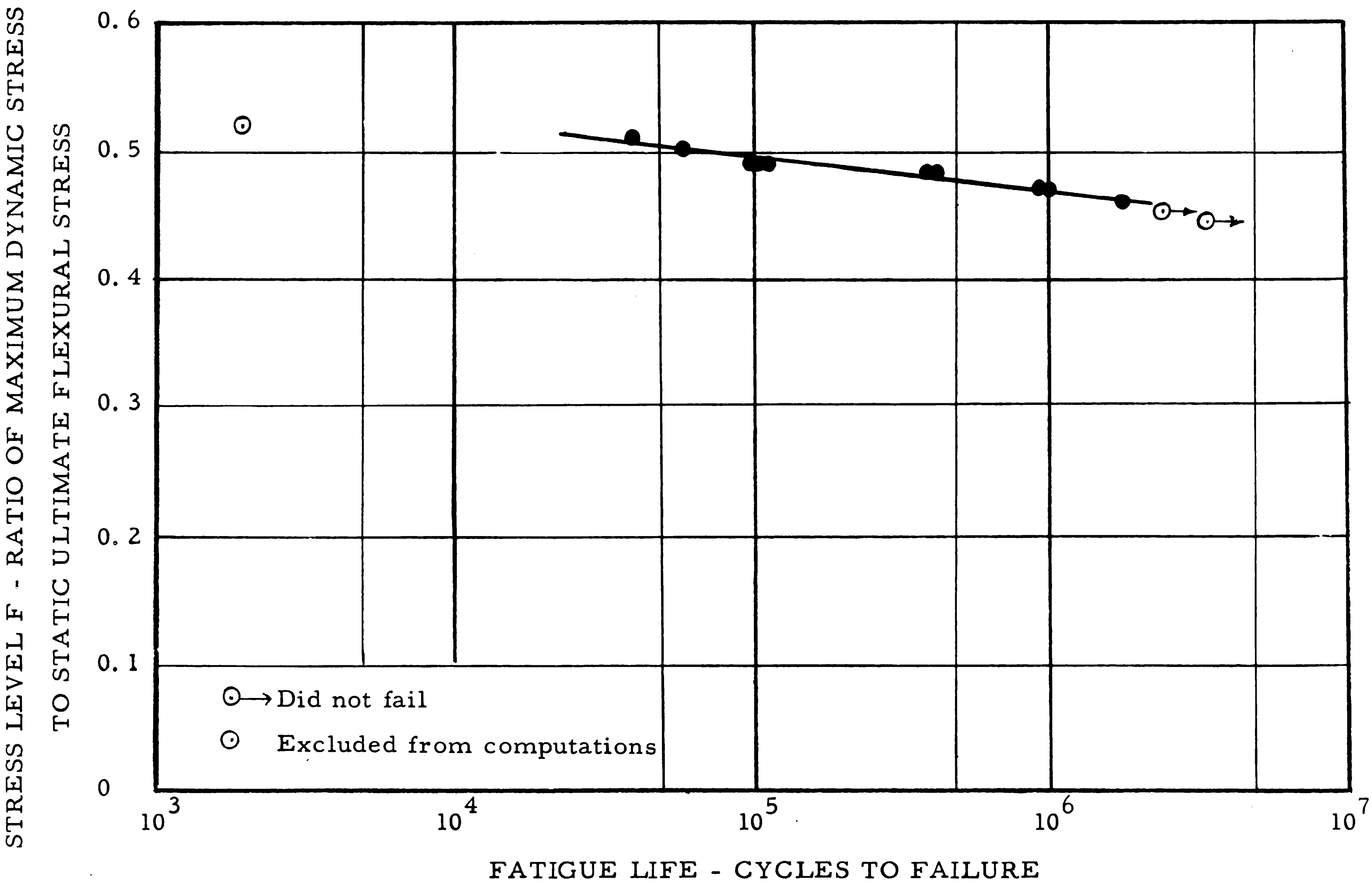


Fig. 9. Behavior of specimens under fatigue loading in Series II.

