CREEP AND SHRINKAGE STRAINS IN PRESTRESSED CONCRETE USING TWO TYPICAL INDIANA AGGREGATES

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by
E.J. MIDGAARD

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

FINAL REPORT

CREEP AND SHRINKAGE STRAINS IN PRESTRESSED CONCRETE USING TWO TYPICAL INDIANA AGGREGATES

TO: K. B. Woods, Director
Joint Highway Research Project

January 24, 1957

FROM: Harold L. Michael, Assistant Director

File: 5-13-1
C-36-58 A

Attached is a final report entitled, "Creep and Shrinkage Strains in Prestressed Concrete Using Two Typical Indiana Aggregates." The report has been prepared by Mr. E. J. Midgaard, initially under the supervision of Professor G. M. Nordby and recently under Professor M. J. Gutzwiller.

This report is the first presented since initiation of a structural area of the Project. The study presents some valuable information for prestressed concrete that used two typical Indiana aggregates.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Assistant Director
Joint Highway Research Project

HLM:hgb

Attachment

cc: D. S. Berry
J. R. Cooper
J. T. Hallett
G. A. Hawkins
F. F. Havey
G. A. Leonards
R. E. Mills
B. H. Petty
Lloyd Poindexter
C. E. Vogelgesang
J. L. Waling

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by

Einar Johan Midgaard
Graduate Assistant

Joint Highway Research Project
Project C-36-58 A
File 5-13-1

Purdue University
Lafayette, Indiana

January 24, 1957

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The research facilities were those of the Structural Department of the School of Civil Engineering at Purdue University. Research was under the general supervision of Prof. Kenneth B. Woods, Head, School of Civil Engineering and Dr. Joseph L. Waling, Professor of Structural Engineering, and under the immediate supervision of Gene M. Nordby and Martin J. Gutzwiller, Associate Professors of Structural Engineering. To Professors Waling, Nordby and Gutzwiller, for their helpfulness and suggestions, the writer is greatly indebted and gives thanks.

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ABSTRACT

Midgaard, Einar Johan. M.S.C.E., Purdue University, January 1957. "Creep and Shrinkage Strains in Prestressed Concrete Using Two Typical Indiana Aggregates." Major Professor: Martin J. Gutzwiller.

This investigation is concerned with the plastic flow, here called creep, in prestressed concrete having the aggregate as the varying factor. The aggregate used was limestone from Ohio and Indiana Stone Company, Greencastle, Indiana and glacial gravel from Western Indiana Sand and Gravel Company, West Lafayette, Indiana. Four different stress levels were used in the concrete. The creep was measured on unloaded beams or in other words the creep due to the prestress force was found. Special shrinkage specimens were made to differentiate between the creep and shrinkage strains. The readings were taken by means of gagepoints embedded in the concrete surface and a 10" mechanical strain gage.

Stranded wire of 3/8" and 1/2" was used as prestress tendons. The 1/2" strand previously has not been used in pretensioned prestressed concrete as it has been feared the high bond stresses expected would cause a slippage between the steel and concrete.

Electrical strain gages were used on the strands in order to check the prestress force and elastic shortening. However, the results obtained are questionable.

The results showed that a concrete having the glacial gravel as aggregate gave less creep than the concrete with limestone aggregate. None of the concretes will produce excessive prestress loss due to shrinkage and creep.

It is not likely that any slippage occurred between concrete and the 1/2" strand. More investigation is needed on this subject.

INTRODUCTION

During the last decade prestressed concrete has become a popular building material replacing ordinary reinforced concrete in buildings and bridges. It has also been used to substitute for steel or wood in items like fenceposts, telegraph-poles, railway ties, etc. In spite of its relatively young age a huge amount of literature exists already and still more research is in progress all over the world.

Prestressed concrete has a number of advantages over ordinary reinforced concrete. It is possible to obtain a crackless structure and hence the whole concrete cross-section is utilized in resisting the external bending moment. This enables the size of section to be reduced and it is possible to use longer spans than with ordinary reinforced concrete. Furthermore we can utilize high-strength steel and high-strength concrete which enables us to reduce dead weight.

The use of high concrete stress levels has introduced another factor which has not been so significant in ordinary reinforced concrete. This factor is the plastic flow under sustained load. Although not absolutely correct it has been customary lately to use the notation, creep, for this plastic flow. The word creep will therefore be used in this paper.

Most structures never get their full design load and if they do it is only for a short period of time. In structures of ordinary reinforced concrete, the concrete will rarely reach the design stress and the creep taking place will be small due to the low stress. The expected creep in such a structure may also be substantially reduced by the use of compressive reinforcement. (15)

In prestressed concrete the prestress force will induce the design stresses which will not be altered (here not thinking of the effect from shrinkage and creep) as long as the structure is unloaded. This high sustained stress will be the cause of appreciable creep. In the compression zone of the unloaded structure there is of course no compression steel to reduce this creep. For the case of the loaded structure the same arguments apply as for other structures. For handling purposes prestressed structures are usually supplied with some compressive reinforcement. Hence it can be concluded that the creep will be most troublesome in structures with temporary loadings.

When prestressing a concrete beam we apply a constant moment throughout the beam. This moment will induce a deflection upward called camber. With permanent loads this camber will be balanced by the load. But for structures as bridges, roofs and flooring, the camber has to be compensated for by topping as terazzo, etc. It is easily understood that the creep strain will have the same effect upon the beam as if an additional moment was applied giving still more camber. Excessive creep will thus produce excessive camber which may give a bad architectural effect and can even make the topping crack.

Another effect is that creep (and shrinkage) will reduce the prestress force. This is usually taken into consideration in design, but excessive creep will reduce the load the structure can take without cracking.

SCOPE OF TEST

The principal objective of this investigation was to find the influence of two widely used Indiana aggregates on creep in prestressed concrete beams. This investigation was also to include shrinkage, creep and elastic losses that will be expected in prestress design. Various stress levels were to be used in the concrete and the creep results obtained to be compared with the present theories of creep. As a secondary objective it was intended to secure additional information on bond, such as anchorage length and slippage, in particular in connection with the use of the $\frac{1}{2}$" strand which here was used in pretensioned prestressed concrete for the first time.

Most concrete aggregates in present use in Indiana have been thoroughly tested at Purdue for properties such as concrete strength, durability etc. This investigation is meant to supplement these present records.

PRESENT THEORIES ABOUT SHRINKAGE AND CREEP

Shrinkage

Before discussing creep the phenomenon of shrinkage has to be dealt with.

Shrinkage is dependent upon many factors. The most accepted theory at present is that the shrinkage is due to capillary pressure in the cement "gel" when colloidal water is evaporating. It is thus in the cement paste the shrinkage is taking place. Because of the effect of water, shrinkage has been found proportional to the amount of mixing water used. It follows also that the less paste used the less shrinkage is obtained, and the higher the drying rate, the more shrinkage takes place.

A proper mix design and steam curing can reduce the shrinkage substantially. Steam curing for instance has reduced the shrinkage by as much as 2/3.

When the cement paste shrinks the aggregate is put under compression and the physical properties of the aggregate will naturally effect the amount of shrinkage.

As a specimen dries out from the surface the shrinkage will put the outer shell in tension and the core under compression. The core will thus tend to reduce the shrinkage. In very thin specimens there will be little or no "core-action" and a higher shrinkage will occur. We can conclude that the rate of shrinkage is dependent upon the size of the specimen, or we may also say, the ratio between the circumference and the area of the cross-section.

Shrinkage is naturally dependent upon time and Kommendant (6) reports the following time relationship:

$$\frac{\varepsilon s\text{(Deferred strain due to shrinkage)}}{\varepsilon st\text{(Total strain due to shrinkage)}} = 1-e^{-t}$$

in which t is a time factor.

Creep

Creep is still not fully understood as to the physical changes concerned although recent research has yielded reliable information on the phenomenon. Most materials show creep under sustained load, the magnitude being affected by factors as temp., humidity etc.

Creep and shrinkage are closely related. One cannot evaluate the creep without having information about the shrinkage at the same time. For example it has been found that the creep is basically dependent upon the curing condition and the drying out conditions. Here the increase in the measured strains when the specimen is permitted to dry out is due to the higher rate of shrinkage. To measure the creep we must prevent the specimen from drying out, or have an unloaded identical specimen under otherwise the same conditions from which to obtain the shrinkage.

The curing time and the age of concrete affect the amount of creep. As these same factors affect the strength and the modulus of elasticity of the concrete it is natural to correlate the creep with the strength and modulus of elasticity. A higher strength concrete should under the same stress give a smaller amount of creep than a lower strength concrete. Roughly it can be said that stresses of the same percent of the ultimate strength will give the same amount of creep if the amount of paste is constant in each of the mixes.

But as the creep is taking place in the cement paste, the creep will be approximately proportional to the amount of cement-paste. An increased concrete strength obtained by increasing the amount of paste will not necessarily reduce the creep.

As the aggregate has an effect on the amount of shrinkage a similar effect might be expected for creep. Experiments have shown that this is true to a certain extent. Troxell and Davis (2) report 2 ½ times as much creep for concrete containing sandstone as that containing dense limestone. The creep of basalt concrete and granite concrete is 1 ½ times the creep of the corresponding limestone concrete. This indicates that creep losses will vary with local materials and must be investigated locally.

Creep and shrinkage is a long time effect and will theoretically go on indefinitely, but the measurable amount will usually reach a constant value after 2-3 years. There are various theories about the rate of creep. One of them is due to Cacout (4) who gives the following equation:

$$\frac{\text{(Deferred strain at age m months)}}{\text{Final deferred strain}} = 1-10^{-.4\sqrt{m}}$$

Note the similarity to the formulae of Kommendant for shrinkage. L'Hermite (4) gives the following values:

40% of deferred strain in 1 month
60% of deferred strain in 3 months
80% of deferred strain in 1 year
90% of deferred strain in 1½ years

These values are in substantial agreement with those of Cacout.

OUTLINE OF TEST PROCEDURE

Two identical beams were to be cast each time, the only difference being the aggregate. Altogether 8 beams or 4 pairs of beams were to be cast using four different stress-levels. The theoretical maximum stress levels would be 1440 psi, 2150 psi, 3150 psi and 2550 psi using 4-3/8", 6-3/8", 9-3/8" and 4-1/2" strands respectively as tendons. The resultant prestress force was to act 1/3 from the bottom. The strains were to be measured at this level corresponding to the theoretical stresses of 960 psi, 1430 psi, 2100 psi and 1700 psi. The beam size decided upon was 6" x 12" x 14' - 4".

Stainless steel gagepoints flush with the beam surface were used to provide the means of measuring the strains.

In view of the present knowledge of the creep rate strain readings were to be taken 1 day, 2 days, 4 days, 8 days, 15 days, 29 days, 60 days and 120 days after release. The last four beams didn't have the age of 120 days at the time the final readings were taken.

At release the camber developed by the elastic strain was to be measured. Although not originally planned, arrangements were made to measure the change of camber with time due to creep.

The beams were numbered consecutively from 1 to 8, beams with the same type of aggregate having the numbers 1, 3, 5, 7 or 2, 4, 6, 8. The 1/2" strands were to be used in beams No. 7 and No. 8. As will be later described arrangements were made to check whether any significant slippage would take place between the concrete and the 1/2" strand upon release.

To evaluate the amount of shrinkage at any time a shrinkage specimen was to be provided for each beam. These specimens had the same cross-section as the beams but only half of the length. Two gage points were supplied on all four faces symmetrical with respect to the centerline.

To get a double check of the prestress force and to evaluate the elastic shortening upon release electric strain-gages were to be placed on the strands. Sufficient waterproofing was to be provided in order to protect the gages from the wet concrete.

DESCRIPTION OF MATERIALS USED

Aggregate

The aggregates chosen were a limestone and a glacial gravel. The limestone was taken from the Ohio and Indiana Stone Co., Greencastle, Indiana. It has a very good field performance record and numerous tests have shown its high quality.

The gravel was taken from the Western Indiana Sand and Gravel Co., West Lafayette, Indiana. Qualitatively it doesn't range as high as the limestone but it has a fairly good field performance record. The limestone beams were given numbers 1, 3, 5, and 7 and the gravel beams were given the numbers 2, 4, 6, and 8.

It was decided to use a maximum size of 3/4" for the aggregate. A better "aggregate effect" might have been obtained with 1" maximum size, but this could possibly have caused some trouble when pouring the 9 strand specimens as the strands were quite close.

Table 1 and Fig. 1 and 2 show the results of the sieve analysis of the two aggregates.

Sand

The sand used was taken from the sand bin in the laboratory. It is delivered to the school from Western Indiana Sand and Gravel Co., West Lafayette, Indiana. The result of the sieve analysis is found in Table 1 as well as in Figs. 1 and 2. It can be seen that it is a little short of finer

TABLE 1

SIEVE ANALYSIS OF SAND, LIMESTONE AND GRAVEL

Percent Passing

Screen Size	Sand	Limestone	Gravel
3/4		99	88.3
1/2			
3/8	100.0	49.0	3.6
4	99.2	12.6	
8	86.75	2.7	
16	61.70		
30	27.30		
50	7.7		
100	2.1		
Fineness Modulus	3.15	6.37	7.07

