AN INVESTIGATION OF CREEP DUE TO BOND BETWEEN DEFORMED BARS AND CONCRETE

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.

PREFACE

This Research Report 113-5F is the final report on those phases of the general project "Splices and Anchorage of Reinforcing Bars," which relate to an investigation of creep due to bond or anchorage stress in pan-joist T-beams. It follows Research Report 113-4, "Shear and Anchorage Study of Reinforcement in Inverted T-Beam Bent Cap Girders," and represents the last report of Project 113. The study reported herein was initiated in order to determine whether bond creep contributed to excess deflections of the pan-joist beams after early removal of metal forms.

Research Report 113-4 included conclusions and recommendations for the design of shear reinforcement in the web of inverted T-beams and recommendations for the design of the T-beam flange to support stringer loads.

Research Report 113-3, Part 2, included conclusions and recommendations regarding splice tests with #11, #14, and #18 reinforcing bars. It superseded previous Research Report 113-2, Part 1, on Tensile Lap Splices.

Research Report 113-1, entitled "Test of Upper Anchorage #14S Column Bars in Pylon Design," by K. S. Rajagopalan and Phil M. Ferguson, published August 1968, covers another separate phase of the project completed earlier.

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ABSTRACT

In an investigation of creep due to bond stress in T-beams, eight specimens, made to represent the behavior of pan-joist T-beams, were subjected to sustained loads applied initially when specimens were 3, 4, 8, and 32 days old. Two levels of tensile stress, 12 ksi and 17 ksi, were maintained during the sustained load period. No evidence of creep due to bond was detected, but there was an indication of initial bond slip in specimens loaded at the earlier ages.

S U M M A R Y

In an investigation of creep due to bond stress in T-beams, eight reinforced concrete beams, made to represent the behavior of pan-joist T-beams, were subjected to a constant load applied to a pair of beams initially when the beams were 3, 4, 8, and 32 days old. One beam in each pair developed a 12 ksi tension stress in steel, and the other developed a tension stress of 17 ksi.

There was evidence of an initial bond softness or slip when loads were first applied to specimens only 3 or 4 days old, but thereafter there was no discernible evidence of bond slip or loss of anchorage in any beams. Specimens loaded at an early age initially displayed fewer cracks than those loaded after a 32-day curing period. In beams loaded early during the curing period, concrete was less stiff than it became later, but the smaller number of cracks tended to stiffen the beams such that beam deflections were not excessively larger than those of the beams loaded after 8 or 32 days of curing time.

IMPLEMENTATION

The influence of creep or bar slip on long-time deflections of pan-joist bridge stringers has been investigated. Sustained loads that maintained a 12 ksi tension stress in one beam and a 17 ksi stress in a second beam were applied initially to each pair of beams after the beams had cured for 3, 4, 8, and 32 days. A total of eight beams was studied. The beams loaded at the earliest age of curing revealed some bond softness or slip upon the initial application of load, but thereafter none of the beams exhibited measurable changes that could be interpreted as creep due to bond.

The transfer of stress from steel to concrete changed with time only as cracking (probably due to shrinkage) developed at a regular spacing of 6 to 12 in. apart in the flexural tension region of the beams. Beams loaded at the earlier curing ages displayed fewer cracks upon initial loading than did the 32-day-cured beams. After several months, the distribution of flexural cracks was virtually the same for all specimens.

It was concluded that excess long-time deflections of the pan-joist T-beams could not be attributed to bond creep. Therefore, no changes in bar anchorage details would be recommended as a remedy to the deflection problem. It was observed that beams less than 4 days old were only some 70 percent as stiff as beams that had cured 32 days.

C O N T E N T S

CHAPTER I

INTRODUCTION

Loads are first applied to most cast-in-place reinforced concrete beams at the instant that formwork is removed from beneath such beams. During pan-joist and similar construction techniques that repeat the use of standard formwork for a succession of identical members, definite economies can be realized from the frequent and rapid reuse of metal forms. In general, the shorter the time interval before forms can be reused, the smaller is the total number of forms necessary to complete a project.

Removal of forms too early after concrete is cast can be undesirable and even dangerous if the concrete is unable to attain adequate stiffness and strength to maintain its shape under the loads imposed when the forms are removed. Concrete gains both stiffness and strength roughly in an exponential ratio suggested by the curves of Fig. 1.[1] Early gains are very rapid, but gains apparently continue several months after concrete is cast. Concrete may appear to be strong enough for the removal of forms at a very early age, when perhaps 50 percent of its total strength has been developed through curing. Although capable of maintaining its own weight, such concrete at 50 percent of its mature strength would have reached only 70 percent of its mature stiffness.

The instantaneous stiffness or modulus of elasticity E is the physical characteristic of concrete most significant in deflection behavior. Reference here is made to instantaneous stiffness, because concrete tends to

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[1] Troxell, G. E., Davis, H. E., and Kelly, J. W., Composition and Properties of Concrete, 2nd ed., McGraw-Hill, New York, 1968, p. 243.

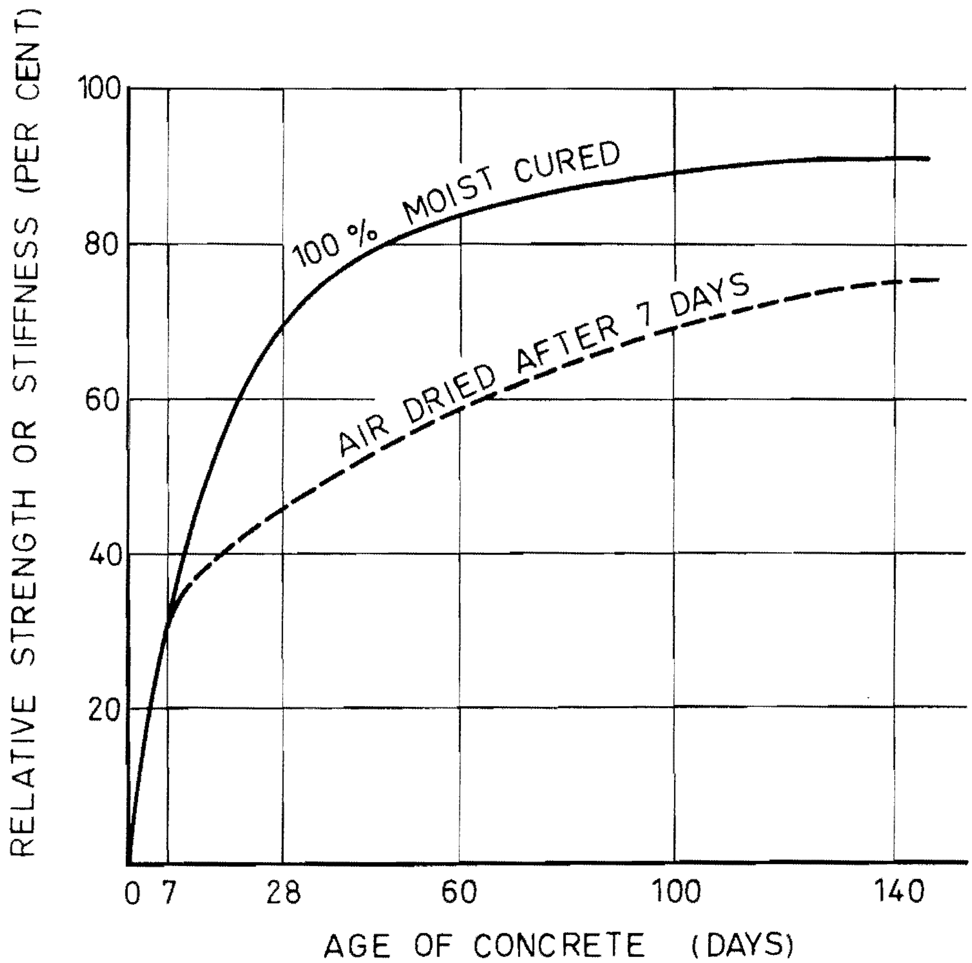

FIG. 1. INFLUENCE OF CURING TIME.

creep under constant pressure or stress. The amount of creep increases with the relative stress level and with the length of time the material has been subjected to constant stress.[2,3] The flow of a material at constant stress has the effect of reducing the apparent stiffness of the material. Creep, or flow deformation, may not be recoverable upon removal of the load that caused the creep deformation. Generally, the creep of concrete has been studied primarily in connection with compression stress, secondarily in connection with tension stress, and rarely if ever in connection with bond stress.

Even though concrete is likely to experience creep deformation when it is loaded at an early age, its strength and stiffness can continue to increase as the concrete continues to cure. Gains in strength and stiffness eventually can stabilize the tendency to creep and in fact halt further deformations due to creep.

The deflection or sag of some reinforced concrete beams was found by the Texas Highway Department to be undesirably high if forms were removed after only two or three days' curing time. The compressive stress developed from the self-weight flexure of these beams was felt to be too small to cause enough creep to account for the undesirable amount of sag. The beams contained rather large #11 and #10 reinforcing bars closely spaced at the bottom of a T-beam stem. It was felt that some portion of the undesirable sag might be attributed to high bond stress or even creep due to high bond stress in such beams.

Conceivably creep might occur due to high compressive stresses at the bearing surfaces of deformed bar lugs. The study reported herein was initiated in order to measure the amount of creep due to bond in beams made to resemble very closely the actual beams that were deflecting too much. Only two variables, the stress level in reinforcement and the age of concrete at the time of initial loading, were planned for investigation in this study. Eight specimens were constructed identical in all details. The specimens were made two at a time, and each pair of specimens cast on the same day

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[2] Concrete Manual, 7th ed., U. S. Department of the Interior, Bureau of Reclamation, 1966, pp. 29-33.

[3] Ali, Iqbal, and Kesler, Clyde E., "Mechanisms of Creep in Concrete," Symposium on Creep of Concrete, American Concrete Institute SP-9, Detroit, 1964, pp. 35-41.

were loaded at the same time in such a way that a tensile stress of 17 ksi was developed in one beam and a tensile stress of 12 ksi was developed in the other beam. Loads once applied were maintained at a constant level for more than 100 days. The eight specimens were held under the constant stress while measurements of surface strains, steel strains, and deflections were kept.

CHAPTER II

TEST PROGRAM

Specimens

The beams that had exhibited an undesirable amount of sag are described in the sketches of Fig. 2. The circular arc formed by the top of the metal pans created rather large fillets between the web and flange of the T-beam. Consequently, there appeared to be an abundance of concrete above the neutral axis to keep flexural stresses low. Generally, four reinforcing bars were placed in the bottom of the 8-1/2 in. wide T-beam stem. The amount of concrete surrounding each bar is normal for T-beam stems. The ratio A/A_s between the area of tensile concrete and the area of tensile reinforcement for a stem containing two #11 and two #9 bars would be

$$A/A_s = \frac{2 \times 3.35 \times 8.5}{2(1.56 + 1.0)} = 11.1 \qquad (1)$$

The ratio expressed by Eq. 1 is the same ratio used in Eq. (10-2) of the 1971 Building Code of the American Concrete Institute as an index for the control of cracking.[4]

The cross section of a test specimen made to represent the cast-in-place T-beam shown in Fig. 2 should possess approximately the same ratio A/A_s and develop approximately the same amount of compression stress as that of the actual beam. The test specimen need not necessarily contain the same amount of steel nor even the same actual depth as the actual bridge beam. Properties of the cross section of a specimen selected for testing are compared with properties of the bridge cross section in Fig. 3. A comparison of stress profiles can be derived from Fig. 3b, which displays a

[4] Building Code Requirements for Reinforced Concrete (ACI 318-71), American Concrete Institute, Detroit, 1971, p. 31.

