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# PRESTRESS LOSSES IN PRETENSIONED PRESTRESSED CONCRETE MEMBERS WITH BENT TENDONS

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## NOMENCLATURE

| | |
|---|---|
| $b$ | Width of concrete section |
| $d$ | Total depth of section |
| $A_c$ | Area of entire concrete section |
| $A_s$ | Area of the prestressing tensile steel |
| $A_t$ | Area of the transformed section |
| C.G.C. | Center of gravity of the concrete section |
| C.G.S. | Center of gravity of steel area |
| C.G. | Center of gravity of the transformed section |
| $y_o$ | Distance between C.G. and C.G.C. |
| $e$ | Eccentricity of C.G.S. with regard to C.G.C. |
| $e_t$ | Eccentricity of C.G.S. based on the transformed cross section |
| $I_t$ | Moment of inertia about the centroid of the transformed cross section |
| $i_t$ | Radius of gyration of the transformed section |
| $F_o$ | Original prestressing force in the cables |
| $F_i$ | Initial prestressing force after transfer of prestress |
| $F$ | Final prestressing force, after all losses |
| $M_x$ | Dead load bending moment at the section considered |
| $w$ | Dead weight of beam per unit length |
| $L$ | Length of beam |
| $x$ | Distance along the length of beam from its end |
| $E_c$ | Modulus of elasticity of concrete |
| $E_s$ | Modulus of elasticity of steel |
| $n$ | Poisson's ratio equals $E_s/E_c$ |

| | |
|---|---|
| $f_{ci}$ | Compressive strength of concrete at time of initial stress (at transfer) |
| $f'_c$ | Compressive strength of concrete at 28 days |
| $f'_s$ | Ultimate strength of prestressing steel |
| $f_s$ | Original stress of the prestressing steel |
| $f$ | Unit normal stress |
| $f_{c.g.s.}$ | Stress at the level of C.G.S. |
| $\epsilon$ | Unit strain |
| $\epsilon_{c.g.s.}$ | Strain at the level of C.G.S. |
| $\epsilon_i$ | Instantaneous strain at transfer |
| $\epsilon_t$ | Creep plus elastic strain at time t |
| C | Creep coefficient equal to $\epsilon_t/\epsilon_i$ |
| r | Ratio of ultimate creep to elastic strain of concrete at transfer |
| P.C. | Prestressed concrete |
| ACI | American Concrete Institute |
| ASCE | American Society of Civil Engineers |
| ASTM | American Society of Testing and Materials |

Other terms are defined, where they first appear

# CHAPTER I

# INTRODUCTION

## 1.1 General Background

The rapid growth in the use of prestressed concrete, especially precast prestressed concrete, in the last decade is one of the most important developments in the construction industry. The impact of this growth - involving theory, design, materials and applications - has been worldwide. The general trend now in the construction industry is toward the ultimate use of prefabricated architectural and structural components. Close supervision and control of materials and a specialized work force in a centralized plant is conducive to high quality product. Except for very large, unwieldly members, the most common method of prestressing is pretensioning, because of its adaptability to mass production in a plant. Plant production is not normally subject to delays due to adverse weather conditions, as often happens to job site operations.

A basic factor to the good design of every prestressed concrete member is an accurate determination of the prestress losses which will occur in that member. In spite of the importance of determining these losses, the several prestressed concrete design codes currently in use in the United States and abroad differ widely and are vague on the methods and criteria to be used by the designer in determining prestress losses. This divergence of opinion among the authorities who issue these codes, indicates the need for further experimental and theoretical studies of these losses.

## 1.2 Definitions

The following definition of prestressed concrete is given by the American Concrete Institute Committee on Prestressed Concrete:

> "Prestressed concrete. Concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from the given external loading are counteracted to a desired degree. In reinforced-concrete members the prestress is commonly introduced by tensioning the steel reinforcement."

There are two methods of application of the prestress: (a) pretensioning in which the reinforcement is tensioned before the concrete is placed, and (b) post-tensioning in which the reinforcement is tensioned after the concrete has hardened. The reinforcement may be either bonded throughout its length to the surrounding concrete, in which case it is called "bonded reinforcement", or not bonded to the concrete in which case it is called "unbonded reinforcement". If the reinforcement is provided at its ends with anchorages capable of transmitting the tensioning forces to the concrete it is called "end-anchored reinforcement".

The layout of a prestressed-concrete beam is controlled by two critical sections: the section of maximum moment and the end sections. Bent tendons may be used to satisfy the allowable stresses at these two critical sections. Modern pretensioning plants have buried anchors along the stressing beds so that the tendons for a pretensioned beam can be bent.

This investigation deals with pretensioned prestressed straight beams with bent tendons.

## 1.3 Prestress Loss in Pretensioned Prestressed Members

Loss of prestress force in prestressed pretensioned concrete members is normally attributed to elastic shortening of concrete, creep of concrete, shrinkage of concrete and creep or relaxation of prestressing strands. The definitions of these terms together with their effect on the prestress loss will be fully explained in Chapter II of this thesis entitled "Theoretical Nature of Prestress Loss".

## 1.4 Historical Background of Prestress Losses

Several investigations have been made quantitatively on the creep of ordinary sand and gravel concrete under sustained loads. The principal investigators are R. E. Davis and H. E. Davis[84,85,86,87,88,89] of the University of California who conducted extensive tests in 1925-1937 to show the effect on the creep of the following: the richness of the mix, gradation of the aggregates, mineral compositions of the aggregates, age at time of applying load, condition of storage as regards moisture, magnitude of stress, reinforcement and alternately applying and releasing loads. Since then other investigators have made extensive tests of creep of concrete cylinders and prisms but very little has been used in predicting the prestress loss due to creep of concrete[90].

Magnel[90] published in 1947 the results of his tests on the plastic flow of sand and crushed stone concrete and the creep characteristics of three kinds of high tensile wire commonly used in Belgium. Based upon his tests, he found that the average plastic flow coefficient is $C = 2.12$. However, he recommended the use of a value of $C = 2.2$. The plastic flow coefficient or creep coefficient, as Magnel calls it, is the ratio of the total strain $\epsilon_t$ attained when the load is kept constant during a certain time, t to the instantaneous strain $\epsilon_i$ of a concrete prism loaded axially up to a certain stress. Thus, $C = \epsilon_t/\epsilon_i$ . Magnel suggested a 15% prestress loss due to creep and shrinkage of concrete.

Recently C. Z. Erzen[48] gave an expression for creep and discussed its applications to prestressed concrete. He showed that the losses due to creep in prestressed concrete beams may be evaluated if the variation of the modulus of elasticity of concrete with time and the creep expressions are given. The problem necessitates the solution of an integral equation. K. Okada[44] discussed this expression and he feels that the Erzen method is not necessarily simple nor satisfactory. It is the writer's opinion that the Erzen expression could be made into a set of curves to facilitate its application.

B. A. Chowdhry[30] summarized the losses as follows: loss due to shrinkage 3.5%, loss due to creep of concrete 10% and loss due to creep of steel 4%, loss due to movement of anchorage 2.5%.

Concerning the prestress loss due to creep in steel, the editor of Reference 81 commented on certain tests by saying, "It appears that the effect of creep in steel can be almost annulled by temporary overstressing of the reinforcement". In Reference 54 the author shows how the loss due to creep in steel is increased by the use of lower strength steel, even though the loss of steel strength is compensated by the use of a larger cross-sectional area. Canta[49] conducted 30 creep tests with varying stresses over a wide range using three different kinds of steel. With cold-drawn steel wire, an almost linear relationship exists between creep strain and logarithm of time, as long as stresses are relatively low. Beyond this, the creep curves tend toward the time axis. Graphs illustrate the effect of stress on creep strain and strain rate. Importance of relaxation is also emphasized.

In 1959 Stussi[24] gave a long-time law for the relaxation of steel wires but this law was questioned by the Dutch Committee.[17]

A good last step in fabricating wire strand or wire is stress relieving.[91] Here the tendons are pulled through an air furnace or a lead bath, which has a temperature of 750 to 800°F, for a few seconds. This process does produce changes in the physical properties of the strand. It increases its ultimate elongation, removes some of the residual stresses and makes the strand much easier to handle on the job. Also, any oil, grease, or foreign materials are removed from the surface of the strands by this treatment. It does reduce the loss due to creep of steel up to 70% of the ultimate strength.

The amount of shrinkage strains varies with many factors[92] and it may range from none to .0010. There may be even an expansion for some types of aggregates and cements. The effect of shrinkage of concrete on the prestress loss has been investigated thoroughly in References 19, 33, 50, 51, 60, 69, 72, 73 and 83, but each gives a different value of shrinkage strain.

A series of tests has been conducted at the University of Michigan to investigate prestress loss.[11,12,13,94] This research will be a continuation of this study and some of the results of these investigators will be discussed later in Chapter VI.

Early attempts to induce a permanent prestress in concrete structures failed because of the major losses produced by plastic flow and shrinkage of concrete. It was not until Freyssinet, the eminent French engineer, discovered that by using high tensile steel with very high elastic limit these losses may be made much less significant compared with the original prestress.

It appears, although the causes of prestress loss are known, that a rapid and satisfactory procedure for evaluating such losses is not available. Perhaps the only kind of loss that could be directly estimated with a rather satisfactory formula is elastic loss. The other three factors are dependent on so many features. They are dependent, for example, on the properties of the materials used and the level of stress.

It appears, therefore, that a thorough study of these losses and their effect on prestressed concrete members is necessary, since a divergence of opinion among the authorities in this field is noted.

## 1.5 Description of the Testing Program

The present program consisted mainly of three test series. Each series consisted of casting a main prestressed concrete beam 18 feet long, a dummy shrinkage beam 4.5 feet long, (both beams are 5x9 inches in cross section), twelve 6x12 inch test cylinders and two dummy cylinders 6x4 inches.

The steel strands of the main prestressed concrete beam have end eccentricity different from the middle third of the beam eccentricity. The beams are symmetrical with respect to their middle section. The strands were made horizontal in the middle third and then inclined upward at the outer thirds of the beams. Each main prestressed beam has different end and middle eccentricities. The shrinkage beams have two unstressed strands located at two-thirds their depth.

The strains of the concrete and the steel were measured by means of Whittemore Extensometers. Brass plugs served as gage points. They were first glued to the side forms at their proper locations. When the forms were removed the exposed ends of the plugs served as gage points. Plugs on the steel are glued to it by means of epoxy type cement.

The cylinder creep test was made by loading three cylinders to a stress level equal to the average original stress of the corresponding prestressed beam. A creep rack assembly is used for such a test.

Shrinkage of concrete was measured by measuring the change of strain of unstressed dummy shrinkage beams and cylinders. The strain readings of the stressed beams and cylinders give the combined effect of creep plus shrinkage, while the dummy unstressed beam and cylinder strain readings give the effect of the shrinkage only. By subtracting the shrinkage effect from the combined effect of shrinkage and creep, one gets the creep effect.

A creep of steel test was also conducted. A strand of the same type used in the prestressed beams was stressed to a stress level equal to that used in stressing the steel of the main beams. A dynamometer was inserted between the end plate of a stressing frame and the strand chuck. This dynamometer was used to record the force in the prestressing strand versus time.

# CHAPTER II

## THEORETICAL NATURE OF PRESTRESS LOSS

### 2.1 What Causes Prestress Loss in Prestressed Pretensioned Concrete Members?

Loss of prestress force in prestressed pretensioned members is normally attributed to four major factors. These factors are (1) elastic shortening of concrete; (2) shrinkage of concrete; (3) creep of concrete; and (4) creep of prestressing strand.

A mathematical expression at the zone of complete anchorage is always available for determining the prestress loss due to the elastic shortening, while determining the losses due to the other factors is essentially empirical. Experimental work is the basis for determining these empirical factors.

### 2.2 Factors which Contribute to Losses

#### 2.2.1 Elastic Shortening of Concrete

When the prestress is transfered to concrete, the concrete member shortens and so does the prestressed steel. This shortening will cause loss of prestress and it occurs immediately after transfer. It depends mainly on the amount of prestress force, its eccentricity and the elastic moduli of concrete and steel at transfer.

Let

$F_o$ = Original prestress force in the cables

$F_i$ = Value of prestress force after transfer

$M_x$ = Bending moment due to the dead load of the member at the section considered

$I_t$ = Moment of inertia of the transformed section

$e_t$ = Eccentricity of the prestress force.

Then the stress in the concrete at the C.G.S. will be:

$$f_{c.g.s.} = -\frac{F_i}{A_t} - \frac{F_i e_t^2}{I_t} + \frac{M_x e_t}{I_t} \tag{2.1}$$

and the unit strain at the C.G.S. will be:

$$\epsilon_{c.g.s.} = f_{c.g.s.}/E_c \tag{2.2}$$

Since strain in concrete equals the change in strain in steel at C.G.S. Then, change in stress in steel due to elastic shortening of the member $= n\, f_{c.g.s.}$ or,

$$F_i = F_o - \text{unit elastic stress loss} \;\; x \;\; A_s$$

$$F_i = F_o - \left(\frac{F_i}{A_t} + \frac{F_i e_t^2}{I_t} - \frac{M_x e_t}{I_t}\right) n\, A_s$$

Solving for $F_i$

$$F_i = \frac{F_o + M_x e_t\, n\, A_s/I_t}{1 + n\, A_s \left(\frac{1}{A_t} + \frac{e_t^2}{I_t}\right)} \tag{2.3}$$

Total elastic loss $= F_o - F_i$ (2.4)

Equation (2.3) provides a good, rapid, direct way of estimating elastic loss of prestress force. Probably the only approximation in this equation is the value selected for concrete modulus of elasticity, since stress-strain curve for concrete is seldom a straight line, even at normal levels of stress.

2.2.2 Shrinkage of Concrete

Shrinkage of concrete is defined as its contraction due to drying and chemical changes, dependent on time and on moisture conditions but not on stress. At least a portion of the shrinkage resulting from drying of concrete is recoverable upon the restoration of the lost water. Thus total shrinkage and the rate of shrinkage are dependent upon the moisture gradient within the concrete. It is the moisture gradient which regulates the flow of water through the capillaries.

The shrinkage of concrete is almost directly proportional to the amount of water employed in the mix. The quality of aggregates is also an important factor. Harder and denser aggregates of low absorption and high modulus of elasticity will exhibit smaller shrinkage. The chemical composition of cement also affects the amount of shrinkage. Much experimental data is available on shrinkage of concrete of different kinds and mixes. The designer should select the shrinkage data derived from mixes which closely resemble the mix to be used and cured under conditions similar to the conditions to which the actual member will be exposed. Shrinkage strains will ordinarily vary from .0001 in/in to .0005 in/in for standard weight mixes.

2.2.3 Creep of Concrete

Creep of concrete is defined as its time-dependent deformation resulting from the presence of stress. When a concrete prism is loaded axially to a certain stress it has an instantaneous strain $\epsilon_i$. If the load is kept constant for a certain period the strain increases to $\epsilon_t$. The coefficient of creep is defined as $C = \epsilon_t/\epsilon_i$. This coefficient varies widely, depending upon the stress level, time, age of concrete at the application of stress, the quantity of mixing water, the strength of concrete, the quality of aggregates and cement, moisture content of concrete, the humidity of the ambient air, and the size of mass.

Very little information is known of the creep of concrete under higher stresses than those stresses allowed in reinforced concrete structures. Moreover, the information available is for creep of concrete under uniform stresses but not under normal prestressed conditions, i.e., where the prestress force is applied eccentrically with accompanying trapizoidal or triangular stress distribution.

Creep of concrete(14) may be due partly to viscous flow of the cement-water paste, closure of internal voids and crystalline flow in aggregates but it is believed that the major portion is caused by seepage of adsorbed water from the gel that is formed by hydration of the cement. Although water may exist in the mass as chemically combined water, and as free water in the pores between the gel particles, neither of these is believed to be involved in creep. The rate of expulsion of the colloidal water is a function of the applied compressive stress and the friction in the capillary channels. The greater the stress, the steeper the pressure gradient with resulting increase in rate of moisture expulsion and deformation. The phenomenon is closely associated with that of drying shrinkage.

It is common practice to assume that the total loss due to creep of the concrete is equal to a constant factor times the elastic losses. This constant factor may vary from 1 to 3, depending on the conditions which affect creep in each particular design.

2.2.4 Creep of the Prestressing Strand

Creep in steel is the loss of its stress when it is prestressed and maintained at a constant strain for a period of time. It can also be measured by the amount of lengthening when maintained under a constant stress for a period of time. Creep varies with steel of different compositions and treatments, hence exact values can be determined only by test for each individual case if previous data are not available. Creep depends also on the level of prestress. For lower stresses creep of steel could be negligible. For stresses higher than 0.55 $f_s'$ creep is known to be increasing rapidly. The original stress of prestressed strands is usually 0.7 $f_s'$ and thus the creep due to steel strands should be taken into consideration.

Approximate creep characteristics, however, are known for most of the prestressing steels now on the market.(15) Compared to stress-relieved wire, the "as drawn" wire has somewhat higher creep. Prestretched wire will have about 2 to 3% creep when subject to 0.5 $f_s'$, but when stressed to 0.7 $f_s'$ the creep will still be no more than 5%. Time-temperature treated wire has practically no creep when subject to 0.5 $f_s'$. At 0.6 $f_s'$ it has slightly more creep than "prestretched" wire, and at 0.7 to 0.8 $f_s'$ the creep becomes excessive. Galvanized wires have about the same creep characteristics as the time-temperature treated wires, and should preferably not be subjected to any stress above 0.6 $f_s'$ without carefully considering the effect of creep.

Creep in stress-relieved strands was determined by W. O. Everling of U. S. Steel Corporation, Cleveland, Ohio, 1953-1955. In general, their characteristics are similar to those of stress-relieved wires. For high tensile bars, some limited tests seemed to show that, for stress up to about 0.55 $f_s'$, creep is not more than 5%.

Creep is most probably due to changes along grain boundaries as a result of plastic deformation of the relatively weak matrix. The matrix is thought to be viscous in nature.

It is a generally accepted fact that total creep of prestressing strands can be reduced by 30 to 60% by overstressing the strand from 5 to 10 per cent for a period of 2 to 3 minutes before anchorage.

# CHAPTER III

## MATERIALS, EQUIPMENT AND TECHNIQUES USED IN THE EXPERIMENTS

### 3.1 Sequence of Operations

The sequence of operations involved in making the beams and the cylinders necessary for each test series is as follows.

1) Cement the plugs on the wooden forms and the plugs on the cables. This was usually done four days before tensioning the cables.

2) Tension the steel strands. This operation was usually performed four days before placing the concrete.

3) Cast the main beam, the shrinkage beam, twelve 6 x 12 inch cylinders and two 6 x 4 inch cylinders. After approximately four hours, moist cure with wet burlap until the concrete attains its desired strength.

4) Strip forms and commence marking and gaging the beams while the concrete is curing.

5) When the concrete attains its desired strength, take initial set of strain readings.

6) Transfer the prestress force and take another set of strain readings.

7) Move to the storage area.

8) Make the concrete cylinder creep test.

## 3.2 Concrete

The mix selected for these experiments was a mix recommended by the Michigan State Highway Department for use in prestressed concrete bridge members. The mix proportions for a one cubic yard batch were specified to be as follows:

| | |
|---|---|
| Cement | = 658 lb. (7 sk./c.yd.) Type III (High Early Strength Cement) |
| Sand (dry) | = 1180 lb. (Glacial sand) |
| Gravel (dry) | = 1850 lb. (Glacial gravel) |
| Water (net) | = 235 lb. |
| Pozzolith #8 | = 1.82 lb. |
| Vinsol | = .07896 lb. (.012% by wt. of cement). |

This mix was designed to give a compressive strength $f'_{ci}$ = 4000 psi at 48 hours and $f'_c$ = 5000 psi at 28 days. Unfortunately, the laboratory was not equipped with a mixer big enough to mix a half cubic yard of concrete at one time, which is the amount needed for each test series. Therefore the concrete was ordered ready mixed from a commercial firm which used Type I cement (ordinary cement) instead of Type III as specified. This resulted in delaying the cutting of the steel strands for about three days until the desired strength of the concrete cylinders was achieved. Placement of the concrete in the forms was facilitated by means of an internal vibrator.

Each series consisted of casting a main prestressed concrete beam 18 feet long, a dummy shrinkage beam 4.5 feet long (both beams are 5 x 9 inches in cross section), twelve 6 x 12 inch test cylinders and two 6 x 4 inch dummy cylinders which were needed for the cylinder creep test as will be explained later.

As soon as the concrete took its initial set (4 to 6 hours) the beams and cylinders were covered with wet burlap. This damp cure was maintained until the cables were cut. Great efforts were made to keep the burlap wet to avoid shrinkage strains prior to cutting the cables.

The strength of the concrete cylinders was checked intermittently until the desired strength was achieved. This took about four days.

Three 6 x 12 inch concrete cylinders and two 6 x 4 inch dummy cylinders were needed for the concrete cylinder creep test. Another two were needed for measuring the shrinkage strain of the concrete cylinders. Brass plugs were attached by means of screws to the inside of the forms for these five 6 x 12 inch cylinders at their proper locations. Each cylinder had four plugs, two on opposite sides of the cylinder. Each two formed a vertical gage length of 10 inches, the upper plug was placed one inch from the top of the cylinder, and the other at 11 inches. After the

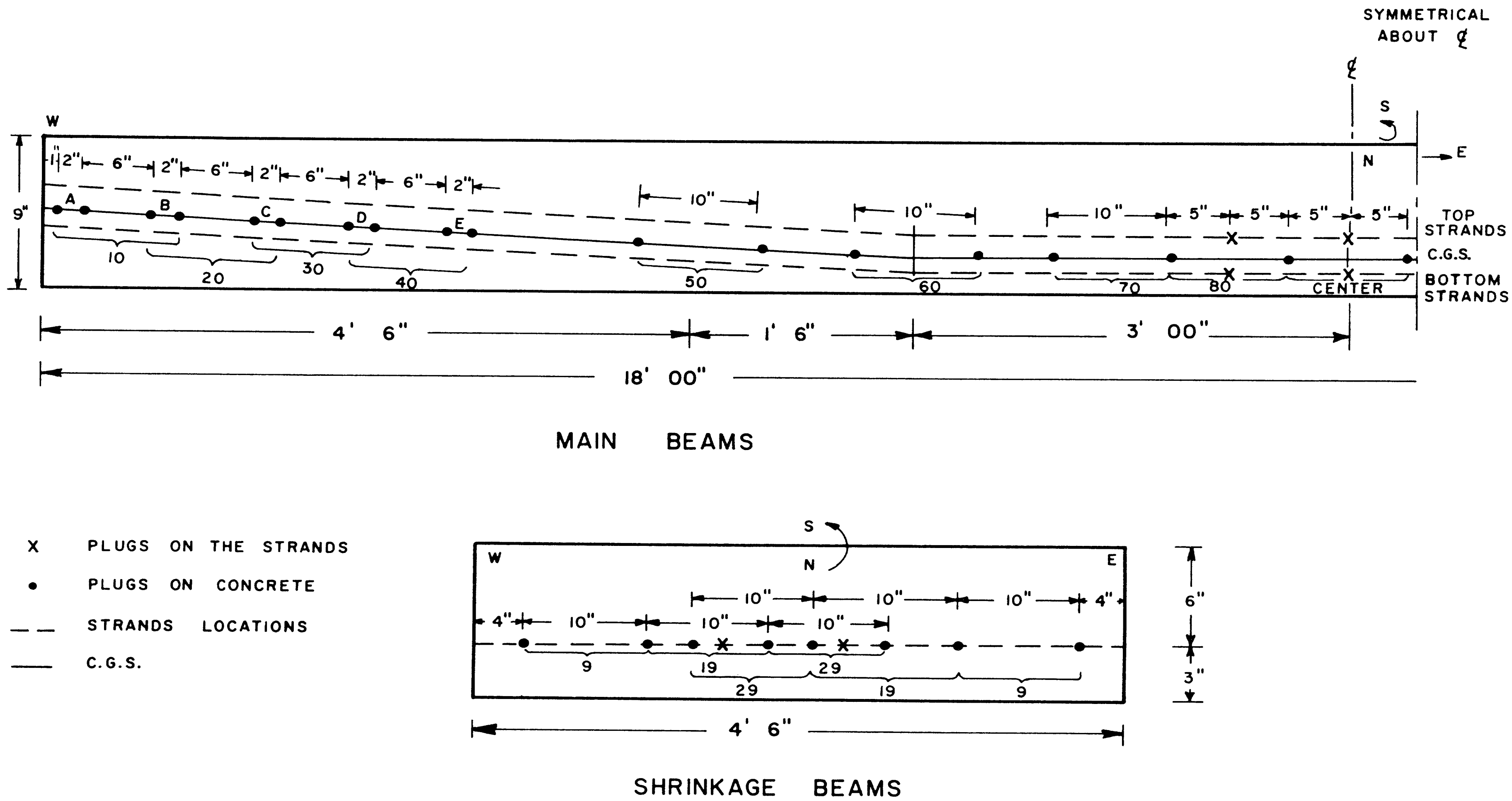



Figure 3.1 Whittemore Plugs Locations.

concrete had hardened and the forms were removed, the screws were taken off and were replaced by other stainless steel screws with punchmarked heads drilled and reamed to fit the points of the Whittemore gages.

Whittemore plugs locations on the main prestressed concrete beams and the shrinkage beams are shown in Figure 3.1. Gages were located on both sides of the members at the elevation of the C.G.S. The average of four symmetrical locations was considered in the analysis of the results.

Two gage lengths were used, one having a 10 inch gage length and the other having a 2 inch gage length. These 2 inch gages were located near the ends of the beams and their function was only to measure the elastic strains near the ends immediately after transfer.

The gage points consisted of 3/8 inch x 1/4 inch tapered brass plugs which were glued to the side forms at their proper locations. These plugs were punchmarked and then drilled and reamed to fit the points of the Whittemore gages. The holes of these plugs were then filled with "plastic clay" so that no glue would get into them and make it difficult to get them cleaned and ready for measuring. The holes were on the faces which were in direct contact with the inside of the wooden forms. These plugs were embedded in the concrete. When the forms were removed the exposed ends of the plugs served as the gage points. The holes were then cleaned to remove the plastic clay. Before cutting the cables the gage lengths were checked by the Whittemore gages and occasionally drilling new holes was necessary. Great care should be taken in tapping the punch for these new holes to prevent loosening the plug from its bond with the concrete.

Care should be exercised, when vibrating the concrete in the forms, to prevent the vibrator from striking any of the plugs. Some plugs were lost for this reason and other plugs were later installed. This was accomplished by drilling a hole in the concrete, at the proper location, just large enough for a new plug and cementing the plug to the concrete by means of hydro-stone. Hydro-stone dries fast and thus will not affect the shrinkage strain of the concrete later.

A great deal of care is required in the gaging operation to achieve good, reliable strain readings. The main precautions are as follows: 1) When drilling the plugs it is essential to drill the holes perpendicular to the plane of the plug's face. Also, it is necessary to drill the holes deep enough to clear the points of the Whittemore gages. 2) Reaming the holes in order that they will fit the Whittemore gages exactly.

Duco cement, of the type used for cementing electrical strain gages, works best for cementing the brass plugs to the forms. The wood forms should be clean and free from oil, paint or old cement. It is best to apply an initial coat of cement to the wood only and work it thoroughly into the grains by hand rubbing. After this coat has dried for about 30 minutes or more, another coat of cement was applied to both the wood and the brass plug, and the plug then set in place. About 24 hours should be allowed for the cement to harden before assembling the forms.

## 3.3 Concrete Cylinder Creep and Shrinkage Test

Creep of concrete under sustained stress, equaling the average compressive stress on the corresponding P.C. beam of the same series $F_o/A_t$ , was determined on 6 x 12 inch concrete cylinders. Companion specimens were fabricated for drying shrinkage under no load. Creep values were obtained by subtracting the drying shrinkage under no load from the length change of the specimens under sustained load. Cylinder ends were capped with sulfer mixtures and were made perpendicular to the long axis prior to assembly in the spring loaded rack shown in Figure 3.2. Length changes were measured with Whittemore extensometer over a 10 inch gage length provided by embedded reference plugs. The concrete cylinder creep and shrinkage tests were conducted in the same storage area as the P.C. beams. There was no way of controlling the temperature or humidity of this storage area.

The creep rack assembly, as shown in Figure 3.2, consists of five circular plates 14 inches in diameter and approximately 1-1/2 inches

Figure 3.2 Creep Rack Assembly-Apparatus for Determining Creep of Concrete Cylinders under Stress.

thick. Four symmetrical holes were drilled through these plates at a radius of 5-3/8 inches, which are slightly larger than four load bars 1-1/4 inches in diameter. The load bars are 5 feet 9 inches long. A lower base plate is connected to the top flanges of two I-beams resting on the ground. Springs are placed on the top of the lower base plate and then another upper base plate on the top of the springs. A 6 x 4 inch concrete cylinder, three 6 x 12 inch test cylinders and then another 6 x 4 inch plug concrete cylinder are placed on the top of the upper base plate. A lower jack plate is placed on the top of the cylinders. Nuts are then inserted on the top of this plate. A 30 ton simplex hydraulic jack is placed exactly at the center of the lower jack plate. The ram of the jack is working against another plate, on the top of which there are two dynamometers, a final upper jack plate and then nuts. The exact force desired on the cylinders could be attained by setting the calibrated strain corresponding to that force on two standard SR-4 Wheatstone bridges and jacking against the middle jack plate between the ram of the jack and the dynamometers. Two dynamometers were used since the capacity of neither one is enough for the desired force. Great care should be exercised to give a concentric loading on the cylinders. This could be achieved by changing the position of the dynamometers to get almost equal strain recordings on the two Wheatstone bridges. When the desired force is recorded on the Wheatstone bridges the nuts on the top of the lower jack plate are tightened only by hand and not by a wrench to avoid any extra load on the cylinders.

3.4 Steel

Seven-wire, cold drawn, high carbon, stress relieved strand steel was used for prestressing. The size of the strands used was 7/16 inch nominal diameter. The modulus of elasticity of the strand was found to be 28,000,000 psi.

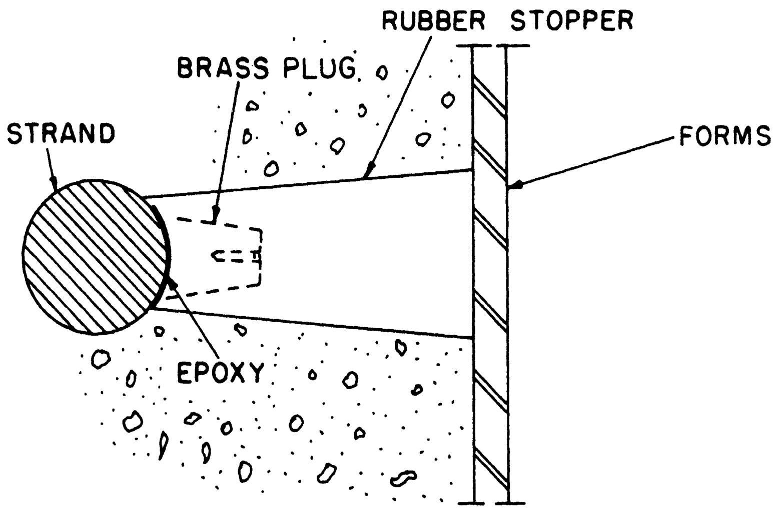
Brass plugs, similar to those used in determining the strain at the surface of concrete, were used to determine the strain in the steel strands. The procedure used in cementing these plugs to the cables is as follows: The strands were tensioned slightly, in their final lay-outs, just enough to make them straight. The wider faces of the plugs, which are of tapered shape, were filed to match the profile of the strands. Devcon 2 ton epoxy adhesive was used to secure the plugs to the strand. The plugs were kept fastened to the strands by means of clamps for at least 24 hours, to permit the epoxy to have sufficient strength to hold the plugs. Then the clamps were taken away and the glue left to dry for another three days. Provisions for the change in the gage lengths between any two plugs due to tensioning the cables should be made. This depends on the modulus of elasticity of the steel, the gage length and the stress. After the epoxy had dried, tapered rubber stoppers covering the plugs extending from the strands to the inside face of the forms were inserted. Holes a little larger than the size of the plugs were necessarily drilled in the rubber so that the plugs would go in and be completely covered by the rubber. After the concrete had hardened and the forms taken away, these rubber stoppers were taken from the concrete. It is necessary to oil the surfaces of the rubber in contact with concrete but caution should be taken to avoid any oil on the strands. Holes in the concrete, left by the removal of the rubber stoppers and extending from the surface of the concrete beam to the plugs which were cemented to the strand, were necessary for Whittemore gage legs to go through and measure the strain of the steel strands. Special Whittemore gage legs (4 inces) were used for this purpose.