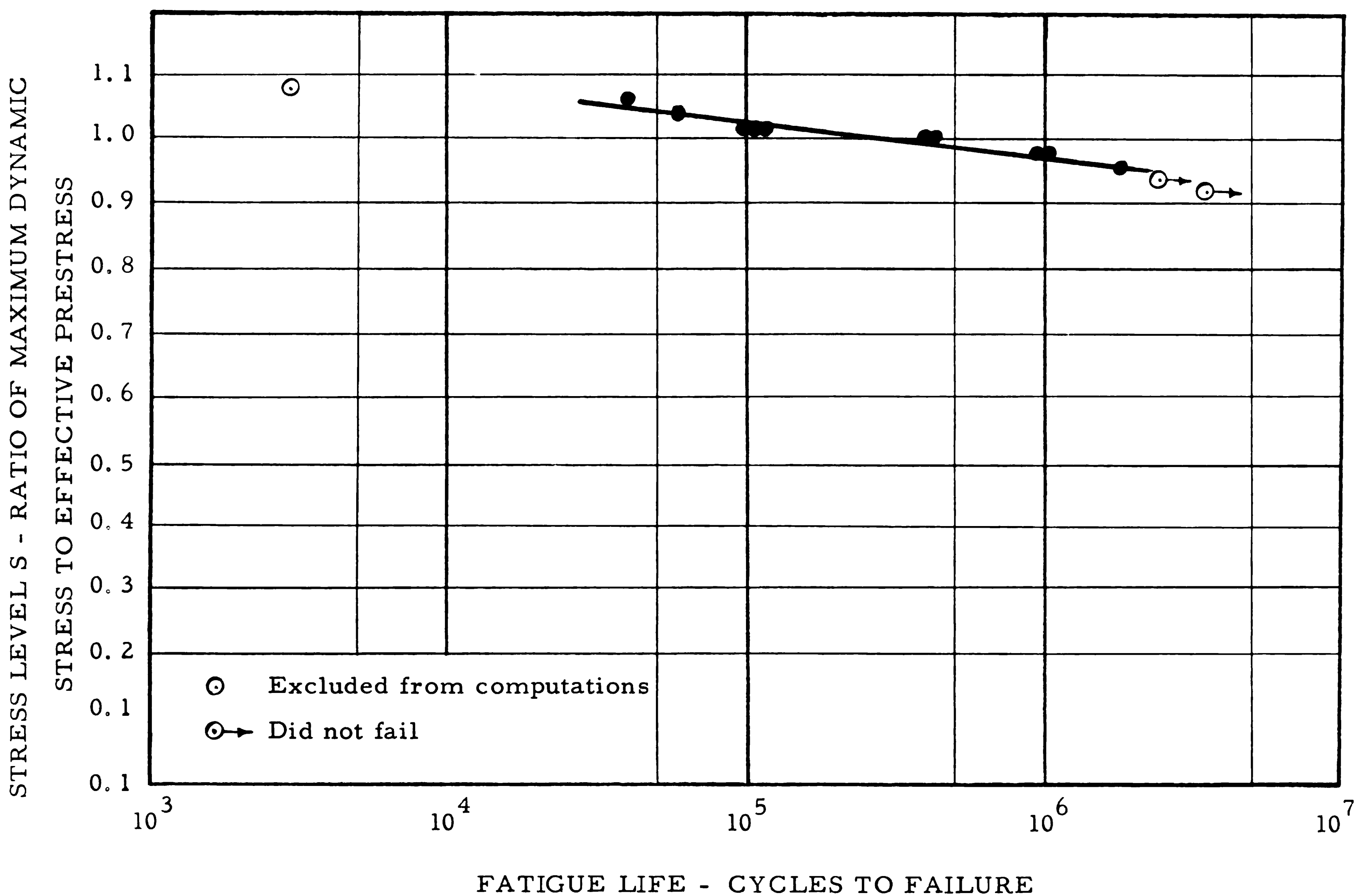


Fig. 10. Behavior of specimens under fatigue loading in Series II.