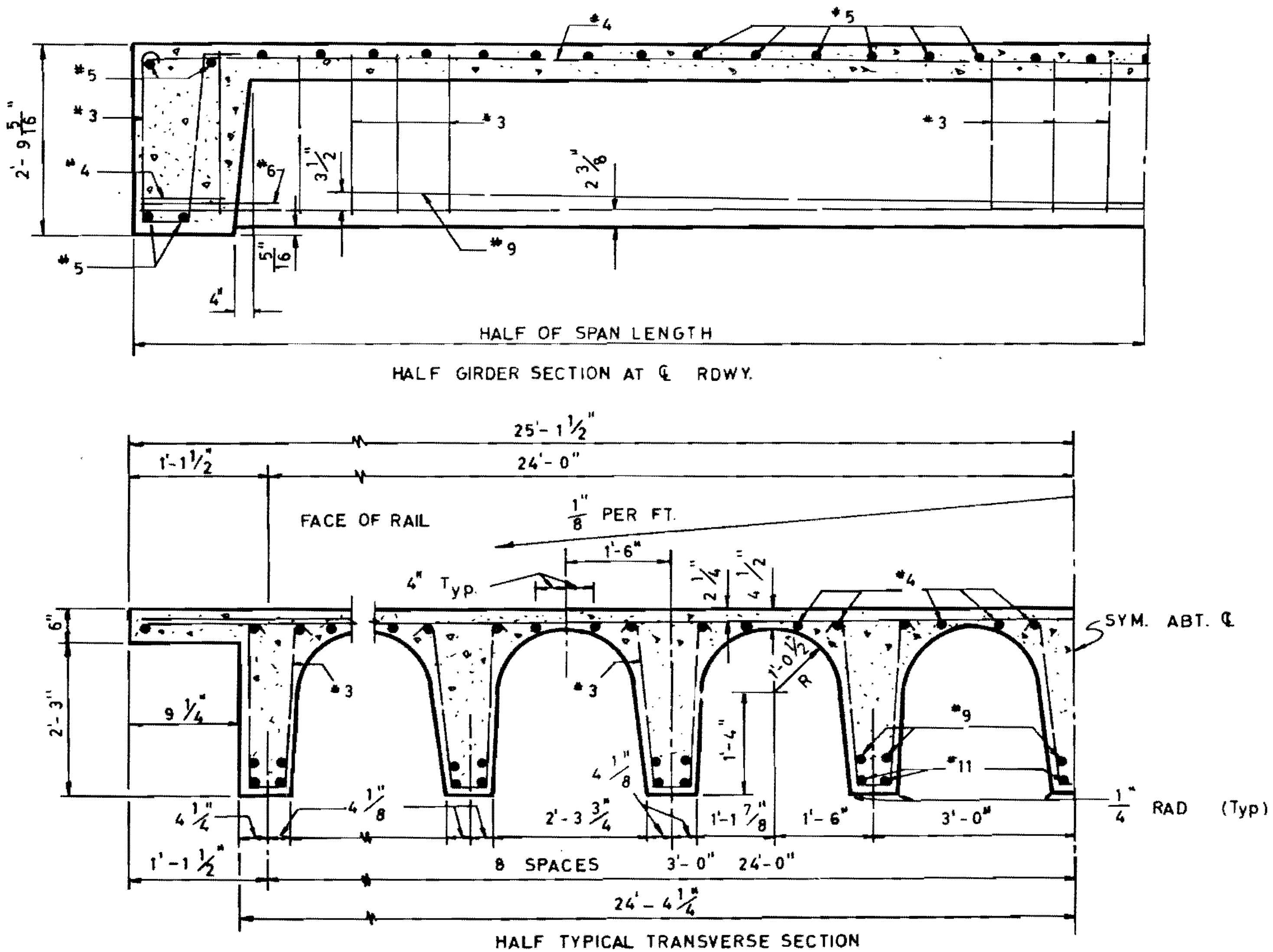


FIG. 2 CONCRETE PAN T-BEAMS

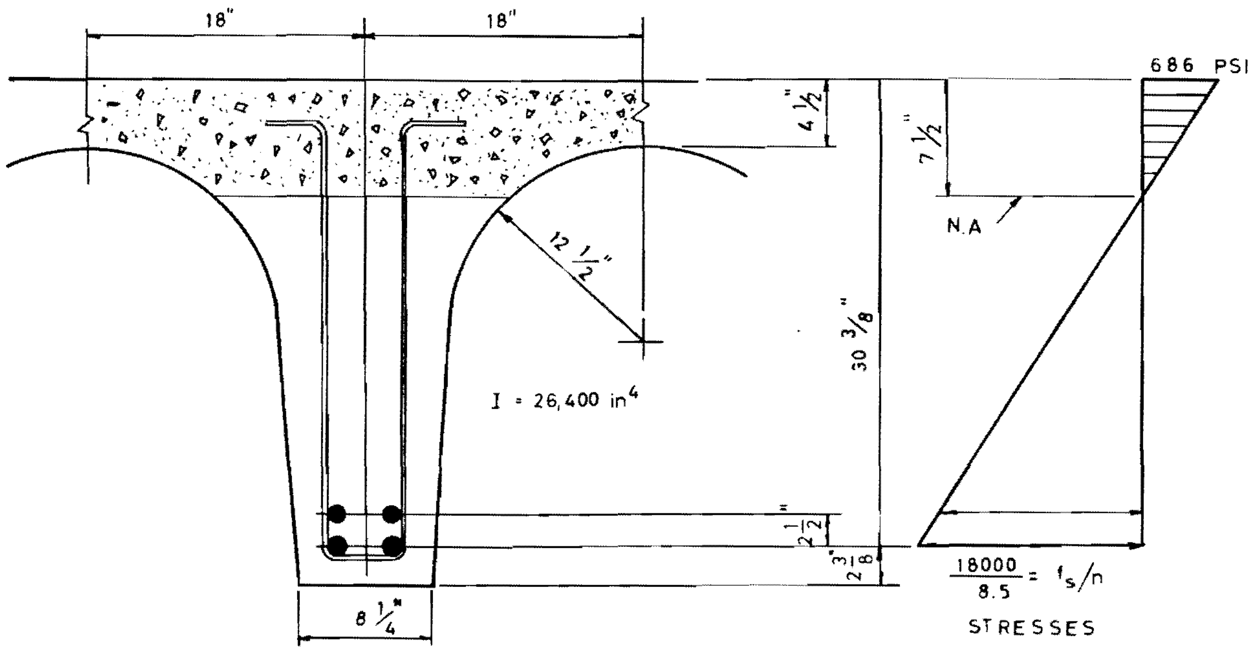


FIG. 3. (a) BRIDGE BEAM SECTION PROPERTIES

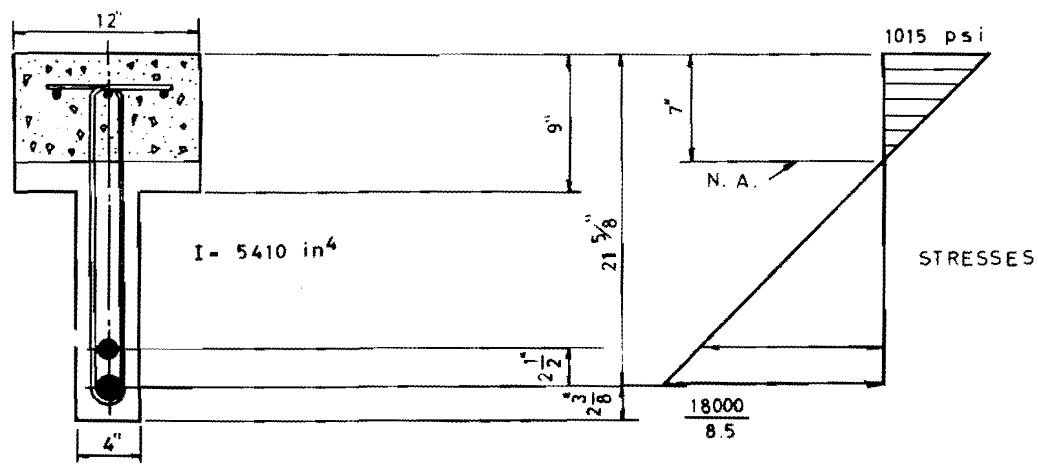


FIG. 3 (b) TEST SPECIMEN SECTION PROPERTIES compressive stress of 1015 psi in the test specimen and a compressive stress of 686 psi in the actual beam when the steel in both the test specimen and the actual beam develop a flexural stress of 18,000 psi. The ratio A/A_s for the test specimen is 10.5, and the clear concrete cover beneath the #11 bar is the same in the test specimen and in the actual bridge beam.

The cross section shown in Fig. 3 was used for a length of 6 ft. at the center of laboratory beams. The remainder of the test specimens possessed a 12 in. wide rectangular cross section of the same overall depth as the test region. The nontest or end regions of the laboratory specimen were made with constant width both to make available as much shear strength as possible in the cross section in those regions subjected to shear under the applied load, and to reduce to a minimum any anchorage problems for the longitudinal flexural bars. Complete details of the dimensions and the reinforcement for all test specimens are shown in Fig. 4.

Materials

Concrete. The concrete mix used for all eight specimens had a water-cement ratio of 5.5 gals. per sack of cement and the cement factor was 6 sacks per cubic yard of concrete. Class I ordinary cement was used. Colorado River gravel with a maximum size of 5/8 in. was used for coarse aggregate, and washed Colorado River siliceous sand was used for fine aggregate. No air entrainment or other additives were used in the mix. The mix and ingredients are the same as those used in the Central Texas area for Texas Highway Department Type C mix with Type 3 aggregate.* A relatively high slump was obtained from the substitution of a 5/8 in. maximum aggregate for the specified maximum size of 1-1/2 in. Specific proportions of the mix are given in Table I below.

TABLE I. PROPORTIONS OF THE MIX

Materials	Percent of Volume	Quantity for 2 Specimens (3.5 cu. yds.)
Water	17.8%	112 gal.
Cement	10.6%	21 sacks
Fine aggregate	27.4%	4270 lbs.
Coarse Aggregate (5/8" diam.)	41.2%	6450 lbs.
Entrained Air	3.0%	400 lbs.
Total	100.0%	

*Texas Highway Department Standard Specifications for Road and Bridge Construction, 1962, p. 480.

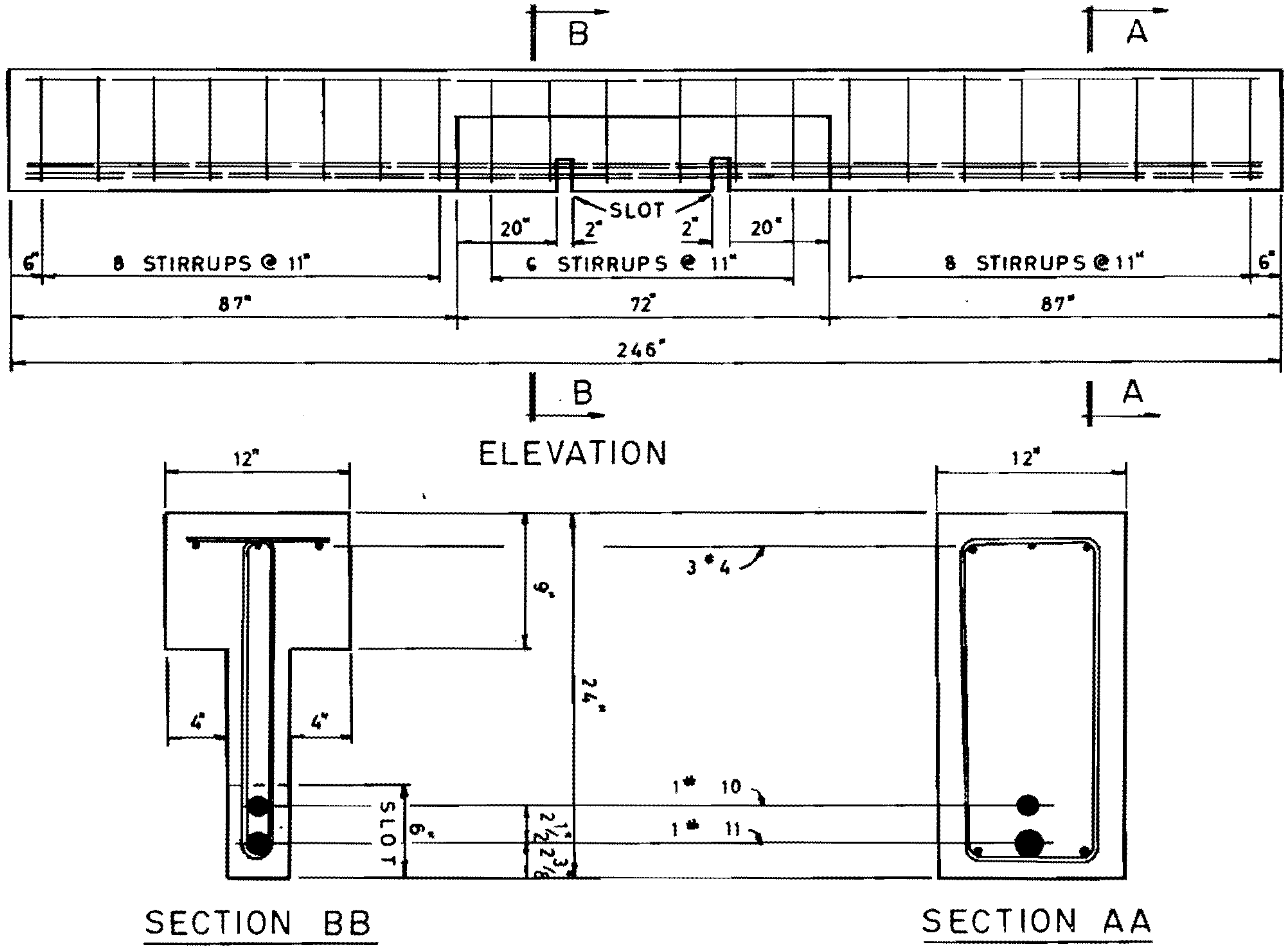


FIG. 4. DETAILS OF TEST SPECIMENS

The mix for the four separate batches provided concrete with an average strength of 3640 psi after 32 days of curing time. Table II contains cylinder compression test information showing the average strength of at least 3 cylinders for each of the strengths listed in the table. The strength on the day of loading is underlined. The data from Table II are shown graphically in Fig. 5. The strength of concrete for all specimens was reasonably uniform, although the rate of strength increase for specimens 1 through 4 began to decrease at an earlier age than for the remaining

TABLE II. CYLINDER COMPRESSION TEST DATA
(Average of 3 cylinders)

	Age of Specimen on Day of Loading			
	32 days	8 days	3 days	4 days
Age of Concrete	BC-1 & BC-2 psi	BC-3 & BC-4 psi	BC-5 & BC-6 psi	BC-7 & BC-8 psi
2 days	2240	1390	1910	2460
			(2010)	
4 days	2320	2080	2120	3120
8 days	2680	2750	3450	3520
32 days	3490	3410	3960	3680
90 days	4160	3660	4080	4530

specimens. The tensile strength for each mix on the day that each specimen was loaded was obtained from split cylinder tests on at least three cylinders. Tensile strength information as well as slump measurements for each mix are shown in Table III.

The concrete mix was easy to work and place in the forms. The concrete was placed in the forms in about three lifts of 6 to 8 in. each. Each lift was vibrated into place with mechanical vibrators. Forms were removed from each specimen on the day after concrete was cast, and specimens were left to dry in the atmosphere of the laboratory after forms were removed. Control cylinders for each mix were removed from forms and cured in exactly the same atmosphere as were the specimens.

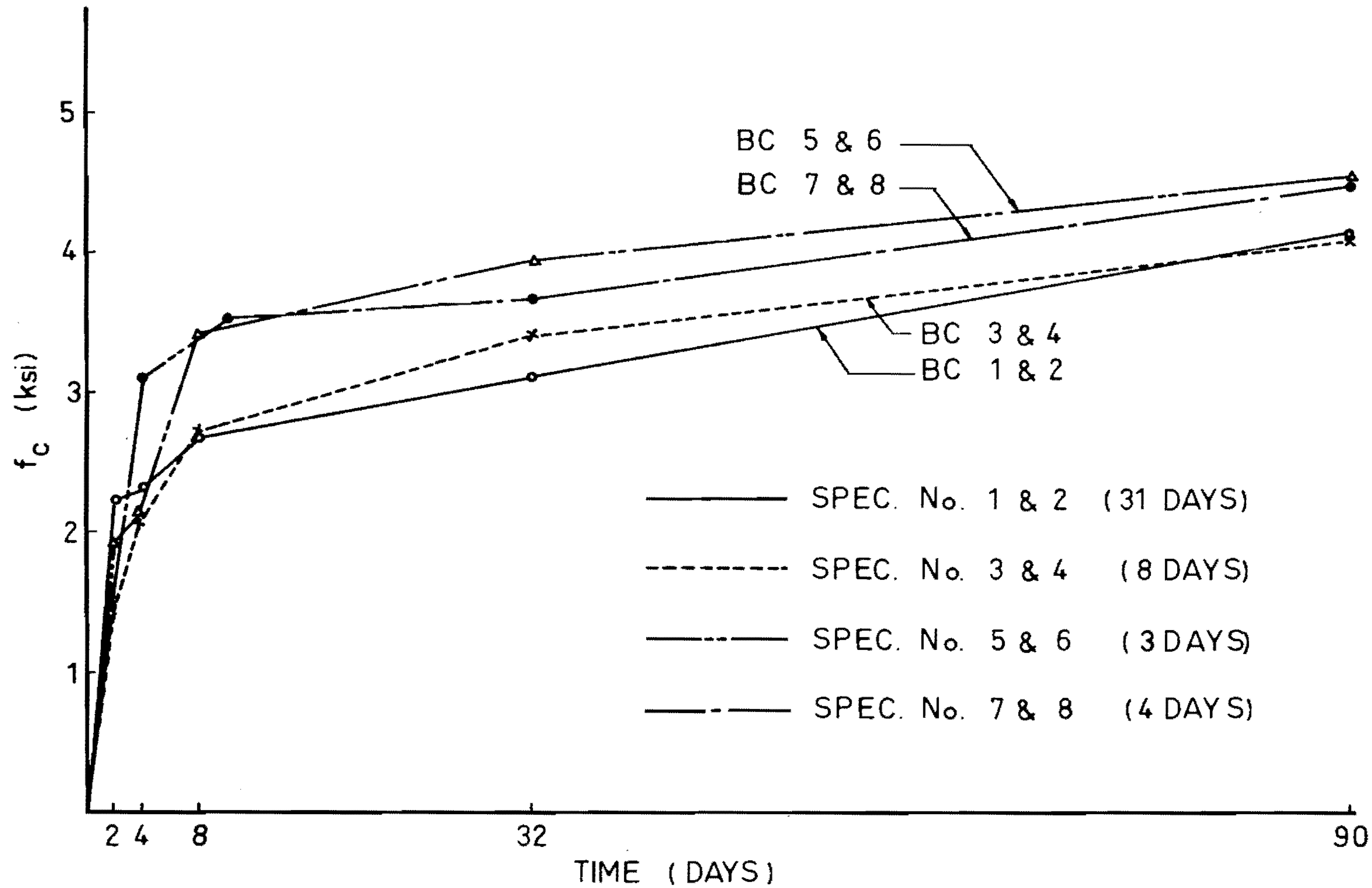


FIG. 5. CONCRETE STRENGTH vs TIME

TABLE III. CYLINDER TENSION DATA
(Average of 3 cylinders)

Loads at failure

Age of Concrete	BC-1 & BC-2 psi	BC-3 & BC-4 psi	BC-5 & BC-6 psi	BC-7 & BC-8 psi	Slump (in.)
32 days	406	---	---	---	5-1/2
8 days	---	266	---	---	4-1/2
3 days	---	---	286	---	6
4 days	---	---	---	274	4-1/2

Reinforcement. The most significant bars used for the study of creep due to bond were the main tension flexural steel bars, a #11 bar with a nominal minimum yield strength of 60 ksi and a #9 bar with a nominal yield strength of 50 ksi. Stress-strain curves for each size bar are shown in Fig. 6 and Fig. 7. Neither bar was loaded to stresses higher than 18 ksi, within which stress range each exhibited a linear stress-strain relationship. The modulus of elasticity E was 29,100 ksi for the #11 bars and E measured 28,400 ksi for the #9 bars. The #4 bars used for shear reinforcement as well as longitudinal bars in the compression zone of each specimen possessed a yield strength of 68 ksi and a modulus of elasticity E = 29,000 ksi. None of the steel in any of the specimens was subjected to stresses in excess of 18 ksi.

The height of lugs on deformations of the main flexural bars was checked against minimum requirements of ASTM Specification A615-68. Lug height for the #11 bars was very close to the minimum acceptable specification standard, but lug height for the #9 bar showed a comfortable excess above the minimum required by the specification. In all cases, the spacing of lugs fulfilled specification requirements. If any creep were to occur due to compression of concrete at bar lugs, by fulfilling ASTM A615-68 the lugs provided a representative bearing surface for concrete.

Loading

Pairs of beams cast on the same day were loaded against one another as shown schematically in Fig. 8. One beam was placed on top of the other

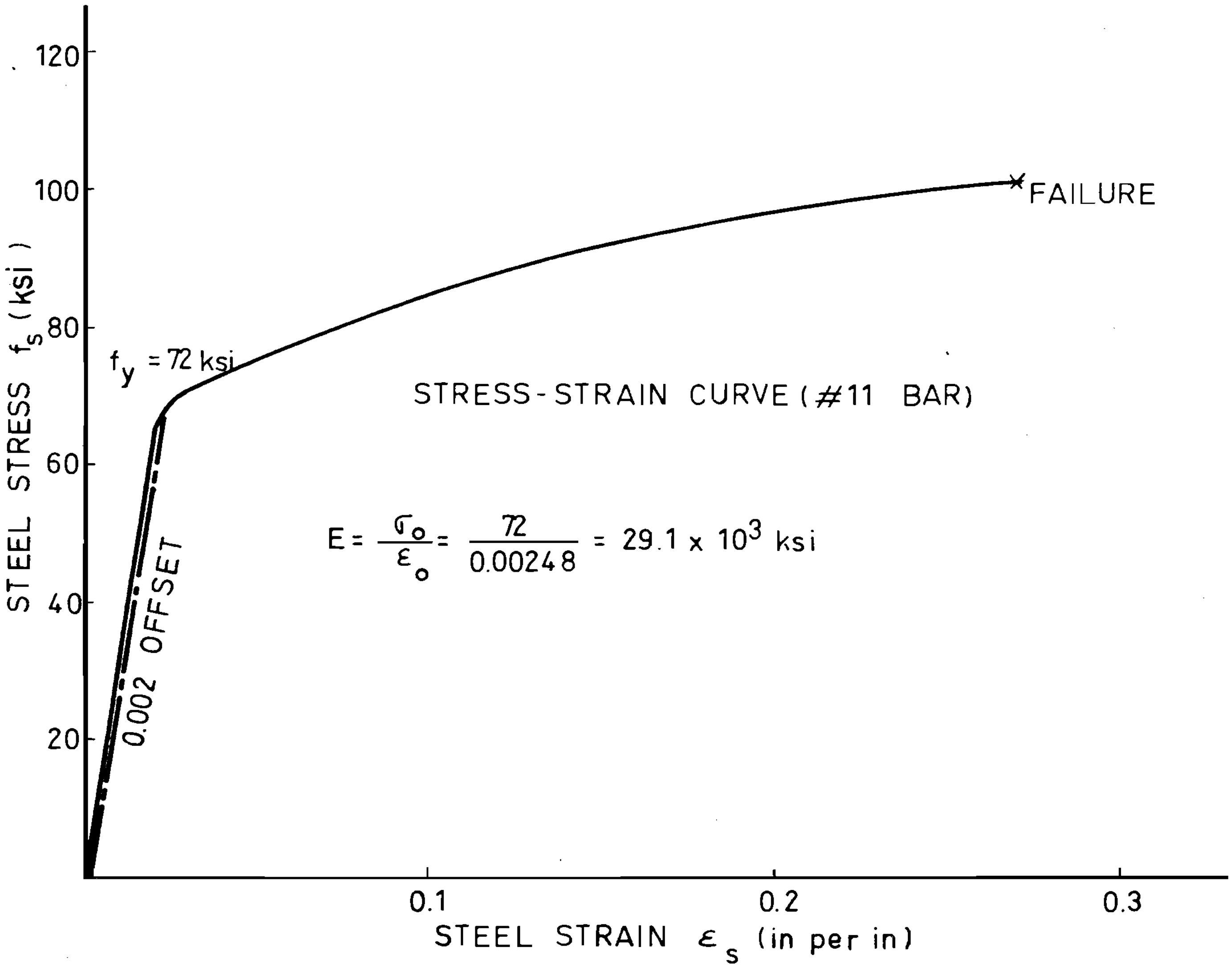


FIG. 6. STRESS-STRAIN CURVE OF No. 11 BAR.