Two major difficulties were encountered in measuring the strain of steel. First, the tensioning of the cables will always loosen the plugs and it is probable that the vibration of the concrete would contribute to this loosening. For these reasons some plugs came off. Wooden extensions to the rubber stoppers were needed at the plug locations on the middle strands. Although these extensions were oiled, great difficulties were encountered in taking them off. This contributed to the loosening of the plugs cemented to the strands. This difficulty was solved by cementing other plugs, instead of those which were loosened, after the concrete had hardened. The same rubber was pressed into the holes to keep the plugs from falling for the period necessary for the epoxy to dry. This treatment was successful. The second difficulty was that longer legs for Whittemore gages are neither stiff nor rigid enough to translate the same elongation between the gage points on the strand to the Whittemore dial gage. This difficulty may be solved by placing the strands closer to the surface of concrete as far as possible and using shorter legs.

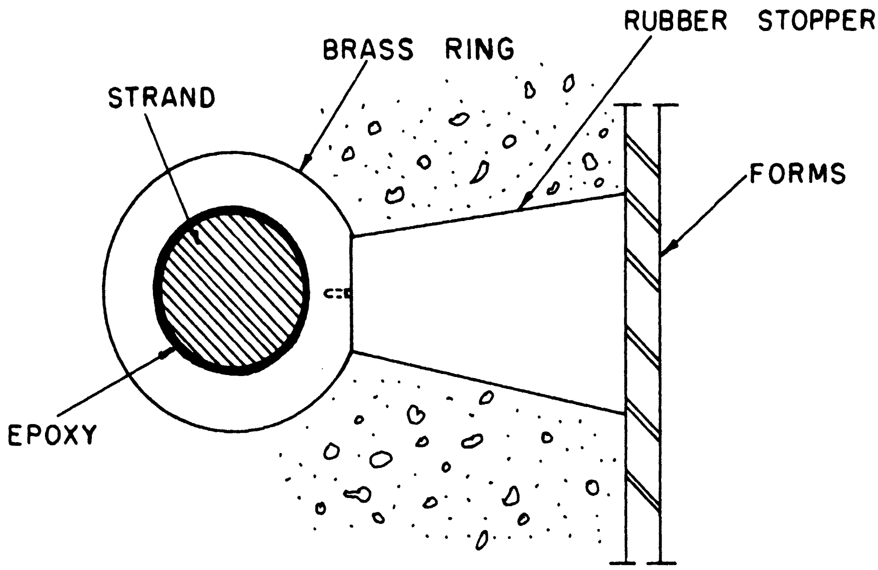
However, the writer suggests the use of brass rings instead of the plugs. This will help to avoid loosening of the plugs. Another provision is that the holes in the plugs should be drilled and reamed after and not before the concrete hardens. The writer believes that such a technique will lead to better results in measuring the strain of the steel strands.

Figure 3.3 shows the technique used together with the suggested one in solving this problem. Figure 3.1 shows the plug locations on the steel strands.

Three layouts of the steel strands were used. The location of the steel was varied in these three layouts to give a C.G.S. concrete stress almost the same in the three series. This permitted the study of the effect of stress distribution, having a practically constant C.G.S. stress, on the creep of concrete. The three layouts of the steel strands, together with the beam cross sections, are shown in Figure 3.4.




A - USED METHOD




B - SUGGESTED METHOD

Figure 3.3 The Used and Suggested Methods of Cementing the Plugs on the Steel Strands.

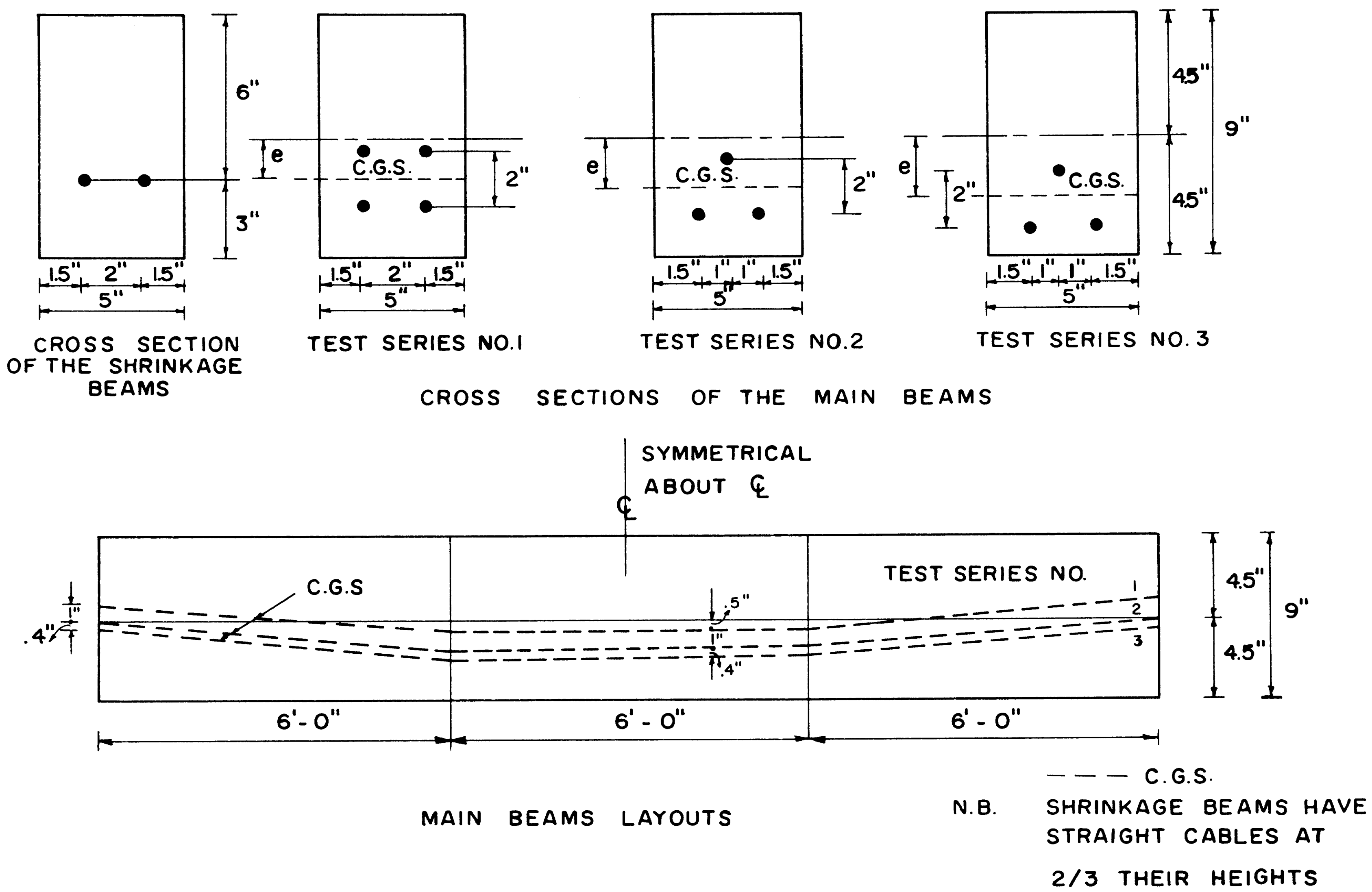



Figure 3.4 Beams Cross Sections and Layouts.

For the three test series, the dummy shrinkage beam has two unstressed strands located at two-thirds the depth of the beams. The reason for the shrinkage beam strands, which has nothing to do with the stresses, is for the purpose of having a dummy shrinkage beam composed of materials similar to those in the main prestressed beam in order to have a similar shrinkage behavior.

For the main prestressed beams, the eccentricity of the steel was kept constant for the middle third of the beams. The three test series have end eccentricities of -1.0 inch, 0.0 inch and 0.4 inch, respectively, and middle third eccentricities of 0.5 inch, 1.5 inches and 1.9 inches respectively. The difference between end and middle third eccentricities is the same in all three test series and it is 1.5 inches. The first P.C. beam has four strands, while the other two have three strands each. They were stressed to an original stress of 175,000 psi (19,057 lb. per cable). The techniques used in stretching the cables and depressing them will be discussed at this time.

## 3.5 Geometry of the Deflected Pretensioned Strands

As mentioned before, the steel strands have end eccentricity which is different than the middle third of the beam eccentricity. The beams need depressing devices at the one-third points capable of taking the upward thrust caused by bending the cables. The device used is shown in Figure 3.5. It consists of a threaded vertical steel rod which goes

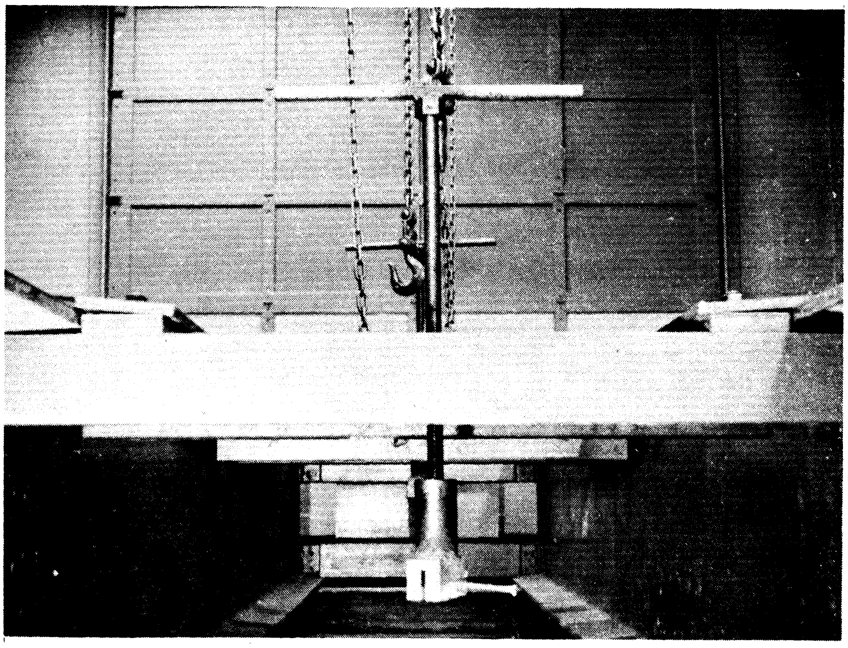


Figure 3.5 Device for Depressing Strands.

through the middle of a steel channel. This rod can be turned to change its elevation by a horizontal arm at its top. The channel is supported under the inside top flanges of the prestressing bed and kept from falling by steel plates connecting it to the top flange. The bottom edge of the steel rod goes into a steel tube and is connected to it in such a manner to permit the turning of the steel tube freely around the rod. Welded to the steel tube is a steel block. A vertical groove is cut in the bottom of the block and a 3/8 inch thick steel plate is inserted in this groove. It is kept in place by a bolt which goes through the block and the plate.

Holes just large enough to permit the cables to pass through are drilled in the steel plate. These holes should match exactly the location of the steel strands at this point of depressing the cables. This plate is left in the concrete. The depressing device is movable by sliding it on the top flange.

### 3.6 Stressing the Steel Strand

The strands were individually stressed by means of a 30 ton simplex hydraulic jack. The total force on the strand was measured by means of a calibrated SR-4 load cell which was inserted between the ram of the jack and the strand chuck against which the ram worked. The strand passes through the center of the load cell (dynamometer) to eliminate eccentricity on the cell. The exact force desired on the strand could be attained by setting the calibrated amount of strain corresponding to that force on a standard SR-4 Wheatstone bridge and jacking the strand until the bridge was balanced. Such an arrangement is shown in Figure 3.6 and Figure 3.7.

"Supreme" brand strand chucks were used to anchor the strand against the end templates of the stressing bed. The strand prestressing arrangement, is shown in Figure 3.7. It consists mainly of two 24WF120 beams, each 30 feet long. They are placed 3 feet apart. An anchorage plate is placed at the anchor end and another plate placed at a distance of 4 feet from the jacking end. The jack works against two vertical channels, connected by welding with enough distance left between them to allow the strands to pass through. These two channels are made movable laterally so that they can be adjusted to the desired location.

The slippage of the chucks was eliminated by jacking the strand to the desired stress, sliding the chuck tightly against the frame, and then releasing the jack which "seated" the chuck tightly onto the strand. The strand was then rejacked to the desired stress and the resulting gap between the chuck was tightly shimmed with split washers; the jack was then released and left so for three to four days. Before casting the concrete the strands were rejacked again to the design stress and the resulting gap, now due to creep in steel, was tightly shimmed with similar split washers. It is the writer's belief

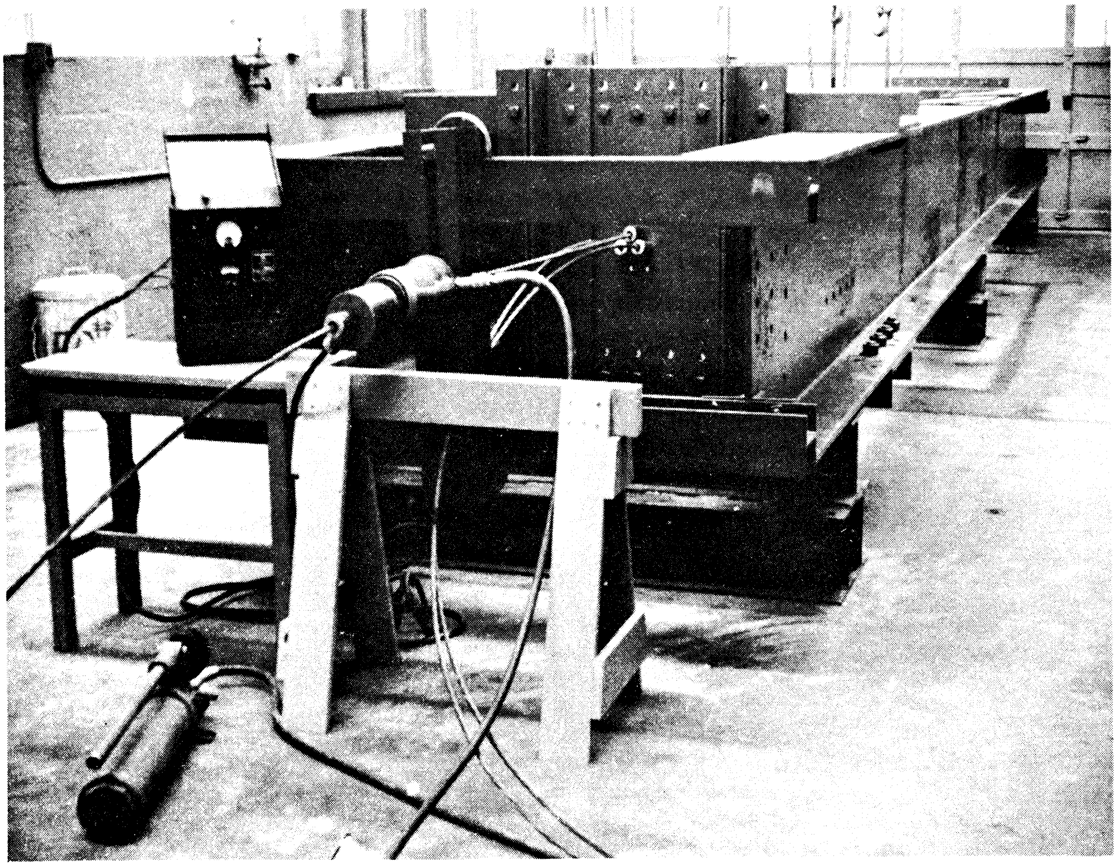


Figure 3.6 The Prestressing Bed.

that by doing so the anchorage losses can be ignored and probably a good percentage of the loss due to the creep of steel could be omitted. This opinion will be justified later from the results of the steel creep test.

### 3.7 Transferring the Prestress Force

The prestress force in the steel strands was transferred to the concrete when the concrete cylinders had attained a minimum compressive strength equivalent to (1/0.6), the maximum compressive strength in concrete at transfer. The prestress force was transferred by simply burning the strands with a gas cutting torch. Care was exercised to cut the strands in a pattern which kept the applied eccentricity as small as possible at all times. Each strand was cut at both ends before proceeding to the next strand.

### 3.8 The Steel Creep Test

A strand 30 feet long, of the same reel of the steel which was used in the P.C. beams, was tested to determine its creep behavior under constant strain. The strand was stressed to a stress level of 175,000 psi between the ends of a rigid testing frame. This stress corresponds to the original steel stress level in the main P.C. beams. The stressing procedure was similar to that used in stressing the strands of the main P.C. beams which was discussed in Section 3.6. A dynamometer, which was previously tested for its creep characteristics and found to not creep under the same load as that in the strand, was inserted between the end plate and the strand chuck. This calibrated dynamometer was used to record the force in the strand. Two dial gages were placed at the ends of the testing frame to measure its movement, if any. There was no movement recorded. The arrangement for the steel creep test is shown in Figure 3.8.

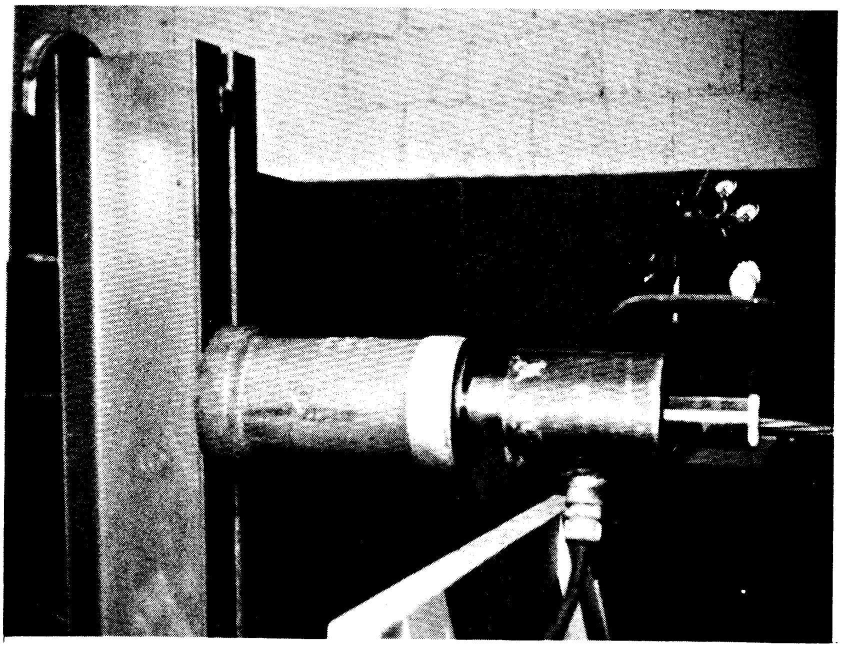


Figure 3.7 Hydraulic Jack and Dynamometer Arrangement Used in Prestressing the Strand.

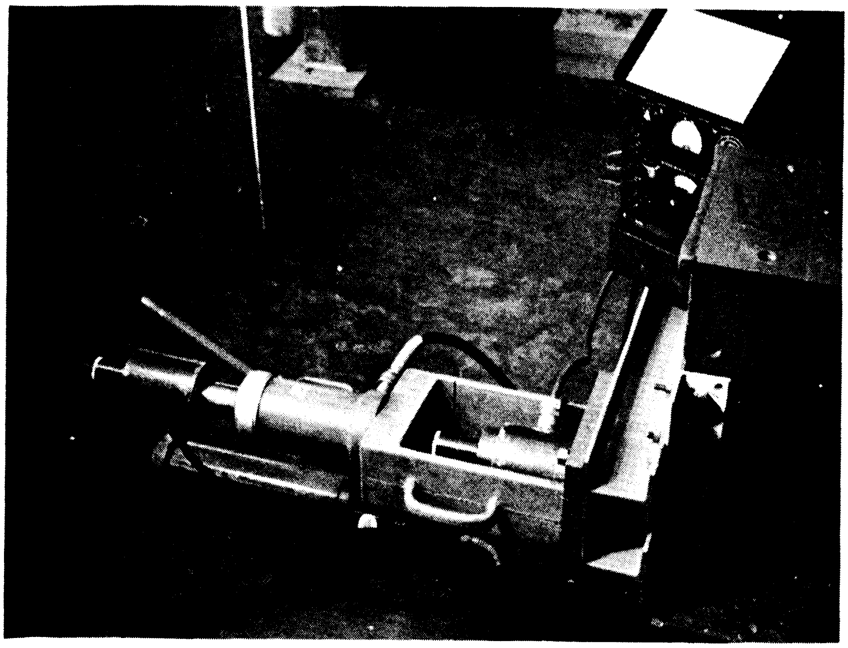


Figure 3.8 Arrangement for the Steel Creep Test.

## CHAPTER IV

## DESIGN OF THE TEST BEAMS

### 4.1 Assumptions

The design of the test beams was based on the following assumptions:

1. Prestressing strand

   $E_s = 27,000,000$ psi

   $f_s'$ (ultimate) $= 250,000$ psi

   $f_s$ (at transfer) $= 175,000$ psi

2. Concrete

   $f_{ci}'$ (at transfer) $= 3000$ psi

   $f_c'$ (at 28 days) $= 4000$ psi

   $E_c = 60,000 \sqrt{f_{ci}'} = 3,794,760$ psi (at transfer)

   $$n = \frac{27}{3.79476} = 7.11 \text{ (at transfer)}$$

3. Losses

   Total creep of steel losses $= .04\ F_o$

   Total shrinkage losses $= .00025\ A_s E_s$

   Total creep of concrete losses $= 1.5$ x Elastic losses

   Elastic losses $(F_o - F_i)$ is obtained from Equation (2.3).

However, after the beams were cast, the actual values of concrete and steel properties, obtained from the tested cylinders and strands, were used in recalculating the stresses in the beams. The modulus of elasticity of the steel strands used in the experiments was found to be 28,000,000 psi and $f_s' = 27,000/.1089 = 247,930$ psi (see Figure 5.1).

## 4.2 Sample Calculations

The stresses at Gage No. 50 of Test Series No. 2 will be illustrated below. The tabulated stresses of all test beams are included in Tables 4.1, 4.2, and 4.3. These values are computed using the IBM 7090 electronic computer.

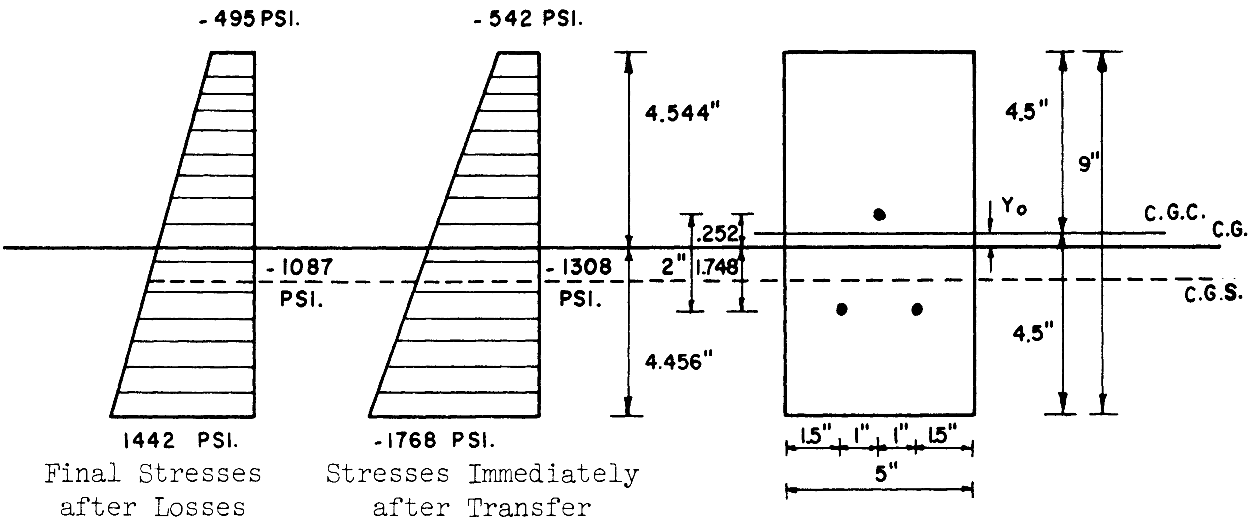



Figure 4.1. Stresses at Gage No. 50 of Test Series No. 2.

This section is at a distance $x = 53.995''$ from the end of the beam and the eccentricity of the steel strands is 1.125". Further properties of the section and the materials are as follows:

Cross sectional dimensions = 5" x 9"

No. of 7/16" strands = 3(0.1089 sq. in./strand)

$A_s = 3 \times .1089 = 0.3267$ sq. in.

$E_c = 4{,}258{,}000$ psi

$n = 28/4.258 = 6.57586$

$$y_o = \frac{(n-1)\ A_s\ e}{A_c + (n-1)\ A_s}$$

$$= \frac{5.57586 \times .3267 \times 1.125}{5 \times 9 + 5.57586 \times .3267} = .043765''$$

$e_t = 1.125 - .043765 = 1.08113''$

$$I_t = \frac{5 \times 9^3}{12} + 45 \times (.043765)^2 + (6.576-1) \times .3267 \times (1.08113)^2$$

$$= 306 \text{ inch}^4$$

$A_t = 45 + (6.576 - 1) \times .3267 = 46.82168$ sq. in.

Calculation of Losses

(1) Due to Elastic deformation of concrete

Weight of beam $= \frac{45}{144} \times \frac{150}{12} = 3.90625$ lb/inch

(Assumed weight of concrete beam = 150 lb/cu foot)

Bending Moment $M_x = \frac{w}{2} x (L - x)$

$$= \frac{3.90625}{2} \times 53.995 \times 162.005$$

$$= 17085.94 \text{ lb inch}$$

$$F_o = 175{,}000 \times .3267 = 57{,}172 \text{ lb}$$

From Equation (2.3), we have:

$$F_i = \frac{F_o + M_x e_t n A_s/I_t}{1 + n A_s \left(\frac{1}{A_t} + \frac{e_t^2}{I_t}\right)}$$

$$= \frac{57{,}172 + 17{,}086 \times 1.08113 \times 6.576 \times .3267/306}{1 + 6.576 \times .3267 \left(\frac{1}{46.82168} + \frac{1.08113 \times 1.08113}{306}\right)}$$

$= 54{,}361$ lb.

Elastic losses $= F_o - F_i = 2{,}811$ lb.

(2) Due to shrinkage of concrete

Shrinkage loss $= .00025 \times 28 \times 10^6 \times .3267$

$= 2{,}287$ lb.

(3) Due to creep of concrete

Concrete creep loss = 1.5 elastic loss

= 4,217 lb.

(4) Due to creep of steel

Steel creep loss = .04 $F_o$ = 2,287 lb.

Total losses = 2,811 + 2,287 + 4,217 + 2,287 = 11,602 lb.

% Age Total losses = 11,602 x 100/57,172 = 20.29%

Final prestress force = 57,172 - 11,602 = 45,570 lb.

Concrete Stresses Immediately After Transfer

$$\text{Top fiber stresses} = -\frac{54,361}{46.82168} + \frac{54,361 \times 4.5437 \times 1.08113}{306} - \frac{17086 \times 4.5437}{306} = -542 \text{ psi}$$

$$\text{Bottom fiber stresses} = -\frac{54,361}{46.82168} - \frac{54,361 \times 4.456 \times 1.08113}{306} + \frac{17086 \times 4.456}{306} = -1768.1 \text{ psi}$$

Similarly:

Stresses at C.G.S. = - 1308.27 psi

Stresses at top cables = - 1126.65 psi

Stresses at bottom cables = - 1399.10 psi

Final Concrete Stresses After Losses

$$\text{Top fiber stresses} = -\frac{45,570}{46.82168} + \frac{45,570 \times 4.5437 \times 1.08113}{306} - \frac{17086 \times 4.5437}{306} = -495 \text{ psi}$$

$$\text{Bottom fiber stresses} = -\frac{45{,}570}{46.82168} - \frac{45{,}570 \times 4.456 \times 1.08113}{306} + \frac{17086 \times 4.456}{306} = -1441.9 \text{ psi}$$

Similarly:

Stresses at C.G.S. = - 1086.9 psi

Stresses at top cables = - 946.725 psi

Stresses at bottom cables = - 1157 psi

TABLE 4.1

CALCULATED VALUES OF PRESTRESS FORCE BASED ON ZERO ANCHORAGE LENGTH
TEST SERIES NO. (1)

$E_s$ = 28,000,000 psi  $A_s$ = .4356 sq. inch  Eccentricity at the ends = -1.0 inch

$E_c$ = 4,807,000 psi (At transfer)  $F_o$ = 76,230 lb  Eccentricity in the Middle Third = .5 inch

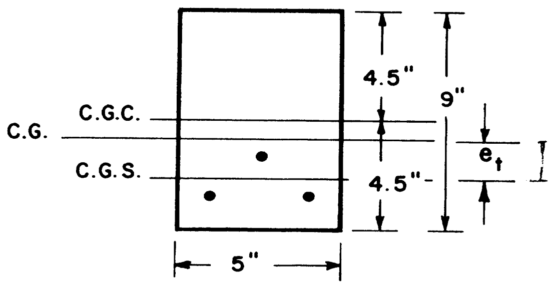
$f'_{ci}$ = 3,680 psi (At transfer)  $A_t$ = 47.1017 sq. inch  Steel Creep Loss = 3,049 lb = (4%)

n = 5.82484 (At transfer)  Shrinkage Loss = 3,049 lb = (4%)

Concrete Creep Losses = 1.5 Elastic Losses, Shrinkage Losses = .00025 $E_sA_s$

| Gage No. | Distance* (inch) | Eccentricity $e_t$ (inch) | $I_t$ (inch$^4$) | $F_i$ (lb) | Elastic Loss $(F_o-F_i)$(lb) | Concrete Creep Loss (lb) | F (lb) | Percentage Elastic Loss | Percentage Total Losses | Concrete Stresses at Transfer in psi at: | | | | | Concrete Stresses after Losses in psi at: | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | Bottom Fibers | Top Fibers | C.G.S. | Top Cables | Bottom Cables | Bottom Fibers | Top Fibers | C.G.S. | Top Cables | Bottom Cables |
| 10 | 5.999 | -.8360 | 305.29 | 71,921 | 4,309 | 6,464 | 59,359 | 5.653 | 22.13 | -596 | -2441 | -1698 | -1903 | -1493 | -485 | -2021 | -1403 | -1573 | -1232 |
| 20 | 13.999 | -.6768 | 304.76 | 72,043 | 4,187 | 6,280 | 59,665 | 5.492 | 21.73 | -722 | -2325 | -1650 | -1828 | -1472 | -584 | -1940 | -1368 | -1519 | -1218 |
| 30 | 21.998 | -.5175 | 304.34 | 72,147 | 4,083 | 6,125 | 59,923 | 5.357 | 21.39 | -853 | -2203 | -1609 | -1759 | -1459 | -687 | -1851 | -1339 | -1468 | -1209 |
| 40 | 29.997 | -.3583 | 304.03 | 72,229 | 4,001 | 6,001 | 60,129 | 5.248 | 21.12 | -987 | -2076 | -1577 | -1698 | -1456 | -795 | -1755 | -1315 | -1422 | -1208 |
| 50 | 53.995 | -.1193 | 303.78 | 72,341 | 3,889 | 5,833 | 60,410 | 5.101 | 20.75 | -1411 | -1661 | -1532 | -1560 | -1505 | -1136 | -1429 | -1279 | -1311 | -1246 |
| 60 | 72.00 | .4777 | 304.25 | 72,280 | 3,950 | 5,926 | 60,255 | 5.182 | 20.96 | -1745 | -1322 | -1557 | -1510 | -1604 | -1405 | -1152 | -1293 | -1265 | -1321 |
| 70 | 88.00 | .4777 | 304.25 | 72,286 | 3,944 | 5,916 | 60,272 | 5.174 | 20.93 | -1719 | -1348 | -1554 | -1513 | -1596 | -1380 | -1179 | -1290 | -1267 | -1313 |
| 80 | 98.00 | .4777 | 304.25 | 72,288 | 3,942 | 5,912 | 60,277 | 5.171 | 20.93 | -1711 | -1357 | -1553 | -1514 | -1593 | -1371 | -1187 | -1289 | -1269 | -1310 |
| Center | 108.0 | .4777 | 304.25 | 72,289 | 3,941 | 5,911 | 60,279 | 5.170 | 20.92 | -1708 | -1360 | -1553 | -1515 | -1592 | -1368 | -1190 | -1289 | -1269 | -1309 |

* Distance from the end of beam.



