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AN INVESTIGATION OF END-ANCHORAGE AND PERFORMANCE OF SR-4 STRAIN GAGES ON A PRESTRESSED CONCRETE BEAM

BY

HOWARD W. NUNEZ, JR.

A

THESIS

submitted to the faculty of the

SCHOOL OF MINES AND METALLURGY OF THE UNIVERSITY OF MISSOURI

in partial fulfillment of the work required for the

Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

Rolla, Missouri

1952

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Approved by- E. W. Carlton
Professor of Structural Engineering

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I am deeply indebted to Prof. E. W. Carlton for his continuous advice, encouragement, and guidance in this work. To Prof. A. V. Kilpatrick I would like to express my thanks for his excellent machine work in making the end-anchorages employed.

My thanks to Mr. K. P. Billner, President of Vacuum Concrete, Inc. of Philadelphia, for the model of the end-anchorage as well as for very helpful advice; to John A. Roebling's Sons for their generous gift of wire used in this project; to Prof. J. K. Roberts for his photographic work; and to Mr. J. H. Senne for his help and advice regarding SR-4 strain gages. I am grateful to K. Delap, J. Latham, G. Latham, and R. Hoffman, all of whom gave willing and able assistance in the pouring, prestressing and testing of the beam.

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INTRODUCTION

In recent years there has been brought to the attention of engineers and architects a new medium of construction--- prestressed concrete. Reinforced concrete has long been recognized as a satisfactory material for use in the construction of buildings, bridges, highways, tanks and other structures. However, most concrete designers will concede that this material has many drawbacks. For instance, in a simple beam of rectangular cross-section, roughly only one half of the concrete is relied on to resist compressive forces. The tensile strength of the concrete is considered to be zero, and steel bars or rods must be inserted to carry the tensile stresses. This assumption necessarily increases the dead loads which must be considered in design.

Prestressed concrete refers to members in which the concrete is subjected to stress before it receives any live loads. This stress is usually in the form of a compressive force which acts on the whole cross-sectional area of the member. In this way the entire area of the member can be used to resist the applied loads.

As in any new field of endeavor, many problems are encountered which must be investigated. In this field of prestressed concrete probably the largest stumbling block is the method of anchoring the high tensile strength wires which are used to induce the compressive stresses in the concrete.

It is the purpose of this paper to investigate one method of end-anchorage and also to investigate the performance of SR-4 strain gages applied to both the prestressing wires and the concrete.

HISTORICAL SKETCH

The basic idea of prestressed concrete is the elimination of tensile stresses by superimposing compressive stresses. One method of accomplishing this is by stretching the reinforcement. This much was known more than sixty years ago. In 1886, the first patent on prestressed concrete was issued to P. H. Jackson of San Francisco. This patent was for a process of tightening steel tie-rods in cast-stone and concrete arches by means of turn-buckles and nuts and bolts. Jackson's scheme was tried under various conditions, but with little or no success. The neutralizing compressive stresses could be induced, but they could not be maintained indefinitely by means of the ordinary bar reinforcement then in use.

The first person to achieve any real degree of success in this new method of construction was R. E. Dill of Alexandria, Nebraska, (1) who applied for a patent in 1925. Dill

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(1) U. S. Progress in Prestressed Concrete, Architectural Record, Vol. 110, No. 2, pp. 148-156.

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produced mainly posts and slabs by the post-tensioning method. In order to overcome the major obstacle encountered by Jackson, Dill employed high tensile strength steel wire which was coated with a plastic substance to prevent bond with the concrete, the wires being tensioned after the concrete had set, thus avoiding any loss of the prestress force due to shrinkage of the concrete.

Since that time, European engineers such as J. Mandl of Austria, Eugene Freyssinet of France, Gustave Magnel of Belgium, Edwald Hoyer of Germany, and Colonnetti of Italy have led the way in making prestressed concrete a revolutionary and highly successful new medium of construction. Their primary work has been done in the field of bridges, of which the 410-foot, two span, continuous box-girder Sclayn Bridge[2]

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(2) Schofield, E. R., Prestressed Concrete Used for Boldly Designed Structures in Europe, Civil Engineering, Sept. 1949, Vol. 19. p. 596.

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is a most beautiful and impressive example. Another striking bridge project is that of the five almost identical prestressed concrete bridges over the Marne River. These bridges, designed and built by Freyssinet, were assembled from precast units and prestressed to form flat-arches spanning 243 feet and having a crown depth of only 37 inches. Due to the fact that all the spans were the same, the bridges were constructed on a mass production basis, as many as sixty identical pieces being made from each mold.

Another example of European practice is a one-story textile mill in Ghent, Belgium, which covers 8 1/2 acres. Here again, mass production methods were employed with the necessary one hundred main girders, 72 feet in length, and the six hundred secondary beams, 45 feet in length, being cast in steel molds. The whole structure was erected in a period of one year with no more than seventy workmen engaged on the job at any one time.

A runway at Orly Airport near Paris, France, constructed by Freyssinet in 1948, points out the possibilities of prestressed concrete for use in highway and airport slab construction. This strip is 1440 feet long, 200 feet wide, and only 6 inches thick, yet it is claimed to be capable of supporting loads of a conventional runway four times as thick. The pavement consists of 40-inch square pre-cast concrete blocks. The blocks were installed using 45-degree diagonal joints, making only transverse prestressing necessary. One inch vertical rollers help to reduce friction in these joints. Freyssinet claimed that this runway could support aircraft three times the size of any then in existence.

Here in America, virtually the only application of prestressing up until the last few years has been in the production of concrete tanks and cylindrical pipe. This process of circular prestressing is somewhat different from linear prestressing and has been perfected to a high degree almost exclusively by American engineers. The reluctance of the American engineer to show other than a passing interest in the European scheme of linear prestressing may be attributed almost entirely to the methods involved.

It has been argued that a great difference exists between the two continents in regard to the relative importance of labor and material. European labor is cheap, while materials are in short supply. American labor is decidedly costly, and materials have been, at least up until the present time, relatively plentiful. For this reason, the European designer is primarily interested in refinements of design to conserve material, while the American designer is more concerned with mechanical methods of construction which can keep labor requirements to a minimum.

In the last few years numerous organizations have been formed in this country, devoted to the study and manufacture of prestressed concrete, and, as a result, remarkable advances have been made toward overcoming some of the prevailing inertia. Constantly being presented are new methods and new ideas which are in many ways simpler and at the same time better than current European practices.

GENERAL DISCUSSION

The term "prestressing" does not exactly describe what is done to the concrete because it is actually a pre-compressive force which is exerted on the member. By various means, a large compressive force is applied to the concrete prior to its being put to use, and this force is maintained to some degree throughout the life of the structure. This pre-compressive force is made so large that, under working loads, the tensile forces developed only reduce the total compressive forces and never actually place the concrete in tension.

The essential difference between the various methods of pre-compressing is in the way in which the forces are applied to the member. This may be done by pre-tensioning or by post-tensioning, the prefix indicating whether the steel is tensioned before or after the concrete has hardened.

In pre-tensioning, the compressive force is applied to the concrete member through the action of bond. Very small, high strength steel wires are strung between two abutments, one fixed and the other movable. The wires traverse the length of the forms and are held in their proper places by holes in the abutments or by spacer blocks. When the movable abutment is forced away from the other one, a stress in the neighborhood of 200,000 psi. can be induced in the wires. While the wires are held in this state, concrete is placed into forms which have been set up around them. When the concrete has hardened sufficiently, the wires are released from the abutments. When the wires are released, they have a tendency to snap back to their original length, much like a rubber band, but the bond of the concrete along their full length prevents them from doing so. Thus, the pre-compressive force is transmitted to the concrete member as a combination of mechanical bond resistance and friction with radial compression. See Fig. 2. This method is particularly suitable for operations involving the manufacture of a large number of similar units, since the wires may be tensioned in lengths of from 200 feet to 1000 feet. A large number of units may then be cast at one time and the wires cut between each unit to separate them. This will eliminate the necessity of end anchorages and jacking operations for each member. However, this method does present a problem in the form of a loss of prestress due to the shrinkage of the concrete after the wires have been released.

In post-tensioning, the compressive force is applied to the concrete through bearing plates at the ends of the member. See Fig. 3. Prior to the placing of the concrete, hollow ducts are placed in the forms. After the concrete has hardened, these are withdrawn, and the wires are threaded through the passages and fastened to some form of jack. Reacting against the ends of the concrete member, the jack stresses the wires to around 120,000 psi. Under this stressed condition they are wedged against an anchorage plate, and when the jack is released, the stress in the wires is transferred to the bearing plate and thence to the concrete.

The purpose of the passages is to keep the wires from being bonded to the concrete. Another method of accomplishing this and, at the same time, eliminating the expense and time necessitated by the ducts is to coat the wires with asphalt or grease or to wrap them in waterproof paper. Where a number of wires are in the form of a cable, a thin metal sheath may be employed to surround the entire cable and prevent bond.

While the post-tensioning method is most adaptable to individual job conditions, it also has great promise in the field of mass produced units. Of special interest are beams and girders built of pre-cast building block units. These units are very inexpensive, yet very strong, and may be made in a wide variety of shapes on standard commercial concrete block machines. The wires may then be strung in passages cast inside the units or in channels on the outside.

A number of the advantages of prestressed concrete are stated herewith:

1. Prestressing can make concrete crackless, which is conducive to greater durability under severe exposure conditions.
2. Prestressing minimizes deflections and reduces the depth of beams and girders and the thickness of slabs, thus giving greater headroom.
3. As compared with conventional reinforced concrete, prestressed concrete permits substantial savings in materials: up to as much as 75 per cent in steel

and approximately 25 per cent in concrete.

4. Prestressing results in maximum rigidity under working loads and maximum flexibility under excessive overloads.
5. Design calculations are considerably quicker and more accurate.
6. The architect can design concrete structures with much cleaner, slimmer lines.
7. Even under extremely heavy loads, the member will return to its original shape as long as the elastic limit of the steel has not been exceeded.

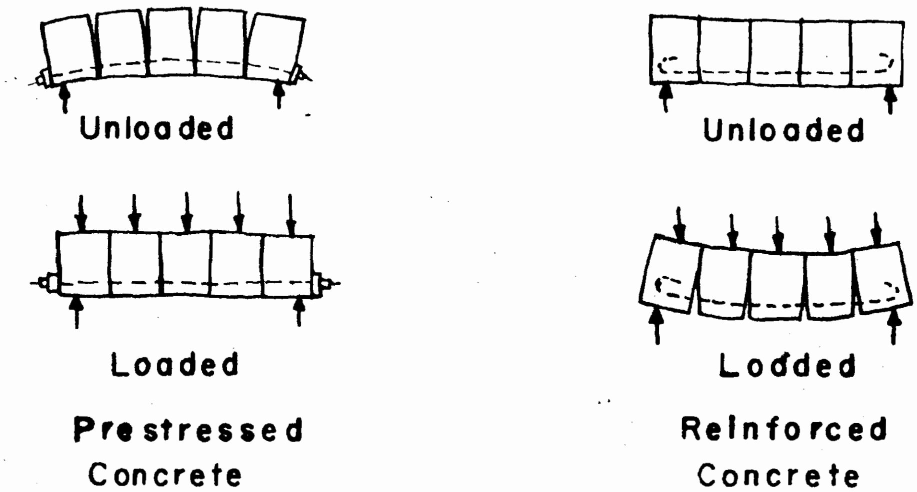

Fig. I Showing Difference Between Prestressed and Reinforced Concrete Under Load.

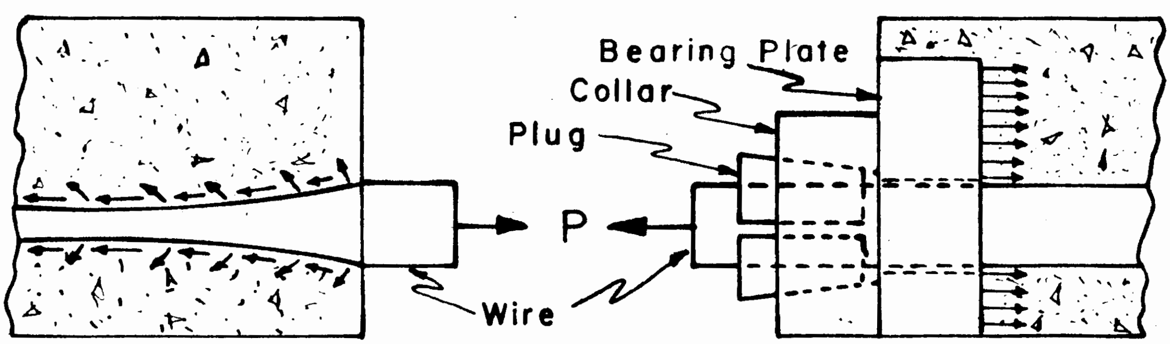

Fig. 2 Compressive Forces Applied By Pre-tensioning.

Fig. 3 Compressive Forces Applied By Post-tensioning.

A DISCUSSION OF METHODS

FREYSSINET METHOD: The Freyssinet method uses a number of cables composed of from 8 to 18 wires each, the wires being separated and spaced by a center core. Bond is prevented between the cable and the concrete either by covering the cable with a sheath before the concrete is placed or by pulling the cable through holes formed in the beam by hollow ducts. The wires of the cable are locked in place at each end of the beam by a wire-gripping device which consists of a cast concrete conical plug and a cast concrete cylinder with a tapered hole through its length. Both units are spirally bound by wire for added strength. The cylinder is cast into the end of the beam at the time of pouring.

After the concrete has hardened sufficiently, all the wires are gripped around the circumference of a unique double-acting hydraulic jack and stressed at about 125,000 psi. Then a center ram on the jack is activated, and the conical plug is driven securely into the tapered hole, securing all the wires under load. Mortar is then pumped into the cable hole to protect the wires against corrosion as well as to provide extra strength through bond.

The features of this method are that all wires in any one cable are tensioned at the same time, thus reducing costs; that the anchorages do not project at the beam ends; and that the stress on the beam is uniform. However, there is a disadvantage in stressing all the wires at once because it is then impossibl to determine whether or not they all have equal stress. If there is slippage of one wire in the wedge, the entire job must be redone.

MAGNEL METHOD: In the Magnel method, which is also a post-tensioning method, the wires are arranged in a series of vertical and horizontal rows to form a cable. There may be as many as 8 wires per layer and 64 wires in all. The gripping device at the end consists of steel plates called sandwich plates. These plates are about 1 1/2-in. x 3 1/2-in. x 6-in., and each has four wedge-shaped slots. Two wires are placed in each slot, and, when they are stressed the required amount, a tapered wedge is driven between them by hand. This process is repeated for all the wires in the cable, the plates being stacked one on top of another. These sandwich plates are later covered over with a cast-in-place extension to the beam. Grout is pumped around the wires for protection and to provide bond.

Since only two wires are tensioned at a time in the Magnel method, there is obviously better control over their stresses; the required jacks may be simpler and smaller; and, because a large number of wires are used in each cable, the total compressive force is very large and quite suitable for long span beams. Obvious drawbacks to this method are found in the expense of machining so large a number of blocks as are required and in the extra time involved in stressing only two wires at a time.

The Magnel method was used on Philadelphia's Walnut Lane Bridge, the first large prestressed concrete bridge in America.

SHORER SYSTEM: The Shorer system, based on the bonding action of the wires, has as its central feature a high strength steel tube which is used as the bearing area for jacking. The prestressing wires are wound about the tube in a helical fashion with a very wide pitch, half in one direction and half in the other. They are kept away from the tube by spacer discs along its length. A patented device at the ends of the tube secures the wires and enables the jack to bear against the tube, thus compressing it and at the same time stretching the wires. When the wires are stretched, they are fastened in place, and the jack is removed. The entire unit, wires and tube, can then be set in the form in the same manner as is standard reinforcement, except that the end of the tube must be accessible when the form is removed. In this manner, the steel tube takes the entire compressive load. After the concrete has hardened, the locking device on the end is removed, and the tube, which has had a bond-breaker applied to it, is also removed. The wires, being fully bonded to the concrete, transfer the entire compressive stress to the concrete. The hole is then grouted.

BILLNER METHOD: The unique feature of the Billner method is that, although it is a post-tensioning process, no end-anchorages are needed.[3] Billner employs high-strength

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(3) Billner, K. P., New Prestressing Method Utilizes Vacuum Process, Journal of the American Concrete Institute, Vol. 22, No. 2, 1950.

wires of comparatively large diameter which are anchored in the concrete by means of large loops formed on the ends. All of the wires except these loops are coated with some material to prevent bond with the concrete. Before the concrete is cast, a thin sheet metal partition is placed at the mid-span so that after the concrete has been cast and has hardened, the beam will be in two halves tied together only by the wires. These two sections are then jacked apart a predetermined distance, the separation plate removed, and the space filled with a high-strength grout. When the grout has hardened, the jacks are released, and the beam is prestressed by the efforts of the wires to regain their original length.

This process has distinct advantages in that all of the wires are stressed as one, and wires 3/8-in. in diameter may be used, thus reducing the number of wires required. Also, the expense of manufacturing and installing end-anchorages is eliminated.

ELECTRIC METHOD: This unique method was developed by K. P. Billner and R. W. Carlson.(4) Steel bars coated with

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(4) Billner, K. P. and Carlson, R. W., Electric Prestressing of Reinforcing Steel, Journal of the American Concrete Institute, Vol. 14, No. 6, 1943.

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a thermoplastic material and having threaded ends are placed in the concrete in the same manner as in ordinary reinforcing, except that the ends are left protruding. After the concrete has hardened, a low voltage, high amperage current is passed through the bars, causing them to heat up. The thermoplastic material melts upon heating and destroys any bond, thus allowing the bars to elongate. Nuts are then tightened against the ends of the beam, and the current is removed. Compressive stresses are set up in the concrete by the contraction of the steel upon cooling. This method proved very successful in making slabs only two inches thick to be used as walls for houses.

<u>ROEBLING METHOD:</u> Although the method developed by John A. Roebling's Sons Company is not entirely different from the Freyssinet or Magnel methods, it does have a number of distinguishing features which make it a very simple and highly effective method of prestressing concrete.

The first such feature of this method is the use of pre-stretched galvanized bridge strand which has been thoroughly developed by Roebling. By using this cable which is made up of a number of high strength wires prefabricated into cable form, a large area of steel may be secured with one end-anchorage, thus greatly reducing the time and labor involved in that step, in some instances enabling only one cable to do what otherwise might require six or eight separate wires.

To accompany these cables Roebling uses a special swaged fitting similar to that used on aircraft control cables. This fitting is a slim cylindrical terminal from

1 1/2-in. to 2 3/4-in. in diameter in one end of which the cable is fastened by pouring molten zinc around the wires, much like standard zinc sockets. A threaded stud is screwed into the other end, and a special nut on this thread bears against the concrete through a steel bearing plate to hold the prestressing force.

This is, of course,a post-tensioning process, and by the very nature of the anchoraging system any loss of the prestressing force over a period of time may be corrected simply by restretching the cable and taking up on the nut.

A standard poured zinc socket about five inches in diameter is used for heavier cables, and the development of a hydraulic jack with a hole through the center of the ram has greatly simplified the stretching of these cables.

A DISCUSSION OF MATERIALS

REINFORCEMENT: Of first concern in the matter of prestressing is the nature of the material that is to be used for reinforcement. Since the primary object of prestressing is the induction of preliminary internal stresses in a concrete member, it becomes essential to use a steel of considerably high strength so that losses of stress in the steel due to elastic and plastic deformations and shrinkage will be only a small part of the initial stress, thereby leaving ample stress to be transmitted to the concrete.