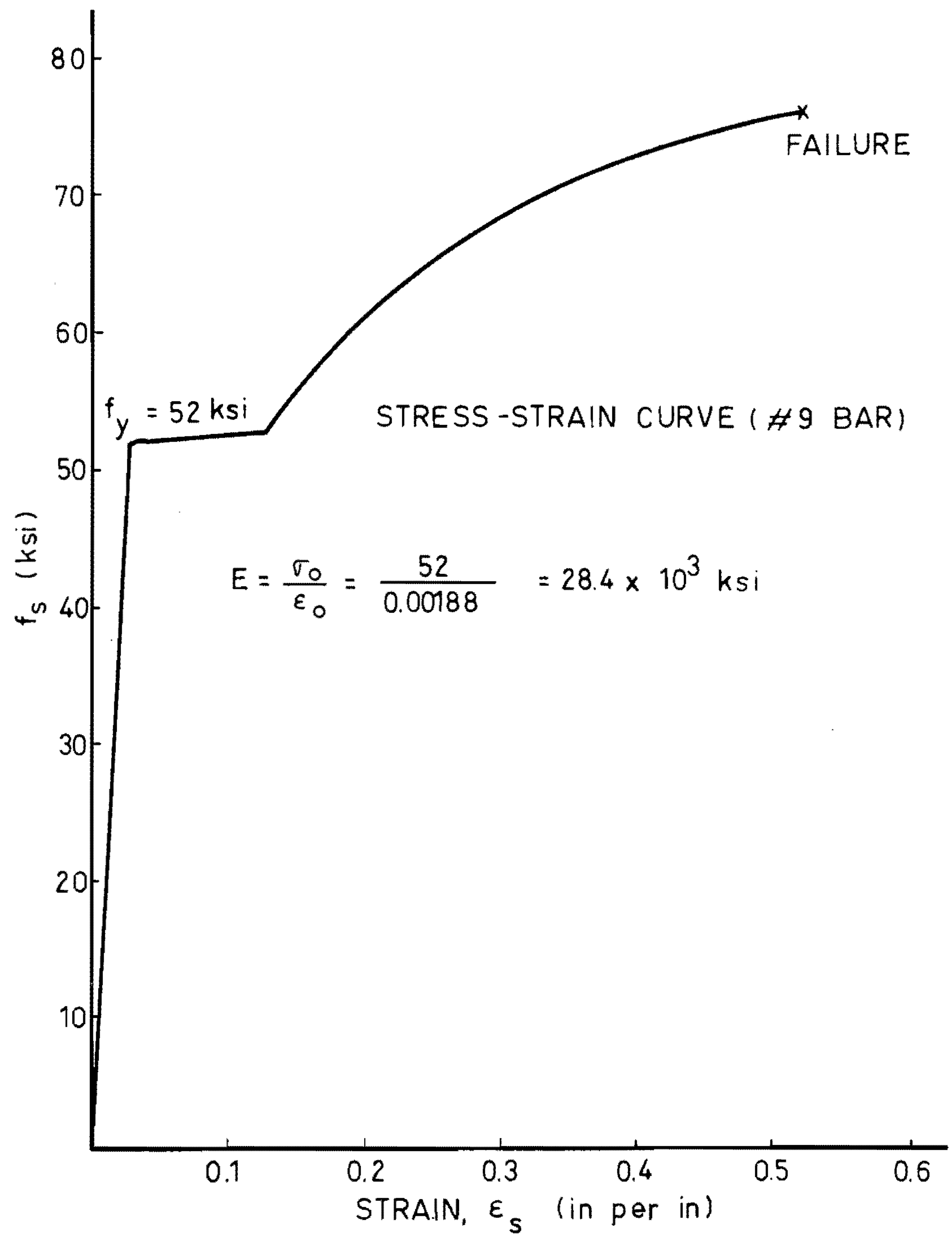


FIG. 7. STRESS-STRAIN CURVE OF No. 9 BAR.

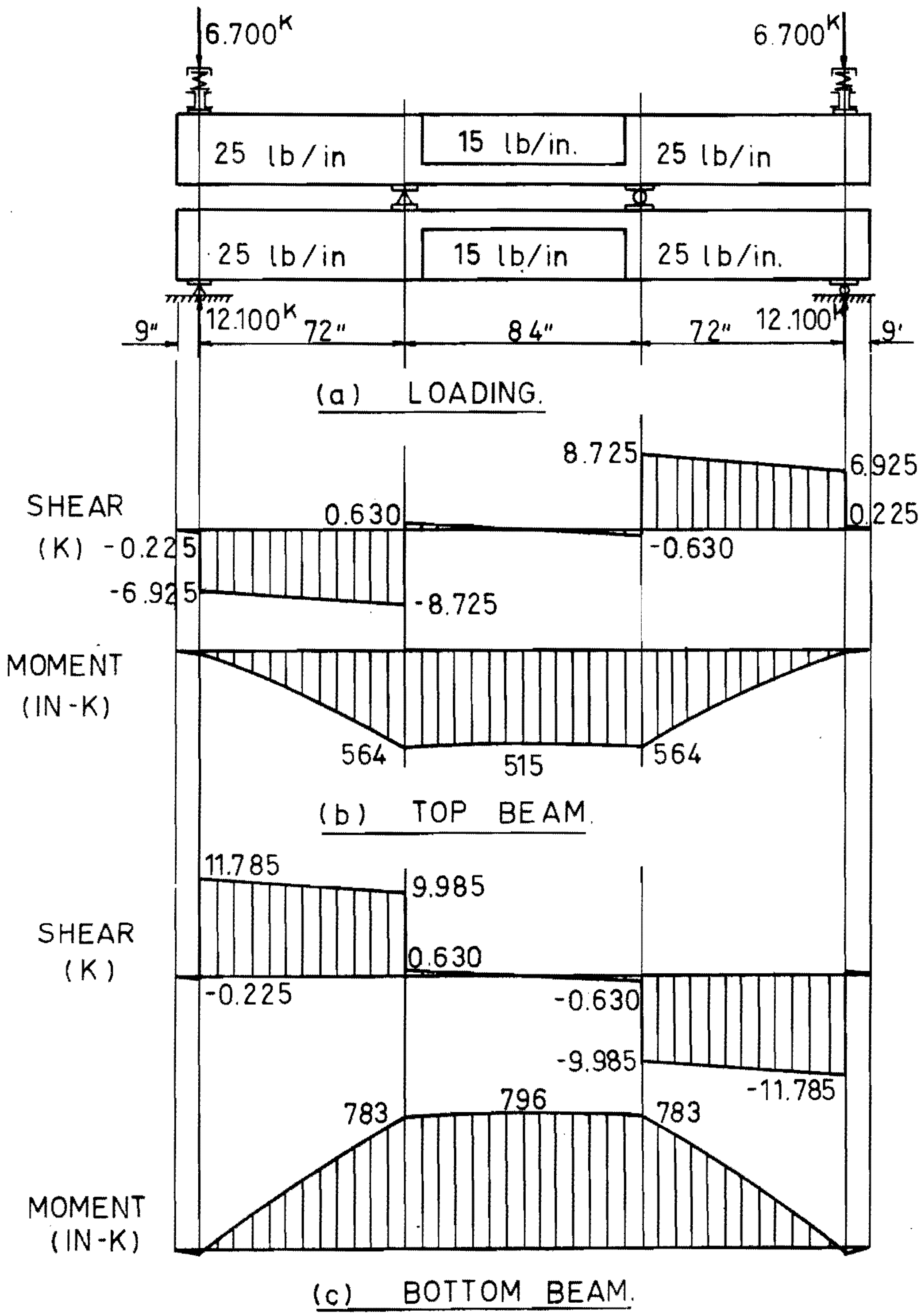


FIG. 8. LOADING ARRANGEMENT

beam before additional loads were applied through spring assemblies located near the ends of the specimen. The weight of the top beam and the weight of the lower beam added to the total moment on the lower beam. Shear diagrams and moment diagrams for the permanent load applied to the beams are given in Figs. 8b and 8c.

The maximum moment at midspan for the top beam was 551 in.-kips, and the maximum moment at the middle of the lower beam was 796 in.-kips. On the basis of properties in a transformed cracked cross section, the tensile stress at the centroid of the steel in the top beam was 11.6 ksi, and the stress in the steel in the bottom beam was computed to be 17.0 ksi. The maximum shear stress near the end of the bottom beam was only 48 psi, a stress that the concrete could sustain without any help from stirrups. Nominal stirrups 11 in. apart were placed in the test region and in the end region of all specimens in order to avoid altogether any complications from shear distress, and the sitrrups served the additional purpose of holding longitudinal bars in proper alignment during the casting of concrete.

With the exception of loads caused by the weight of each specimen, any external forces applied to the specimen would tend to decrease as the creep deformation of specimens allowed a reduction in resistance to load. However, loads applied through a highly flexible mechanical system would vary only slightly as beams deflect. An assembly of heavy springs was used at each end of the test specimens, and small displacements of the beams resulted in only small changes in the force transmitted through the spring assembly. An auxiliary benefit of the calibrated spring assemblies was derived from the use of the calibrated springs to maintain the desired constant level of load. A diagram of the spring assemblies is shown in Fig. 9.

The spring assemblies were calibrated simply by compressing the top channel against the springs and recording changes in dial indicator readings as the top channel was forced closer to the bottom channels. The total travel of the assembly exceeded 2 in., and the desired test force of 6.7 kips for the assembly required slightly more than 1 in. of displacement in the spring system. Dial indicators revealed spring displacements to 0.001 in. which corresponds to a force of approximately 7 lbs. Force was applied to

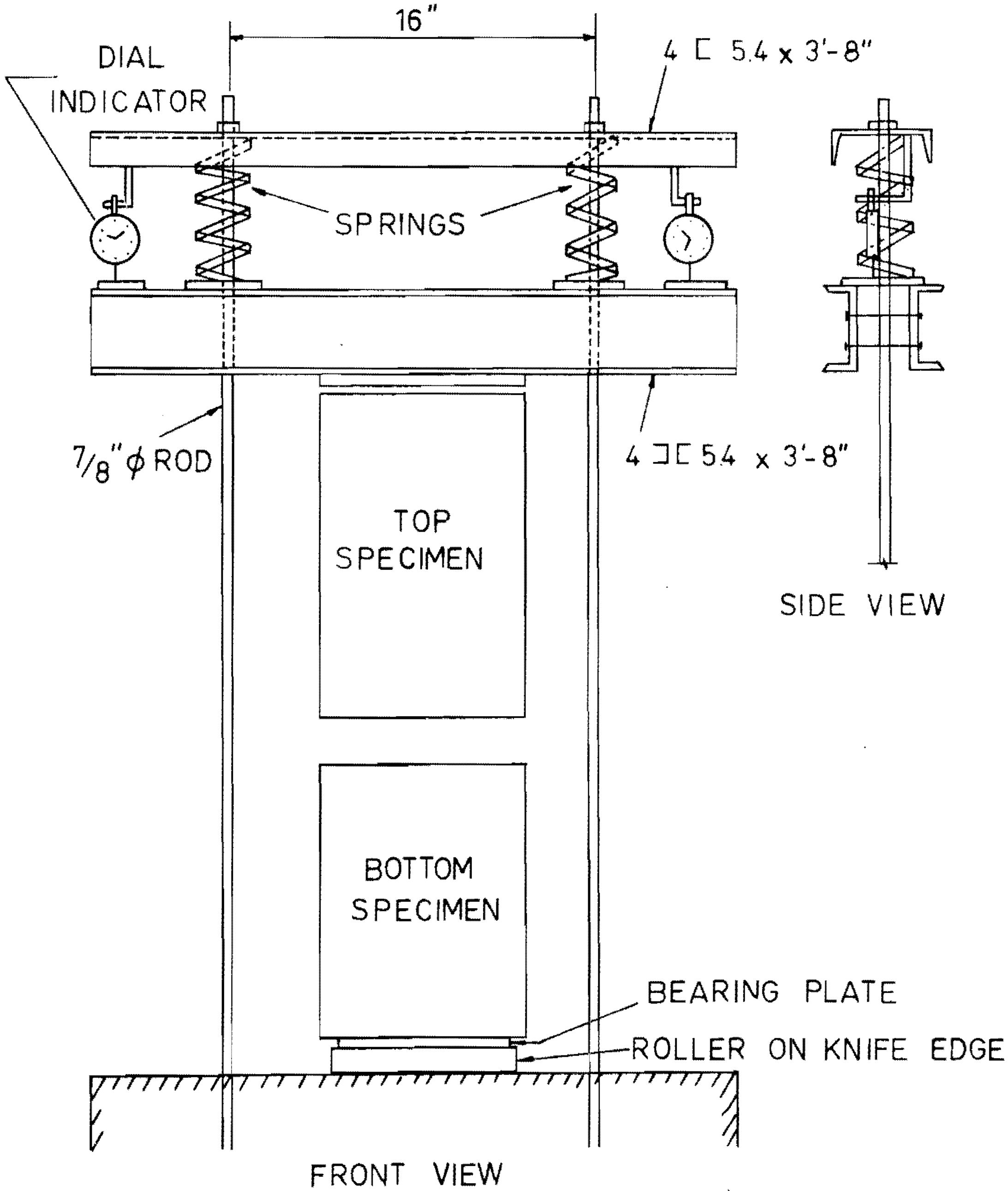


FIG. 9. DETAILS OF THE SPRING ASSEMBLY

the spring assemblies on the specimens simply by tightening the nuts on the anchor rods that formed the sides of the loading halter. Periodically during the sustained load period, dial indicators were read and nuts on the anchor rods were adjusted to maintain a constant force through the springs. The loading system as used, with periodic adjustments of the load, maintained a force that varied less than 150 lbs. from the desired force of 6700 lbs. at the ends of each specimen.

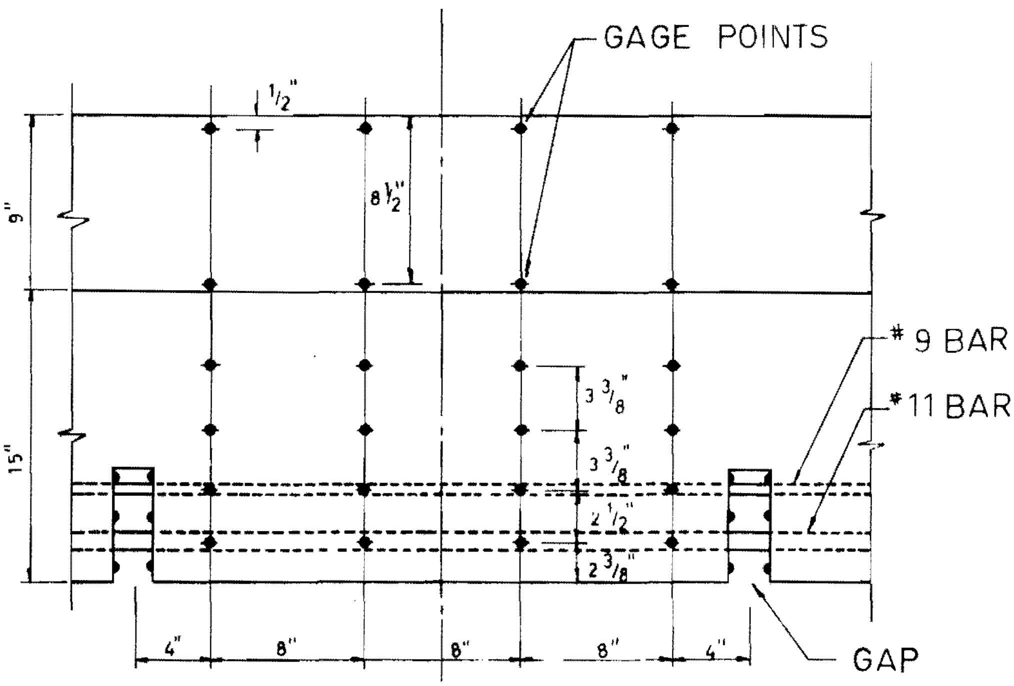
Measurements

Loads were measured and maintained as already described. Attempts to detect creep due to bond were made by measuring at periodic intervals the following quantities:

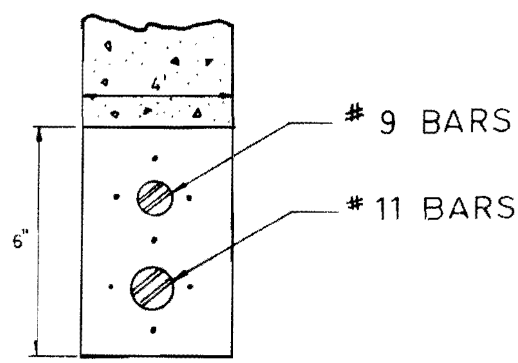
(1) horizontal surface strains up and down the sides of the test specimens,
(2) strains across a gap in which concrete was omitted in order to expose reinforcing bars, and
(3) the deformation along the tension face of the specimens.