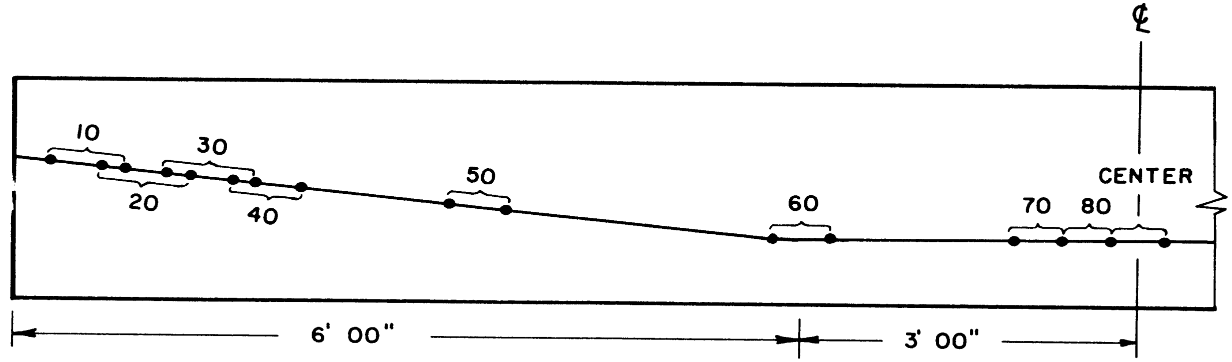



TABLE 4.2

CALCULATED VALUES OF PRESTRESS FORCE BASED ON ZERO ANCHORAGE LENGTH
TEST SERIES NO. (2)

$E_s$ = 28,000,000 psi — $A_s$ = .3267 sq. inch — Eccentricity at the ends = 0.0 inch

$E_c$ = 4,258,000 psi (At transfer) — $F_o$ = 57,173 lb — Eccentricity in the middle third = 1.5 inch

$f'_{ci}$ = 3,300 psi (At transfer) — $A_t$ = 46.8628 sq. inch — Steel creep loss = 2,287 lb = (4%)

n = 6.57586 (At transfer) — Shrinkage loss = 2,287 lb = (4%)

Concrete Creep Losses = 1.5 Elastic Losses, Shrinkage Losses = .00025 $E_sA_s$

| Gage No. | Distance* (inch) | Eccentricity $e_t$ (inch) | $I_t$ (inch$^4$) | $F_i$ (lb) | Elastic Loss ($F_o$-$F_i$)(lb) | Concrete Creep Loss (lb) | F (lb) | Percentage Elastic Loss | Percentage Total Losses | Concrete Stresses at Transfer in psi at: Bottom Fibers | Top Fibers | C.G.S. | Top Cables | Bottom Cables | Concrete Stresses after Losses in psi at: Bottom Fibers | Top Fibers | C.G.S. | Top Cables | Bottom Cables |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 10 | 5.999 | .1201 | 303.78 | 54,661 | 2,512 | 3,767 | 46,320 | 4.393 | 18.982 | -1228 | -1107 | -1169 | -1151 | -1178 | -1035 | -943 | -991 | -977 | -997 |
| 20 | 13.999 | .2803 | 303.90 | 54,646 | 2,527 | 3,790 | 46,282 | 4.419 | 19.049 | -1312 | -1022 | -1176 | -1133 | -1198 | -1099 | -878 | -995 | -963 | -1012 |
| 30 | 21.998 | .4404 | 304.12 | 54,618 | 2,555 | 3,832 | 46,211 | 4.469 | 19.170 | -1398 | -933 | -1189 | -1120 | -1224 | -1164 | -808 | -1004 | -952 | -1031 |
| 40 | 29.997 | .6006 | 304.43 | 54,576 | 2,597 | 3,895 | 46,106 | 4.542 | 19.360 | -1487 | -840 | -1209 | -1113 | -1257 | -1232 | -735 | -1018 | -944 | -1055 |
| 50 | 53.995 | 1.0811 | 305.97 | 54,362 | 2,811 | 4,216 | 45,572 | 4.916 | 20.290 | -1768 | -542 | -1308 | -1127 | -1399 | -1442 | -495 | -1087 | -947 | -1157 |
| 60 | 72.00 | 1.4416 | 307.69 | 54,108 | 3,064 | 4,596 | 44,939 | 5.359 | 21.400 | -1989 | -300 | -1426 | -1176 | -1551 | -1603 | -300 | -1168 | -975 | -1265 |
| 70 | 88.00 | 1.4416 | 307.69 | 54,125 | 3,047 | 4,571 | 44,980 | 5.330 | 21.320 | -1965 | -326 | -1419 | -1176 | -1540 | -1579 | -326 | -1161 | -976 | -1254 |
| 80 | 98.00 | 1.4416 | 307.69 | 54,131 | 3,042 | 4,563 | 44,994 | 5.320 | 21.300 | -1957 | -335 | -1416 | -1176 | -1536 | -1571 | -335 | -1159 | -976 | -1251 |
| Center | 108.0 | 1.4416 | 307.69 | 54,132 | 3,040 | 4,560 | 44,999 | 5.317 | 21.290 | -1954 | -338 | -1415 | -1176 | -1535 | -1569 | -338 | -1158 | -976 | -1249 |

* Distance from the end of beam.

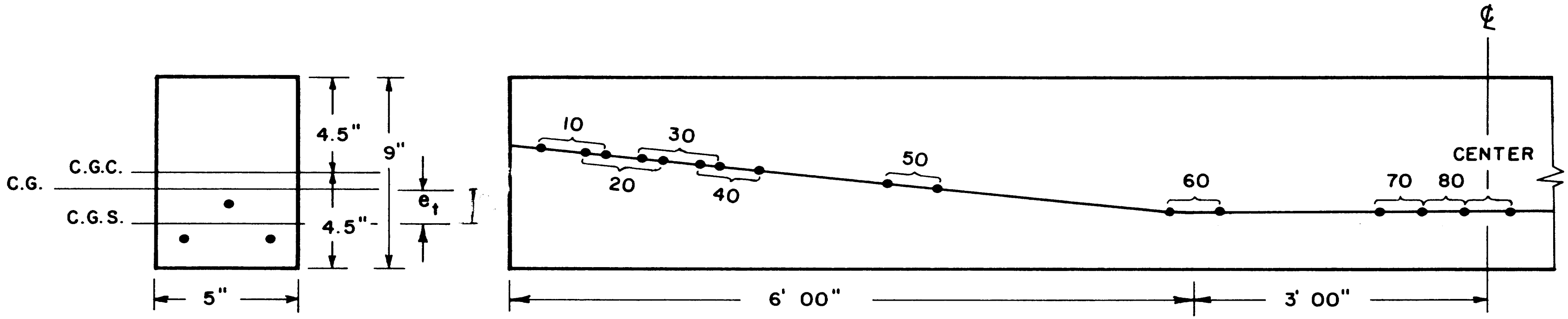


TABLE 4.3

CALCULATED VALUES OF PRESTRESS FORCE BASED ON ZERO ANCHORAGE LENGTH
TEST SERIES NO. (3)

$E_s$ = 28,000,000 psi    $A_s$ = .3267 sq. inch    Eccentricity at the ends = .4 inch

$E_c$ = 4,178,000 psi (At transfer)    $F_o$ = 57.173 lb    Eccentricity in the middle third = 1.9 inch

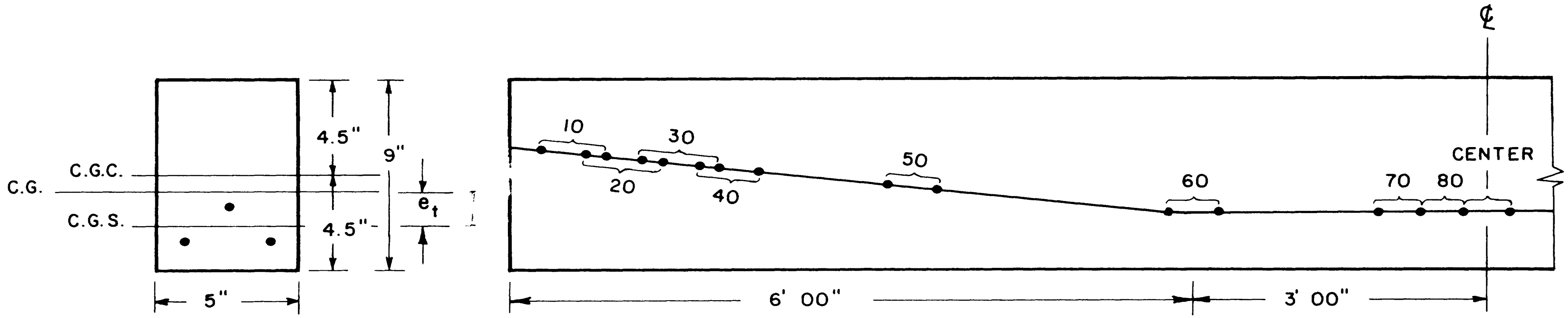
$f'_{ci}$ = 3,760 psi (At transfer)    $A_t$ = 46.8628 sq. inch    Steel creep loss = 2,287 lb = (4%)

n = 6.70177 (At transfer)    Shrinkage loss = 2,287 lb = (4%)

Concrete Creep Losses = 1.5 Elastic Losses, Shrinkage Losses = .00025 $E_sA_s$

| | | | | | | | | | | Concrete Stresses at Transfer in psi at: | | | | | Concrete Stresses after Losses in psi at: | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Gage No. | Distance* (inch) | Eccentricity $e_t$ (inch) | $I_t$ (inch$^4$) | $F_i$ (lb) | Elastic Loss ($F_o$-$F_i$)(lb) | Concrete Creep Loss (lb) | F (lb) | Percentage Elastic Loss | Percentage Total Losses | Bottom Fibers | Top Fibers | C.G.S. | Top Cables | Bottom Cables | Bottom Fibers | Top Fibers | C.G.S. | Top Cables | Bottom Cables |
| 10 | 5.999 | .5041 | 304.24 | 54,534 | 2,639 | 3,958 | 46,002 | 4.615 | 19.538 | -1532 | -792 | -1205 | -1095 | -1260 | -1287 | -674 | -1016 | -925 | -1061 |
| 20 | 13.999 | .6641 | 304.61 | 54,481 | 2,691 | 4,038 | 45,869 | 4.708 | 19.770 | -1613 | -707 | -1229 | -1095 | -1297 | -1345 | -608 | -1033 | -924 | -1088 |
| 30 | 21.998 | .8242 | 305.07 | 54,414 | 2,758 | 4,137 | 45,703 | 4.824 | 20.061 | -1696 | -618 | -1260 | -1100 | -1340 | -1405 | -539 | -1054 | -926 | -1119 |
| 40 | 29.997 | .9842 | 305.63 | 54,333 | 2,839 | 4,258 | 45,502 | 4.965 | 20.413 | -1781 | -527 | -1297 | -1111 | -1389 | -1465 | -468 | -1080 | -932 | -1154 |
| 50 | 53.995 | 1.4643 | 307.91 | 54,004 | 3,169 | 4,753 | 44,677 | 5.542 | 21.855 | -2046 | -234 | -1447 | -1179 | -1581 | -1650 | -237 | -1183 | -974 | -1288 |
| 60 | 72.00 | 1.8245 | 310.21 | 53,665 | 3,507 | 5,261 | 43,830 | 6.134 | 23.337 | -2252 | +.33 | -1602 | -1268 | -1769 | -1787 | -54 | -1287 | -1030 | -1415 |
| 70 | 88.00 | 1.8245 | 310.21 | 53,686 | 3,486 | 5,229 | 43,883 | 6.098 | 23.244 | -2229 | -25 | -1592 | -1266 | -1756 | -1765 | -80 | -1278 | -1028 | -1403 |
| 80 | 98.00 | 1.8245 | 310.21 | 53,693 | 3,479 | 5,219 | 43,900 | 6.086 | 23.214 | -2221 | -34 | -1589 | -1265 | -1751 | -1757 | -89 | -1275 | -1028 | -1398 |
| Center | 108.0 | 1.8245 | 310.21 | 53,696 | 3,477 | 5,215 | 43,906 | 6.081 | 23.203 | -2218 | -37 | -1588 | -1265 | -1750 | -1755 | -91 | -1274 | -1028 | -1397 |

* Distance from the end of beam.



## CHAPTER V

## RESULTS OF THE TESTING PROGRAM

### 5.1 Modulus of Elasticity of the Steel Strands

The modulus of elasticity curve for the 7/16 inch diameter cable is shown in Figure 5.1. This data was obtained from a tensile test conducted at the University of Michigan. The net area of the cable was 0.1089 sq. inch. The elongations of a 24 inch effective gage length were measured by means of an Extensometer.

From this test, the modulus of elasticity of the steel strand was found to be 27,988,338 psi and was considered in all computations as 28,000,000 psi. The ultimate strength was found to be 247,930 psi.

### 5.2 Modulus of Elasticity of Concrete

Modulus tests were run on 6 x 12 inch concrete cylinders cast with each test series. These cylinders were cured under the same conditions as the shrinkage and main beams. They were wrapped with burlap and kept wet for the same period as the beams. One test was performed immediately, prior to transfer of the prestress force, and a second one was performed at a cylinder age of 28 days.

However, compression tests were performed before cutting the cables to assure the desired strength. All specimens were loaded to failure at a constant rate of strain equal to .005 inches per minute. The strains were measured by means of a dial gage attached to the concrete cylinders. The secant modulus taken at one-half of the ultimate strength was used in the computations. The results of the individual tests are shown in Figures 5.2 through 5.7.

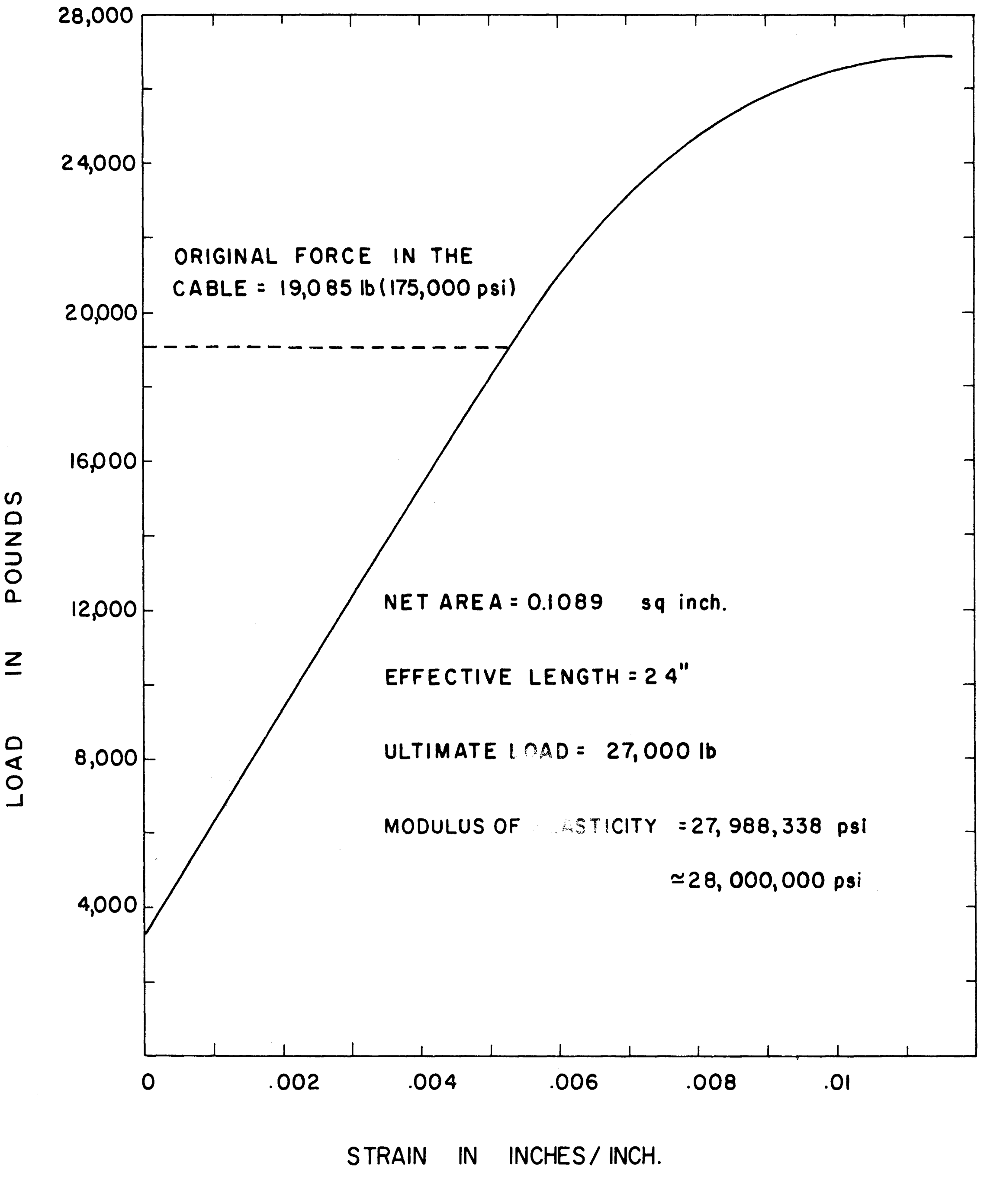



Figure 5.1 Steel: Modulus of Elasticity.

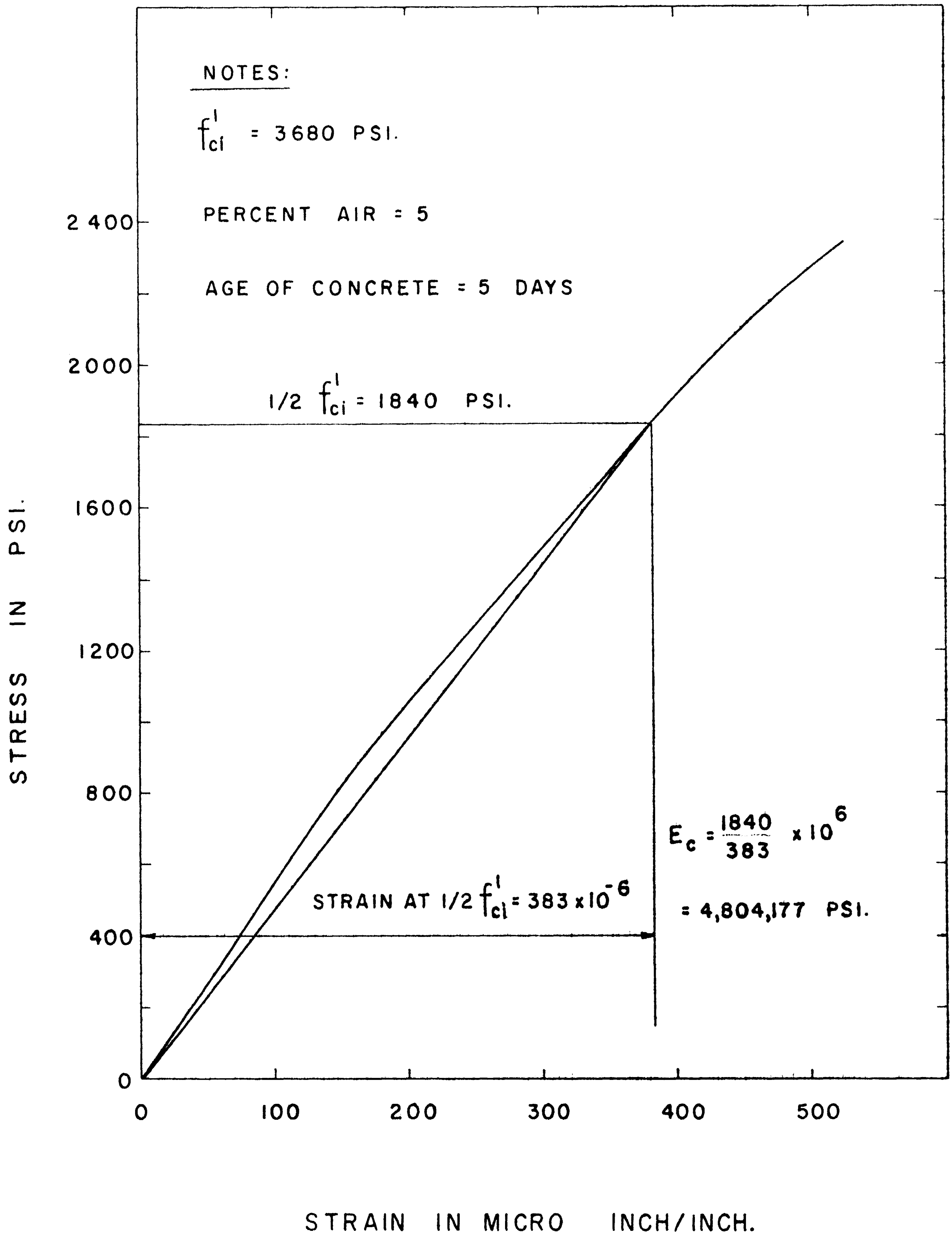



Figure 5.2 Concrete: Modulus of Elasticity - Test Series No. 1.

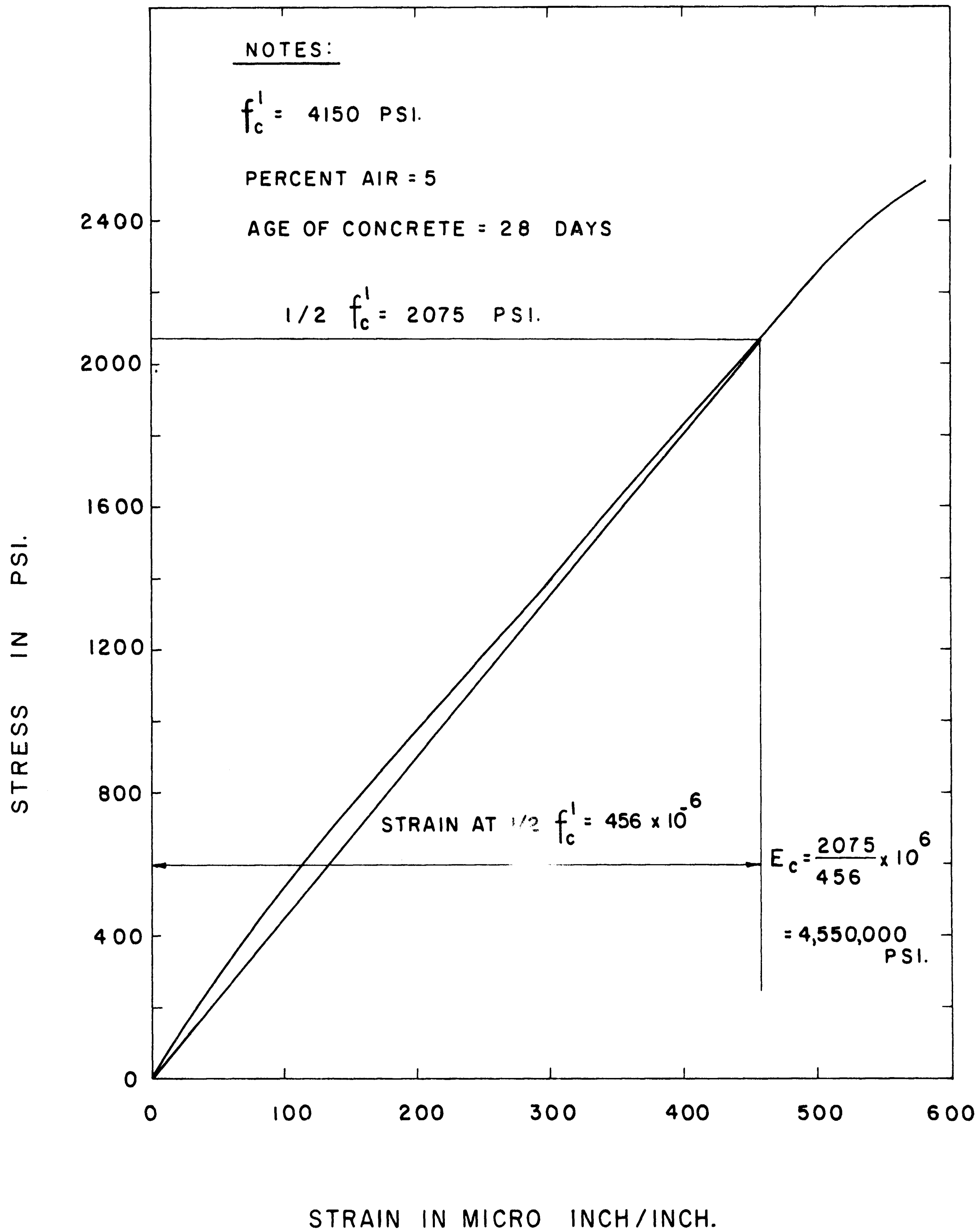



Figure 5.3 Concrete: Modulus of Elasticity - Test Series No. 1.

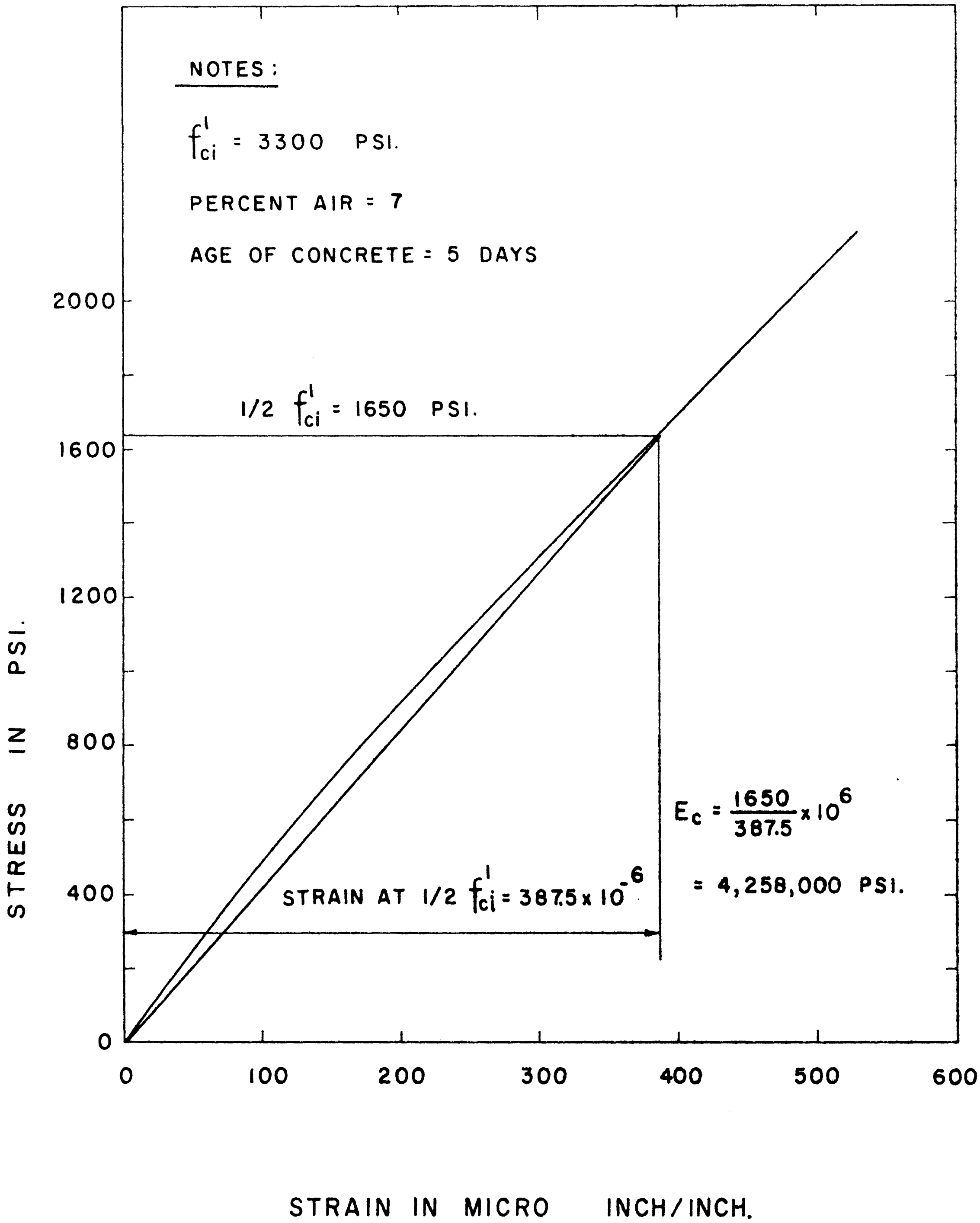



Figure 5.4 Concrete: Modulus of Elasticity - Test Series No. 2.