There are numerous varieties of high strength steel on the market today, ranging from the fine piano wire which is in common usage in Europe to steel bars 1 1/8-in. in diameter which are used by one concern in England. In between these two are wires of varying sizes which are either stretched singly or coiled into cables and stretched several at a time. The Roebling bridge strand cable is a notable example of the latter. It should be noted that wires in the sizes of 0.010-in. to 0.100-in. are used almost exclusively in bonded prestressed concrete because it is difficult to provide end-anchorages for wire of this small size when it is used in post-tensioned prestressed concrete.

Typical of the wire being used in post-tensioned prestressed concrete is the 0.276-in. diameter cold drawn wire manufactured by the John A. Roebling's Sons Co. This wire, which would have a very small cross-section for use in ordinary reinforced concrete design, is used both in single strand form and in coiled cables containing from 3 to 5 wires each. When this wire is used singly, the matter of end-anchorage becomes rather difficult, but when made into cables, the use of a Roebling swaged terminal affords a very convenient method of anchoring the wires. One drawback of this wire when used singly is the very large coil curvature that is imparted to it. This wire is usually delivered in six-foot diameter coils which have a free coil curvature of over twelve feet. This makes the wire somewhat unmanageable for placing in forms.

The effect of creep on the wire used in prestressing is also of importance. Studies have shown(5) that there is a

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(5) Godfrey, H. J., Steel Wire for Prestressed Concrete, Proceedings of the First United States Conference on Prestressed Concrete, 1951, p. 151.

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definite relationship between the creep of cold drawn wire and the elastic properties of the material. These studies indicate, however, that the amount of creep is negligible if the stress is at, or below, the proportional limit of the wire.

Since the elastic properties of prestressing wire are of utmost importance, it is necessary to specify and control the stress-strain characteristic of the wire. This characteristic may be determined by means of the modulus method, in which a minimum modulus of elasticity is required at a specified stress, or by means of a permanent strain or offset method. Roebling Co. uses what is known as the elongation method which requires a minimum stress at 0.7 per cent elongation. This method eliminates the drawing of stress-strain curves, and material meeting this requirement will have the necessary elastic properties.

In most specifications for prestressing wire some account will be taken of elastic deformation of the wire and also of creep and shrinkage. This is generally done by specifying an initial and a final prestress. These forces usually differ by from 15,000 psi. to 35,000 psi., the initial prestress force generally being taken as about 50 to 60 per cent of the ultimate strength of the wire. It has been found [6] that under average conditions about 25 per cent of the above losses occurs within 20 days, 50 per cent within 60 days, and 75 per cent within 130 days after prestressing. After that the rate of loss approaches a horizontal straight line condition. For this reason it seems advisable to employ some means of end-anchorage which would allow a restressing of the wires to compensate for a part of this loss.

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(6) Dobell, C., Prestressed Concrete Tanks, Proceedings of the First United States Conference on Prestressed Concrete, 1951, p. 10.

<u>CONCRETE:</u> The strength of concrete specified for prestressed construction is generally higher than that for conventional reinforced concrete. Concrete specified for prestressed construction will usually have a minimum 28-day cylinder strength of 5,000 psi., which is probably the top value for conventional concrete.

There are many factors which combine to affect the strength of concrete. Some of these are aggregate size and quality, fineness of cement, composition of cement, size of the section, water-cement ratios, admixtures, and curing. While all of these are important, there are some over which the concrete maker has little control; however, others such as the water-cement ratio and the method and time of curing, both of which may be greatly varied, can be determined and controlled by the concrete maker to produce a certain strength concrete. The exact relationship of these factors has been set forth in other literature. (7)

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(7) Design and Control of Concrete Mixes, Portland Cement Association.

In general terms, it may be said that the lower the water-cement ratio, the stronger the concrete. For Type I Portland cement the 28-day ultimate compressive strength for a 6 gal. per sack water-cement ratio is about 5500 psi. whereas for a 4 gal. per sack water—cement ratio the strength is about 7500 psi.[8] Another way of looking at this same

(8) Ibid., p. 7.

thing is that with the lower water—cement ratio a given strength will be reached in a shorter period of time, which is very important in prestressed construction.

An accompanying feature of this low water-cement ratio is a marked decrease in the slump of the concrete. Most conventional concrete will have a slump of from three to five inches, whereas the average concrete used for prestressed construction will have a slump of from one to two inches. In nearly all instances this low slump will make the placing in forms, particularly of sections with thin webs and closely spaced reinforcement, extremely difficult. On the Walnut Lane Bridge in Philadelphia,[9] specifications called for

(9) Baxter, S. S., Construction of the Walnut Lane Bridge, Proceedings of the First United States Conference on Prestressed Concrete, 1951, p. 47.

5400 psi. concrete with a maximum two inch slump. This slump was maintained until requests from the field resulted in increasing the slump to three and one-half inches in order that the concrete could be more easily placed. The strength of the concrete, which had been running over 7000 psi., was materially reduced but still maintained above 5400 psi.

Vibration may be used very advantageously in placing concrete for prestressed works. It is a most effective means of reducing the water content because of the stiffer consistencies and harsher mixtures which may be placed by this method. In order to be fully effective, however, vibration should be of a high frequency, in the neighborhood of 6000 v.p.m. Either external or internal vibrators or a combination of the two may be used.

Still another important factor in the quality of concrete is the effect of bleeding, which may be defined as the forcing upward of mixing water by the gravitational settlement of the heavier constituents of fresh concrete. Tests have shown[10] that as a result of bleeding there is a

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(10) Kennedy, H. L., High Strength Concrete, Proceedings of the First United States Conference on Prestressed Concrete, 1951, p. 128.

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marked decrease in the compressive strength of concrete from the bottom to the top of a relatively deep section. See Fig. 4. Because of this reduction of strength, the ability of concrete to resist the applied stress in a conventionally reinforced concrete beam is frequently almost inversely proportional to the stresses existing in accordance with the theory of planar distribution of stresses, as shown in Fig. 5. In other words, due to bleeding the concrete is weakest where it should be strongest. It might be mentioned that advantage may be taken of this fact by casting beams or slabs upside down.

The age and, correspondingly, the strength of the concrete at which the prestressing force is applied is of considerable importance. Some specifications allow a somewhat higher compressive strength at the transfer of prestress than at the application of design loads. Average figures for these are from 2/3 f'_c to 0.8 f'_c at transfer of prestress and from 0.3 f'_c to 0.4 f'_c at application of load, f'_c being the 28-day cylinder strength.

One reason for allowing a higher compressive stress when prestress is first transferred to the concrete is that the initial concrete stresses induced at that time decrease considerably as the steel prestress decreases, owing to the effect of creep and shrinkage.

In conventional reinforced concrete it is customary to disregard all tensile stresses in the concrete, but such an attitude is not justified in the design of prestressed concrete. One major purpose of prestressing concrete is to prevent cracking by making the concrete perform homogeneously. The steel is designed to produce this effect. Actually reduced. The once-cracked section performs as if it had never been cracked, and the prestress is as effective as ever.

When prestress is first transferred to the concrete, compression is induced in the bottom fibers. Some designs will require the stress in the top fibers to be zero under this initial loading, but there is no harm in allowing some tensile stresses as long as they are kept below the value that causes cracking, which is generally about 0.15 f'_c. Allowing tensile stresses in the top fibers at transfer of prestress will generally result in better proportioned sections. As the allowable tensile stress is increased at the top, the centroid of the prestressing wire can be moved closer to the bottom, thus becoming more effective in inducing compressive stresses in the bottom fibers.

The term "fully prestressed" refers to prestressing in which no tension is allowed in the bottom fibers under design load. Structures in which some tension is allowed may be called "partially prestressed". Where safety against cracking is of greatest importance, a high degree of prestress should be provided. On the other hand, in structures in which cracks at or near working loads are not objectionable, the desired characteristics can be obtained by applying a smaller prestress force. Where applicable, partial prestressing may result in considerable savings in prestressing labor and fittings as well as in concrete. The subject of partial prestressing has been discussed in detail by

P. W. Abeles.[11]

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(11) Abeles, P. W., The Principles of Prestressed Concrete, N.Y., Ungar Publishing Co., 1949.

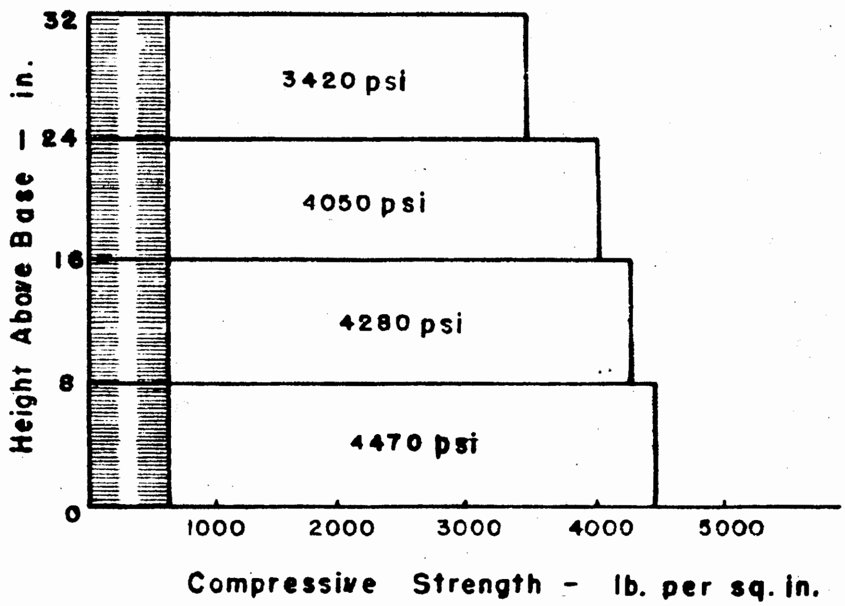
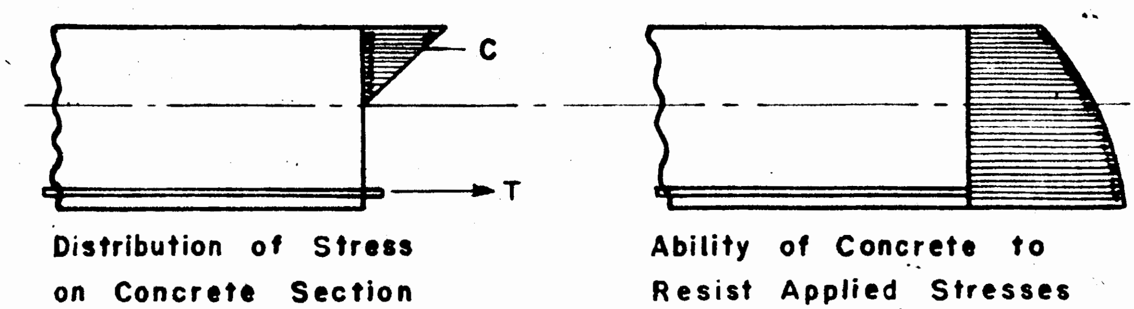


Fig. 4 — Effect of bleeding on uniformity of concrete strength.



MATERIALS USED IN TEST BEAM

FORM: Since the beam itself was to be 6-in. x 8-in. x 10-ft. 6-in., it was necessary to make the sides and the bottom of the form from two thicknesses of 3/8-in. plywood securely nailed together since 3/4-in. plywood was not available in the required lengths. Exterior grade plywood was used throughout. The bottom and ends were grooved to receive the sides, and bracing was used at the quarter points to prevent bulging of the sides. Both ends of the form were cut out to fit around the five anchoring collars which held the wires in place. A piece of 1/2-in. half-round was nailed to the bottom at one end to form a keyway in the bottom of the beam which prevented lateral movement of the beam while being tested.

Steel bearing plates 4 1/2-in. x 6-in. x 5/8-in. were fastened to each end of the form by 5/16-in. bolts through the form. Each plate was drilled to allow passage of the five wires. Since these plates were to remain in the finished beam, two 3/8-in. x 4-in. carriage bolts were screwed into threaded holes in the upper corners of each plate and allowed to extend into the concrete to provide anchorage. The entire inside of the form was painted with several coats of motor oil to prevent bond of the concrete. See Figs. 6-9.

CONCRETE: To secure the full advantages of prestressed concrete requires considerably higher concrete strength than is customary in ordinary reinforced concrete. To secure higher strength concrete the water-cement ratio was reduced to 4 gal per sack. In order to reduce the curing time to a minimum, high-early strength cement was used. The mix was as follows, with all materials conforming to A.S.T.M. specifications:

	1 Cu. Ft. Yield
Water - (4 gal per sack of cement)	11.9 lbs.
Cement - Type III High-Early	33.7 lbs.
Sand - All passing #4 sieve, F.M.-2.30	46.3 lbs.
Gravel - Crushed limestone, 3/4" max. size	59.0 lbs.

This mix resulted in a very stiff, dry concrete which had a slump of from 1 1/2-in. to 2-in. and was rather difficult to place. It was necessary to force the concrete under and around the wires with the fingers as the spacing between the wires was too small to allow adequate rodding. Above the level of the wires considerable rodding was necessary in order to obtain good filling of the form.

The concrete was mixed in a small portable electric mixer of two and one-half cubic feet capacity. Since the total volume of the concrete required, including eight standard 6-in. x 12-in. cylinders, was five and one-half cubic feet, it was necessary to make the concrete in three batches: two of two cubic feet each and one of one and one-half cubic feet. Care was taken to keep the batches as nearly the same as possible, and test cylinders were taken from each batch.

CURING: The beam was moist cured by placing about 1-in. of sand over the top and sprinkling with water twice a day. The sand was covered with used cement sacks to help retain the moisture. The test cylinders were stored in a moist closet. Three were tested at the time of application of the prestress, and four at the time of testing of the beam.

REINFORCEMENT: The reinforcing wire used in this project was furnished by the John A. Roebling's Sons Company of Trenton, New Jersey. Data furnished on this special acid steel prestressed concrete wire is as follows:

Dia.	0.276-in.
Ultimate Strength	240,000 psi.
Min. Value at 0.7% Elongation	180,000 psi.
Min. Ult. Elongation in 10" Gage Length	4%
Design Stress	120,000 psi.
Tensioning Stress	135,000 psi.

An average stress strain diagram is shown in Fig. 10. Five wires were used in the beam, three straight and two parabolic. A light wire frame was used to keep the two parabolic wires on the same level as the straight wires at the mid-section. Each wire was left extending two feet beyond one end of the beam to provide for tensioning at a later date. Bond between the wires and the concrete was prevented by sheathing each wire in 7/16-in. electrical loom. (Figs. 15 and 16) This method proved very satisfactory and much more convenient than painting each wire with asphalt or wrapping with waterproof paper. The inside diameter of the loom was just slightly larger than the diameter of the wire. The loom was cut and the wire exposed for a length of 2 3/4-in. at each strain gage location. The arrangement and numbering of the wires is indicated in

Figs. 11 and 12. Due to the large radius coil curvature of the wires, it was necessary to apply a slight initial tension to each wire to straighten it and to hold it in the proper place during placing of the concrete.

END-ANCHORAGE: The model of the end-anchorage used on this beam was furnished by Mr. K. P. Billner, President of Vacuum Concrete, Inc. of Philadelphia. The units used were machined by Prof. A. V. Kilpatrick of the Mechanical Engineering Department of the Missouri School of Mines and Metallurgy, who suggested and incorporated several changes to increase the gripping power of the plugs and to adapt them to the size wire used.

Each unit consisted of a steel collar 1 3/4-in. square and 3/4-in. thick with a tapered hole in the center and a steel plug 1-in. long, turned to the same taper as the hole and drilled to slip over the wire. Both collar and plug were machined from cold-rolled steel. The four collars for the parabolic wires were milled to a slant on the back side so that the axis of the hole would be parallel to the axis of the wire as it emerged from the bearing plate. The plug was tapped with a 5/16-in. x 24-NF thread cutter to provide a toothed gripping surface adjacent to the wire. Each plug was also slit into quarter segments in the manner of a collet in order to help increase the gripping power. All plugs were case hardened and carburized by Mr. Gene Langston of the Metallurgy Department of the Missouri School of Mines and Metallurgy. This unit is dimensioned in Fig. 13 and pictured in Fig. 14.

SR-4 STRAIN GAGES: A total of twenty-two type A-7 SR-4 strain gages was used in the beam: four on the center wire, three on each of the other wires, and three on both the top and bottom of the beam at the centerline. Fig. 11 shows the arrangement and numbering of these gages. Two of the test cylinders were also instrumented by placing two gages on the sides of each cylinder, spacing the gages 180^{o} apart.

Theory: Essentially an SR-4 strain gage consists of a length of very fine special alloy wire in the neighborhood of 0.001-in. in diameter arranged to form a grid pattern and bonded to a paper or bakelite base. In use, the gage is cemented to the member to be tested and is thus strained uniformly with the test member in either tension or compression. As the member is strained, the cross-sectional area of the gage wires change, thus causing a change in the resistance of the wires. This change in resistance may be measured with a sensitive Wheatstone bridge or by special instruments such as the Baldwin-Southwark Model K Strain Indicator which is calibrated directly in micro inches per inch and thus gives the correct value of the strain when the bridge is balanced. Strains due to temperature changes may be eliminated by introducing as a fixed resistance in one arm of the bridge a similar type gage known as a compensating gage which is cemented to the same type material and kept under the same conditions as the active gage.

Application: The method of applying these gages was substantially the same as outlined by J. H. Senne.[12]

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(12) Senne, J. H., Investigation of Stress and Crack Distribution in Concrete Slabs Containing Welded Wire Reinforcement, Missouri School of Mines and Metallurgy, 1951. pp. 13-15.

After first sanding the wires to remove all rust and scale and wiping with a swab soaked in acetone until no more dirt came off, the SR-4 strain gages were cemented to the cleaned areas, using a heavy coat of Duco cement. Each gage was clamped for three hours, using a clamp (Fig. 15) devised by Mr. Senne, after which time the clamp was removed and the cement allowed to air dry for a period of three days. All gages were then coated with molten cerese wax as the first step in waterproofing.

Waterproofing of the gages when placed in concrete is extremely important since any intrusion of water to the gages themselves will destroy their usefulness by giving false indications of resistance on the strain indicator. Not only is the exclusion of moisture important; it is equally important that all moisture and cement solvent be eliminated from the gages before waterproofing. For this reason, it is essential that the gages be allowed to dry thoroughly and to age before waterproofing is begun. Heat may be used to hasten this process.

After the wax was applied, short neoprene-covered external lead wires were soldered to the gage lead wires. These lead wires were later brought out of the concrete and soldered to longer leads which were connected to the bridge balancer. Next, the entire gage area was given two coats of liquid rubber cement, care being taken that all exposed wire was covered. Since there was to be a substantial elongation of each wire under the prestressing load, a 3/4-in. sponge rubber pad 1/4-in. thick was wrapped around the wire just in front of each gage, serving as a cushion to prevent stripping-off of the gage. Also, the lead wires were doubled back on themselves so that they would have some slack to allow for movement. Following this, the gages were given three fairly thick coatings of 3M Special Weather-Strip Adhesive. This rubber adhesive covered the entire gage area and extended over the electrical loom for a short distance on both sides. Finally, the entire area was covered with electrician's rubber tape. One gage was cemented to a short length of wire and waterproofed by the above process. This gage and wire were placed in a 6-in. x 12-in. cylinder at the time of casting the concrete and served as a temperature compensating gage during the testing.