(1) Surface Strain Measurement. An 8-in. Berry gage was used to measure surface strains along the depth of each specimen. Gage points containing standard holes for the Berry gage were made in Demec gage tabs which were then applied to the surface of the concrete with epoxy. The Demec gage tabs were applied in four vertical rows on each side of each specimen at six different levels through the height of each specimen. Thus, 48 Demec gage points and 18 measuring stations existed on each side of each specimen. The Demec gage lines 8 in. apart were located at midspan in the test region of each specimen. The location of Demec gage points for surface strain is indicated in Fig. 10a.

Berry gage readings could be made to the nearest thousandth of an inch, and some practice with the Berry gage was necessary in order that consistent and repeatable readings could be made. At best, readings to the nearest 0.001 in. over an 8-in. gage length represent an accuracy of only 125 microinches. The cracking of concrete is never consistent between fixed gage stations, but crack patterns across the 24 in. space, represented by three adjacent 8 in. stations, provided a reasonable average for the number of discrete cracks in the test region. For purposes of data reduction, the



(a) DETAILS SURFACE STRAIN POINTS



(b) SECTION AT BARS IN GAP

FIG. 10. DETAILS OF GAP STRAIN POINTS

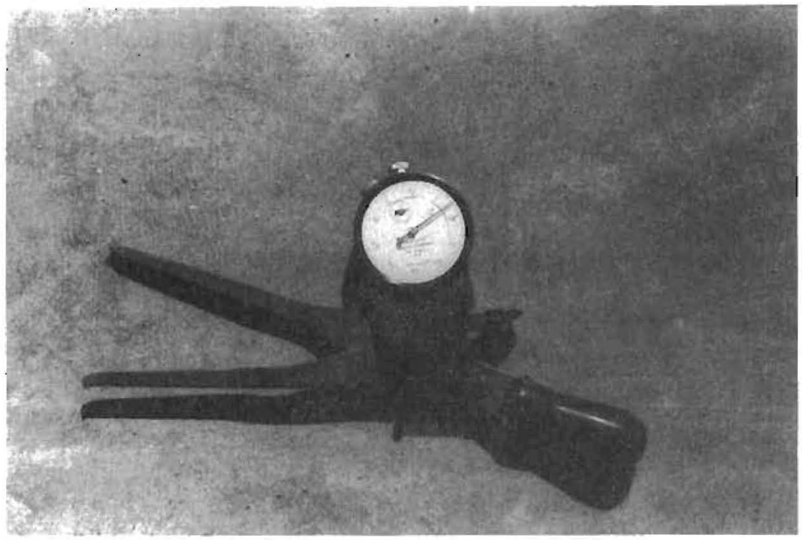
change in surface strain at all six 8-in. gage locations (three stations on each side of a specimen) at a specific height on the specimen were averaged. Strain profile data represent the average change in strain through the 24-in. gage region on both sides of each test specimen.

(2) Gap Strains. A 2-in. wide void of concrete was created around tensile reinforcement at 2 stations for every specimen. Any loss of bond around reinforcement should have created changes in the space across the gap different from changes in strain where no gaps existed. Demec gage points on each face of each gap were placed above, below, and beside the main tensile reinforcement as indicated in Fig. 10b. Changes in the distance across the 2-in. gap were detected with an inside dial caliper manufactured by the Mueller Gages Company and illustrated in Fig. 11a.

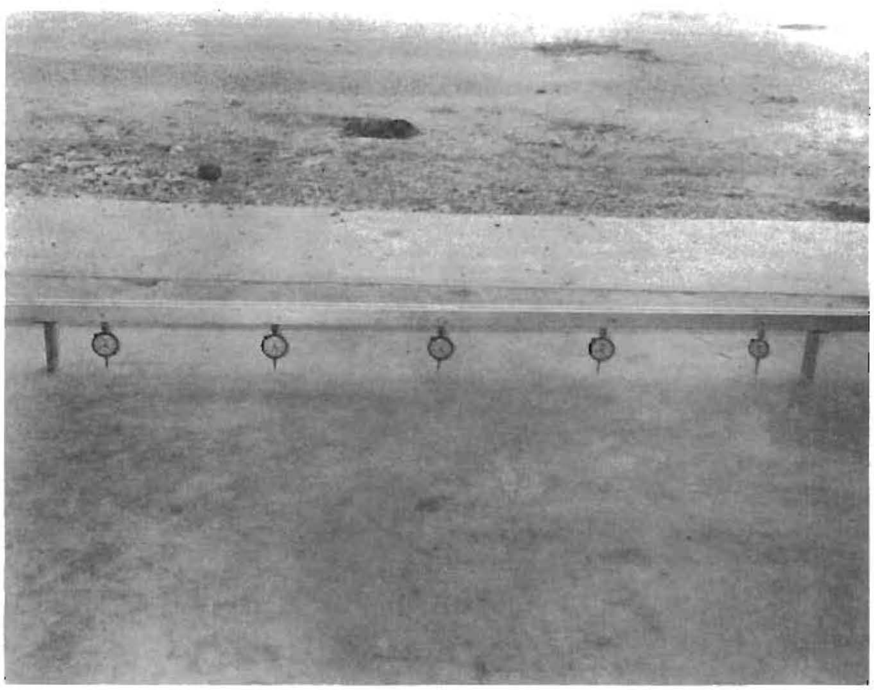
Readings of the inside dial caliper could be made to the nearest 0.0001 in., providing a strain resolution approximately to the nearest 50 microinches across the 2-in. gap. Data reported for gap strain represents the average for both gaps on a specimen and all seven gage stations in each gap. Thus gap strains reported here represent the average of 14 readings.

(3) Deflection Measurements. The deflection of beams was measured only across the constant moment test region of each specimen. Five dial indicators were attached to a gage bridge consisting of a 6-ft. long aluminum bar with two posts to serve as bearings at fixed locations near the end of each test region. Consequently, the deflection readings reported here involve only the deflections due to curvature through the test region. Metal tabs were attached to the test specimen with epoxy to provide a flat surface for each dial plunger to bear.

Dial indicators were calibrated to the nearest 0.001 in., and readings were estimated to the nearest 0.0001 in. Deflections reported here represent displacements of the center dial from the chord connecting the gage points at the exterior or end dial. Deflections are accurate only to the nearest 0.001 in. The deformation gage is illustrated in Fig. 11b.



(a) Inside caliper gage



(b) Deformation gage

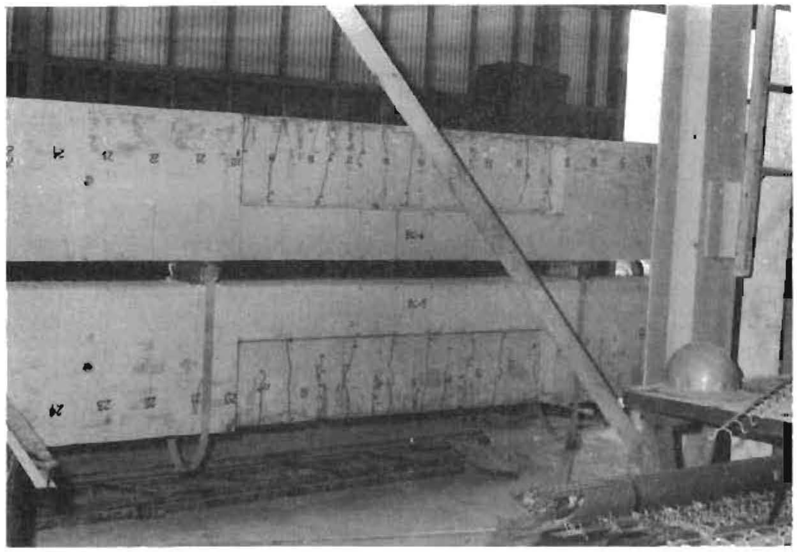
Fig. 11. Deformation gages.

CHAPTER III

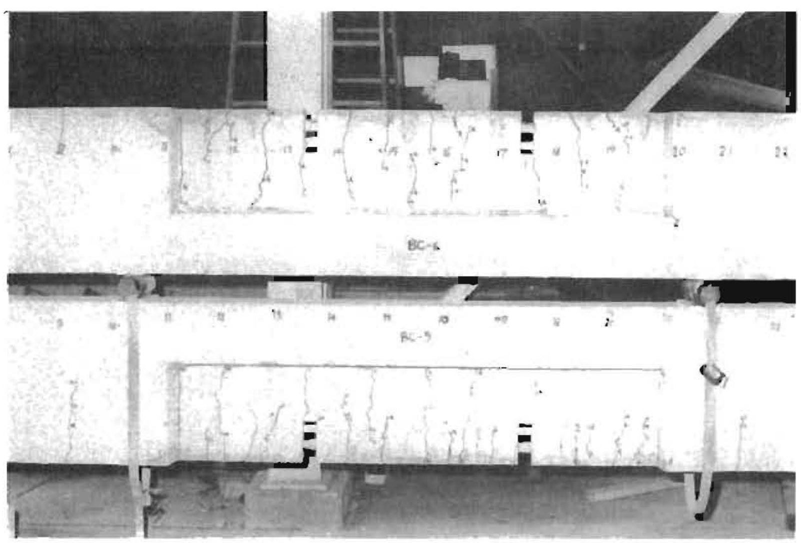
RESULTS

Eight specimens were observed in this study. Pairs of beams loaded against one another were subjected to initial load at four different intervals of time after concrete was cast. Obviously, the age of the concrete was identical for the two beams loaded against one another. The age and concrete strength for each pair of specimens were listed in Table II. Table II contains age and concrete strength information for the day on which loads were applied initially and for the strength of concrete 32 days after the concrete was cast. Load was maintained on each pair of specimens for more than 2000 hours during which all gages were read at periodic intervals. Spring assemblies were tightened to the desired load level before each set of readings was made.

Cracks that occurred in the concrete during the initial loading of each specimen were noted and traced along the exterior surface of the beam. Subsequently, as further cracking developed and cracks propagated still further, their progress was noted and traced along the surface of each specimen. A number indicating the sequence of the time of marking was written at the end of each crack. The specimens loaded at an early age (three or four days) displayed fewer cracks. Initially, cracks were smaller than those which could be seen on the surface of the specimens after a longer interval of time under load. Eventually, as concrete continued to cure and dry in the rather arid atmosphere of the laboratory, all specimens developed practically identical crack patterns, i.e., the size and distribution of cracks varied only slightly among specimens at the same level of tension stress in the main reinforcement. Photographs in Figs. 12 through 15 show crack patterns after specimens had been loaded for at least 90 days. Crack locations numbered 4 or less existed when the initial load was reached. All cracks in the test region can be seen to be almost vertical, indicating the

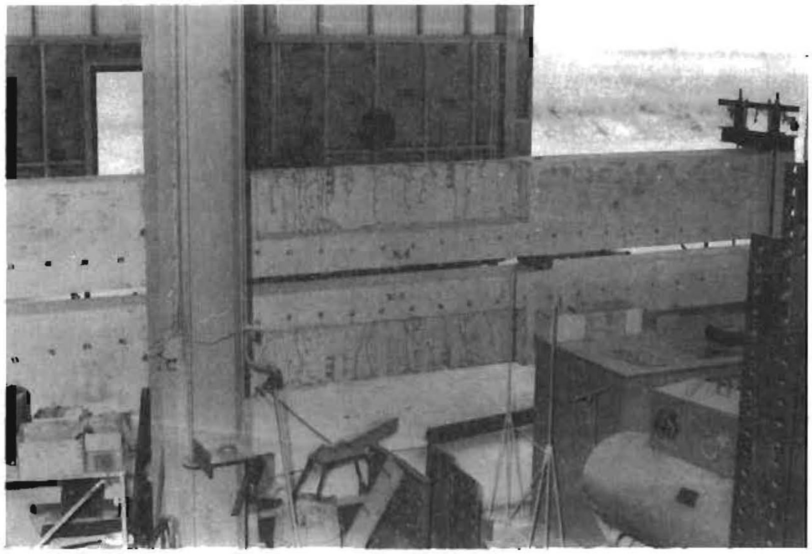


(a) West face



(b) East face

Fig. 12. Crack distribution after three months. Initial loading at age 3 days.



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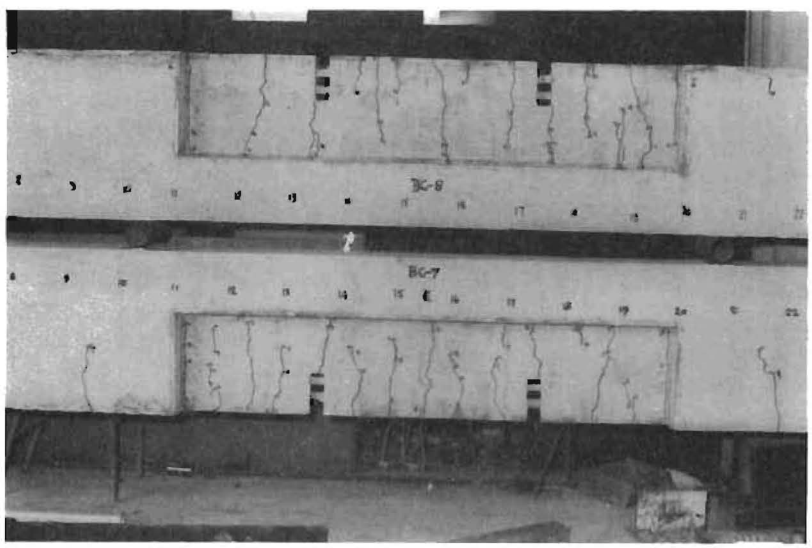
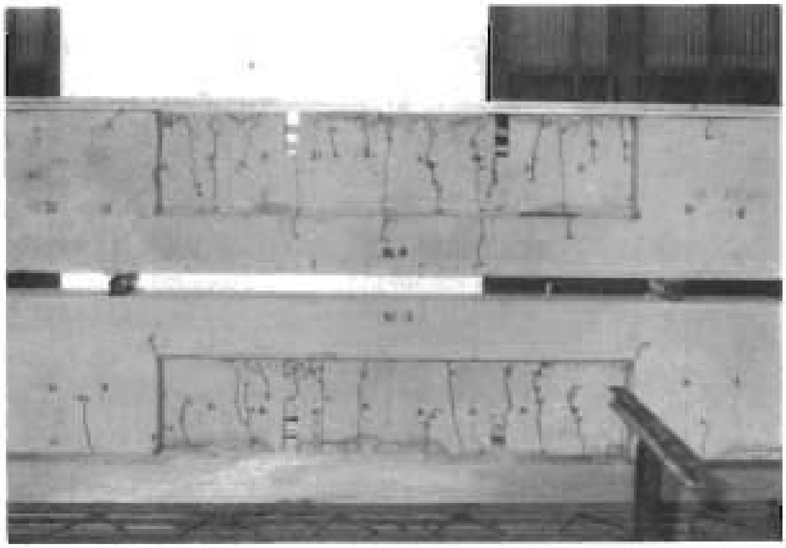
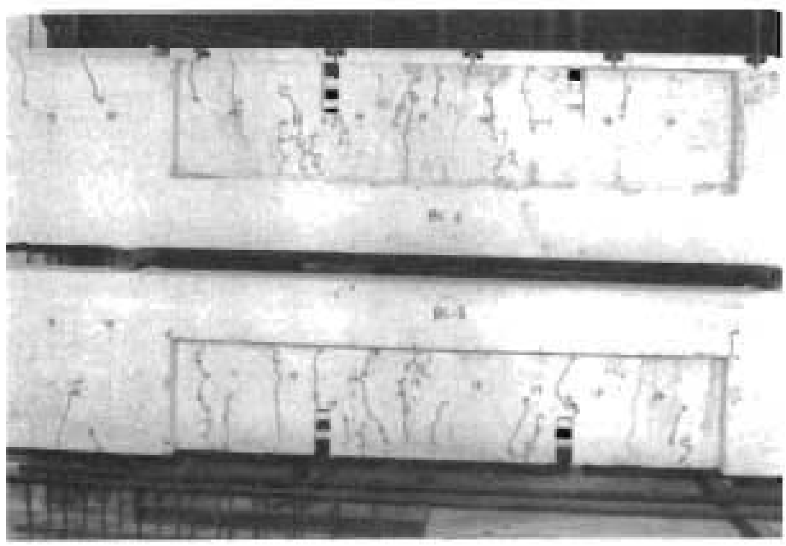


(b) East face

Fig. 13. Crack distribution after three months. Initial loading at age 4 days.