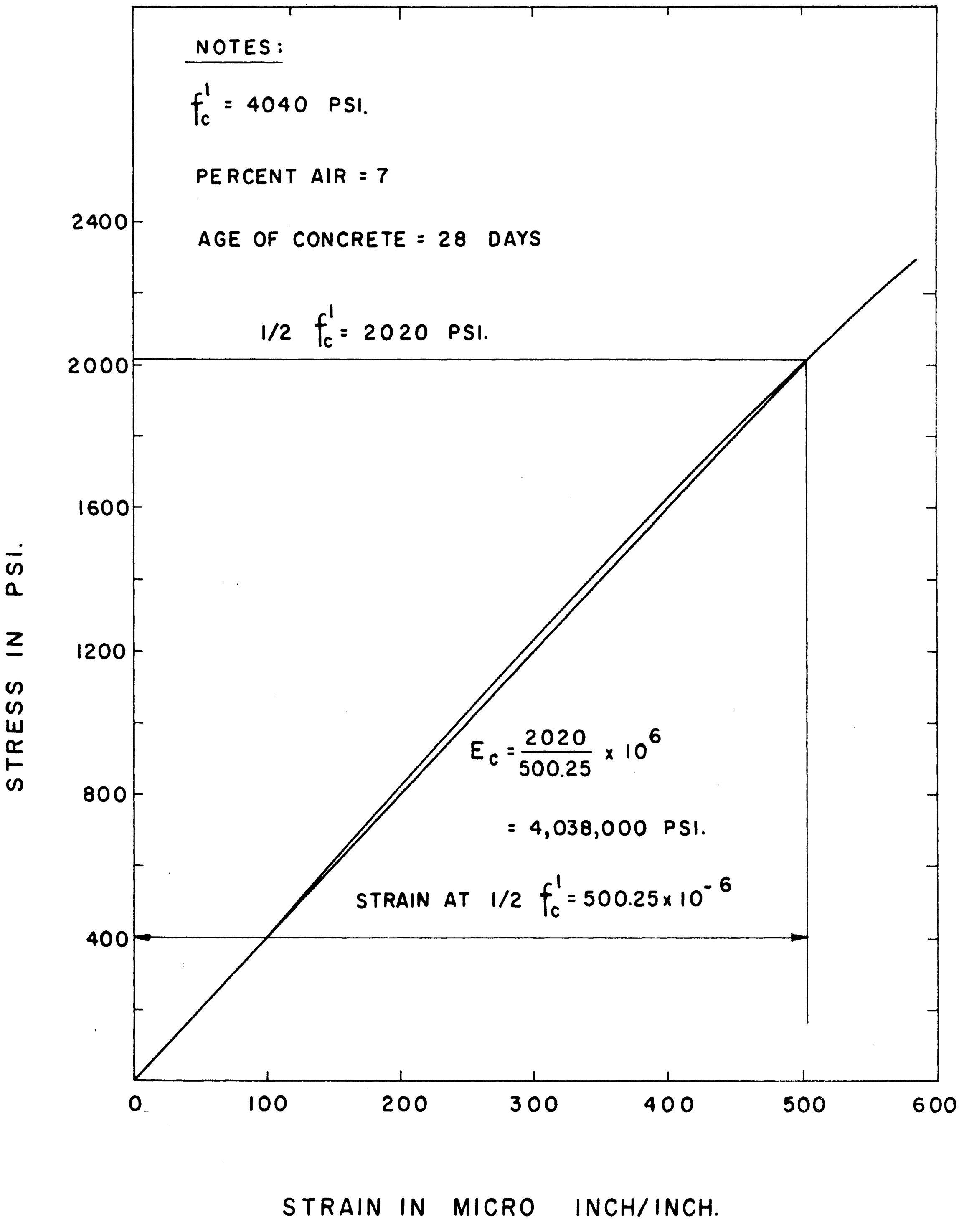



Figure 5.5 Concrete: Modulus of Elasticity - Test Series No. 2.

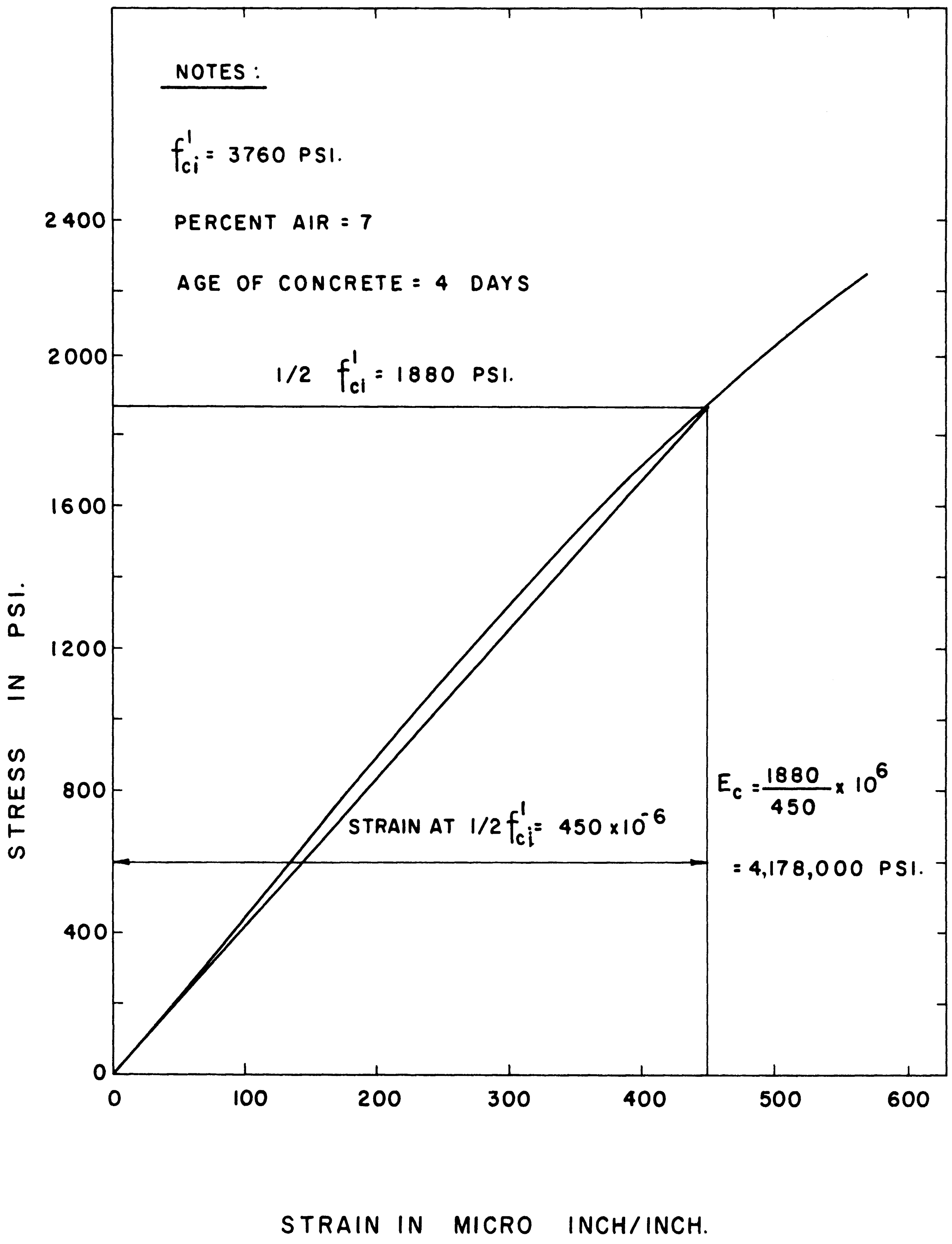



Figure 5.6 Concrete: Modulus of Elasticity - Test Series No. 3.

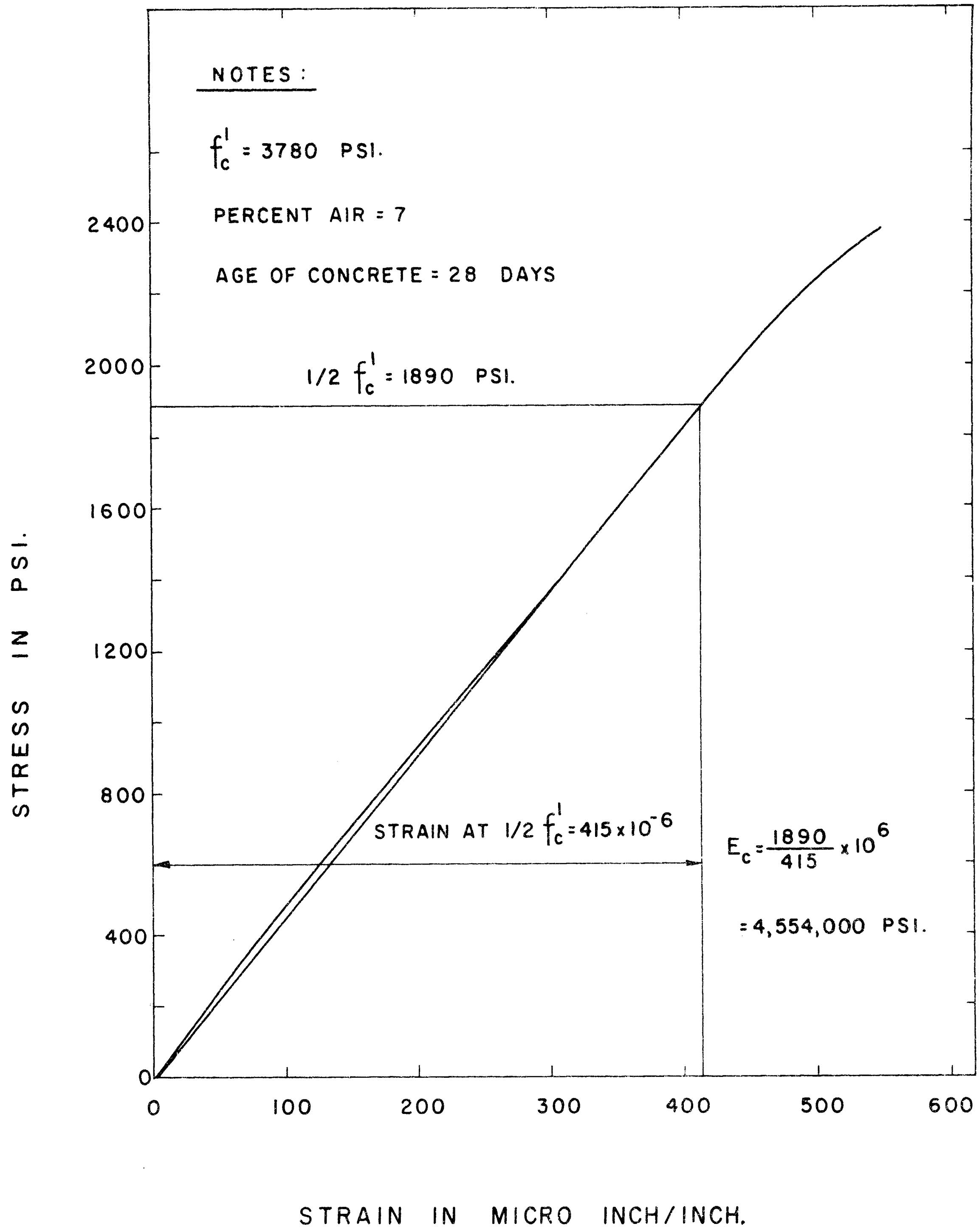



Figure 5.7 Concrete: Modulus of Elasticity - Test Series No. 3.

5.3 Calibration of the Pressure Cells (the Dynamometers)

The total force in the strands and the load applied to the concrete cylinders in the creep tests was measured by means of two calibrated SR-4 load cells which were inserted between the ram of the jack and the strand chuck, against which the ram worked. In the case of the cylinder creep tests they were placed on the top of the lower jack plate. The exact force desired could be obtained by setting the calibrated amount of strain corresponding to that force on a standard SR-4 Wheatstone bridge and then jacking the strand until the bridge was balanced. The calibration curves of the load cells are given in Figure 5.8.

5.4 Elastic Losses of Concrete and Anchorage Length of Steel Strands

The measured elastic shortening of concrete at various gage points along the beams was obtained from the differences between strain readings taken immediately before and after transfer of the prestress force. Since the steel remains bonded to the concrete, except near the ends, the strain of both concrete and steel at the C.G.S. is the same. The strain at the C.G.S. at the zone of complete anchorage at any point times the modulus of elasticity of steel will represent the elastic loss of prestress, while the strain at the C.G.S. times the elastic modulus of concrete at transfer will represent the stresses in concrete at the C.G.S.

The measured elastic shortening, concrete strains and concrete stresses at the C.G.S. immediately after transfer are recorded in Tables 5.1, 5.2 and 5.3 and the concrete stresses at this stage are plotted in Figures 5.9, 5.10 and 5.11. These values are plotted versus the distance from the end of the beam. These curves show clearly how the concrete

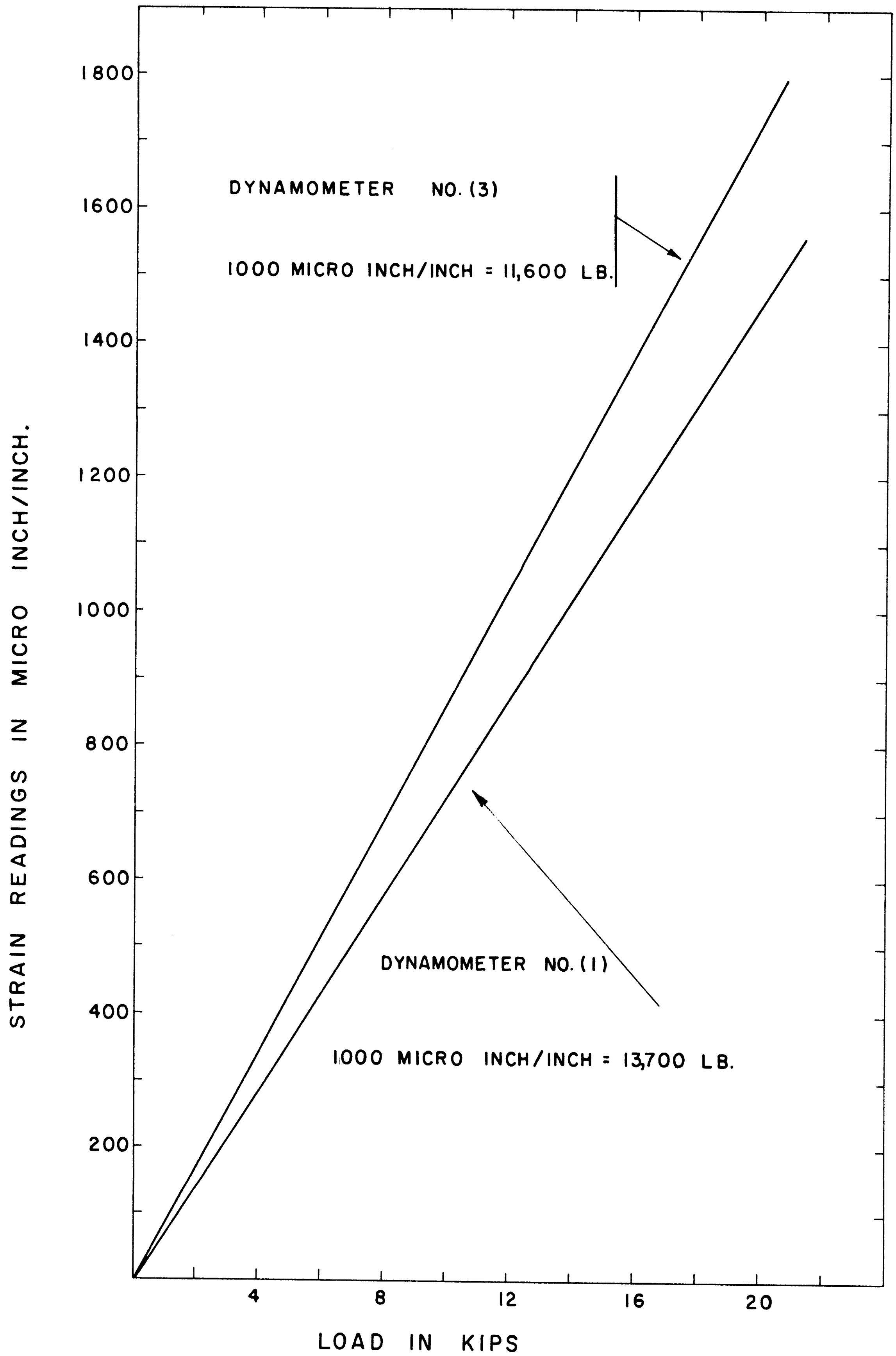



Figure 5.8 Calibration of the Dynamometers.

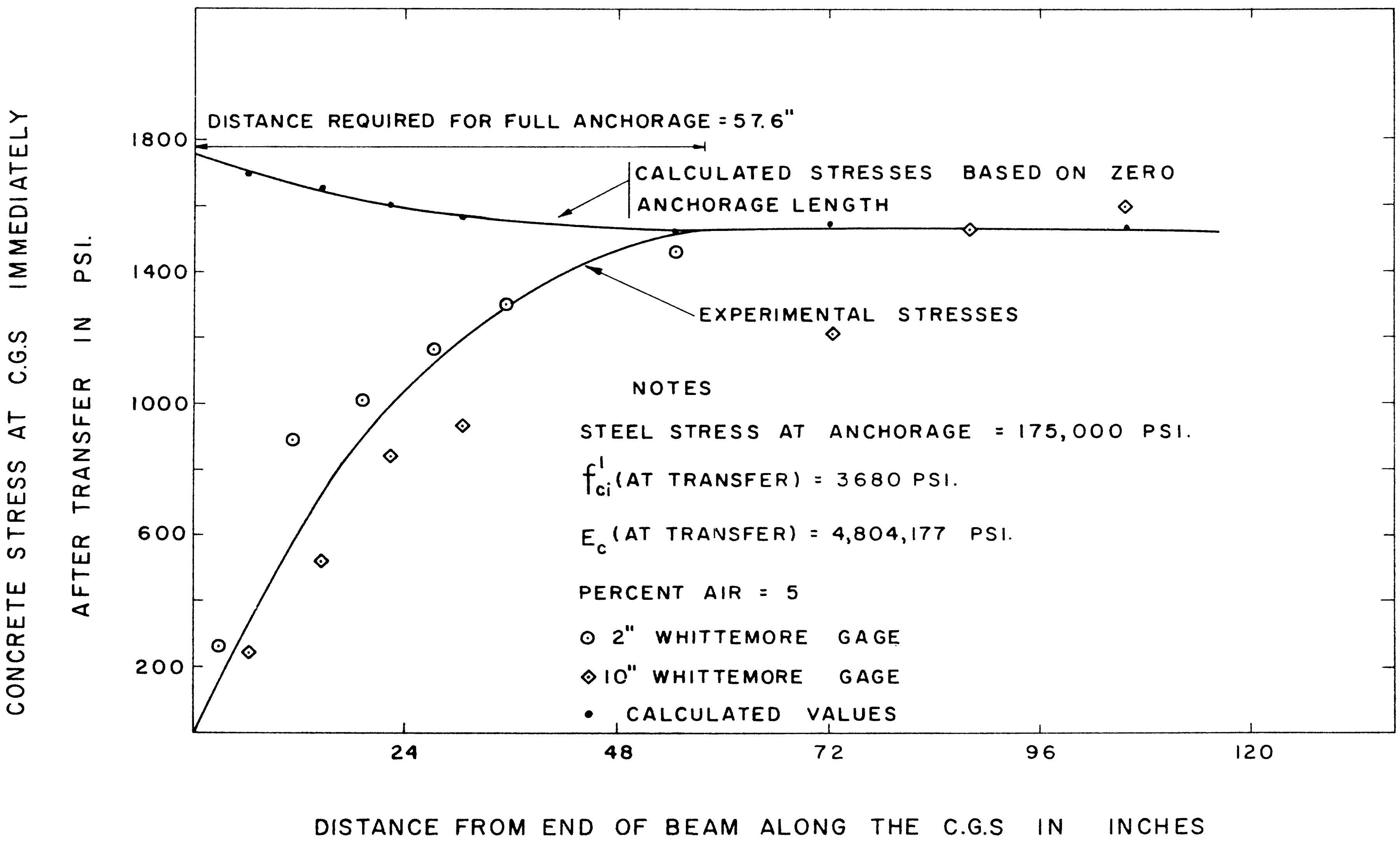



Figure 5.9 Anchorage of 7/16" Ø Seven Wire Uncoated Strand - Test Series No. 1.

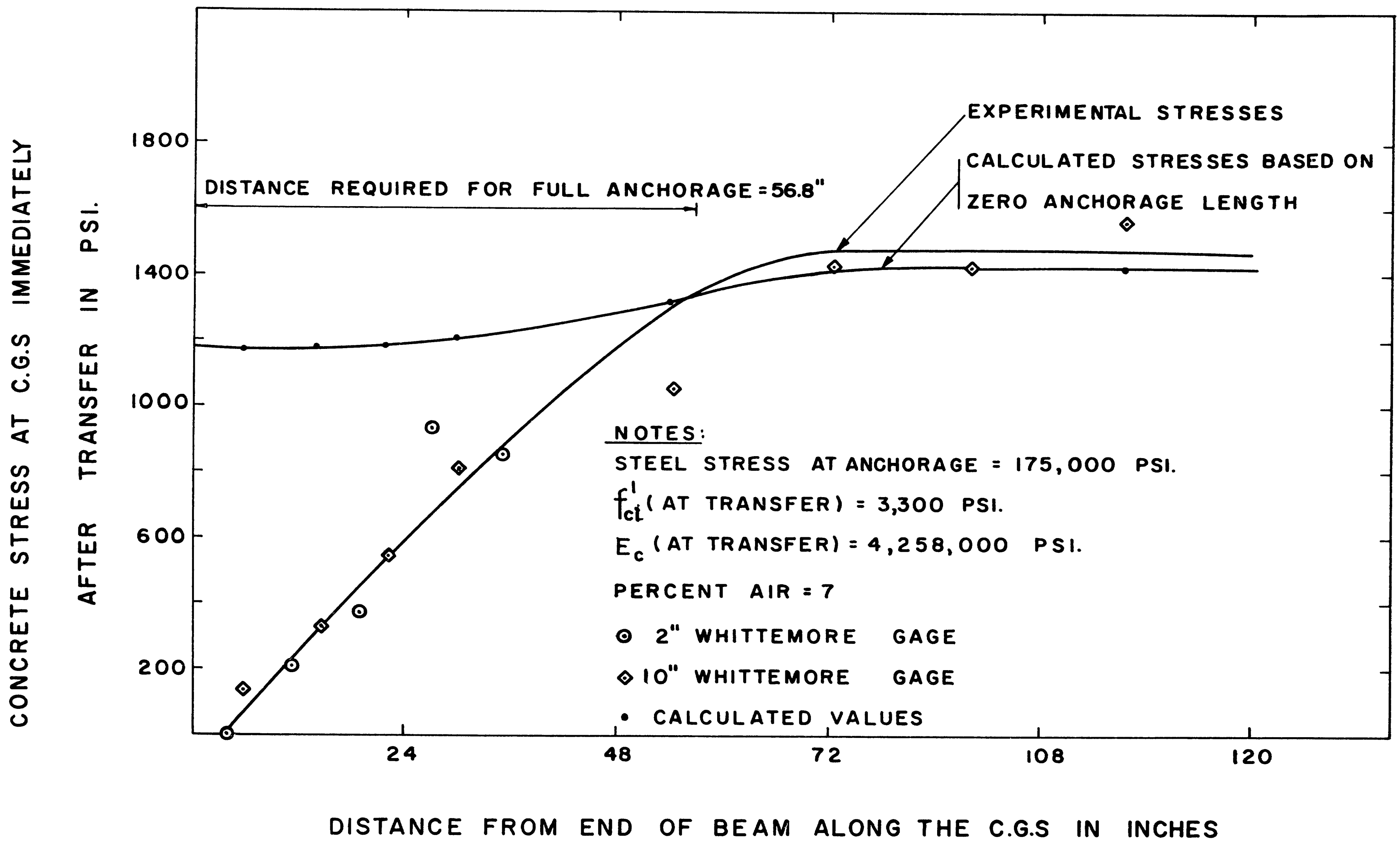



Figure 5.10 Anchorage of 7/16" ∅ Seven Wire Uncoated Strand - Test Series No. 2.

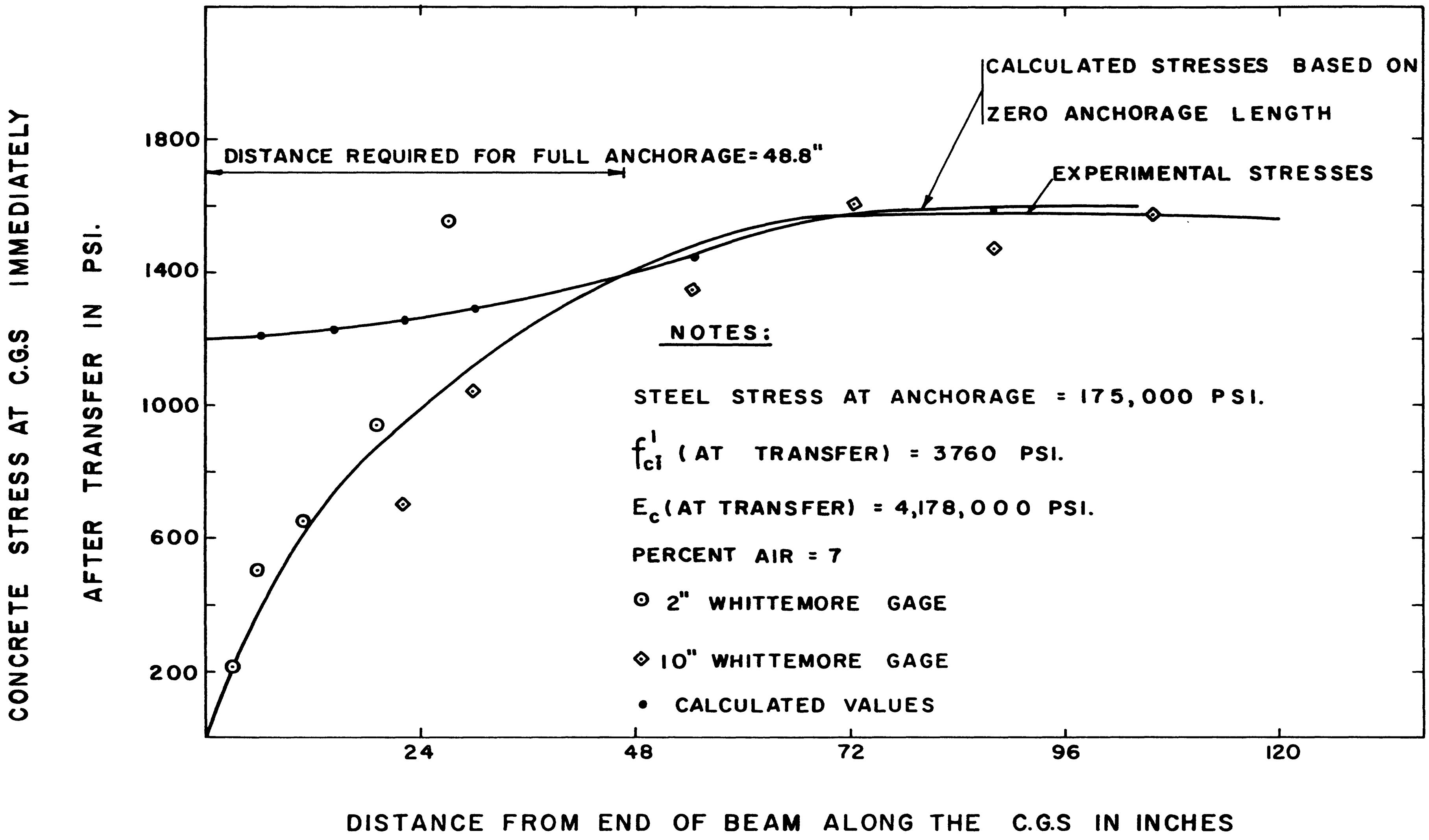



Figure 5.11 Anchorage of /16" ø Seven Wire Uncoated Strand - Test Series No. 3.

TABLE 5.1

CONCRETE STRESSES AT C.G.S. IMMEDIATELY AFTER TRANSFER - TEST SERIES NO. 1

$E_s$ = 28,000,000 Psi $A_s$ = .4356 sq. inch

$E_c$ = 4,807,000 Psi $A_c$ = 45.0 sq. inch

| Gage No. | *Distance From end of Beam in inches | Avg. Measured El. Shortening Inch | Gage Length Inch | Avg. Measured Elastic Strain of Concrete $\epsilon_c$ | Concrete Stresses $f_c=E_c x \epsilon_c$ Psi |
|---|---|---|---|---|---|
| A | 3.0 | .00011 | 2 | .000055 | 264 |
| B | 11.0 | .000375 | 2 | .000186 | 894 |
| C | 19.0 | .000425 | 2 | .000212 | 1,019 |
| D | 27.0 | .000490 | 2 | .000245 | 1,180 |
| E | 35.0 | .000540 | 2 | .000270 | 1,300 |
| 10 | 6.0 | .00051 | 10 | .000051 | 245 |
| 20 | 14.0 | .00109 | 10 | .000109 | 524 |
| 30 | 22.0 | .00175 | 10 | .000175 | 841 |
| 40 | 30.0 | .00195 | 10 | .000195 | 937 |
| 50 | 54.0 | .00304 | 10 | .000304 | 1,461 |
| 60 | 72.0 | .00253 | 10 | .000253 | 1,216 |
| 70 | 88.0 | .00320 | 10 | .000320 | 1,538 |
| Cent. | 108.0 | .00335 | 10 | .000335 | 1,610 |

* Distances are measured along the length of the cables, except for gages Nos. 60, 70 and Center.

TABLE 5.2

CONCRETE STRESSES AT C.G.S. IMMEDIATELY AFTER TRANSFER - TEST SERIES NO. 2

$E_s$ = 28,000,000 Psi      $A_s$ = .3276 sq. inch

$E_c$ = 4,258,000 Psi      $A_c$ = 45.0 sq. inch

| Gage No. | *Distance From end of Beam in inches | Avg. Measured El. Shortening Inch | Gage Length Inch | Avg. Measured Elastic Strain of Concrete $\epsilon_c$ | Concrete Stresses $f_c=E_c x \epsilon_c$ Psi |
|---|---|---|---|---|---|
| A | 3.0 | 0 | 2 | 0 | 0 |
| B | 11.0 | .00010 | 2 | .000050 | 213 |
| C | 19.0 | .000175 | 2 | .000087 | 370 |
| D | 27.0 | .00044 | 2 | .00022 | 937 |
| E | 35.0 | .00040 | 2 | .00020 | 852 |
| 10 | 6.0 | .00033 | 10 | .000033 | 140 |
| 20 | 14.0 | .00076 | 10 | .000076 | 324 |
| 30 | 22.0 | .00130 | 10 | .000130 | 554 |
| 40 | 30.0 | .00189 | 10 | .000189 | 805 |
| 50 | 54.0 | .00247 | 10 | .000247 | 1,052 |
| 60 | 72.0 | .00337 | 10 | .000337 | 1,435 |
| 70 | 88.0 | .0033 | 10 | .000333 | 1,418 |
| Cent. | 108.0 | .00368 | 10 | .000368 | 1,567 |

* Distances are measured along the length of the cables, except for gages Nos. 60, 70 and Center.

TABLE 5.3

CONCRETE STRESSES AT C.G.S. IMMEDIATELY AFTER TRANSFER - TEST SERIES NO. 3

$E_s$ = 28,000,000 Psi $A_s$ = .3267 sq. inch

$E_c$ = 4,178,000 Psi $A_c$ = 45.0 sq. inch

| Gage No. | *Distance From end of Beam in inches | Avg. Measured El. Shortening Inch | Gage Length Inch | Avg. Measured Elastic Strain of Concrete $\epsilon_c$ | Concrete Stresses $f_c=E_c x \epsilon_c$ Psi |
|---|---|---|---|---|---|
| A | 3.0 | .00010 | 2 | .000050 | 209 |
| B | 11.0 | .00031 | 2 | .000155 | 648 |
| C | 19.0 | .00045 | 2 | .000225 | 940 |
| D | 27.0 | .00074 | 2 | .00037 | 1,546 |
| E | 35.0 | .00090 | 2 | .00045 | 1,880 |
| 10 | 6.0 | .00024 | 10 | .000120 | 501 |
| 20 | 14.0 | .00099 | 10 | .000099 | 413 |
| 30 | 22.0 | .00168 | 10 | .000168 | 702 |
| 40 | 30.0 | .00249 | 10 | .000249 | 1,040 |
| 50 | 54.0 | .00324 | 10 | .000324 | 1,353 |
| 60 | 72.0 | .00383 | 10 | .000383 | 1,600 |
| 70 | 88.0 | .00351 | 10 | .000351 | 1,466 |
| Cent. | 108.0 | .00378 | 10 | .000378 | 1,579 |

* Distances are measured along the length of the cables, except for gages Nos. 60, 70 and Center.

stresses at this stage change from zero at the ends to maximum at the center. The calculated values of concrete stresses at the C.G.S. based on zero anchorage length obtained from Tables 4.1, 4.2 and 4.3 are also plotted on the same graphs. A study of these graphs shows the good agreement between the calculated and the measured stresses at the full anchorage portion of the beams. It shows also the "anchorage length". The anchorage length is that length measured from the end of the beam after which the prestressing force will exert its full strength on the concrete beam. The anchorage length in these curves was obtained by measuring the distance from the end to that point at which the calculated and the experimental curves meet. For the three tests series, 1, 2, and 3, these anchorage lengths are 57.6" (.267 L), 56.8" (.263 L) and 46.8" (.216 L) respectively.

