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FIELD INVESTIGATION OF PRESTRESSED REINFORCED CONCRETE HIGHWAY BRIDGES

INSTRUMENTATION FOR LONG-TERM FIELD INVESTIGATIONS

by

R. J. REYNOLDS

W. L. GAMBLE

Issued as part of ~~Program~~ Progress Report No. 2
of The Field Investigation of Prestressed
Reinforced Concrete Highway Bridges
Project IHR-93
Illinois Cooperative Highway Research Program

Conducted by

THE STRUCTURAL RESEARCH LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
ENGINEERING EXPERIMENT STATION
UNIVERSITY OF ILLINOIS

in cooperation with

THE STATE OF ILLINOIS
DIVISION OF HIGHWAYS

and

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

UNIVERSITY OF ILLINOIS
URBANA, ILLINOIS
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1. INTRODUCTION

1.1 Introductory Remarks

The use of prestressed concrete in bridge construction in this country has prompted many investigators to conduct laboratory investigations of the various behavior characteristics of prestressed girders. While the laboratory investigations have been numerous and extensive, there have been relatively few long term field investigations.

For this reason the Department of Civil Engineering and the Engineering Experiment Station of the University of Illinois, in cooperation with the Illinois Division of Highways and the Bureau of Public Roads, undertook a field study of the behavior of prestressed concrete bridges.

This report is a survey of instrumentation which has been developed to study the behavior of bridge structures in the field.

At this time a field investigation of a four-span prestressed concrete bridge is underway. Future reports will discuss the details and results of the field investigation, including the instrumentation used. The recommendations included in this report reflect some of the experiences of the initial phases of the field investigation.

The work reported herein covers phase 1 (a), Development of Instrumentation, of the project's work schedule.

1.2 Object and Scope

A study was undertaken to develop an instrumentation program applicable to a field investigation of prestressed concrete bridge. This study was designed to be directly applicable to a multi-span bridge in which each span consists of several pretensioned, precast I-section girders. The deck is composite with the girders, and is reinforced to provide live-load continuity. Instrumentation was either sought or developed to measure those parameters which would best describe the long time behavior of a prestressed concrete structure. The evaluation of short time dynamic effects were not included in this study; however, the instrumentation described is useful for measuring the effects of static live loads.

The parameters to be measured are as follows: (a) stress and strain in the prestressing steel, (b) strains in the concrete as indicated by internal and external gages, (c) temperature gradients across beam sections, (d) deflections of the structure, and (e) relative humidity in the concrete.

The criteria employed in the selection of the various instruments were that they should: (a) be sturdy in order that they would withstand common fabrication and handling procedures, (b) be stable over long periods of time, (c) be temperature compensated from 0 F to 135 F, (d) be waterproof, (e) have sufficient sensitivity, and (f) be as uncomplicated and inexpensive as possible.

Information on the relative merits of alternate methods of measuring the above parameters is presented and discussed, and suggested methods are described.

Deflection measuring devices are discussed in Chapter 2, gages for determining strains in the concrete are described in Chapter 3, and temperature measurements are discussed in Chapter 4. The determination of the forces and strains in the prestressing steel is discussed in Chapter 5 and gages for measuring relative humidity in the concrete are described in Chapter 6.

A beam was cast and tested in the laboratory in order to investigate the behavior of several of the instruments. The experiments are described in Chapter 7. Chapter 8 contains the summary and conclusions.

1.3 Acknowledgements

This study was carried out as a part of the research under the Illinois Cooperative Highway Research Program Project IHR-93, "Field Investigation of Prestressed Reinforced Concrete Highway Bridges." The work on the project was conducted by the Department of Civil Engineering, University of Illinois, in cooperation with the Division of Highways, State of Illinois, and the Bureau of Public Roads, U.S. Department of Transportation.

At the University, the work covered by this report was carried out under the general administrative supervision of W. L. Everitt, Dean of the College of Engineering, Ross J. Martin, Director of the Engineering Experiment Station, N. M. Newmark, Head of the Department of Civil Engineering, and Ellis Danner, Director of the Illinois Cooperative Highway Research Program and Professor of Highway Engineering.

At the Division of Highways of the State of Illinois, the work was under the administrative direction of Virden E. Staff, Chief Highway Engineer, and J. E. Burke, Engineer of Research and Development.

The program of investigation has been guided by a Project Advisory committee consisting of the following:

Representing the Illinois Division of Highways:

J. E. Burke, Engineer of Research and Development

Floyd K. Jacobsen, Bureau of Research

C. E. Thunman, Jr., Engineer of Bridge and Traffic Structures, Bureau of Design, Representing the University of Illinois:

Narbey Khachaturian, Professor of Civil Engineering,

C. P. Siess, Professor of Civil Engineering.

Acknowledgment is due to Mr. W. J. Mackay, Illinois Division of Highways, who contributed materially to the guidance and progress of the program.

The investigation is directed by Dr. M. A. Sozen, Professor of Civil Engineering as Project Supervisor. Immediate supervision of the investigation is provided by Dr. W. L. Gamble, Assistant Professor of Civil Engineering, as Project Investigator.

2. DEFLECTION MEASURING DEVICES

2.1 General

For a composite girder-slab bridge the deflected shape is one of the most sought after parameters. In particular, when dealing with prestressed girders, the camber may have an important role in the fabrication of the final composite structure.

The following discussion will deal with the various means of measuring the mid-span deflection of a simply-supported beam from a straight line extending from one end support to the other. In actual practice the bearing length at each support of a prestressed girder may be on the order of two percent of the total length of the girder. It will be assumed throughout this discussion that the span length extends center-to-center of the bearings.

It was decided that the instruments should be sufficiently accurate to measure deflections to the nearest 0.001 ft. To obtain this accuracy, the following systems, among others, are possible: (a) sighting with a surveying instrument, (b) a truss system with either mechanical dial gages or electrical gages, (c) a hydraulic "water level" system using graduated pipettes connected in series and fitted with a colored fluid, and (d) by measuring the deviation from a straight line established along a beam by means of a tensioned wire.

2.2 The Surveyor's Level Technique

The use of a surveyor's level is probably the most practical and straight forward of the various techniques suggested, Fig. 1. This is partly due to the fact that the instrument is not attached to the beam and thus does not undergo the transportation and construction hazards to which the other systems would be subjected.

The deflection at mid-span of a beam is measured by comparing the level rod reading at the center of the span with the average of the readings at the two ends as in the following equation:

$$\delta_t = \frac{R_r + R_\ell}{2} - R_m - \Delta_i \qquad (2.1)$$

where R_r = level reading at the right end of the beam,

R_ℓ = level reading at the left end of the beam,

R_m = level reading at mid-span,

Δ_i = deflection before release of prestress, corresponding to initial non-linearities of the beam, and

δ_t = deflection at the center of the beam at the time t.

One scheme using a surveyor's level to obtain the required precision is common to both the Structural Research Laboratory at the University of Illinois and the Portland Cement Association (1)*. This scheme uses a Wild N-3 precise surveyor's level rod. The level contains an optical vernier system that allows readings to be made to the nearest 0.0005 ft. The line of sight can be displaced, internally, through a distance of 0.02 ft by tilting a thick glass plate located in front of the objective lens. The principle is shown schematically in Fig. 2.

A light-weight level rod was made of 1-in. square aluminum tubing and was made in two easily separated 6-ft sections. A polished ball was used as the base of the level rod. The scale was two ft long, was made of laminated plastic, and had divisions of 0.02 ft each machined onto it. The scale could be attached to the metal rod with two bolts at any six-in. increment. The scale is shown in Fig. 3.

* Numbers in parentheses refer to references in the bibliography.

A center-bubble rod-level was used to insure that the rod was held vertical.

To insure that deflections are always taken at the same points on a beam, it is necessary to establish permanent reference points. This can be done by casting metal plates 1 in. square by 1/8 in. thick into the beam surfaces. The plates may be welded to 2 in. lengths of No. 3 deformed reinforcing bar which will serve as anchors.

Since a bridge girder will eventually be covered with an overlying slab, it is necessary to cast the deflection plates into both the top and bottom surfaces of the beam so that deflections can be measured from either above or below. It was found that the bottom plates could be located in the beam by gluing them to the pallet of the casting bed with a liquid resin or white glue such as Elmer's Glueall. The glue held the deflection plates in position during the casting operation but did not offer any difficulties during the form stripping operation. The top plates can be placed in the fresh concrete after the last finishing operation or can be set in holes drilled in the hardened concrete.

Setting the plates accurately in the top surface of the beams before the concrete hardenes is relatively easy. However, proper positioning in the fresh concrete of the bridge deck is difficult and setting the plates in holes drilled after the concrete hardened allows much better precision in locating the plates.

While every effort should be taken to insure that the deflection plate is level, a standard procedure of positioning the steel ball in the base of the level rod at the center of the deflection plate whenever a reading is being taken should be adopted. This will minimize any error due to a tilted plate. Seats can be drilled into the plates to help position the end of the level rod.

2.3 The Truss System

The idea of establishing a reference line along a beam from which deflections could be measured has merit but there are several drawbacks.

A truss or beam spanning between adjacent piers, Fig. 4, would form a convenient reference point for a deflection gage under the test structure. Unfortunately, the reference can easily be established only after the girder is in place on the piers. It does not seem possible to develop a practical truss scheme for measuring deflections due to release of the prestressing force and at other times prior to final positioning of the girders. A more serious problem, however, is that the truss would have to be installed below the bridge, using up some of the valuable headroom over the roadway below. If there were no diaphrams between the girders, the truss could be placed between adjacent girders.

The truss could be a steel bar joist. The lightest joist capable of spanning 70 ft would weigh at least 500 lbs, which is too heavy to handle conveniently by hand. An aluminum truss light enough to handle could be used, but it would be relatively flexible.

The truss system, or just a post, Fig. 5, could be considered for use with a bridge crossing a stream if headroom is not a problem. If such is the case, several deflection measuring devices could be considered. They are: (a) a mechanical dial gage, (b) a linear-variable differential-transformer, or LVDT, and (c) a slide wire. With deflection plates mounted in the beam the dial gage would seem to be the most practical solution. It would be necessary to secure the gage to the truss in such a manner that it would not be affected by adverse weather conditions or vibrations of the bridge.

While the LVDT has the desired short-term accuracy, it and the associated electronic equipment usually behave erratically over long periods of time and under varying weather conditions.

An electrical-resistance slide-wire device could be mounted on the truss, with the slider connected to the bridge. Again, the precision attainable is adequate, but the long term stability is probably not.

The mechanical gage has sufficient stability but it is difficult to read remotely.

2.4 The Water-Level Technique

A hydraulic or water level system has been used to measure deflections in some European work (2). Essentially, the system consists of three vertical graduated tubes (for a single beam span) connected by a horizontal tube, as shown in Fig. 6. A liquid is placed in the system, and the levels in the graduated tubes are noted. After the structure has deflected, the level of liquid in the three tubes is again recorded, and the deflection is found from the differences, using Eq, 2.1 as was done for level readings.

Such a system was tested in the laboratory. The readings were found to be reliable to within about 0.05 in. There were problems of the meniscus in the relatively small tubes used, and slightly more precise readings could be obtained if larger tubes and point gages to locate the liquid surface were used. Barges and Marecos (2) reported sensitivity to 1 mm (0.04 in.) in a span of a few hundred feet. Their system was not well described, unfortunately.

The system is flexible, and could be used through the entire life of the girder, but would always be susceptible to physical damage. The particular system used in the laboratory is described in Sec. 7, but a more rugged system would have to be used in the field. The laboratory system was connected with plastic tubing; copper tubing could be used in the field.

2.5 The Taut-Wire Technique

A simple means of measuring the deflection of a beam is to establish a reference line with a tensioned wire and note the deviation from the lines to a previously marked point, as sketched in Fig. 7. With some refinements it may be possible to obtain an accuracy of 0.01 to 0.02 in. The refinements would consider (a) the diameter of the wire, (b) method of application of a constant tensioning force, (c) the scale used in measuring the deviation, and (d) the means of sighting the scale.

3. STRAIN MEASURING DEVICES

3.1 General Remarks

In an investigation of the long-term behavior of prestressed concrete structures, information on the changes in local strains and on the changes in strain gradients are necessary. A variety of instruments have been used to make these measurements.

Electrical resistance strain gages are widely used for short term measurements, and some types are stable enough for long-term use. All of these gages utilize the physical phenomenon that stretching an electrical conductor increases its resistance. The resistance changes are generally measured by means of a Wheatstone bridge circuit, and the changes are related to strain by the proper calibration constants. The most successful gages for long-term usage incorporate at least two arms of the Wheatstone bridge into the gage itself.

Vibrating-wire strain gages, operating on the basis of the principle that changing the length of a tensioned wire changes the natural frequency of vibration of the wire, have been developed for structural applications. Such gages have satisfactory long-term stability.

Mechanical strain gages, using dial indicators to mechanically measure movements between reference points attached to the structure, are widely used. When adequate temperature compensation procedures are followed, suitable long-term stability can be achieved. Many kinds of mechanical strain gages have been used, and two of these will be described later. Optical devices for measuring the movements have been used (10).

Optical strain gages, which indicate strains by the presence of diffraction and interference bands, are available.

References 3 through 15 contain information on the various types of strain gages.

Some of the electrical resistance gages and vibrating wire gages are suitable for embedment in concrete structures. The mechanical gages and some of the vibrating wire gages are suitable for use in measuring surface strains.

The performance requirements for embedded gages are relatively severe. The gages must be sturdy so that vibration during concrete placement will not damage them. Further, the gage must be stable over long periods of time and under temperature variations of 0° F to at least 140° F. The high upper bound on the temperature is necessary because prestressed girders are often steam cured. The strain range should be on the order of 2500×10^{-6} in compression and 500×10^{-6} in tension. The sensitivity should be on the order of $5 - 10 \times 10^{-6}$. The instrument readings should be either temperature compensated or capable of simple compensation. Monitoring of the gage should be possible from remote locations without serious deterimental effects due to additional lead-wire length.

3.2 Internal Strain Gages

3.2.1 Bonded Wire Strain Gages

Bonded-wire strain gages, such as the well known SR-4 gages, have the measuring element attached directly (with proper electrical insulation) to the material in which strain is to be measured. The gage element then lengthens or shortens with the material to which it is attached.

Several techniques can be used to measure internal strains in concrete with such gages. If reinforcing steel is present, the gage can be attached to the steel and waterproofed before the concrete is cast.

If no steel is present, other techniques must be used. Baldwin-Lima-Hamilton manufactures a gage known as the Valore gage. This is a SR-4 resistance gage which is mounted on a piece of brass shim stock. The brass is then folded around the gage and soldered shut, after provision for the lead wires has been made. The gage is embedded directly in the concete.

Another possible technique is to embed a short length of small reinforcing bar in the concrete and measure the strains in the bar. Such bars, sometimes called "pencil bars" may be instrumented as complete four-arm bridge circuits using ordinary SR-4 strain gages. Development work on these gages has been done by the Ohio River Division Laboratories of the Corps of Engineers.

Long term stability of the gages and of the electrical measuring circuits is generally a problem with the SR-4 type of bonded wire gage. However, if short-term transient strain measurements at various time intervals are required, the SR-4 gages can often be used satisfactorily, even in difficult environments.

3.2.2 Carlson Strain Meter

The Carlson strain meter (4,5) is an unbonded-wire gage, in that the strained resistance element is not attached continuously to the strained material. The gage is set with an initial tension in the elements, and the linear range is limited by the strain at which the element goes slack or yields. Consequently, the gage must be preset to give the maximum strain range in the proper direction.

The Carlson electric wire strain meter is a device which is made to be embedded in concrete to measure the internal deformations (4,5). It responds to changes in dimension and in the temperature of the concrete. The Carlson meter was originally developed for embedment in mass concrete in dams, but has also been used in a few structural applications.

The meter consists of two fine tensioned-wire resistance-coils connected to themeter body in such a manner that when the concrete contracts the tension in one of the coils is reduced while the tension in the other is increased. Figures 8 is a photograph of three strain meters, one complete and two partly assembled. The two coils make up two arms of a four-arm Wheatstone bridge. The Carlson Test set consists of one fixed and one variable resistance, and makes up the remaining two arms of the four-arm bridge circuit. Readings from the meter are taken by null-balancing the Wheatstone bridge as indicated by the internal galvanometer in the test set. Power for the circuit is provided by two 1.5 volt batteries.

The two resistance coils are encapsulated in a brass sheath and the enclosure is filled with oil and nitrogen to prevent corrosion and moisture damage. The corrugated section of this cover permits the meter to deform easily. The outside cover which breaks the bend between the meter body and the concrete is made up of a fibrous-cotton material that is secured to tbe brass sheath by adhesive tape. The meter is designed so that the concrete will produce strains in the gage due to bearing action against the metal lugs at the ends.

Each meter is connected by means of 3-conductor, rubber covered cable. Corrections in the calibration constants for long lead wires must be made; extra lead resistance reduces the sensitivity slightly.

The following are the manufacturer's specifications for the standard SA-10 strain meter:

Range in expansion, millionths (minimum)	400
Range in contraction, millionths (minimum)	800
Least Reading, strain, millionths (approx.)	3.8
Least reading, temperature	0.1°F
Gage Length	10 in.
Flange diameter	1.25 in.
Body diameter	1.15 in.
Weight	1.16 lb.
Price (1967)	41

The average strain ranges available within the linear range of the gage are about twice the minimum ranges shown, according to the manufacturer. The meter can be preset, during manufacturing, to give a maximum range in one direction and only a small range in the other, if desired.

For the present field investigation, modified gages with gage lengths of 7.75 in. were obtained. This was done to obtain about 20 percent greater range. In addition, the gages were preset to give the maximum possible range in compression and only a small range in tension.

Seven Carlson strain meters were used in one bridge structure during this investigation. A reinforced concrete beam containing one meter was made and tested in the laboratory to investigate the behavior of the instrument. This test is described in Chapter 7.