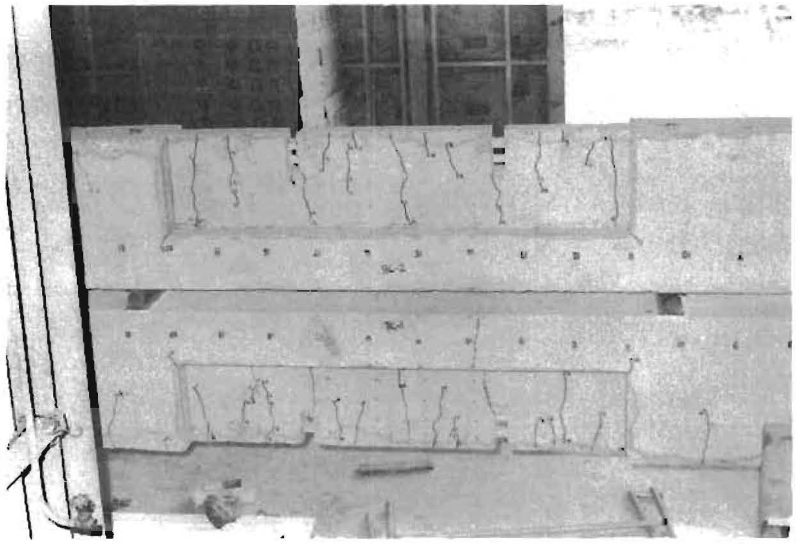


(a) West face

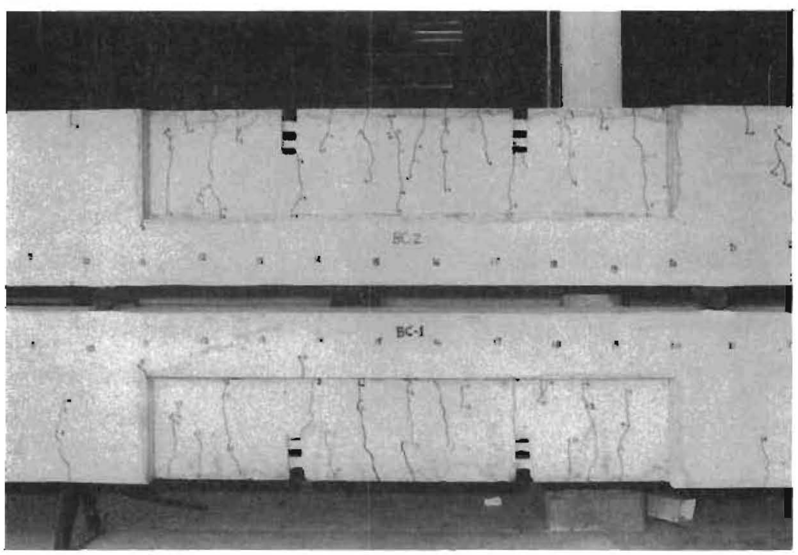


(b) East face

Fig. 14. Crack distribution after three months. Initial loading at age 8 days.



(a) West face



(b) East face

Fig. 15. Crack distribution after three months. Initial loading at age 32 days.

existence of flexural stress and little, if any, shear stress. No cracking occurred parallel to reinforcement, and no evidence of bond splitting can be seen in the test specimens.

Table IV contains a numerical summary of crack size from major cracks in each specimen at various ages after the day of initial loading. Generally, the cracks were measured near the level of main tension reinforcement. The table shows rather clearly that initial cracks were smaller in the concrete loaded at earlier ages; but the final size of cracks after four months under load did not differ appreciably among the four specimens called lower and the four specimens called upper beams (lower beams developed 17 ksi steel stress and upper beams developed only 12 ksi steel stress). Since the moisture in a specimen that is cured in dry air should decrease with the age of the specimen, differences in the spacing and size of crack should be attributed to the drying and shrinkage of concrete. Brittleness in tension of plain concrete tends to increase as concrete dries.

TABLE IV. SIZES OF CRACKS

Age of Loading	Beam Position	Day of Loading (in.)	1 Wk. (in.)	2 Wks. (in.)	2 Mos. (in.)	4 Mos. (in.)
3 days	Lower	0.0020	0.0035	0.0035	0.0040	0.0045
	Upper	0.0020	0.0025	0.0030	0.0030	0.0035
4 days	Lower	0.0025	0.0030	0.0030	0.0035	0.0045
	Upper	0.0015	0.0025	0.0030	0.0030	0.0035
8 days	Lower	0.0030	0.0035	0.0035	0.0040	0.0050
	Upper	0.0020	0.0025	0.0030	0.0035	0.0040
32 days	Lower	0.0035	0.0040	0.0040	0.0045	0.0055
	Upper	0.0025	0.0025	0.0030	0.0035	0.0040

Surface Strains

All surface strain data reported here represent the average strain for six measuring stations (three on each side of the beam) at the level indicated. Initial readings were taken before the beams were placed in the loaded position, and three sets of readings were made as load was being applied. After the permanent load was reached, strain readings were made at periodic intervals. Early during the sustained load period, readings were made every few hours. Later, time intervals were measured in days, and eventually time intervals lasted weeks between strain readings. Maximum strains occurred nearest the compression face and nearest the tension face of each specimen. The change with time of average tension strain at the reinforcement and maximum measured compression strain is displayed for each specimen in Figs. 16 through 19. Each of the figures contains measured strain data for the two beams cast and loaded simultaneously, plotted against a log scale of time (in hours) under constant load.

For the beams loaded at an age of 3 days the graphs of Fig. 16 indicate that all compression strains increased at a steady rate during the first 1000 hours under load. During the next 700 hours surface strains at the 17 ksi steel increased more than 150 microinches. The steel stress corresponding to a tension strain of 590 microinches would be 17 ksi. Surface strains adjacent to the 12 ksi steel increased from 330 to 390 microinches during 1000 hours. For a strain of 390 microinches, the corresponding steel stress would be 11.3 ksi. During the same 1000 hours, compression strains increased approximately 150 percent. During the second 1000 hours under load, <u>all</u> strains indicate an overall shortening of both specimens, probably due to the fact that the shrinkage rate overtook the creep rate.

Beams loaded at the age of 4 days show in Fig. 17 that after a sharp increase during the first 4 hours, all strains varied with time much the same as those observed in Fig. 16. Tension strains near the 17 ksi steel increased from 560 to 600 microinches in 1000 hours, and strains adjacent to the 12 ksi steel increased from 330 to 430 microinches. During the same 1000 hours, compression strains increased at least 100 percent. Again, an overall shortening occurred during the second 1000 hours under load.

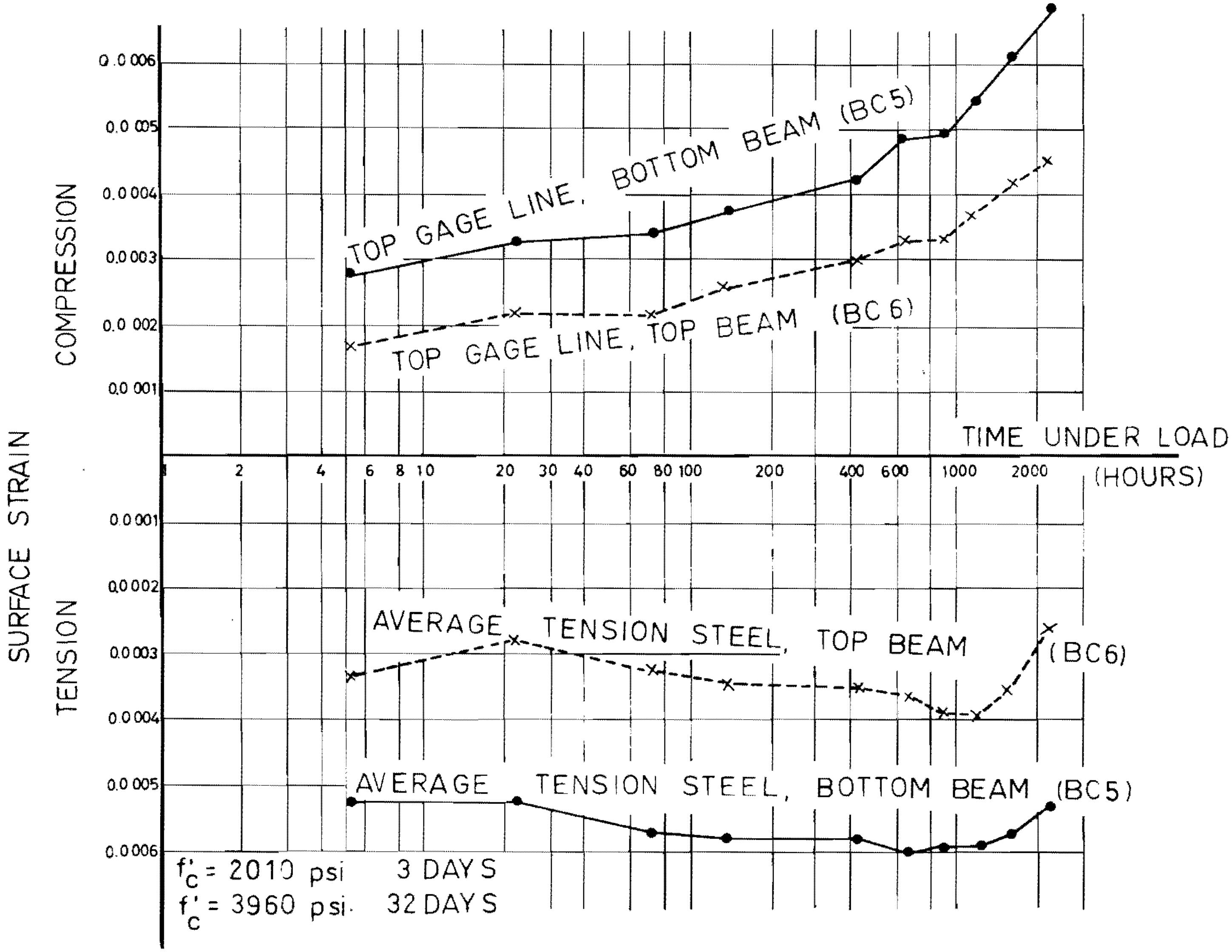


FIG. 16 MAXIMUM STRAINS FOR SPECIMENS LOADED AT AGE OF 3 DAYS

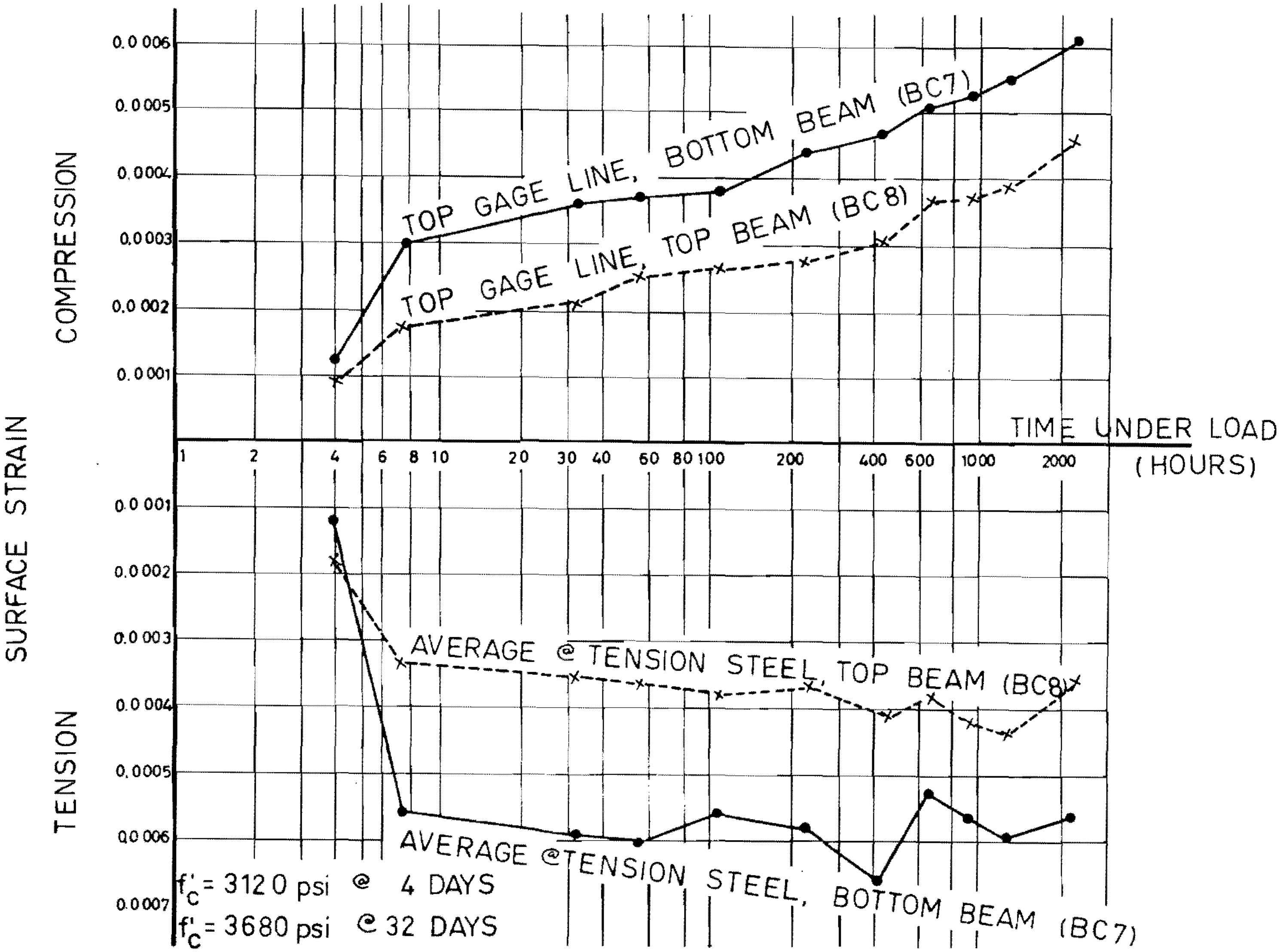


FIG. 17. MAXIMUM STRAINS FOR SPECIMENS LOADED AT AGE OF 4 DAYS.

Measured strains for specimens loaded at an age of 8 days are shown in Fig. 18. Demec gages on the top beam (BC4) were mounted with an improper epoxy mixture, and reliable data could not be accumulated. The erratic nature of strain graphs in Fig. 18 suggests that other gage tabs may have been mounted improperly. The curves do indicate large increases in all strains during the sustained load period. Adjacent to 17 ksi steel, strains increased from 450 to 620 microinches after the 21-hour reading, and strains increased from 250 to 500 microinches adjacent to 12 ksi steel. Compression strains in the bottom beam (with 17 ksi steel) appeared to increase about 250 percent.

Beams loaded at an age of 32 days displayed strain-time behavior shown in Fig. 19. Small increases in tension strains were measured similar to the beams loaded at an age of 3 days and 4 days. At 17 ksi steel, strains increased from 600 to 680 microinches in 1500 hours under constant load. The tendency for an overall shortening due to shrinkage in the specimen loaded at a 32-day age was less prominent than in specimens loaded at very early ages.

The magnitude of strain at every level through the height of each specimen seven to ten hours after the time of loading and also 400 or more hours after the time of initial loading are displayed in Figs. 20 through 23. The readings after 400 hours were considered representative of behavior after strength developed and before shrinkage became prominent. Also shown in these figures are lines displaying the linear strain profile determined on the basis of elastic behavior in a cracked transformed cross section. The modulus of elasticity E_s used for the theoretical strain profiles was taken as 29000 ksi and a neutral axis was taken at a depth of 7 in.