The procedure for applying the gages to the concrete was a little different. After the concrete was cured and removed from the form, the surfaces where the gages were to be applied were ground smooth with a grinding wheel and then brushed with a wire brush. Next, pieces of 0.080-in. thick celluloid were cemented to these prepared surfaces with a heavy coat of Duco cement. This was allowed to dry for two days, after which the gages were cemented to the celluloid and allowed to dry for two more days. The gages were then given a coat of wax, and the entire areas were painted with lacquer to prevent moisture from entering. Finally, the lead wires were soldered on. This same procedure was followed in applying gages to the test cylinders. (See Fig. 17)

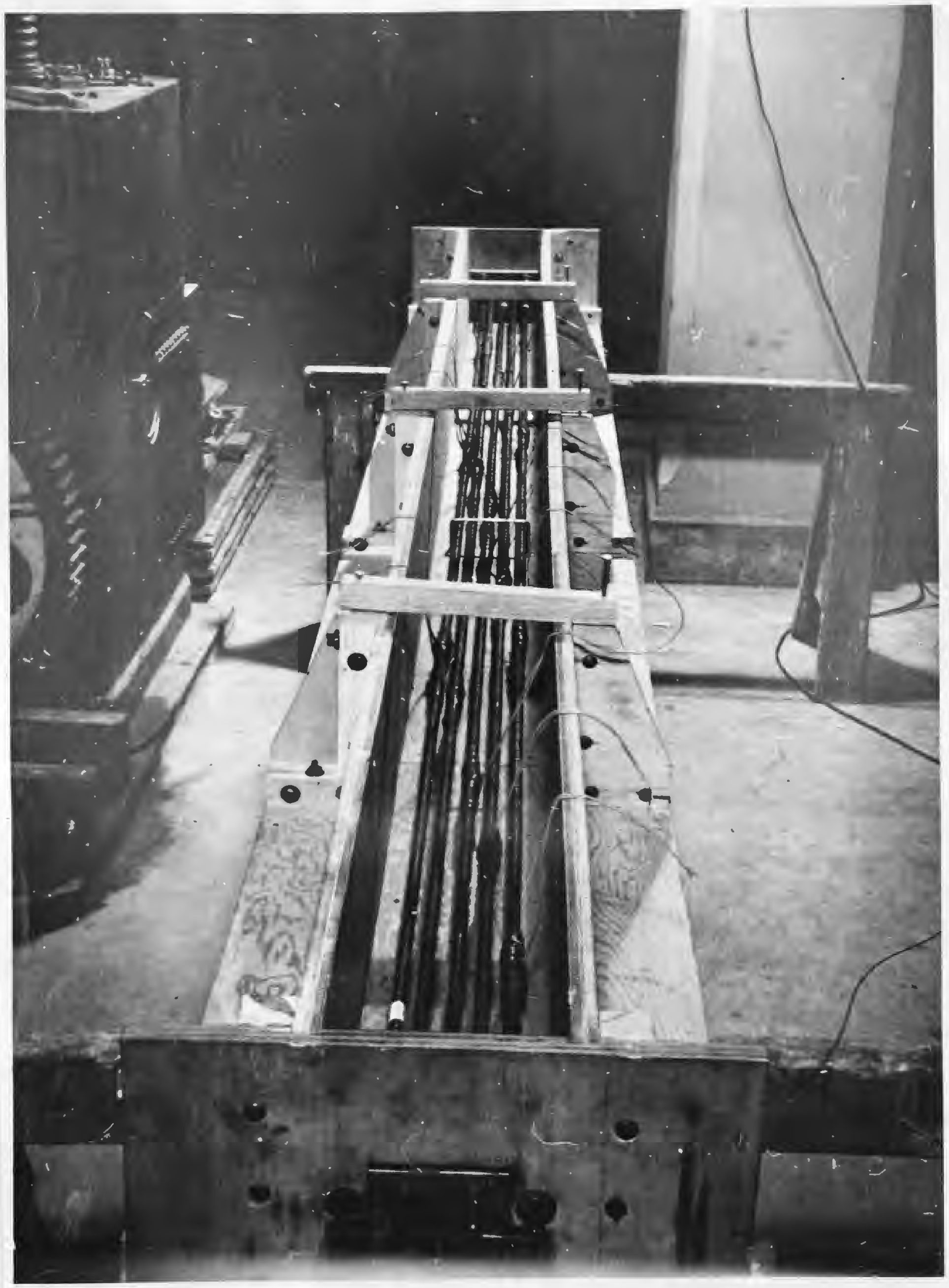


Fig. 6
General view of form, showing bracing and method of bringing out strain gage lead wires. Prestressing wires are under slight tension.

Fig. 7
Interior of form showing bearing plate, anchoring bolts, loom-covered wires and, in lower left, SR-4 strain gage wrapped with rubber tape.

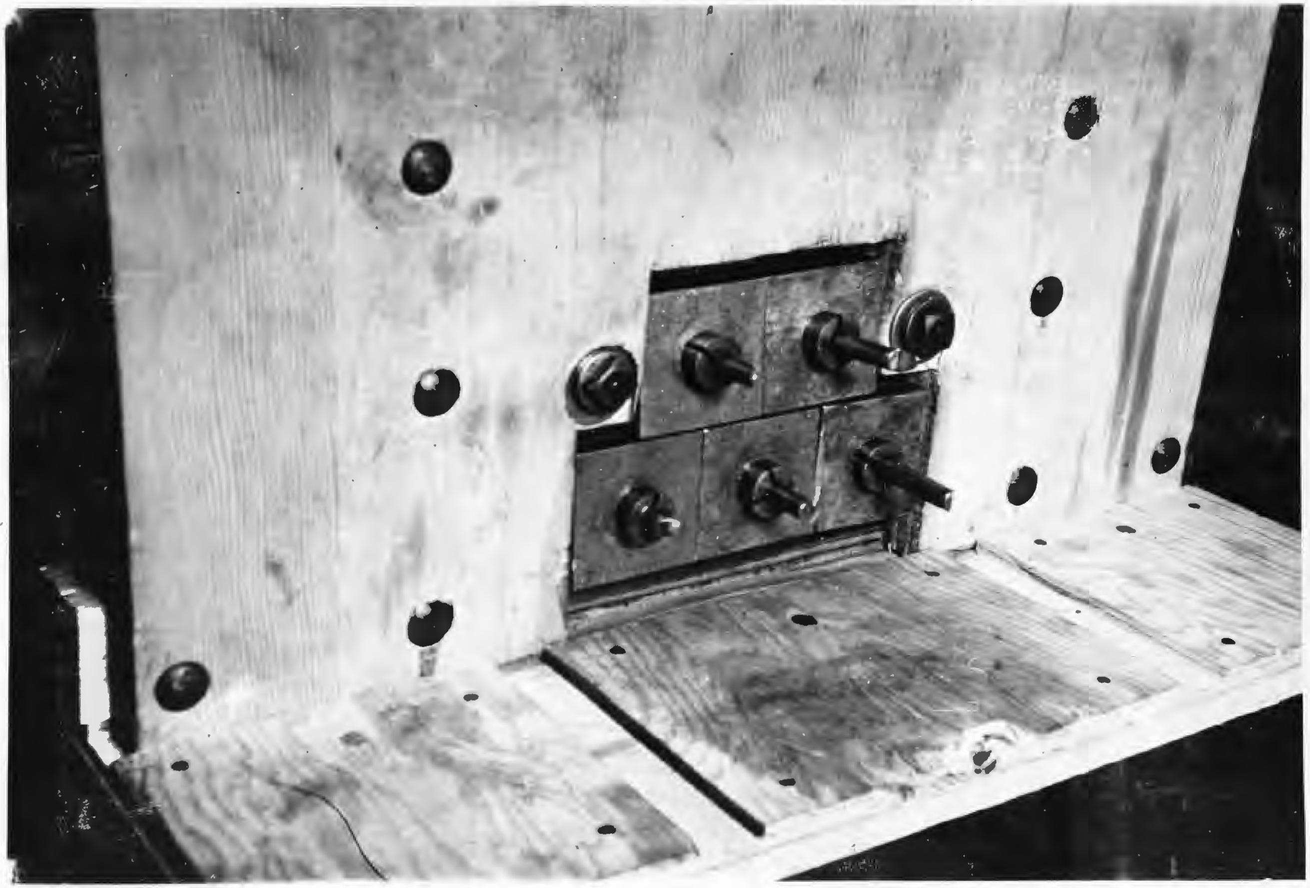


Fig. 8
Form end before casting concrete, showing end-anchorages bearing against bearing plate.

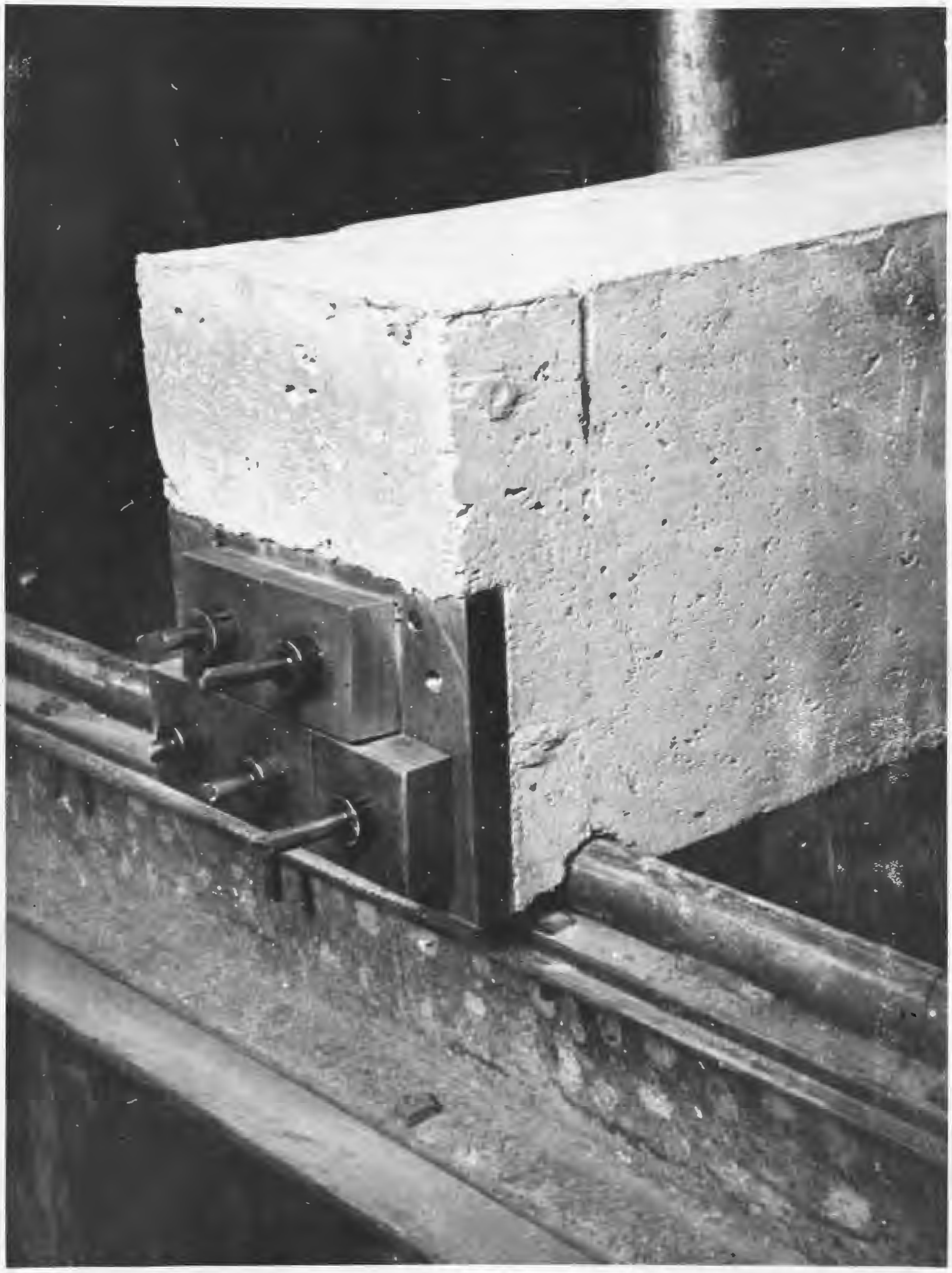


Fig. 9
View of beam end, showing bearing plate, end-anchorages, and keyway.

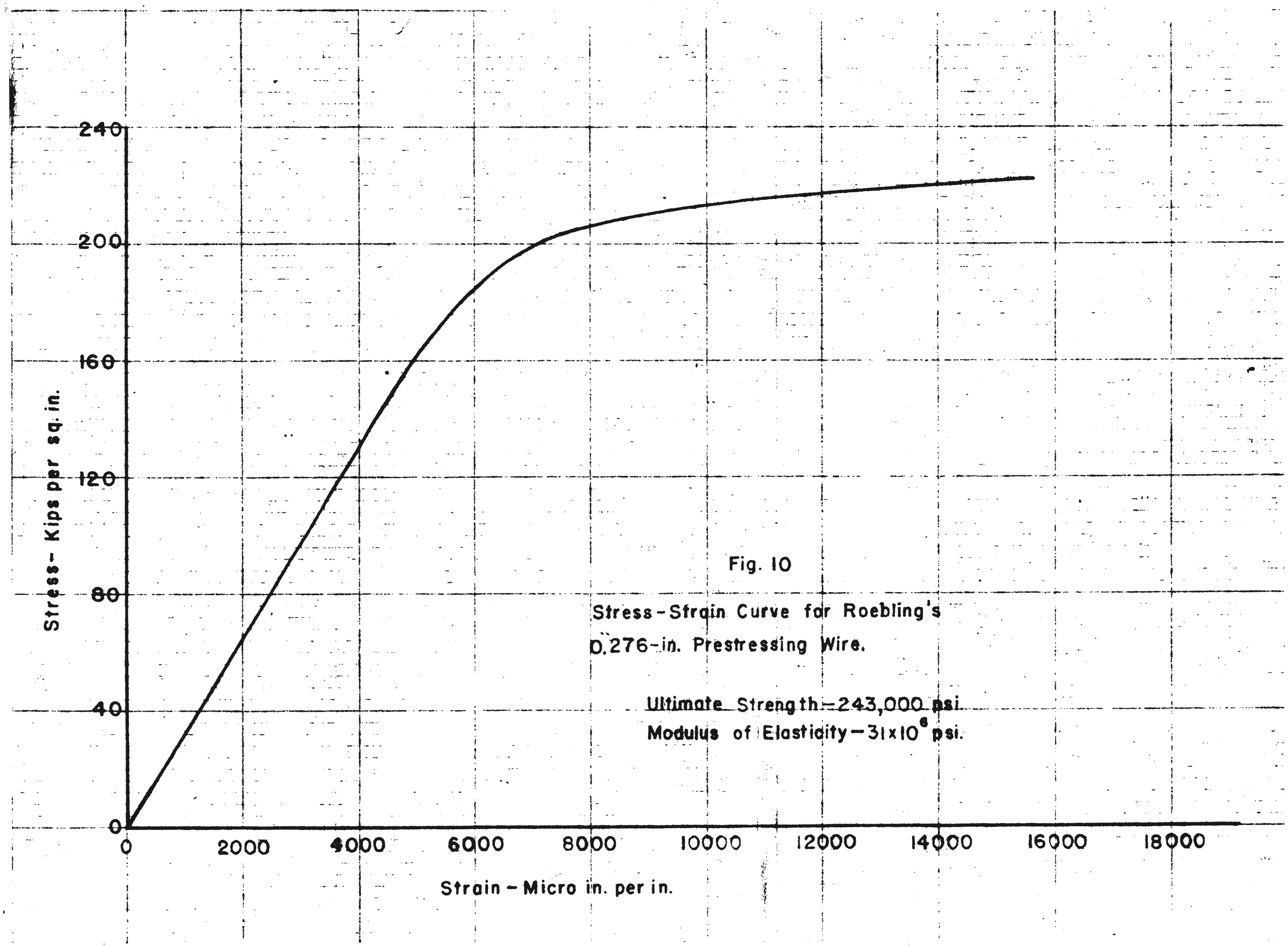


Fig. 10

Stress-Strain Curve for Roebling's 0.276-in. Prestressing Wire.

Ultimate Strength - 243,000 psi.

Modulus of Elasticity - 31×10^6 psi.

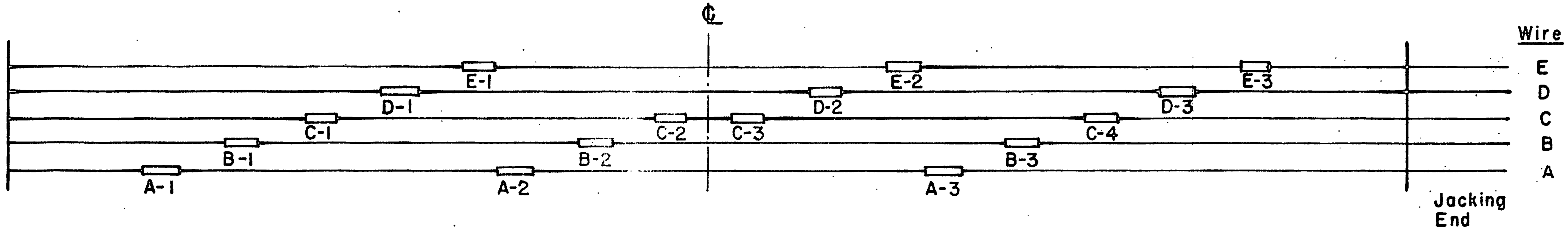
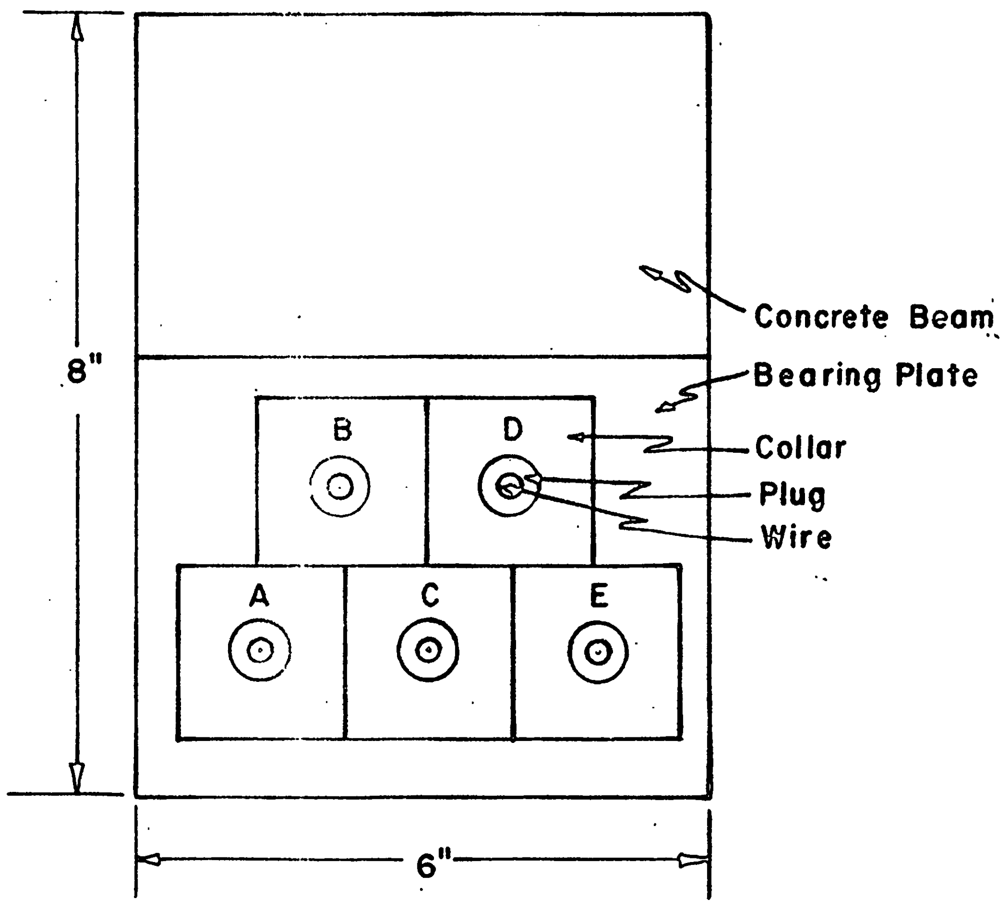
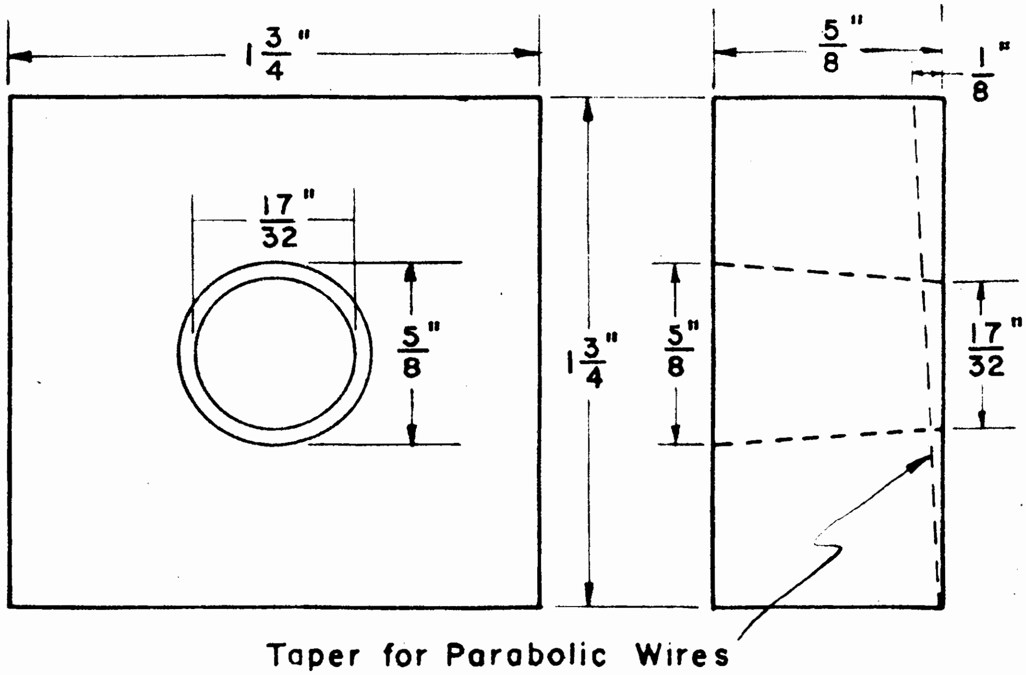