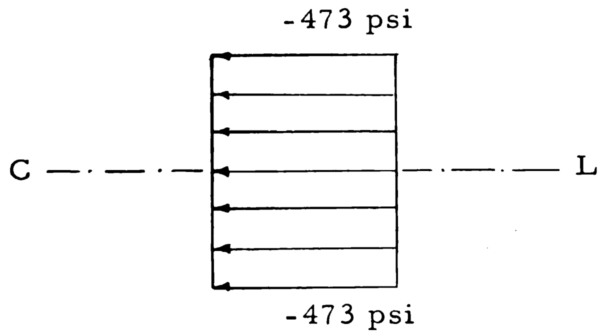
VI. DISCUSSION OF RESULTS

In Fig. 10, it was observed that the fatigue failures did not occur when the stress ratio S was 0.942 and below; and the fatigue loadings did not affect subsequent static loading to failure. These were in agreement with Abeles (3, 4, 5). Therefore, if the beams were fatigue loaded within the ratio S equal to or less than 0.942, no failures could occur under reversal of stress. Thus the fatigue strength of the pre-tensioned, prestressed concrete members with prestressing steel placed at the neutral axis and subjected to reversal of loading will be less than the design strength. Typical stress patterns are shown in Fig. 11.

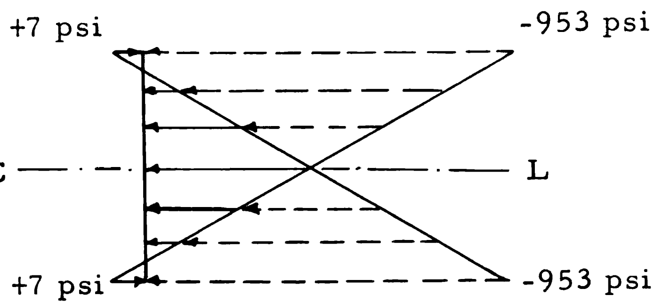
At the bottom of Table 2 and Table 3 are indicated the members which did not fail in fatigue and which were tested statically under third point loading. These members which were loaded to failure statically did not show any change in the ultimate strength due to fatigue loading. The typical fracture surfaces of beams are shown in Fig. 12.

In Fig. 9 the stress ratio F was varied from thirty-six to fifty-two percent. It was observed from computations that the ratio of failure loads in fatigue loadings to that in static ultimate loadings was approximately 45 percent. Thus if the stress ratio F varied from 0 to 45 percent, no fatigue failures could occur.

In Table 2 and Table 3 eight beams are listed which were dynamically loaded such that the entire cross-section of the beams was in compression. Since the beams were prestressed by placing the prestressing wires at the neutral axis, no variation in the prestress



Uniform Prestress

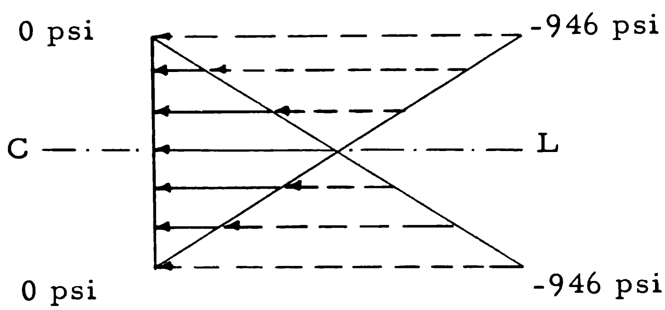


Dynamic load ± 480 psi

Stress ratio S = 1.016

Stress ratio F = 0.491

Fiber stress varies from 7 psi (tension) to 953 psi (compression)

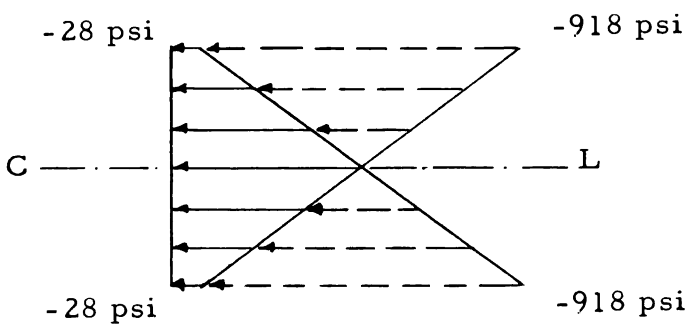


Dynamic load ± 473 psi

Stress ratio S = 1.00

Stress ratio F = 0.483

Fiber stress varies from 0 psi (tension) to 946 psi (compression)



Dynamic load ± 445 psi

Stress ratio S = 0.942

Stress ratio F = 0.455

Fiber stress varies from 28 psi (compression) to 918 psi (compression)

Fig. 11. Stress variations under dynamic load.

force could occur under reversal of loading. Thus variation in the stress on the cross-section of the beam was confined to concrete stresses. This condition led to the fatigue failure of concrete in flexural compression and such failure was observed in three beams. It appears that concrete can fail in flexural compression.