3.2.3 Internal Vibrating Wire Gages

The vibrating wire gage, also known as the acoustic or sonic gage, has been used extensively in England and Europe (3, 6, 18, 19). The long-time stability and sensitivity have been reported to be very satisfactory. A research program on prestressed concrete pressure vessels conducted by the General Atomic Division of General Dynamics has recently made use of over 150 vibrating wire gages and their performance has been found acceptable (16)

The measuring element in the gage is a tensioned wire fixed between two points a few inches apart in a structure. The wire is "plucked" magnetically and its frequency of vibration is determined. Changes in strain in the structure change the tension in the wire, and consequently alter the frequency. The changes in frequency may be related to the changes in strain through calibration data.

Vibrating wire gages are manufactured by Deakin Phillips Electronics Ltd., Middlesex, England, the Maihak firm of Germany, and Perivale Controls Co. Ltd., London. The gage sensitivity is comparable to the Carlson meter but the gages do not have means of measuring the temperature of the concrete. Phillips states that the temperature coefficient for their gage is approximately that of the concrete. The vibrating wire gages cost \$25 to \$55 . The support equipment, such as the Deakin Phillips Strain and Temperature Measuring Unit or the Maihak Receiver, cost from \$500 to \$2,000.

As internal gages, both the vibrating wire gage and the Carlson strain meter have shown comparable stability characteristics over a long time. Satisfactory performance of vibrating wire gages in use for over 10 years has been reported (18). Some Carlson Strain meters which were placed prior to 1940 can still be read (17). A major drawback is their susceptibility to damage due to concrete placement.

When either gage has been secured in a reinforcement cage without a protective covering other than the gage covering itself, lose due to concrete placement of up to one-third of the gages has been encountered. In one case 30 out of 150 vibrating wire gages were lost due to internal vibration during concrete placement (16). In the present field investigation, 2 of 7 Carlson meters were lost.

In resolving the problem of damage to internal strain gages due to concrete placement, suggestions have been made that the gage should first be cast in a small concrete briquette and that the briquette be cast into the test specimen (3). While the basic idea is reasonable, there are problems that must be met. The briquette must be cast of the same concrete mix as the specimen and cast within 1-2 days of the test specimen casting. In a prestressed beam that has strands on a small grid pattern, the maximum size of the briquette is limited. Lastly, the homogeneity of the briquette with the test specimen must be verified. With the exception of the size limitations these problems have been discussed in the literature (3, 19) relative to vibrating wire internal strain gages, and apparently have been resolved.

3.3 External Strain Gages

3.3.1 Electrical Strain Gages

To collaborate and supplement the results of the internal strain gages, external gages may be employed. Either electrical gages such as the SR-4 gage or vibrating wire gages, or mechanical gages are often considered.

The SR-4 gage is not often used because of its relatively unstable nature over long periods and its sensitivity to extreme temperature and moisture conditions.

The vibrating wire gage shows considerable promise as an external gage. Work in this country by General Atomic is currently under way to develope such a gage. Preliminary results have indicated a satisfactory performance over long times. There are problems involved in attaching the gage to concrete bridge girders and protecting it during construction, but successful applications have been made, and development work is continuing. Earlier work was carried out in England (6).

3.3.2 Mechanical Strain Gages

3.3.2.1 Whittemore and Munich Mechanical Strain Gages

The Whittemore and Munich mechanical gages which have been used in the field investigation are shown in Fig. 9. The gages have gage lengths of 10 in. and 50 cm, respectively. Both gages are direct reading gages, without mechanical multiplication of movement of the points on the gages.

The Whittemore gage has a 10 in. gage length and the least reading on the dial is 0.0001 in. The minimum strain reading is 0.000,010. The maximum travel is 0.10 in., giving a strain range of 0.01. The readings can usually be repeated to within 0.0001 in., once the perator has acquired some experience.

The Munich gage, developed at the Technical University of Munich, has a 50 cm gage length and the least reading on the dial is 0.001 mm. The corresponding minimum strain reading is 0.000,002. The maximum travel is 4 mm and the strain range is 0.008. The readings can usually be repeated to within about 0.003 mm.

Mechanical gages using optical measuring systems have been used (10). There are several problems with mechanical strain gages which have had to be resolved. The primary questions concern the following:

(a) the form of gage points and reference points to be used,

(b) the manner of attaching the reference points to the structure,

(c) the type of standard bar to be used for temperature compensation, and

(d) the dependence of the reliability and repeatability of the strain readings on the operator's experience with the gage.

The gage point is the metal tip of the strain gage and the reference point is a metal piece attached to the structure. The reference point generally has a small hole drilled in it, and the point of the gage fits into the hole. The change in distance between adjacent gage points is the average strain times the gage length. The form of the gage and reference points affects the reliability and repeatability of the readings.

a) <u>Gage Points</u>

The gage point may take the form of a conical point, spherical ball, or a spherical seat as shown in Fig. 10 (a). To match the conical point a cylindrical hole in the reference point is generally employed, a conical hole is used with the Brittish "Demec" gage.

To match the spherical ball, a conical hole, a spherical seat or a cylindrical hole may be used. To match the spherical seat a spherical ball attached to the gage plug would be used as shown in Fig. 10 (b).

A disadvantage of the conical or spherical point in a cylindrical hole is that the edges of the hole have a tendency to wear, and in fact must be worn slightly before reproducible readings can be taken. The spherical point is larger than the cylindrical hole and bears on the edge of the hole. The conical point is sensitive to slight tipping of the gage, while the spherical point is much less sensitive.

The spherical ball in a spherical seat would be an acceptable solution since the wearing problem is greatly reduced except for the fact it has proven to be very difficult to match the sizes of the spherical ball and the spherical seat to the tolerances necessary.

Several of the various methods have been used in the laboratory. The use of spherical points on the strain gage and conical holes in the reference points which are attached to the structure has given the most satisfactory results, and has been adopted for the field study.

b) Attachment of Reference Points to Structure

Once the geometry of the plug hole was determined, the material for the reference point and the means of attaching the point to the beam must be determined. For a field investigation, stainless steel is preferred. The point can either be glued to the surface of the beam or it can be grouted or in some manner cast into the beam. The common laboratory procedures have been to use Eastman 910, a high strength fast setting glue, to attach gage reference points to the surface or members.

Unfortunately, all of the glues used to date have the disadvantage of being sensitive to high humidity and even with the waterproofing procedures recommended the glues will deteriorate in time. Even the most careful waterproofing techniques have not proven sufficient to insure that a point will remain in position on a concrete surface if the specimen is outdoors. A point that protrudes from the surface of the beam is also subject to handling and construction hazards. It was found necessary to install the reference points in the beam rather than on the surface.

There are at least two methods of installing a reference point in the beam: (a) it may be cast into the beam, or (b) a hole may be drilled in the beam after the form has been stripped, and the reference point can be anchored mechanically and/or grouted into the hole. Casting either the gage plug or an insert into which the point is later screwed has the advantage of insuring good bond between the concrete and plug. Unfortunately, a workable scheme for casting the inserts in the proper places requires drilling holes in the formwork and bolting the insert to the form before the concrete is cast. Drilling holes in steel forms in a precasting plant is usually impossible, but this can be done where expendable wooden forms are being used.

A 1-in. long piece of No. 4 deformed bar which has a 1/4 in. hole drilled and tapped through most of its length makes a convenient anchor when 1/4-in. diameter cap screws are used for the gage reference points. The bar, plus a beveled washer which forms a countersink for the head of the cap screw, is bolted to the inside of the form as shown in Fig. 11a. After the concrete has set, the bolt is removed. After the forms are removed the beveled washer is removed and the cap screw forming the final gage point is installed. A stud locking solution should be used with this cap screw. The final position is as shown in Fig. 11b.

In a current investigation, the points were installed in holes drilled in the beam after the forms were removed. The method used consisted of drilling a 7/16-in. diameter hole 1-1/4-in. into the beam and then countersinking with a 1/2-in. diameter drill. A "Red-Head" mechanical anchor was then driven into the hole. This anchor has a 1/4-in. diamter hole 3/8-in. tapped deep. This permitted a 1/4-in. cap screw to be inserted in the anchor. It was originally planned to secure the cap screw with Loctite Stud Lock, but the plaster grout which was put in the holes before the anchors were set eliminated the need for it. The cap screws were 1/4-in. diameter by 3/8-in. long stainless steel. The depth of the predrilled hole was such that the top of the cap screw would be flush with the surface of the concrete. The final installation looks about like that shown in Fig. 11b.

The first experiences with the mechanical anchors demonstrated that the installation of the anchors and bolts took too much time, although the results were generally acceptable. Drilling the 7/16 in. holes took more time that was readily available.

An alternative simpler, scheme of setting the cap screws was then developed. A 1/4-in. diameter hole was drilled to about one-in. depth and cleaned out. Then a quick-hardening epoxy paste (Concressive No. 1201) was ejected into the hole, and a 1/4-in. diameter by 3/4-in. long stainless-steel cap screw was pushed into the epoxy in the hole. The epoxy hardened within 15 to 20 minutes. The cap screw head protrudes slightly from the surface of the concrete, but this has not yet proved to be a serious problem.

The epoxy used was a thick paste material. The two components were mixed and then placed in a plastic-lined cloth pastry bag which was equipped with a metal nozzle. The epoxy was forced into the hole by squeezing the pastry bag in the same way a baker uses the bag to decorate a cake. After the cap screw was installed, a conical gage hole was drilled with a No. 1 center drill into the top surface of the cap screw. The drill used for both the masonry drilling and the gage hole drilling was a two-speed hammer-drill. For the masonry drilling, it was geared in the hammer-drill position at 1250 rpm, and for the steel drilling it was in the drill positon at 750 rpm.

There were two major difficulties encountered in positioning gage points in this manner: (a) since the Whittemore gage has only 1/10-in. of travel, the tolerance on positioning the masonry hole was tight, and (b) despite the fact that cutting oil was used in drilling the conical holes, numerous center-drill bits were broken due to the hardness of the stainless steel, overheating of the drill bit, and the eccentricities encountered when hand drilling was employed. Because of the shape of a No. 1 center drill, it was necessary to drill the hole about 1/8-in. deep, i.e., to a depth such that the full conical shape is formed without obtaining a cylindrical shaft, as is shown in Fig. 12. The drill bit generally broke along line A (Fig. 12) and it was impossible to finish or redrill the same hole.

Drilling a hole in concrete to within 1/10-in. of the desired positon is difficult, but when this problem was compounded by having to drill more than one hole in the bolt head, the results were frequently unsatisfactory. Particular difficulty has been encountered where a square grid pattern of gage lines was being developed in the anchorage zones of a beam. Positioning a gage hole at a location where two vertical and two horizontal gages lines tied in was found to be not practically possible with this technique.

Stainless steel cap screws were used in the first phase of the field investigation. However, a few ordinary mild steel cap screws were eventually used. After a year, it can be seen that the stainless steel is a superior material, in that only minor rust development has occurred. The few mild steel bolts have rusted badly, in spite of the presence of light machine oil. The stainless steel is much harder to drill, but the results have shown its use to be necessary.

c) Standard Bars

The standard bar has the functions of providing a reference gage line for checking the strain gage itself and providing temperature compensation. The two most common standards for the mechanical strain gages used are Invar, a steel alloy which has a very low thermal coefficient of expansion, and steel, which has a coefficient about the same as concrete. The standards that have been used in the field investigation are a 2 in. square by 12 in. bar for the Whittemore gage and a 1-7/8 in. square by 22 in. steel bar for the Munich gage. Relatively massive standards were selected so that they would respond slowly to temperature changes, as is the case for full sized structural members. It was determined experimentally, by means of strain measurements, that when the steel bars used were taken from room temperature (about 70° F) and placed in a freezer (about 0°F), the bars would reach that temperature in 5 to 6 hours.

Standards of this size are also stiff enough so that it is not necessary to position the bars on a perfectly smooth surface to avoid bending the bars while a reading is being taken. This is of particular advantage when working under field condtions where the standard often must be located on a rough surface.

The primary problem that arises with any temperature compensating standard is the representative quality of the bar compared to the structure. The manner of storage and frequently the manner of final positioning of a beam specimen may be such that only one flange or area may be exposed to direct sunlight. This induces temperature gradients across a section, thus causing strain gradients.

It is not possible to completely compensate for the temperature induced length changes by using one standard bar. Several would be required, but this is not practical under either field or laboratory conditions.

One possible way of eliminating this problem would be to take all readings in the morning before dawn. The test structure would have had at least eight to ten hours to come to thermal equilibrium with the surrounding air. The standard bar would then be most nearly representative of the test specimen. Such timing is seldom practical.

In the present investigation, the standard bar has been placed on the concrete in a position that seems to be best represent the specimens. The standard is placed on the concrete structure, but in the shade. Since the bars are darker colored than the concrete, placing them in the sun could produce a substantial error because of radiant heating of the bars.

An alternate method of temperature compensation is to use a standard bar which does not change length with temperature changes in the range desired. The temperature of the surface of the structure has to be determined by use of a surface thermometer, and corrections made analytically rather than mechanically. A completely temperature compensated standard can be made from a combination of steel and Invar bars.

d) Mechanical Strain Gage Use

It has been found that two experienced strain gage operators often will obtain different readings on any given gage line using the same instrument. The differences will be consistent, however, and no errors are introduced as long as the corrections for differences in the standard bar readings are made.

A few hours of practice in using the mechanical strain gages are necessary before consistent readings can be obtained. Because of differences in the grips on the gages, obtaining consistent readings with the Whittemore gage has proven easier than with the Munich gage. The Whittemore gage has a handle designed for one-handed use if necessary, and it is arranged so that forces parallel to the gage line are not easily transmitted to the measuring parts of the gage. The Munich gage must be held with both hands, and care is required to prevent transmission of forces into the gage body. The gage body is flexible enough to allow appreciable errors if the technique of using the gage is not consistent.

4. TEMPERATURE MEASURING DEVICES

4.1 General Remarks

It has been established that a temperature gradient across a reinforced concrete section can produce curvatures in a girder that may be of the same order of magnitude as those curvatures produced by the loading on a girder. For this reason, it is helpful to establish the temperature gradient in a test girder under field conditions. There are three general types of internal temperature measuring devices. These include: (a) thermocouples (b) thermometers, and (c) electrical resistance gages or semi-conductors.

4.2 The Thermocouple

A thermocouple consists of an electrical circuit such as that shown in Fig. 13. When wires of any two unlike metals are joined to form a complete circuit, it is found that an electromotive force exists in the circuit whenever the junctions A and B are different temperatures.

For a given pair of metals the emf depends on the difference in temperature between the two junctions. The more common metals used to make up a thermocouple are (a) Iron-Constantan, (b) Chromel-Alumel and (c) Copper-Constantan.

To use the thermocouple as a thermometer, one junction (the "reference junction") is usually placed in contact with a body of known temperature, such as ice water (32°F), and the other junction is placed in contact with the body whose temperature is to be measured. A potentiometer is then placed in the circuit and the induced emf is measured. A potentiometer is then placed in the circuit and the induced emf is measured. For a copper-constantan thermocouple with the reference junction at 32°, the induced emf is on the order of 0.5-1 millivolts at room temperatures. A potentiometer with a suspension type galvanometer is used to measure the emf in the field investigation. The calibration constant for a copper-constantan thermocouple is 0.022 M.V. per degree F. The indicating meter on the instrument can be read to $\pm$ 0.01 M.V., giving a sensitivity of $\pm$ 0.5°F.

The advantages of a thermocouple indicate simplicity, stability over a long time, sensitivity, and low cost.

To protect the thermocouple junctions from damage during casting in the field investigation, the junctions were encased in a 3/16-in. I. D. copper tube of a length sufficient to extend above the finished surface of the concrete. A practical limit to the length of copper tubing seems to be four to six feet, because of the problems in threading the wire. With the junction held at one end of the tube, the tubes were filled with molten wax, taking care that the junctions did not touch the tubes.

With the lead wire extended above the finished surface of the concrete, additional wire, of the same materials (this is important to avoid creating more active junctions), can be attached to the original wire. A splicing solder must be chosen that will not influence the reading on the potentiometer. For copper-constantan, common thermocouple materials, silver solder is the preferred splicing material.

An easy way to protect the long lead wires from the splice to a remote reading location is to encase the wire in 1/4-in. I.D. by 1/16 in. wall thickness clear polyetheylene tubing. The tubing provides a flexible, waterproof casing and has been found to be satisfactory for this use.

4.3 The Thermometer

A common mercury thermometer can be adapted for measuring the internal temperature of a concrete beam by drilling or preforming an access hole to the point desired. This method can be considered practical in cases where the desired points are readily accessible once the girder is in place on the piers.

One major disadvantage to the use of the thermometer exists. The boundary of air in an access hole between the thermometer and the walls of the hole acts as an excellent insulator to the thermometer. Discrepancies in temperature measurements of 10°F. or more have been attributed to the boundary layer insulation.

4.4 The Electrical Resistance Element

The electrical resistance gage can be in one of two forms: (a) a wire, such as an SR-4 gage, or (b) a semi-conductor.

Two examples of a wire gage whose resistance is sensitive to temperature are the SR-4 gage and the Carlson meter discussed in Chapter 3. The SR-4 gage can be bonded to a bar, cast into the concrete in such a manner to insure that no bond with the concrete is developed, and can be expected to behave in a stable and consistant manner (13).

The Carlson Test Set is arranged so that readings can be taken of both the ratio of the two coil resistances and of the total resistance of the two coils in series. By measuring the total resistance and by using the fact that the total resistance is a function of the temperature, it is possible to obtain the temperature in the concrete in the immediate vicinity of the meter. The calibration constant used in computing the temperature change is about 8.50 degrees per ohm, resulting in a minimum reading of about 0.1°F. The resistance at 0°F is given by the manufacturer.