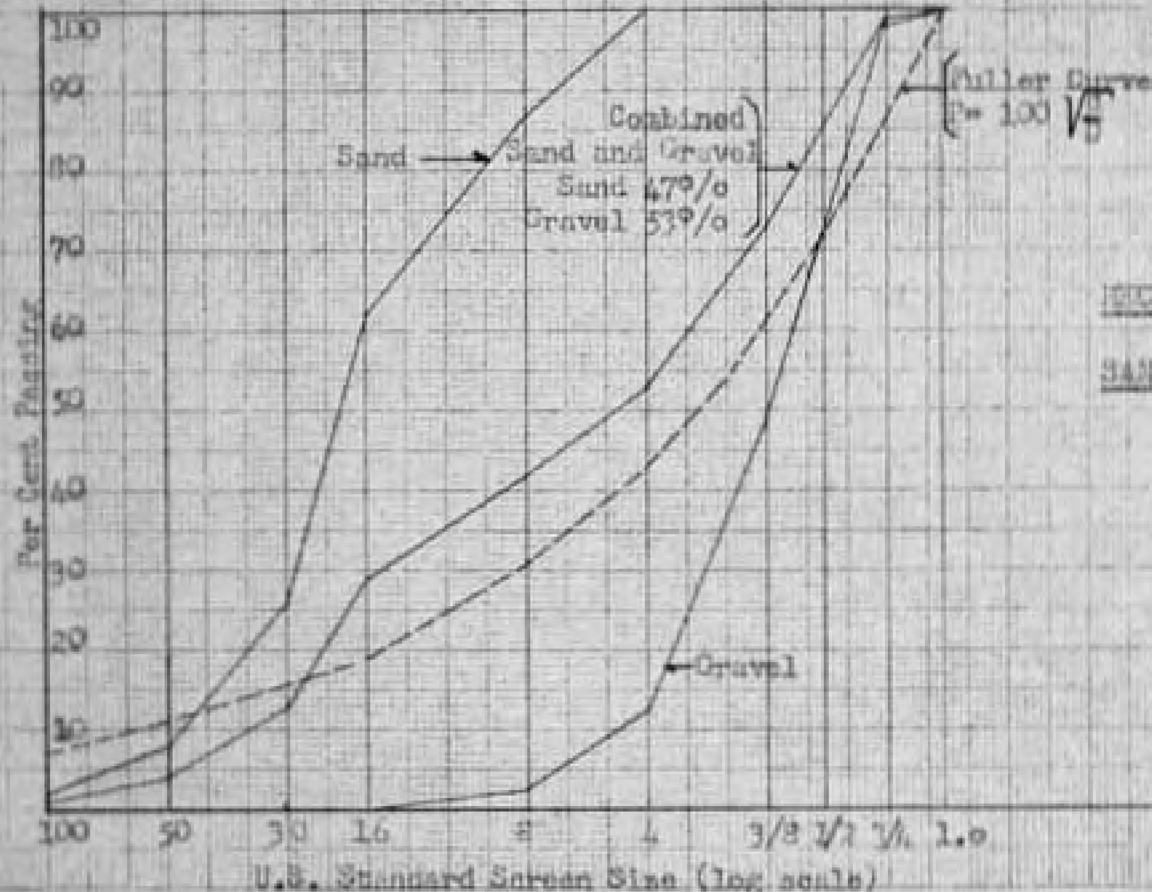


FIG. 1

MECHANICAL ANALYSIS

OF

SAND AND LIMESTONE

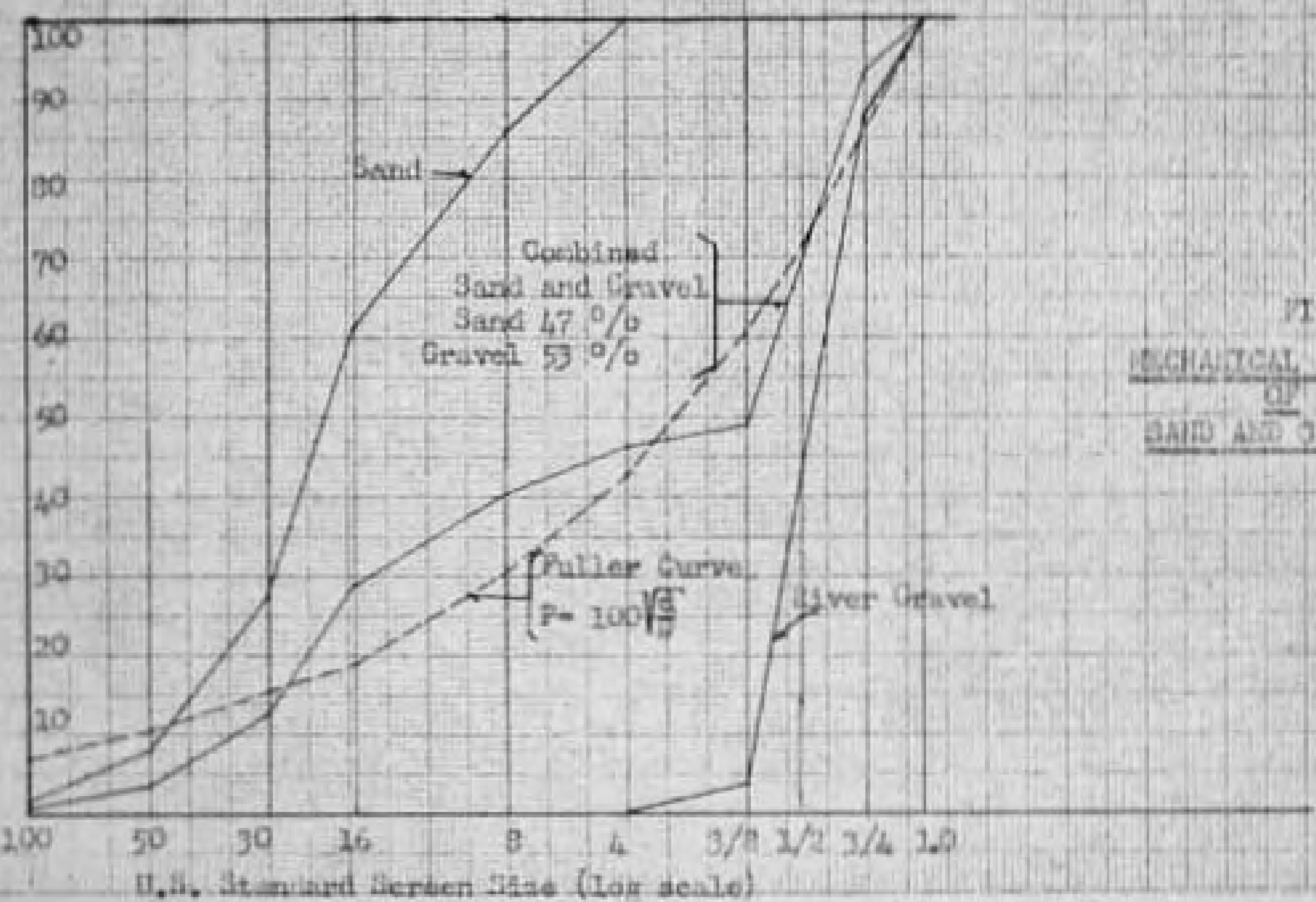


FIG. 2

MECHANICAL ANALYSIS
OF
SAND AND GRAVEL

fractions which means it will need more cement than an ideal sand. This was not found to be too objectionable.

Cement

As the next pair of beams could not be cast before the preceding pair had been released and stored away, the use of ordinary Portland cement would have been too time-consuming. With the use of high early strength cement the curing time could be reduced to one fourth. A somewhat higher shrinkage would occur, but it was decided to use the high early strength cement as high early strength cement is very likely to be used in prestressed concrete. If the lab. had been equipped with steam curing equipment ordinary Portland cement could have been used.

Concrete Mix

Prestressed concrete requires higher strength than generally can be expected from ready-mix. As it also was desired to get well controlled concrete in all beams, a very close control with the mixing was necessary. The use of ready-mix would not have provided the necessary control. The small batches involved, approx. 12 cubic feet, appeared rather small for a truck. It was decided to mix the concrete in the lab, and a mix had to be designed.

All trial mixes were made by hand. Effort was made to get a high strength concrete with good workability. The minimum strength was set at 5000 psi and a slump of one inch was intended. The various mixes tried are listed in Table 2. The sand and the aggregates were stored indoor

TABLE 2

DATA FOR TRIAL MIXES

Number	Proportions	w/c	Slump	Ultimate Strength	Cure Conditions	Remarks
1	1:2.07:2.36	0.477	0	5280	7 days moist	Limestone
2	1:1.86:2.57	0.472	0	4510	7 days moist	Limestone
3	1:2.07:2.36	0.457	0	5340	7 days moist	Gravel
4	1:1.86:2.57	0.493	¼	4265	7 days moist	Limestone
5	1:2.07:2.36	0.515	2½	4920	7 days moist	Limestone
6	1:2.07:2.36	0.505	1¼	5010	7 days moist	Gravel
7	1:1.86:2.57	0.494	1 3/4	5035	7 days moist	Gravel
8	1:2.07:2.36	0.515	1½	5500	7 days moist	Gravel
9	1:2.07:2.36	0.515	1½	5060	7 days moist	Limestone

TABLE 3

SLUMP AND CYLINDER TESTS FOR CONCRETE IN BEAMS

Date of Pouring	Beam No.	Slump	Type Curing	Ultimate Strength
6-21-56	1	1 1/2	7 days moist	5800 psi
7-9-56	3	1 1/2	7 days moist	5270 psi
7-28-56	5	2 3/4	9 days moist	5800 psi
8-14-56	7	1 1/2	9 days moist	6300 psi
6-21-56	2	2	7 days moist	5640 psi
7-9-56	4	2	7 days moist	5240 psi
7-28-56	6	3	9 days moist	5450 psi
8-14-56	8	2 3/4	9 days moist	6000 psi

and maintained an almost constant water content. This was a great help, of course, in maintaining a constant quality of the concrete.

It was expected that both slump and strength would increase when the concrete was machine-mixed. This proved to be particularly true for the gravel concrete with regard to the slump.

Trial mixes 8 and 9 were found satisfactory. As the mixer had a capacity of about two and a half cubic feet the charges to be used would be composed of the following quantities: 62.2 lbs. cement, 32.0 lbs. water, 129 lbs. sand, and 147 lbs. coarse aggregate. Table 3 shows the average slump and strength for the beams. The higher slump values for beams 5 and 6 is most likely due to more moist sand. The inner part of the sand heap is more moist and the men filling the sacks took the sand from there to be less bothered by dust.

Tendons

As previously mentioned 3/8" and 1/2" stranded wire was used. The steel strand has several advantages in prestressing work. First, it combines several wires into one element which can be tensioned as a unit eliminating the need for tensioning seven or more separate wires in its place. In addition, it is claimed that the spiral grooves around the strand add greatly to the bond by mechanical anchorage.

The strands are composed of seven wires, one of them being the core. The modulus of elasticity of the strands was not determined as part of this project as it is a very tedious operation. The manufacturer gives 28,000,000 psi as an average. Other tests give 28,750,000 psi (10) and 28,700,000 (7). In this test the jack-dials were used as the main load-indicator and the measured elongation was used only as a check using the manufacturer's value of E. Other pertinent information is given in Table 4.

TABLE 4

DATA FOR TENDONS

Nominal Diameter	Area (square inches)	Ultimate Strength lbs.	Design Load lbs.	Tensioning Load lbs.
3/8"	0.0799	20,000	11,200	14,000
1/2"	0.1438	36,000	20,160	25,200

Recent literature reveals that the 3/8" strand is widely used commercially in pretensioned prestressed concrete and has proved reliable. The one half inch strand has only been used in post-tensioned prestressed structures. Looking at the Table 4 we see that the tensioning load of one-half inch strand is 1.8 times the load of the 3/8" strand while the diameter is only 1.33 as large. To anchor the load in a 1/2" strand a substantially higher bond stress is expected to develop. It has been feared that this high bond stress will lead to a slippage between the concrete and the steel, and for this reason designers have considered 3/8" strand as a ceiling value in pretensioned prestressed structures.

FABRICATION EQUIPMENT

Prestressing Bed

No means of producing pretensioned prestressed items existed at the University at the time this experiment was planned. A prestress bed had to be designed and built before the experiment could start.

Certain restrictions were encountered in the design of this prestressing bed. The fund allocated for this project was limited and economic considerations weighed heavily in all the phases of planning. A large amount of the space in the laboratory was already occupied by other projects in progress and the beams were to be cast end to end thus requiring a free length of approx. 40 feet.

The reasons for casting the beams end to end were the following:

1. The capacity of the jacking equipment could be reduced.
2. The two beams would get exactly the same prestress force.
3. The effect of differential slippage in the anchorage grips would be reduced by about 50%. The strand always slips somewhat before the grips "bite in". A differential slippage will produce a difference in stress in the strands. This difference in stress will be $\Delta\sigma = \Delta\epsilon E$ where $\Delta\epsilon = \frac{s}{\ell}$ is the diff. slip per unit length of the strand. The difference in stress due to this slippage therefore will decrease with increasing length.

The only space available was the area on two sides of the 600,000 lbs. testing machine. The strand would have to pass between the legs of the machine thus obstructing the use of the lower part of the machine. Advance planning and coordination with the other projects was worked out so that the use of the machine could be arranged in the periods when the strands didn't close it off.

In the floor of the Structures Laboratory there are a number of holes that can be used for various loading arrangements. The holes are of two types, one with threads for vertical loads and another with smooth surface for horizontal loads. All of the holes are three feet deep and approximately three inches in diameter, made of ordinary unfinished steel pipe. The holes were used to take up the prestress load in the following way.

Steel plugs, twelve inches long, were made to fit the thrust holes. A wooden peg was first inserted to support the plug and make it protrude two inches above the floor level. Tiedown bolts were made for the threaded holes. A plug and a bolt in place are shown in Figure 3 and to show the real dimensions a duplicate has been placed beside each.

On top of the bolt and the plug was placed a 24 inch high wide flange girder as shown in Figure 4. The bottom flange had been turned out to fit the plug exactly while some clearance was allowed for the bolt.

At this stage of the assembling of the prestressing bed we would have four girders, two at each end of the room, six feet apart and parallel with the direction of the future concrete beams. A set of crossbeams was bolted to the girder

FIG. 3 PLUG AND TIEDOWN BOLT FOR PRESTRESSING BED

FIG. 4 GIRDER FOR PRESTRESSING BED

as can be seen on Figure 5 which shows the fixed end of the prestressing bed. The jacking end had in addition a movable set of crossbeams, Figures 6 and 7.

The jacks were not expected to hold the load for days and as they might be needed elsewhere some locking device was needed; they were made of steel tubing and could be adjusted to any required length.

The girders and beams were borrowed from the University Physical Plant and fabricated to the proper dimensions in the machine shop and later assembled in the laboratory. To provide for adjustments in the connections high strength bolts were used and the holes were given an oversize of 1/8" instead of the usual 1/16".

The following procedure was used to place the desired load on the strands.

The strands were inserted through the sandwich plates at the fixed end. The washers and anchorage grips were placed on the strand. The positioning plates and endplates for the forms were then placed on the strands and the free end of each strand was inserted through the sandwich plate at the jacking end. The positioning plates were made of plywood and were placed next to the sandwich plate and between the flanges of the crossbeams. The endplates were blocked up until the strands would be horizontal upon tightening. Using a simple lever arm system, Figure 8, one strand at a time was stressed and the washer and anchorage grip pushed fully home. It was hoped in this way to get approximately the same initial

FIG. 5 FIXED END OF PRESTRESSING BED

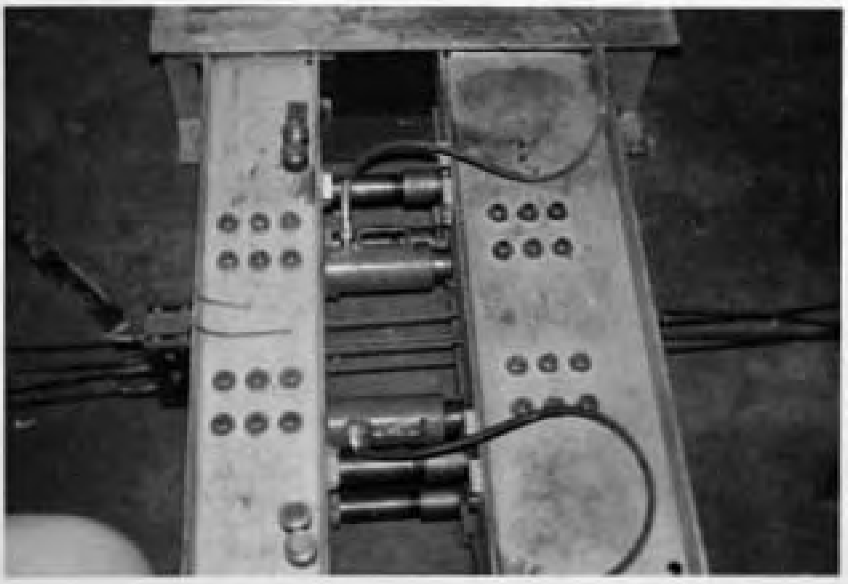


FIG. 6 JACKING END OF PRESTRESSING BED

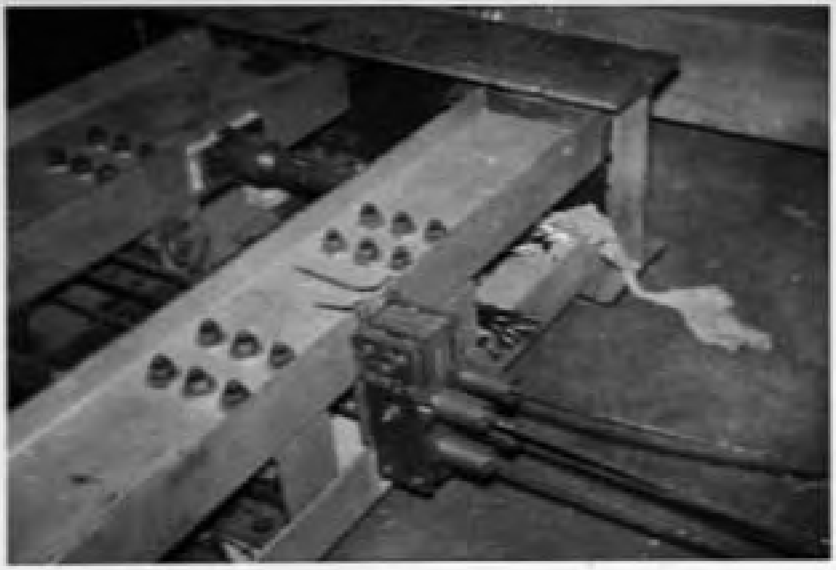


FIG. 7 JACKING END OF PRESTRESSING BED

FIG. 8 PRELIMINARY STRESSING OF STRANDS

load in each of the strands. During this operation the jacks were fully collapsed.

The jacks were then applied up to full load to make sure that all slip occured in the grips and make them "bite in". After that the load was usually reduced to some value less than the desired load. It was experienced during the jacking operation that a larger distance between the jacks would have been beneficient. But this would have complicated if not rendered impossible the assembly of the bed.