From the results of Table 1, it was observed that slight slippage of prestressing wires could occur during the fabrication of the members. However slippage of wires did not occur during fatigue loading, thus, small wires show greater resistance to bond. This was in agreement with Iomata (10).

Failure of all beams under dynamic loading was due to fatigue failure of the concrete. Vertical cracks were observed in all beams which failed under dynamic loading; the typical fracture surfaces of the beams are shown in Figs. 13 and 14. Thus in pre-tensioned prestressed concrete using thin wires, the fatigue failure of concrete is more likely to occur than fatigue failure in the wires.

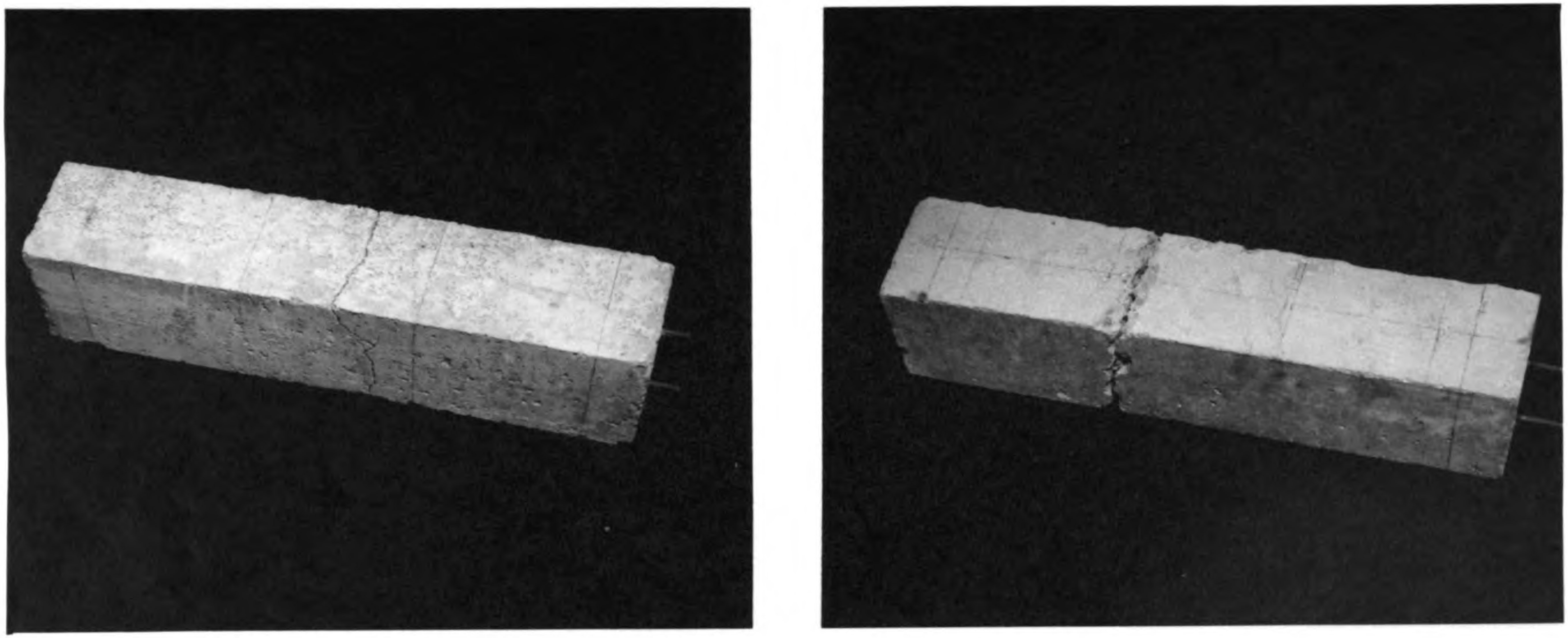


Fig. 12. Typical fracture surfaces of specimens under the static third-point loading.