Fig. 11 Location of SR-4 Strain Gages on Prestressing Wires

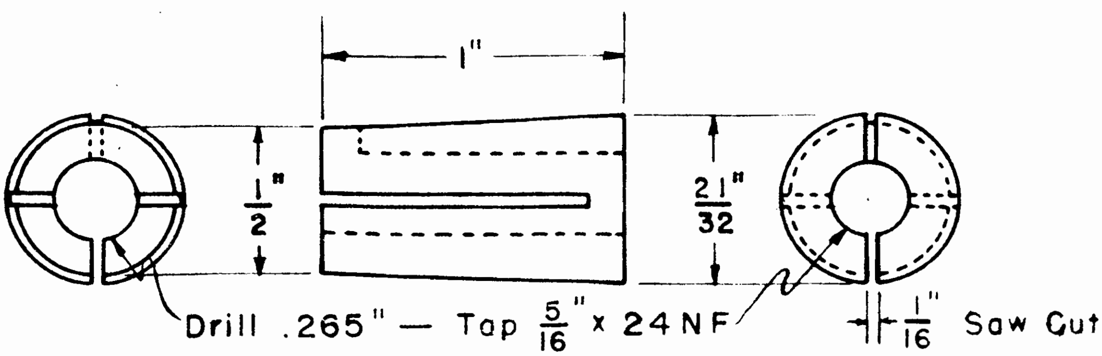
Wires A, C & E Straight; Wires B & D Parabolic.

Fig. 12 Location of Wires at End of Beam.





Collar — Cold Rolled Steel



Plug — Case Hardned Steel

Scale: $1\frac{1}{2}$" = 1"

Fig. 13

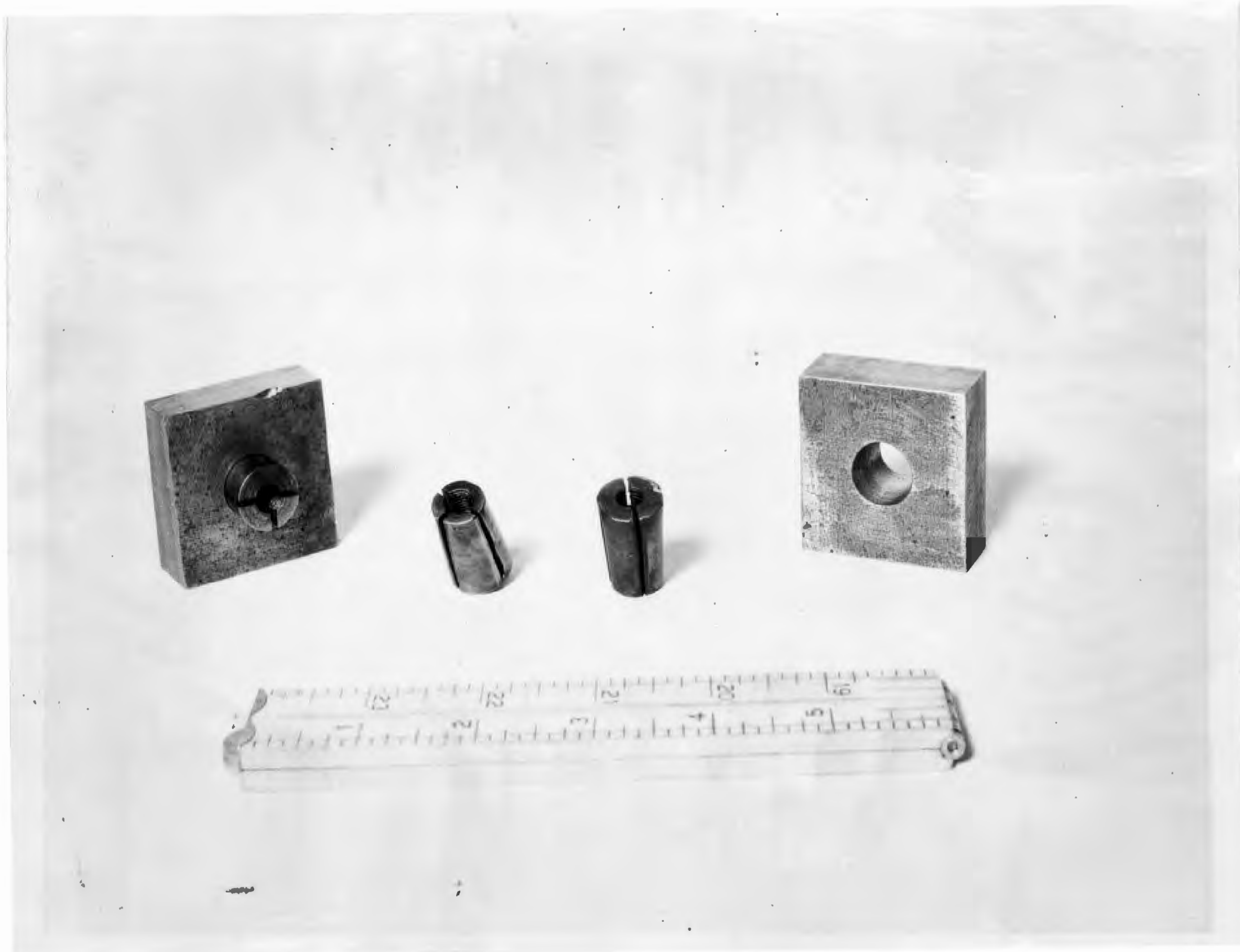


Fig. 14
Collar and plug used as end-anchorage.

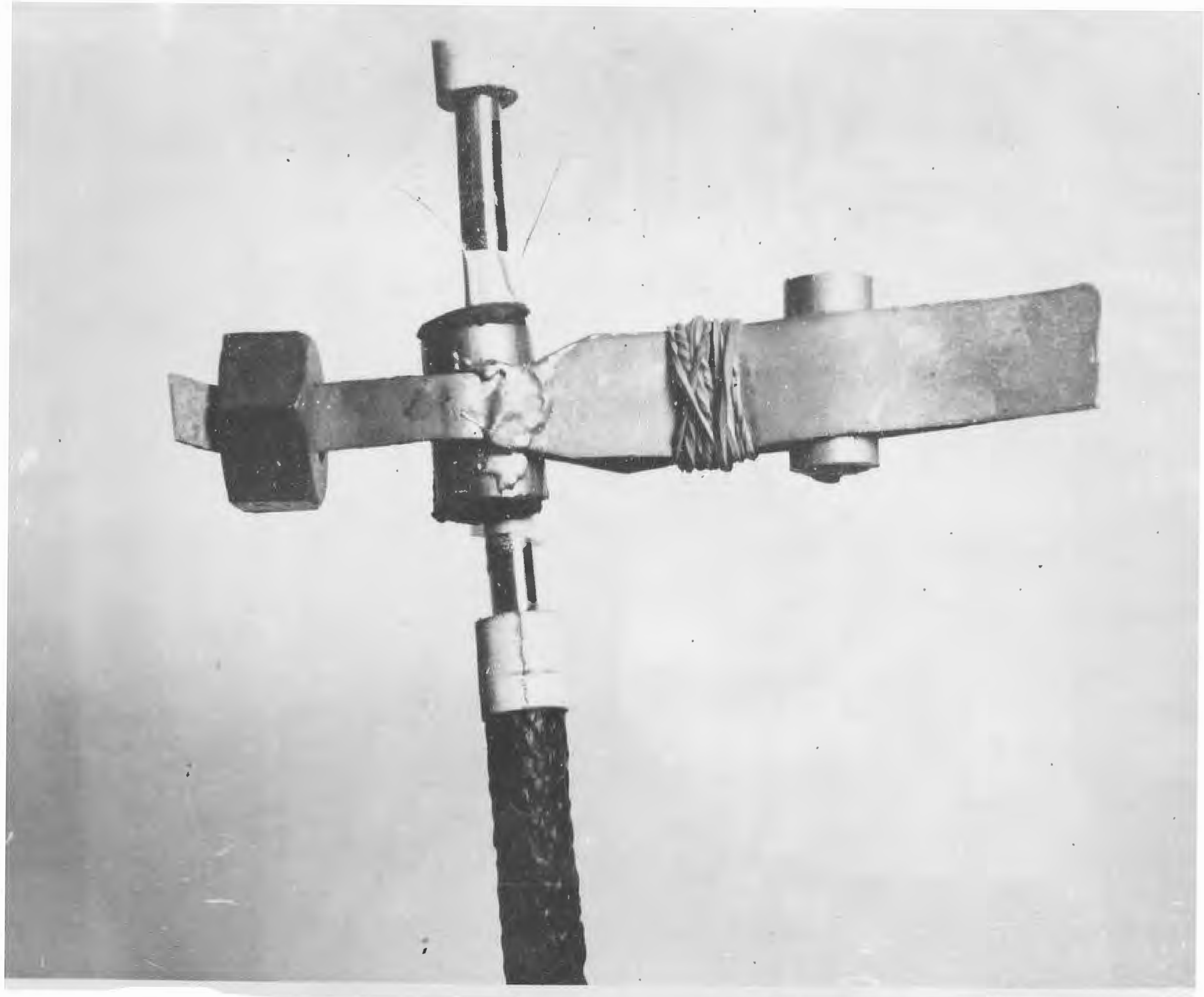


Fig. 15
SR-4 Strain Gage being comented to wire under pressure of special clamp. Note gage lead wires.

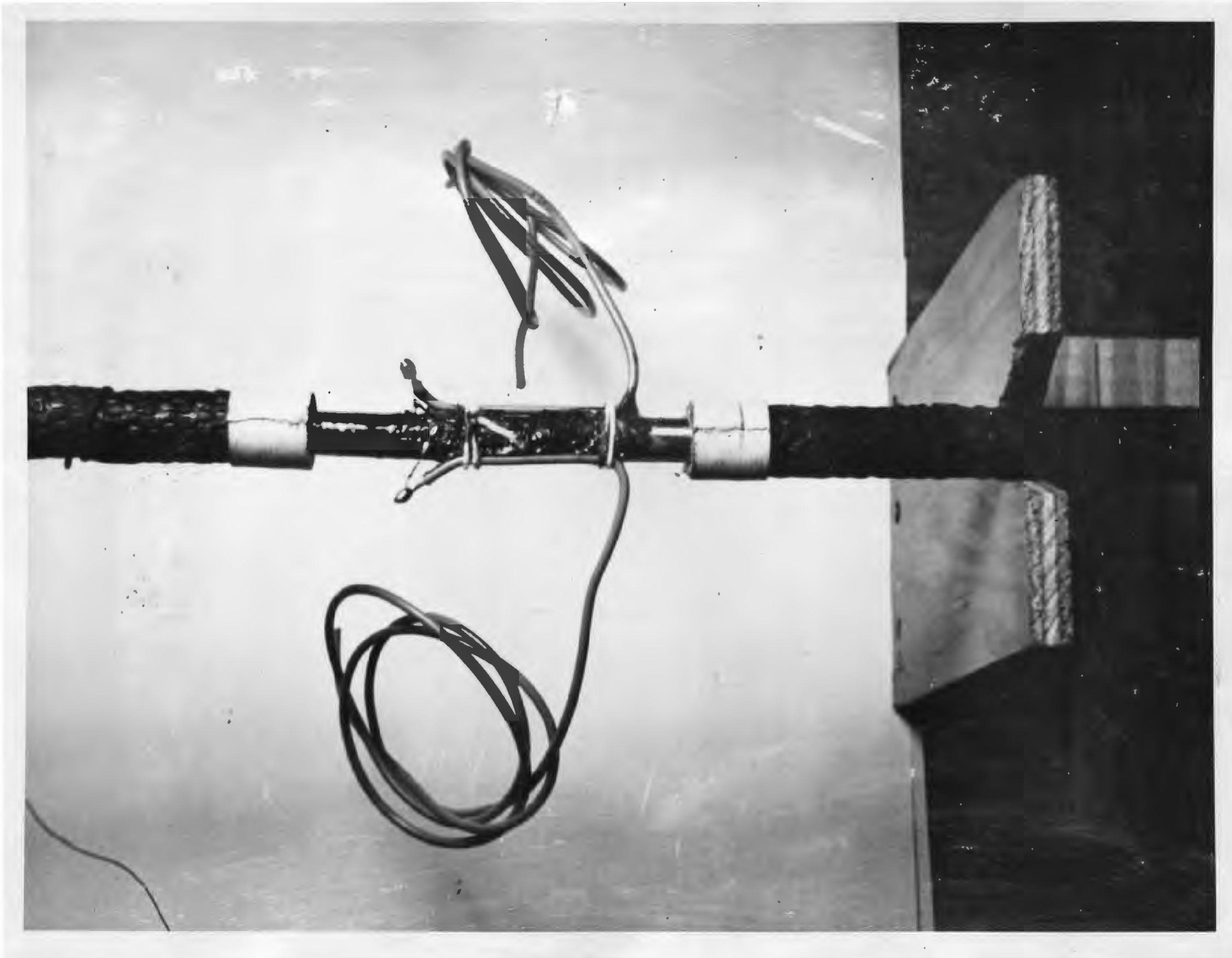


Fig. 16
External lead wires soldered to strain gage leads and one coat of rubber cement applied. Note electrical loom covering prestressing wire.

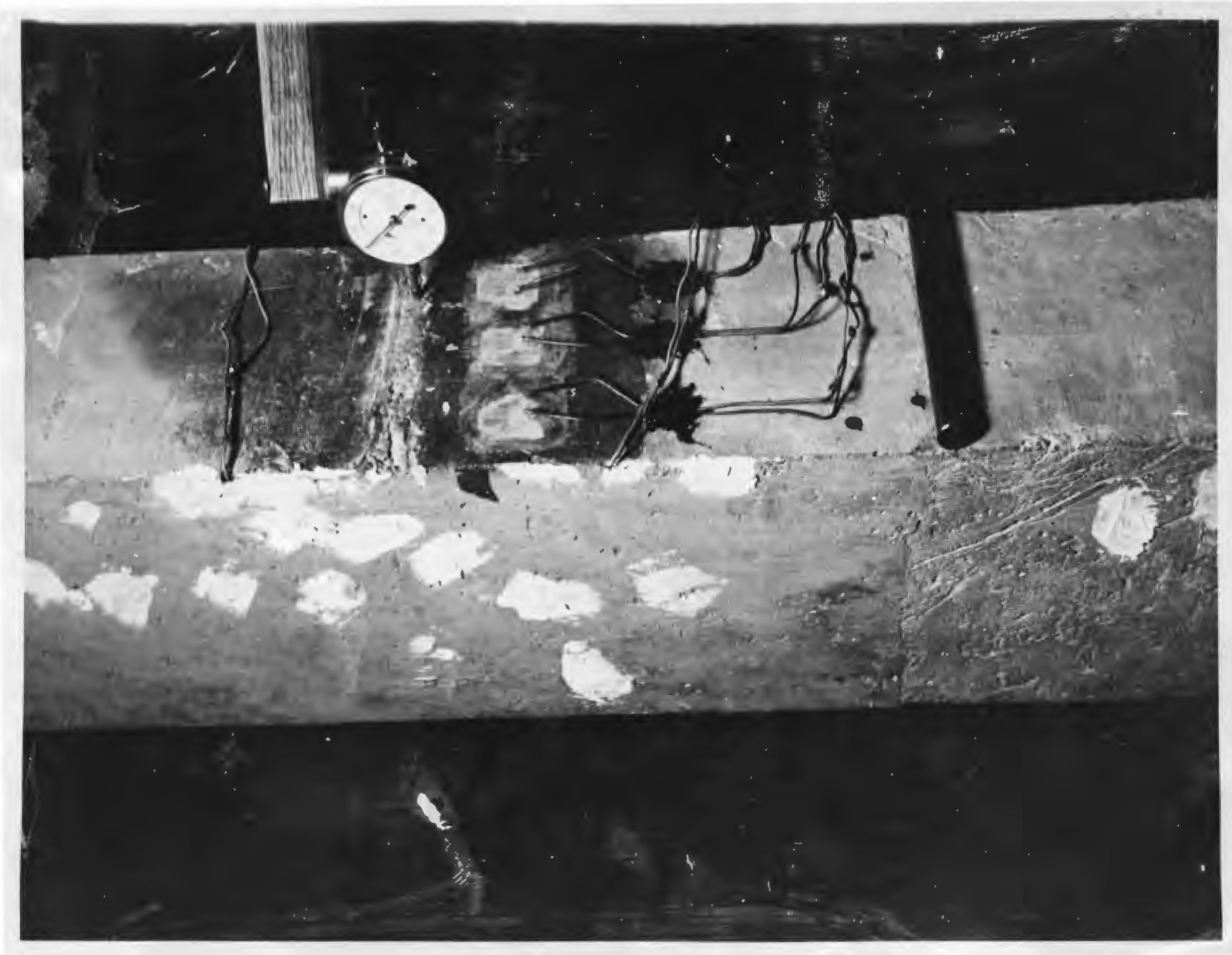


Fig. 17
SR-4 Strain Gages and Ames Dial on top of beam. The Ames Dial is at mid-span.