The two ends of the prestressing bed were exactly underneath the crane-rail. This simplified the operations rather considerably as it was operated with loads around 600 pounds. Special care was maintained to eliminate any possibility for accidents.

Sandwich Plates (Figure 9)

The sandwich plates were made from 4 1/2" x 3/8" x 24" plates obtained from a previous rivet-project. The small thickness, 3/8", made it necessary to provide washers on the strand to distribute the loads on to the pertinent plates. Plates in groups of four were machined down to exactly the same width to secure good load distribution. Small pieces of pipe were used to provide the intervals for the strands. Five bolts, two at each end and one at the center, held the plates together. The washers were 2" x 2" x 1" steel plate.

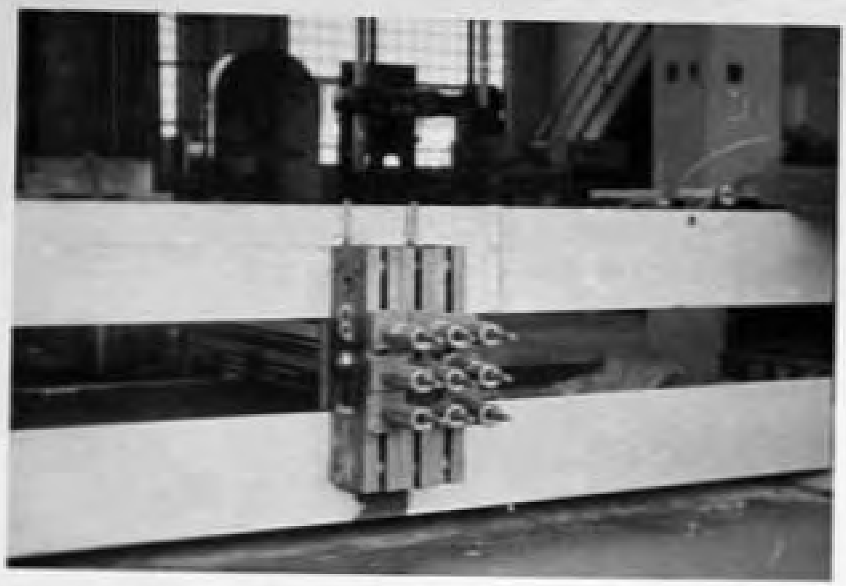


FIG. 9 SANDWICH PLATES

FIG. 11 SHIMMING OF BOTTOM SECTION OF FORM

Anchorage Grips

The main body of the grip is a cylinder, the inner surface of which is conical. Three steel wedges lie between the strand and the cone. When loading the cylinder, i.e., when jacking, the cone will press the wedges against the strand. The wedges have "teeth" that bite into the strand and secure good anchorage.

Grips for 3/8" strand were purchased but the anchorage grips for the 1/2" strand (Fig. 7) had to be manufactured and fabricated to fit in the machine shop.

Jacks

Blackhawk S-80 Porto Power 50 ton hydraulic jacks with total load gages at the pump were used.

At 35 tons, the maximum load in this experiment, the jacks became difficult to operate. Except for a defective valve they worked quite satisfactorily. The dialgages were adjusted in the testing machine immediately before each load application. The dial gage did not show correct load readings for load applied over the full range of the gage. It was adjusted to show the correct load in each case.

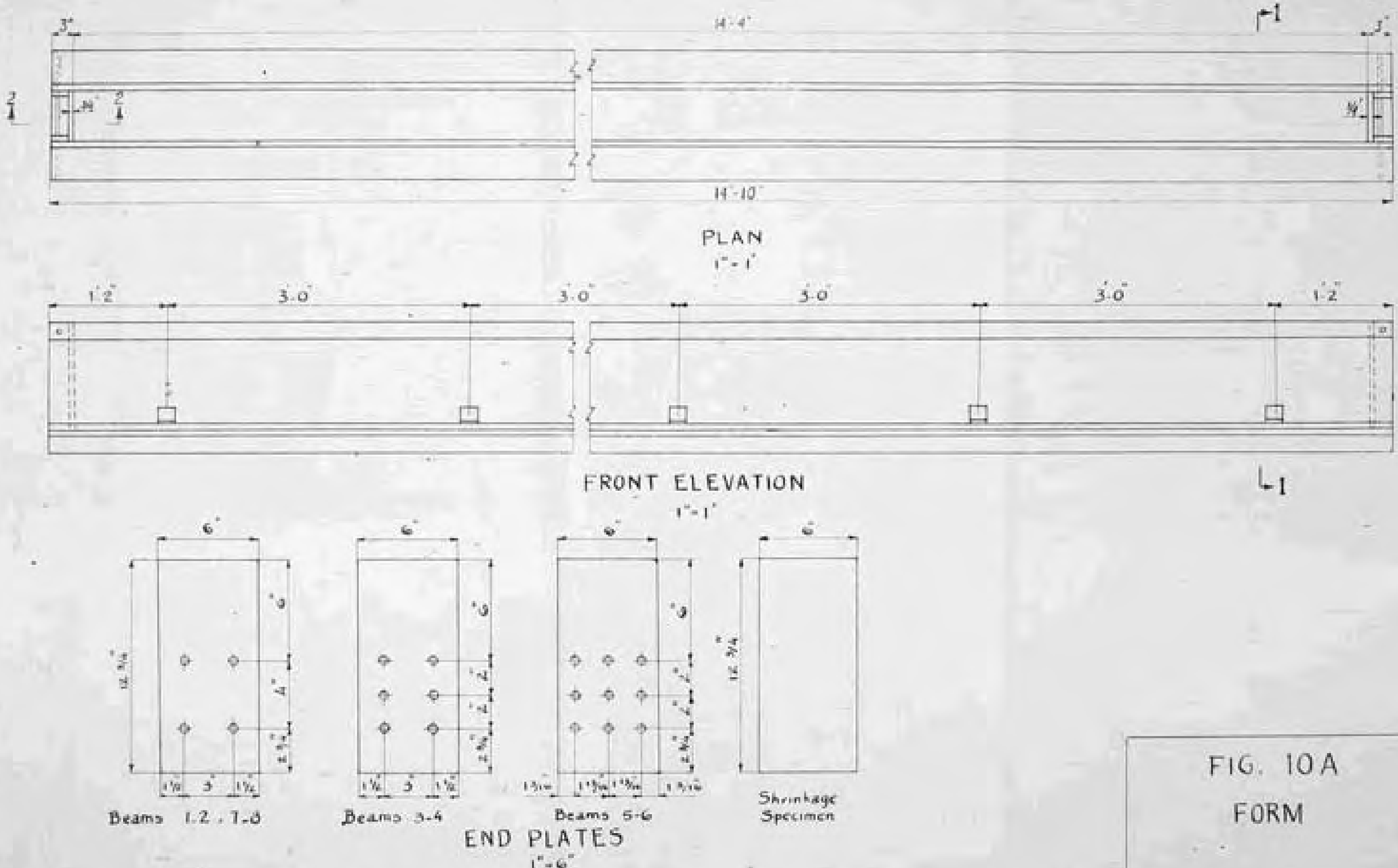
The jack piston had an engraved scale to show the elongation. This was used to check the elongation of the strands. This provided a second check for the load as we had no experience with the dependability of the jacks. To base the load upon the elongation was impossible. The stress and strain in the strands at the instant they obtained a straight configuration could not be determined.

The jacking operation was time consuming and numerous small adjustments had to be made before everything proceeded as planned.

Forms

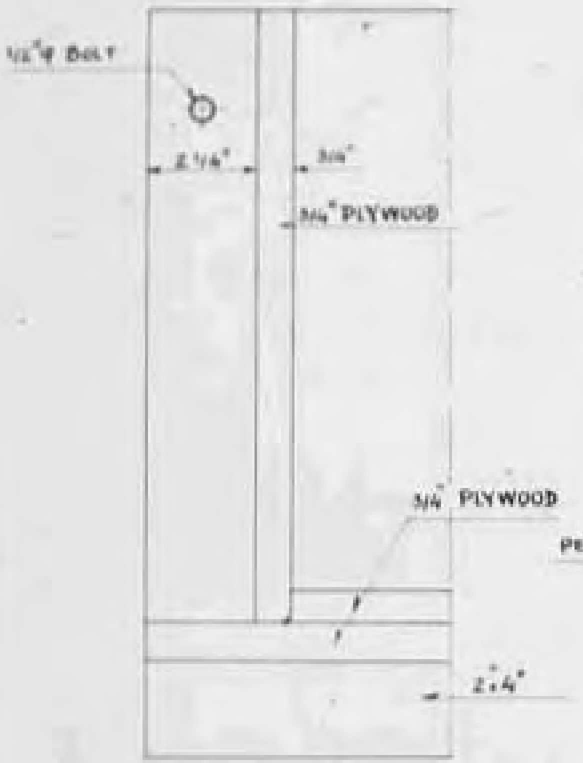
One set of forms was to be used for all four sets of beams in the present project as well as other sets of beams in a future project. Because of the necessity of re-using the forms it was necessary to be able to replace worn parts of the form (Figs. 10A and 10B).

The distance between the lower face of the beam and the loaded strands was adjusted by the use of planks and shims (Fig. 11).



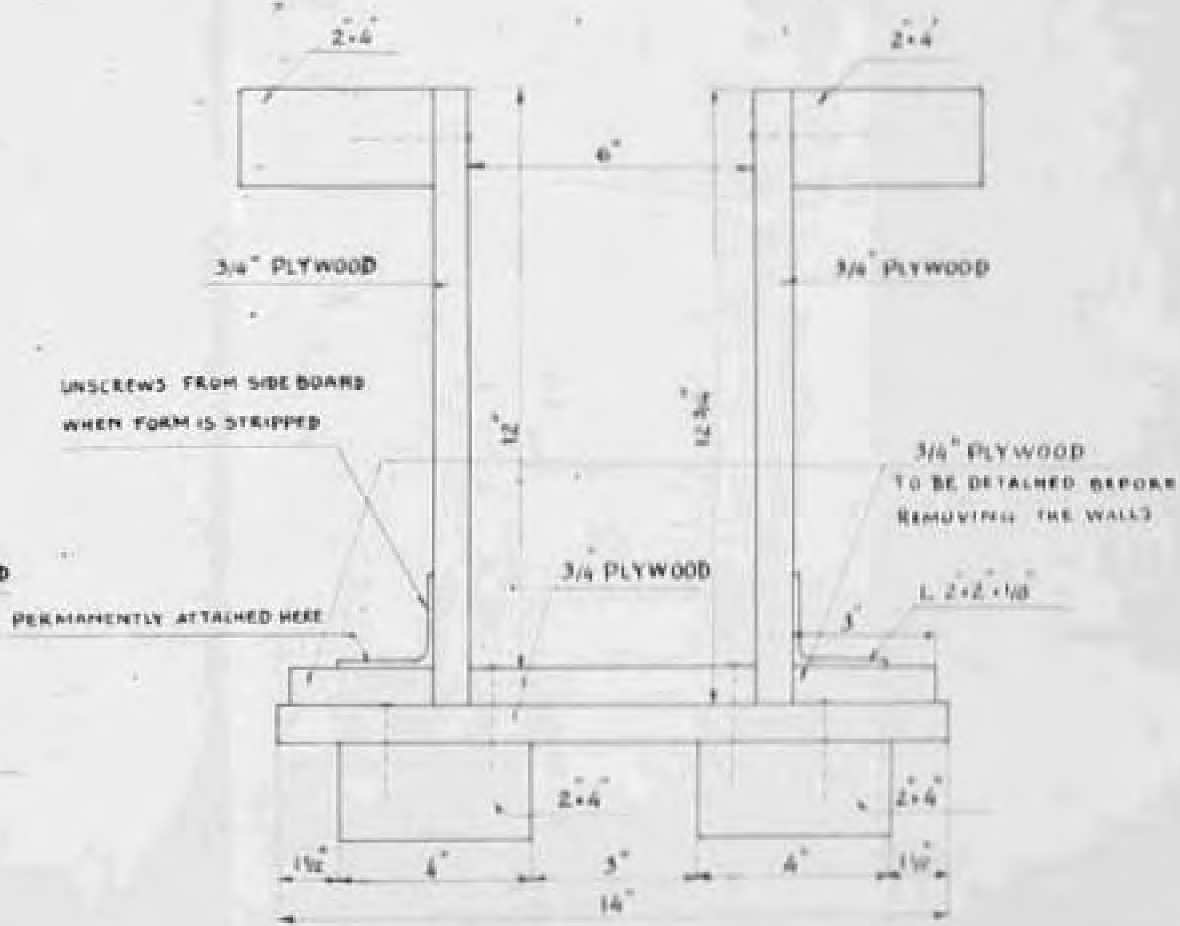
14'-4"
14'-10"
PLAN
1" = 1'
1'-2"
3'-0"
3'-0"
3'-0"
3'-0"
1'-2"
FRONT ELEVATION
1" = 1'
Beams 1-2, 7-8
Beams 3-4
Beams 5-6
Shrinkage Specimen
END PLATES
1" = 6"

FIG. 10A
FORM



SECTION 2.2

1" = 3"



SECTION 1-1

1" = 3"

FIG. 10B

FORM

DETAILS

PROCEDURE FOR FABRICATION OF ONE SET OF BEAMS

After the strands had been properly connected with the prestressing bed the electric straingages were applied to the strands. The baseplate of the testing machine was greased and then covered with multiple layers of tarpaper. Tarpaper was also put on the floor under the forms. If not already done the strands were now given an overload of ten per cent for about an hour to take out the creep in the steel. The bottom sections of the forms were thereafter blocked and shimmed up to the correct level. Sidewalls, fitted with gagepoints, were put in place, bracing fixed, and the forms were ready for pouring. Before the forms were assembled the strands were cleaned with carbon tetrachloride to ensure that the strands were free of oil. The forms on the other hand were well oiled.

The mixer used was a batch-mixer with a capacity of approximately 2.5 cubic feet. The charges were already sacked in the basement and brought up before mixing started. Water had to be weighed up for each batch. Five men were the usual crew. One operated the mixer, one the vibrator, one handled the wheel barrel and two men were making slump tests and test cylinders. Slump tests were made for every second batch. Ten cylinders were made for each beam. The number was set that high as nothing was decided definitely, as to what to do with the beams when this project was finished. All limestone cylinders were numbered from 1 to 40 and all gravel cylinders from 51 to 90. For example, beam number 3 would have cylinders 11 through 20 and beam number 8, cylinders 81 through 90. As will be recalled the limestone beams had the numbers 1, 3, 5, 7, whereas the gravel beams had the numbers 2, 4, 6, and 8.

All mixing and pouring was done within one and one half hour, rinsing and general clean-up took another hour and one half.

After a couple of hours the top of the beams were covered with sacks and kept moist for the whole curing period. (Figures 12 and 13).

At the day of release two cylinders for each beam were broken to determine whether or not the concrete had sufficient strength.

The forms were then stripped and the rest of the cylinders brought out of the moisture room. Readings for electric strain gages, with mechanical strain gage between gagepoints and for camber determinations were taken and the beams could be released. The release was done by transferring the load to the jacks again and then letting the jacks collapse slowly. All readings were then repeated. To find the modulus of elasticity at release, modulus of elasticity tests were run on four more cylinders. (Figure 14) For the last four beams the modulus of elasticity tests were made in the big testing machine. That meant that these tests could not be performed before the strands had been cut and the beams moved aside. With beam numbers 5 and 6 the man with the abrasive saw did not show up, and the modulus of elasticity test had to be postponed until the next day.