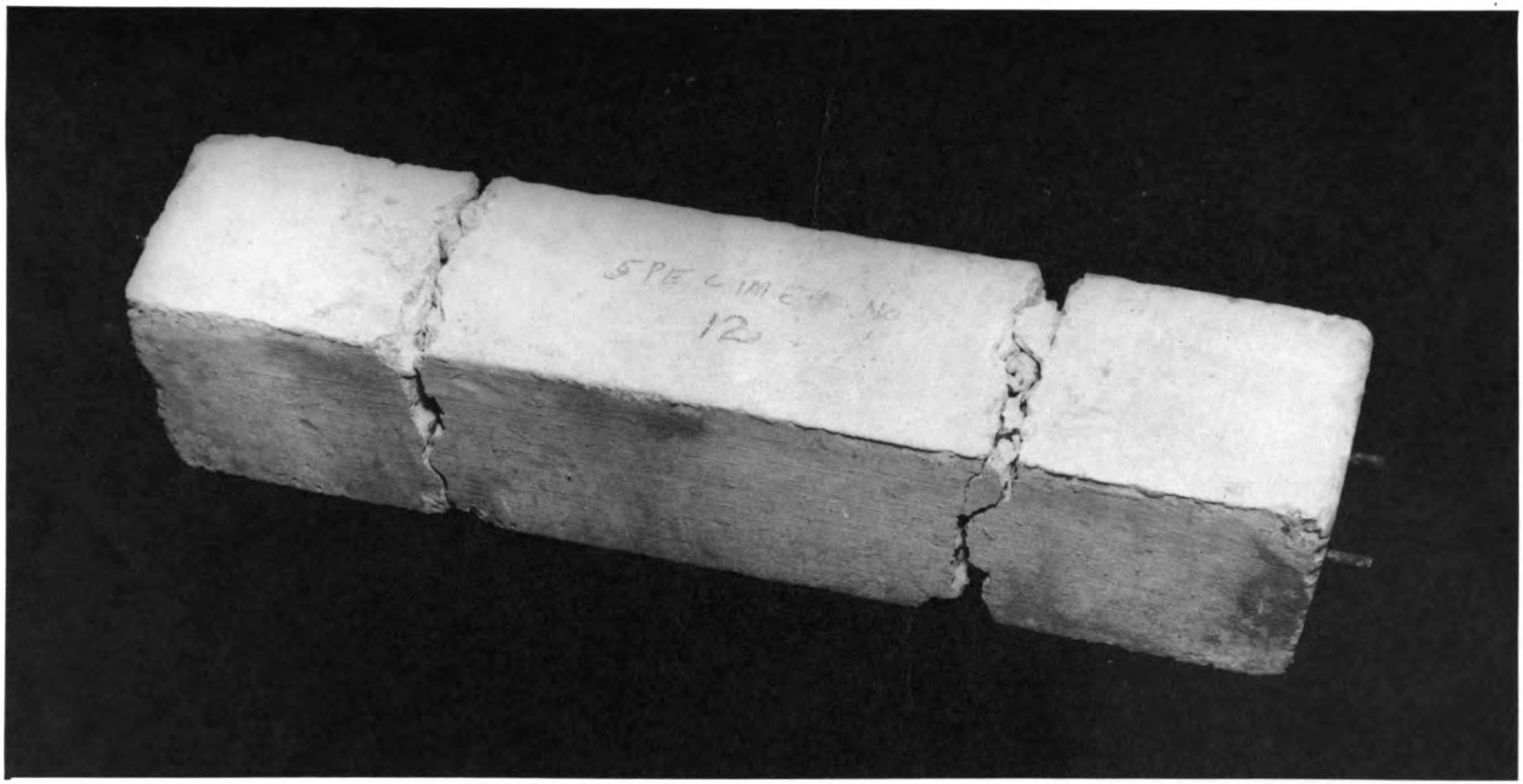
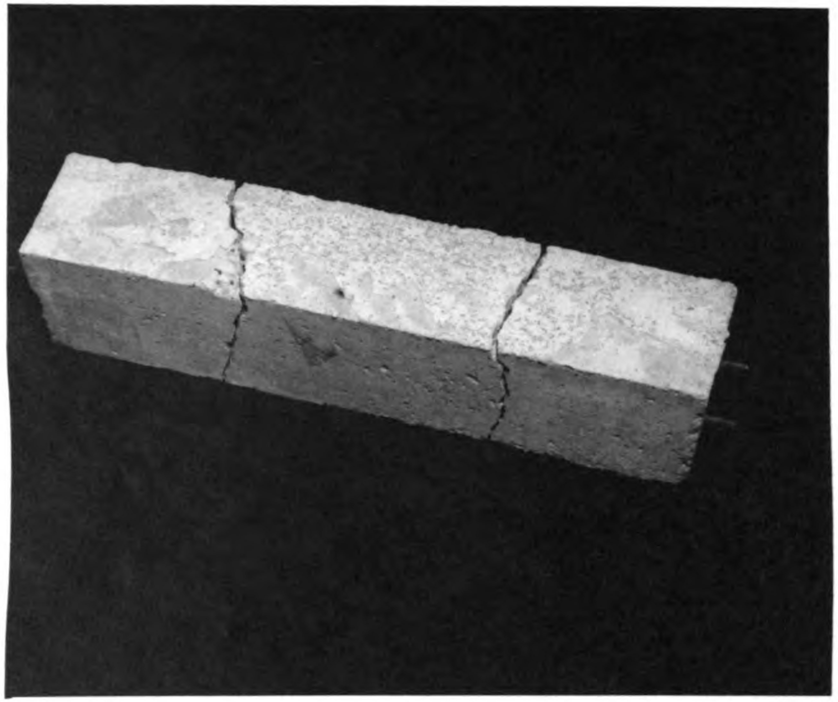