TEST CYLINDERS

Three of the standard 6-in. by 12-in. test cylinders were subjected to compressive testing at the time of application of prestress, which corresponded to a concrete age of seventeen days. It was noted, when the forms were removed, that four of the cylinders showed marked honeycombing, indicating insufficient rodding at the time of pouring. The other three cylinders were very sound and smooth. The cylinders were all capped on both ends with plaster of paris in the recommended manner. All cylinders were tested on a 200,000-lb. Tinius-Olsen Universal Testing Machine.

The first cylinder tested proved entirely unsatisfactory. Apparently the cylinder was inaccurately placed in the machine, or the load was applied too fast because the cylinder broke before any load readings could be taken.

Cylinder number two was one of the two which were instrumented with SR-4 strain gages. Extreme care was taken in centering this cylinder, and the load was applied at the slowest speed. The movement of the loading head was stopped at intervals, and the applied load and corresponding strain on the two gages were recorded. This information is incorporated in the stress-strain diagram, Fig. 21. This cylinder was loaded to the full capacity of the testing machine--200,000-lb., corresponding to a stress of a little over 7000 psi.--the only indications of failure being a small flaked-off place on the bottom. See Fig. 18. When all the load was removed from the cylinder, both gages returned to zero, indicating no permanent set.

The third cylinder tested (without SR-4 strain gages) was also loaded to the capacity of the machine without complete failure. At stresses of 6220 psi., 6530 psi., and 6960 psi. pronounced crushing was noted around the top and bottom, there being considerable honeycombing at these points. At a load of 200,000-lb (7070 psi. stress) the machine was shut off. After the machine had stopped and while the beam was being brought into final exact balance, the cylinder broke.

At the time of testing the beam, the concrete age being thirty-one days, the remaining four cylinders were subjected to compression testing. Cylinder number four (Fig. 19) was the second of the instrumented cylinders. Data was taken as on cylinder number two. This cylinder showed a definite crushing strength of 6855 psi. Cylinder number five was loaded to 190,660-lb.--6730 psi.--at which load it failed. Fig. 20 shows this cylinder after failure. Cylinders number six and seven were loaded to 200,000-lb.--7070 psi.--and showed no indication of failure whatsoever.

The stress-strain diagram (Fig. 21) shows the average stresses for cylinders number two and four, the two instrumented cylinders. From this curve the modulus of elasticity was calculated by the three recognized methods; i.e., by the tangent to the curve at the origin, by the tangent to the curve at the working stress, and by the secant to the curve at the working stress. The results are shown in the following table:

Modulus of Elasticity:

Tangent at origin	8.06×10^6 psi.
Tangent at 2400 psi.	7.98×10^6 psi.
Secant at 2400 psi.	8.15×10^6 psi.
Average	8.06×10^6 psi.

Fig. 18
Cylinder No. 2 after loading to limit of testing machine--7070psi. Small flaked-off area near bottom was only indication of failure. Note SR-4 Strain Gage.

Fig. 19
Cylinder No. 4 after failure: stress-6855 psi.
Note SR-4 Strain Gage on side.

Fig. 20
Cylinder No. 5
Breaking strength 190,660 lbs.--6730 psi.

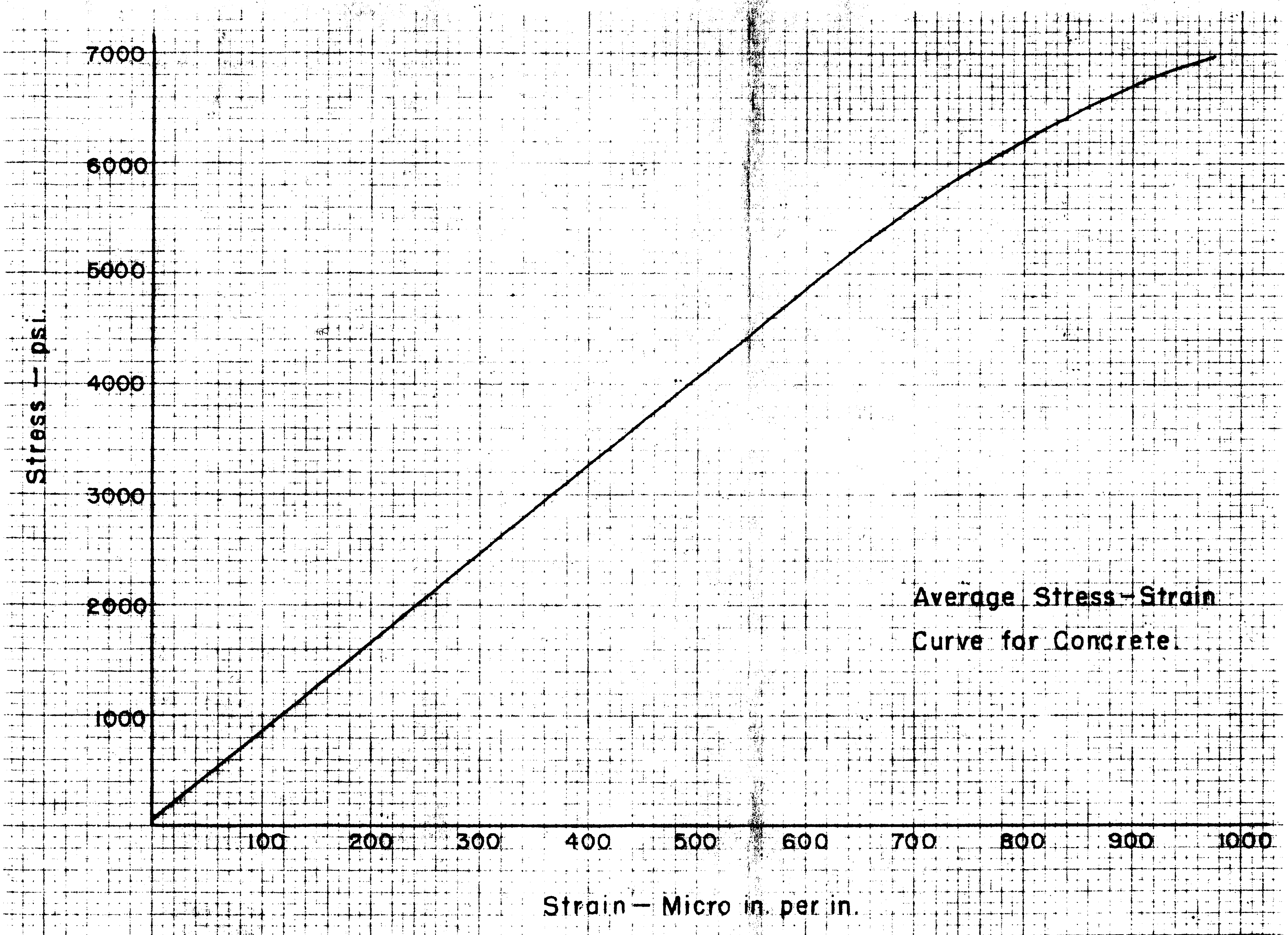

7000
6000
5000
4000
3000
2000
1000
Stress — psi.
100
200
300
400
500
600
700
800
900
1000
Strain — Micro in. per in.
Average Stress-Strain
Curve for Concrete.

Fig. 21

PRESTRESSING

The beam was removed from the form after curing for seven days and was placed on the testing frame. There was practically no honeycombing whatsoever on the beam, and the concrete appeared very dense and sound. Strain gages were applied to the beam and extensions were soldered to the lead wires and connected to the bridge balancing units. The sixteen strain gages on the wires were connected to an Anderson 24-switching unit Bridge Balancer, and the six surface gages on the outside of the concrete beam were connected to a Baldwin-Southwark 12-switching unit Bridge Balancer. Each of these units was in turn connected to a Baldwin-Southwark Model K Strain Indicator which recorded strains directly in micro inches per inch. At this point all the gages were balanced to record zero. This balance was not changed throughout the testing.

The testing frame used was a modification of the frame designed by J. H. Senne.(13) The modification consisted of

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(13) Ibid., p.24.

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lengthening the frame from a five-foot span to a ten-foot span by substituting longer I-beams for the sides. The two vertical T-sections and the cross I-beam were kept at mid-span. The two ends of the frame were the same ones used by Mr. Senne. This changed the capacity of the frame from 3-ft. x 5-ft. to 3-ft. x 10-ft. Extensions were welded to the T-sections to increase the distance between the top of the concrete beam and the cross-beam.

The concrete was prestressed by stretching each wire individually with a jacking unit which consisted essentially of two Simplex 30-ton hydraulic rams of the center-hole type. One of these rams was used to pull the wire, and the other was used to push the plug home into the collar when the desired wire stress was reached. These two units were positioned bottom-to-bottom by two steel rods bent to form U-bolts and bolted through a 3/8-in. steel plate at the top of the pushing ram. A hole was bored in the center of this plate so that the 2 1/4-in. diameter piston of the ram could pass through it. Welded around this hole was a 4 1/2-in. length of 2 1/2-in. pipe which had a 2 1/2-in. to 1-in. pipe reducer welded to it. This reducer was machined flat on the small end to secure good bearing against the steel collars. The reducer, pipe, and plate unit served as a stand-off to hold the rams away from the end of the beam so that the wires not being stretched could be kept out of the way. A follower made from a piece of 3/4-in. steel rod, drilled 5/16-in. throughout its length, was turned to a tight fit and pressed into the center hole of the pushing ram. This rod served as an extension of the piston of the pushing ram and acted against the plug to push it home.

In this way the pulling ram was able to react through the stand-off unit against the end of the beam itself as the wire was being stretched and was independent of the pushing ram which was used only for setting the plug. Both rams were connected to pumping units by 48-in. lengths of flexible hose. The pumping unit for the pushing ram was a standard Simplex RP-6 pump designed for use with the Simplex hydraulic rams. The pumping unit for the pulling ram was adapted from the hydraulic unit of a war surplus aircraft tail hoist. See Figs. 22-26.

The wire tensioning process involved the following steps:

1. A collar and plug unit was slipped over a wire to bear against the bearing plate in the end of the beam opposite the jack. This plug was driven in tight until the threads inside the plug gripped the wire.
2. A collar and plug unit was slipped over the same wire to bear against the bearing plate at the jack end of the beam. This plug was left loose, but care was taken to see that it remained in the hole in the collar.
3. The jacking unit was then slipped over the wire, the stand-off end bearing against the collar. The wire passed through the follower rod and the center hole of both rams.
4. Another collar and plug unit was slipped over the end of the wire to bear against the piston of the pulling ram. This plug was also driven tight so that it would not slip on the wire.

5. A strain reading of 4200 micro inches per inch was set on the Strain Indicator. This strain corresponded to a wire stress of about 136,000 psi. The switching unit was set to one of the gages on the wire to be stretched.
6. The pulling ram was activated, and the wire was stretched until the Strain Indicator returned to zero, indicating that the wire had reached the desired stress.
7. While this load was being held, the pushing ram was activated and the plug driven home as tightly as possible.
8. Pressure on the pulling ram was released, the collar and plug at the end removed, and the whole jacking unit taken off.
9. This same procedure was repeated on all five wires. The wires were stressed in the following sequence: C,D,B,E,A.

Considerable difficulty was encountered in the process of prestressing. Almost all of the difficulty was due to the inadequacies in the end-anchorage units. It was found that it took several driving blows to set the plug satisfactorily at the end opposite the jack. Each of the plugs at this end showed some slipping during the stretching of the wires. When this happened, it was necessary to release the jacks and to use a sledge hammer and a short length of steel rod bored at one end and slipped up against the plug to reset the plug.

On two of the wires--A and E-- this slipping proved particularly bad. In both cases the wire very suddenly slipped all the way through the plug and about one inch up into the beam. The absence of bond made it possible to drive these wires back through the beam and to reset the collar and plug around them. The sudden and excessive movement of these two wires inside the beam caused all the strain gages on both wires to be stripped off and become inactive. When these two wires were finally stretched, a pressure dial on the hydraulic unit of the pulling ram was used as an indication of the load applied.

On the other three wires a total of five gages was lost in the prestressing process, leaving only five out of sixteen strain gages still active when the testing was begun.

Owing to the unforeseen difficulties encountered, the complete process of prestressing required two days. Records of the time of stressing each wire were kept. Also, strain readings were taken on both the wires and the concrete for a period of twelve days from the time of prestressing until the time of first testing, the purpose being to observe any loss of prestress which might occur over that short period. The results of this observation are shown on Figs. 27-29 in which the average stress in each wire is plotted against time. The loss of prestress is figured from the time that prestressing was completed, because, as may be seen from the graphs, the stretching of each successive wire caused a sharp reduction of stress in the wires already tensioned.

The graphs may be summarized as follows:

Wire	Initial Prestress Psi.	Final Prestress After 12 Days Psi.	Loss In Prestress Psi.	Per Cent Loss
A	Gages Inactive			
B	156,250	141,000	15,150	9.8
C	138,900	134,750	4,150	3.0
D	70,700	64,400	4,150	8.9
E	Gages Inactive			

It should be noted that there was considerable slipping of the anchorage on wire D, resulting in a loss of 65,300 psi. or 48 per cent from the desired 136,000 psi. Several attempts were made to correct this loss, but a stress higher than the initial 70,700 psi. could not be maintained. Apparently, the threads in the plug had cut into the wire in such a manner that the wire was not actually separate from the plug, and the plug was simply moving with the wire.

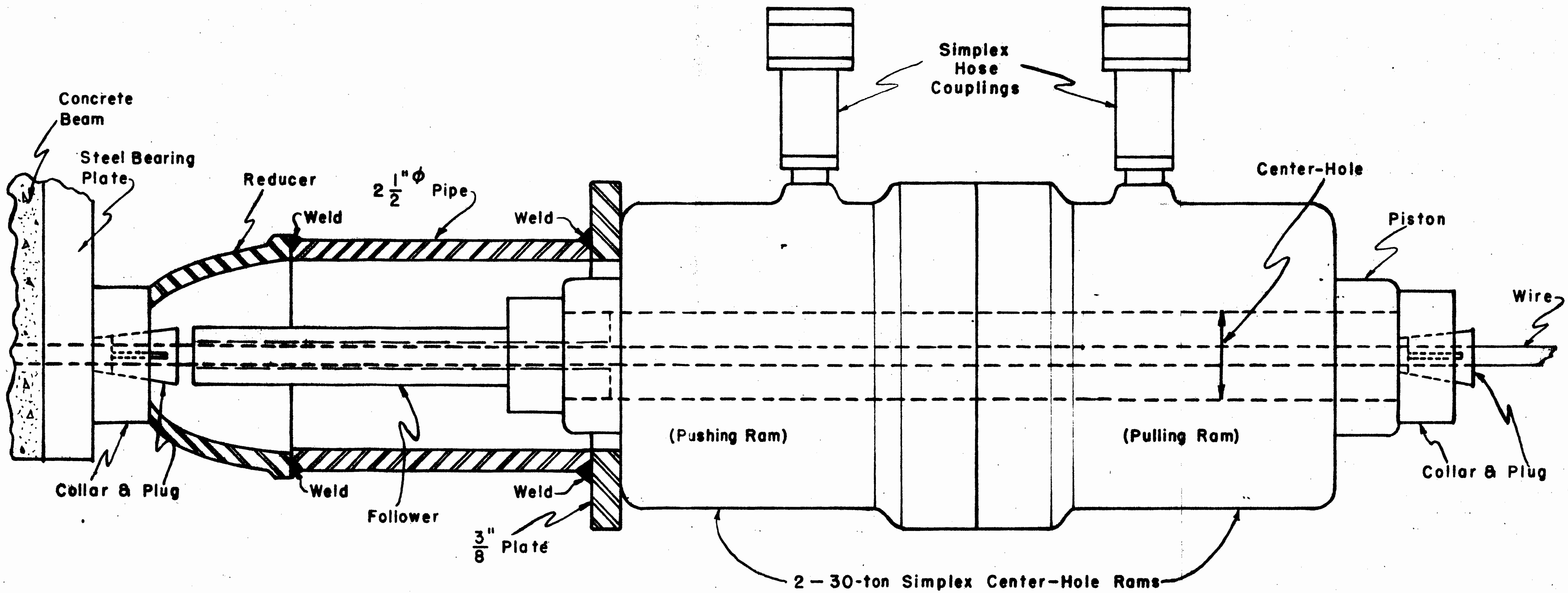


Fig. 22 Diagrammatic Sketch of Prestressing Unit

(Stand-Off Assembly Shown in Half-Section)

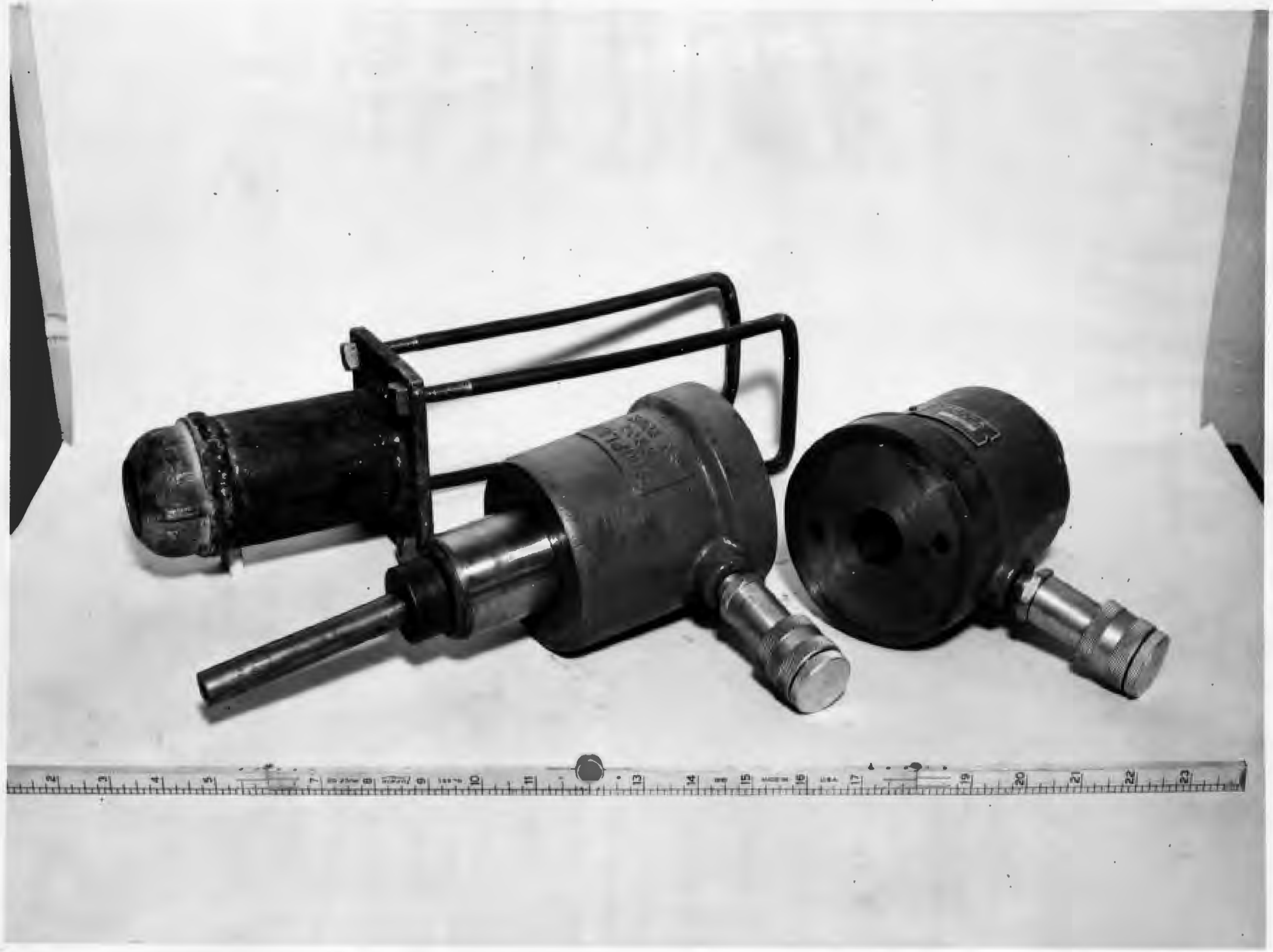


Fig. 23
Component parts of prestressing unit. Left to right: stand-off unit, pushing ram with follower, pulling ram.