FIG. 12 BEAMS IMMEDIATELY AFTER POURING

FIG. 13 AFTER POURING, BEAMS COVERED UP

FIG. 14 MODULUS OF ELASTICITY TESTS

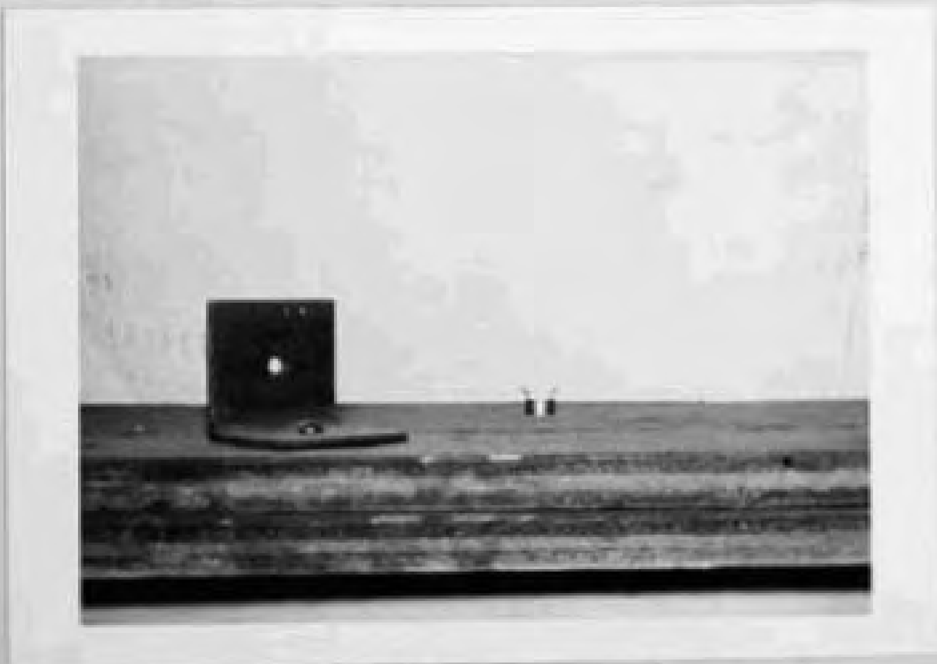


FIG. 15 GAGEPOINT IN PLACE

After the strands had been cut, the beams were taken away, placed on supports, and a complete set of readings was taken once more.

After cleaning the forms and a general clean up the operation could be started for the next set of beams.

INSTRUMENTATION AND MEASURING DEVICES

Gage Points

Surface strain readings were used in this project. The gage points were made of stainless steel and were 1/2" in diameter and 1/4" thick with a 1/8" hole drilled through the center for the points of the mechanical gage. The gage points were furnished with two additional holes to fit 5/16" nails and the points were then nailed to the forms. The nails were driven only partly through and then bent over to the side (Fig. 15). There was no chance for the gage points to pull out upon stripping the forms as the nails themselves were anchored in the concrete. The centerhole was filled with tar to prevent it from being filled with cement slurry.

The gage points were placed at the level of the resultant of the prestress force or 4" from the bottom on each side of the beam. The gage points were placed at intervals of 5" starting at the end face of the beam for a length of 40" and 10" intervals from that point to the center line of the beam. All gage points were placed in one half of the beam. The 5" intervals provided strain readings to evaluate the anchorage length. To obtain a reading for the first 5", using the 10" Whittemore mechanical strain gage, an extension was arranged at the end of the beam (Fig. 16). Two gage points were placed, symmetrical to the centerline, on the top and bottom face of the beams. Between these points at the centerline was placed a gage point to be used for the creep camber readings. That made a total of 24 gage points for each beam.

The shrinkage specimens had two gage points on each face at the centerline to give a total of eight.

On the last four beams gage points at 2" intervals were placed at the ends for a distance of 12" and readings were obtained by a 2" Whittemore gage. That increased the number of gage points to 44. Figure 16 shows the gage points near the end of the specimen. The nails can be seen sticking out of the concrete. They were cut off before readings were taken. Figure 17 shows the forms equipped with gage points; the form for the shrinkage specimens is on the right.

The points proved to be very successful. In most cases there was little doubt of the reading, no hesitating dial needle. The strain kept close to the evaluated average all the time. One reason they worked so well is that they were accurately made in the machine-shop. Points that came out of place had to be re-drilled in the laboratory and these holes did not give the same performance as the holes drilled in the machine-shop.

Electric Straingages

According to the plan there should be electric strain gages at the centerline for each beam to evaluate the elastic shortening and to control the prestress force. It is evident that to get an electric strain gage put on a strand requires a very exacting technique. The diameter of each wire was

FIG. 16 END OF BEAM

FIG. 17 BEFORE ASSEMBLING OF FORMS

approximate 1/8" (3/8" strand) and the gage had to be very narrow. The best S.R. 4 strain gage for this use is the A-12 gage. Its dimensions are 1/8" x 1", and the width can be trimmed considerably. This gage has been used previously (10) on strands but is no longer commercially available. A-7 and A-18 gages were available for use on this project. They are appreciably shorter and wider, 3/16" x 1/4" and 3/16" x 1/8" respectively. The width given is the minimum trim width. The gage covered almost the whole exposed circumference of the wire.

Before application of the gages the wire was cleaned with emery paper and carbon tetrachloride to insure good bond. After the width of the gage had been trimmed down as much as possible it was covered with cement (Duco or Armstrong) and applied to the wire. It was kept in place by wrapping rubber bands around the entire strand. The rubber band was removed after about one day's curing. The gages were then exposed to heat-lamps for additional curing.

The leads were connected to the gages and secured to the strand by insulation tape. The gages were then waterproofed with hot petrolastic asphalt. To make the asphalt stick better the strand was kept warm while the asphalt was applied. Care was taken to make the asphalt fill the space between the wires. Fig. 18 shows the finished waterproofing of the 1/2" strand.

FIG. 18 WATERPROOFED ELECTRIC STRAINGAGES

FIG. 19 CAMBER DETERMINATION, SUSPENSION OF COPPER WIRE

Camber Determination

To measure the camber created at release a fairly simple device was used. A copper wire was suspended between two bolts projecting from the concrete six inches from the end (Fig. 19). A load at each end kept the wire straight. At midspan a piece of graph paper was glued to the face of the beam, (Fig. 20). By use of a square the relative position of the wire with respect to the graph paper was determined before and after release.

On beams 5 and 6, a level was used to evaluate the camber and for 7 and 8 the device seen on Fig. 21 was used. Both the latter methods had weak points in that the beams moved relative to the measuring device. On release the shortening of the strands outside the beams would make the beams move. The floor not being level and the forms being on shims caused irregular movements.

To measure the camber due to creep, dial gages were placed between the beams at midspan as showed at Fig. 22. Gagepoints provided the supports at both ends of the gage. The readings would be the relative movements between two beams and correction had to be made to get the correct deflection.

Slip-in Readings for 1/2" Strand

As it has been feared that the high bond stresses developed with the 1/2" strand would lead to a significant slippage between the concrete and the strand, an arrangement

FIG. 20 CAMBER DETERMINATION, COPPER WIRE AT CENTERLINE

FIG. 21 CAMBER DETERMINATION, USE OF DIAL GAGE

FIG. 22 GAGE FOR MEASURING CREEP CAMBER

FIG. 23 GAGES IN PLACE FOR SLIP-IN READINGS

was made to measure the slip-in or the movement of the strand with respect to the end face of the beam. (Fig. 23). Wire clips were clamped to the strand. A dial gage was bolted to the previously mentioned steel extension. The point of the gage touched one of the nuts of the clips. It was intended to obtain only the magnitude of the movement, not the exact value. If the strand should slip it would still get the correct value provided the face of the nut was exactly in a plane normal to the axis of the strand. Even in the event this were not true the error might not be too large.

HANDLING AND STORING

As the beams were made they were stored as shown in Fig. 24 and Fig. 8. Six inch pipe were used as supports. The beams had to be placed a distance apart to enable a person to take readings on the sides facing each other. Shrinkage specimens are seen standing vertically. They were all stored between the beams and the wall as can be seen in Fig. 8

For handling purposes the beams were provided with a hook 1 foot from each end. The shrinkage specimens had a hook at one end face in addition.

FIG. 24 STORING OF BEAMS

RESULTS

Table 5 gives the average values of total strain, shrinkage only and creep only. Each number represents the average of about 18-20 readings. Readings which were definitely out of line with the rest were omitted; these amounted to about 10%. Strains within the anchorage zone were not included. The last readings taken for beams No. 5 and 6 and No. 7 and 8 were taken at the time of 111 days and 94 days respectively.

Table 6 lists the total strains and creep strains after three months and the expected final strains. The expected total creep is calculated on the assumption that 60% of the final strain has taken place after three months. L'Hermite (4) suggests this value for creep only, but as the shrinkage has much the same rate it has been assumed to be valid for creep and shrinkage as well. Total loss and percentage loss of prestress are listed too.

Total strain, creep only and shrinkage only are plotted on graphs in Fig. 25-28. The first set of beams, beams No. 1 and 2 is plotted on Fig. 25, and the following sets on the next figures. In order to differentiate between the curves the shrinkage and creep curves have been translated to the right. Knowing the total strains, Cacout's curve can be plotted. With the use of the values in Table 6, Cacout's curve has been drawn with broken lines for the total strain and the creep.

The creep versus stress is plotted on Figures 29 through 32 for various days after release. A straight line is drawn to indicate a straight line relationship where it seemed justifiable to do so. The solid line is for the limestone and the broken line is for the gravel. On Figure 32 the values have been taken from the curve instead of from the table to plot the creep at 1,2 and 4 days.

Table 7 gives the measured and calculated elastic strains as well as the pertinent stresses. The calculated strains were derived in the following way. Using the flexure formulae we will have for the stress at any point:

$$f_c = -\frac{P}{A} - \frac{Pec}{I} + \frac{M_d c}{I}$$

Where

- P = Initial prestress force reduced for elastic shortening
- A = Area of concrete cross-section
- I = Moment of inertia of concrete cross-section
- e = Eccentricity of prestress force
- c = Distance from neutral axis to the fiber where the stress is to be found
- M_d = Moment due to dead load at quarterpoint

The calculated elastic shortening will be;

$$\varepsilon = \frac{f_c}{E_c}$$

Where

- E_c = Modulus of elasticity of the concrete found from cylinder tests.

Figures 33 through 45 give the results of modulus of elasticity upon which the calculated strains in Table 7 are based.

Table 8 gives the calculated and measured camber. The camber was calculated by the elastic theory and using the conjugate beam method we arrive at the equation:

$$\delta \text{ midspan} = \frac{fs \quad e\ L^2}{8E_c I_c}$$

Where

- fs = prestress force, reduced for the elastic shortening
- e = eccentricity of prestress force
- L = Span length
- E_c = Modulus of Elasticity for concrete
- I_c = Moment of inertia of concrete cross-section

The calculated camber was corrected for dead load deflection.

Assuming the creep to be just a continuation of the elastic shortening the camber due to creep should be in the same proportion to the elastic camber as the creep to the elastic shortening.

Table 9 gives the readings of the gage points on the top and bottom of the beams. The numbers given are readings of single gage length and cannot be taken to be as reliable as the readings given in Table 5. The expected magnitude of the creep 1/3 from the bottom calculated with use of the top and bottom readings is given and for comparison the measured strains are also listed (taken from Table 5).

Figures 46 through 53 give the actual measured camber and the calculated camber based upon the calculated instantaneous camber and the creep. The measured camber is drawn as a solid line and the calculated camber is drawn as a broken line.

The slip in readings for the 1/2" strand are shown in Table 10. To differentiate between the strands some kind of reference system had to be made. The part of the beam with the gage points was called the northern part, assuming the beams in the direction North/South. With a top and a bottom row of strands, the strands were named Top-East, Top-West, Bottom-East and Bottom West, abbreviated to TE, TW, BE, and BW. The surface condition of the strands before pouring is also given.

The anchorage length of the 1/2" strand is given in Table 11 and for the 3/8" strand in Table 12.

For comparison the slip-in value for a 5/16" strand as given by Echols (3) is given in Table 11. This slip-in value represents an average of several readings where the concrete strength was the same as in this experiment. Using Guyon's formulae for elastic bond the theoretical anchorage length has been evaluated for the 1/2" strand.

Guyon's formulae:

$$3\lambda = \frac{3\ E_s}{Pi}\ G_o$$

Where

3λ = Transmission length
E_s = Modulus of elasticity of the strand
Pi = Initial prestress
G_o = Slip-in value

TABLE 5

TOTAL STRAIN, SHRINKAGE AND CREEP (AVERAGE)

Total Strains

Beam No.	Number of days after release 1	2	4	8	15	29	59	120
				Microinch/inch				
1	85		190	230	280	371	452	554
2	71		166	214	254	367	422	521
3	84	148	168	261	329	397	498	637
4	73	130	158	220	292	366	472	610
5	124	155	240	294	378	474	604	734*
6	100	135	205	256	350	445	537	700*
7	57	132	179	263	363	472	653	725**
8	63	133	168	243	359	464	610	711**
				Shrinkage				
1	31		126	142	180	252	317	387
2	30		117	148	179	270	313	380
3	2	43	34	91	131	182	255	333
4	7	43	42	72	141	172	233	348
5	51	62	106	111	151	215	278	341*
6	32	40	76	93	143	203	288	369*
7	29	74	91	120	175	472	323	382**
8	25	68	68	97	178	244	331	400**
				Creep				
1	54		64	88	100	119	135	167
2	41		49	66	75	97	109	141
3	82	105	134	170	198	215	243	304
4	66	87	116	148	151	194	233	262
5	73	73	134	183	227	259	278	393*
6	68	95	129	163	207	242	249	331*
7	28	58	88	143	188	254	330	343**
8	38	45	100	146	181	220	279	311**

*Age: 111 days

**Age: 94 days

TABLE 6

CALCULATED FINAL STRAIN* AND STRESS LOSSES IN STEEL

Beam number	1	2	3	4	5	6	7	8
Shrinkage (3 month)	355	348	302	293	327	340	382	392
Expected Final Shrinkage	590	580	505	489	500	566	634	653
Stress loss	16500	16250	14150	13700	15650	15879	17750	18300
Percent loss	9.40	9.3	8.1	7.8	8.9	9.0	10.2	10.5
Creep (3 month)	150	125	268	250	263	315	340	310
Expected Final Creep	250	208	445	417	605	524	566	517
Stress loss	7000	5820	17450	11700	16950	14680	15850	14500
Percent loss	4.0	3.3	7.1	6.7	9.6	8.4	9.0	8.3
Elastic Shortening	211	201	314	285	559	521	476	456
Stress loss	4900	5630	8800	7800	15650	14600	13300	12750
Percent loss	3.4	3.2	5.0	4.5	8.9	8.4	7.6	7.2
Total stress loss	29400	27700	35400	33400	47880	45150	46900	45550
Percent loss	16.8	15.8	20.2	19.0	27.5	25.8	26.8	26.0

* Strains in microinch/inch

TABLE 7

MEASURED AND CALCULATED ELASTIC SHORTENINGS

Beam Number	1	2	3	4	5	6	7	8
Measured strain *	211	201	314	285	559	521	476	456
Calculated strain	236	228	338	321	538	478	417	400
Stress, extreme fiber	1440	1440	2150	2150	3150	3150	2550	2550
Stress, 1/3 from bottom	960	960	1430	1430	2100	2100	1700	1700
Measured strain, electr.	225	238			430	265	470	465

* Strains in microinch/inch

TABLE 8

MEASURED AND CALCULATED CAMBER

BEAM NO.	1	2	3	4	5	6	7	8
Calculated, inch	0.0977	0.0948	0.139	0.135	0.229	0.207	0.176	0.163
Measured, Wire	0.110	0.130	0.160	0.110	0.202	0.200	0.190	0.190
Percent, Difference	13	37	13	19	12	3.5	8	16
Measured, Otherwise					0.16	0.13	0.120	0.125

TABLE 9

MEASURED CREEP STRAINS ON TOP AND BOTTOM FACE *

		1	2	4	8	15	29	60	120 Days	
Beam 1	Top	0		28	47	64	75	79	112	
	Bottom	130		Gage point lost in the concrete						
	D/3 from bottom calc									
	- " - measured									
Beam 2	Top	21		20	46	40	84	96	124	
	Bottom	50		69	175	179	188	225	249	
	D/3 from bottom calc	41		52	132	133	154	102	207	
	-"- measured	41		49	66	75	97	109	141	
Beam 3	Top	5	0	15	25	53	78	100	134	
	Bottom	130	151	106	151	210	239	294	359	
	D/3 from bottom calc	87	100	45	108	158	185	230	284	
	-"- measured	82	105	134	170	198	215	243	304	
Beam 4	Top	8	2	13	9	0	33	60	77	
	Bottom	113	130	154	220	253	292	345	394	
	D/3 from bottom calc	78	88	107	149	169	206	250	288	
	-" - measured	66	87	116	148	151	194	233	262	
Beam 5	Top	-19	-20	-15	5	9	42	42	47	111 Days
	Bottom	129	187	247	352	410	465	513	-	
	D/3 from bottom calc	80	118	160	236	275	324	356	-	
	- " - measured	73	73	134	183	227	259	278	393	