It was observed during the testing of beam LB1 that the meter was sensitive to small changes in temperatures. Sudden changes in the room temperature due to an outside door being opened could be detected with the meter within 5 minutes. The meter was protected by 1.5 in. of concrete. The temperatures found during the field phase of this investigation have been in good agreement with the thermocouple readings.

The semi-conductor is mentioned only as a case of recent interest (13). Its temperature sensitivity is up to four orders of magnitude larger than the SR-4 type gage. Its major drawbacks are moisture sensitivity and expense. Further development with the semi-conductor is being undertaken principally by the aerospace industry.

5. INITIAL STRAND FORCE MEASUREMENTS

One major objective of the field investigation was the determination of the initial prestressing force and its change with time.

The time-dependent changes in the prestressing force after release can be determined from strain measurements in the concrete at the level of the strands, assuming knowledge of the stress-strain characteristics of the strand. This approach assumes plane strain distributions and adequate bond between the concrete and steel. Relaxation of the reinforcement is not taken into account.

During the prestressing operation, the force is usually determined by measuring the oil pressure in the hydraulic ram used. The relationship between pressure and force is known from calibration tests. Checks are made by comparing computed and measured strand elongations.

In order to obtain information on the forces remaining after anchorage and removal of the jack and up to the time of release of the prestressing force, dynamometers can be installed between the anchor bulkheads and the grips holding the strands.

A thick-walled aluminum cylinder which is loaded parallel to the axis of the cylinder has been found to form the basis of a suitable dynamometer. The dynomometer is instrumented as a four-arm Wheatstone bridge measuring circuit made up of four SR-4, type A-7 gages, two positioned longitudinally 180° apart. The dynamometer is a six in. long cylinder with 1-5/16 in. O. D., 5/8 in. I. D., and is made of 6061-T6 aluminum. It had been determined from previous experience that the six in. length was necessary so that the gages, located at the center, would not be influenced by end bearing effects.

Because 6061-T6 aluminum does not initially have a linear stress-strain relationship it was necessary to preload the dynamometers to 110 per cent of the maximum calibrated load several times to obtain a linear relationship. The dynamometers were designed to accomodate a 7/16 in. diameter prestressing strand and were calibrated to a maximum load of 26,000 lbs.

The gages were wired as four-arm bridge circuits in order to amplify the actual strain by a factor of 2.6 and to provide reasonable stability of the measuring circuit. The resultant sensitivity was about 40 lbs. per dial division, that is, per 10×10^{-6} in./in. of strain as indicated on a Baldwin Strain Indicator.

Fig. 14 shows a set of dynamometers in place at an end ~~anchorage with their lead wires connected to a central switchbox~~ which in turn is connected to a Baldwin Strain Indicator.

The arrangement shown in Fig. 14 permits obtaining the initial prestressing force as well as the time-dependent changes up to the time of release.

In order to measure the strain in the prestressing steel after release, SR-4 or other electric gages have been employed in the past. The construction sequence, in the present investigation, did not allow time for installation of such gages without undue interruption of the schedule. In addition, the waterproofing and stability problems associated with the SR-4 gages seemed to preclude their use.

All data on time-dependent strains after release of the prestress have been derived from measurements of strain in the concrete rather from measurements directly on the steel.

6. RELATIVE HUMIDITY INDICATIONS

It is well known that the moisture content of a reinforced concrete member may have a significant effect on time-dependent deformations. Consequently, instrumentation necessary to measure the moisture content on internal relative humidity of the concrete has been investigated on a cursory basis.

Several methods for determing the relative humidity of concrete have been developed by the Portland Cement Association, Research and Development Division. A brief description of these methods will be given below and their possibility of application to a field investigation will be investigated.

The first method is a destructive one and is used primarily to determine the relative humidity of one concrete blcok sampled from a large group of blocks. A representative block is broken with a chisel into small lumps of 3/4 to 2 inch maximum dimension. The lumps are placed in a metal basket which is sealed in a vaportight container. A blower in the container circulates the air inside to facilitate reaching humidity equilibrium between the air and the concrete. A hygrometer on the container indicates the relative humidity of the air. These humidity readings are corrected by calibration curves based on previous tests.

The second method and the others to follow are non-destructive and are used on nearly any size specimen. In the second method, a containeris placed on and sealed to the surface of the concrete specimen. The enclosed air eventually comes to the same humidity as that of the surface concrete. A hygrometer then measures the relative humidity of the air.

A third method has been used to measure the relative humidity at interior points in concrete. A 1-1/4-in.diameter hole with with a metal sleeve is cast to the desired depth in the member. An electrical hygrometer is fitted in a piston which makes a tight seal with the metal sleeve. The moisture of the exposed concrete at the bottom of the hole comes to equilibrium with that of the enclosed air. Again, the humidity of the air is measured.

The fourth and newest method is also for measuring the relative humidity at the interior of a member (21, 22). A 3/16-in. brass tube is cast the desired depth into the member. The probe-type Monfore gage which is lowered into the tube is 1/8-in. in diameter and 4 to 8 in. long. A Dacron thread in the probe is the moisture sensitive element whose change in length is measured by the change in electrical resistance of a tensioned wire attached to the Dacron. A conventional strain bridge is used to measure the resistance change. The humidity wells are filled with solid brass rods at all times except when measurements are taken.

All the hygrometers are calibrated in air of known relative humidities over a series of salt solutions. Other similarities of the methods are that the time per measurement is relatively great and that the temperature of the specimens must unfortunately be nearly the same for all measurements.

7. LABORATORY TESTS

7.1 General

Test beam LB1 was a reinforced concrete beam which was tested to provide a means of investigating the strain sensitivity, temperature sensitivity, and long-term stability of a Carlson strain meter. To collaborate the data obtained from the Carlson meter and also to train the research personnel in the use of the mechanical strain gages, external strain gage lines were installed for both the 10-in. Whittemore and the 50-cm Munich gages. In addition, a water-level deflection measuring system was installed on the beam, and the deflection measurements obtained were compared with mechanical dial gage readings.

The beam was loaded to about 60 percent of its capacity for ten days before being loaded to failure.

7.2 Description of LB1

The beam was a ten-ft long reinforced concrete beam with a gross cross-section 6 by 12 in. It was simply supported on a nine-ft span and was loaded at the third-points. The effective depth to the centroid of the tension steel was 10.5 in.

Two No. 7 high-strength reinforcing bars supplied the tension reinforcement, and a stress-strain curve is shown in Fig. 15. The reinforcement ratio was 0.019. A high reinforcement ratio and high strength steel were used so that the concrete strains would be large enough to test the capacity of the Carlson meter.

Stirrups bent from No. 3 deformed bars were spaced at 5 in. throughout the shear spans. A single No. 2 bar was placed in the compression zone in order to support the stirrups. This bar was interrupted near mid-span. The yield strength of the No. 3 bars was about 50 ksi.

The concrete mix was 1 : 3.92 : 4.17 by weight of cement, sand, and gravel respectively. The water : cement ratio was 0.70 by weight. The gravel was 1 in. maximum size limestone and the sand was Wabash River sand. Type III cement was used. The slump was one-in. The beam was moist cured for seven days and air-dried for three days before loading. The compressive strength was 5000 psi and the modulus of rupture was 510 psi at 10 days. The compressive strength at failure of the beam (20 days) was 6000 psi.

The concrete was mixed in a six-cubic-foot capacity horizontal rotating drum mixer for three minutes. Six-6 by 12 in. control cylinders and one modulus of rupture beam were taken from each of two batches. The beam was cast in two horizontal layers and was compacted with an immersion type vibrator.

7.3 Instrumentation of LB1

A Carlson strain meter was installed in LB1 at mid-span. The center of the gage was 2-1/8 in. below the top fiber. It was secrued to the reinforcement cage with annealed steel wire. It was found that dents were made in the brass sheathing if the wire was tied too tightly, so the meter was secured with several loose wires.

External strain gage lines were also established on the beam at mid-span. The gage layout is shown in Fig. 16. Gage lines were established at (a) the level of the centerline of the Carlson meter, (b) 4-9/16 in. from the top fiber, corresponding approximately to the depth to the neutral axis based on the cracked, transformed section, and (c) along the center line of the tension steel, 10 1/2 in. from the top fiber. The gage reference points used were 1/4 in. diam hex-head cap-screws, 5/8 in. long, which were drilled with 3/32 in. diam spherical holes.

Two alternate schemes were used to secure the cap-screws to the beam. The first was a 1/2 in. diam reinforcing bar 1 in. long that had a 1/4 in. diam by 3/4 in. deep tapped hole along its longitudinal axis as described in Sec. 3.4.2.2(b). The second scheme employed the use of the same 1/2 in. bar but with the addition of a small loop of No. 2 plain bar welded to the untapped end. The addition of the loop aids in the bonding of the bar but this was found to be unnecessary. Tapered washers were used to form countersinks for the cap screw heads.

Mid-span deflections relative to the supports were measured with a 0.001 in. sensitivity mechanical dial gage on a stand under the beam. A small steel plate was glued to the beam to provide a flat surface for the dial gage to bear against.

The water-level scheme installed used three 5 cc pipettes, one located at each support and the third at mid-span, as the measuring elements. Rubber stoppers were glued to the concrete surface at each of the three points with Eastman 9-10 glue. The pipettes were inserted, tips upwards, through holes in the stoppers and connected in series with 1/4 in. I.D. clear polyethylene tubing, and a glass Y-connection. Water, colored with red food coloring, was used to fill the system. The system is shown schematically in Fig. 17.

Readings of the fluid levels were taken at various load levels, and the deflections due to superimposed load were found as described in Sec. 2.4.

A deflection of the beam produced changes in the fluid level in all three tubes. In addition, temperature changes and increases in the diameter of the connecting plastic tubing due to the small sustained pressures changed the readings in all three vertical tubes. These changes do not affect the results, however, since only the movements of the center relative to the supports are required.

The fluid level was read at the bottom of the meniscus. The graduations on the pipette were about 1/8 in. apart and readings were estimated to the nearest tenth of a division.

It was found that the length of division of each of the three pipettes were not the same. For 5 cc of fluid volume, five divisions on the pipettes extended 7.15 in. on the center pipette, 7.27 in. on the west pipette, and 7.11 in. for the east pipette. Consequently, small correction factors were necessary in reducing the data.

7.4 Test Procedure for LB1

The loading and support conditions for the beam are shown in Fig. 17. The beam was placed in a screw-type testing machine. Initially, the load was applied with the hydraulic jack reacting against the top loading head of the testing machine and the load was measured with an electrical dynamometer. The loading sequence was as follows:

i) Load in increments to 5 kips and unload,

ii) Reload to 5 kips and increase by increments to 20 kips and unload,

iii) Reload to 10 kips and increase by increments to 20 kips and unload,

iv) Reload to 20 kips and increase by increments to 30 kips and unload,

v) Reload to 30 kips, using the testing machine instead of the hydraulic jack and dynamometer to apply and control the load,

vi) Maintain 30 kip load for 10 days, and

vii) Unload and reload to failure.

At each increment of load all instruments were read.

It was found that the screw-type testing machine would maintain the deflection constant, but with a time-dependent decrease in load. The sustained 30 kip load was placed on the beam at 2:45 pm 30 April 1966, and was removed at 1:16 pm 10 May 1966, during which time the midspan deflection increased from 0.43 to 0.57 in. and the concrete strain as indicated by the Carlson meter increased from 853×10^{-6} in./in. to 1598×10^{-6} in./in.

It was necessary to adjust the load during the first five hours of the sustained loading period at approximately 1/2 hours intervals. After this time adjustments and readings were made 2 or 3 times a day. At no time did the sustained load drop below 29 kips.

7.5 Behavior of Beam and Instrumentation.

The beam was subjected to the six loading cycles described in the preceding section. The load-deflection curve for the midspan section is shown in Fig. 18. The deflections shown were measured with a mechanical dial gage.

The maximum load was 52.7 kips, at which load the concrete at the compression surface crushed, and the load capacity decreased rapidly. The beam did not exhibit appreciable ductility, but this was expected because of the presence of 1.9 percent of high-strength reinforcement. The load capacity predicted on the basis of an analysis taking the measured stress-strain curve for the reinforcement into account was 55.6 kips, five percent greater than was measured. The predicted steel stress at failure was 94 ksi.

The time-dependent increase in deflection under the 30-kips sustained load is shown in Fig. 19. The deflection increased 32 percent during the ten days.

The deflections were also measured with a water-level system made of pipettes and tubing, as described earlier. In Fig. 20, the deflections measured with the dial gage are plotted against the deflections measured with the water-level system. The solid line at the 45 degree slope indicates the line of perfect agreement.

It can be seen that the points, each representing a deflection measurement made by means of a mechanical dial gage and another made with the water-level device, lie generally along a straight line. The line is not coincident with the 45 degree line which represents complete agreement, but the two are approximately parallel. The apparent shift upwards of the line from the correct position could be caused by an error in the zero reading with the water-level device.

The two measurements generally agree within about 0.05 in., although there is appreciable scatter. The consistency of the measurements could probably be improved. Larger diameter tubes would have reduced the importance of the surface tension effects. The effects of surface tension could be overcome by disturbing the system by squeezing the tubing just before taking the readings. Unfortunately, this was not discovered until the test had been underway for some time.

It must be concluded that the water-level deflection measuring technique, at least as used in this laboratory experiment, must be somewhat modified and refined before it can be used successfully to measure deflections.

The strain measured with the Carlson meter during the test is plotted as a function of applied load in Fig. 21. As was the case with the deflections, there is no ductility indicated, and crushing of the concrete occurred while the rate of increase of load was still quite high. The maximum measured strain was about 0.0024. The Carlson meter was located 2 1/8 in. below the top of the beam, and the computed neutral axis depth was about 4 1/2 in.at failure. From the relative depths, the surface strain at failure appears to have been about 0.0045.

After the failure occurred the Carlson meter could no longer be balanced.

The time-dependent increase in strain during the ten days of sustained load is plotted in Fig. 22. During the ten days the strain increased by 85 percent over the initial value. In the same period the deflections increased 32 percent .

Mechanical strain gage readings were taken at the level of the Carlson meter in an attempt to check the Carlson meter. The readings from both mechanical gages and from the Carlson meter during the sustained load period are plotted against time in Fig. 23. It is immediately obvious that the Carlson meter provided the check rather than being checked. The scatter in the readings from the mechanical gages was excessive, and the use of the mechanical gages was carefully reviewed as a result of this test.

Both mechanical gages were equipped with spherical gage points, and spherical holes were used in the reference points in the beam. After this test, the gage points were examined, and it was found that the sizes of the points were somewhat different. The gage points on the Whittemore gage were 0.002 to 0.004 in. different in size, and all were at least 0.004 in. smaller than the 3/32-in. spherical holes in the reference points. No reasonable method of improving the precision of the fit was apparent, so the decision to change the shape of the hole in the reference point to conical was made. Subsequent laboratory and field experience have supported this decision.

The repeatability of the gage readings improved substantially when the holes were changed to conical shapes. A bonus factor also appeared; the conical holes do not need to be "worn-in" before repeatable readings can be obtained. The spherical holes (and cylindrical holes as well) had to be used several times before consistent readings could be obtained.

The measured deflections are plotted as a function of the measured concrete strains in Fig. 24. Initially there is a short straight line which indicates the response in the uncracked stage of behavior. Then the slope of the line changes, as cracking developed, and a second straight segment is seen. A third straight segment corresponds to the increase in strain and deflection during the sustained load period, and the fourth segment corresponds to the final loading to failure. The fourth segment remains linear to a strain of about 0.0022, and then curves somewhat. The last two points are for loads of 51 and 52 kips, respectively, and the beam failed at 52.7 kips.

It does not seem possible to determine whether the final irregularities were due to changes in the beam or in the strain gage. However, it appears that the gage was capable of producing consistent, linear readings to a strain of at least 0.0022. Information on the response to higher strains is inconclusive.

8. SUMMARY AND CONSLUSIONS

This report has described various instrumentation suitable for use in a field investigation of the long term performance of prestressed or reinforced concrete structures.

Deflection measuring devices and methods are described in Chapter 2. The simplest and most straight-forward method of determining deflections appears to be to use a surveyor's precise level and a suitable level rod. A water-level device was described; it may be suitable in some cases. The use of a taut wire or string-line was also discussed.

Strain measuring devices were discussed and described in Chapter 3. Carlson strain meters are currently being used in the field phase of the investigation, and their performance has been satisfactory. The Carlson meter is made for embedment in concrete. Vibrating wire strain gages which are suitable for embedment in concrete are available, and their long-term stability and sensitivity make them useful for a field investigation, although none have been used in this project.

Surface strain can be measured satisfactorily with various mechanical strain gages, assuming adequate temperature compensation methods are used. The Whittemore strain gage has been successfully used in the field phase of this investigation. The gage points used have spherical tips. The reference points which are attached to the structure are drilled with conical holes into which the spherical tips fit. Some types of vibrating-wire gages are suitable for measuring surface strains.

Temperature measurements at internal points in the structure may be taken with thermocouples, as described in Chapter 4.

The dynamometers used to measure the strand forces during the process of manufacturing the girders of the test structure are described in Chapter 5. The aluminum sleeve-type dynamometers, which were instrumented with SR-4 electrical resistance strain gages, were fitted over the strands between the anchor bulkhead and the strand grips at the time of stressing of the tendons.

A few devices suitable for measuring the relative humidity at internal points in concrete members are described in Chapter 6. All of the devices seem to have serious drawbacks concerning temperature compensation which makes their use in a field investigation relatively difficult. None of these gages has been used in the field investigation phase of this project.

A reinforced concrete beam was manufactured and tested in the laboratory in order to investigate the behavior of the Carlson strain meter and the water-level deflection device. The results of this test are described in Chapter 7. The Carlson meter appeared to respond in a linear, consistent manner. During the 10-day sustained loading phase, the stability of the gage readings appeared to be very good. The water-level deflection device appeared to be capable of consistently producing readings within about 0.05 in. of those from a mechanical deflection gage, which may be satisfactory for some applications.