Strain profile graphs in Figs. 20, 21, and 23 indicate that the measured compression strains compare almost exactly with computed strains based on the elastic behavior of cracked transformed cross sections. Data from Fig. 22 for specimens loaded at 8 days were erratic because of poor epoxy and loose Demec gage points. The figures show quite clearly that plane sections before bending did not remain plane after bending, and the variation from a linear distribution of strain through the depth of a beam was greatest

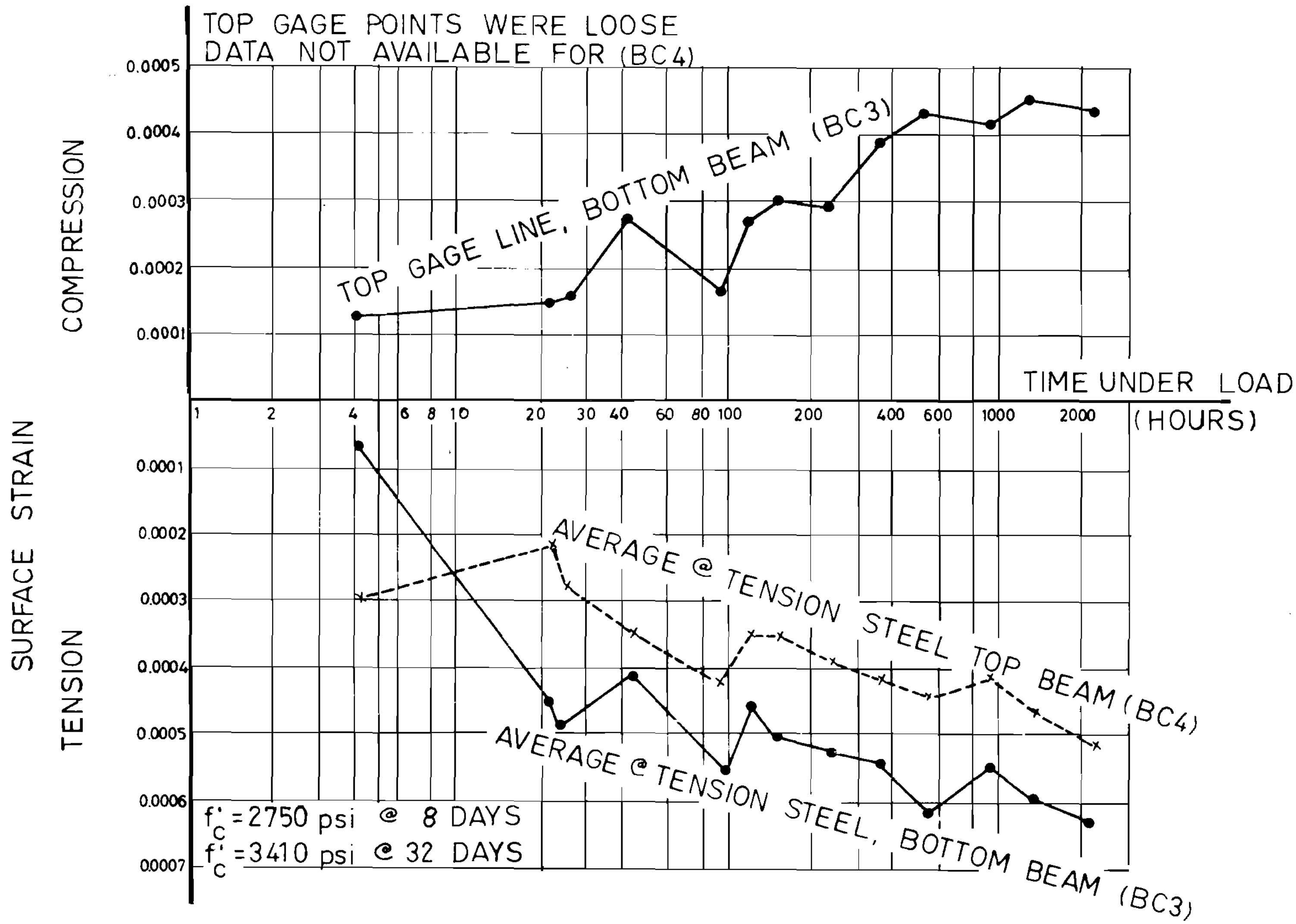


FIG. 18. MAXIMUM STRAINS FOR SPECIMENS LOADED AT AGE OF 8 DAYS.

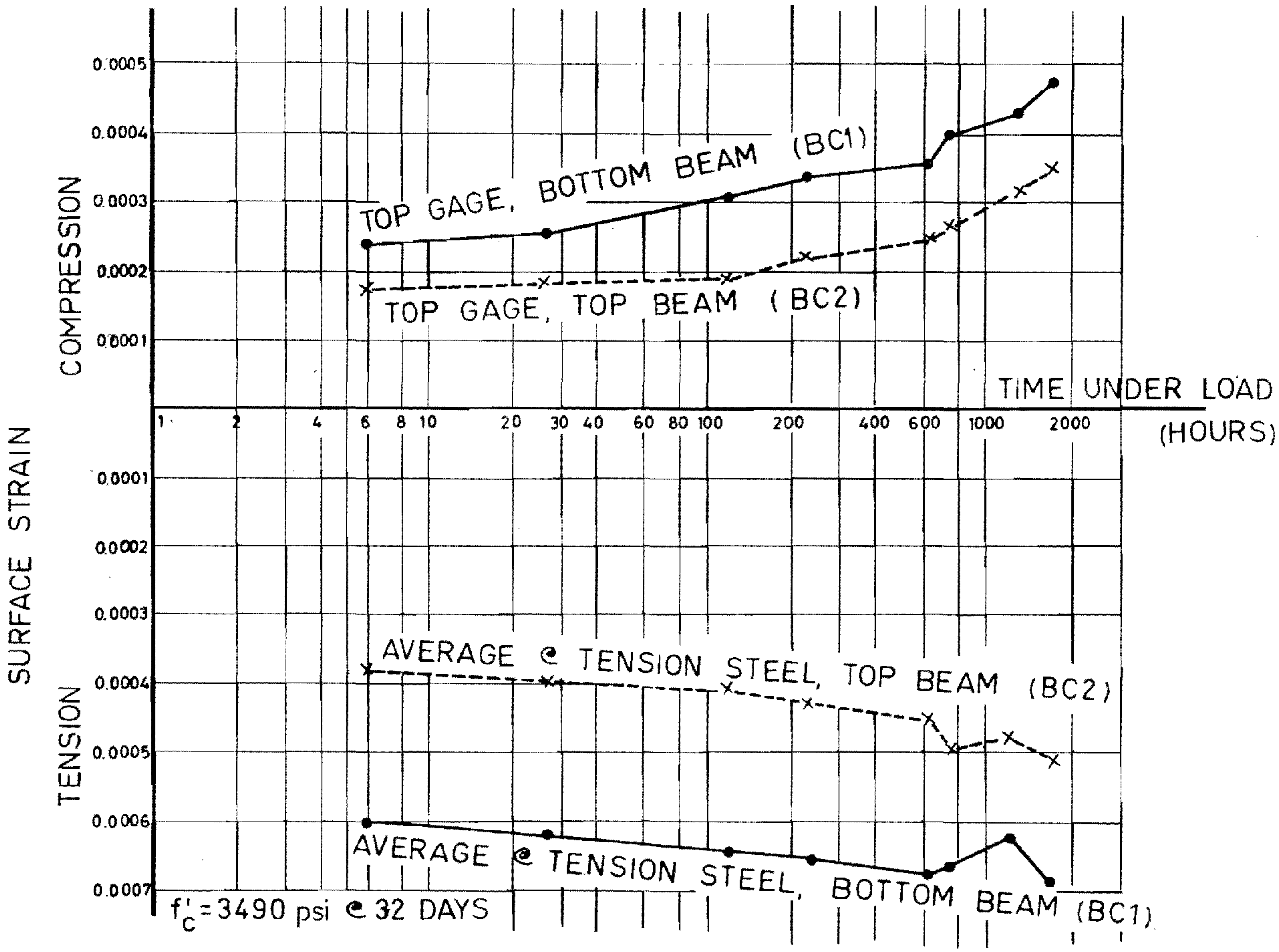


FIG. 19 MAXIMUM STRAINS FOR SPECIMENS LOADED AT AGE OF 32 DAYS.

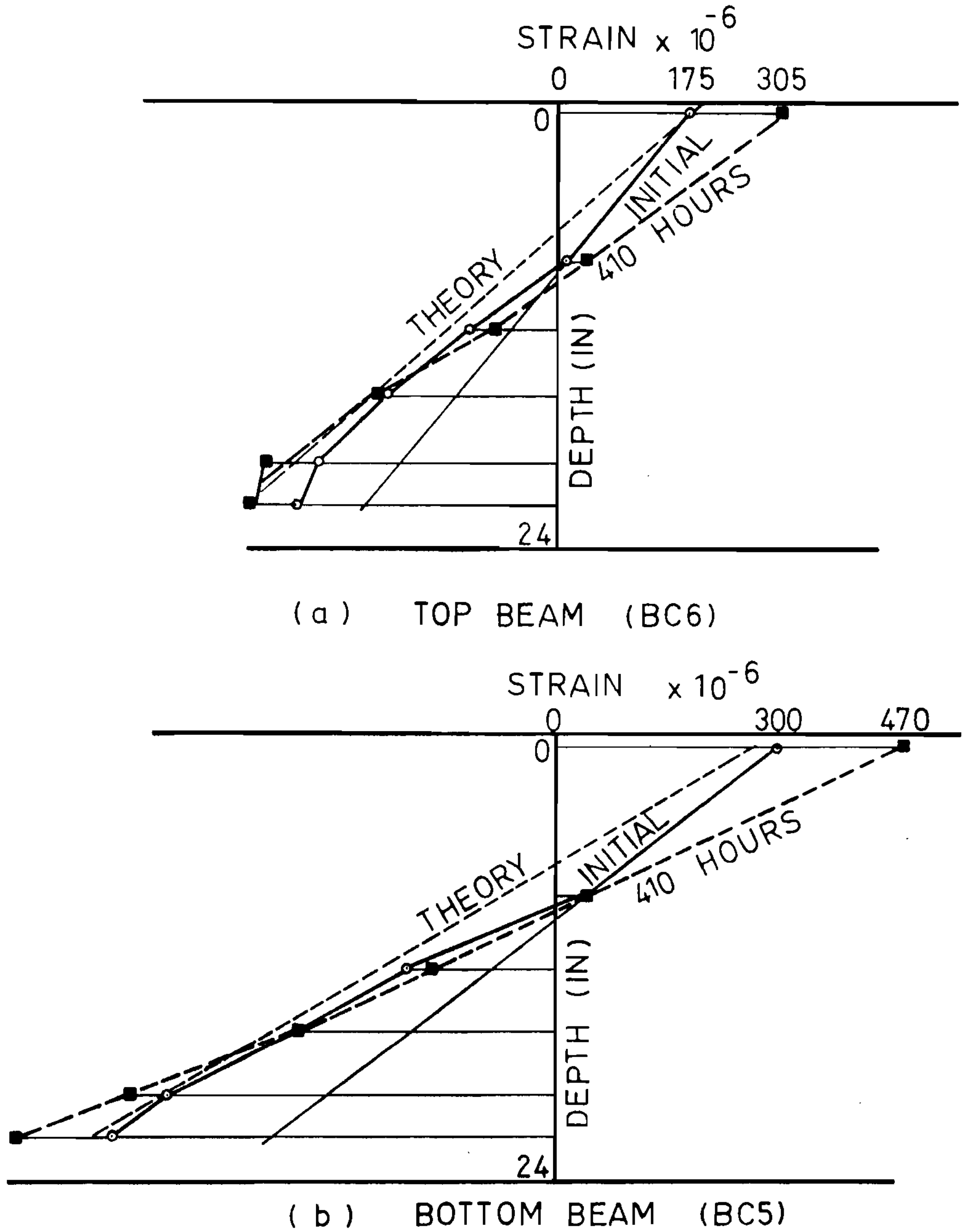


FIG. 20. STRAIN PROFILES -BEAMS LOADED AT 3-DAY AGE.

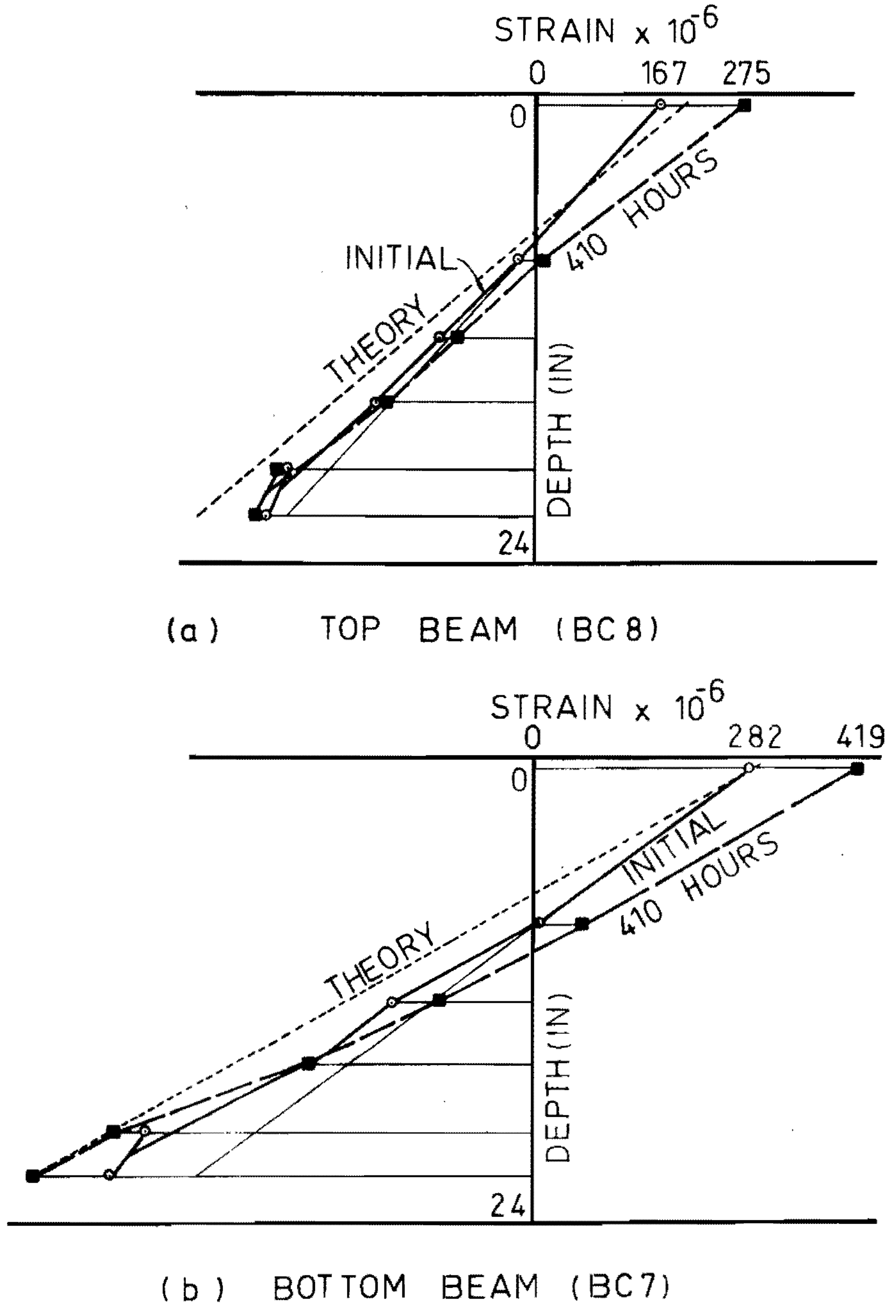


FIG. 21. STRAIN PROFILES - BEAMS LOADED AT 4-DAY AGE.

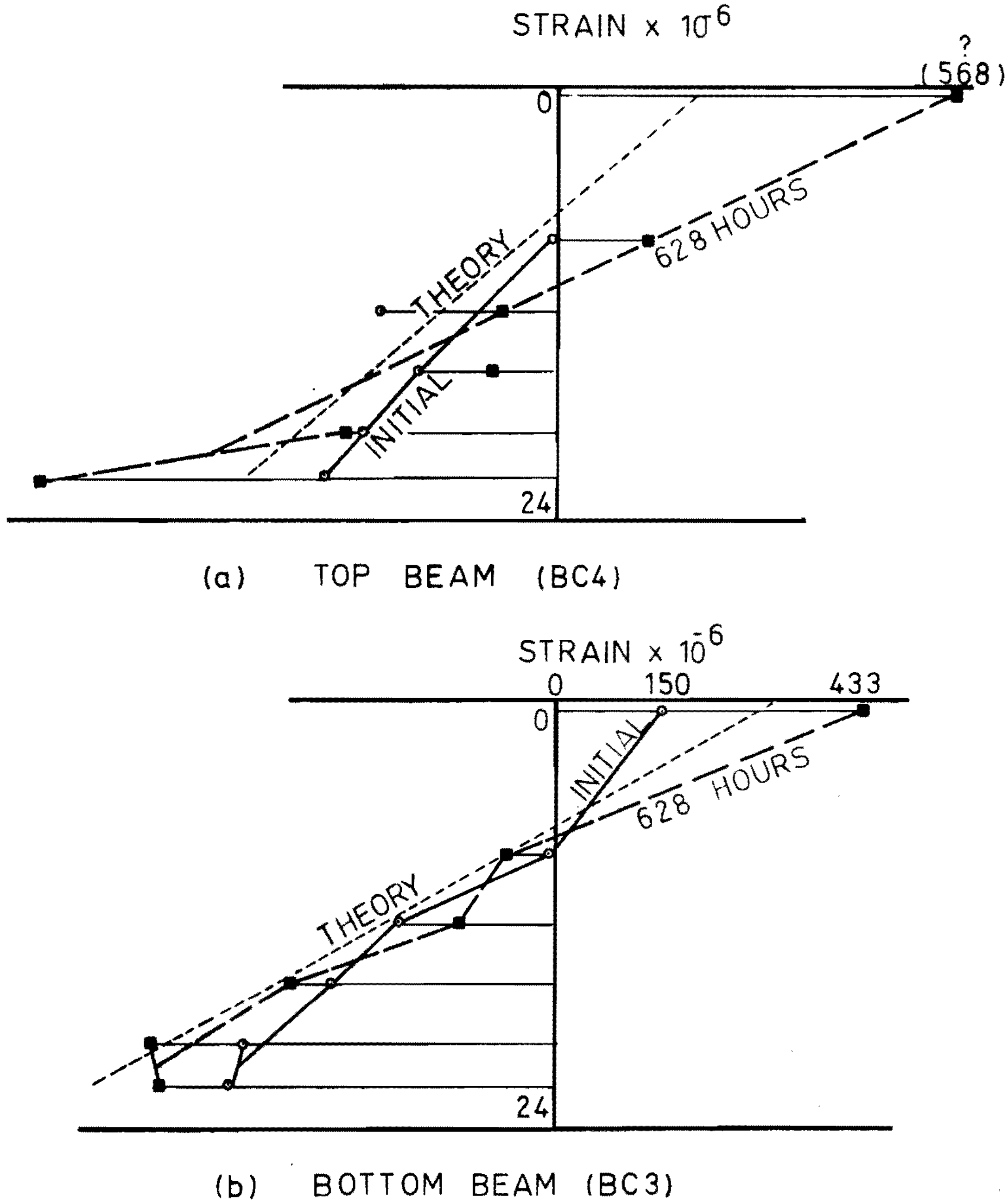


FIG. 22. STRAIN PROFILES—BEAMS LOADED AT 8-DAY AGE.