* Strains in microinch/inch

TABLE 9 (continued)

		1	2	3	4	5	6	7	8
Beam 6	Top	-10	0	10	27	45	64	124	79
	Bottom	68	150	195	252	304	360	377	464
	D/3 from bottom calc	42	100	133	177	218	262	292	335
	- " - measured	68	95	129	177	207	242	249	331
Beam 7	Top	0	4	14	39	46	19	110	95
	Bottom	0	96	149	218	274	299	-	-
	D/3 from bottom calc	0	65	104	158	168	206	-	-
	- " - measured	28	58	88	143	188	254	330	311
Beam 8	Top	3	7	27	3	49	79	69	111
	Bottom	70	116	163	206	262	347	444	470
	D/3 from bottom calc	48	79	116	149	191	258	319	350
	- " - measured	38	45	100	146	181	220	279	311

TABLE 10

"SLIP-IN" READINGS FOR 1/2" STRAND

	Beam No. 7				Beam No. 8			
Strand	T.W.	B.W.	B.E.	T.E.	T.W.	B.W.	B.E.	T.E.
Condition of Strand	Clean-clean rusty	clean-rusty	Clean	Clean	Clean-rusty	Clean-rusty	Rusty	Rusty
Slip in. 10^{-3} at release	56.7	62	68	79.8	68.5	59.3	35.4	30.8
- " - 3 hours	59.6	64.8	72.7	87.2	75.0	61.2	37.5	33.1
- " - 1 Day	62.6	71.4	73.2	93.9	75.0	76.2	47.0	38.0
- " - 2 Days	63.6	73.8	74.0	96.4	77.0	76.2	47.6	39.2
- " - 4 Days	64.3	74.4	74.1	97.3	78.3	77.7	47.7	40.7
- " - 8 Days	65.6	76.0	74.1	100.6	83.4	80.8	47.7	44.3
- " - 15 Days	65.6	76.4	74.3	101.4	84.4	81.3	47.8	45.1
- " - 29 Days	66.4	76.4	78.0	102.3	86.3	82.3	48.1	46.6
- " - 60 Days	67.3	77.8	80.4	102.6	90.6	84.2	48.2	47.6
- " - 94 Days	67.8	78.9	81.8	103.0	92.3	87.1	48.5	47.9

TABLE 11

SUMMARY OF SLIP-IN VALUES

	Beam No. 7				Beam No. 8			
Strand	T.W.	B.W.	B.E.	T.E.	T.W.	B.W.	B.S.	T.E.
Condition of Strand	Clean Clean Rusty	Clean Clean Rusty	Clean	Clean	Clean Rusty	Clean Rusty	Rusty	Rusty
Slip-in in.10^{-3}	56.7	62	68	79.8	68.5	59.3	35.4	30.3
Slip-in 5/16" Strand, in.10^{-3}	17.6 - prestress: 92500 psi							
Calculated Anchorage Length	28"	30"	33"	39"	33.5"	29"	17.5"	15"
Measured Length at Release	17"		22"		18"		12"	
Measured Length, 3 month	17-18"		22-23"		20-21"		13-14"	

TABLE 12

ANCHORAGE LENGTHS

Beam No.	1	2	3	4	5	6
West Side	9" - 10"	7" - 8"	7" - 8"	6" - 7"	7"	7"
East Side	9" - 10"	7" - 8"	8" - 9"	7" - 8"	7" - 8"	7" - 8"

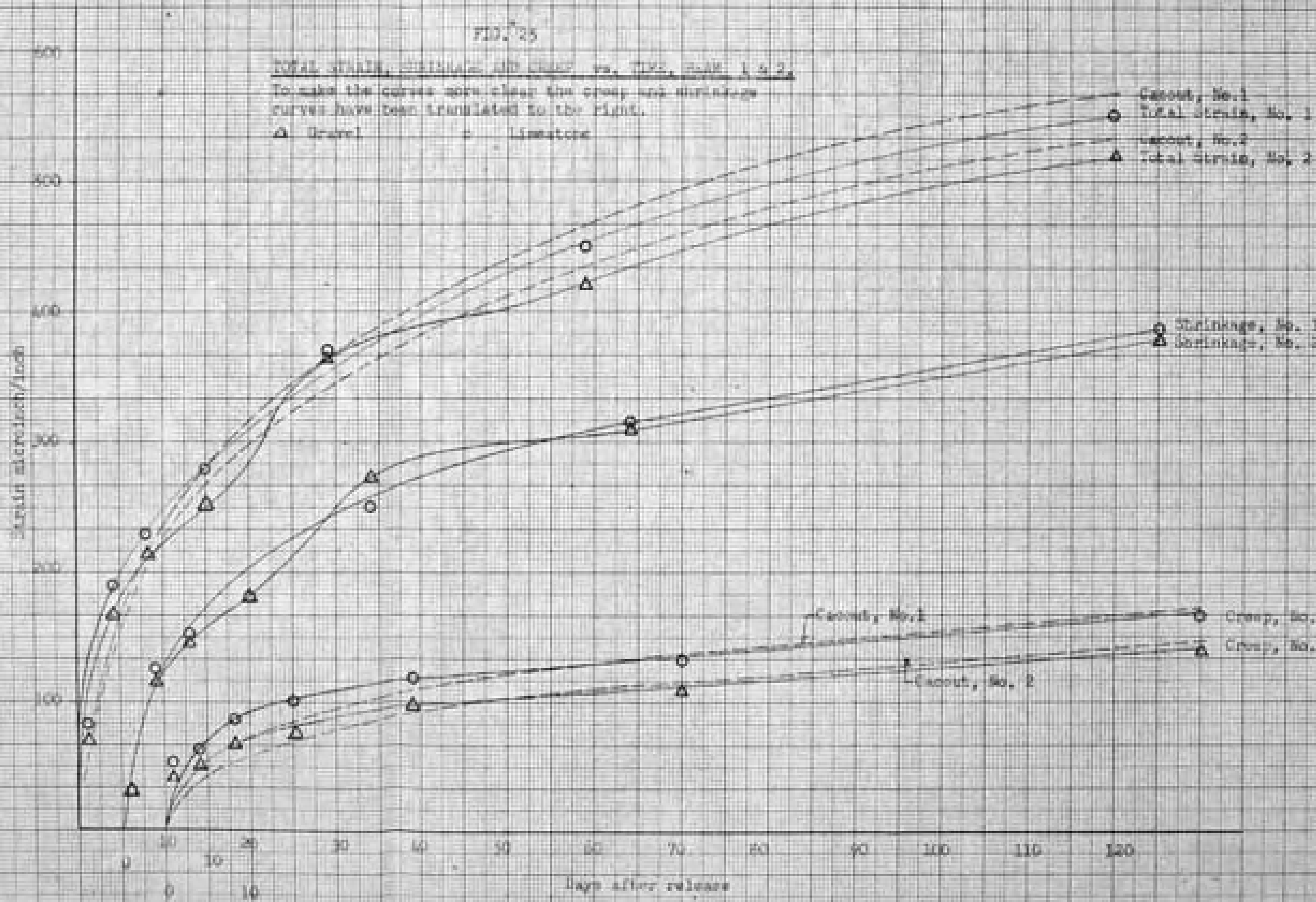


FIG. 25

TOTAL STRAIN, SHRINKAGE AND CREEP vs. TIME, BEAM 1 & 2.

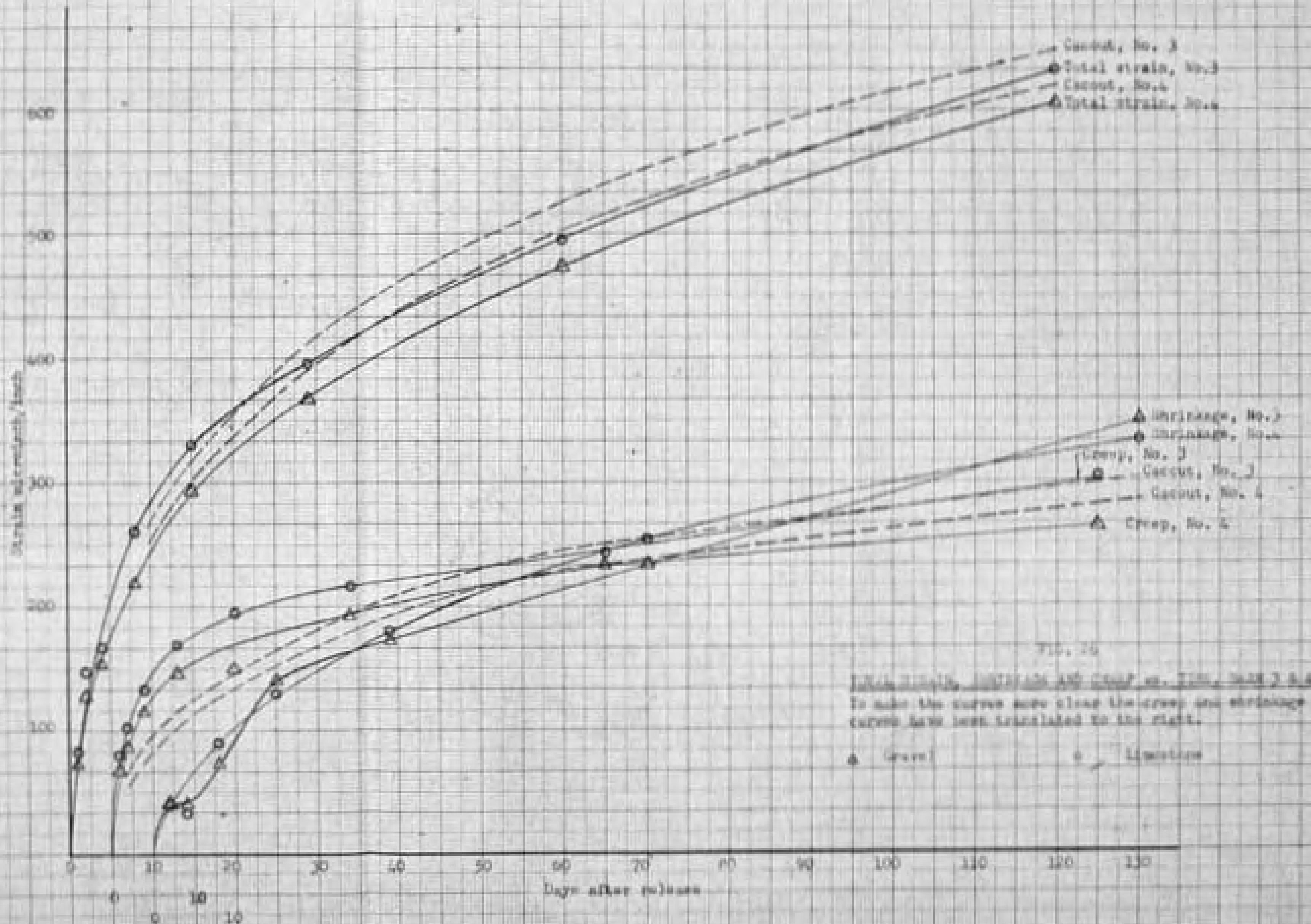


FIG. 26

TOTAL STRAIN, SHRINKAGE AND CREEP vs. TIME, BARS 3 & 4

To make the curves more clear the creep and shrinkage curves have been translated to the right.

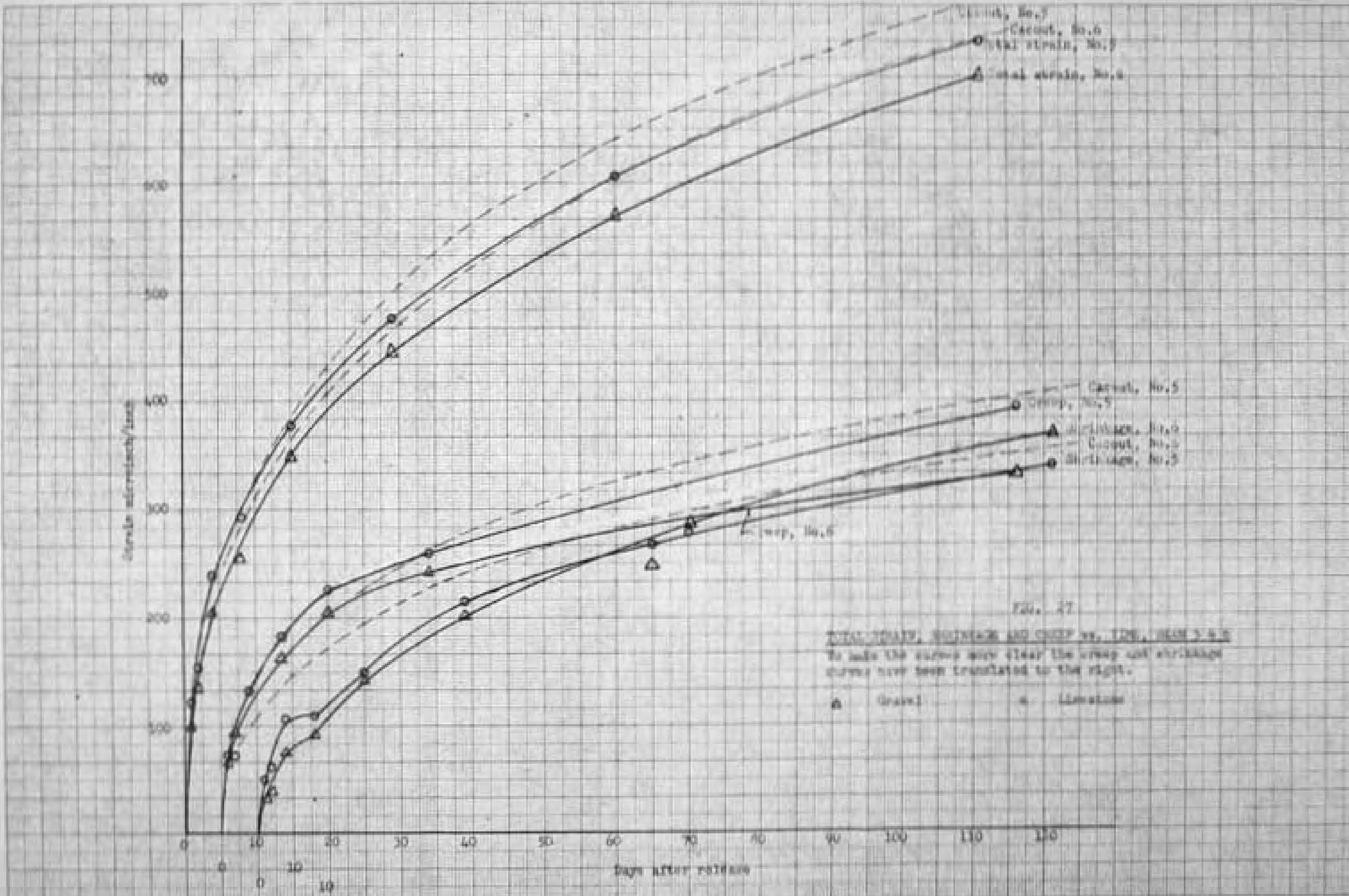


FIG. 27

TOTAL STRAIN, SHRINKAGE AND CREEP vs. TIME, BEAM 5 & 6

To make the curves more clear the creep and shrinkage curves have been translated to the right.