## 5.5 Creep and Shrinkage Curves

As stated before, each series of the three tests consisted of a shrinkage beam 4.5 feet long supported at several points to avoid bending stresses as shown in Figure 5.12, a main prestressed beam 18 feet long simply supported on concrete blocks as shown in Figure 5.13, three creep cylinders supported at the bottom on springs and loaded to a stress equal to $F_o/A_t$ of each corresponding test series as shown in Figure 3.2, and two shrinkage cylinders. Each of the creep and shrinkage cylinders had two Whittemore gages on opposite sides of the cylinders. Their center was at the mid-height of each cylinder.

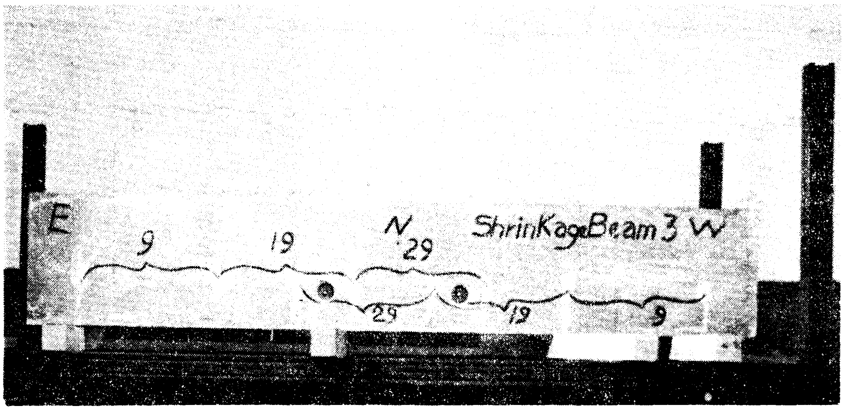



Figure 5.12 A Typical Shrinkage Beam. (Note the brass plugs on the cable.)

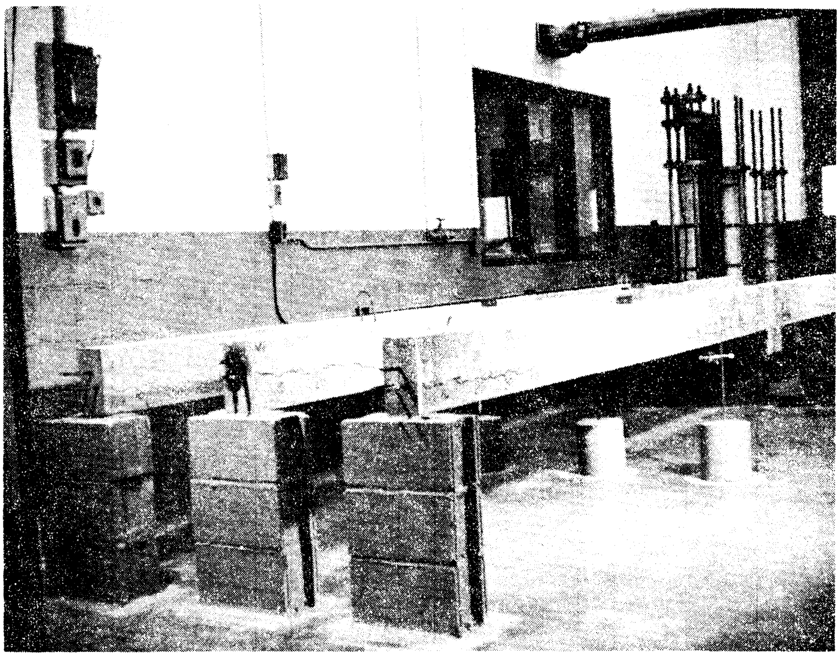


Figure 5.13 The Main Prestressed Concrete Beams in the Storage Area. (Note the use of concrete blocks as end supports and the arrangement of measuring the growth of camber at the center of the beams.)

The shrinkage versus time curves for each test series are shown in Figures 5.14, 5.15 and 5.16. A typical Whittemore gage data sheet for calculating the shrinkage strains from which these curves were plotted is given in Table 5.4.

The shrinkage strain was measured by dividing the difference in Whittemore gage readings at the time considered and that reading at the time of transfer by the gage length. The average reading of all the plugs on the concrete beam and that on the cables was considered. The average of the four readings on the two shrinkage cylinders was also plotted.

These shrinkage curves do not represent pure shrinkage, but rather represent the effects of changes in temperature and humidity as well as shrinkage. The relative himidity curves are also included on the same curves. It was not possible to control the atmospheric conditions of the storage area. The numerous small fluctuations in the shrinkage curves are due to these changes in atmospheric conditions.

Curves of the elastic and creep strain of concrete versus time and given for the cylinder creep test and at four locations on the main prestressed concrete beam. This is shown in Figures 5.17, 5.18 and 5.19. Each reading of the cylinder test represents the average of the six readings on the three creep cylinders, while the beam readings represent the average of four symmetrical readings located at the same distance from the beam ends. All readings are taken at the C.G.S. The four plotted locations on the prestressed concrete beams are at distances of .0608L, .139L, 0.33L and 0.5L, i.e., at gages number 20, 40, 60 and center.

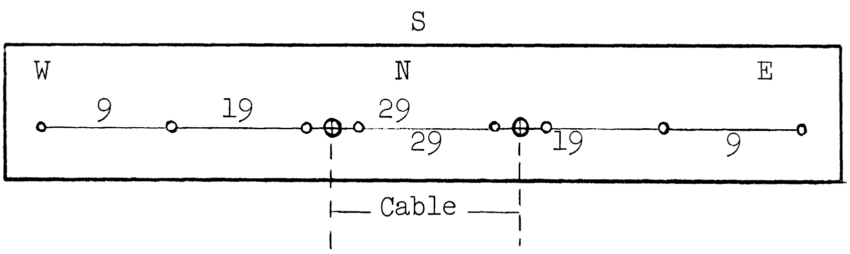
TABLE 5.4

WHITTEMORE GAGE DATA SHEET (SHRINKAGE)

Date - 1/11/1963
Conc. Date - 11/5/1962
Time - 1:00 P.M.
Temps. - 66° - 64°
Beam - S-3
Cylinders - S-3-1 & S-3-2

Age - 67 days
Humidity - 90%
Standard Bars:
Beam & Cylinders (10.07050)
Cable (10.08100)

| Gage No. | | Initial Reading Inch x $10^5$ | Final Reading Inch x $10^5$ | Shortening Inch x $10^5$ | Average Shrinkage Inch/Inch | Average Shrinkage Inch/Inch | Remarks |
|---|---|---|---|---|---|---|---|
| | NE | 3960 | 3670 | 290 | | | |
| 9 | NW | 6440 | 6190 | 250 | .000275 | | |
| | SE | 8830 | 8570 | 260 | | | |
| | SW | 2670 | 2370 | 300 | | | |
| | NE | 7655 | 7330 | 325 | | | |
| 19 | NW | 4350 | 4075 | 275 | .000282 | | |
| | SE | 4940 | 4670 | 270 | | | |
| | SW | 4280 | 4020 | 260 | | .000284 | |
| | NE | 6585 | 6230 | 355 | | | |
| 29 | NW | 9680 | 9310 | 370 | .000317 | | |
| | SE | 4250 | 3970 | 280 | | | |
| | SW | 4925 | 4660 | 265 | | | |
| Cable | N | 5600 | 5340 | 260 | .000240 | | |
| | S | 4900 | 4680 | 220 | | | |
| Cylinders | 1N | 2965 | 2640 | 325 | | | |
| | 1S | 6570 | 6210 | 360 | .000350 | .000350 | |
| | 2N | 2840 | 2490 | 350 | | | |
| | 2S | 8045 | 7680 | 365 | | | |




WHITTEMORE PLUGS LOCATION

TABLE 5.5

WHITTEMORE GAGE DATA SHEET (CREEP)

Beam B-3

Date - 1/11/1963
Age of Concrete - 67 days
Deflection - .015 inch

| Gage No. | Initial Reading Inch x $10^5$ | Final Reading Inch x $10^5$ | Shortening Inch x $10^5$ | Average creep + Elastic + Shrinkage Inch/Inch | Average Shrinkage Inch/Inch | Average Creep + Elastic Inch/Inch | Remarks |
|---|---|---|---|---|---|---|---|
| NE | 5725 | 5345 | 380 | | | | |
| 10 NW | 2755 | 2405 | 350 | .000365 | .000284 | .000081 | |
| SE | 5560 | 5230 | 330 | | | | |
| SW | 7685 | 7285 | 400 | | | | |
| NE | 0550 | 0000 | 550 | | | | |
| 20 NW | 3345 | 2800 | 545 | .000530 | .000284 | .000246 | |
| SE | 6950 | 6450 | 500 | | | | |
| SW | 1835 | 1310 | 525 | | | | |
| NE | 10135 | 9435 | 700 | | | | |
| 30 NW | 6735 | 5995 | 740 | .000727 | .000284 | .000443 | |
| SE | 6610 | 5880 | 730 | | | | |
| SW | 5825 | 5085 | 740 | | | | |
| NE | 7970 | 7080 | 890 | | | | |
| 40 NW | 5030 | 4190 | 840 | .000888 | .000284 | .000604 | |
| SE | 6920 | 6025 | 895 | | | | |
| SW | 7490 | 6565 | 925 | | | | |
| NE | 7150 | 6060 | 1090 | | | | |
| 50 NW | 7010 | 5810 | 1200 | .001094 | .000284 | .000810 | |
| SE | 3750 | 2750 | 1000 | | | | |
| SW | 5200 | 4115 | 1085 | | | | |
| NE | 6940 | 5760 | 1310 | | | | |
| 60 NW | 6975 | 5845 | 1075 | .001227 | .000284 | .000943 | |
| SE | 3660 | 2500 | 1280 | | | | |
| SW | 7715 | 6585 | 1243 | | | | |
| NE | 4075 | 2875 | 1200 | | | | |
| 70 NW | 7175 | 6125 | 1050 | .001098 | .000284 | .000814 | |
| SE | 6100 | 5000 | 1100 | | | | |
| SW | 7275 | 6235 | 1040 | | | | |
| NE | 5440 | 4200 | 1240 | | | | |
| 80 NW | 8885 | 7675 | 1210 | .001216 | .000284 | .000932 | |
| SE | 1970 | 0770 | 1200 | | | | |
| SW | 9200 | 8010 | 1190 | | | | |
| Cent N | 6450 | 5220 | 1230 | | | | |
| Cent S | 7680 | 6455 | 1225 | | | * | |
| Cent Up | 5410 | | 810 | .000810 | .000284 | .000526 | |
| Cent Do | 5340 | 3750 | 1590 | .001490 | .000284 | .001206 | |

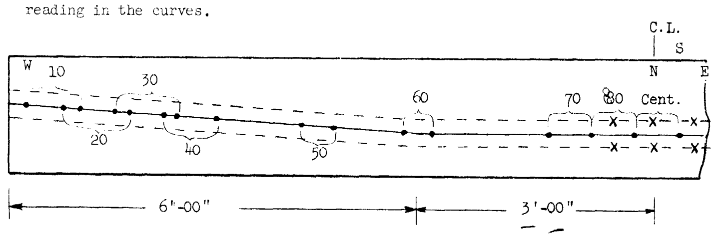
CABLES

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| NW Up | 9920 | 8850 | 1070 | | | | |
| NE Up | 7350 | 6490 | 860 | .000965 | .000284 | .000681 | |
| SW Up | | | | | | | |
| SE Up | | | | | | | |
| NW Do | 5350 | 4085 | 1265 | | | | |
| NE Do | 8900 | 7550 | 1350 | .001300 | .000284 | .001016 | |
| SW Do | 4735 | 3445 | 1290 | | | | |
| SE Do | 4840 | 3545 | 1295 | | | | |

CYLINDERS

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1N | 2495 | 1185 | 1310 | | | | |
| 1S | 4860 | 3500 | 1360 | | | | |
| 2N | 6115 | 4865 | 1250 | .001310 | .000350 | .000960 | |
| 2S | 2120 | 0830 | 1290 | | | | |
| 3N | 3545 | 2245 | 1300 | | | | |
| 3S | 4510 | 3160 | 1350 | | | | |

* Gages No. 80 and Centers are added together and are considered to be the center reading in the curves.




WHITTEMORE PLUGS LOCATION

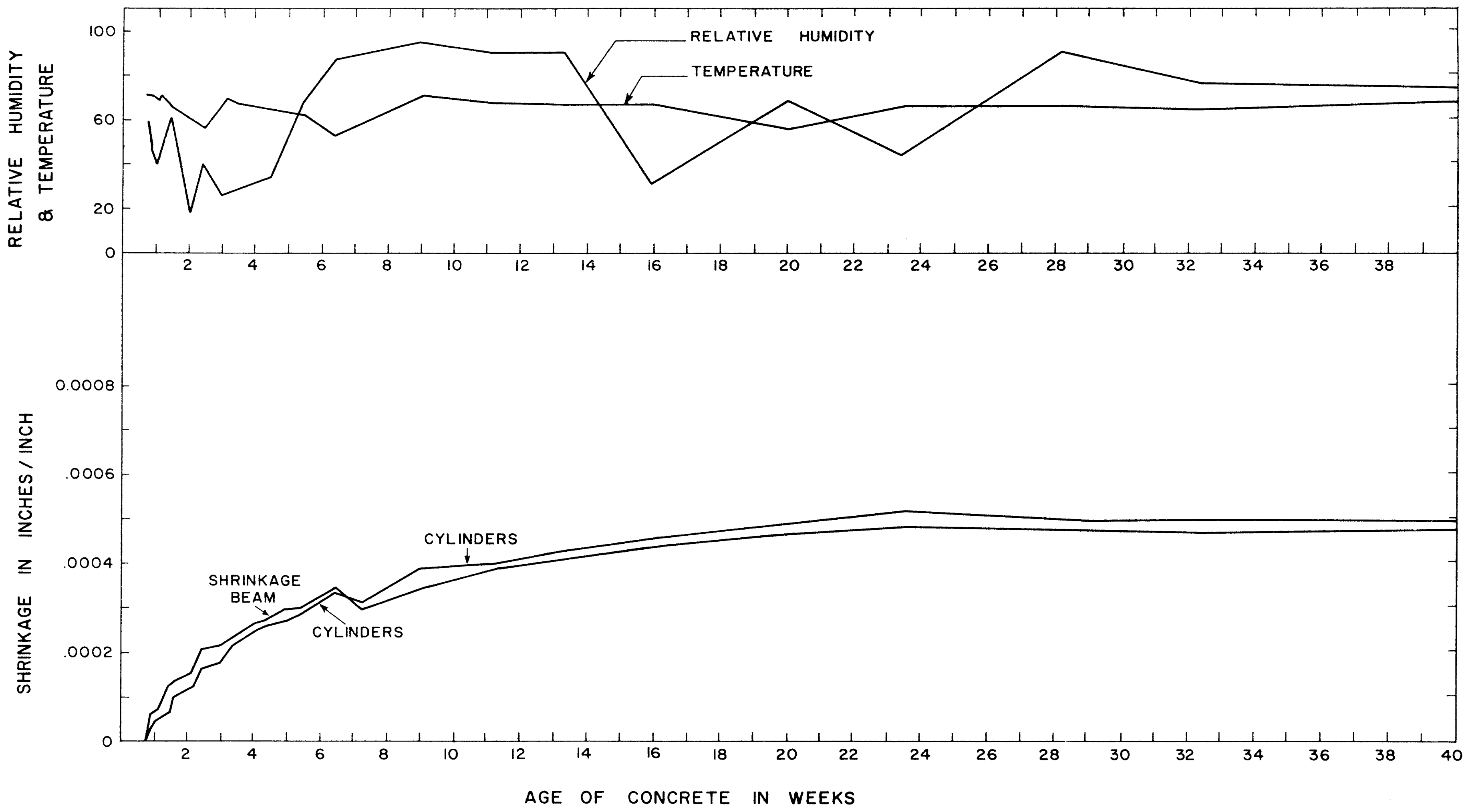



Figure 5.14 Shrinkage vs. Time Curves - Test Series No. 1.

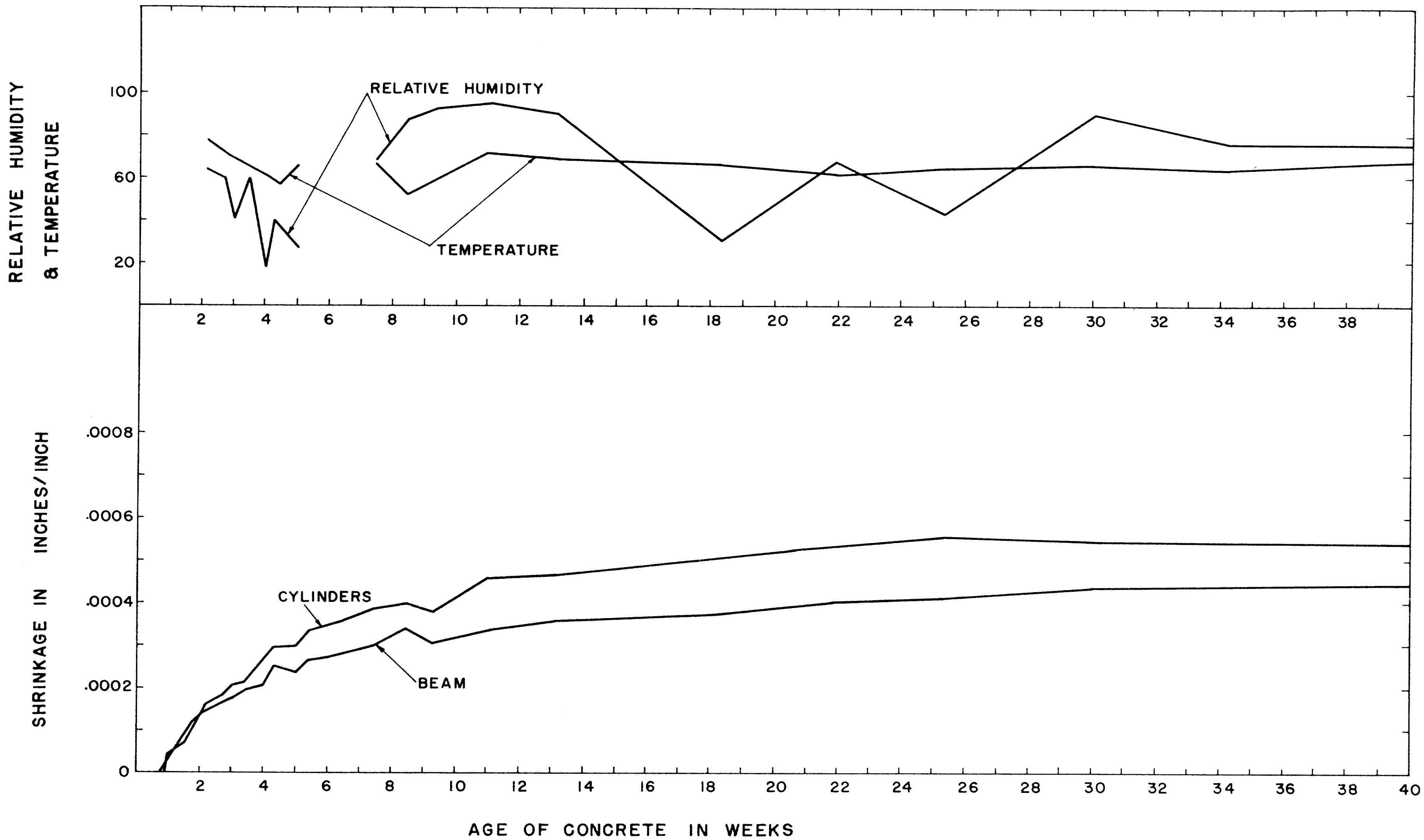



Figure 5.15 Shrinkage vs. Time Curves - Test Series No. 2.

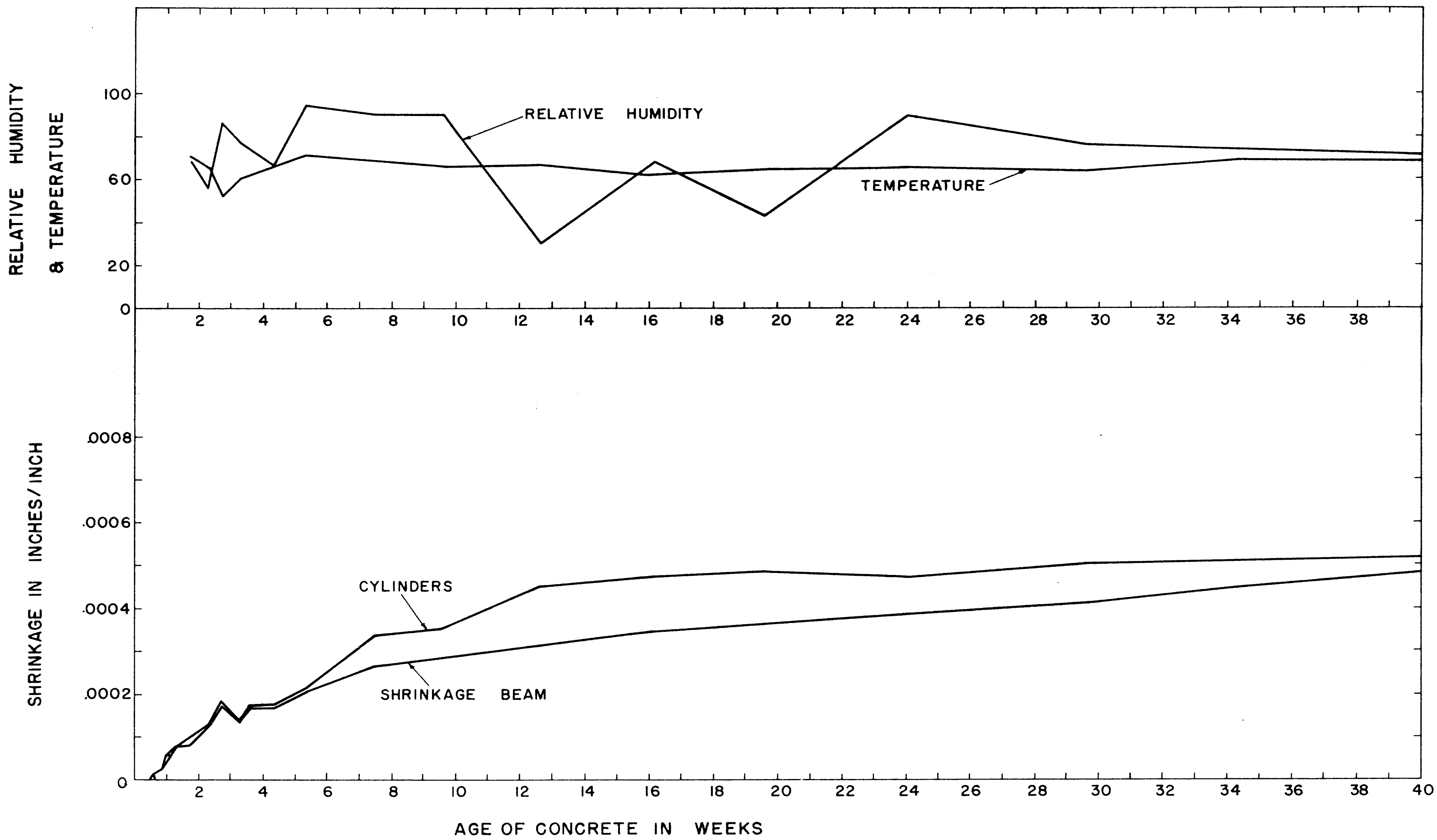



Figure 5.16 Shrinkage vs. Time Curves - Test Series No. 3.

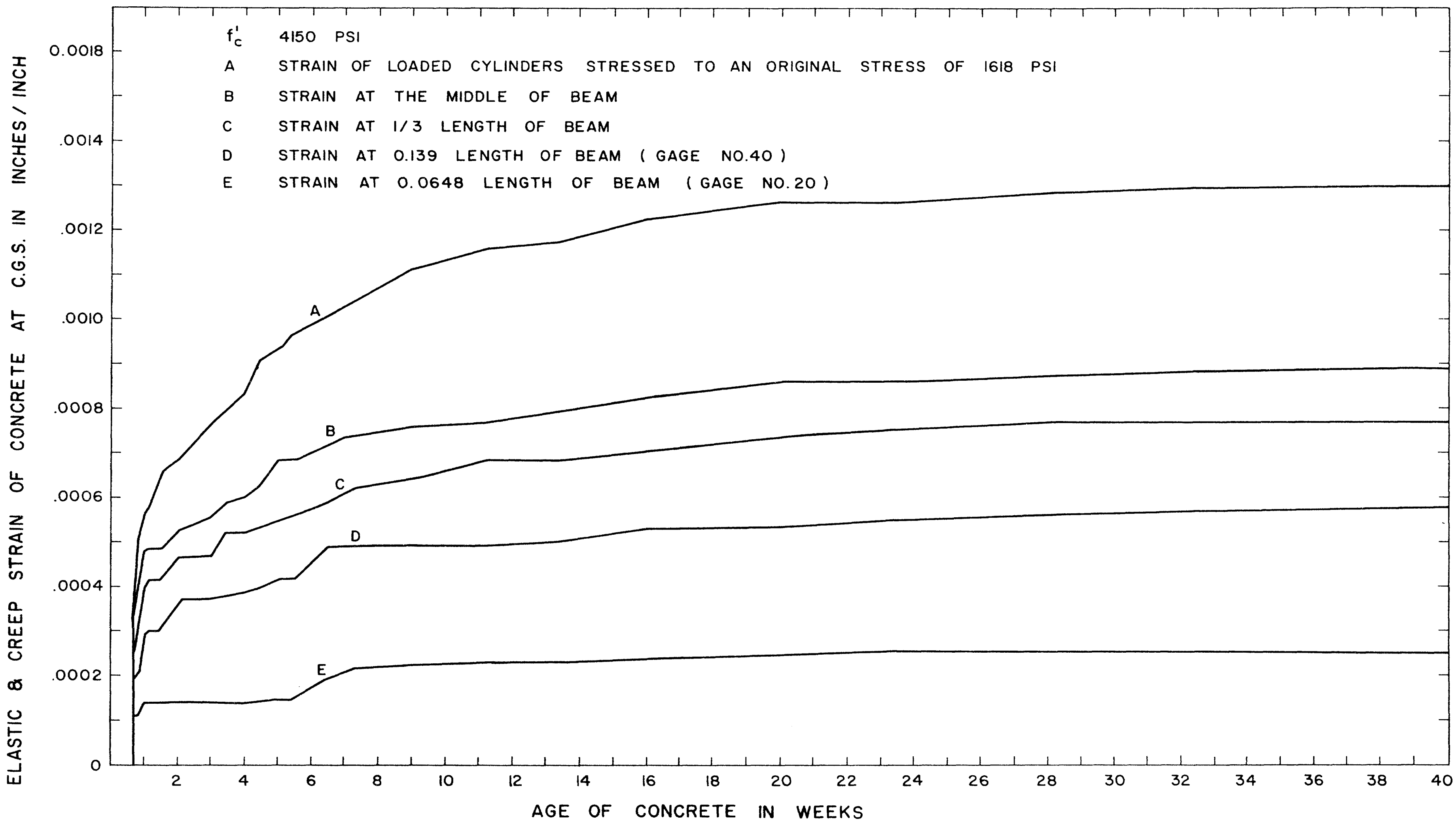



Figure 5.17 Strain vs. Time Curves - Test Series No. 1.

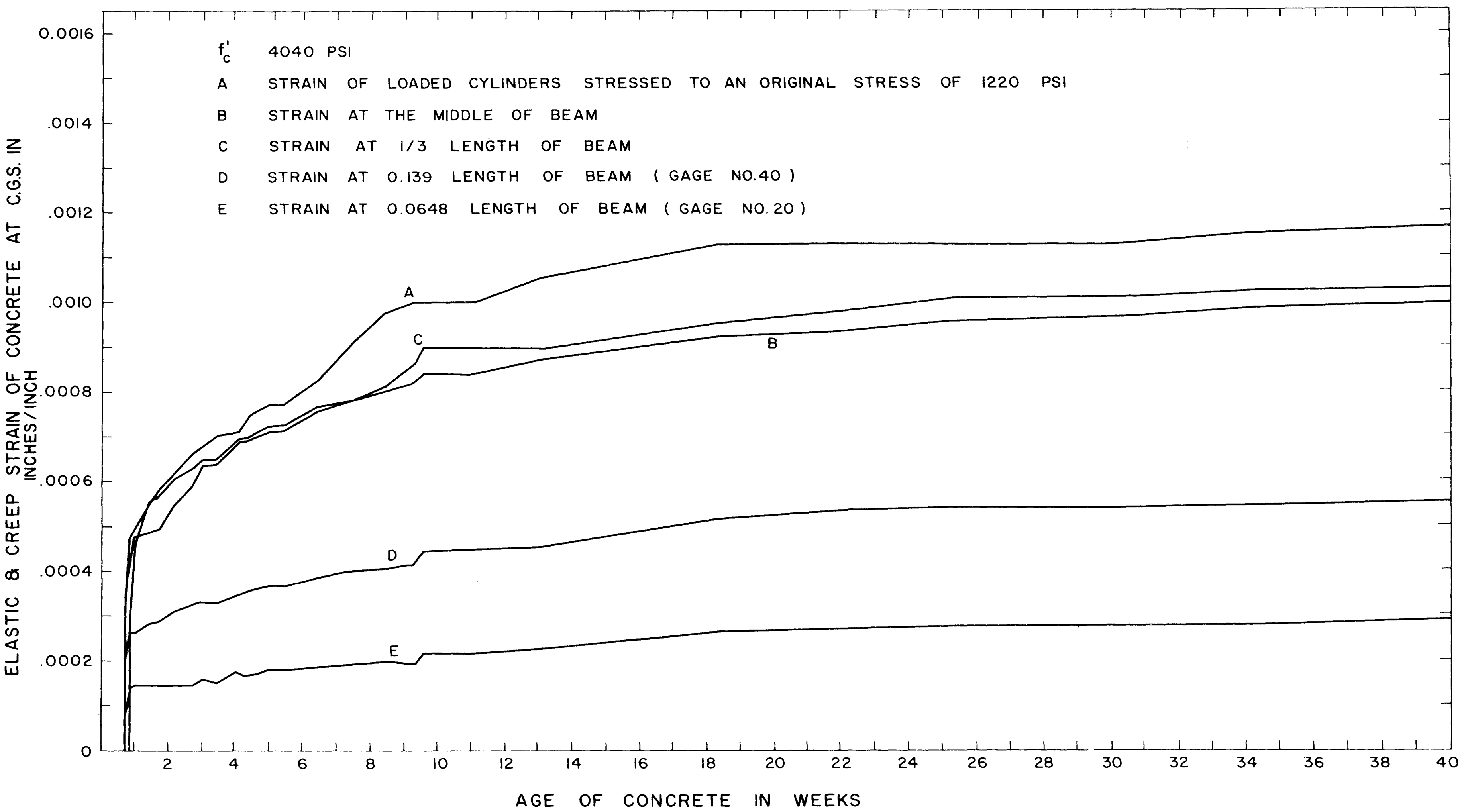



Figure 5.18 Strain vs. Time Curves - Test Series No. 2.