Fig. 24

Prestressing unit assembled.

Fig. 25
Jacking unit. Left to right: Simplex pump to activate pushing ram, hose connections, prestressing unit, hydraulic pump to activate pulling ram.

Fig. 26
Jacking unit in position for stressing a wire. Strain gage switching units at extreme right.

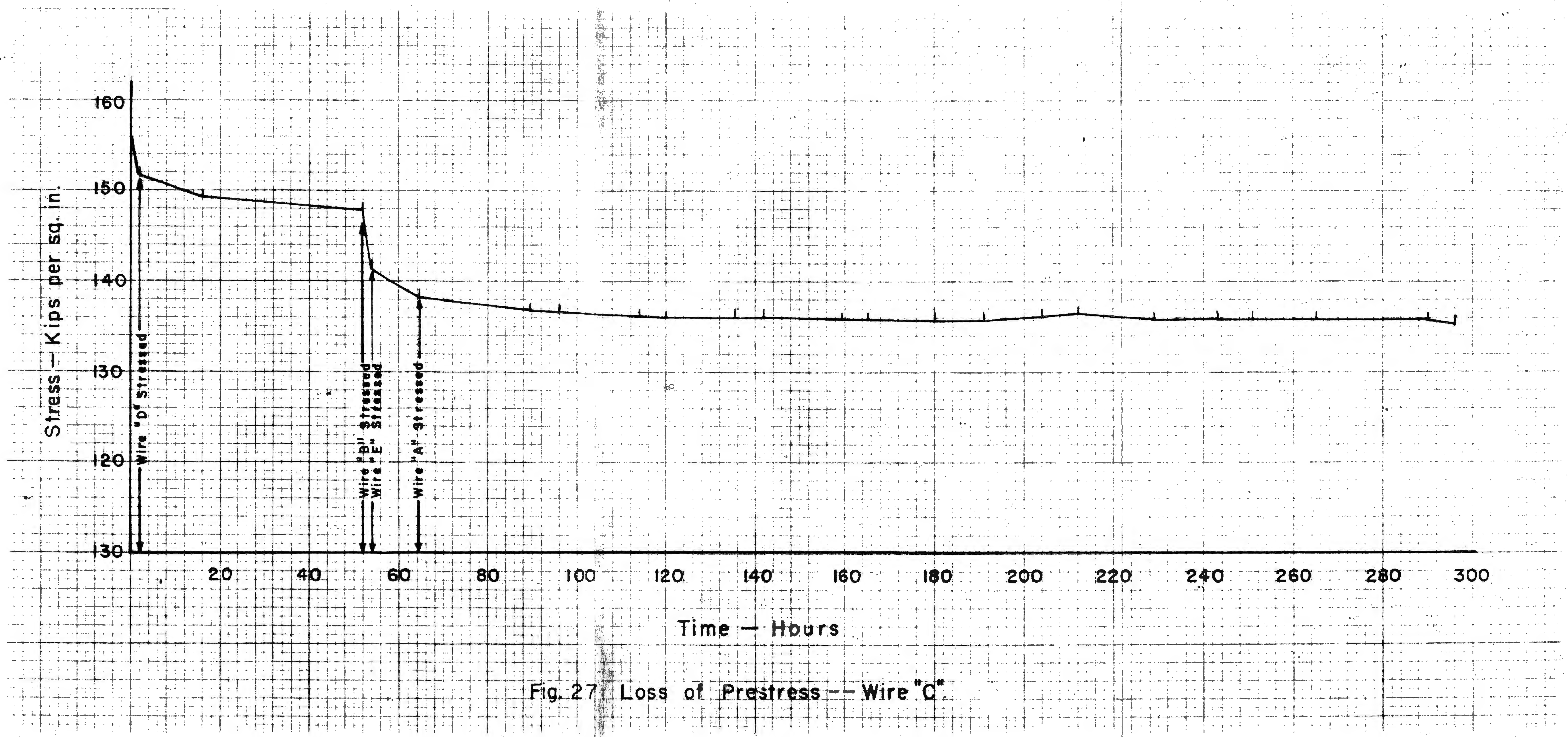


Fig. 27 Loss of Prestress -- Wire "C"

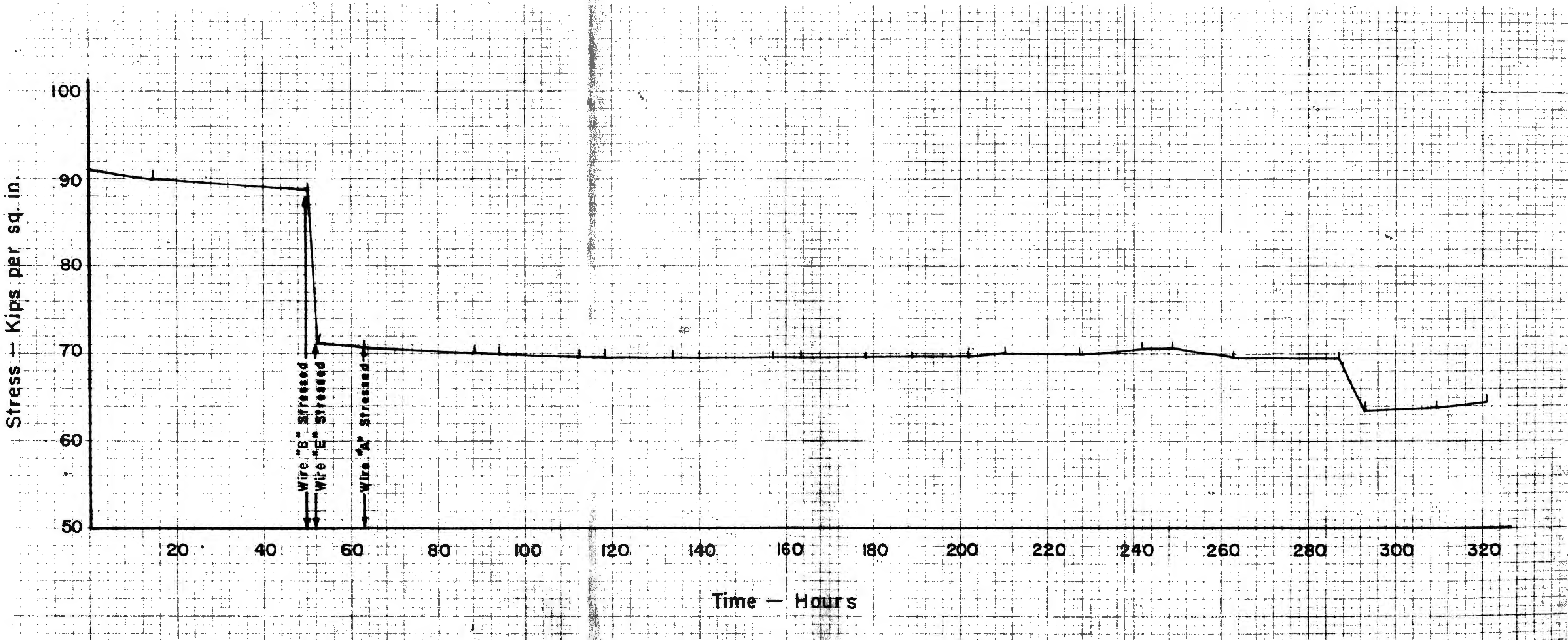


Fig. 28 Loss of Prestress -- Wire "D".

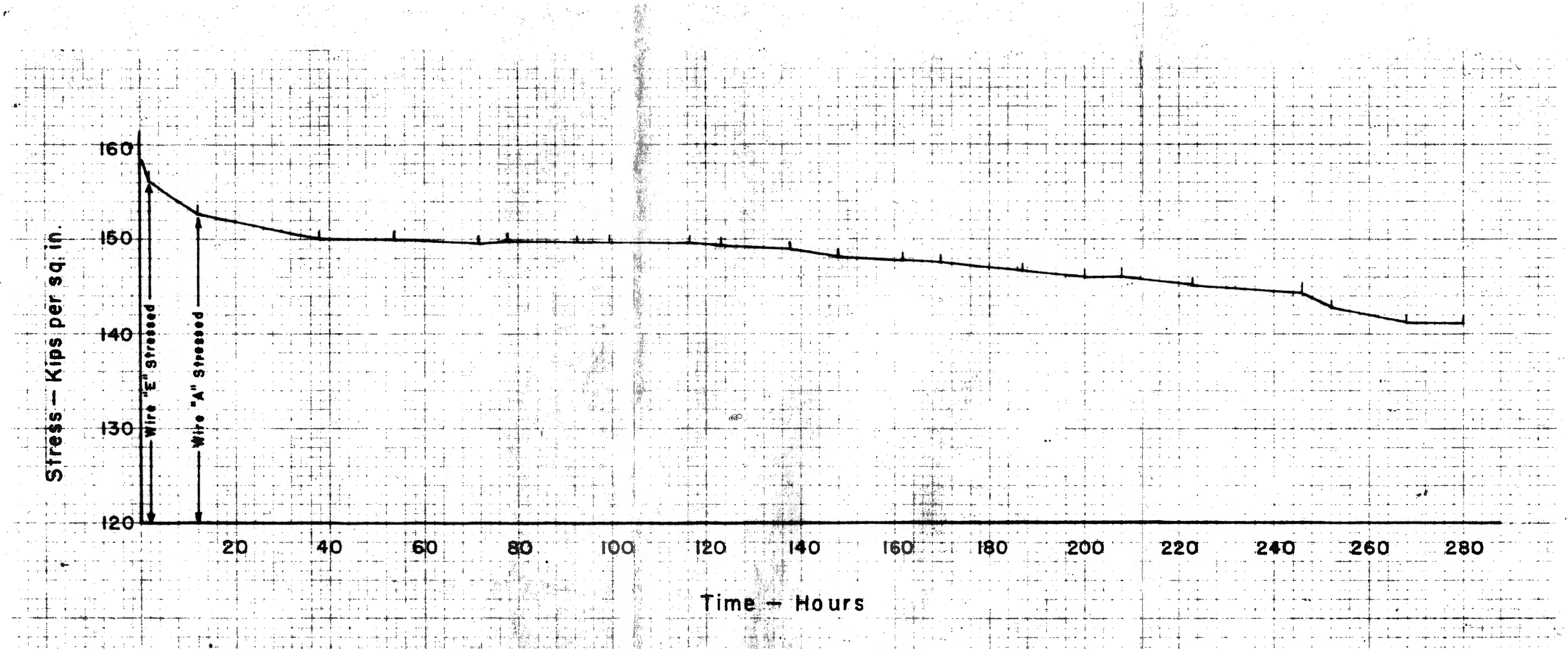


Fig. 29. Loss of Prestress -- Wire "B".

TEST PROCEDURE AND RESULTS

The testing of the beam was done in two stages. In the first stage, the load was carried to slightly beyond the point where cracking occurred. The results of strain readings taken during this loading were tabulated and interpreted prior to the second stage of loading in which the beam was tested to destruction.

The load was applied by an 8-ton hydraulic jack located at the mid-span of the beam. The load was transmitted to the beam through two 4-in. I-beams whose reaction points were eleven inches on either side of the mid-point of the beam. Two short lengths of 5/8-in. steel rods served to transfer the load from the I-beams to the concrete beam. This arrangement was decided upon so that there would be pure moment and no shear at the center of the beam and so that clearance would be provided for the strain gages located on the top surface of the beam at the center. The applied load was measured by means of a loading cell placed between the top of the jack and the cross-beam of the testing frame. All loads mentioned in the discussion which follows are loads measured at the jack.

This loading cell was made of a piece of magnesium rod 2 1/2-in. long with a diameter of 1 3/4-in. and was instrumented with four type A-7 strain gages. Two measuring gages were cemented on opposite sides of the rod, parallel to the axis, thus preventing any error which might be introduced by eccentricities in loading. These gages were connected in series. The other two gages served as compensating gages and were cemented to the rod 180^{o} apart and 90^{o} from the measuring gages with their axes perpendicular to the axis of the rod. These gages were also connected in series. All gages were affixed in the same manner as described for those placed on the wires, and the loading cell was wrapped with electrician's plastic tape. The loading cell was then calibrated in the Tinius-Olsen testing machine, using loading increments of 250 pounds up to 20,000 pounds. A piece of sheet lead was placed at each end of the loading cell to provide positive bearing against the jack and the cross-beam.

The first stage of loading was begun by applying loads to the beam in increments of 250 pounds. After each increment of load was applied, strain gage readings were taken on all the active gages, and deflections were read from Ames dials. Five of these dials were fixed to an independent stand and arranged so that they were evenly spaced along the top of the beam, one dial being at the center line. The Ames dials read in thousandths of an inch up to one inch.

The loading was continued in 250 pound increments until a total load of 3000 pounds was applied. All readings were taken, and the load was released. Then the zero load readings were checked. These readings varied from the initial readings by from 0 to 10 micro inches for the wires and by from 30 to 45 micro inches for the concrete. The maximum deflections under the 3000 pound load was 0.091-in.

Loading was started again and taken up to 6000 pounds in 250 pound increments, at which point the loading increment was changed to 500 pounds and the load continued to 9000 pounds. Strain and deflection readings were taken at each new increment.

The first visible crack appeared between 7500 and 8000 pounds of load. This crack was directly under one on the points of application of the load to the beam. When the load of 9000 pounds was reached, large cracks had opened under both load points and near mid-span. Smaller cracks had appeared between these. Fig. 34 shows very clearly the size and extent of the largest crack under a load of 9000 pounds. This was the first crack to appear. The maximum width of this crack was about 1/16-in., and it extended over half the depth of the beam. The maximum deflection under the 9000 pound load was 0.344-in.

After sufficient readings and investigations were made while holding the 9000 pound load, the jack was released and the load returned to zero. Again zero load strain and deflection readings were taken. Variations of from 0 to 140 micro inches from the initial strain readings for the wires and of from 15 to 35 micro inches from the initial strain readings for he concrete were noted. No permanent deflection was noted.

Upon removal of the load all cracks closed completely. Fig. 35, which was taken after removal of the load, shows the same area as Fig. 34. Here may be seen how completely the largest crack closed. The trace of the crack may be seen in the flaked-off whitewash with which the side of the beam was painted in order to make cracks more visible.

Loading was resumed and carried to 10,000 pounds in increments of 1000 pounds. The same cracks that had appeared before opened under a load of 7000 pounds. The width of the largest crack was about 3/32-in., and it extended 4 7/8-in. from the bottom of the beam. The maximum deflection under this load was 0.5418-in. or 1/410 of the span. This load of 10,000 pounds was approximately 1.9 times the design load of 5300 pounds. When the load was released, all cracks again closed, and the beam returned to its original position.

The second stage of testing, in which the beam was loaded to destruction, was begun six days later. Load was applied in increments of 500 pounds up to 10,000 pounds, at which point the increment was changed to 250 pounds. The first cracks were again observed at 7000 pounds of load. All cracks followed the same pattern and spread as before.

The first failure occurred at 11,500 pounds. This was a failure, not of the concrete, but of one of the end-anchorages on wire C, resulting in the loss of all tension on that wire. This, of course, materially reduced the prestress force on the beam, causing considerable extra deflection accompanied by a reduction of load down to 9250 pounds. The deflection measured with a ruler (the Ames dials having become inactive) just before this anchorage broke was 1 1/16-in. The deflection just after the break was 1 5/16-in.

After this failure, loading was resumed and carried up to a load of 10,750 pounds and a deflection of 1 13/16-in., at which load wire B pulled through its end-anchorage.

It was during this stage of loading that the beam first began to show sign of crushing. The largest of the cracks had spread to within about 1-in. of the top, and the concrete in this area began to spall and flake off as more load was applied. All crushing occurred just to the inside of one of the steel rods which served as reactions for the I-beams. Figs. 36, 37 and 38 show the location and extent of this crushing. After the first anchorage failed at 11,500 pounds load, it was impossible to reach this load again; therefore, the load at which the concrete failed could not definitely be determined.

After wire B pulled through its end-anchorage, the load was released. The tension in the three remaining wires caused the beam to resume its original position almost completely. Crushed concrete in the cracks prevented them from closing entirely.

The process of applying and releasing load was continued until wires D and E failed. The concrete continued to crush slightly in the same area. Finally, with only wire A still active, the beam was loaded to approximately 7000 pounds. The jack was extended to its limit which resulted in a beam deflection of approximately 5-in. The largest crack opened to about 1-in. at this deflection. When this load was released, the beam still attempted to resume its original position, there being a permanent deflection of 7/16-in. Most of this permanent deflection was due to concrete particles which had wedged themselves in the crack openings.

Figs. 39 and 40 show a comparison between fiber stresses for the top and bottom as calculated from the design prestressing force and as taken from SR-4 strain readings. The stresses on the bottom of the beam are in fairly close agreement up to the point at which the beam cracked. At this load the actual stresses began to fall off. This was due to the fact that cracks occurred on both sides of the strain gages, thereby isolating them from the tensile forces. The compressive stresses in the top fibers of the beam show marked deviation from the calculated stresses. This may be explained by the fact that the short gage length of the strain gages resulted in the recording of stress concentrations which were very high rather than average stress distributions.