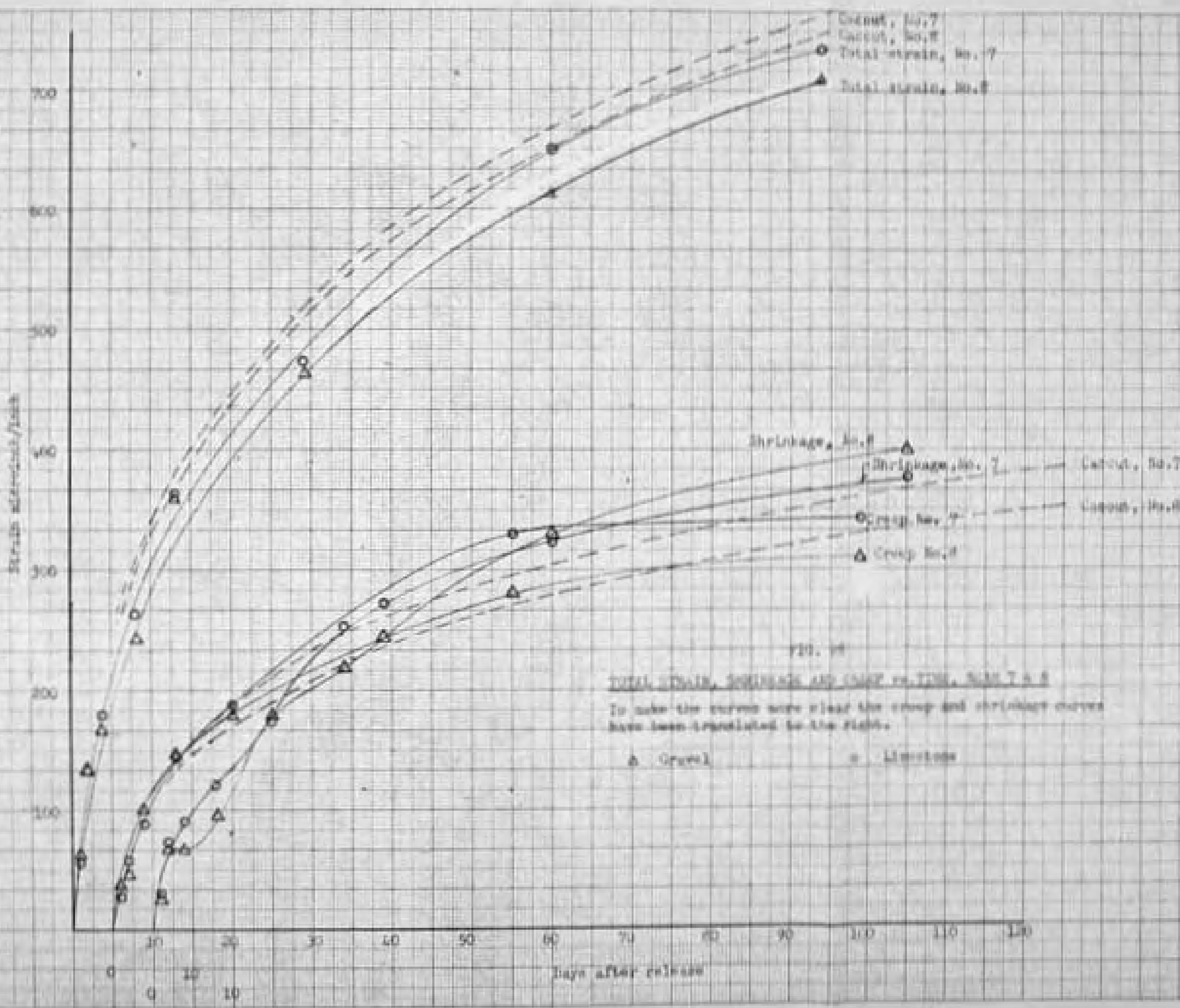


FIG. [illegible]

TOTAL STRAIN, SHRINKAGE AND CREEP vs. TIME, SLABS 7 & 8

To make the curves more clear the creep and shrinkage curves have been translated to the right.

△ Gravel   o Limestone

FIG. 29

CREEP vs. STRESS

o = Limestone Δ = Gravel

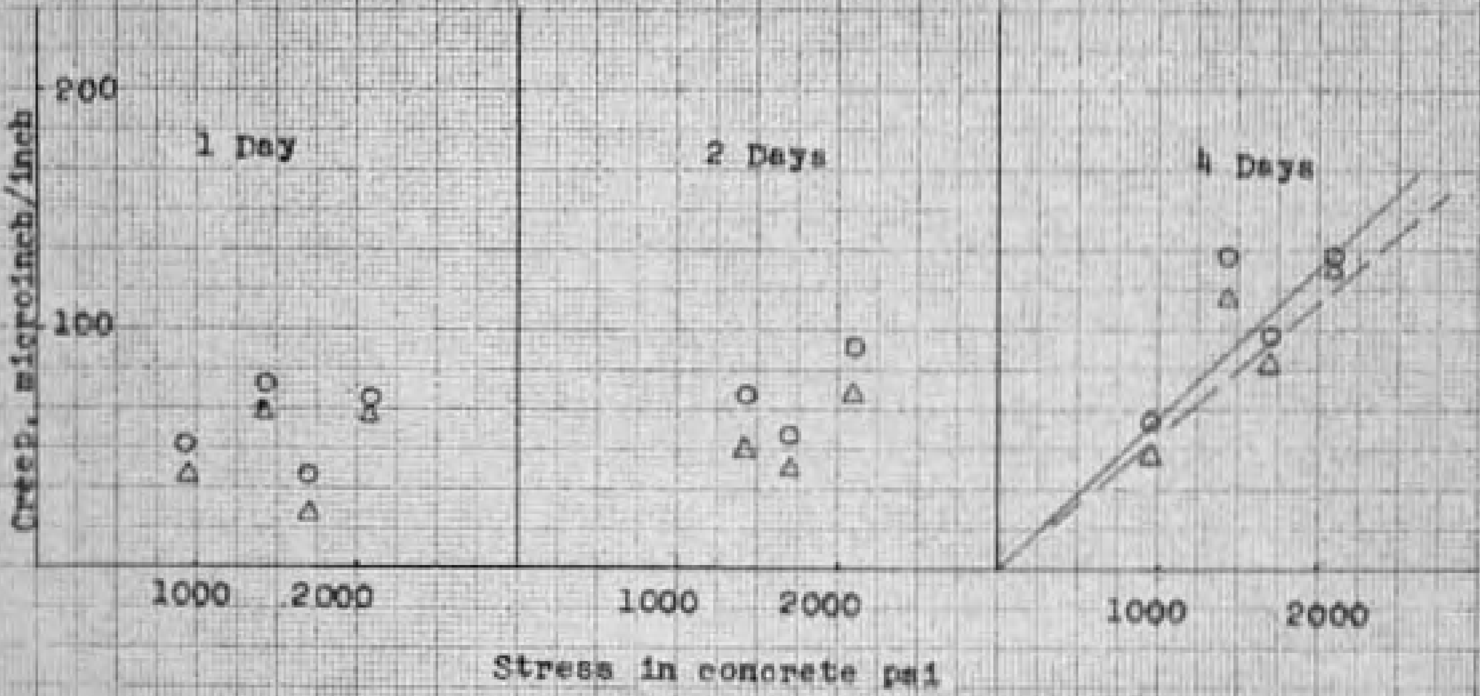


FIG. 30

CREEP vs. STRESS

○ = Limestone △ = Gravel

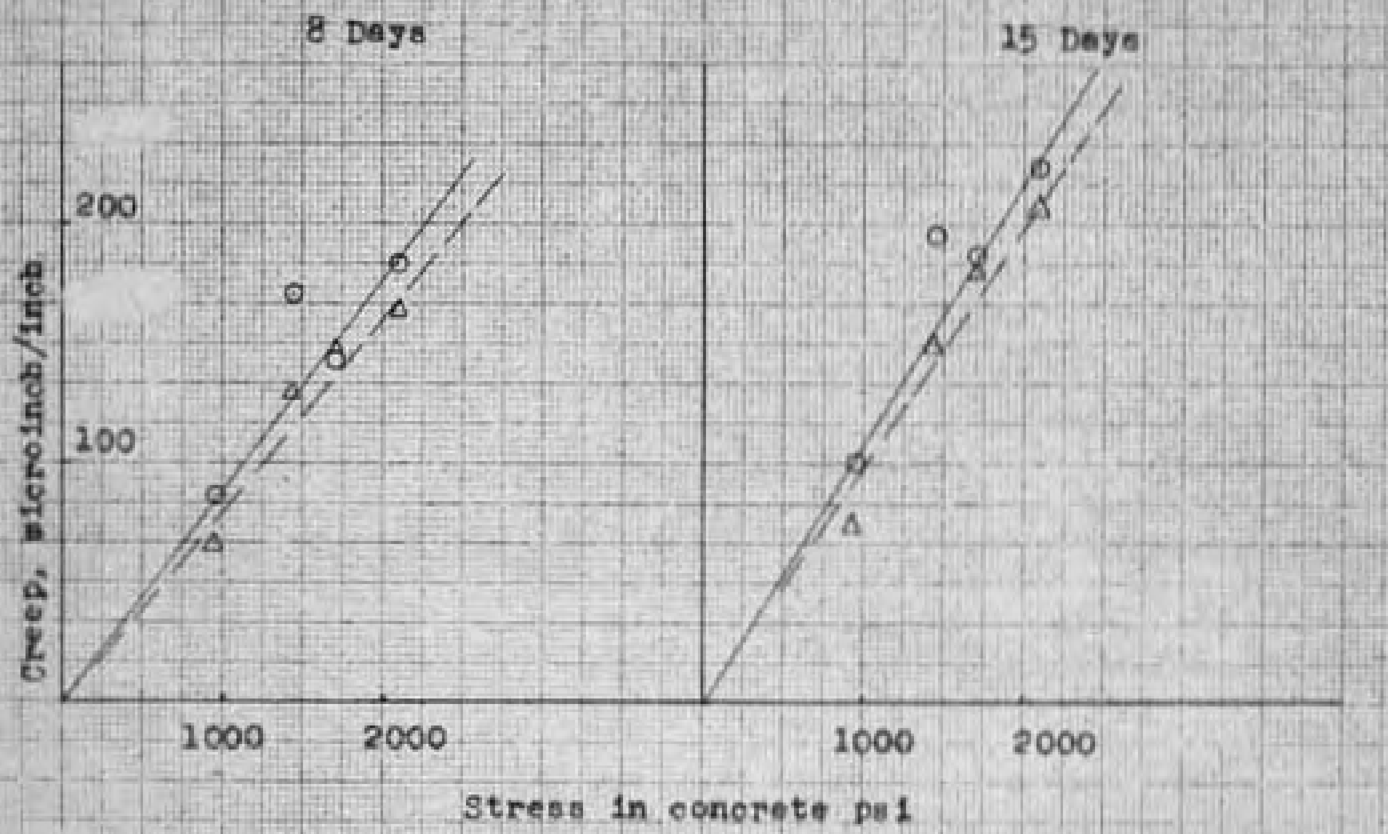


FIG. 31

CREEP vs. STRESS

o = Limestone Δ = Gravel

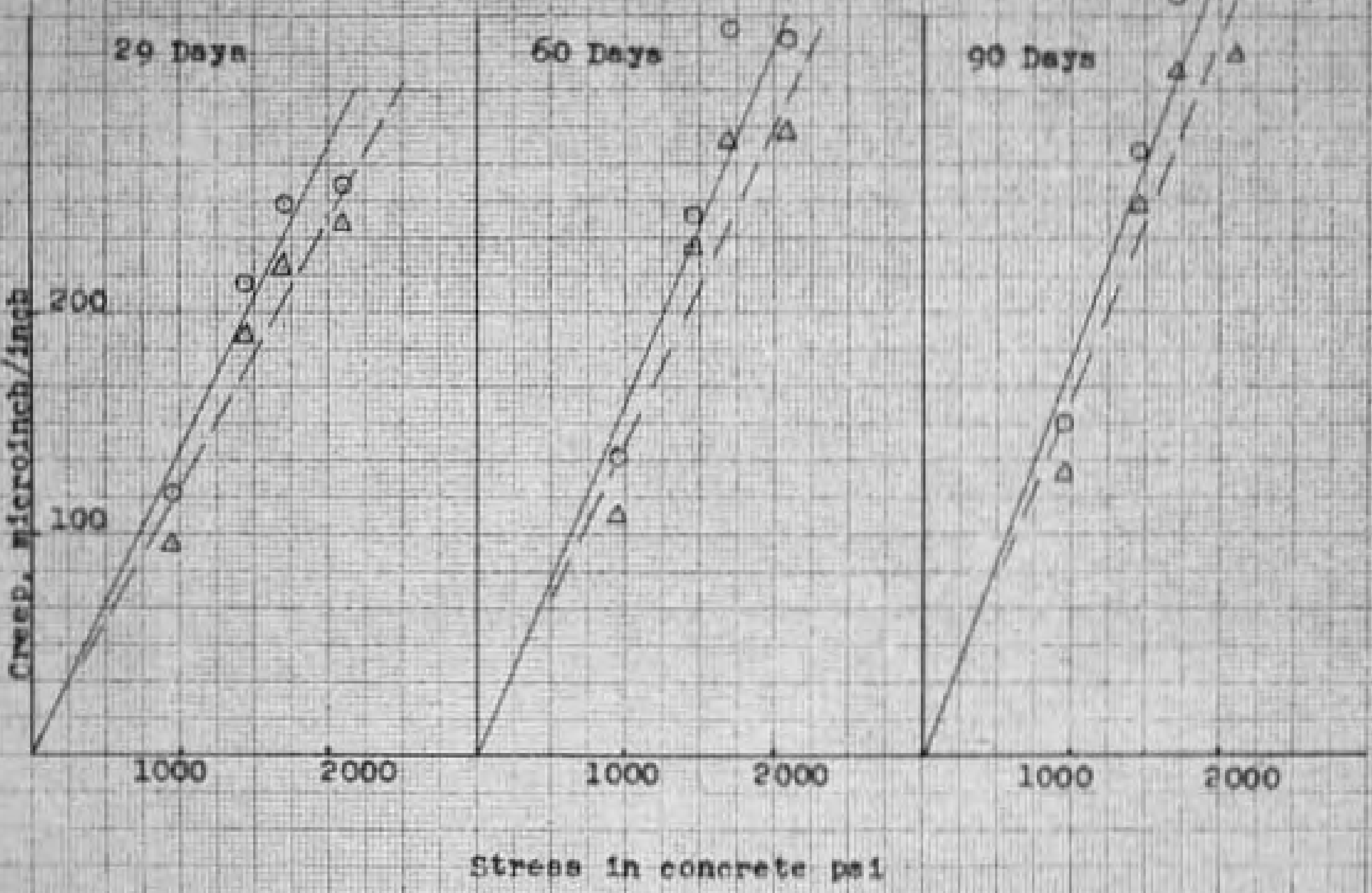
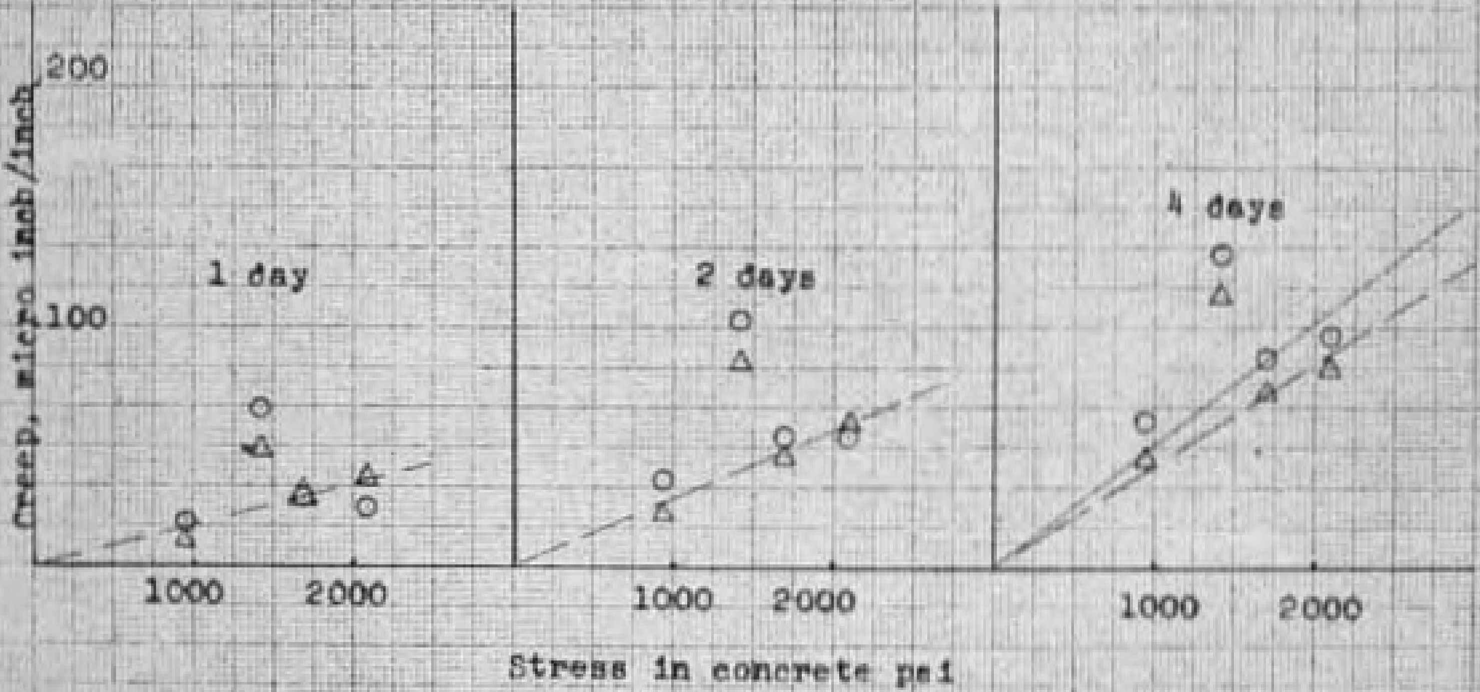