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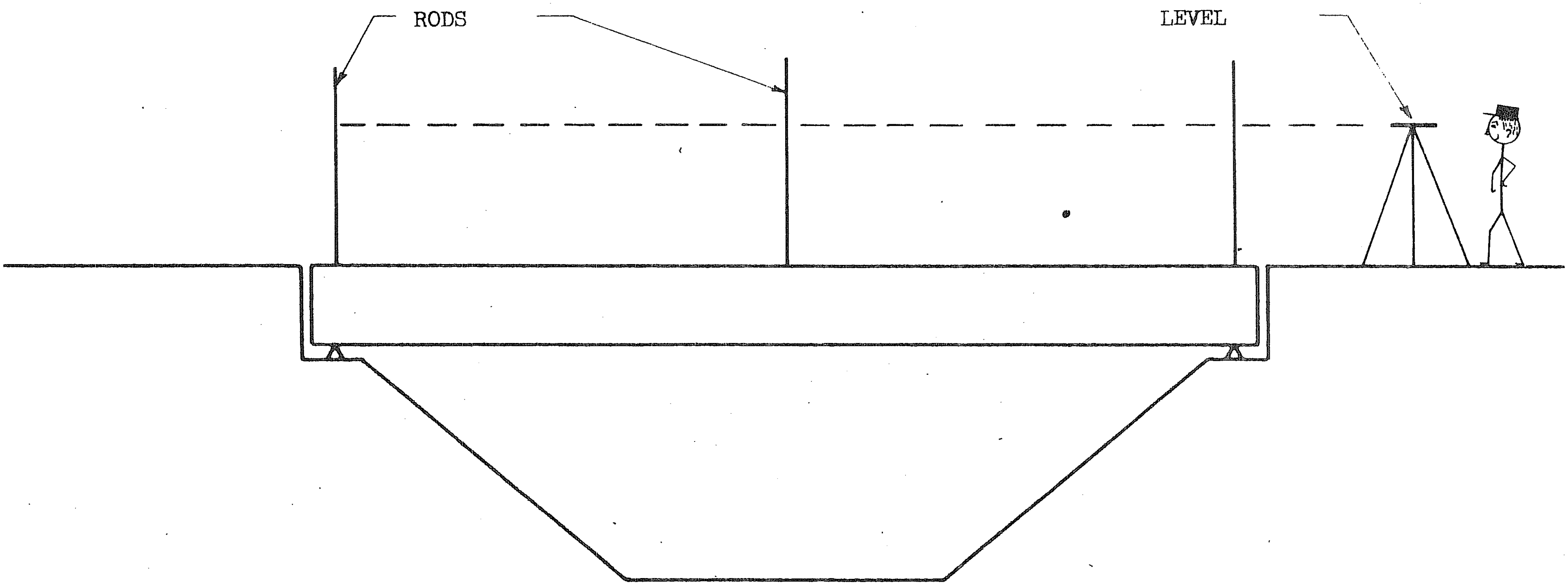
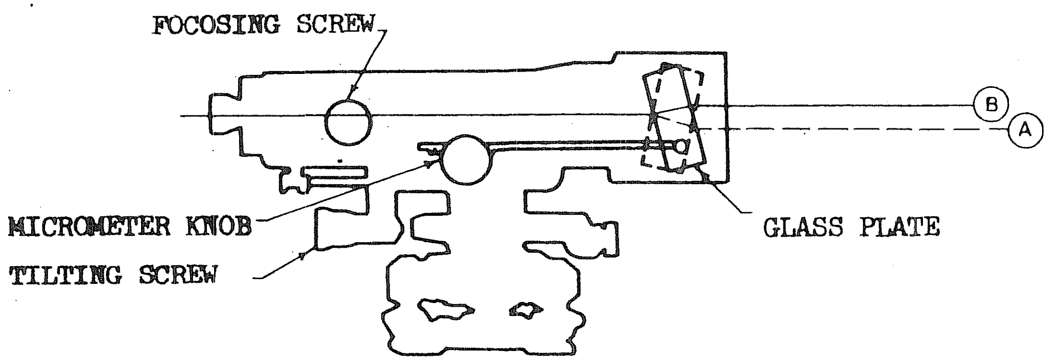
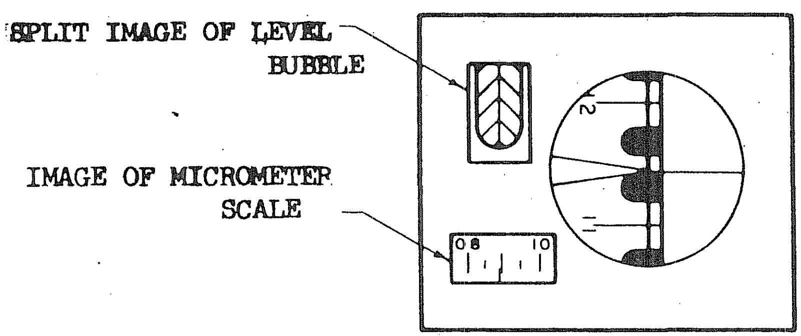


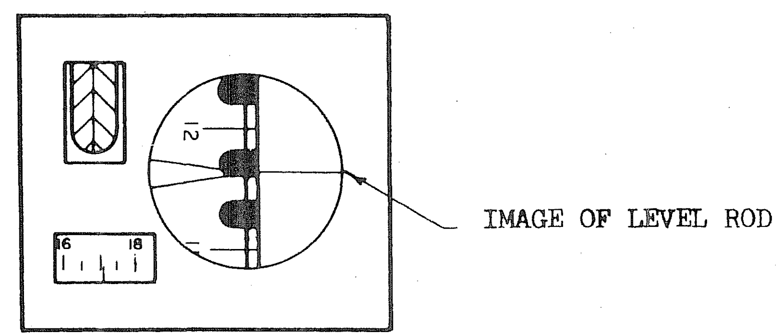
FIG. 1 LEVEL AND ROD DEFLECTION MEASUREMENTS



WILD N-3 LEVEL



Line of Sight in Position A
Before Adjustment



Line of Sight in Position B
Adjusted for a Reading (1.177 ft.)