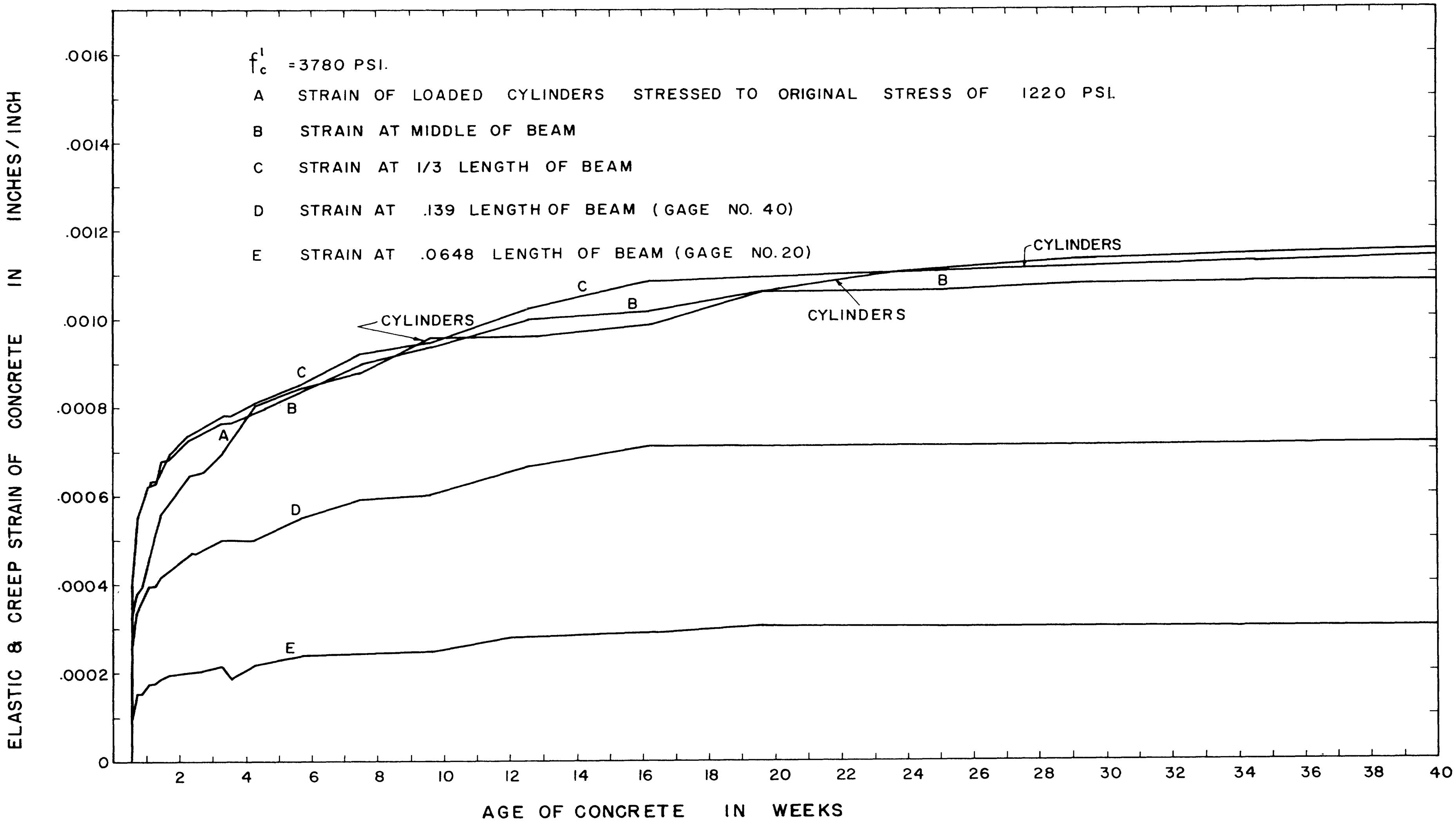



Figure 5.19 Strain vs. Time Curves - Test Series No. 3.

At any gage point, the difference in the strain readings immediately before and after applying the load on concrete (cutting the cables in the case of the prestressed beams and applying the loads on the creep cylinders) will represent the elastic shortening. These elastic strains are shown in Figures 5.17, 5.18 and 5.19 as vertical lines at the age of transfer.

The measured creep of concrete was obtained by first finding the combined value of the creep and elastic strains. This combined value was obtained from the difference between two strain readings. The first reading represents the combined effect of the elastic strain at transfer, creep of concrete strain and shrinkage strains. The second reading represents the cumulative shrinkage of the corresponding shrinkage beam or cylinders. The combined value of creep and elastic strains minus the elastic strain at transfer gives the creep strain.

Table 5.5 represents a typical Whittemore gage data sheet for creep calculations. This table should accompany another one for shrinkage calculations, as that of Table 5.4.

## 5.6 Growth of Camber in the Prestressed Concrete Beams

Growth of camber in the prestressed concrete beams was measured by means of dial gages located exactly under the center of the beams. Each dial gage was attached to a vertical steel rod inserted in a concrete cylinder. These concrete cylinders were cast on the same days as the corresponding test series, and were placed directly on the floor. For this arrangement see Figure 5.20. The ends of the prestressed concrete beams were simply supported on concrete blocks (see Figure 5.13).

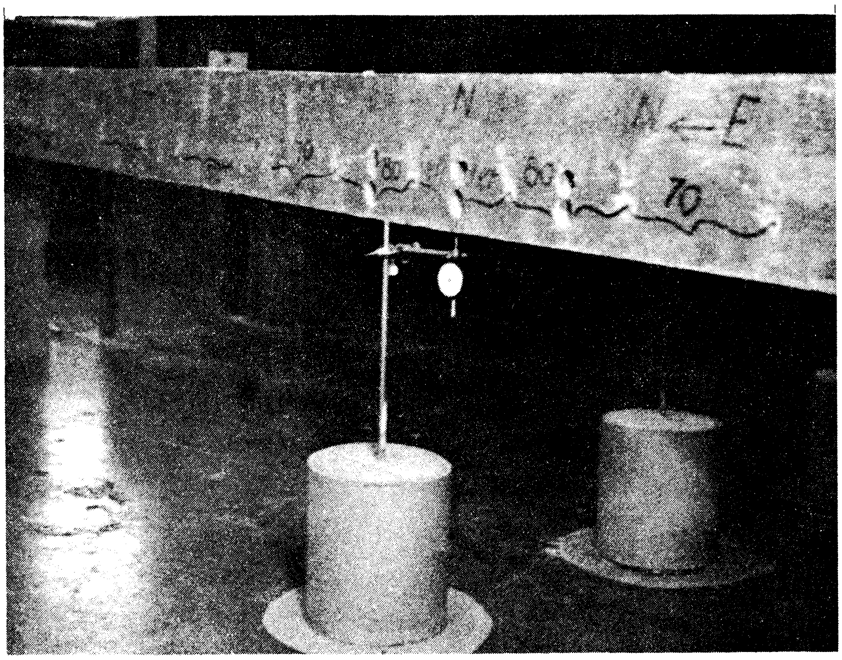



Figure 5.20 Arrangement for Measuring Growth of Camber of the Prestressed Concrete Beams.

These blocks were placed directly on the floor and were built at their specified locations. They were filled with concrete of the same mix as test series Number 2, which was the earliest one. These supports are believed to be very rigid supports, thus the assumptions were made that their deflections were zero and no dial gages were placed there. A 11" x 3" x 1/4" steel plate was placed at the middle of the blocks on the top of each support. The ends of the beams were placed on the top of these plates. The edges of the beams coincided with the outer edges of the plates, thus providing direct supports for the beams equal only to the width of the steel plates.

Great effort was made in the first and second series to measure camber. Camber is the central upward deflection, immediately after cutting the cables, due to the application of the prestress force on the concrete beam. These efforts were unsuccessful, due to the disturbances caused by horizontal as well as vertical movements of the prestressed concrete beams at the time of cutting the cables.

The growth of camber curves are shown in Figures 5.21, 5.22 and 5.23. There is no similarity whatsoever between any two of these curves. This, of course, is due to the different eccentricities of steel used in the beams.

## 5.7 Creep of the Steel Strands

As defined in Section 2.2, creep of steel is the loss of its stress when it is stressed and maintained at a constant strain for a period of time, or the amount of lengthening when maintained under a constant stress for a period of time. Neither of these definitions is applicable

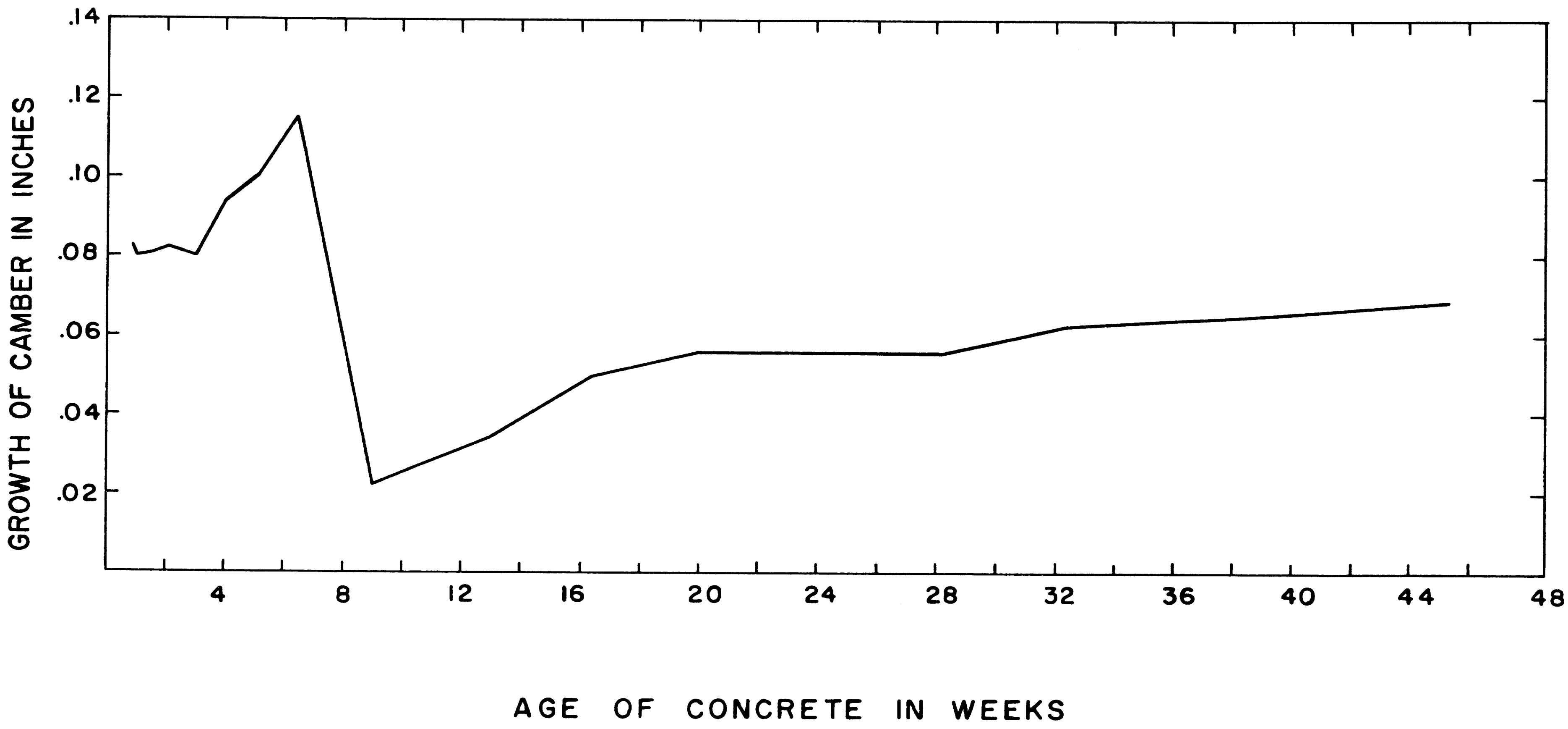



Figure 5.21 Growth of Camber of the Prestressed Beam - Test Series No. 1.

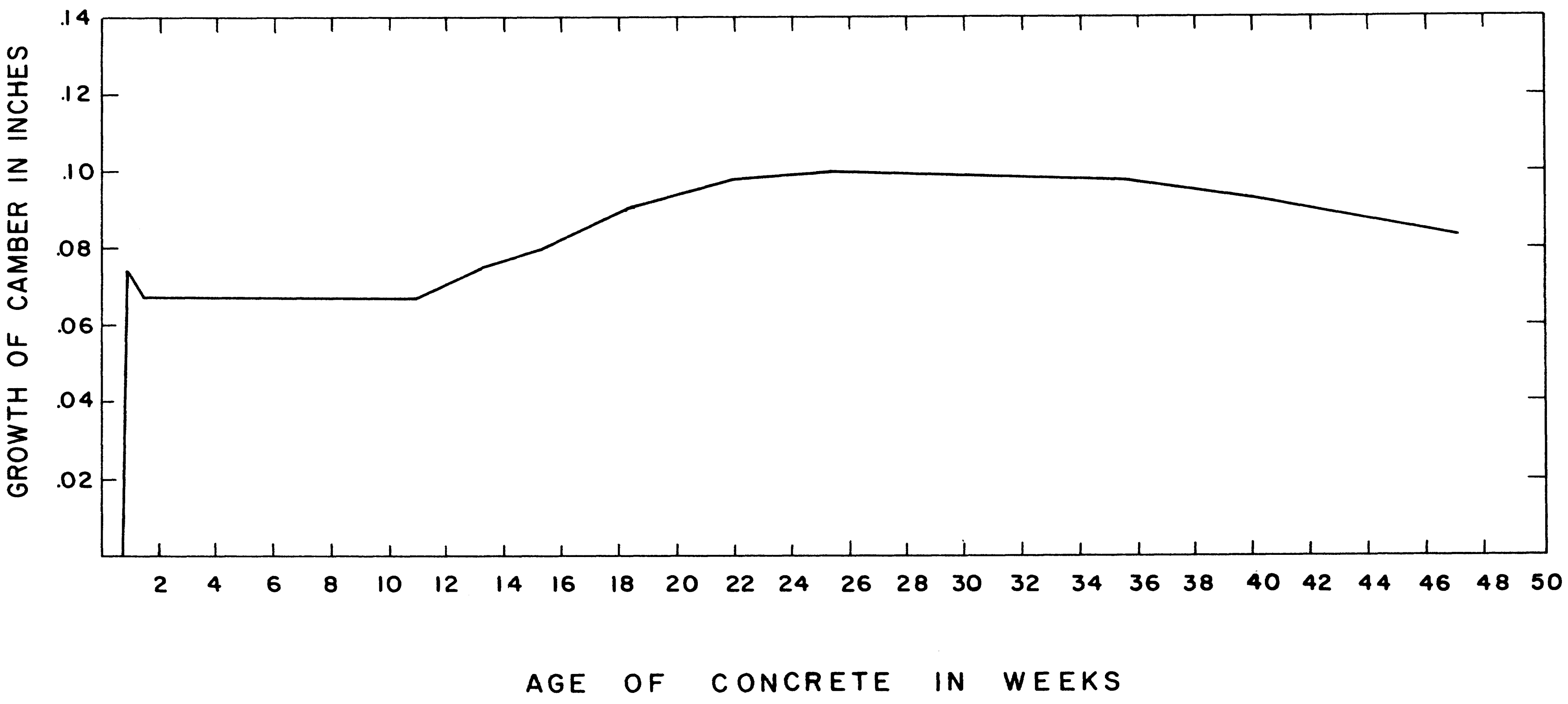



Figure 5.22. Growth of Camber of the Prestressed Beam - Test Series No. 2.

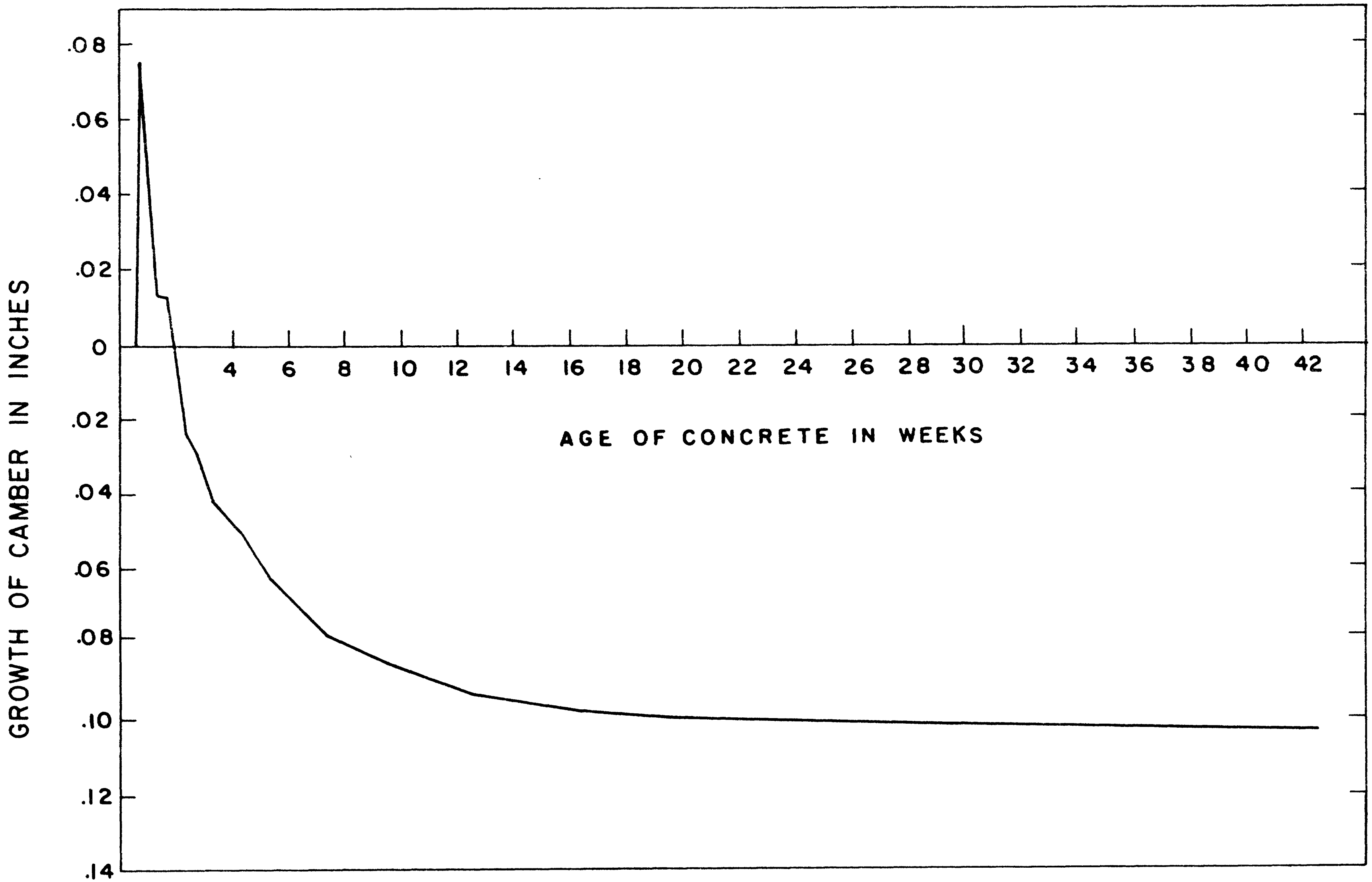



Figure 5.23 Growth of Camber of the Prestressed Beam - Test Series No. 3.

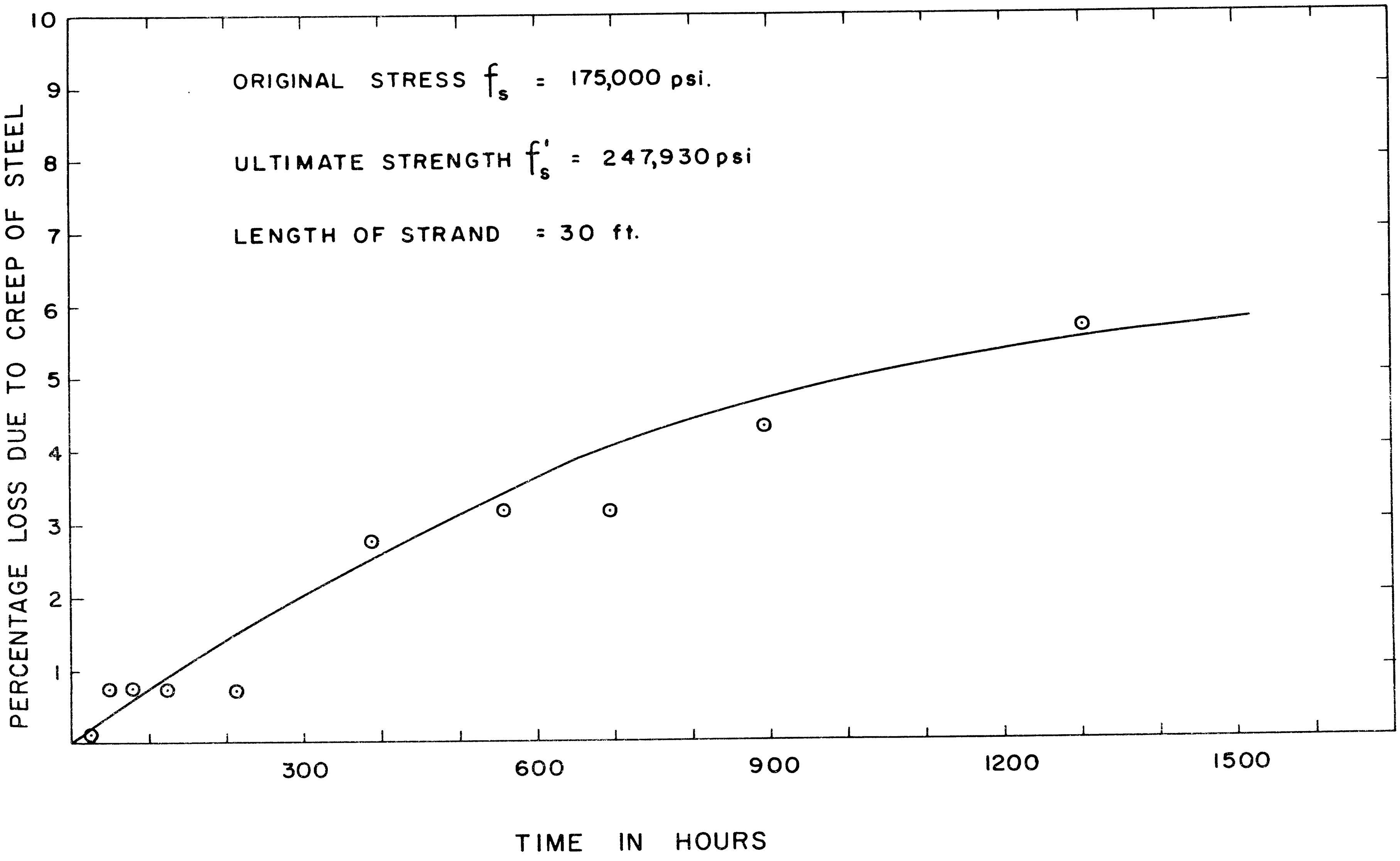



Figure 5.24 Percentage Loss of Stress Due to Creep of Steel vs. Time Curve.

in the case of prestressed concrete beams. The fact is, that creep or relaxation of steel in prestressed concrete beams will occur with simultaneous changes in stress and strain of the steel strands. This is due to creep of concrete and its shrinkage. This investigation is beyond the scope of this thesis although the writer is aware of its importance.

It is to be noted that tests like those discussed in measuring creep and shrinkage of concrete will not yield direct data on the loss attributable to creep of the steel strand.

The test conducted in this investigation is to measure the loss of steel due to its relaxation under constant strain. The procedure was outlined in Section 3.8 and Figure 5.24 represents the percentage loss of prestress after it has been stressed to a stress of 175,000 psi versus time.

## CHAPTER VI

## DISCUSSION

### 6.1 General Discussion

A large number of factors affect prestress force losses; thus it is not an easy task to generalize the results of this investigation. However, the writer did try to vary different parameters in this research, for example, the different eccentricities used in the test program. This made it possible to study the effect of stress distribution, having a constant C.G.S. stress, on the creep of concrete. One important factor, which contributed considerably to the results, was the use of the wrong type of cement. As mentioned before in Section 3.2, the concrete was ordered ready-mixed from a commercial firm and they used Type I cement (ordinary cement) instead of Type III (high early strength cement) as specified. This resulted in delaying the cutting of the steel strands for about three days until the desired strength of the concrete cylinders was achieved. Great efforts were made to avoid shrinkage strains before cutting the cables. This was done by covering the concrete with wet burlap for that period. The temperature of the storage area was kept with a reasonable limit of between 60° and 80°F. There was no way whatsoever of controlling the humidity. Tests were conducted to get the concrete and steel properties necessary for the calculations. This may have contributed considerably to the agreement between the calculated and the experimental results as will be shown later. It would have been preferable to base the concrete modulus on at least three tests, instead of on one test. The method used in averaging at least four individual strain readings to obtain the strain for a specific section of the beam, should have minimized the experimental errors. The technique used in stressing the strands as discussed in Section 3.6 and the use of well calibrated pressure cells is believed to have minimized the variation of the steel stress at anchorage from the design value.

## 6.2 Modulus of Elasticity Tests of Concrete

Some standards suggest that concrete cylinders should be subjected to a load of about one-half the ultimate strength and then unloaded before taking any strain readings. This is to secure a setting of the specimen and to make any necessary adjustment to either the compressometer or to the specimen. However, this procedure was not exercised since the loads in the actual beams are applied directly to the concrete without any preloading. It is assumed that the results obtained without any preloading of the test specimens will be closer to the actual action of the concrete. The secant modulus of elasticity of concrete at half the ultimate load was used in the analysis. The stress strain curves of concrete as shown in Figures 5.2 to 5.8 are not straight lines, even at lower stress levels. The writer believes that the nonlinearity of the stress strain curve is due partly to the low strength of concrete used. Within the range of concrete stresses in the beams (maximum stress $\simeq 0.6\ f'_{ci}$), the modulus of elasticity of concrete is almost equal to the secant modulus at half the ultimate strength, which can be concluded also from Figures 5.2 to 5.8.

## 6.3 Elastic Shortening of Concrete

The difference between strain readings taken immediately before and after the transfer of the prestress force is the elastic strain of concrete. That difference times the concrete modulus of elasticity will represent the concrete stresses, while that difference times the steel modulus of elasticity is numerically equal to the elastic loss of prestress in the zone of complete anchorage. An expression for the elastic loss was derived in Section 2.2.1 and the concrete stresses at C.G.S. immediately after transfer of prestress obtained by this expression were plotted in Figures 5.9 to 5.11 together with the measured concrete stresses at the same location. In the region of full anchorage, a very good agreement between the computed and the measured concrete stresses is noticeable. The following factors contribute to this agreement.

1. In expression (2.3) the actual steel and concrete properties obtained from actual tests for each test series were used. Modulus of elasticity of concrete was obtained from one specimen, although it would be better if it was obtained from at least three specimens.
2. Efforts were made to avoid any variation in the actual cross-sectional dimensions from the assumed section. The transformed area and the moment of inertia corresponding to that transformed area were used in the computations.
3. As mentioned before in Section 3.6, the slippage of the chucks was eliminated by jacking the strand to the desired stress, sliding the chuck tightly against the frame, and then releasing the jack which "seated" the chuck tightly

onto the strand. The strand was then rejacked to the desired stress and the resulting gap between the chuck was tightly shimmed with split washers. The jack was then released and left so for three to four days. The same procedure was repeated at the day of concreting.

4. Apparently there is no appreciable loss due to the creep of steel in the period between the time of concreting and transferring of the prestress, which ranges between four to five days. This could be attributed to two factors:

   A. The creep of these particular steel strands (about 0.75%) within this period is not considerable as shown from the results of a steel creep test (Figure 5.24).

   B. Immediately before casting the concrete the strands were rejacked again to the design stress after it has been left stressed to the design stress for three to four days and the resulting gap, now due to creep in steel and slippage of the chucks was tightly shimmed with similar split washers. This, the writer believes, did minimize the creep of steel within the first five days and thus it would be considered nil in the computations.

5. Shrinkage of concrete before transfer of the prestress force could contribute to the loss of prestress before transfer. Efforts were made to avoid any shrinkage of concrete before transfer by keeping the concrete covered by wet burlap.

However, there must be some shrinkage strain but at what age the concrete has attained sufficient strength and developed enough bond with the steel to induce shrinkage strains in the steel cannot be determined.

6. The method used in averaging four individual strain readings to obtain the strain for a specific section of the beam should have minimized the experimental errors.

## 6.4 Anchorage Length of the Steel Strands

The dependence on bond to transmit prestress between steel and concrete necessitates the use of small wires to ensure good anchorages. However, the use of seven-wire strand, like that used in this research program, with its excellent bonding characteristics, in lieu of individual smooth wires, will permit the use of larger diameters of the steel. The friction between the steel and concrete, which develops when the diameters of the individual wires increase when the strands are cut, a function of Poisson's Ratio, will contribute to decrease the end anchorage length of the wires.

The following are the factors which affect the anchorage length:

1. Size of the wire diameters.
2. Surface conditions of the strands.
3. Concrete strength.
4. Spacing of the wires.

In a series of tests conducted by J. R. Janney* to investigate the nature of bond in pretensioned prestressed concrete, he gave the

* Janney, J.R. "Nature of Bond in Pre-tensioned Prestressed Concrete." - Journal of the Amer. Conc. Inst., Proceedings, 50, (May, 1954), 717.

following remarks:

1. The ability of pretensioned wire to transfer stress to the concrete through bond does not vary a great deal with wire size within the range of 0.1 to .276 inch in diameter.

2. If the bonding of pretensioned wire and concrete is largely a frictional phenomenon, one might not expect the differences relative to concrete strength, since it is doubtful that the coefficient of friction between steel and cement paste is much affected by paste quality. However, concrete quality probably influences the ability of the concrete to sustain the radial pressure that results from the increase in wire diameter.

3. Rusted wires develop the full transfer of prestress at a more rapid rate and in somewhat less distance from the free end.

4. The length required to transfer the pretension force to the concrete varied from about one to three feet from the ends of the members, depending on the factors previously discussed.

Zuidema(12) in a limited series of tests conducted at the University of Michigan, Ann Arbor, Michigan, concluded that the surface conditions do effect considerably the anchorage length. In his tests, he obtained an anchorage length, in general, of less than those obtained by the writer for the same wire diameter and surface conditions. This will be due to the strength of concrete he was using. Also, Patel(94) in in his series of tests of three prestressed concrete members having the same wire diameters and surface conditions, by using the proposed mix with high early strength cement, obtained an average value of anchorage length of 24 inches. This value is considerably less than the writer's values. This is attributed again to the strength of his mix at transfer. It may be concluded from the previous discussions that the strength of concrete is definitely an important factor in the anchorage length.

It may be noticed that in Test Series No. 2, Figure 5.10, there is no transfer of prestress in the end region of three inches. This might be due to turning of the cables suddenly at transfer, which will result in the breaking of bond at the end and slippage could occur resulting in zero stress in the cable.

Reduction of the anchorage length to a minimum is important only in special instances, where large bending moments and/or shear forces occur near the ends, as with railroad ties or short cantilevers. In such cases it is necessary to select a suitable type of prestressing steel insuring special mechanical bond. For simply supported and uniformly loaded beams, however, the anchorage length will be of little influence since the bending moment increases less rapidly than the prestress is developed. Reduction of the anchorage length to a minimum may result in transferring the prestress to the concrete suddenly and may result in localized high stresses at the ends.