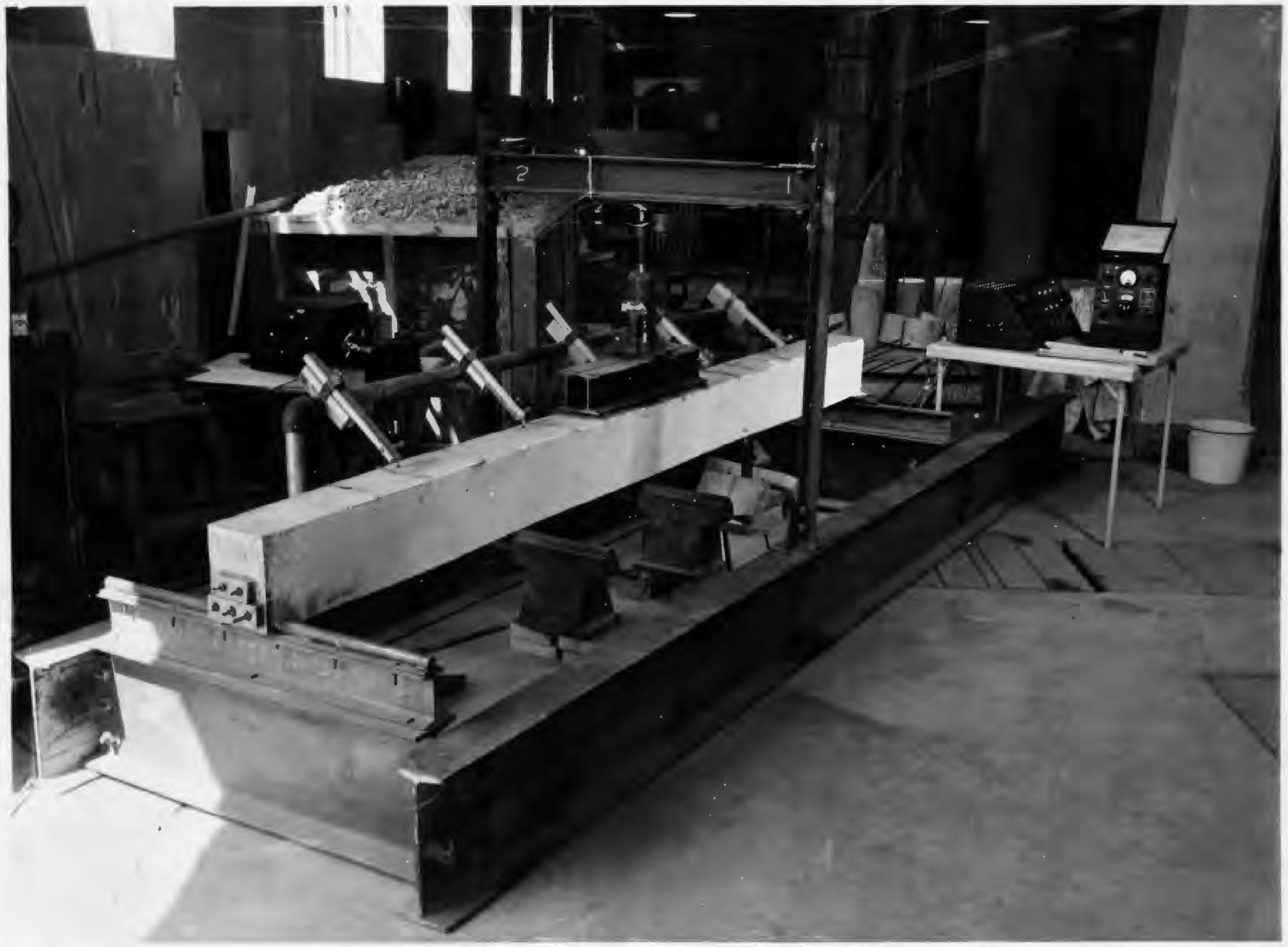


Fig. 30
Beam prior to beginning test. Left to right: Anderson unit for measuring load, jack and loading cell on beam, Baldwin-Southwark unit for measuring wire and concrete strains.

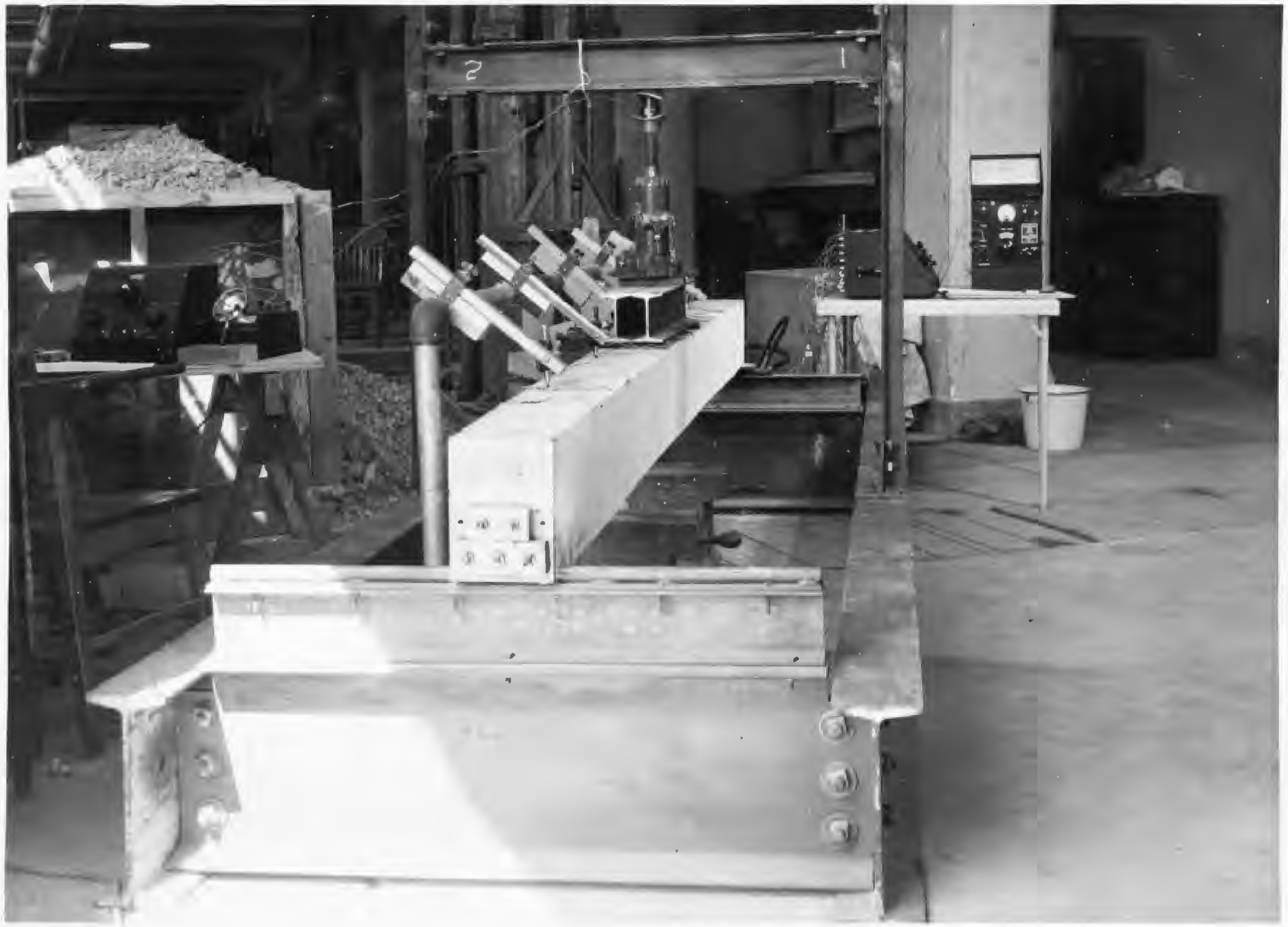


Fig. 31
General view of beam and test frame.

Fig. 32
General view of test procedure. At left: applying load and reading Ames Dials. At right: reading and recording strain measurements.

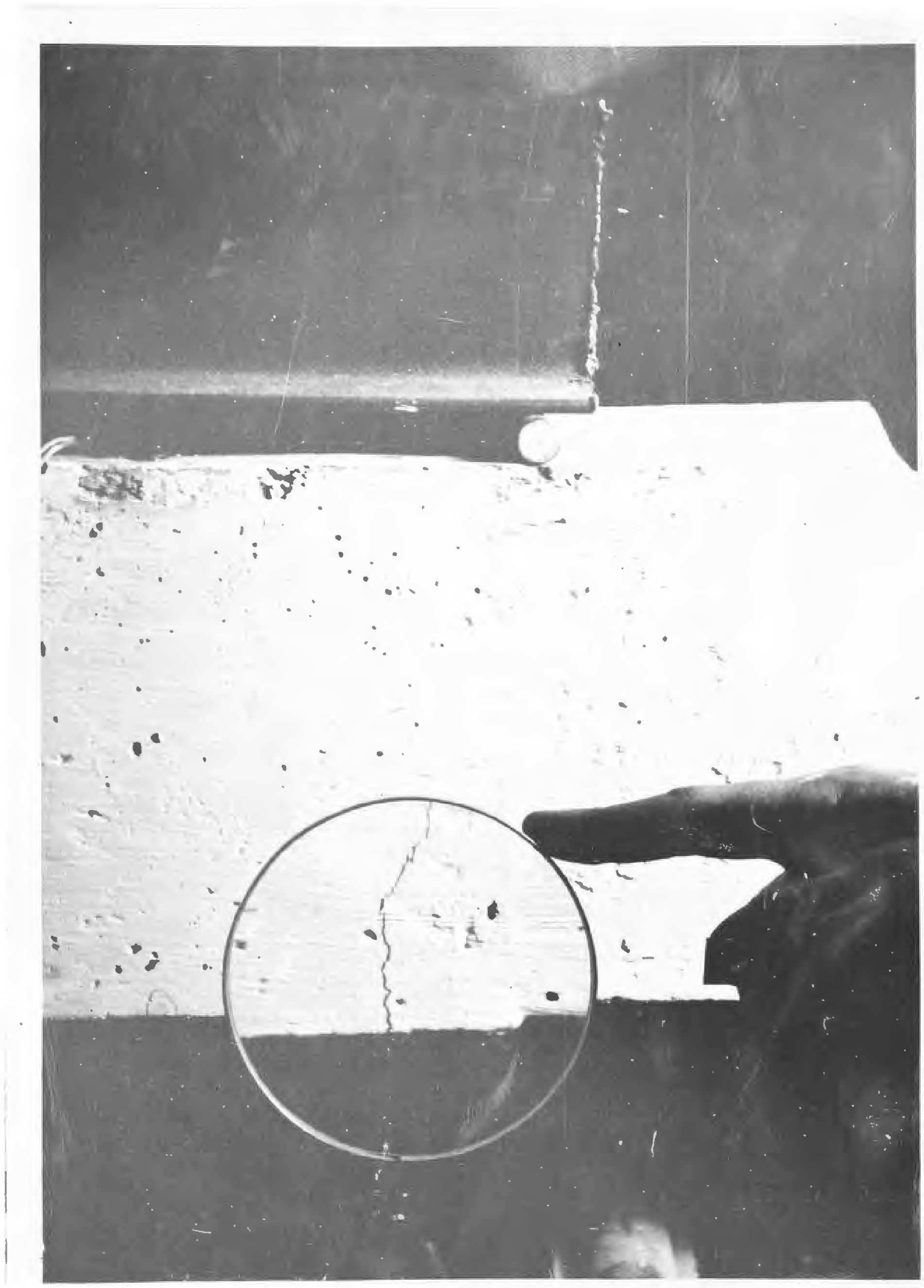


Fig. 33
View through magnifying glass, showing location of largest crack under 9000 pound load.

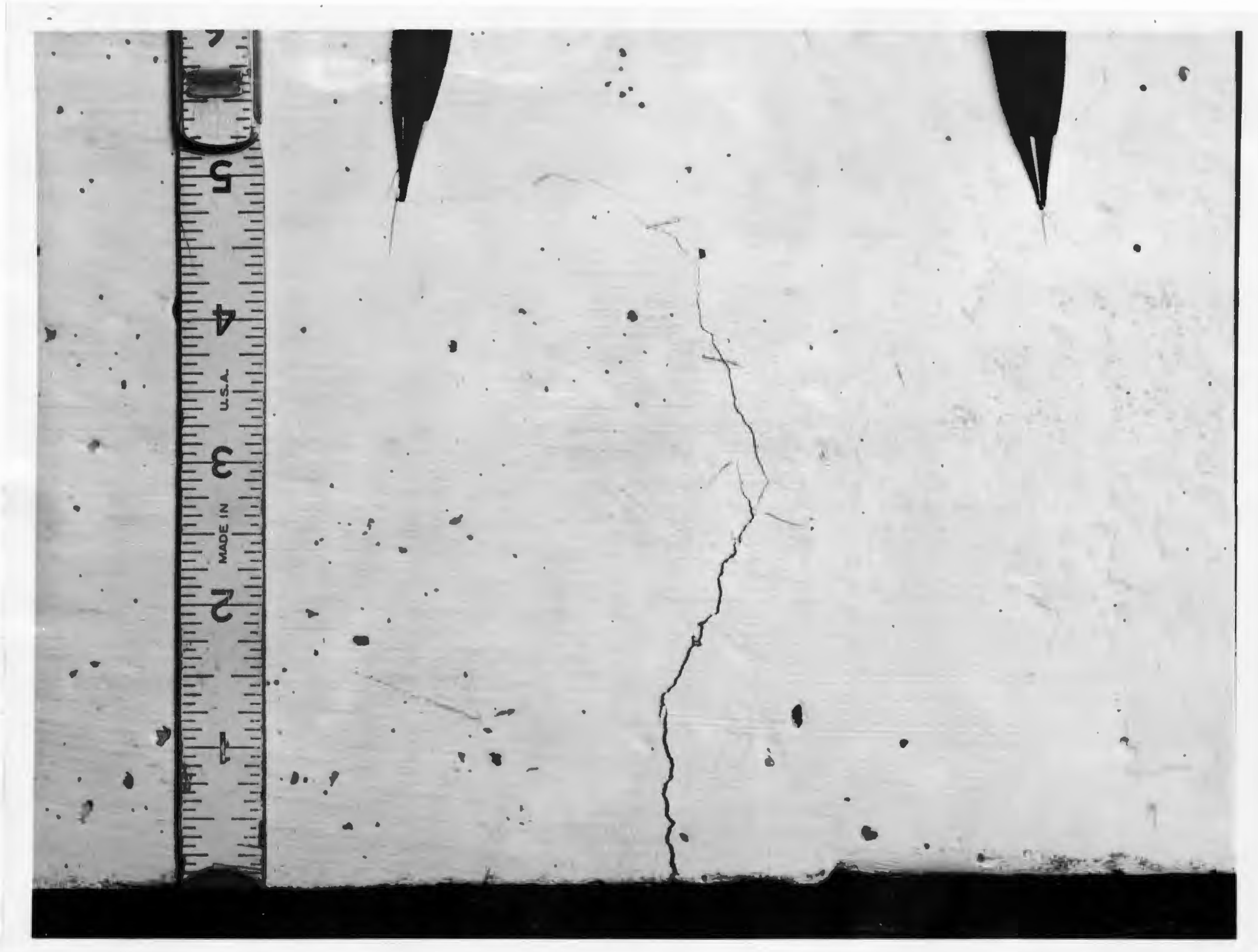


Fig. 34
Size and extent of largest crack opening under load of 9000 pounds.

Fig. 35
Same area as Fig. 34, showing closure of crack after removal of load. Note trace of crack in flaked-off whitewash.

Fig. 36
Beam under load after four of five wires had failed. Deflection approximately 5-in. Beam returned nearly to normal after removal of this load.

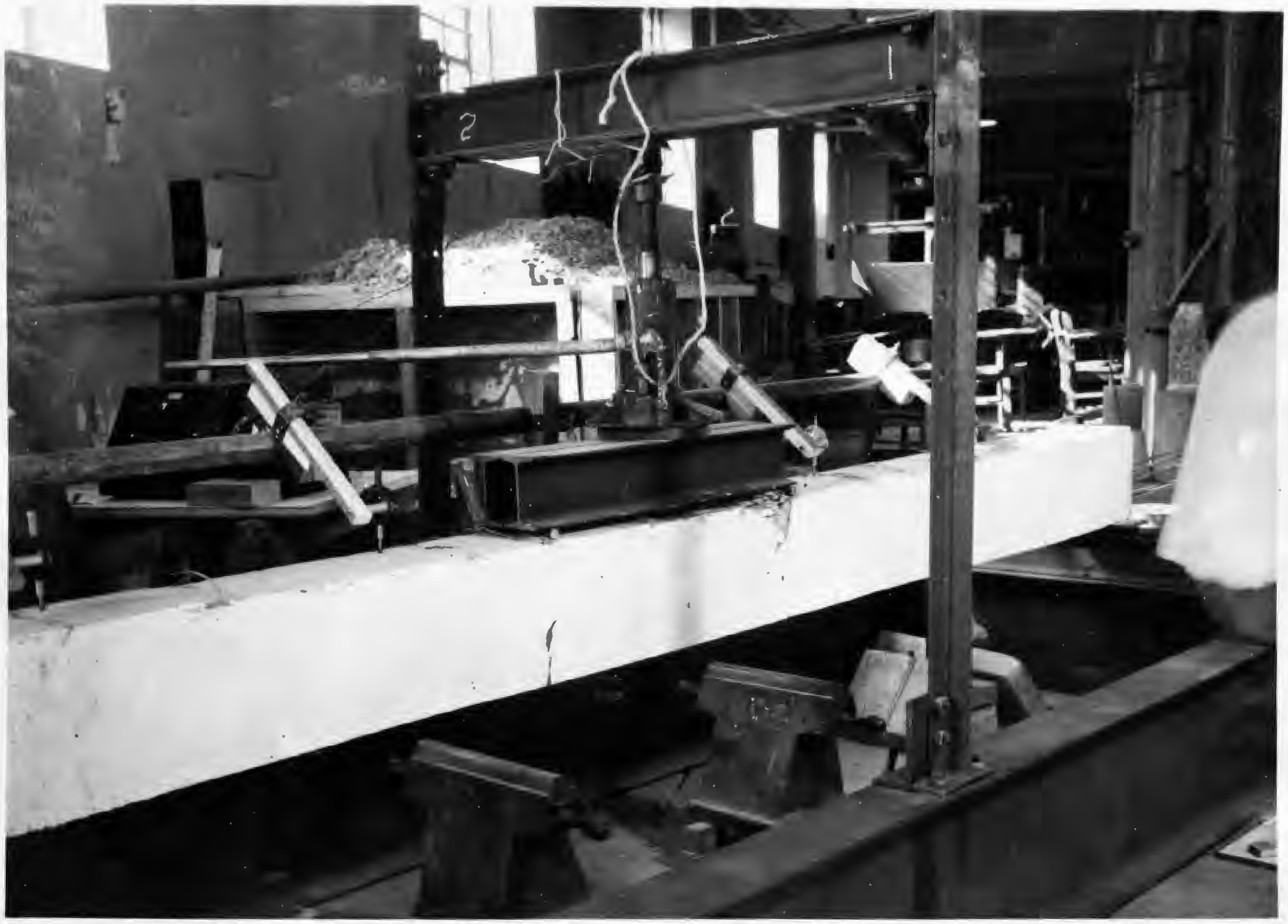


Fig. 37
Beam after completion of test and removal of load.

Fig. 38
Beam after removal of loading assembly, showing location and extent of crushing zone.

MEASUREMENTS TAKEN DURING FINAL STAGE OF TESTING

Wire and Concrete: Top numbers = Stress (psi.), Bottom numbers = Strain (micro in. per in.).
Deflection (in.) measured at center line.