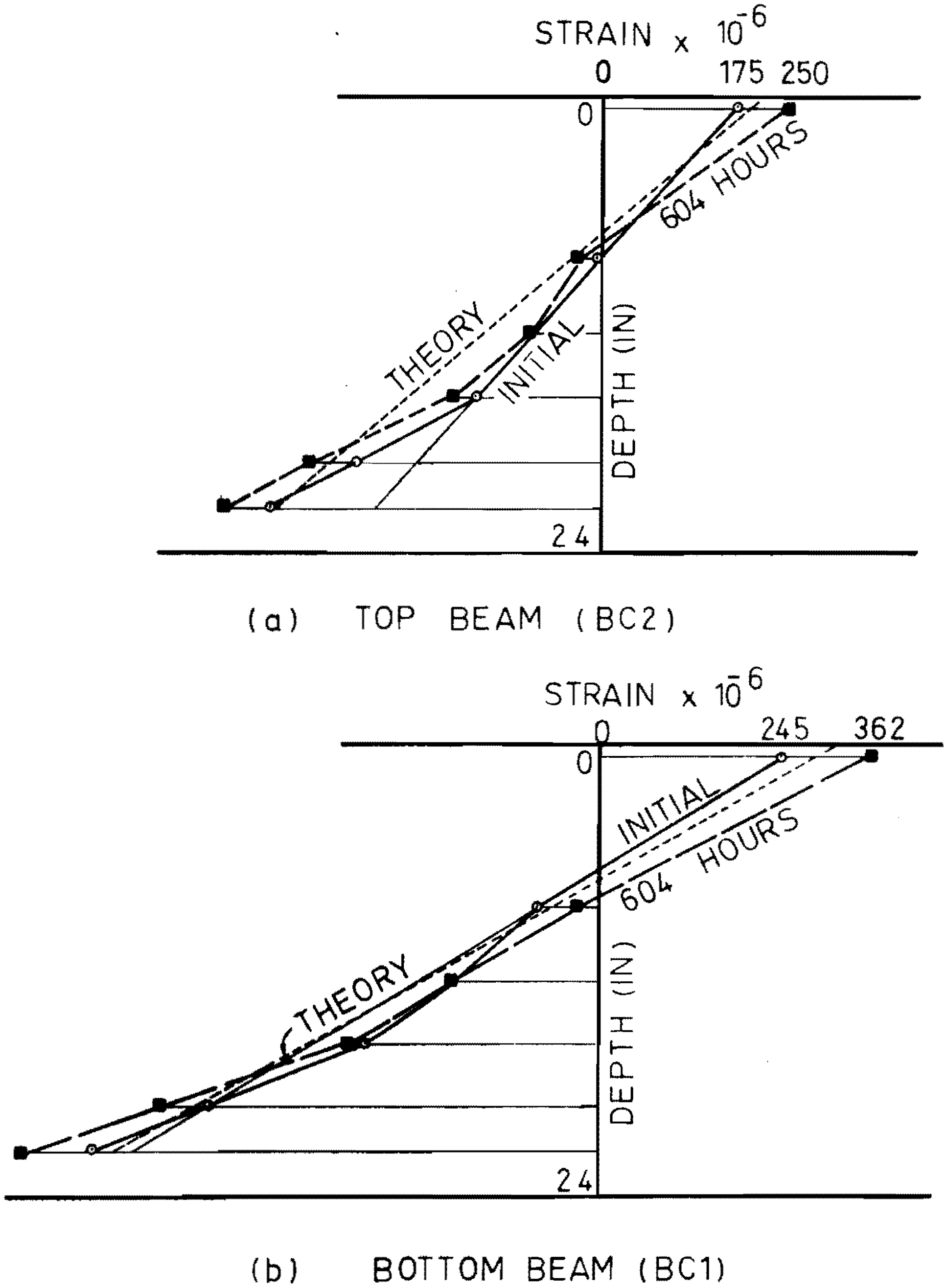


FIG. 23. STRAIN PROFILES – BEAMS LOADED AT 32-DAY AGE.

in those specimens loaded at the earliest age. Figure 20 indicates that in the specimens loaded at the age of 3 days the measured tension strains initially were 50 percent greater than the tension strains obtained by extending the linear strain profiles from the compression zone. In contrast, Fig. 23 shows that the measured tension strains in the specimen loaded at an age of 32 days were less than 20 percent greater than the tension strains corresponding to a linear extension of the compression strain profile.

Increases in the amount of compression strain were expected for all specimens subjected to a load sustained over a long period of time. As a rule of thumb, long-time deflections in reinforced concrete beams can be taken as twice those expected during a short time under load.[5] Figures 20 through 23, for specimens which contained tension steel at 17 ksi, indicate after 400 hours under load an increase in compression strain approximately 60 percent of the strain at the time loads were first applied. Specimens which contain steel at a stress of 12 ksi indicate the same order of magnitude for the increase of compression strain after more than 400 hours under sustained load. Each figure indicates also that after 400 hours under load, each specimen tended to develop a linear variation of strain from top to bottom. The variation of strain through the depth of the beam in each case was more linear after creep had occurred.

Gap Strains

Changes in the dimensions across the gaps at which tension reinforcing bars were exposed were detected by means of an inside caliper dial. All data reported here represent the average of the readings for the 14 measuring points, 7 at each of two gaps per specimen. In all cases the gaps tended to become wider during the first few hundred hours under permanent load, but thereafter the gap sizes remained relatively constant, decreasing slightly during the remaining 1500 to 1800 hours under load. Graphs that display changes in gap strain with time are given for each specimen in Figs. 24 through 27. Shown on the same graphs are the curves for surface strain at the centroid of the bars through the same gaps.

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[5] ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63), American Concrete Institute, Detroit, p. 40.

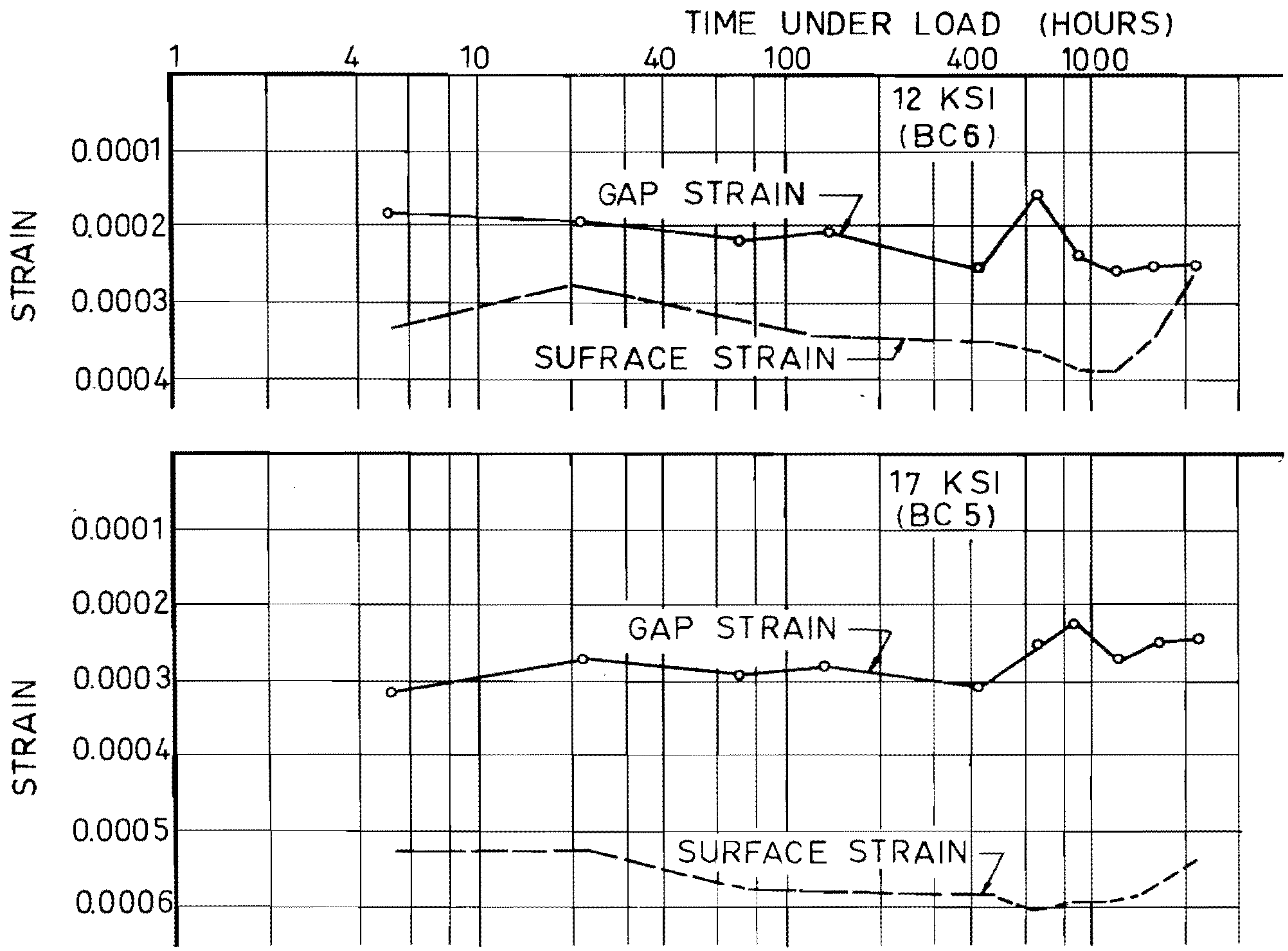

FIG. 24. STRAINS AT TENSION REINFORCEMENT -3 DAY LOADING AGE.

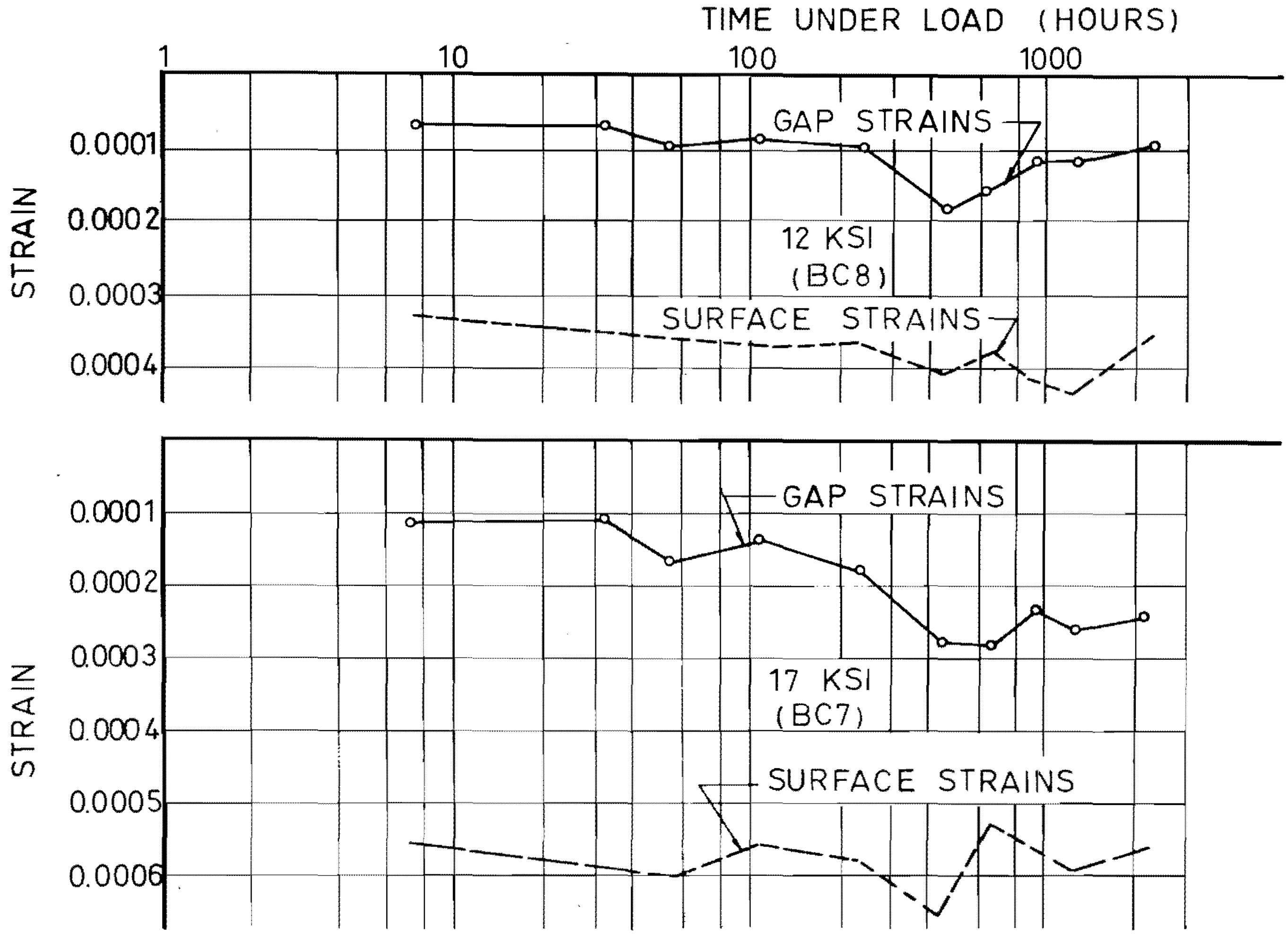

FIG. 25. STRAINS AT TENSION REINFORCEMENT - 4 DAY LOADING AGE.

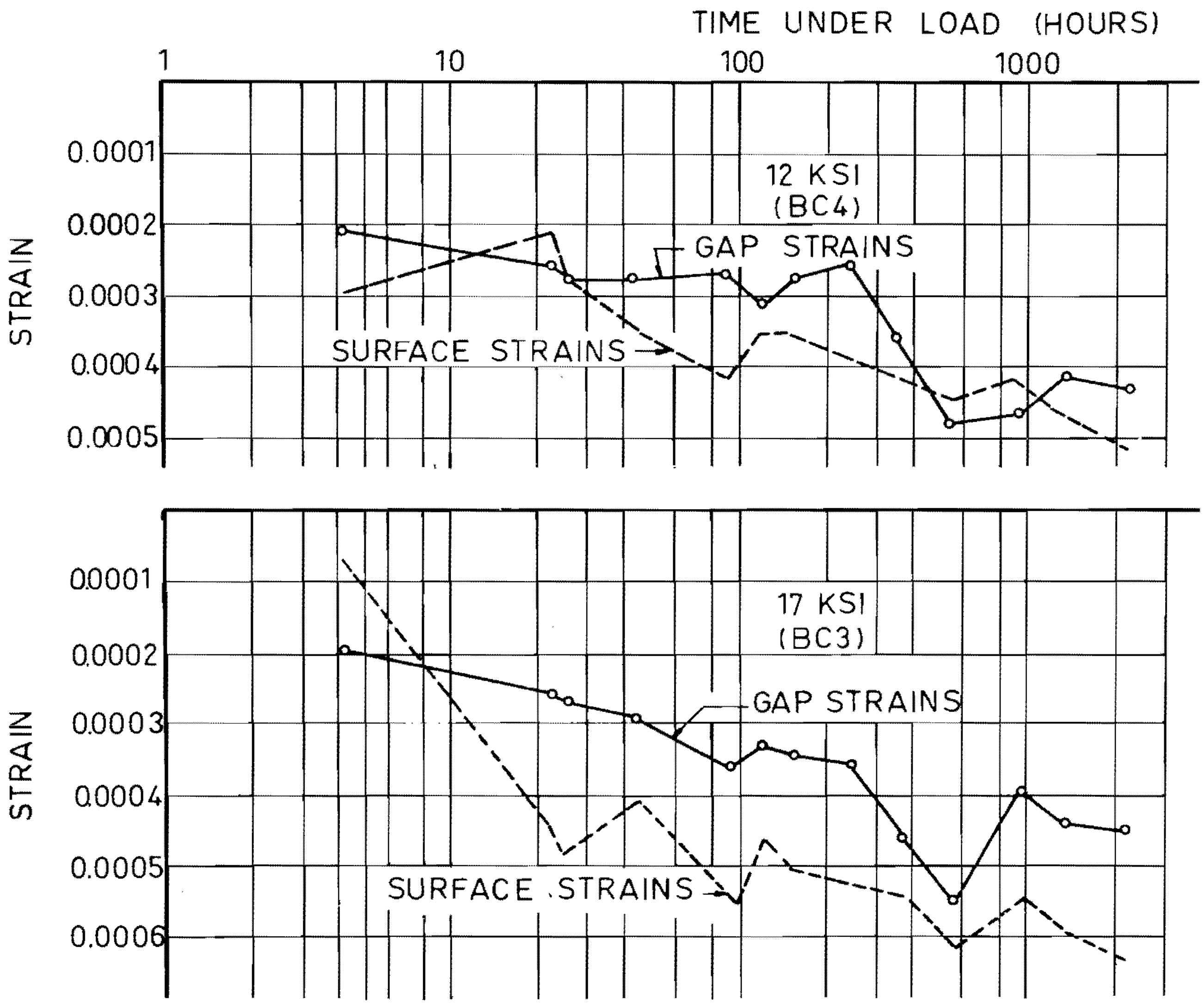

FIG. 26. STRAINS AT TENSION REINFORCEMENT -8 DAY LOADING AGE.