FIG. 32

CREEP vs. STRESS

Strain values taken from creep curve

o - Limestone Δ - Gravel



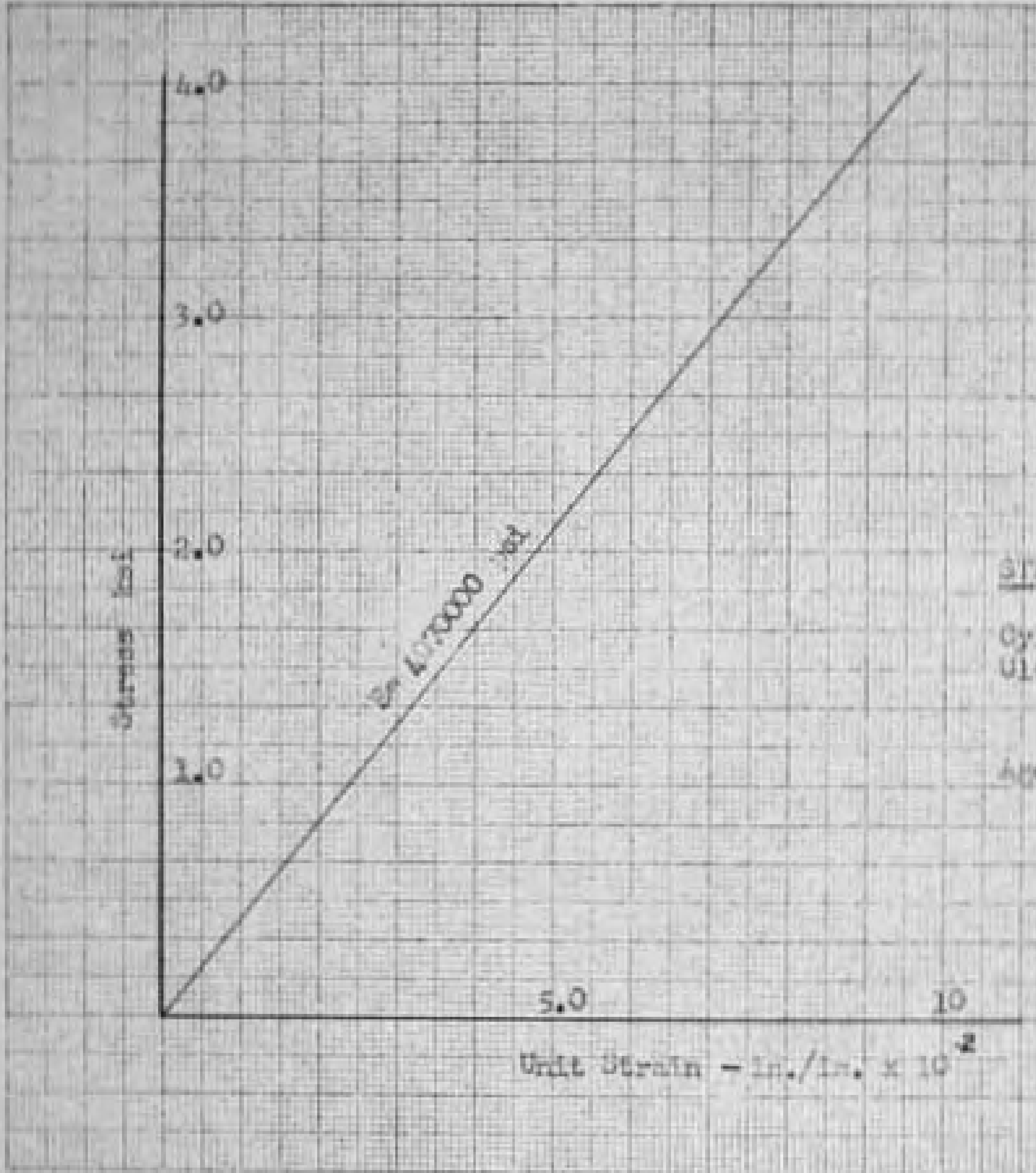


FIG. 33

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no.2, Beam no.1
Ultimate strength: 5690

Type Curing:
Age: 7 days moist, 1 day dry

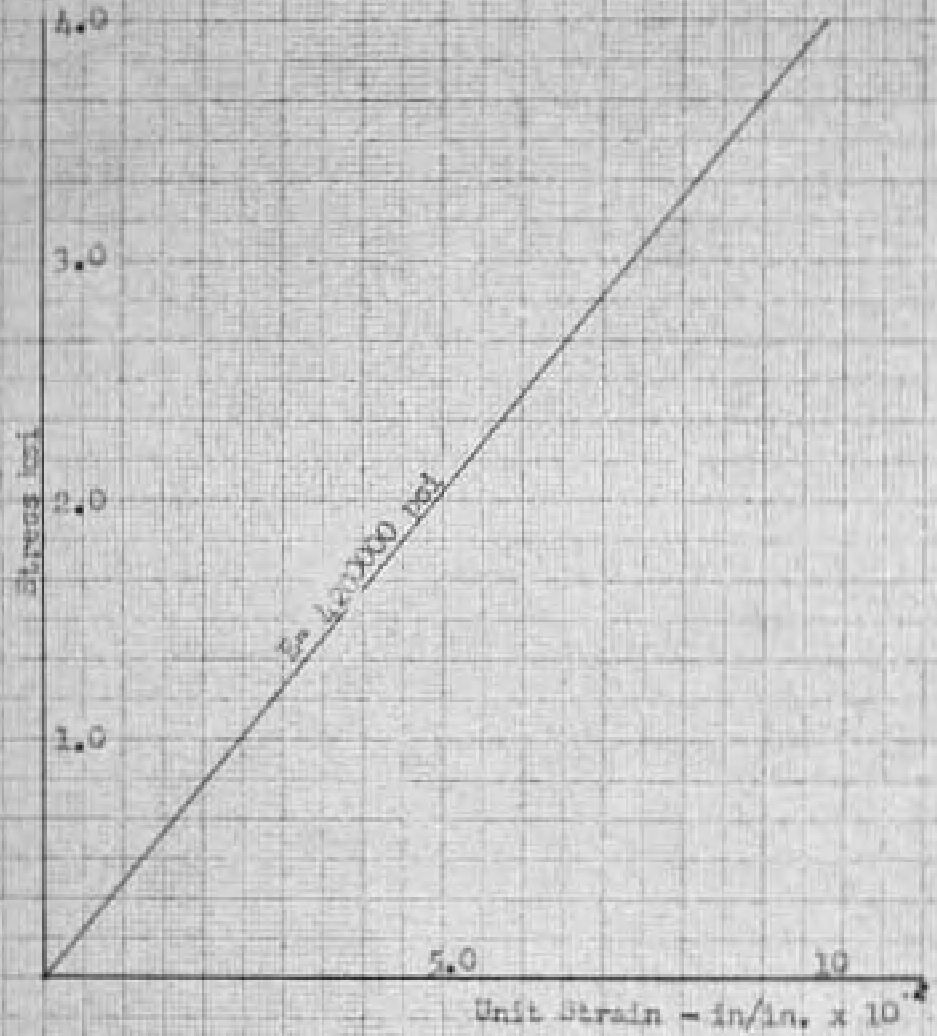


FIG. 34

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 52, Beam no. 2
Ultimate strength: 5550

Type Curing:
7 days moist, 1 day dry

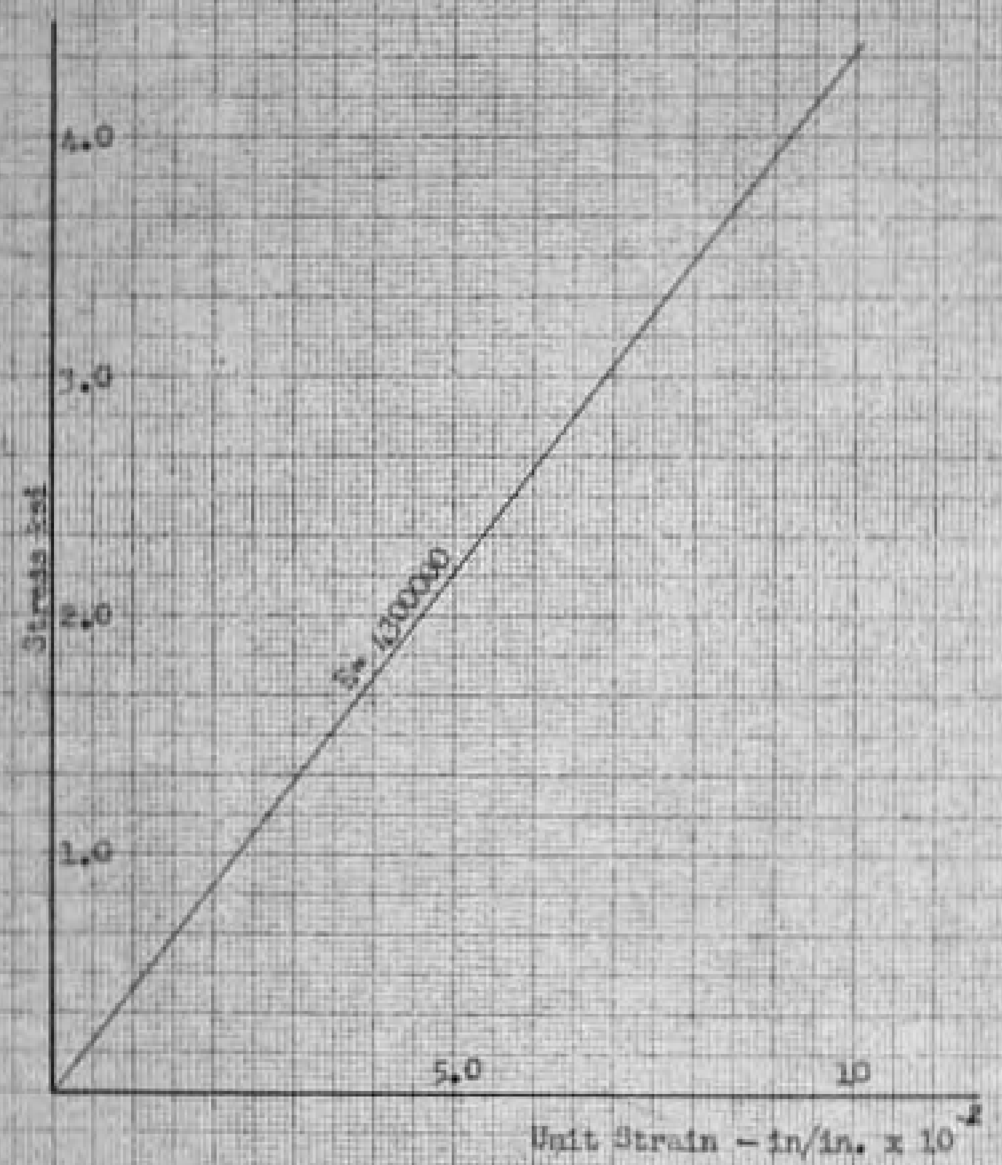


FIG. 35

COMPRESSION TEST
STRESS vs. UNIT STRAIN

Cylinder no. 12, Beam no. 3
Ultimate strength: 5100 psi

Type Curing:
9 days moist, 1 day dry

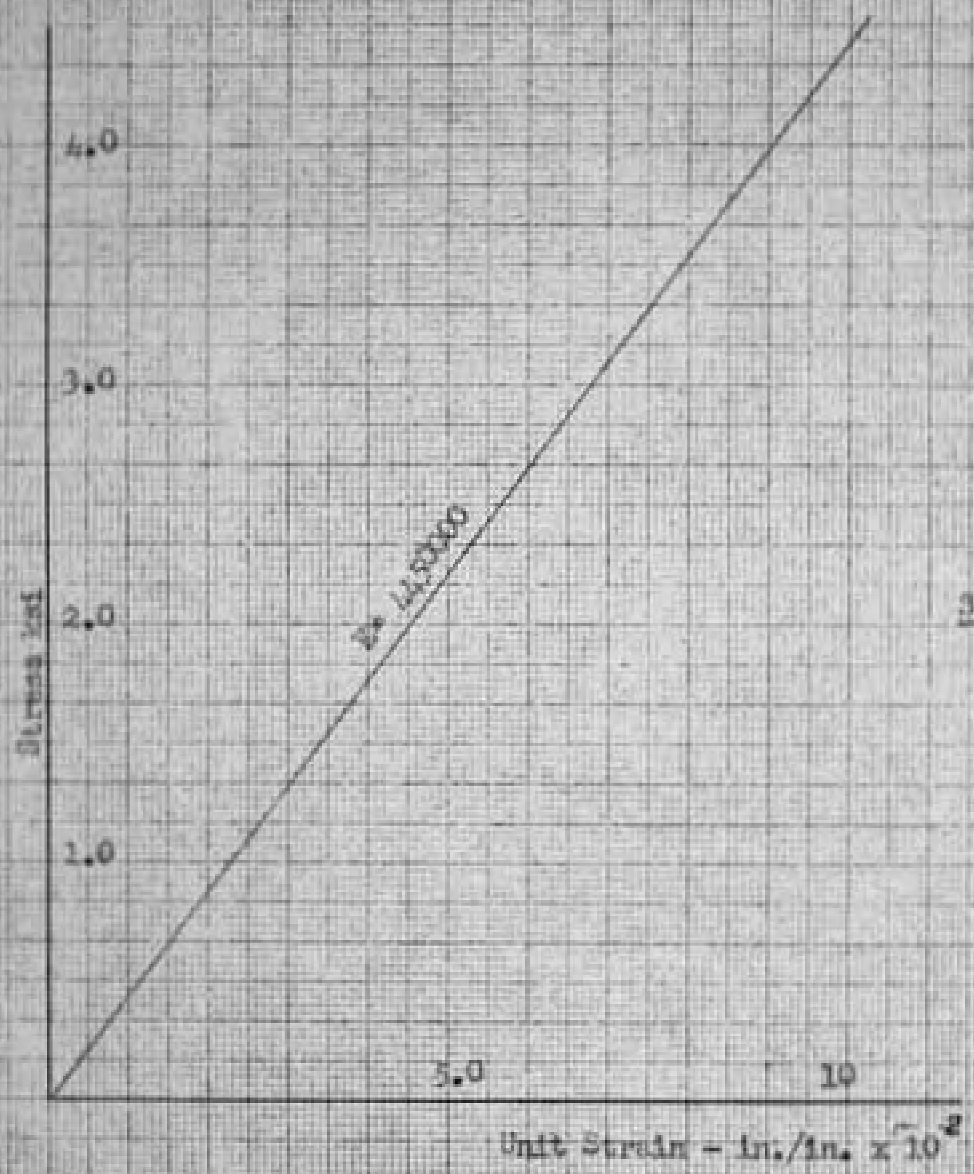


FIG. 36

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 62, Beam no. 4.
Ultimate strength: 5600 psi