## 6.5 Shrinkage of Concrete

Table 6.1 summarizes the shrinkage strains measured at the age of 40 weeks, together with the ultimate shrinkage strains for the shrinkage beams and cylinders. The ultimate shrinkage strains are extrapolated values obtained by extending further the shrinkage time curves shown in Figures 5.14, 5.15 and 5.16 until they are asymptotic to a horizontal line. It appears from these curves that the ultimate shrinkage strains and the shrinkage strains at the age of 40 weeks are very close to each other, which means that concrete at this age has attained most of its shrinkage. It should be noted also that this measured shrinkage strain is only a relative strain, not absolute, since it represents the difference in the shrinkage strain between the age of transfer and 40 weeks, which is assumed to be the effective shrinkage strain to be used in prestress loss calculations. Whether shrinkage could affect prestress loss before the age of transfer is questionable and ambigious. It should be noted also that these shrinkage curves do not represent pure shrinkage but rather represent the effects of changes in temperature and humidity as well as shrinkage. It is clearly seen from Figures 5.14, 5.15 and 5.16 that, in general, the rate of change of shrinkage strain with time is opposite of that rate of change of humidity. In some parts where the two rates are of the same sign, the rate of change of temperature with time will be opposite to that sign. Of course, there are a few exceptions, which may be related to experimental errors. It may also be seen that the temperature variations were kept small, between 60°F to 80°F, except for short periods. This may eliminate the temperature effect on shrinkage strain to some extent.

From Table 6.1 it appears that an average ultimate shrinkage strain of the beams is .0005 inches/inch for all the test series. This shrinkage strain characterizes this particular kind of mix and was exposed to the same weather conditions.

TABLE 6.1

AVERAGE MEASURED SHRINKAGE STRAINS OF CONCRETE AT THE AGE OF 40 WEEKS AND THE ULTIMATE SHRINKAGE STRAINS

| Test Series No. | $f'_c$(psi) | Av. Shrinkage Strains in In./In. at the Age of 40 Wks. | | Ultimate Shrinkage Strains in In./In. | |
|---|---|---|---|---|---|
| | | Beam | Cylinders | Beam | Cylinders |
| 1 | 4150 | .00048 | .00050 | .00050 | .00050 |
| 2 | 4040 | .00045 | .00055 | .00048 | .00055 |
| 3 | 3780 | .00049 | .00052 | .00052 | .00055 |

It appears also from Figures 5.14, 5.15 and 5.16 and Table 6.1 that shrinkage of cylinders is more than shrinkage of beams. This is related to the reinforcement of beams which resists shrinkage. The ratio of average cylinder shrinkage to average beam shrinkage was 1.06.

It appears also from these curves that about half of the shrinkage strains occurred during the first 28 days. This ratio confirms work of other investigators.[12,94] Thus a shrinkage strain of .00025 inches/inch, which was assumed in Section 4.1 in calculating the prestress loss, appears to be very low. The actual measured shrinkage strain is twice as much as the assumed shrinkage strain.

The last Tentative Recommendations for Prestressed Concrete published by ACI-ASCE Joint Committee 323 recommended a value of shrinkage strain between .0002 and .0003. However, in the new ACI Code there is no definite value of this shrinkage strain.

At the same water-cement ratio, the writer concluded from his and Patel's[94] investigations, that the concrete made with Type I and

Type III portland cement with the same water-cement ratio generally showed similar drying shrinkage values.

However, each mix has its own shrinkage characteristics and the designer should select the shrinkage data derived from mixes which closely resemble the mix to be used and exposed to similar weather conditions.

6.6 Creep of Concrete

Table 6.2 summarizes the average measured creep and elastic strains of concrete measured at the age of 40 weeks, together with the ultimate creep strains at four locations on the main beams and the cylinders. The ultimate creep strains are extrapolated values obtained by extending further the creep time curves shown in Figures 5.17, 5.18 and 5.19 until they are asymptotic to the horizontal line. In the same table the ratios between the ultimate measured creep strain and the measured elastic strain at the same locations are given.

Table 6.2 and Figures 5.17, 5.18 and 5.19 will now be discussed.

1. The ratio between ultimate creep strain and the elastic strain at the middle section of the beam is not constant. This ratio depends upon the stress level, the strength of concrete, age of concrete at the application of stress, the quality of aggregates and cement, and the moisture content of the concrete. The relation between this ratio and the stress level is taken care of indirectly by multiplying the elastic strain, which is a function of the stress level, by this ratio to get the ultimate creep strain. What

TABLE 6.2

AVERAGE MEASURED CREEP STRAINS AND ELASTIC
STRAINS OF CONCRETE

| Test Series No. | | 1 | 2 | 3 |
|---|---|---|---|---|
| $f'_c$ in psi at 28 days | | 4150 psi | 4040 psi | 3780 psi |
| Avg. Creep + Elastic Strains In Inches/Inch at the Age of 40 Wks. at: | .0608L | .00026 | .00029 | .00030 |
| | .139L | .00058 | .00055 | .00071 |
| | .333L | .00077 | .000103 | .00115 |
| | .5L | .00089 | .00099 | .00108 |
| | Cylinders | .00130 | .00116 | .001135 |
| | | (1618)* | (1220)* | (1220)* |
| Ultimate Creep + Elastic Strains in Inches/Inch at: | .0608L | .00026 | .00030 | .00030 |
| | .139L | .00060 | .00056 | .00071 |
| | .333L | .00077 | .00103 | .00117 |
| | .5L | .00090 | .0010 | .0011 |
| | Cylinders | .00130 | .00118 | .00115 |
| Elastic Strains In Inches/Inch at: | .0608L | .00011 | .00008 | .00010 |
| | .139L | .00019 | .00019 | .00024 |
| | .333L | .00026 | .00034 | .00038 |
| | .5L | .000335 | .00037 | .00038 |
| | Cylinders | .000335 | .00033 | .00032 |
| Ultimate Creep Strain in Inches/Inch at: | .0608L | .00015 | .00022 | .00020 |
| | .139L | .00041 | .00037 | .00047 |
| | .333L | .00051 | .00069 | .00079 |
| | .5L | .000565 | .00063 | .00072 |
| | Cylinders | .00095 | .00085 | .00083 |
| Ratio of Ultimate Creep Strain to Elastic Strain at: | .0608L | 1.36 | 2.75 | 2.00 |
| | .139L | 2.16 | 1.95 | 1.96 |
| | .333L | 1.96 | 2.03 | 2.08 |
| | .5L | 1.69 | 1.70 | 1.89 |
| | Cylinders | 2.83 | 2.56 | 2.60 |

*Number in parenthesis is the original stress of the concrete cylinders.

remains are certain other factors, in which each factor individually is a function of the strength of concrete. In such a case, that ratio is primarily a function of strength of concrete. In Figure 6.1 this ratio is plotted as a function of $f'_c$ at 28 days; the results of other investigators[12,94] are also plotted. The equation

$$r = 28.5/\sqrt[3]{f'_c} \qquad (6.1)$$

which is plotted on the same figure seems to conform to a reasonable degree of accuracy with the test results. In this equation:

$r$ = ratio of ultimate creep strain to elastic strain at transfer.

$f'_c$ = concrete strength at the age of 28 days in psi.

Such an equation, although it is approximate in nature and needs more experimental verification, gives a reasonable estimate of $r$, much better than if it is assumed a constant value. In plants, in which there is close supervision, control of materials and a specialized work force, the writer believes that a ratio ($r$) may be taken as 1.6 for concrete strength of $f'_c$ (at 28 days) in the neighborhood of 6,000 psi.

2. Cylinders stressed to an average stress of $(F_o/A_t)$ of their corresponding beams, behaved differently in the three test series. In Test Series No. 1, in which the beam has the smallest eccentricity, the ratio between the ultimate creep

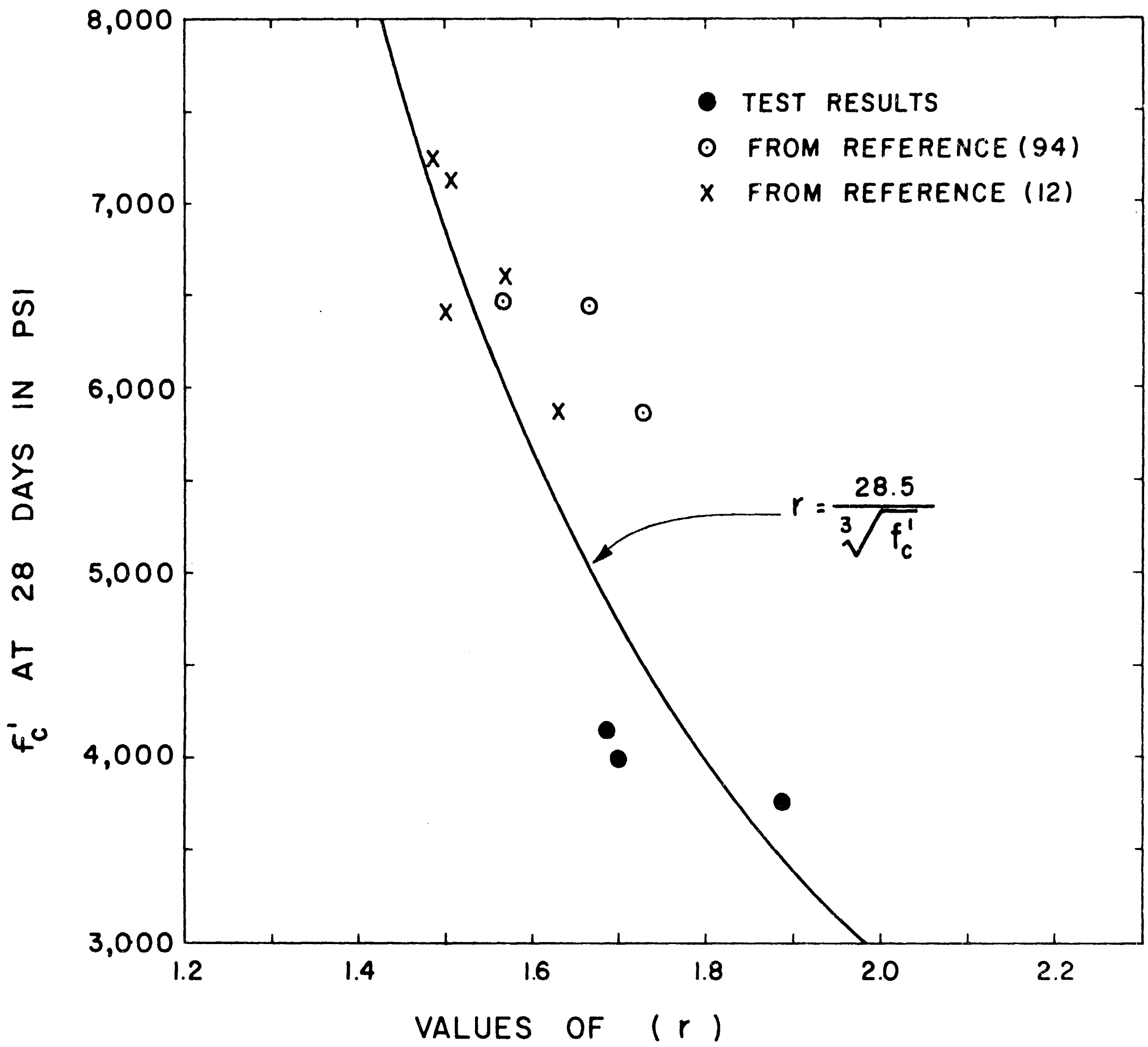



Figure 6.1 Ratio of Ultimate Creep Strain of Concrete to Elastic Strain at Transfer (r) vs. $f_c'$ at 28 Days.

strain and elastic strain of the cylinders and beam is 1.45. This ratio decreases, as the eccentricity increases, until it becomes almost one for eccentricity just below the Kern limit. In general, creep in cylinders is more than that in beams as was expected. A definite relation between creep of cylinders and creep of P.C. beams cannot be derived from this experiment, nor from Patel's investigation.(94) Cylinders under stress are subject to a considerably less rate of decrease of load than beams. This is related to the various losses of prestress force in the cables. Even with cylinders stressed to the average original stress in the beams, the creep of cylinders is higher. This means that the distribution of stress is definitely playing a role in the creep of concrete.

3. From Table 6.2 the average ratio between creep strain and elastic strain at L/3, which is the point of cable bending, is 15% higher than the average ratio at the middle section of the beams. Since there is a vertical component of the prestressing force of bent tendons, it will produce shearing stresses. The stresses along the cables, and thus the strains, will be higher than if the cables are straight. However, since the angle of the inclined tendons in this investigation is too small to produce such an effect, it is believed that at the point of cable bending there may be some stress concentrations which result in higher creep strains.

6.7 Creep of Steel

Figure 5.24 shows the creep of the steel under constant strain (seven wire, cold drawn, high carbon stress relieved strands), stressed to the same original stress (0.7 $f_s'$) as the steel of the prestressed concrete beams.

A percentage loss in steel stress, due to creep, of 5% is noticeable at the age of 1000 hours. Beyond this time the creep curve is still increasing reaching an ultimate value of 6%. These values conform in general with those obtained in References 93 and 94. However, contradictory to Reference 94, the rate of stress loss due to creep of steel is not high in the early ages, but this rate conforms with that obtained in Reference 93. The agreement between the computed and measured elastic losses as given in Section 6.3, indicates also that creep of steel is not excessive in the early ages. This comparison is made for the same kind of steel stressed to the same stress level of 0.7 $f_s'$. It may be noticed in the same figure that points do not fall exactly on the creep time curve. This is due to the effect of temperature fluctuations to weather conditions. The writer is not in a position to generalize these observations, since only one test was conducted on a specific kind of steel stressed to a specific stress level.

In prestressed concrete beams in which the steel stresses fall rapidly after cutting of cables, due to elastic shortening, creep and shrinkage of concrete, loss due to creep of steel is less than 6% obtained in this investigation. This reduction in the steel stresses may reach 0.55 $f_s'$, at which the creep of steel is found to be negligible.[93]

An approximate value of 4% loss due to creep of steel may be predicted in this investigation of which very little occurred before transfer of stress. In plants in which the procedure exercised in this report to avoid creep of steel before transfer (Section 3.6) is impractical, the same value may be predicted of which only 1% occurred before transfer of prestress.

It may be emphasized that these values refer to this kind of steel (super stress relieved manufactured by U.S.S.) stressed to the same stress level of $0.7\ f_s'$.

6.8 Total Losses

Table 6.3 summarizes the total prestress losses at the C.G.S. for each prestressed concrete beam based on actual measured data of creep and shrinkage strains. These losses are given for the two locations at mid section and $L/3$ section.

The average percentage total losses as given in Table 6.3 is 28%. It is not intended here to give the prestress concrete losses as a percentage, as improperly assumed in some design procedures, but mainly to indicate that there is a substantial amount of prestress lost. The designer should compute these losses as accurately as possible to assure good economical design procedure.

A question here may be asked. As the force in the cables decreases due to creep and shrinkage, does elastic losses decrease? In an attempt to answer this question Birkenmaier[71] gives the following equation that represents the losses due to creep, shrinkage of concrete and

TABLE 6.3

ULTIMATE PRESTRESS LOSS IN THE PRESTRESSED BEAMS

| Test Series No. | Location | $F_o$ (lb.) | $F_i$ (lb.) | $\frac{\text{Creep Strain}}{\text{Elastic Strain}}$ | Shrinkage Strain | Elastic Losses (lb.) | Concrete Creep Losses (lb.) | Shrinkage Losses (lb.) | Steel Creep Losses (lb.) | Total Losses (lb.) | F (lb.) | % Elastic Loss | % Loss Due to Concrete Creep | % Loss Due to Shrinkage | % Total Losses |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | $\frac{L}{3}$ | 76,230 | 72,280 | 1.96 | .0005 | 3,950 | 7,742 | 6,098 | 3,049 | 20,839 | 55,391 | 5.18 | 10.15 | 8. | 27.34 |
| | $\frac{L}{2}$ | 76,230 | 72,289 | 1.69 | .0005 | 3,941 | 6,660 | 6,098 | 3,049 | 19,748 | 56,482 | 5.17 | 8.74 | 8. | 25.91 |
| 2 | $\frac{L}{3}$ | 57,173 | 54,108 | 2.03 | .00048 | 3,064 | 6,220 | 4,391 | 2,287 | 15,962 | 41,211 | 5.36 | 10.88 | 7.68 | 27.92 |
| | $\frac{L}{2}$ | 57,173 | 54,132 | 1.70 | .00048 | 3,040 | 5,168 | 4,391 | 2,287 | 14,886 | 42,287 | 5.62 | 9.55 | 7.68 | 26.04 |
| 3 | $\frac{L}{3}$ | 57,173 | 53,665 | 2.08 | .00052 | 3,507 | 7,295 | 4,757 | 2,287 | 17,846 | 39,327 | 6.13 | 12.75 | 8.32 | 31.22 |
| | $\frac{L}{2}$ | 57,173 | 53,696 | 1.39 | .00052 | 3,477 | 6,572 | 4,757 | 2,287 | 17,093 | 40,080 | 6.08 | 11.49 | 8.32 | 29.90 |

the elastic losses taking into consideration the "elastic resilience:"*

$$X = K\,(1 - e^{-xr}) \qquad (6.2)$$

in which

$X$ = loss of prestress in units of force due to shrinkage and creep of concrete and elastic shortening considering elastic resilience.

$x = \dfrac{n\mu}{1 + n\mu}$ where $n$ = Poisson's Ratio

and $\mu = \dfrac{As}{Ac\,\dfrac{i_t^2}{e_t}}\left(e_t + \dfrac{i_t^2}{e_t}\right)$

$r = \dfrac{\text{Ultimate creep strain of concrete}}{\text{Initial elastic strain}}$ (for the cylinders)

$$K = F_o - \frac{M_x}{e_t + \dfrac{i_t^2}{e_t}} + q\,E_c A_t\,\frac{\dfrac{i_t^2}{e_t}}{\dfrac{i_t^2}{e_t} + e_t} \qquad (6.3)$$

where $i_t$ is the radius of gyration of the transformed section

$q$ = ultimate shrinkage strain/r.

Birkenmaier assumed that $E_c$ is constant with time which is on the safer side and he assumed also that creep and shrinkage function have the same form, which is a reasonable assumption as shown from Figures 5.14 to 5.19. In his expression the ultimate value of creep of concrete cylinders of the same mix must be used.

* Elastic resilience is the term used herein to indicate the change in the elastic shortening of concrete with time, due to the decreasing prestressing force.

In this investigation the values of creep of concrete given are actually the combined effect of creep of concrete and elastic resilience. In Figure 6.2 the actual creep strains and the measured creep strains at the C.G.S. of the middle section of the prestressed concrete beams are given. The actual creep strains are computed after making the following two assumptions:

1. The modulus of elasticity of concrete is the same as that at time of transfer of the prestress. An assumption on the safer side.
2. Loss due to creep of steel is 4% of the original prestressing force and this does not change with time. An assumption that makes some error in the first ten weeks but beyond that time creep of steel can be assumed constant.

By these two assumptions the prestress force $F$ at any time $t$ is

$$F = F_o - (.04\ F_o + \epsilon\ A_s E_s) \qquad (6.4)$$

in which

$\epsilon$ = total measured strain at the C.G.S. in inches/inch.

The stresses due to this force $F$ at the C.G.S. are

$$f_{cgs} = -F \left\{\frac{1}{A_t} + \frac{e_t^2}{I_t}\right\} + \frac{M_x e_t}{I_t} \qquad (6.5)$$

and thus the actual elastic strain is

$$\epsilon_{cgs} = f_{cgs}/E_c \qquad (6.6)$$

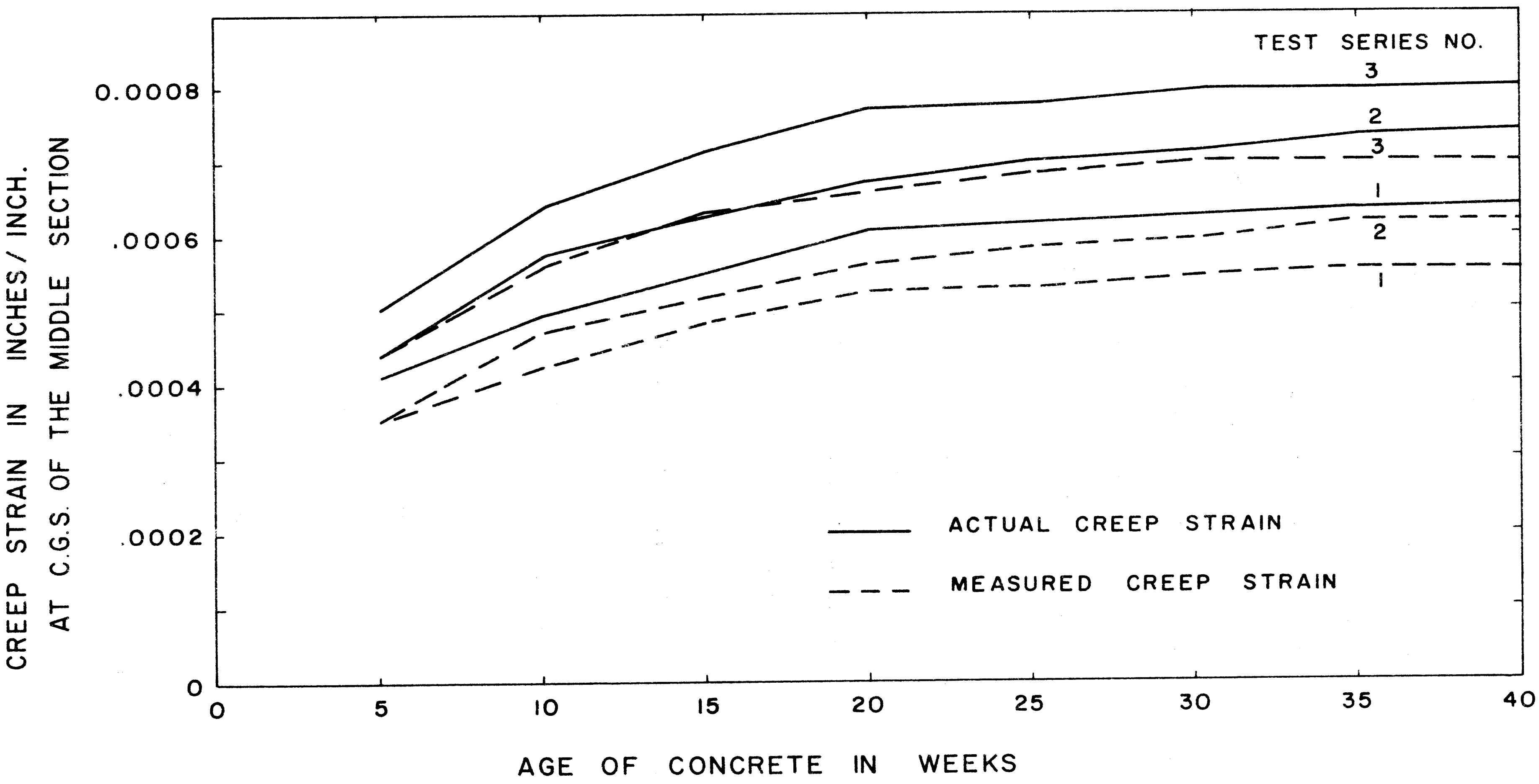



Figure 6.2 Creep of Concrete at the C.G.S. of the Middle Section.

By deducting this value and the shrinkage strain from the total strain one gets the actual creep strain. Table 6.4 gives the values of the actual creep strains which are plotted in Figure 6.2.

By doing so, the ratio between ultimate creep strains to the original elastic strains at the C.G.S. of the middle section raises to 1.94, 2.03 and 2.17 for the three test series respectively, an average increase of about 16% from the apparent ratios. This might explain why creep of cylinders is higher than creep of prestressed concrete beams.

In Table 6.5 the Birkenmaier expression, as given in Equation (6.2), is computed at the middle section of the three beams. In comparing the losses as given by his expression with the measured losses due to creep, shrinkage and elastic shortening of concrete, his expression gives smaller values. This may be due to the lower r ratios used in this investigation, since cylinders were subjected to a decreasing load and not to constant sustained loads.

Figures 6.3 to 6.5 represent the creep plus elastic strain versus time curves at the middle section of the prestressed beams. These curves are given at the top fibers, top cables, C.G.S., bottom cables and bottom fibers. It was hoped that these curves would be satisfactory and accurate enough to give a good picture of the strain distribution, and thus the stress distribution, at the middle section of the beams. From this distribution, the actual force in the cables could be computed by considering a free body diagram of half the beam. Due to the difficulties encountered in taking the strain reading on the cables as discussed in Section 3.4, these readings were not satisfactory as shown from the same curves. In general, they gave strain readings of

TABLE 6.4

ACTUAL CREEP AND ELASTIC STRAIN AT THE C.G.S. OF THE MIDDLE SECTION OF THE PRESTRESSED CONCRETE BEAMS

| Test Series No. | Age in Weeks | Creep + Elastic Strain x $10^6$ | Shrinkage Strain x $10^6$ | Total Strain x $10^6$ | Strain x $E_sA_s$(lb) | F(lb.) | $f_{cgs}$ (psi) | Actual Elastic Strain x $10^6$ | Actual Creep Strain x $10^6$ | Measured Creep Strain x $10^6$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 (335)* | 5 | 685 | 295 | 980 | 11,953 | 61,228 | 1310 | 273 | 412 | 350 |
| | 10 | 760 | 360 | 1120 | 13,661 | 59,520 | 1272 | 265 | 495 | 425 |
| | 15 | 810 | 425 | 1235 | 15,063 | 58,118 | 1241 | 258 | 552 | 475 |
| | 20 | 860 | 465 | 1325 | 16,161 | 57,020 | 1217 | 253 | 607 | 525 |
| | 25 | 865 | 480 | 1345 | 16,405 | 56,776 | 1212 | 252 | 613 | 530 |
| | 30 | 880 | 470 | 1350 | 16,466 | 56,715 | 1211 | 252 | 628 | 545 |
| | 35 | 890 | 470 | 1360 | 16,588 | 56,593 | 1207 | 251 | 639 | 555 |
| | 40 | 890 | 480 | 1370 | 16,710 | 56,471 | 1205 | 251 | 639 | 555 |
| | Ult. | 900 | 500 | 1400 | 17,076 | 56,105 | 1197 | 249 | 651 | 565 |
| 2 (370)* | 5 | 720 | 235 | 955 | 8,736 | 46,150 | 1189 | 279 | 441 | 350 |
| | 10 | 840 | 320 | 1160 | 10,612 | 44,274 | 1137 | 267 | 573 | 470 |
| | 15 | 890 | 365 | 1255 | 11,481 | 43,405 | 1112 | 261 | 629 | 520 |
| | 20 | 930 | 390 | 1320 | 12,076 | 42,810 | 1096 | 257 | 673 | 560 |
| | 25 | 955 | 410 | 1365 | 12,487 | 42,399 | 1084 | 255 | 700 | 585 |
| | 30 | 965 | 435 | 1400 | 12,807 | 42,079 | 1075 | 252 | 713 | 595 |
| | 35 | 990 | 440 | 1430 | 13,082 | 41,800 | 1067 | 251 | 739 | 620 |
| | 40 | 990 | 450 | 1440 | 13,173 | 41,713 | 1065 | 250 | 740 | 620 |
| | Ult. | 1000 | 480 | 1480 | 13,539 | 41,347 | 1055 | 248 | 752 | 630 |
| 3 (380)* | 5 | 820 | 190 | 1010 | 9,239 | 45,647 | 1330 | 318 | 502 | 440 |
| | 10 | 945 | 290 | 1235 | 11,298 | 43,588 | 1264 | 303 | 642 | 565 |
| | 15 | 1010 | 335 | 1345 | 12,304 | 42,582 | 1232 | 295 | 715 | 630 |
| | 20 | 1060 | 365 | 1425 | 13,036 | 41,850 | 1208 | 289 | 771 | 660 |
| | 25 | 1065 | 390 | 1455 | 13,310 | 41,576 | 1199 | 287 | 778 | 685 |
| | 30 | 1080 | 415 | 1495 | 13,676 | 41,210 | 1188 | 284 | 796 | 700 |
| | 35 | 1080 | 450 | 1530 | 13,996 | 40,890 | 1177 | 282 | 798 | 700 |
| | 40 | 1080 | 490 | 1570 | 14,362 | 40,524 | 1166 | 279 | 801 | 700 |
| | Ult. | 1100 | 520 | 1620 | 14,820 | 40,066 | 1151 | 275 | 825 | 720 |

*Values in parentheses are calculated elastic strain x $10^6$ at transfer at C.G.S. of the middle section.

TABLE 6.5

BIRKENMAIER EQUATION OF LOSSES

| Test Series Number | 1 | 2 | 3 |
|---|---|---|---|
| r of Cylinders | 2.83 | 2.56 | 2.60 |
| Ultimate Shrinkage | $500 \times \overline{10}^{6}$ | $480 \times \overline{10}^{6}$ | $520 \times \overline{10}^{6}$ |
| q | $177 \times \overline{10}^{6}$ | $187 \times \overline{10}^{6}$ | $200 \times \overline{10}^{6}$ |
| $i_t^2/e_t$ | 13.52 | 4.60 | 3.63 |
| K | 113,300 | 81,820 | 79,250 |
| x | .0528 | .0568 | .0657 |
| xr | .149 | .1454 | .171 |
| $(1 - e^{-xr})$ | .139 | .135 | .157 |
| X(lb.) | 15,749 | 11,048 | 12,442 |
| $\epsilon E_s A_s$ from Table 6.4 | 17,076 | 13,539 | 14,820 |

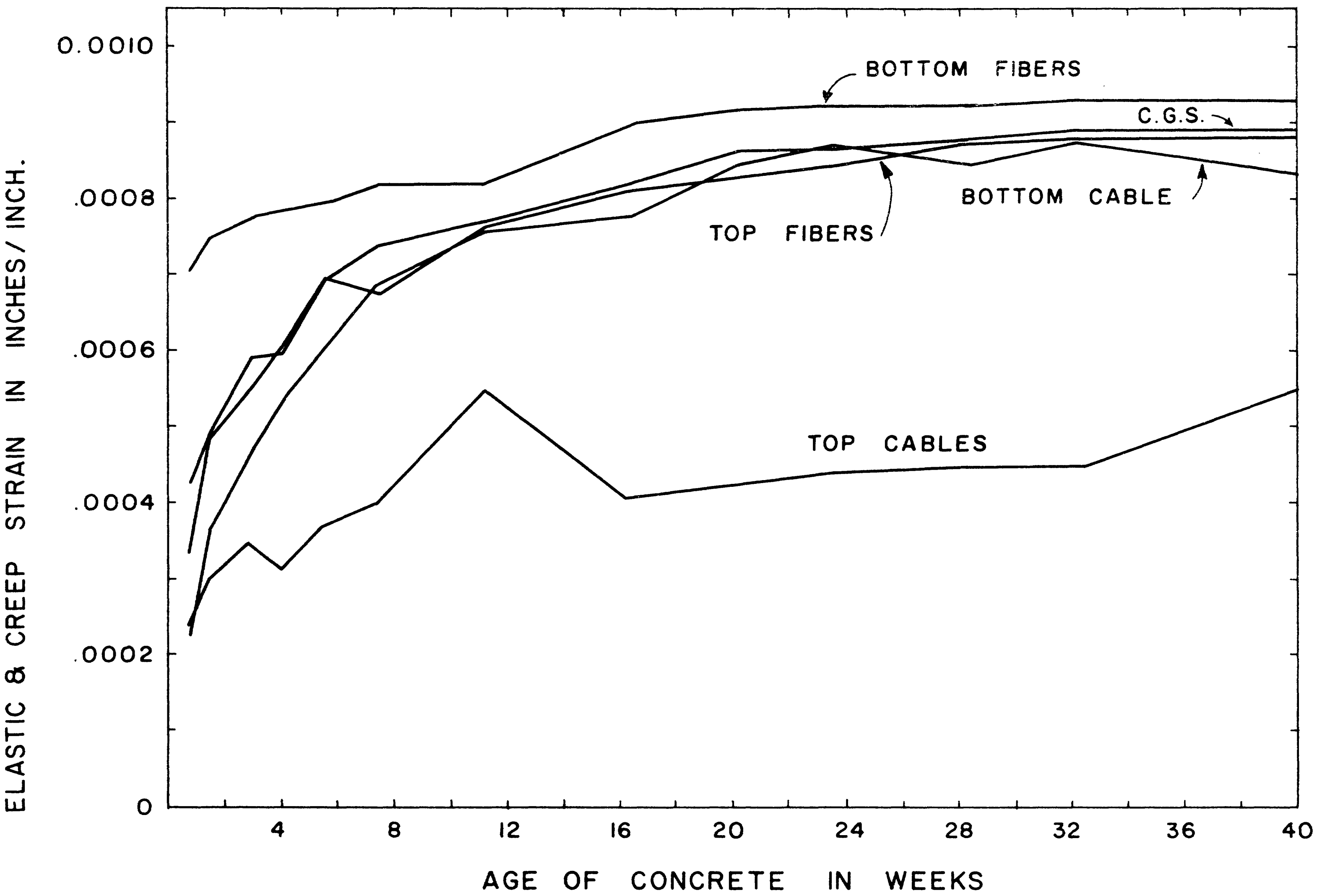



Figure 6.3 Strain vs. Time Curves at the Middle Section of the Prestressed Beams - Test Series No. 1.