FIG. 2 SCHEMATIC REPRESNTATION OF USE OF WILD N-3 LEVEL

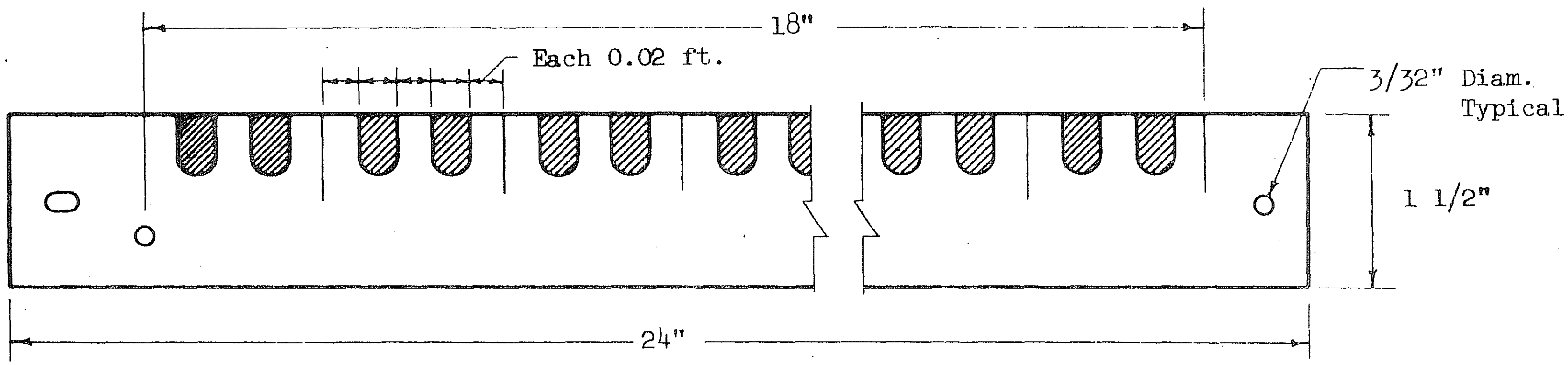


FIG. 3 SCALE FOR LEVEL ROD

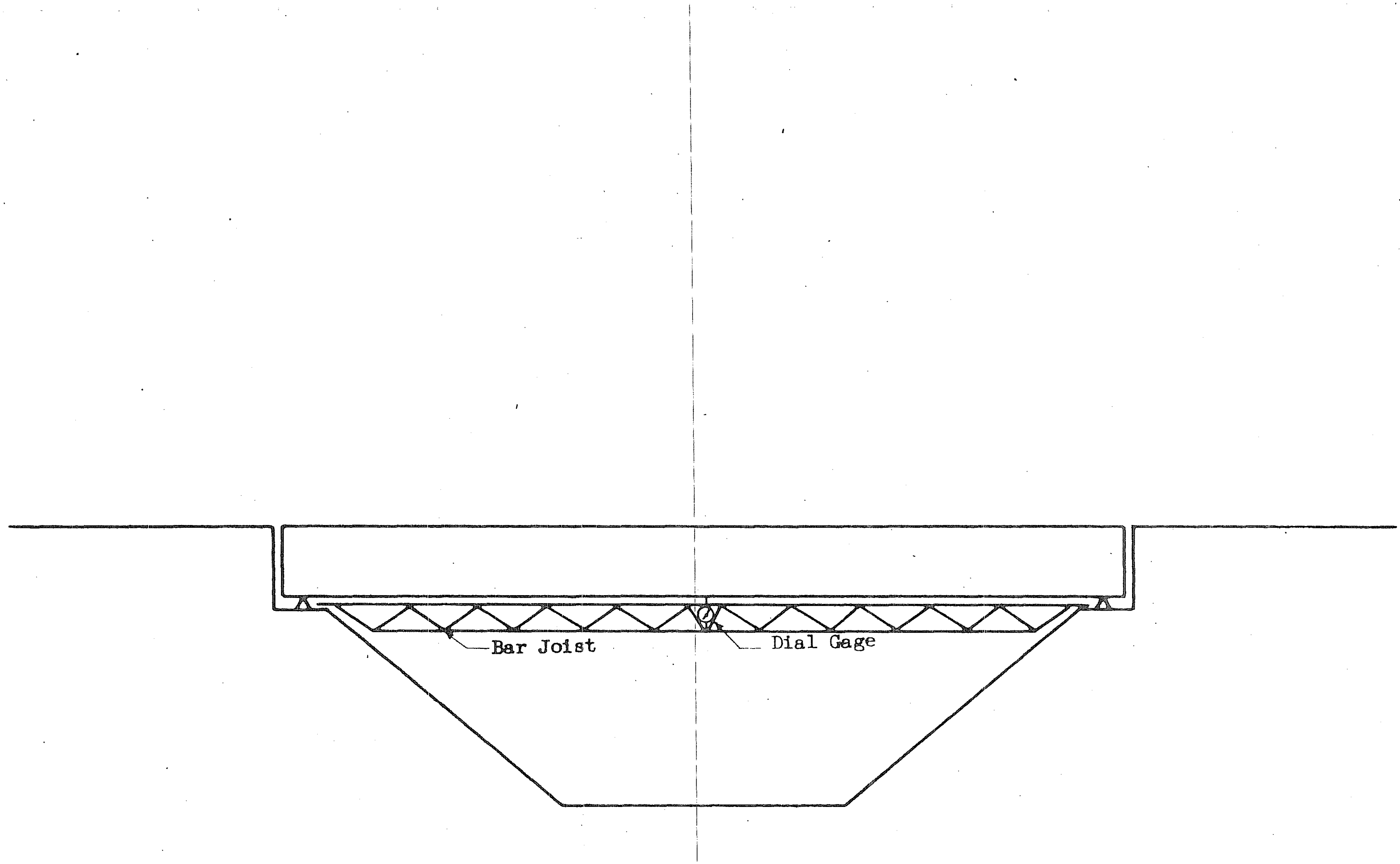


FIG. 4 TRUSS-SUPPORTED DEFLECTION GAGE

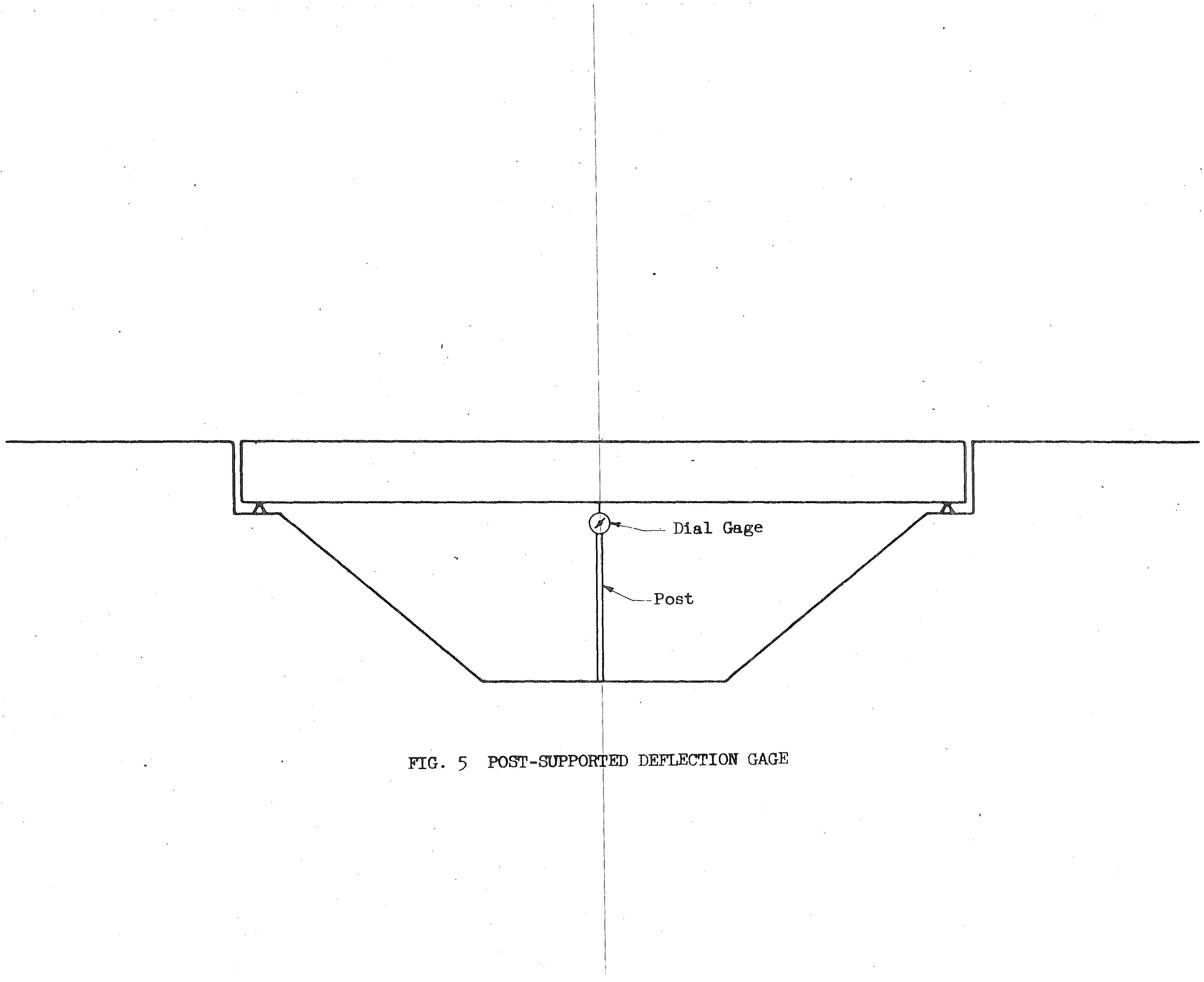


FIG. 5 POST-SUPPORTED DEFLECTION GAGE

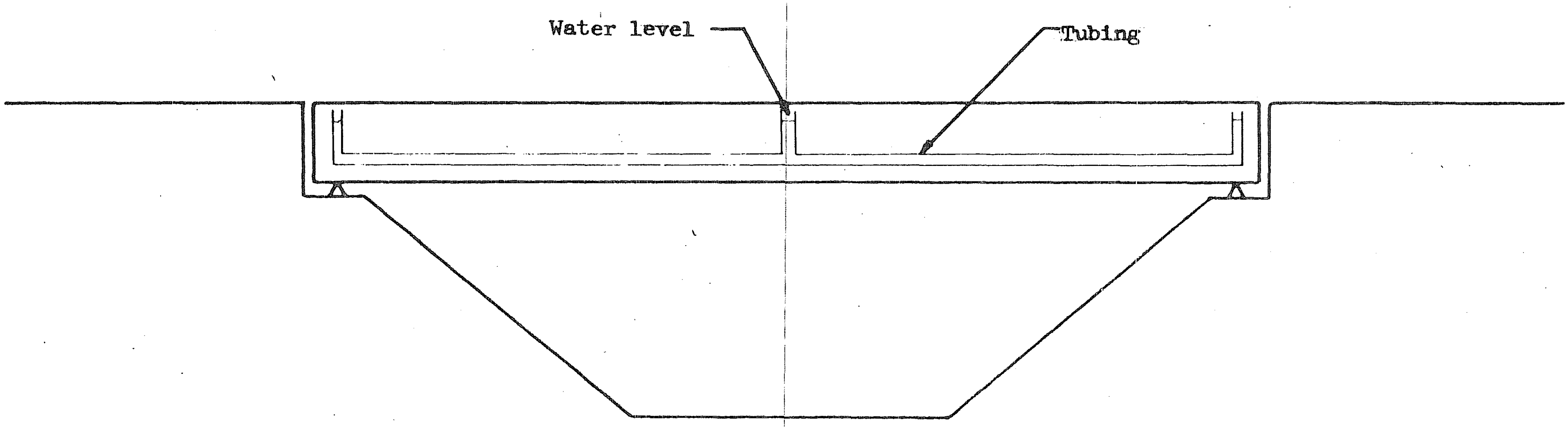


FIG. 6 WATER-LEVEL DEFLECTION DEVICE

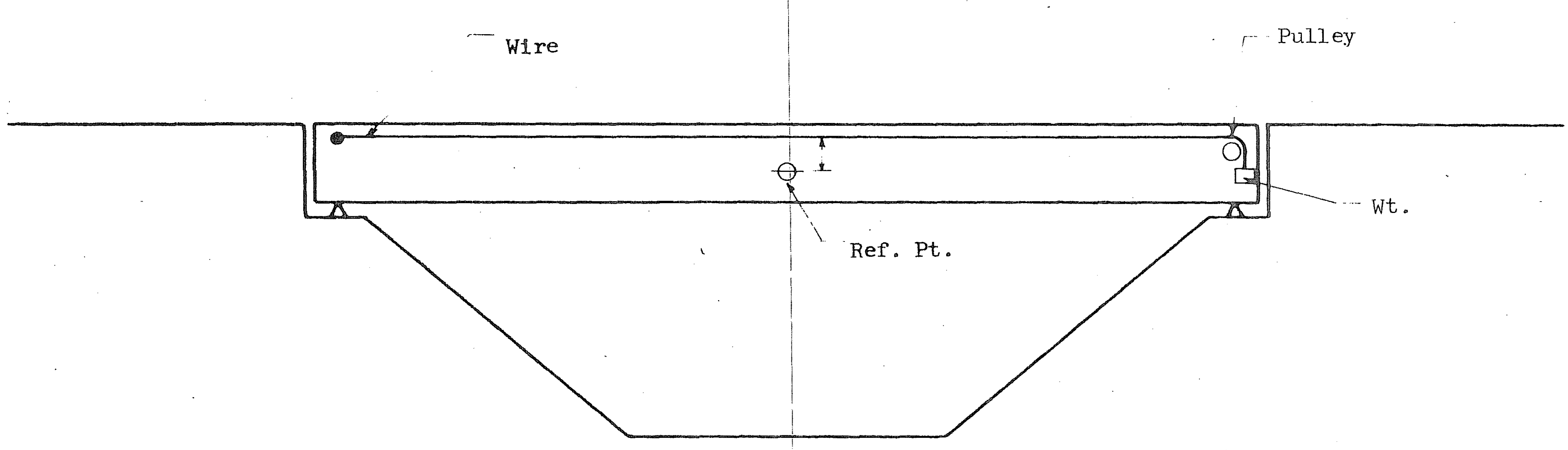