		Load in kips															
		0.0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5
Wire	B	12860 3470	13,900 3484	13,920 3490	13,980 4509	14,010 4519	14,070 4539	14,120 4553	14,180 4572	14,220 4586	14,280 4605	14,320 4619	14,390 4640	14,450 4662	14,550 4693	14,660 4728	14,810 4778
	C	12,370 3990	12,470 4015	12,510 4035	12,590 4060	12,620 4072	12,700 4095	12,740 4110	12,820 4135	12,870 4150	12,920 4168	12,970 4184	13,050 4210	13,110 4230	13,210 4261	13,320 4298	13,480 4348
	D	6,990 2255	7,030 2268	7,050 2275	7,090 2288	7,130 2300	7,150 2308	7,190 2320	7,220 2330	7,250 2340	7,290 2350	7,330 2366	7,380 2380	7,430 2398	7,500 2419	7,590 2448	7,700 2485
Concrete	Bottom	-1,964 244	-1,693 210	-1,496 186	-1,239 154	- 951 118	- 752 93	- 457 57	- 242 30	- 16 2	143 18	333 41	476 59	554 69	565 70	581 72	543 67
	Top	-1,589 197	-1,684 209	-1,698 211	-1,753 218	-1,817 225	-1,910 237	-2,018 250	-2,179 270	-2,299 285	-2,515 312	-2,705 336	-2,966 368	-3,283 407	-3,643 452	-4,093 508	-4,633 575
Deflection		0.0000	0.0208	0.0355	0.0545	0.0820	0.1030	0.1245	0.1460	0.1625	0.1805	0.1990	0.2210	0.2430	0.2695	0.2995	0.3380

-Denotes Compression

		Load in kips															
		8.0	8.5	9.0	9.5	10.0	10.25	10.50	10.75	11.00	11.25	11.50					
Wire	B	14,980 4833	15,180 4895	15,410 4952	15,650 5046	15,950 5144	16,120 5199	16,360 5277	16,690 5382	16,920 5458	17,260 5567	17,880 5765					
	C	13,660 4403	13,860 4470	14,140 4560	14,380 4640	14,730 4750	14,940 4820	15,140 4885	15,510 5000	15,740 5078	16,060 4180	— —					
	D	7,860 2535	8,030 2590	8,250 2660	8,460 2730	8,260 2825	8,960 2890	9,150 2950	9,480 3058	9,700 3130	10,010 3230	10,760 3470					
Concrete	Bottom	511 63	511 63	403 50	309 38	296 37	256 32	236 29	277 34	271 33	265 32	215 27					
	Top	-5,272 654	-5,890 731	-6,686 830	-7,354 912	-8,080 1002	-8536 1059	-8897 1104	-9650 1197	-9903 1229	-10,720 1330	-11,270 1398					
Deflection		0.3845	0.4455	0.4975	0.5580	0.6345	0.6860	0.7095	0.7320	0.7620	0.7908	1.0625					

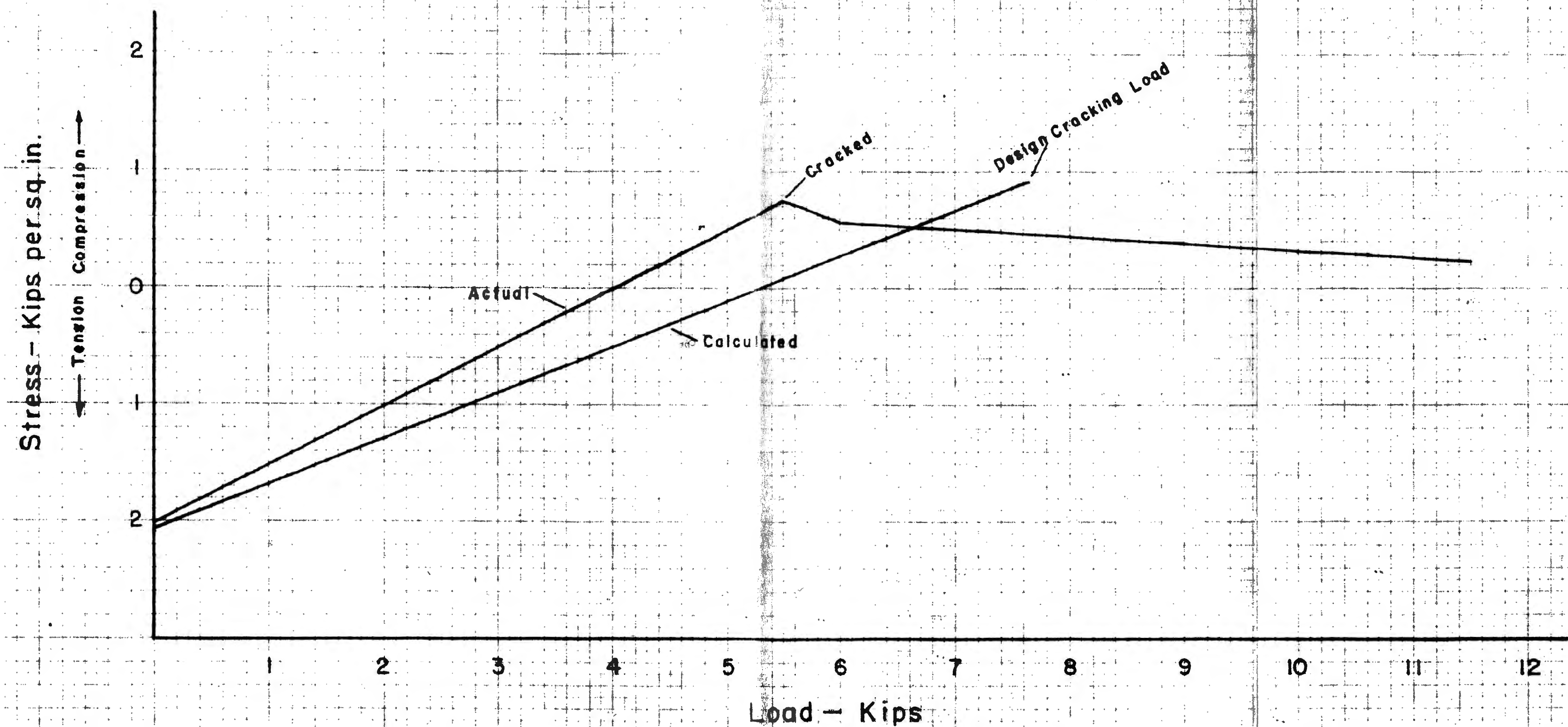


Fig. 39 Relationship Between Calculated and Actual Stresses in Bottom Fibers.

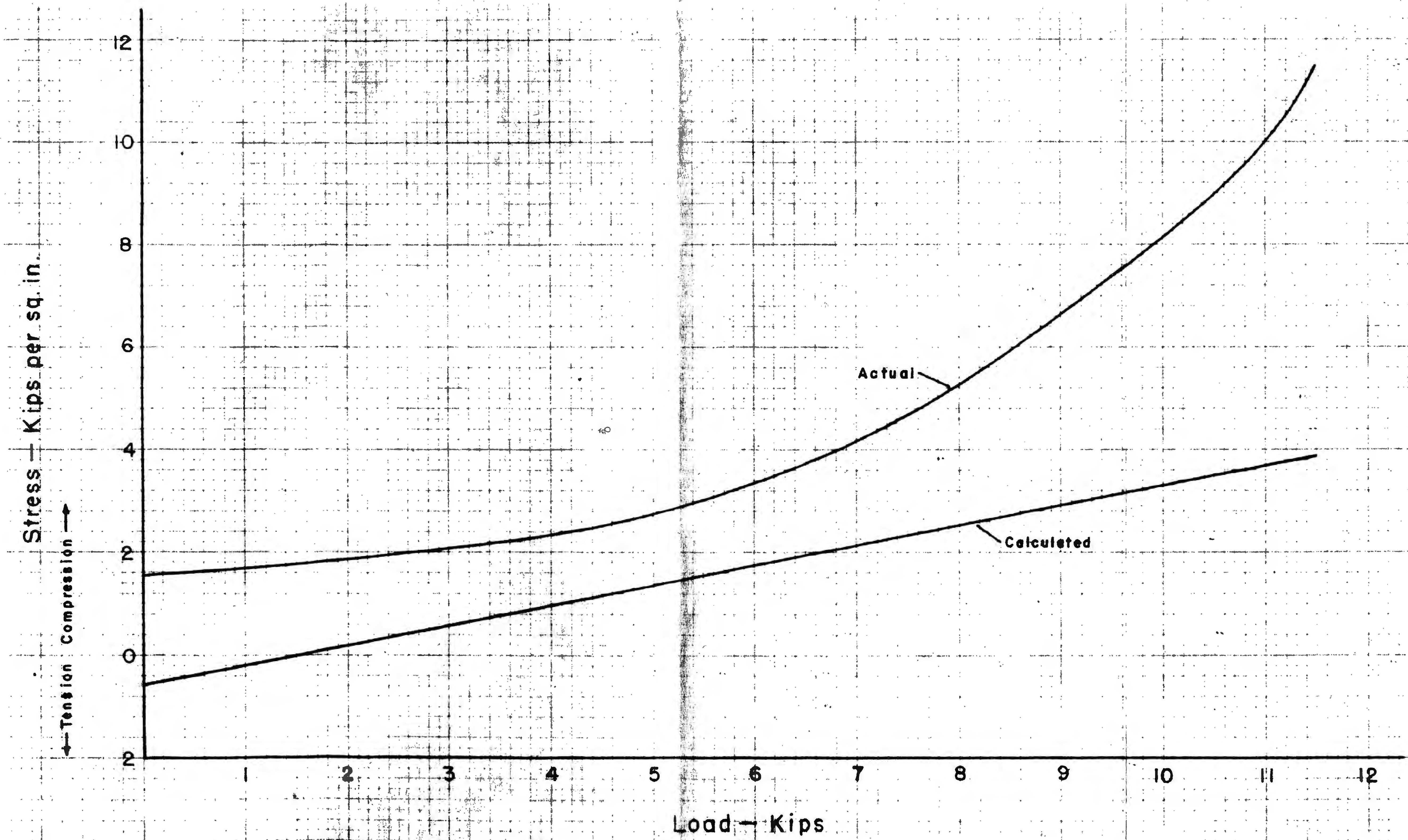
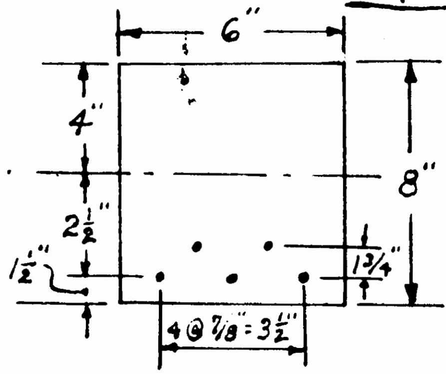


Fig.40 Relationship Between Calculated and Actual Stresses in Top Fibers.

Calculations



Span - 10'-0"
f_s (Initial) - 135,000 psi.
f_s (Final) - 120,000 psi.
5 wires @ 0.598 $\square''$ = 0.30 $\square''$
All wires have eccentricity of 2.5" at mid-span

$A = 48\ in^2$ $\quad S = 64\ in^3$ $\quad I = 256\ in^4$

Dead Load Moment - $\frac{50 \times (10)^2 \times 12}{8} = 7500''^{\#}$

Fiber Stress - $\frac{7500}{64} = \begin{matrix} -117 \text{ psi. Top} \\ +117 \text{ psi. Bottom} \end{matrix}$

Prestress Force

Initial - $0.30 \times 135{,}000 = 40{,}500^{\#}$
Final - $0.30 \times 120{,}000 = 36{,}000^{\#}$

Fiber Stress

Initial - $\frac{40{,}500}{48} = -845$ psi.

Final - $\frac{36{,}000}{48} = -750$ psi.

Moment Due to Eccentricity

Initial - $2.5 \times 40{,}500 = 101{,}000''^{\#}$
Final - $2.5 \times 36{,}000 = 90{,}000''^{\#}$

Fiber Stress

Initial - $\frac{101{,}000}{64} = \pm 1580$ psi. top / bottom

Final - $\frac{90{,}000}{64} = \pm 1405$ psi. top / bottom

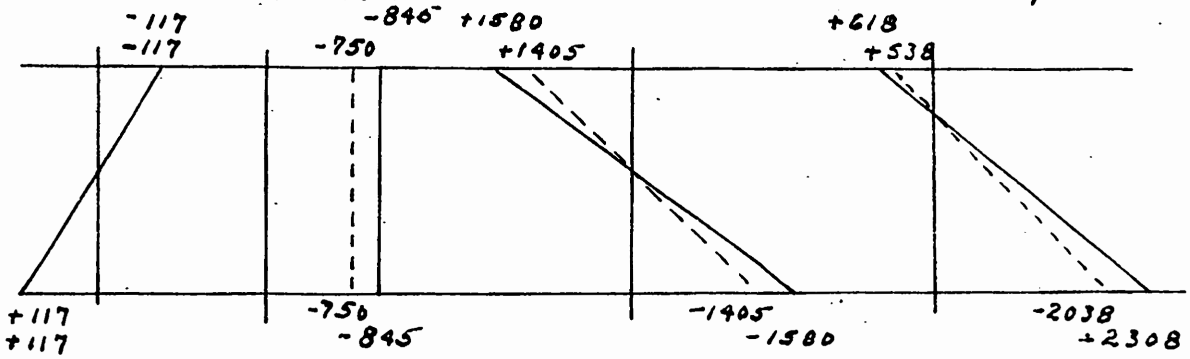
Total Fiber Stress

Initial: Top: $-117 - 845 + 1580 = +618$ psi.

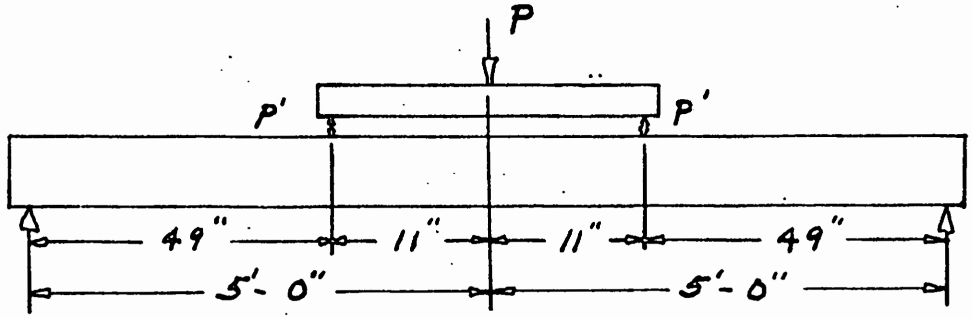
Bottom: $+117 - 845 - 1580 = -2308$ psi.

Final: Top: $-117 - 750 + 1405 = +538$ psi.

Bottom: $+117 - 750 - 1405 = -2038$ psi.



Stress Distributions



Loading Arrangement

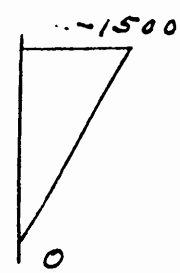
$f'_c = 6000$ psi. $f_c = 0.45 f'_c = 2400$ psi. $f_t = 0.15 f'_c = 900$ psi.

Design Load (Zero stress in bottom)

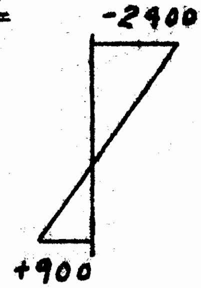
$M = 2038 \times 64 = 130{,}000''^{\#}$

$P' = \frac{130{,}000}{49} = 2650^{\#}$ $\quad P = 2P' = \underline{5300^{\#}}$

Equivalent uniform load $= \underline{433^{\#}/ft.}$



Load at Cracking ($f_t = 900$ psi. bottom)

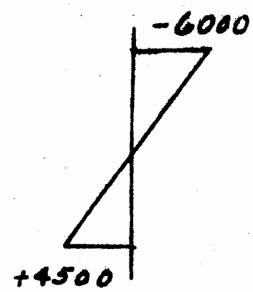


$M = (2038 + 900) \times 64 = 188{,}000''^{\#}$

$P' = \frac{188{,}000}{49} = 3830^{\#}$ $\quad P = 2P' = \underline{\underline{7660^{\#}}}$

Equivalent uniform load $= \underline{\underline{625^{\#}/ft.}}$

Load at Failure ($f_c' = 6000$ psi.)



$M = (6000 + 538) \times 64 = 418{,}000''^{\#}$

$P' = \frac{418{,}000}{49} = 8530^{\#}$ $\quad P = 2P' = \underline{\underline{17{,}060^{\#}}}$

Equivalent uniform load $= \underline{\underline{1390^{\#}/ft.}}$

CONCLUSIONS AND RECOMMENDATIONS

The test beam very clearly demonstrated the remarkable elastic properties which are inherent in prestressed concrete structures. The absence of cracks under design load and the closing of cracks caused by greater loads are also characteristic of a properly designed prestressed concrete beam.

It will be noted from the design calculations that cross-sectional areas of 0.3-sq. in. of reinforcing steel and 48-sq. in. of concrete were used in this beam to carry a design load of 5300 pounds on a 10-ft. span. Analysis of a conventional reinforced concrete beam for the same load and span will show that 0.83-sq. in. of steel and 91-sq. in. of concrete would be required. Thus prestressing in this one instance resulted in a saving of about 36 per cent in steel and about 53 per cent in concrete. This saving, while substantial in itself, could have been made still greater by using an I cross-section for the prestressed concrete beam. This type cross-section with its thin web is recognized as the best for prestressed design where shear is not important, but it cannot be used satisfactorily in conventional reinforced concrete design.

As a whole, the testing procedure and the results obtained were quite satisfactory. The instrumentation of the prestressed wires with the type A-7 SR-4 strain gages was very satisfactory; however, it is believed that more efficient methods of protecting the gages may be devised. Near the jacking end of a beam there is a great tendency for a gage to strip off due to the rather large elongation of the wire at that point. The author believes that by employing more newly developed methods of affixing strain gages, using epon resin as a cementing material, and by using somewhat smaller lead wires it would be possible to enclose the gages and the leads in the electrical loom completely, bringing all the lead wires out at the end. This would considerably reduce the time and labor required to waterproof the gages and would entirely prevent the gages from being stripped off, since they would all move inside the loom as the wire is lengthened under stress.

It is also recommended that a different type strain gage--possibly a type A-9 with a six inch gage length--be used to record concrete strains on the surface of a beam in place of the type A-7 with its one-quarter inch gage length. This short gage length tended to record stress concentrations rather than an average distribution of stress. The extremely high compressive stresses in the concrete shown in Fig. 40 indicate this. It is also recommended that when two or more gages are used to record the same type of strain in the same area on a concrete beam, they be connected in series, thereby giving an average strain reading for the area with only one setting of the strain indicator. This would also apply to strain gages used on test cylinders.

The prestressing unit developed for this project proved eminently successful for this type of end-anchorage. Only one change is suggested. If the pumping unit were of sufficient size, and if the proper type of quick cross-over valve could be installed in the hose lines at the pump, it would then be possible to operate both rams with the same pump. This would make the equipment as well as the prestressing operation much simpler.

The plug and collar system of end-anchorage used in this project could be modified in several ways. This unit failed before the concrete beam began to show any signs of crushing. Failure was due to the shearing off of the threads inside the plug. This threaded area must be sufficiently tough and hard to be able to bite into the prestressing wire which in itself is extremely hard. To assure this gripping the threads must be very sharp and preferably quite fine. Also, it is believed that a much thinner plug with a much flatter taper would have stronger gripping power.

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VITA

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He received his early education in Natchez, graduating from Natchez High School in June 1943. The same month he was inducted into the Army. His twenty-nine months of service included four months with an ASTP unit at Randolph-Macon College in Ashland, Virginia, and nine months of overseas service with an engineer combat battalion in the ETO. He was separated from the Army in November 1945.

In January 1946 he entered the Alabama Polytechnic Institute in Auburn, Alabama, where he received his B.S. in C.E. in March 1949. In June 1948 he was married to Miss Dorothy Till of Hammond, Louisiana.

Following graduation he worked as an architectural draftsman in Natchez until September 1949 when he accepted a position as Instructor of Civil Engineering at the Missouri School of Mines and Metallurgy at Rolla. At the same time he started work on an advanced degree in Civil Engineering.

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