Fig. 13. Typical fracture surfaces of specimen under fatigue loading.



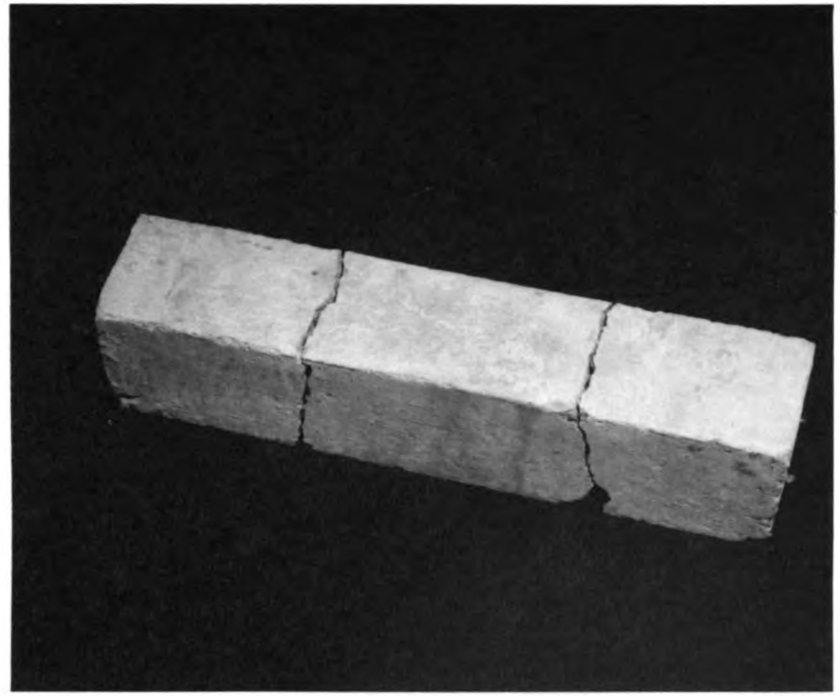


Fig. 14. Typical fracture surfaces of specimens under fatigue loading.

VII. CONCLUSIONS

From the observed results of these experiments, the following general observations are made:

a. Fatigue strength of beams under reversal of loading was approximately 94 percent of design strength.

b. The ultimate strength of prestressed beams under static loading was not affected by previous dynamic loading of 2.5 million cycles.

c. Ratio of failure loads (reversal) in fatigue loadings to that in static ultimate loadings was approximately 45 percent.

d. Under fatigue loading in which the entire cross-section of the member remained in compression, flexural compression failures occurred in the concrete.

e. Thin wire showed good bond resisting property in dynamic loading.

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