FIG. 7 TAUT-WIRE DEFLECTION DEVICE

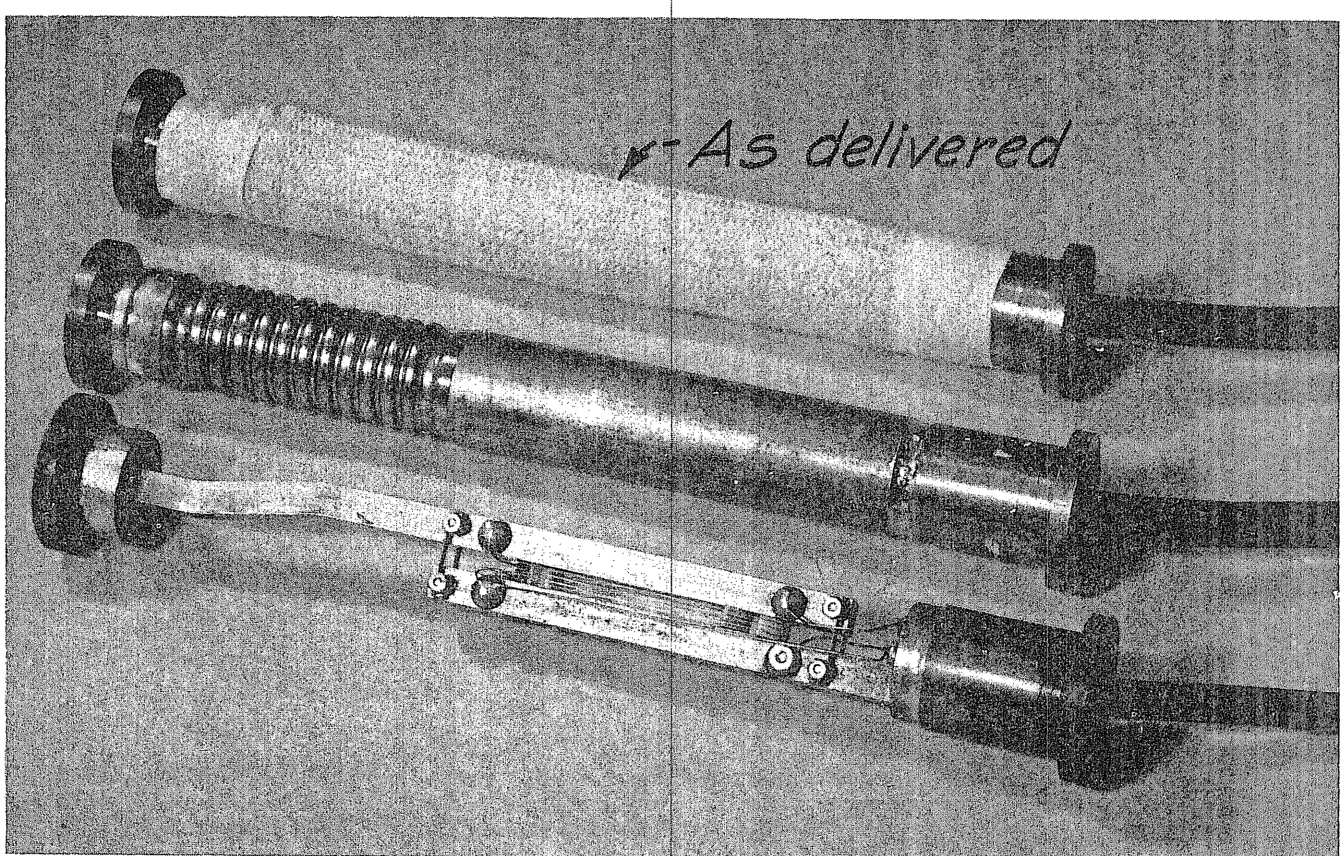
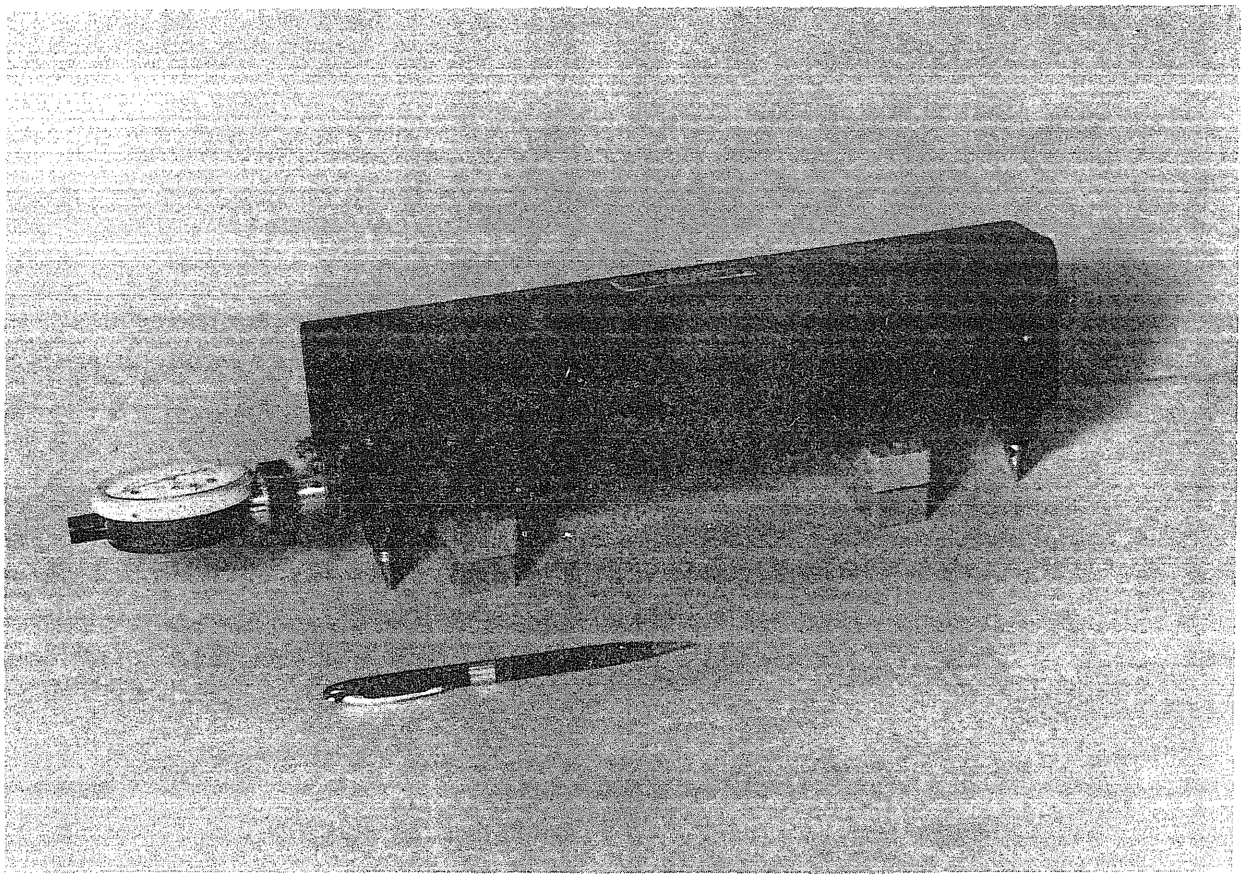
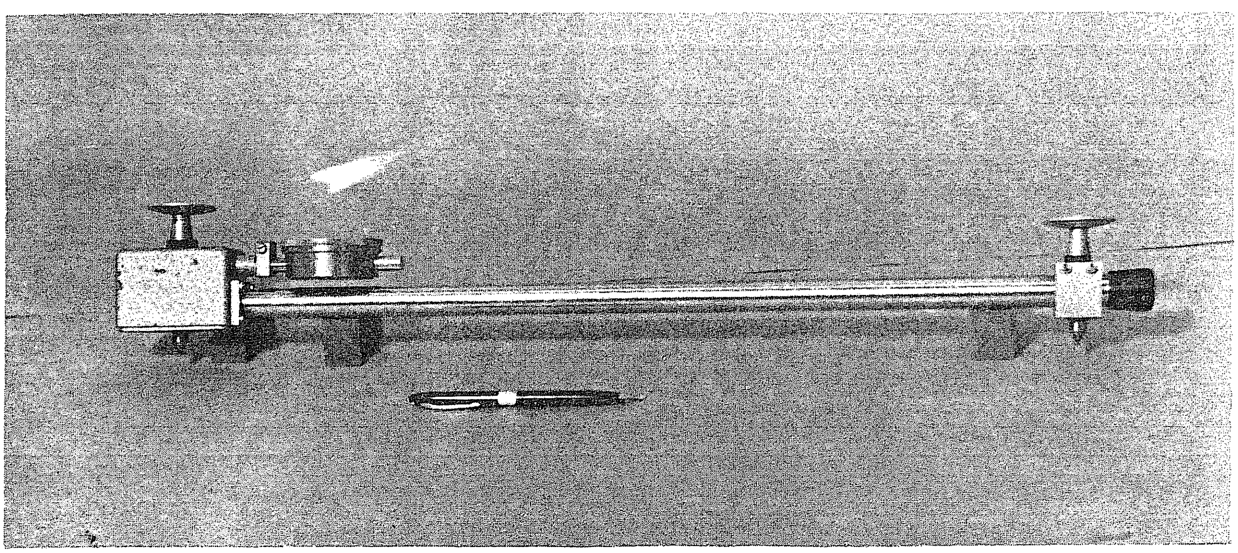


FIG. 8 PHOTOGRAPH OF CARLSON STRAIN METER



(a) Whittemore Strain Gage



(b) Munich Strain Gage

FIG. 9 PHOTOGRAPHS OF MECHANICAL STRAIN GAGES

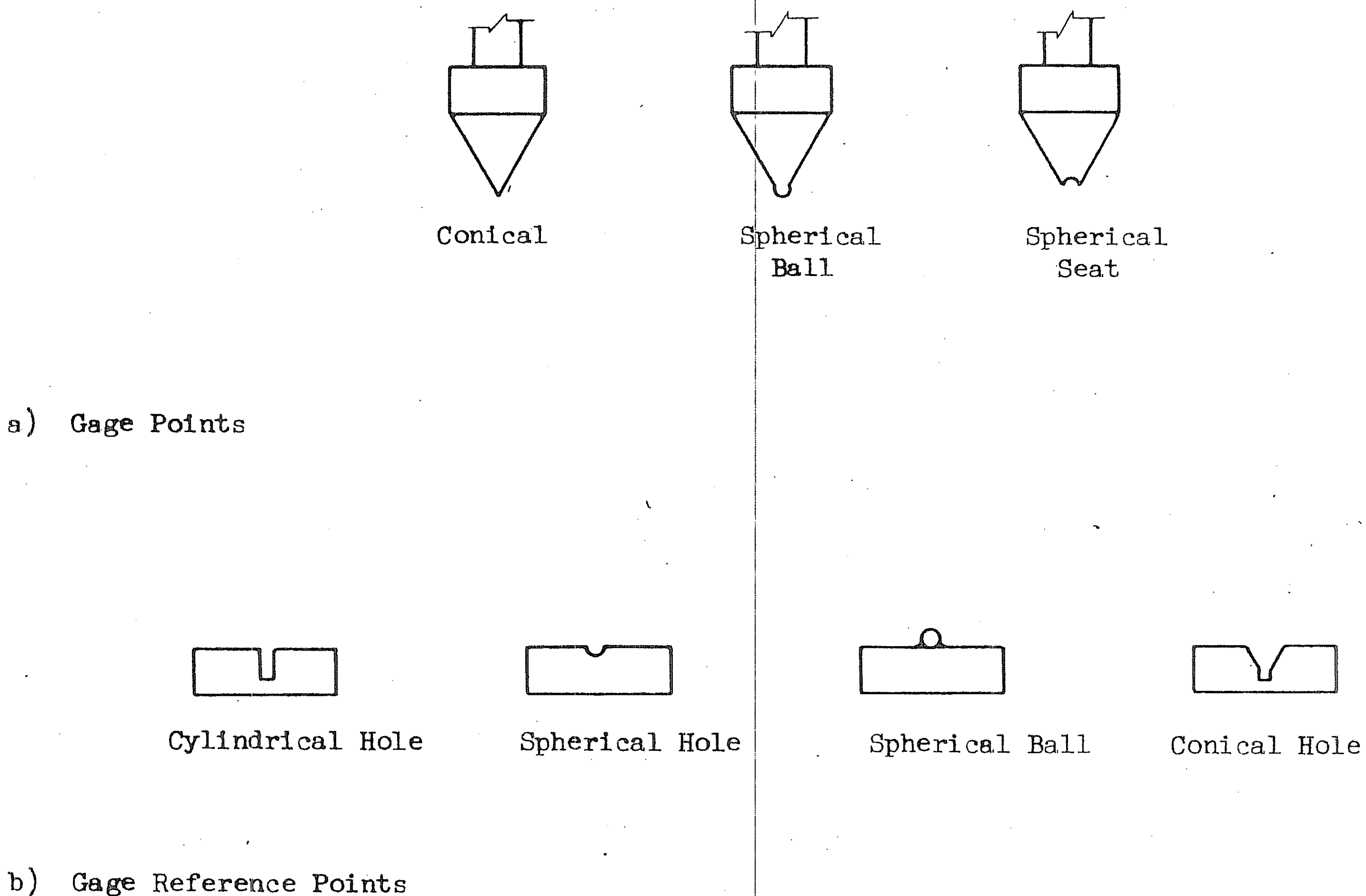
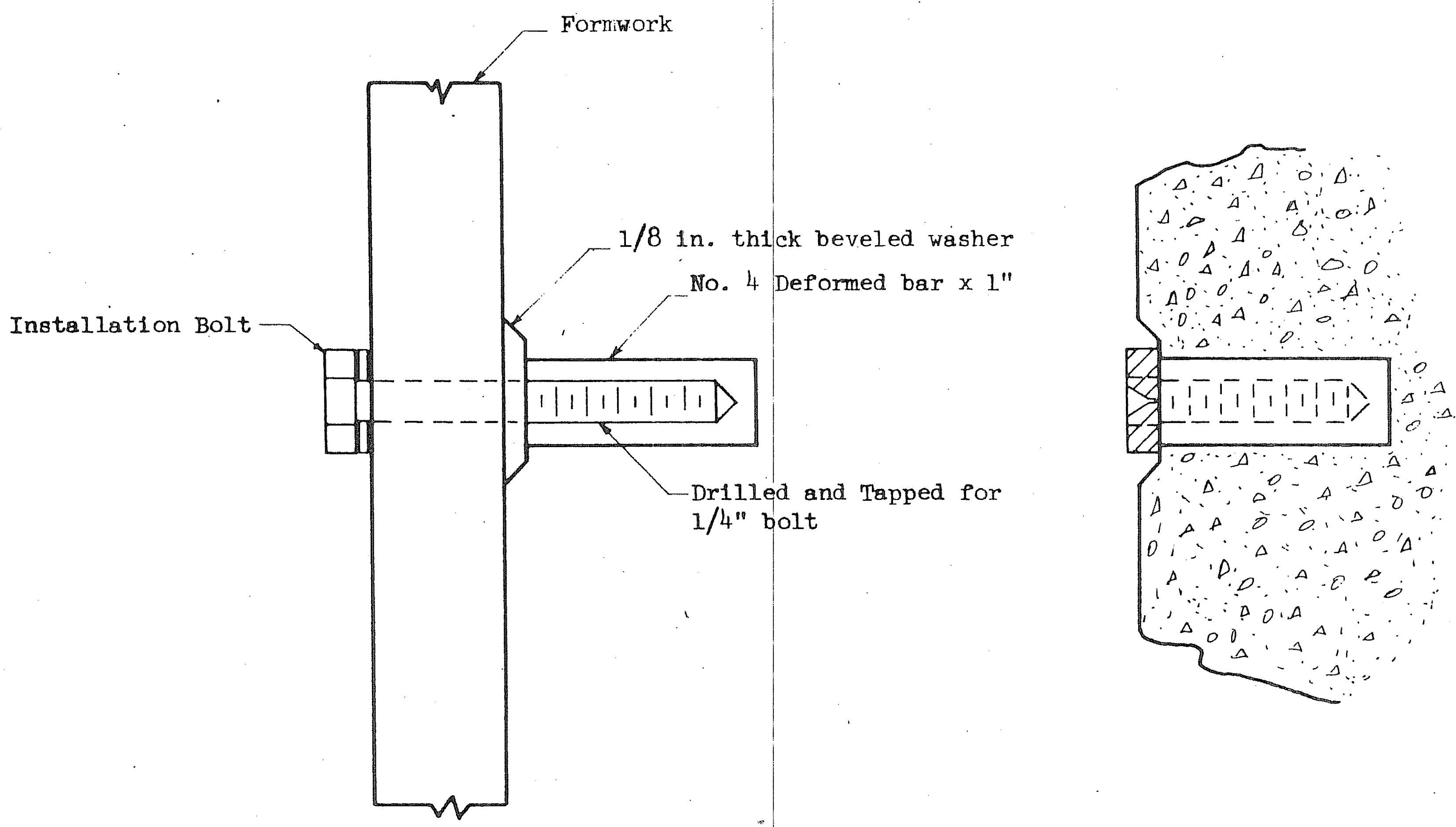