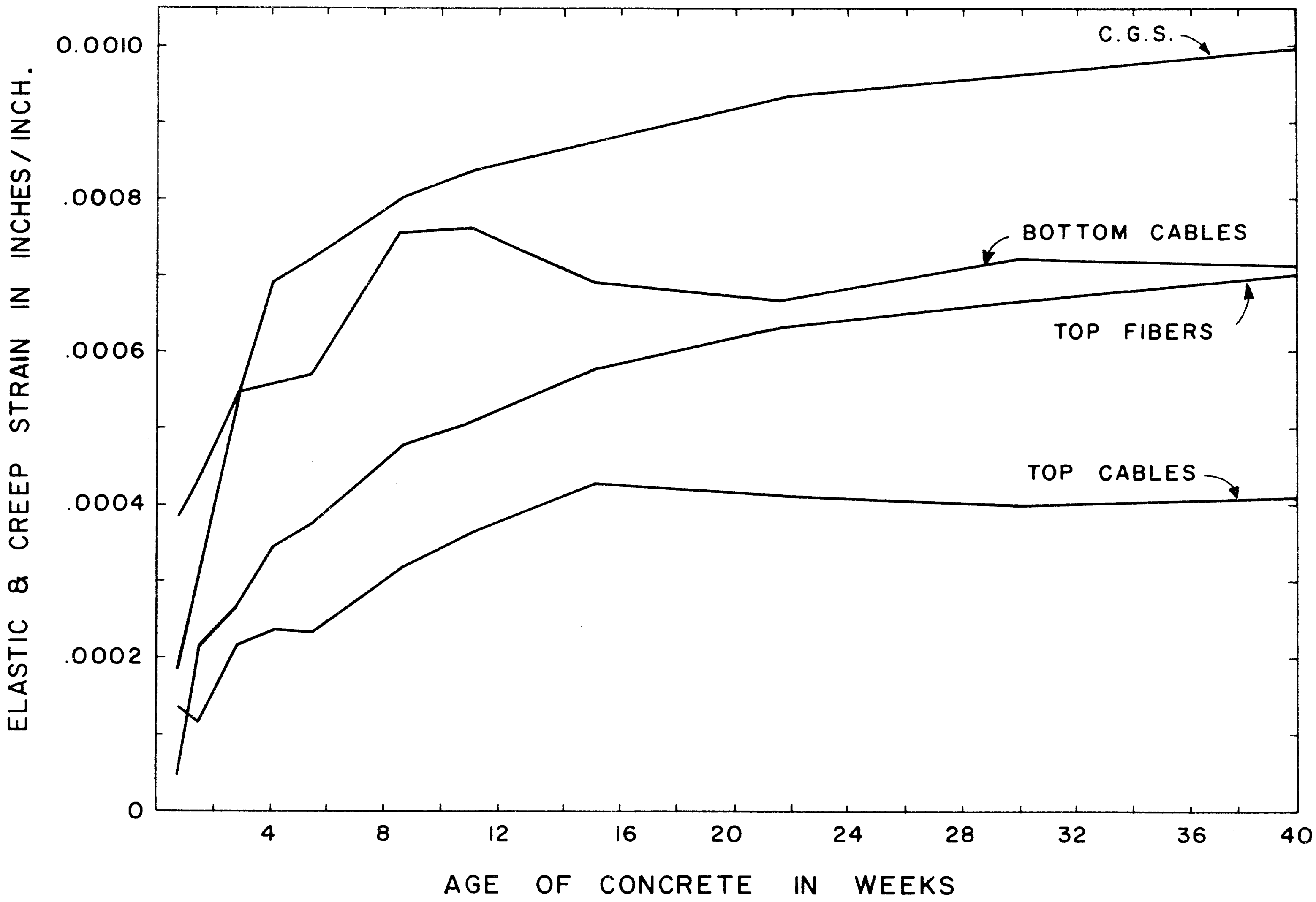



Figure 6.4 Strain vs. Time Curves at the Middle Section of the Prestressed Beams - Test Series No. 2.

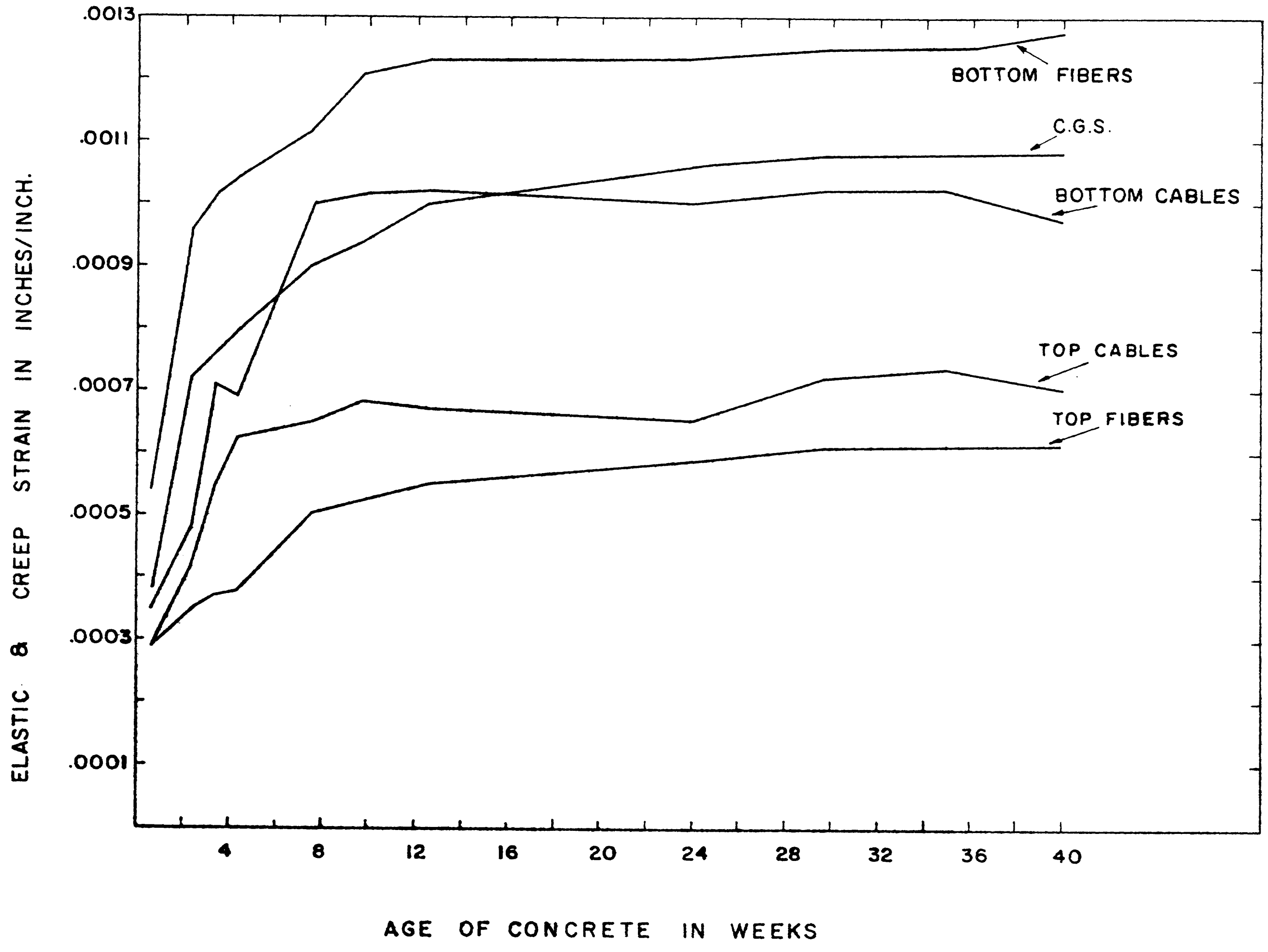



Figure 6.5 Strain vs. Time Curves at the Middle Section of the Prestressed Beams - Test Series No. 3.

less than was expected and they showed, in some intervals, a negative rate of change of strain.

## 6.9 Growth of Camber of the Prestressed Beams

Figures 5.21 to 5.23 represent the growth of camber at the middle section of the prestressed beams. There is no similarity whatsoever between any two of these curves. This, of course, is due to the different eccentricities of steel used in the beams. In general, growth of camber is dependent on two major factors, the rate of decrease of the prestressing force F and the rate of increase of curvatures, due to the differential creep strain between the top and bottom fibers. If the creep effect is more than the loss of prestress effect, the growth of camber is positive, i.e., upwards, and vice versa. In Figure 5.21 there is a sudden change in this growth at about the age of six weeks, which may be due to external disturbance of the dial gage.

## CHAPTER VII

## CONCLUSIONS AND RECOMMENDATIONS

### 7.1 Conclusions

From the testing program conducted in this investigation, the following conclusions are derived:

1. Within the range of concrete stresses in the beam (maximum stress $\simeq 0.6 f'_{ci}$), the modulus of elasticity of concrete is almost equal to the secant modulus at half the ultimate strength. Section 6.2.
2. In the region of full anchorage there is a very good agreement between the computed and measured concrete stresses at transfer. Section 6.3.
3. There is no appreciable loss of prestress due to creep of the steel in the period between the time of concreting and transferring of the prestress, which ranges between four to five days. Section 6.3.
4. The strength of concrete is definitely an important factor in the anchorage length. Section 6.4.
5. An average ultimate shrinkage strain of .0005 inches/inch is obtained, of which about half occurred during the first 28 days. Section 6.5.
6. The ratio between average cylinder shrinkage to average beam shrinkage is 1.06. Section 6.5.
7. At the same water-cement ratio, concrete made with Type I and Type III portland cement showed similar drying shrinkage values. Section 6.5.

8. The ratio between ultimate creep strain and the elastic strain at transfer is mainly a function of concrete strength at 28 days. Section 6.6.

9. Cylinders stressed to an average original stress of $(F_o/A_t)$ of their corresponding beams, showed in general higher creep values than beams. Section 6.6.

10. The average ratio between creep strain and elastic strain at the points of cable bending is 15% higher than that ratio at the middle section of the beams. Section 6.6.

11. An approximate value of 4% loss due to creep of steel may be predicted of which very little occurred before transfer of the prestress. Section 6.7.

12. The average percentage total losses in the middle third of the beams is 28%. Section 6.8.

13. If the data on creep of concrete is obtained from cylinder creep tests under sustained loads, the effect of change in the elastic shortening of concrete with time due to the decreasing prestressing force must be considered by using Birkenmaier's equation. Section 6.8.

14. The actual ultimate creep strain of the beams, with the effect of the decreasing prestressing force is 16% higher than the apparent creep strain, which is obtained by assuming constant elastic strain. Section 6.8.

## 7.2 Recommendations

To determine the prestress losses which will occur in pretensioned prestressed concrete beams with or without bent tendons, the following recommendations are suggested.

1. Elastic losses can be accurately evaluated by equating the strain of concrete to the change in the strain of steel at the C.G.S. at transfer. (Section 2.2.1 and Equation (2.3)), in the region of full anchorage.
2. Equation (6.1),

$$r = 28.5/\sqrt[3]{f'_c}$$

in which r is the ratio between ultimate creep strain to elastic strain at transfer, can be used to predict the concrete creep strain of the prestressed concrete beams. Such an equation is approximate in its nature and needs more experimental verification. However, in the absence of any creep of concrete data, it may be used.
3. Loss of prestress due to shrinkage of concrete has a wide margin of variation. The designer should select the shrinkage data derived from mixes which closely resemble the mix to be used and cured under conditions similar to the conditions to which the actual member will be exposed.
4. Loss of prestress due to creep of steel depends on the type of steel to be used and the level of prestress. For cold drawn, high carbon, stress relieved strand steel, stressed to an original stress of $0.7\ f'_s$, this loss may be taken as 4% in which only 1% occurs prior to transfer of prestress.
5. Care should be taken to avoid loss of prestress due to slippage of the chucks. This loss can be reduced by

immediately rejacking the cable and shimming the resulting gap tightly by split washers.

6. The effect of the elastic resilience as discussed in Section 6.8 should be considered by using Birkenmaier's Equation (6.2), if the available data on creep of concrete are for concrete cylinders and not for prestressed beams.

7. At points of cable bending, there may be some stress concentration and web reinforcement might help in reducing this effect.

8. For simply supported beams under uniform loads the reduction of the anchorage length to a minimum may result in localized high stresses at the ends.

## BIBLIOGRAPHY


1. Kingham, R. I., Fisher, J. W. and Viest, I. M. Creep and Shrinkage of Concrete in Outdoor Exposure and Relaxation of Prestressing Steel. Special Report No. 66, Highway Research Board, (1961), 103-131.

2. Pauw, Adrian and Breen, E. J. Field Testing of Two Prestressed Concrete Girders. Highway Research Board Bulletin 307, (January 1962), 42-63.

3. Delarue, J. Plastic Flow and Prestressed Concrete (in French). Institut Technique du Batiment et des Travaux Publics (Paris), 13, No. 149, (May 1960), 425-446.

4. Mattock, A. H. "Precast Prestressed Concrete Bridges. Creep and Shrinkage Studies." Journal, Research and Development Laboratories, PCA, 3, No. 2, (May 1961), 32-66.

5. Vik, B. Addendum to the Stress Redistribution Due to Shrinkage and Creep in Case of Multistrand Prestressing (in German). Beton und Stahlbetonbau (Berlin), 55, No. 8, (August 1960), 185-187.

6. Cernica, J. N. and Charignon, M. J. "Plastic Strain in Prestressed Concrete Beams under Sustained Load." Journal, Amer. Concrete Inst., (August 1961), 215-222.

7. Hansen, T. C. "Creep and Stress Relaxation of Concrete." Proceedings, Swedish Cement and Concrete Research Institute at the Royal Institute of Technology, Stockholm, No. 31, (1960), 112.

8. Jevtic, D. Relaxation, Creep, Fatigue Tests and Tests of Behaviour at High Temperatures of Steel Wires for Prestressed Concrete (in French). RILEM (Paris), No. 4, New Series, (October 1959), 66-73.

9. Base, G. D. and Rowe, R. E. "Test on a 120-Ft. Span Prestressed Concrete Beam." Proceedings, ASCE, 86, ST 9, (September 1960) 1-26.

10. Cottingham, W. S., Fluck, P. G. and Washa, G. W. "Creep of Prestressed Concrete Beams." Jour. Amer. Concrete Inst., (February 1961), 929-936.

11. Compomanes, N. V. The Prestress Losses and the Flexural Strength of Lightweight Prestressed Concrete Beams. Ph.D. Thesis, University of Michigan, Ann Arbor, Michigan, 1951.

12. Zuidema, M. L. A Study of Losses of Prestress Force in Pre-Tensioned, Concentrically Prestressed Members. Structural Engineering Research, University of Michigan, Ann Arbor, Michigan, 1959.

13. Walker, H. C. A Study of Shrinkage and Creep of Lightweight Aggregate under Prestressed Conditions. Structural Engineering Research, University of Michigan, Ann Arbor, Michigan, 1959.

14. Troxell and Davis. Concrete. McGraw-Hill Civil Engineering Series. New York: McGraw-Hill Book Company, Inc., 1956.

15. Lin, T. Y. Design of Prestressed Concrete Structures. New York: John Wiley and Sons, Inc., 1956.

16. Corbin, Margaret. Loss of Prestress. Special Bibliography No. 113, Research and Division Library, PCA, prepared for Joint ACI-ASCE Committee 323 on Prestressed Concrete, April 1961.

17. Relaxation of High-Tensile Steel for Prestressed Concrete. (Dutch Committee "Betonstaal"), Cement (Amsterdam) (August 1960), 859-861.

18. Hansen, T. C. Creep and Stress Relaxation of Concrete. Sweden. Svenska Forskningsinstitutet for Cement och Berong... Handlingar Nr. 31, (1960), 112.

19. Simonov, M. Z. and Karapetian, K. S. Shrinkage and Creep of Lightweight Concretes in Prestressed Concrete (in Russian). Beton i Zhelezobeton, (October 1960) 450-454.

20. Delarue, J. Creep and Prestressed Concrete (in French with English summary). R.I.L.E.M. (4), (October 1959), 32-52.

21. Jevtic, D. Relaxation, Creep, Fatigue Tests and Tests of Behavior at High Temperatures of Steel Wires for Prestressed Concrete (in French with English summary). PCA Foreign Lit. Study No. 300.

22. Selva, S. de la. "Study of Reduction of Stress Due to Creep of Anchorag Cables in Prestressed Concrete" (in Spanish). Ingenieria (Mexico) 29 (1) (January 1959), 22-35. Abstract in Eng. Index, 1959.

23. Strycker, R. de. "Creep of Hard Prestressing Steel in the Range of So-called Ordinary Temperatures. Effect of Stress Variations and Effect of Temperature." Revue de Metallurgie, 56, (1), (January 1959), 49-54. (Publ. No. 7 of the Groupement Belge de la Precontrainte, session held on January 15 in Brussels, 1958) Great Britain, C. and C.A. Cj. 87, PCA Foreign Lit. Study No. 253.

24. Stussi, Fritz. "On the Relaxation of Steel Wires" (in German with English and French summaries). International Assn. for Bridge and Structural Engrg., 19, (1959), 273-286.

25. Zielinski, Jerzy. "Losses of Prestressing Force." Concrete and Constructional Engineering, 54, (May 1959), 165-170.

26. Abeles, Paul W. "Losses of Prestressing Force." Concrete and Constructional Engineering, 53, (9), (September, 1958), 331-340; Part I in (8), (August, 1958), 285-296.

27. Abeles, Paul W. "Prestressed Concrete with Pre-Tensioned Steel." Concrete and Constructional Engineering, 53, (2), (February, 1958), 83-89.

28. Base, G. D. Tests to Determine the Loss of Prestress in Small, Uniformly Prestressed Units. Great Britain. Cement and Concrete Assn. Technical Report TRB/293, (March, 1958), 6 pp.

29. Cestelli, Guidi C. "Experimental Research into the Loss of Stress of Prestressing Cables" (in Italian). G. Gen. Civ., 96 (1), 1958, 20-31. Abstract in B.S.A. 31, (7), (July, 1959), 1224.

30. Chowdhry, B. A. "The Loss of Prestress in Prestressing Cables." Indian Concrete Journal, 32, (8), (August, 1958), 275-277, 283.

31. Fritz, Bernhard. "Proposals for a More Accurate Interpretation of Prestress Losses caused by Friction in Straight and Curved Tendons" (in German, with Spanish, English and French summaries). Federation Internationale de la Precontrainte, 3rd Congress, Berlin, 1958. Session II, Paper No. 9, (1958), 1-15.

32. Jurkovich, W. J. "A Method for Prescribing the Prestressing Force." Jour. of Prestressed Conc. Inst., 2, (4), (March, 1958), 81-83.

33. Kluge, Ralph W. "Field Studies of Prestress Loss Due to Shrinkage and Creep." Jour. of the Prestressed Conc. Inst., 3, (1), (June, 1958), 63-69.

34. Knesch, Rudolf. "Measurements of Prestress Losses Due to Friction in Doubly Curve 100 t Tendons of the Augusta-Victoria Bridge in Berlin" (in German, with Spanish, English, and French summaries). Federation Internationale de la Precontrainte, 3rd Congress, Berlin, 1958. Section II, Paper No. 11, (1958), 1-4.

35. Kowalczyk, R. "Losses Due to Friction in Curvilinear Post-Stressing Cables" (in German). Inzynieria i Budownictwo,(1), (1958), 6-10. Abstract in Jour. American Concrete Inst., 31, (4) 355a, (October, 1959); Proceedings, 56.

36. Kowalczyk, R. and Zielinski, J. "Prestress Losses and Errors in Their Evaluation" (in German). Federation Internationale de la Precontrainte, 3rd Congress, Berlin,(1958). Session II, Paper No. 12, (1958), 1-9.

37. Mittelmann, Goswin. "Measurement of Friction in Prestressed Concrete" (in German). Beton und Stahlbetonbau, 53, (1), (January, 1958), 4-7.

38. Papsdorf, Werner and Schwier, Fritz. Creep and Relaxation of Steel Wire, Particularly at Slightly Elevated Temperatures. Great Britian, Cement and Concrete Assn., Cj 84, (1959), 36 pp. Translated from Stahl und Eisen, 78, (14), (July 10, 1958), 937-947.

39. Purandare, N. N. "Loss of Prestress Due to Curvature in the Cable." Indian Concrete Journal, 32, (9), (September, 1958), 304-305.

40. Bannister, J. L. and Butler, L. H. "Automatic Control and Recording of Load during Relaxation Tests on Prestressing Wires." Magazine of Concrete Research, 9, (26), (August, 1957), 105-108.

41. Cestelli, Guidi C. "Experiments on Loss of Tension Caused by Cable Friction." World Conference on Prestressed Concrete, San Francisco, California, (July, 1957). Preprint Proceedings, (1957), 17-1 to 17-9.

42. Dawance, G. and Chagneau, A. "Measured Losses in Prestress" (in German). Supplement to Annales de L'Inst. Tech. Batim... Trav. Publics, 10, (120), (December, 1957), 1344-1353. Abstract in J.A.C.I., 29, (11), (May, 1957), 1015e. Proceedings, 54, France...Serie: Beton Precontraint (26).

43. Murphy, P. E. Behaviour of Prestressed Concrete Beams under Long-Time Loading. M.S. Thesis, University of Illinois, Urbana Illinois, September, 1957.

44. Okada, Kiyoshi. Discussion of: "An Expression for Creep and Its Application to Prestressed Concrete" by C. Z. Erzen. Jour., Amer. Concrete Inst., 28, (12), Proceeding, 53, (June, 1957), 1195-1198.

45. Yedvabnyy, V. I. "Calculation of the Prestress Due to Friction in Design of Prestressed Structures with Draped Bundled Reinforcement" (in Russian). Beton i Zhelezobeton, (9), (1957), 373-375.

46. Bergfelt, Allan. "Losses of Cable Force at Prestressing." Internat'l Assn. for Bridge and Structural Engineering, 5th Congress, Lisbon, Preliminary Report, (1956), 1023-1044.

47. Chatterjee, B. K. and Bobrowski, J. "Friction Losses in Prestressing under Site Conditions." Civil Engineering and Public Works Review, 51, (601), (July, 1956), 765-767.

48. Erzen, C. Z. "An Expression for Creep and Its Application to Prestressed Concrete." Jour. Amer. Concrete Inst., 28, (2), (August, 1956), 205-213, Proceedings, 53.

49. Canta, G. M. "Some Creep Tests on Steels for Prestressed Concrete." Second Congress of the Fed. Internationale de la Precontrainte, Amsterdam, 1955; Session II, Paper No. 4, (1955), 16 pp.

50. Finsterwalder, Ulrich. "Results of Creep and Shrinkage Measurements in Prestressed Concrete Structures" (in German). Beton und Stahlbetonbau (Berlin), 50, (1), (January, 1955), 44-50.

51. Sager, W. The Influence of Creep and Shrinkage on Prestressed Concrete Construction. (Der Einfluss des Kriechens und Schwindens...) Dusseldorf, Werner Verlag, (1955), 176 pp.

52. Stephenson, H. K. and Jones, T. R. "Method Presented for Computing Steel Prestress Losses." Civil Engineering, 25, (8), (Vol. p. 510-612), (August, 1955), 62-64.

53. Stoll, Ulrich W. "Tests Needed to Determine Stress-Loss Factors." Civil Engineering, 25, (12), (Vol. p. 855-856), (December, 1955), 57-58.

54. Anon. "Prestressed Concrete: Influence of Steel Quality on Loss of Stress Caused by Elastic and Plastic Deformation of Concrete." Betonstein-Zeitung, 20, (1), (Wiesbaden), (January, 1954), 10-11.

55. Baldauf, H. "Approximate Calculations for the Loss of Prestress Through Creep and Shrinkage in Prestressed Members Arranged in Several Layers." Bauplanung u. Bautechnik, 8, (5), (1954), 200-202.

56. Baldauf, H. "Approximate Calculation of Prestress Losses Due to Creep and Shrinkage." Beton u Stahlbetonbau, 49, (9), (September, 1954), 214-216.

57. Bergfelt, Allan. "Friction and Jack Losses for Prestressed Concrete" (in Swedish). Betang, 59, (1), (1954) 53-65.

58. Koebel, F. E. "Stress Losses in Prestressed Concrete." Eng. News-Record, 153, (19), (November 4, 1954), 37.

59. Komendant, A. E. "Causes of Prestress Losses." Eng. News-Record, 153, (26), (December 23, 1954), 7-8.

60. McLean, G. A Study of the Effect of Time-Dependent Variables in Prestressed Concrete. Dept. of Civil Engineering (Civil Eng. Studies), University of Illinois, Urbana Illinois.

61. Spare, G. T. "Prestressing Wires - Stress Relaxation and Stress-Corrosion Up to Date." Wire and Wire Products, 29, (12), (December, 1954), 1421-4, 1492-3.

62. Weyel, E. "Effect of the Quality of the Steel on the Loss of Prestress Through the Elastic and Plastic Deformation of the Concrete in Prestressed Reinforced Concrete Building Units." Betonsteinztg, 20, (1), (1954) 10-12.

63. Zinsser, R. "Time Yield of Steel Wires Stressed within Range of Fatigue under Pulsating Tensile Stresses." Stahl u Eisen, 74, (3), (January 28, 1954), 145-51.

64. Cooley, E. H. Estimation of Friction in Prestressed Concrete, Cement and Concrete Association Booklet Cb5, London, (October, 1953), 16 pp.

65. Cooley, E. H. Friction in Post-Tensioned Prestressing Systems. Cement and Concrete Association Research Report (1), London, October, 1953.

66. Gifford, F. W. "Creep Tests on Prestressing Steel." Magazine of Concrete Research, (14), (December, 1953), 71-74.

67. Fritz, "Balancing Friction Loss During Prestressing." Beton u. Stahlbeton bau, 48, (10),(1953), 225.

68. Guyon, Yves. Prestressed Concrete. New York: John Wiley and Sons, Inc., (1953), 543 pp.

69. Neunert. Effect of Creep and Shrinkage on Prestressed Concrete Structural Units. (Der Einfluss des Krichens und Schwinden auf Vorgespannte Stahlbetonbauteile), Dissertation, T. U. Berlin, 1953.

70. Umstatter, H. "On Creep and Relaxation" (in German). Schweiz. Arch., 19, (6), (June, 1953), 184-191.

71. Birkenmaier, M. "Calculation of Stress Loss in Prestressed Concrete" (in German). Schweizerische Bauzeitung, 70, (45), (November, 8, 1952), 635-638.

72. Dawance, G. "Tests Concerning Creep and Shrinkage Losses in Prestressed Concrete." Int. Assoc. Bridge and Structural Engineering Publication, 12, (1952), 109-123.

73. Kiyoshi, Okada. "Shrinkage and Plastic Flow of Prestressed Concrete." Journal of the Japan Society of Civil Engineers, 37, (2), (Tokyo), (January, 1952), 25-28.

74. Schwarz, R. "Contribution to the Calculation of Creep Losses in Prestressed Structural Members in Reinforced Concrete" (in German). Bauingenieur, 27, (3), (March, 1952), 85-90.

75. Dehan, E. and Louis, H. "Measurement of Stresses and Their Variations in Accessible Wires in Prestressed Concrete Structures." Annales des Travaux Publics de Belgique, 103, (2), 1950, 201-56.

76. Freyssinet, Eugene. "Prestressed Concrete: Principles and Applications." Jour., Inst. Civil Engineers, 33, (4), (February, 1950), 331-380.

77. Ross, A. D. "Loss of Prestress in Concrete." Civil Engineering, 45, (527), (London), (May, 1950), 307-309.

78. Schwarz, R. "Prestressing Losses and Deflections of Prestressed Concrete Bridge Due to Creep and Shrinkage" (in German). Bauingenieur, 25, (1), (January, 1950), 1-8.

79. Anderegg, F. O. "What will be Prestress Loss Due to Creep?" J. Am. Conc. Inst., 20, (1), (September, 1948), 86-87; Proceedings, 45.

80. Bjuggren, Ulf. "What will be Prestress Loss Due to Creep?" J. Am. Conc. Inst., 20, (1), (December, 1948), 343; Proceedings, 45.

81. Anon. "The Yield Stress and Creep of Steel." Concrete and Constructional Engineering, 42, (1947), 105-106.

82. Coff, L., Abeles, Paul W. and Schorer, H. Discussion of "Analysis and Design of Elementary Prestressed Concrete Members" by H. Schorer. J. Amer. Conc. Inst., 18, Proceedings, 43, (June, 1947), 88-1 to 88-9.

83. Abeles, P. W. and Authors Staley, H. R. and Peabody, Dean, Jr. "Shrinkage and Plastic Flow of Prestressed Concrete." J. Amer. Conc. Inst., 18, (4), Proceedings, 42, (December, 1946), 244-1 to 244-4.

84. Davis, R. E. "Flow of Concrete under Sustained Compressive Stress." Proceedings of ACI, Vol. XXIV, (1928), 303.

85. Davis, R. E. and Davis, H. E. "Flow of Concrete under Sustained Compressive Stress." Proceedings of the ASTM, Vol. 30, Part II, (1930), 707.

86. Davis, R. E. and Davis, H. E. "Flow of Concrete under the Action of Sustained Loads." Proceedings of the ACI, Vol. XXVII, (1931), 837.

87. Davis, R. E., Davis, H. E. and Hamilton, J. S. "Plastic Flow of Concrete under Sustained Stress." Proceedings of the ASTM, 34, Part II, (1934), 354-386.

88. Davis, R. E., Davis, H. E. and Brown, E. H. "Plastic Flow and Volume Changes of Concrete." Proceedings of the ASTM, 37, Part II, (1937), 317.

89. Davis, R. E. Plastic Flow of Concrete. Annual Report for 1937-38, The Engineering Foundation, 32.

90. Magnel, G. "Creep of Steel and Concrete in Relation to Prestressed Concrete." Journal of ACI, Vol. XIX, No. 6, February, 1948.

91. Peck, L. Roy. Prestressed Concrete. Reprinted from 1961 Concrete Industries Yearbook. A Pit and Quarry Publication, Chicago, Illinois.

92. Pickett, G. "Shrinkage Stresses in Concrete." Journal of ACI, 42, (January, 1946), 165-204.

93. Everling, W. O. Steel Wire for Prestressed Concrete. The First National Prestressed Concrete Short Course, Maritime Base, St. Petersburg, Florida, October, 1955.

94. Patel, R. Prestress Loss in Pretensioned Prestressed Beams. Ph.D. Thesis, University of Michigan, Ann Arbor, Michigan, (not published yet).