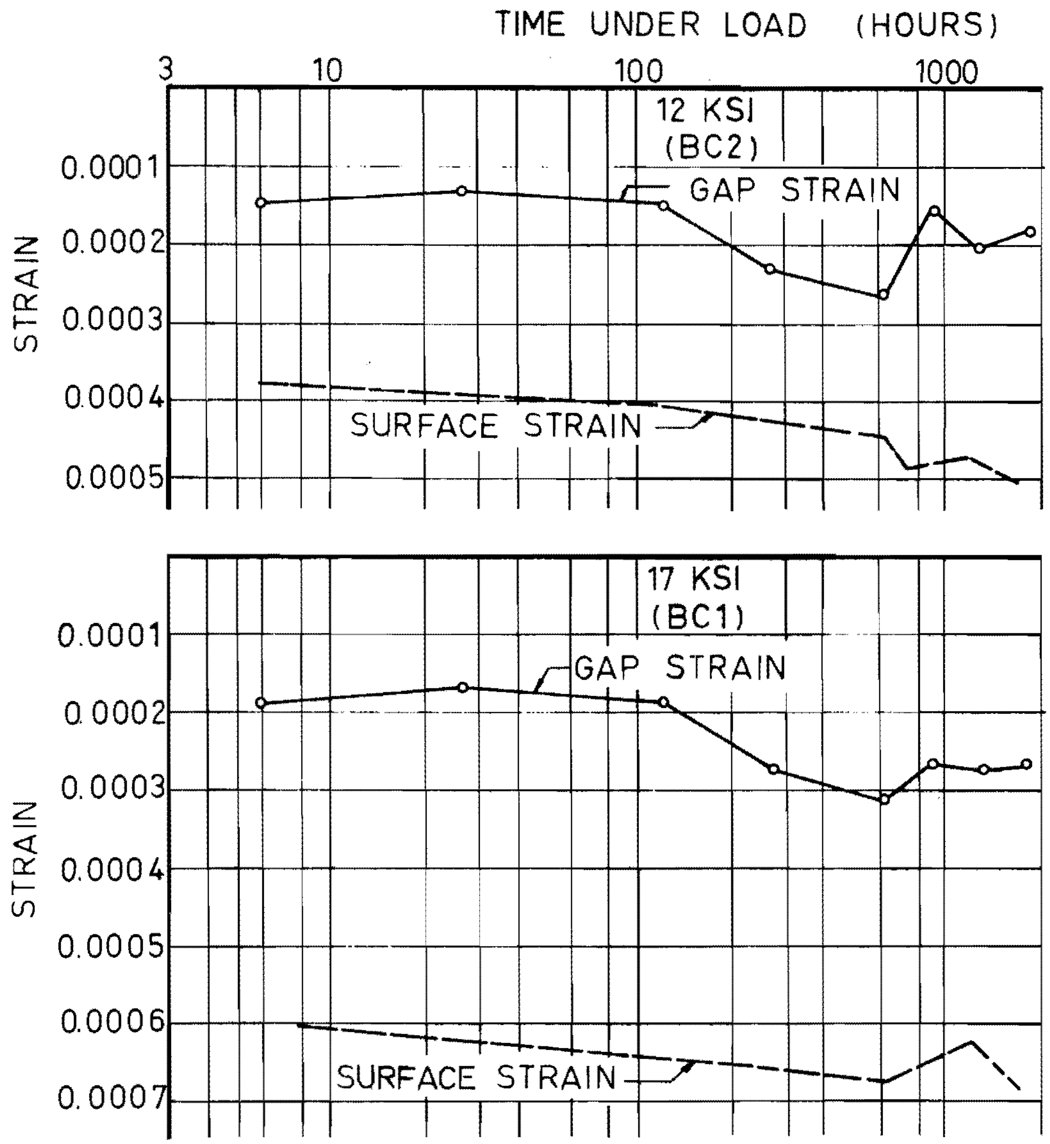

FIG. 27 STRAINS AT TENSION REINFORCEMENT 32 DAY LOADING AGE

The change in strain across the gaps was in all cases smaller than the surface strains alongside tension steel. For specimens loaded at an age of 3 days, gap strains shown in Fig. 24 were only one-third those measured at the surface of concrete. For specimens loaded at an age of 4 days, gap strains at 12 ksi steel were about two-thirds the surface strains, and gap strains at 17 ksi steel were about one-half the surface strain shown in Fig. 25. Figure 26 shows gap strains almost the same as surface strains in the specimens loaded at an age of 8 days, but for the specimens loaded at 32 days, Fig. 27 shows gap strains about one-third to one-half the surface strains.

In all cases gap strains and surface strains appeared to change at about the same rate under the sustained loads. If any bar slippage or creep due to bond took place, the rate at which gap strains changed should have differed from the rate at which surface strains changed. With the exception of data for specimens loaded at an age of 8 days, there were only small changes in <u>any</u> measured strains overall near tension reinforcement.

Major flexural cracks occurred directly above gaps for all specimens except those loaded at an age of 8 days, as shown by photographs of Figs. 12 through 15. Since the major flexural cracks relieved tensile stretching of concrete at the gaps, the extension of the gap (measured between faces of concrete) should be less than the overall extension of the reinforcement. In the absence of main flexural cracking above gaps in specimens loaded at 8 days (Fig. 14), faces of concrete at gaps separated almost the same moment as the extension of steel bars.

<u>Beam Deflection</u>

The deflection of beams was measured along a 6-ft. distance in the constant moment region. A summary of results from deflection data is presented in Table V. Deflections in Table V refer to the deflections at midspan from a chord connecting supports for the deflection dial measuring bridge. The measuring system was not a satisfactory mechanism for accurate data. Since the gage frequently was moved about and dial indicator plungers reacted against pads not absolutely level, much of the data was erratic. Curves had to be fitted to deflection data plots in order to obtain the information in Table V.

TABLE V. DEFLECTIONS IN A 6-FT. REGION AT MIDSPAN

Age at Time of Loading (Days)	Steel Stress (ksi)	Initial Deflection (in.)	Time of Load Until Second Reading (hours)	Second Reading Deflection (in.)
3	12	0.023	410	0.029
4	12	0.020	410	0.026
8	12	0.029	604	0.031
32	12	0.023	404	0.028
3	17	0.058	410	0.067
4	17	0.041	410	0.047
8	17	0.040	604	0.051
32	17	0.036	404	0.050

The specific values of deflections reported in Table V are known to be questionable; therefore, the data are presented here only to reveal trends of behavior. A comparison of deflections for the specimens loaded at 3 days and those loaded at 32 days after casting indicates insignificant differences for the beams containing 12 ksi tension steel. In beams containing 17 ksi tension steel, the specimen loaded at an age of 3 days deflected 35 to 50 percent more than did the specimen loaded at an age of 32 days.

Deflections should have been monitored with a permanent, stationary system, and the amount of deflection should have been measured over a longer length of each beam.

For the beams observed in this series of tests, the level of compression stress was not great (between 600 psi and 1000 psi), and creep due to compression in concrete permitted almost 100 percent increase in compression strains after 2000 hours. The deflection measuring system failed to reveal overall increases in deflection corresponding to the observed changes in compression strains, and the strain profiles in Figs. 20 through 23 do not indicate large overall changes of curvature.

Summary of Results

The initial application of load caused fewer cracks in specimens 3 days old than in specimens 32 days old. After the specimens 3 days old at the time of initial load had remained under load for 30 or 40 days, they displayed almost the same size and number of cracks as did the specimens 32 days old when first loaded. It was felt that the 3-day-old concrete had not experienced much shrinkage. Even though its tensile strength was due to increase somewhat with age and further curing, the increase in tensile stress caused by shrinkage during curing time appeared to contribute as much to cracking as did the tension stress in concrete caused by flexural loading. The absence of some 50 percent of flexural cracks at the time of initial loading helped keep deformations almost as low for specimens loaded at the 3-day age as for specimens loaded at the 32-day-age.

At the gaps or openings in concrete at tensile bars, the measured extension of the gaps when load was applied was noticeably smaller than the average extension of the tensile region in the remaining parts of the specimens. As each reinforcing bar stretches due to flexure, the adjacent concrete must stretch similarly or else it cracks (ruptures in tension). The measured surface strains were in the order of 600 microinches for 18 ksi bars and 350 to 400 microinches for 11 ksi bars, and they corresponded well with average tension strains computed for reinforcement. The extension of gaps would suggest a much smaller steel strain, but actually the face of each gap tended to pull away with the bar as suggested in Fig. 28. Figure 28 illustrates why gaps apparently "opened" less than the bars were stretched. Nonetheless, the general trends or changes in gap strains closely parallels that of surface strains with time, indicating that no measurable change in bond occurred under the sustained load.

In specimens loaded at the 3-day and 4-day age, gaps appeared to open further than in the specimens loaded at an age of 32 days. Possibly the 32-day-old concrete was stronger and stiffer, tending to transmit shear stresses more effectively in order to "follow" the reinforcement out of the face of the gaps.

If there existed among specimens any basic difference in the transfer of tensile force from steel to concrete by bond, evidence of such difference

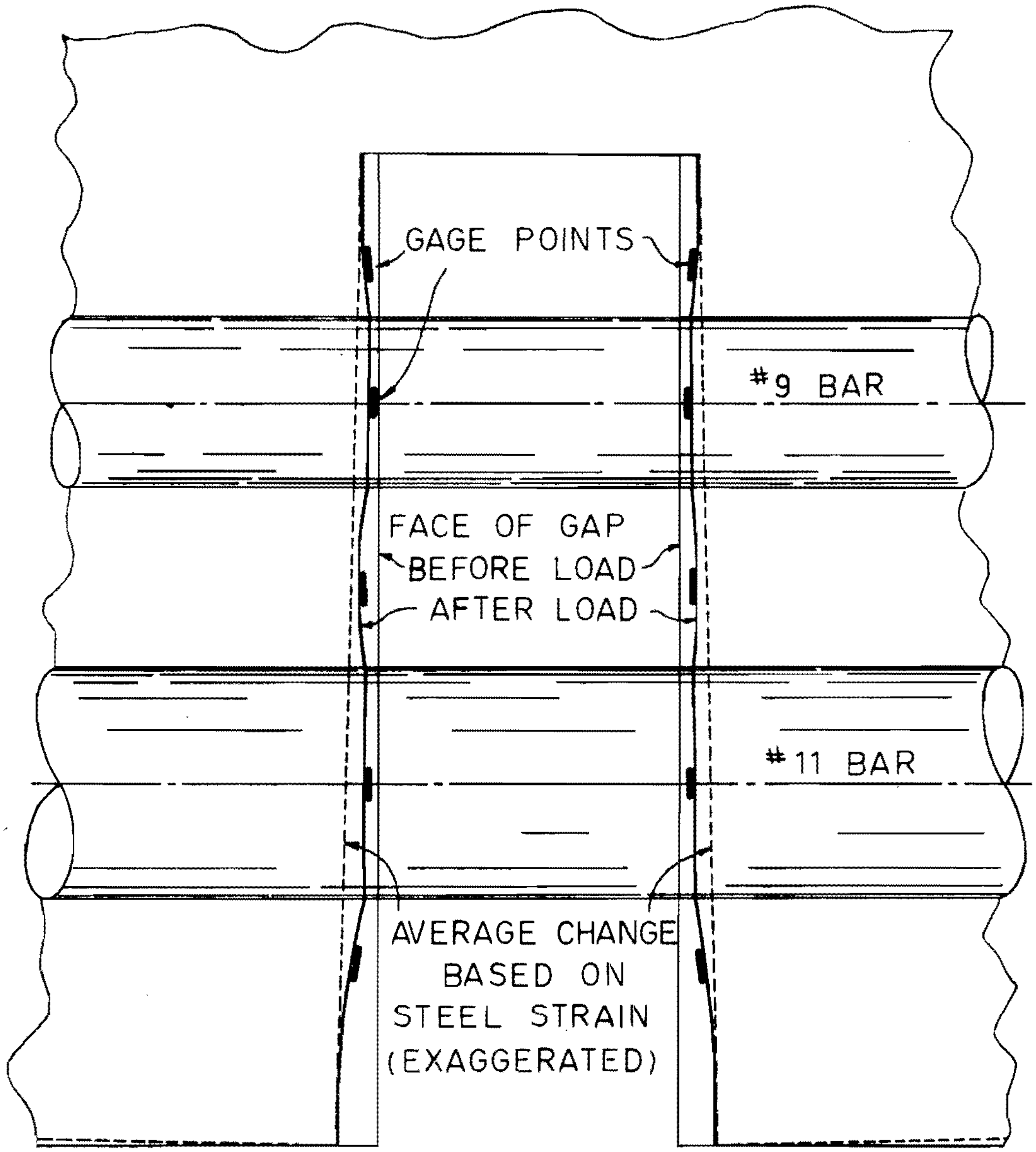

FIG. 28.- DEFORMATIONS AT GAPS.

should be revealed by comparing changes between gap strains and surface strains. The absence of discernible differences indicates instead that there was no evidence of creep due to bond. Indeed, there was no evidence that the age of concrete at the time of loading had any influence at all on the mechanism of bond.

The strain profiles given in Figs. 20 and 21 indicate that specimens loaded at the early age of 3 or 4 days experienced a variation of strain which was not linear through the depth of each beam. Under initial loads, strains of tension steel were disproportionately higher than compressive strains. Specimens 8 and 32 days old at the time of initial loading show in Figs. 22 and 23 somewhat less nonlinear strain variation with depth. The apparently high tensile strains under initial loads might be attributed to a bond softness or even some slip in immature concrete at the time of loading.

Deformations in beams loaded at an early age appear to be larger than those of beams loaded at later ages for two reasons observed in these tests:

(1) Concrete cured for only 3 or 4 days is not as stiff as it will be at a later age. Initial deformations are recoverable only if initial load is removed as concrete hardens.

(2) Some softness in concrete at bar lugs may result in a bond softness that permits an apparent initial slip in immature concrete loaded too early. Unless initial slip were followed by progressive increases in slip, creep due to bond cannot be said to exist. No evidence of change in slip was detected.

CHAPTER IV

CONCLUSIONS

In an effort to determine whether creep due to bond between concrete and steel reinforcement contributed to undesirably high deflections in metal pan T-beams, eight beams with a T-shaped test region were subjected to loads that were maintained for at least 2000 hours. Half of the beams were loaded to produce a tensile stress of 12 ksi, and the other half were loaded to produce a tensile stress of 17 ksi in the flexural steel. Pairs of specimens, one specimen with each steel stress, were loaded when the beams were 3 days, 4 days, 8 days, and 32 days old.

During the period in which permanent loads were maintained, deformation readings were taken at the surface of concrete and across a 2-in. gap in which concrete was omitted and steel bars were exposed.

An attempt to measure deflection across a 6-ft. section of each beam subjected to constant moment was not successful. The movable deflection measuring system was itself too flexible, and the 6-ft. test length was too short to give precise results.

On the basis of data reported here, the following conclusions were reached:

(1) There was no discernible evidence of creep due to bond or anchorage slip during the time in which loads were maintained on each specimen.

(2) Upon initial loading, all specimens exhibited at the level of tensile reinforcement strains higher than those consistent with a linear variation of strain through the depth of the beams. The specimens loaded at the earlier ages displayed a larger amount of nonlinear strain than did the specimens loaded at later ages.

(3) If the larger, nonlinear strains at reinforcement are regarded as a bond slip or softness, a significant part of excess deformation in specimens loaded at an early age could be attributed to such bond softness.

(4) Concrete loaded at the age of 3 or 4 days was significantly weaker and less stiff than 8-day-old and 32-day-old concrete. Beams of softer concrete should be expected to deflect more than beams of stiffer concrete.

(5) Specimens loaded at an early age initially displayed fewer cracks than corresponding specimens loaded at the 32-day age. Eventually new cracks developed such that the same crack distribution appeared in all specimens after 2000 hours under load. The absence of initial cracks had the effect of stiffening early loaded beams.

(6) Cracks were thought to develop and spread as shrinkage proceeded with drying of beams in the arid atmosphere of the laboratory during the summer. Possibly some part of excess deflection in specimens loaded at an early age could be avoided if beams were kept wet for several days after forms are removed.

Recommendations

Further studies directed toward optimizing the removal time of metal forms should explore the following:

(1) In specimens loaded at an age of 2 to 4 days, compare the deformation of wet and dried specimens. If larger deformations can be avoided by wetting, the advantage should be demonstrated.

(2) Test specimens should be made with intentional excesses of air entraining additives to measure the worst conditions of any pozzolanic action that restrains hardening of the specimens.

(3) Total deflections should be measured over a specimen length of at least ten times the depth.

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