FIG. 10 FORMS OF STRAIN GAGE POINTS AND REFERENCE POINTS



a) Installed in forms

b) After final installation

FIG. 11 INSTALLATION OF STRAIN GAGE REFERENCE POINTS IN STRUCTURE

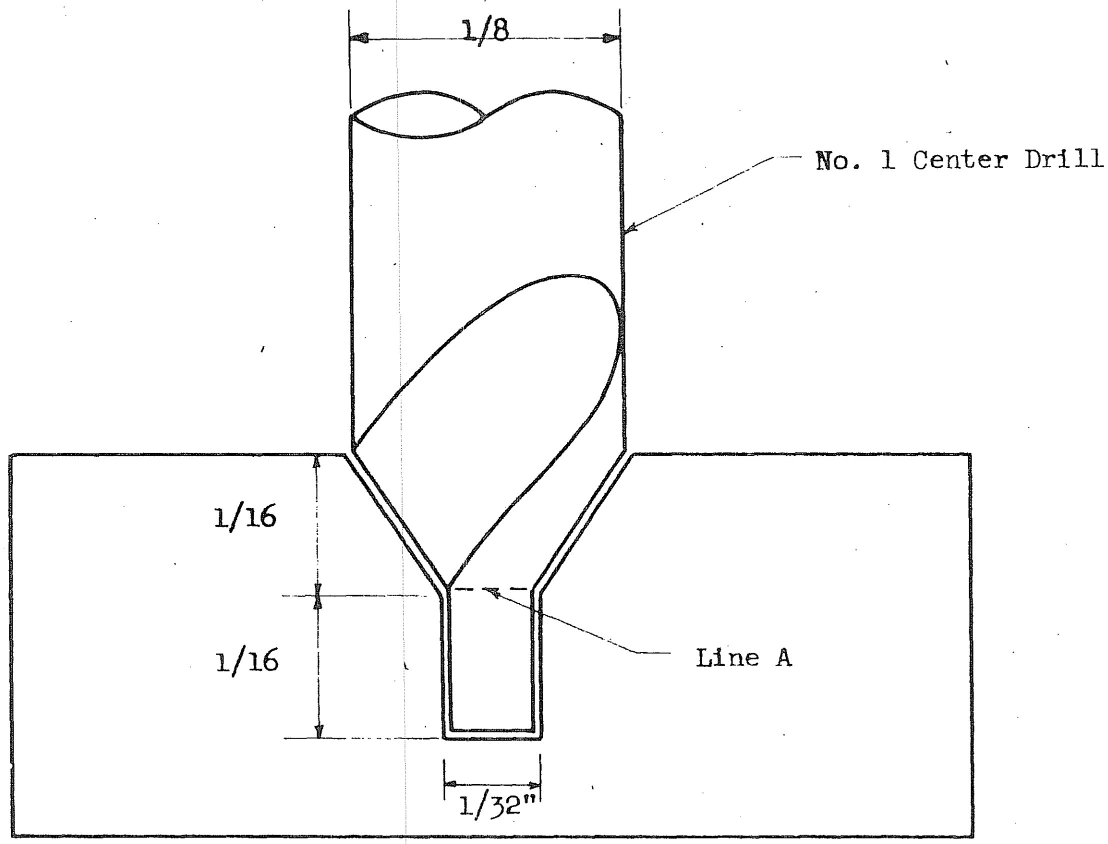


FIG. 12 DETAIL OF NO. 1 CENTER-DRILL AND GAGE HOLE

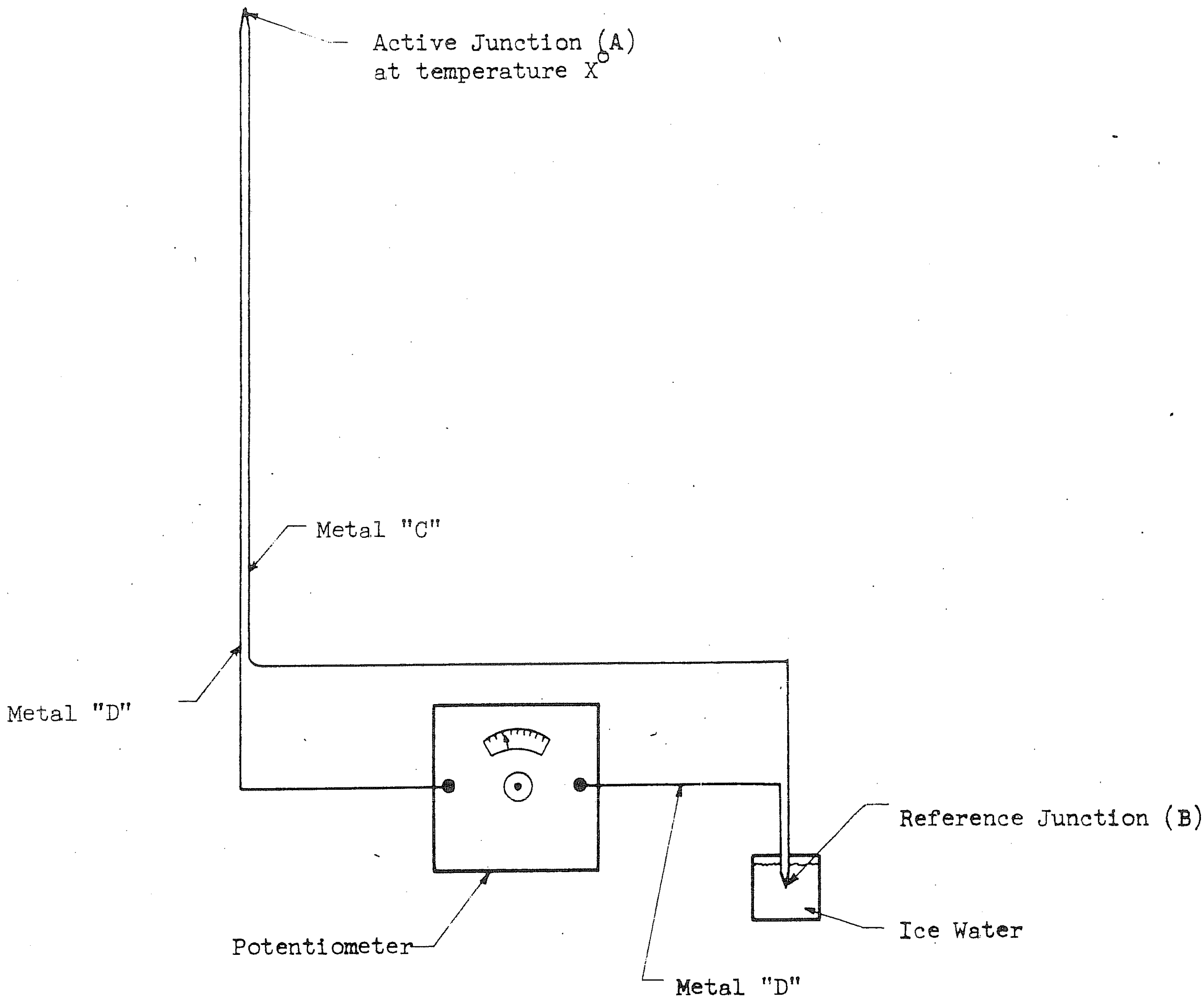


FIG. 13 SCHEMATIC DRAWING OF THERMOCOUPLE CIRCUIT

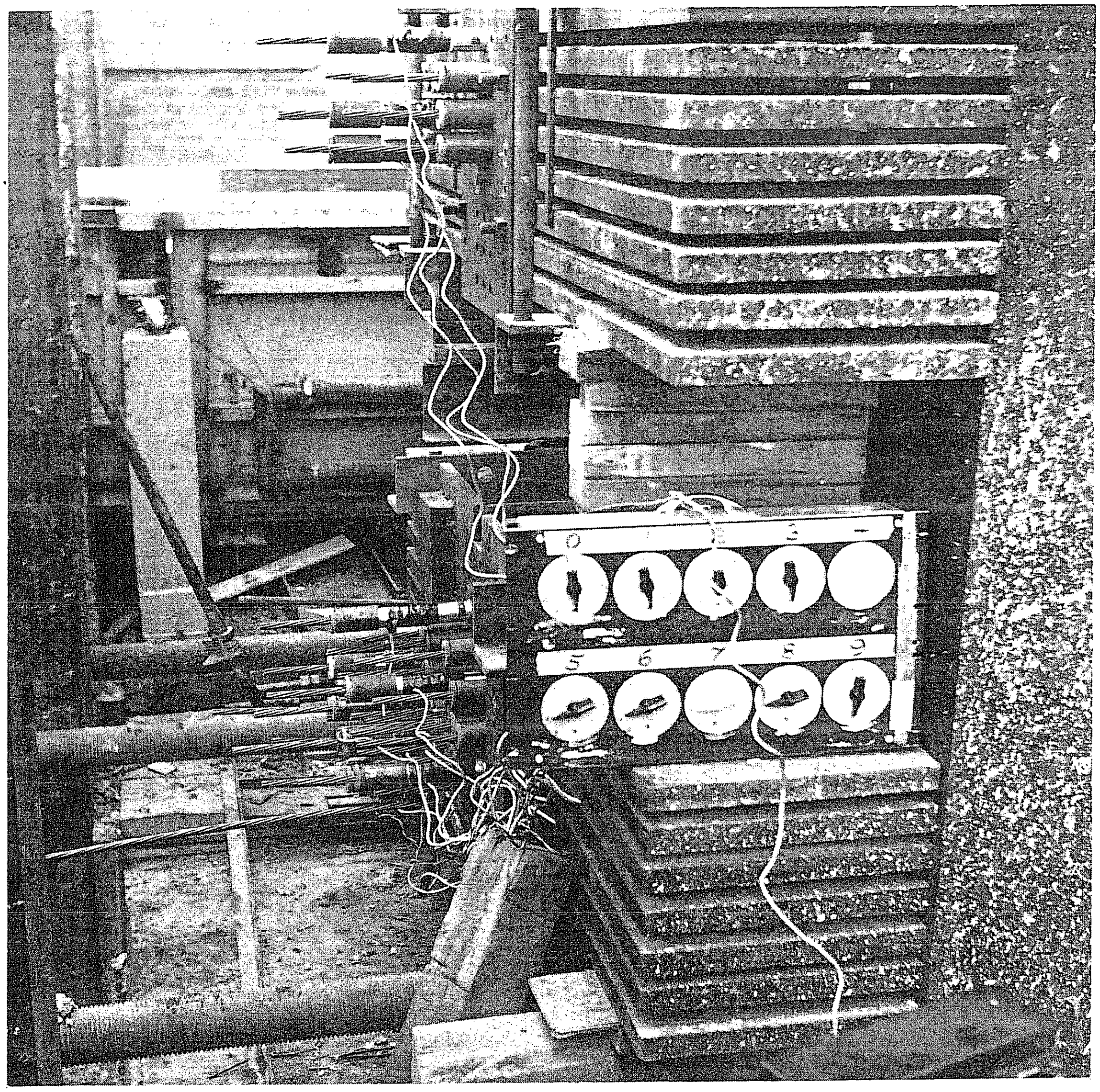


FIG. 14 PHOTOGRAPH OF STRAND - FORCE DYNAMOMETERS

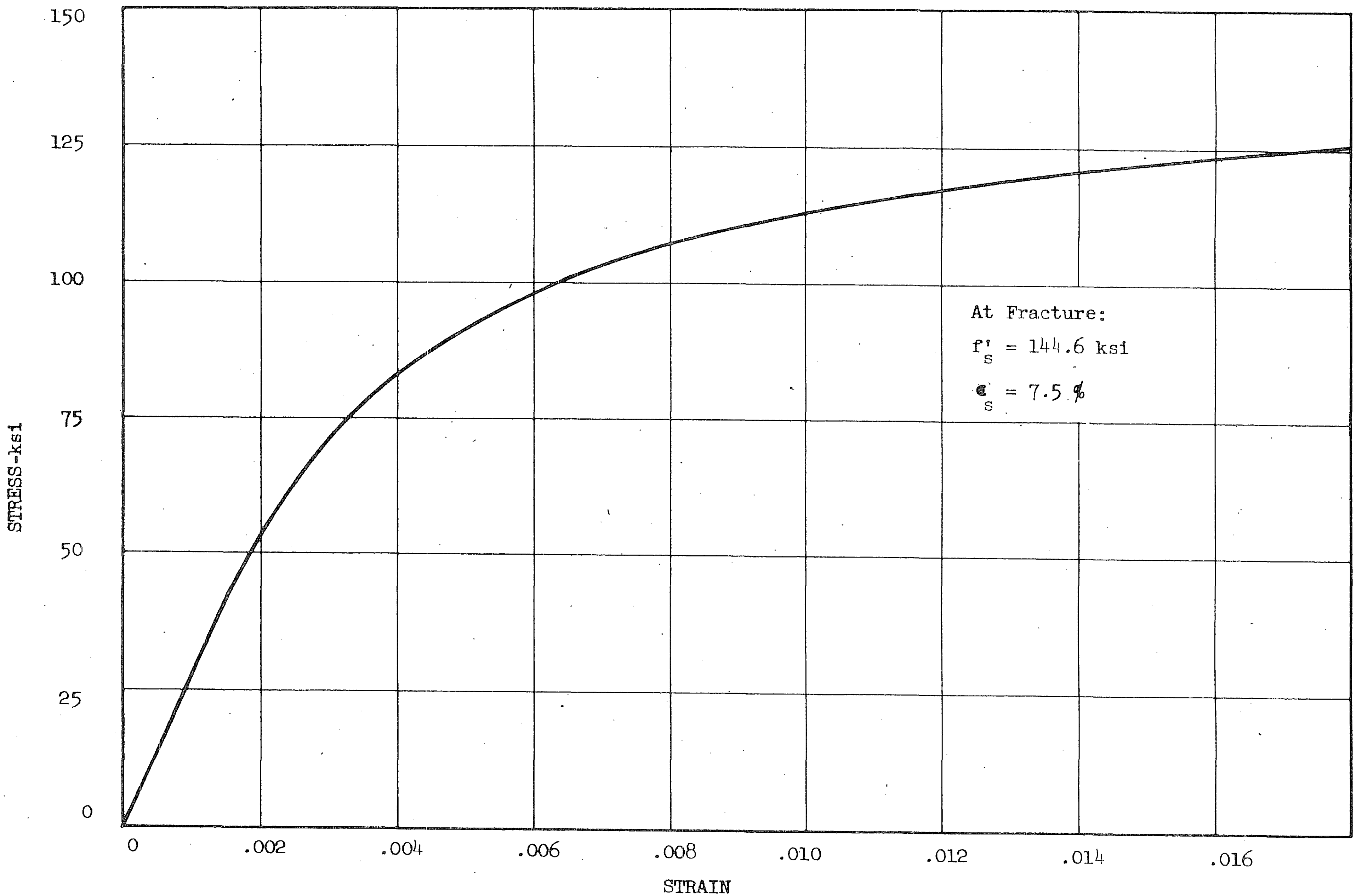
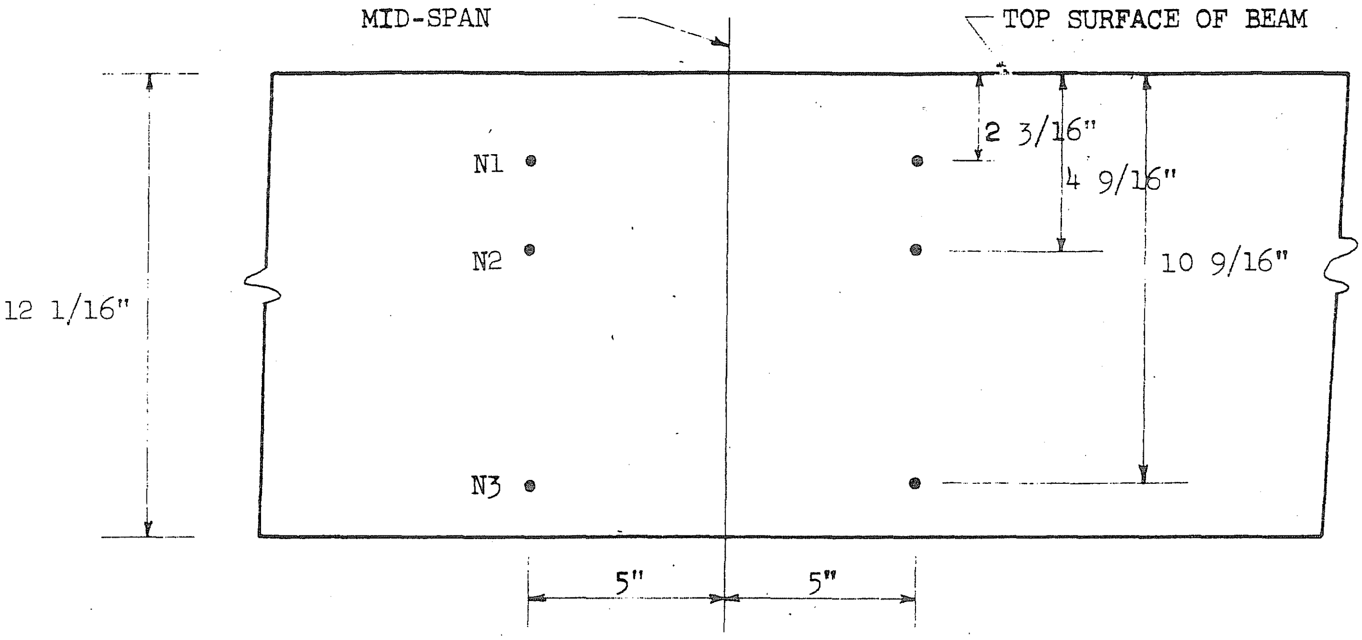
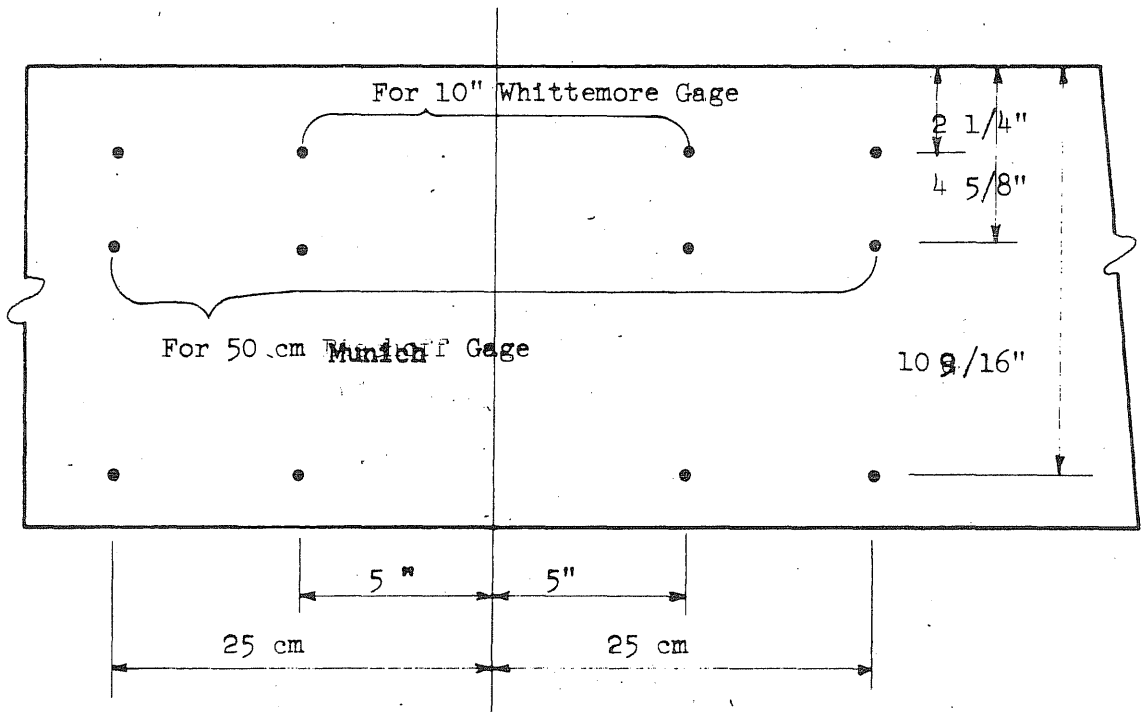


FIG. 15 STRESS-STRAIN CURVE FOR NO. 7 BAR FROM BEAM LB 1



a) Mechanical Strain Gage Lines on N. Side of Beam



b) Mechanical Strain Gage Points on S. Side of Beam

FIG. 16 LOCATION OF MECHANICAL STRAIN GAGE LINES ON BEAM LB1

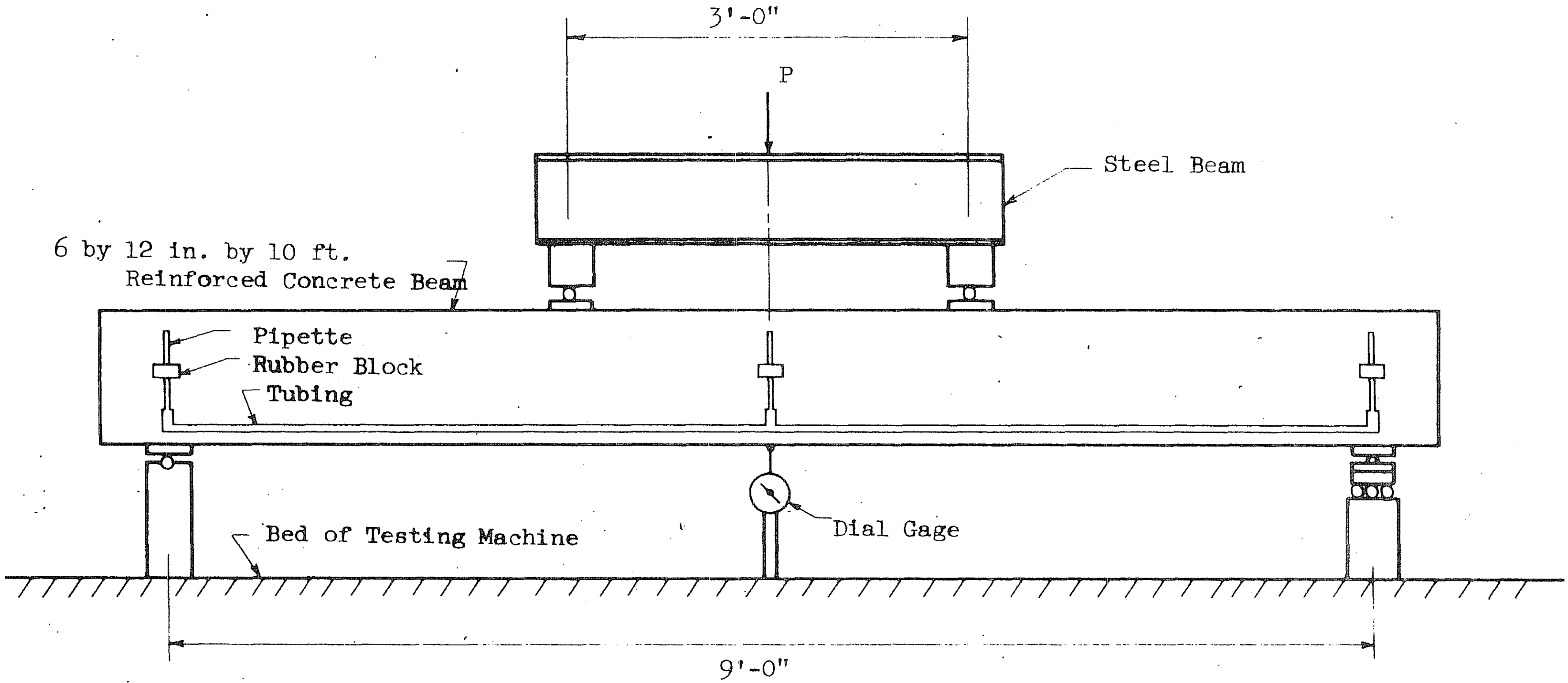


FIG. 17 LOADING AND SUPPORT ARRANGEMENTS FOR BEAM LB1

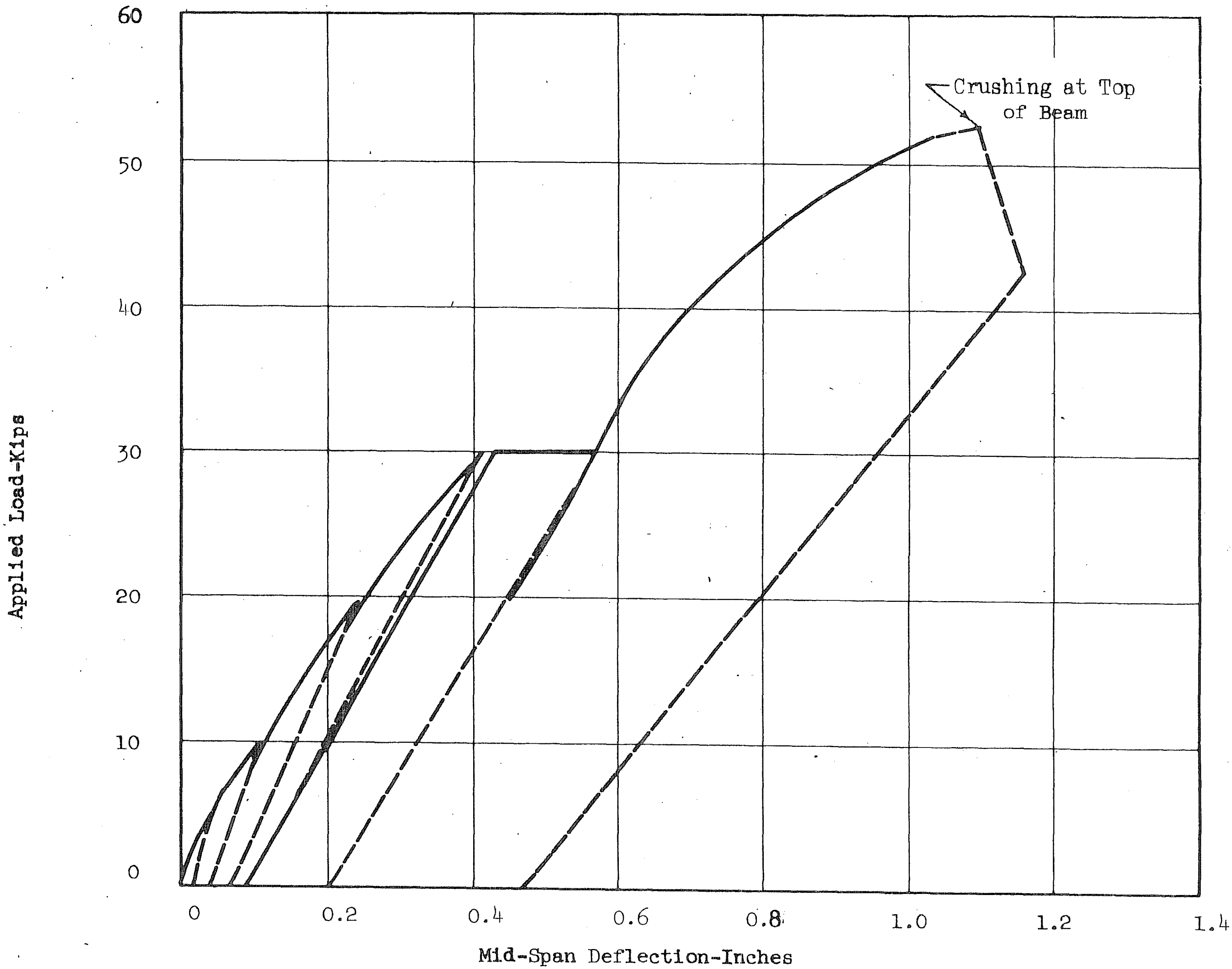


FIG. 18 LOAD-DEFLECTION CURVE, TEST BEAM LB-1

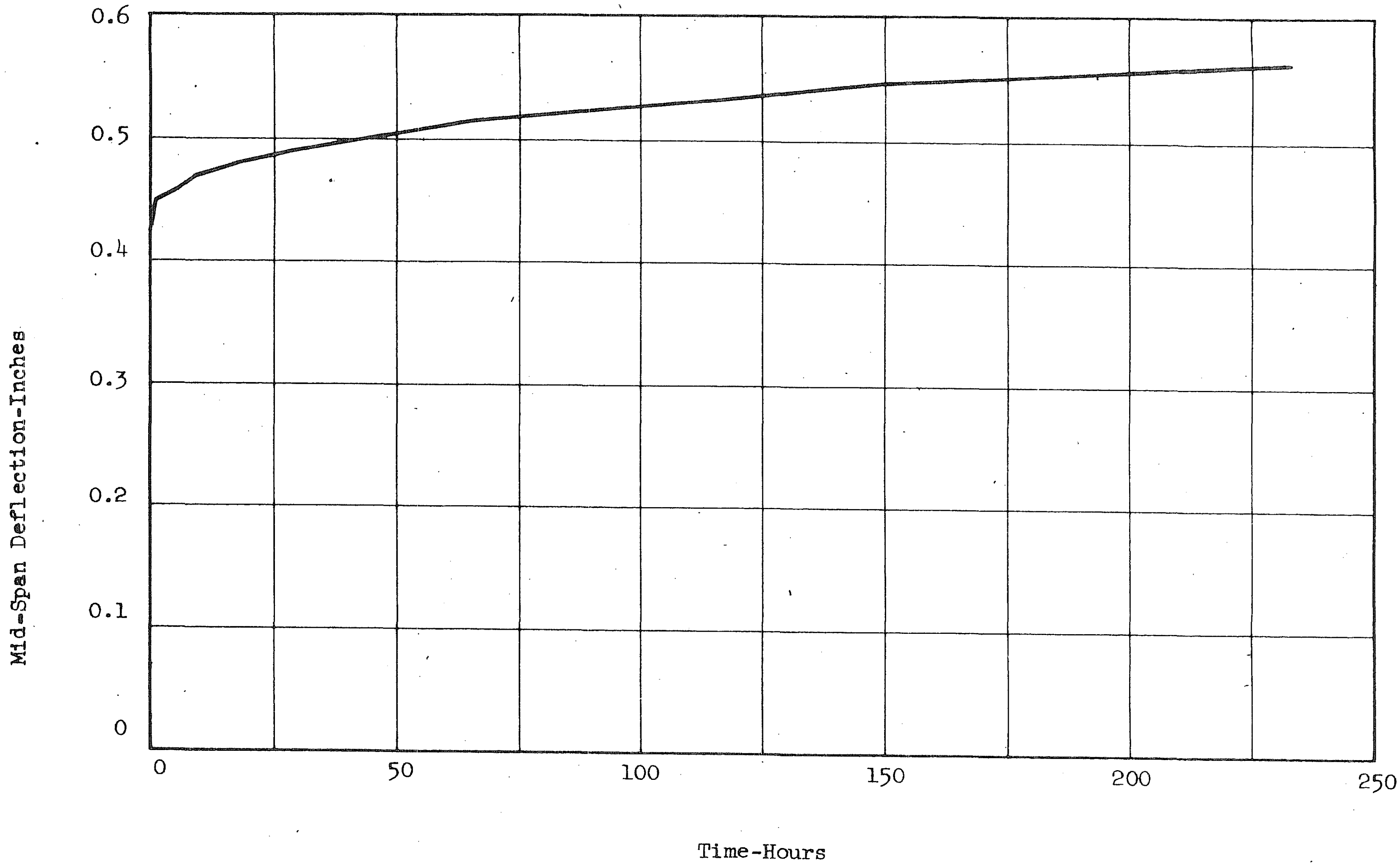


FIG. 19 TIME-DEFLECTION CURVE FOR BEAM LB-1 P=30 kips

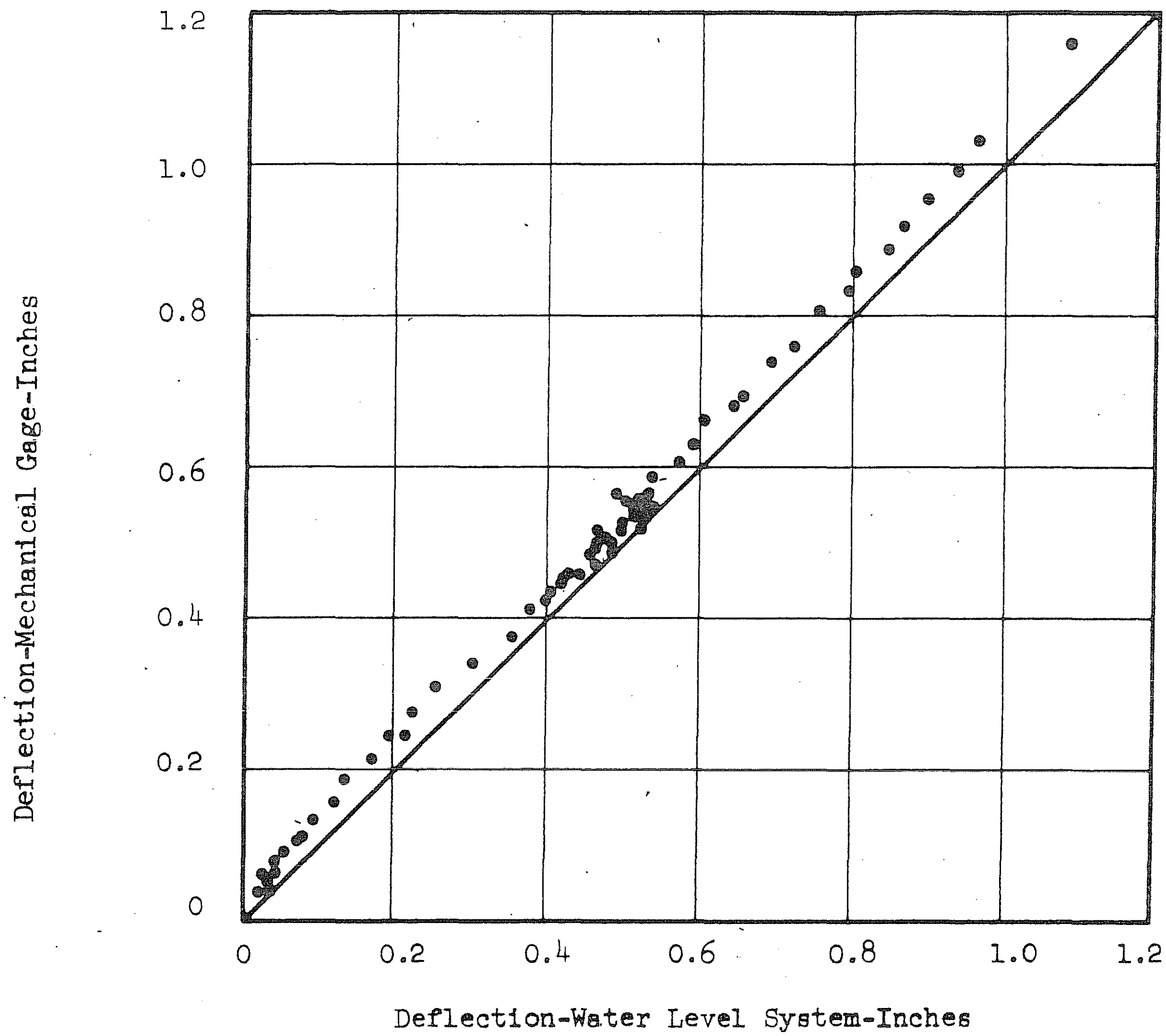


FIG. 20 COMPARISON OF MECHANICAL AND WATER-LEVEL DEFLECTION MEASUREMENTS

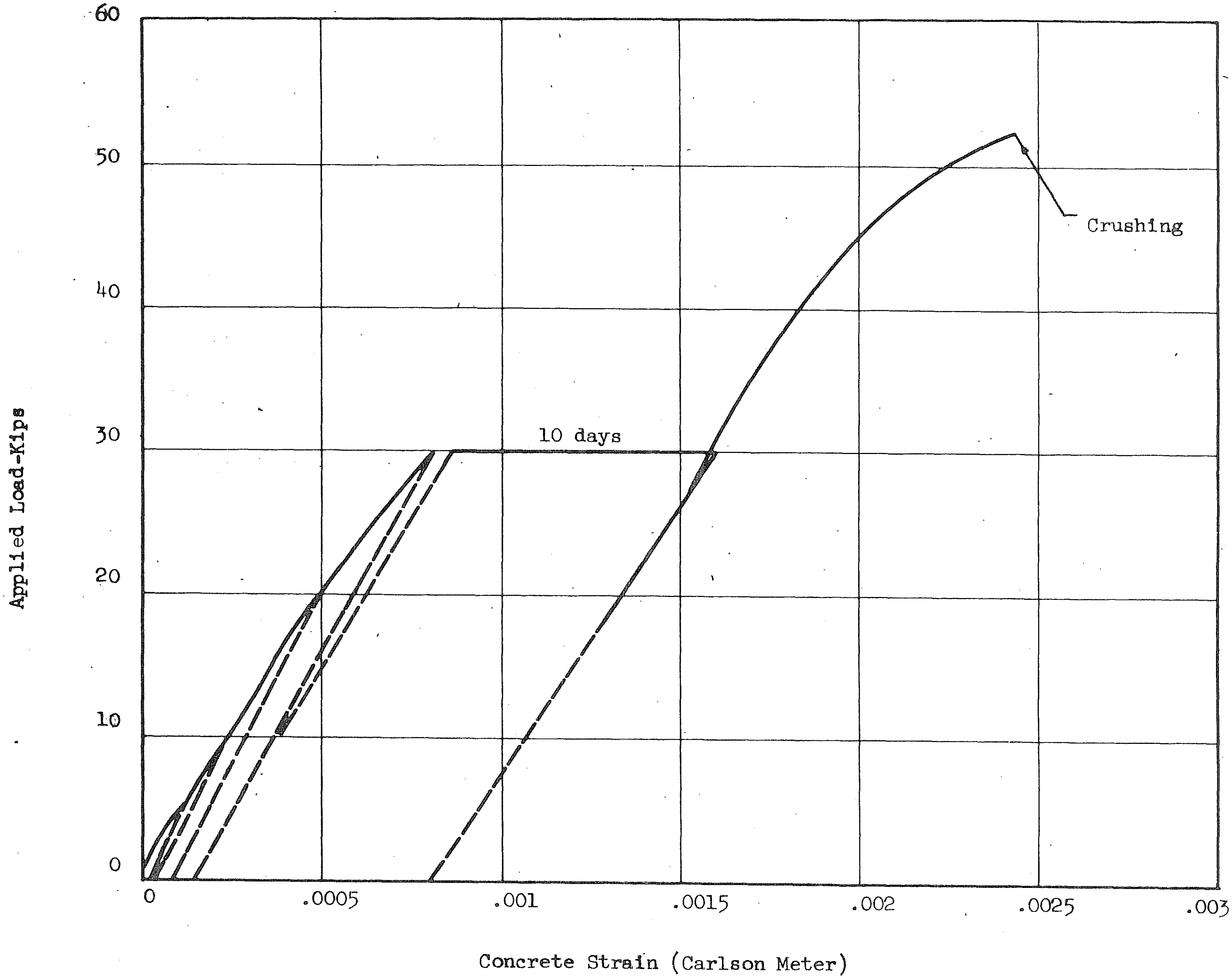


FIG. 21 LOAD-CONCRETE STRAIN CURVE FOR BEAM LB-1

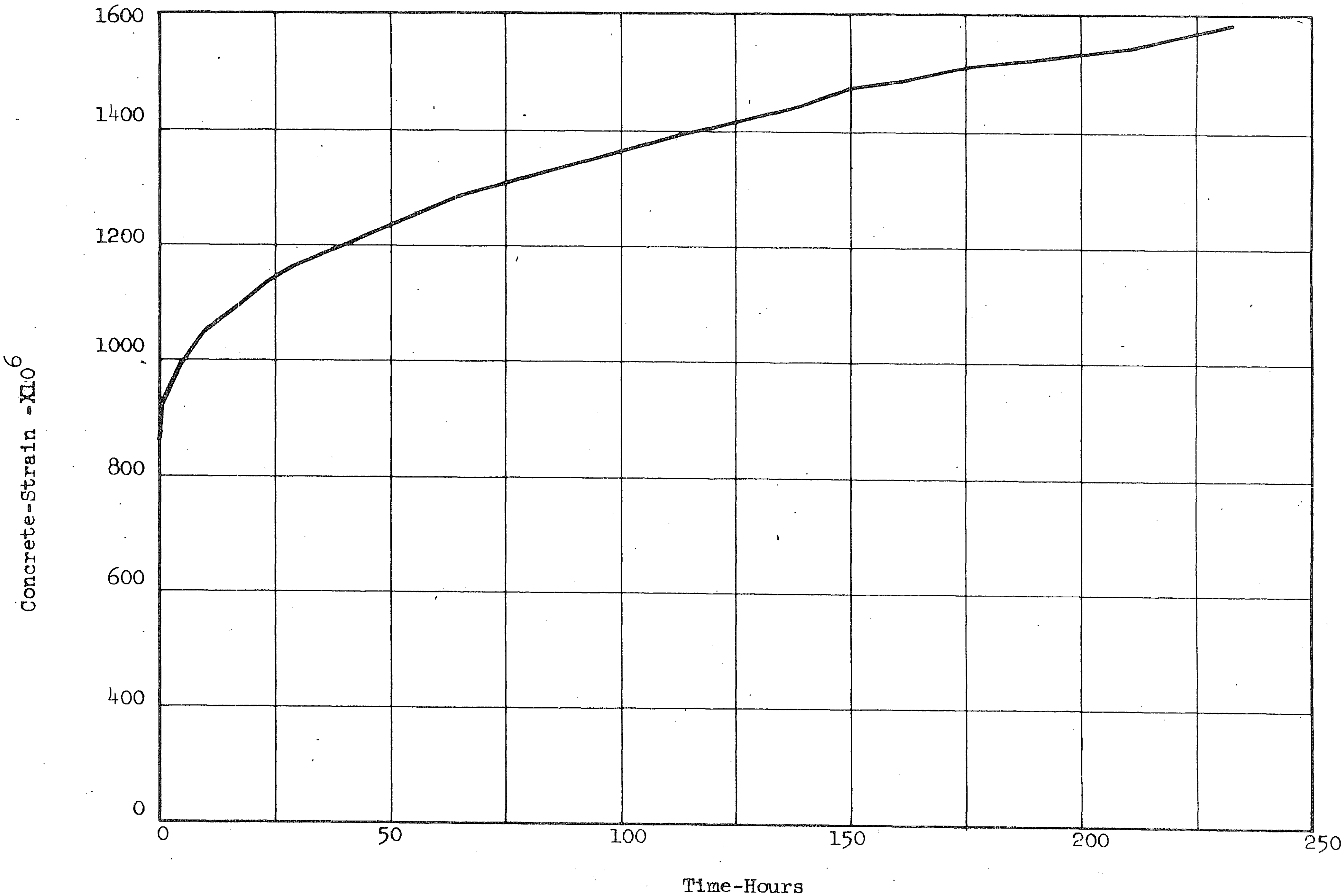


FIG. 22 TIME-CONCRETE STRAIN CURVE FOR BEAM LB-1 P = 30 kips

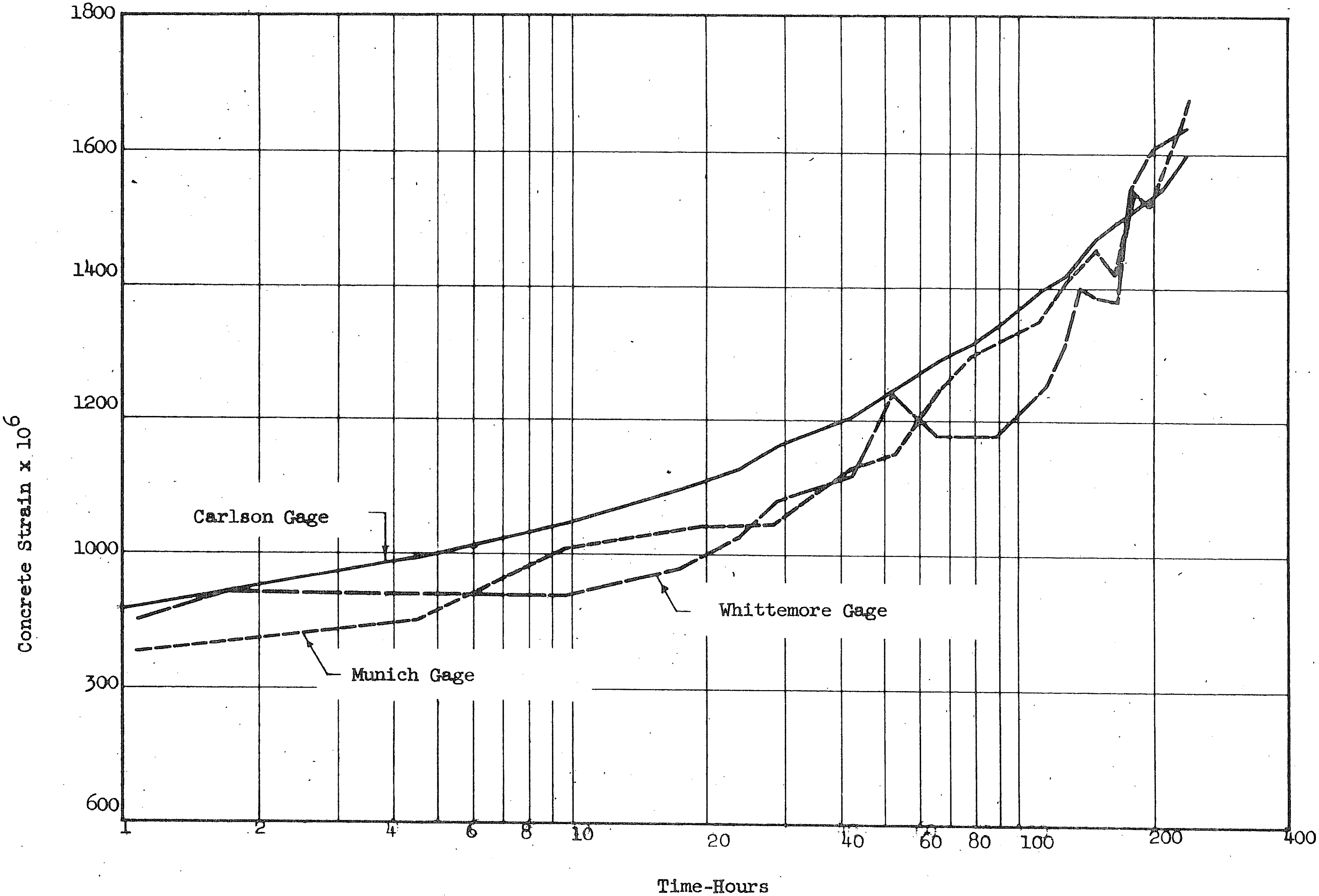


Fig. 23 COMPARISON OF STRAIN-GAGE READINGS DURING SUSTAINED LOADING

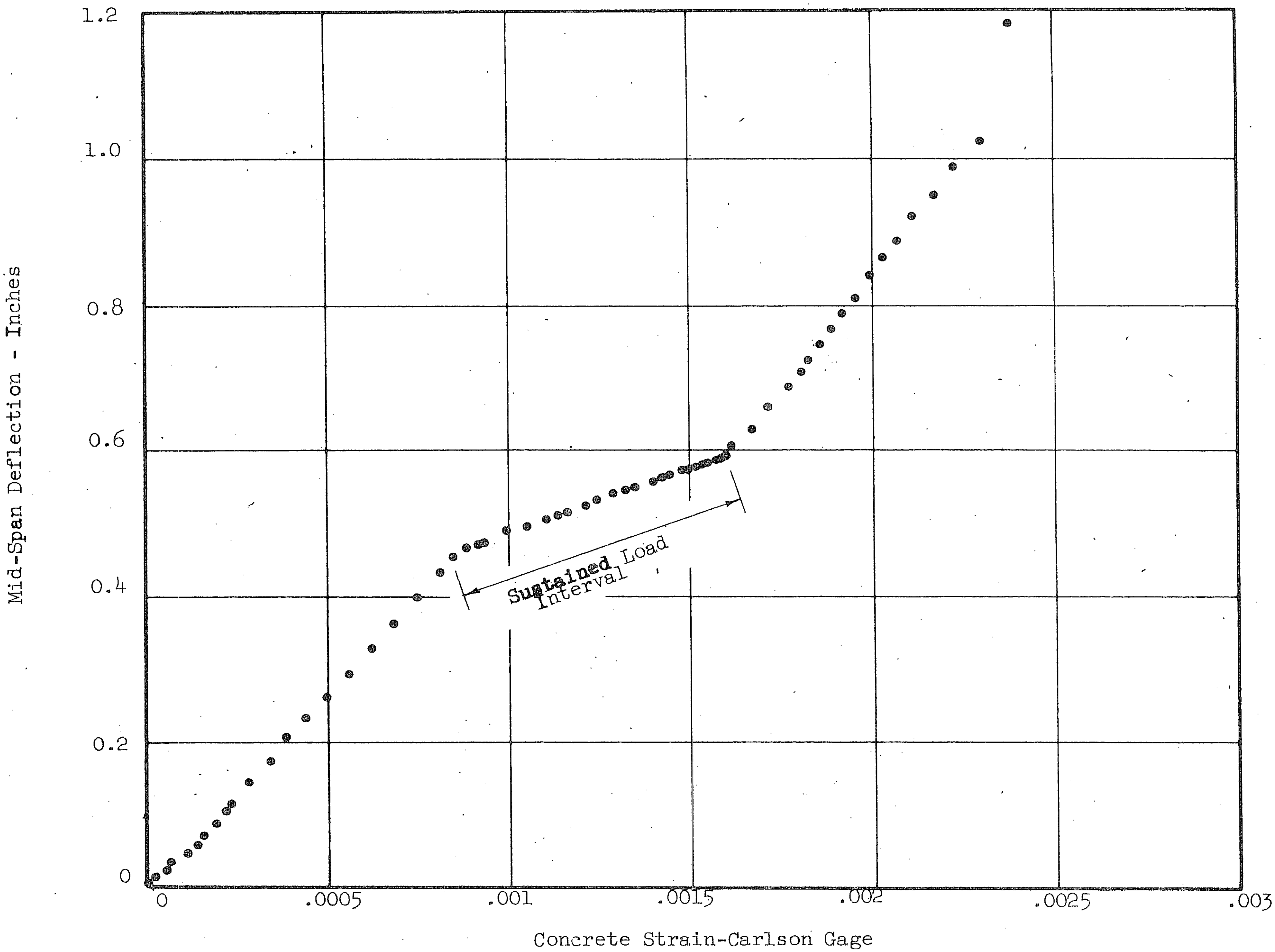


FIG. 24 DEFLECTION-CONCRETE STRAIN RELATIONSHIP, BEAM LB-1