Type Curing:
9 days moist, 1 day dry

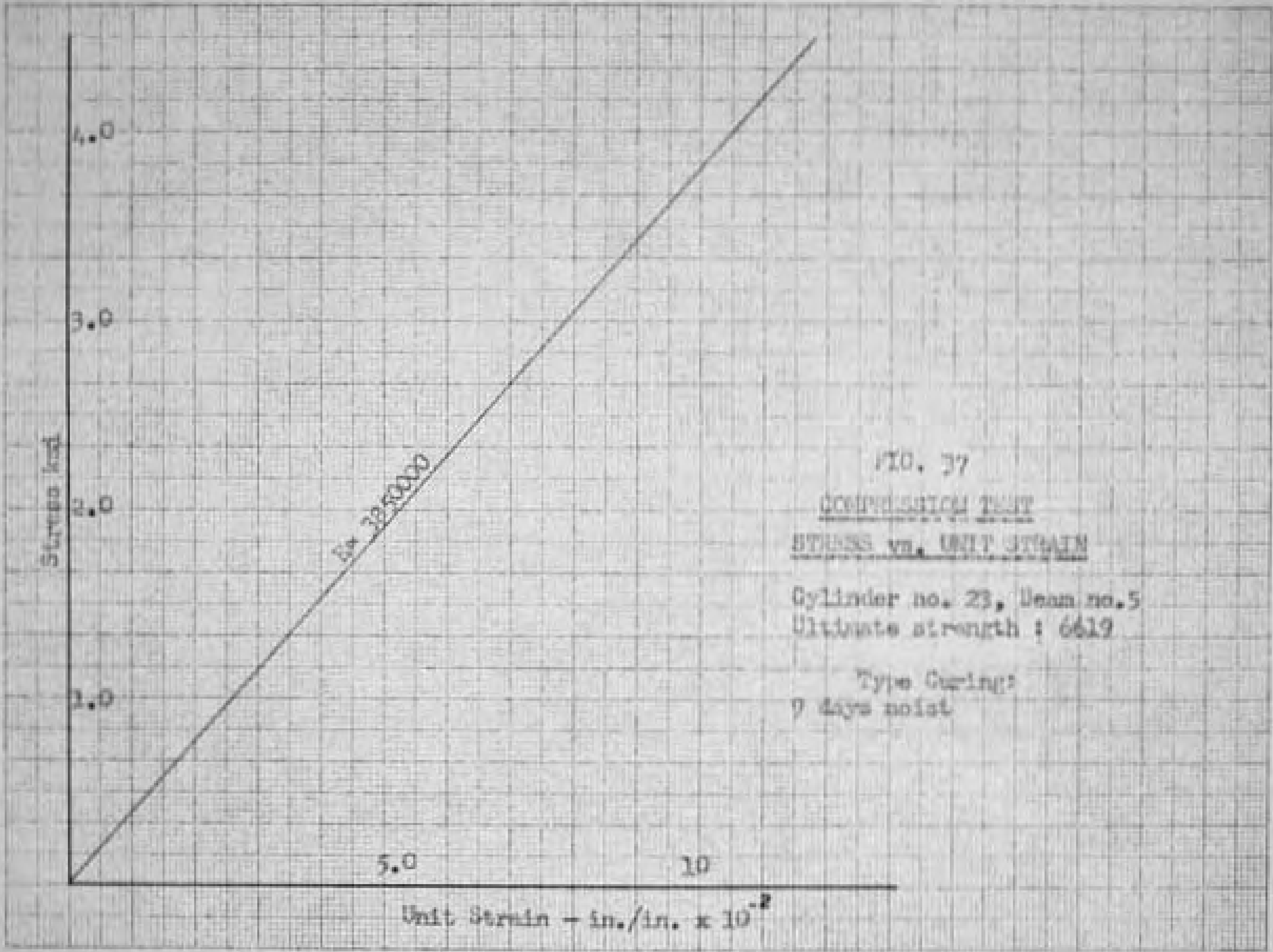


FIG. 37

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 23, Beam no.5
Ultimate strength : 6619

Type Curing:
7 days moist

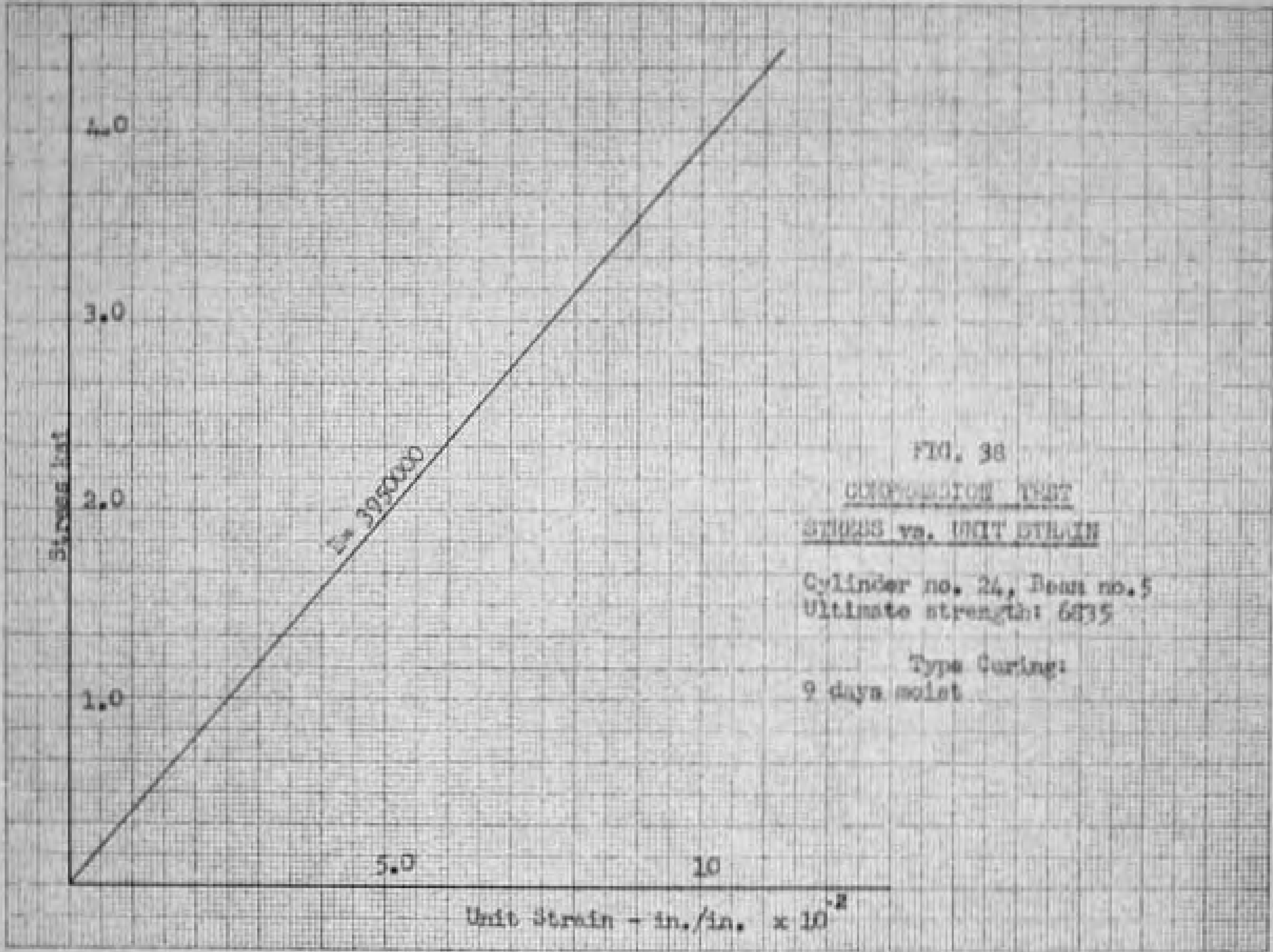


FIG. 38

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 24, Beam no.5
Ultimate strength: 6835

Type Curing:
9 days moist

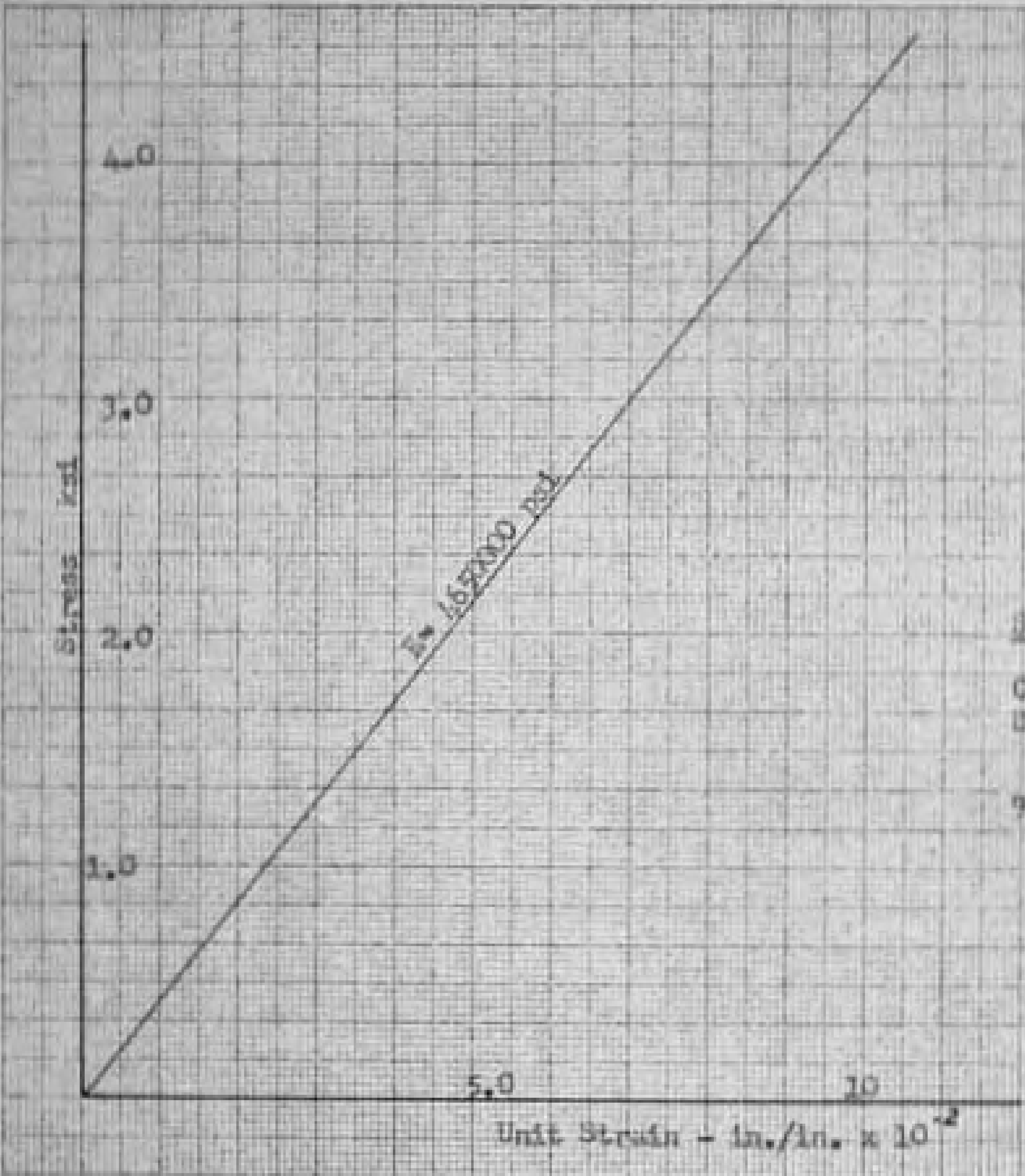


FIG. 39

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 79, Beam no. 6
Ultimate strength: 6133

Type Curing:
9 days moist

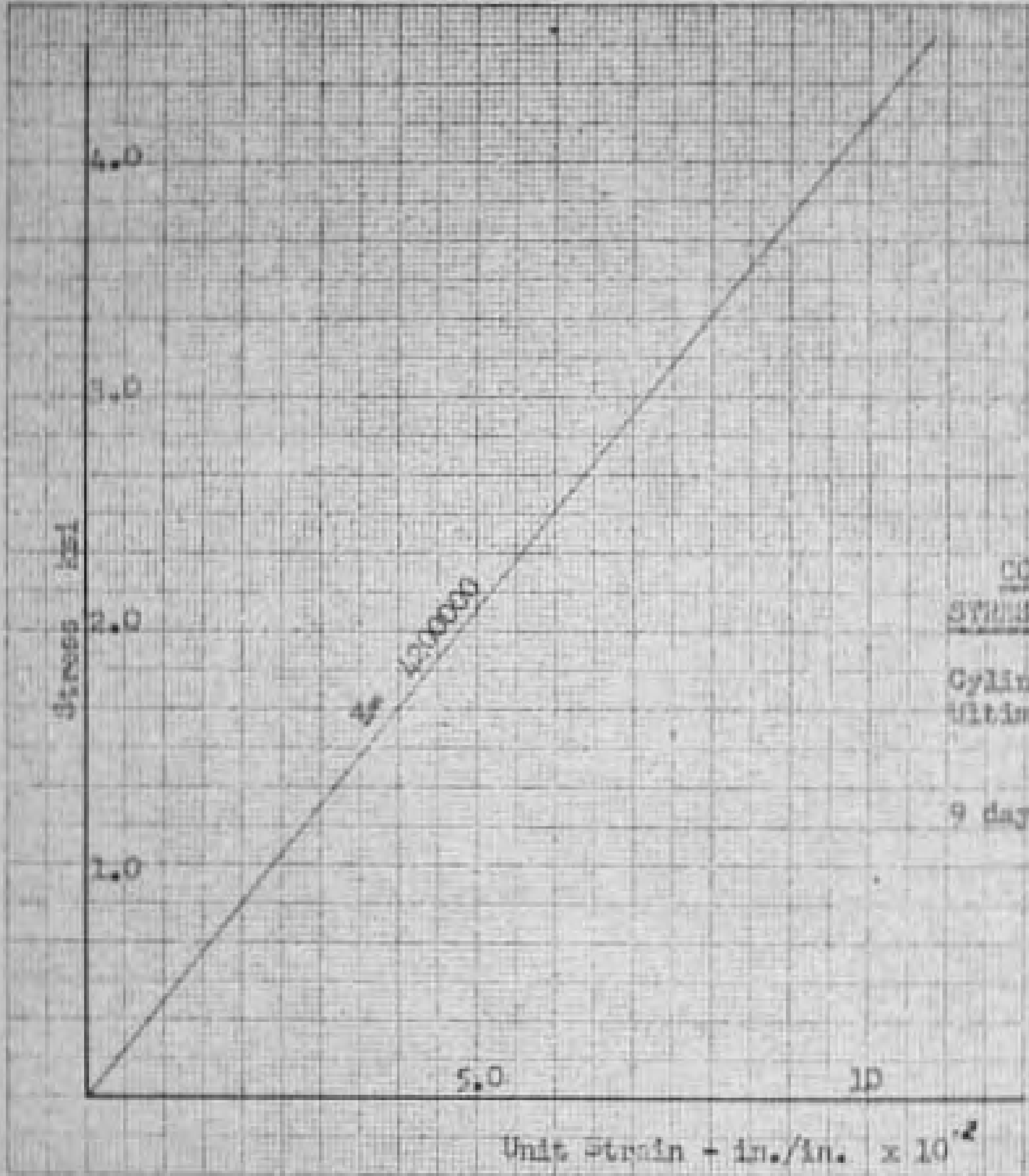


FIG. 40

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 74, Beam no. 6
Ultimate strength: 6409

Type Curing:
9 days moist

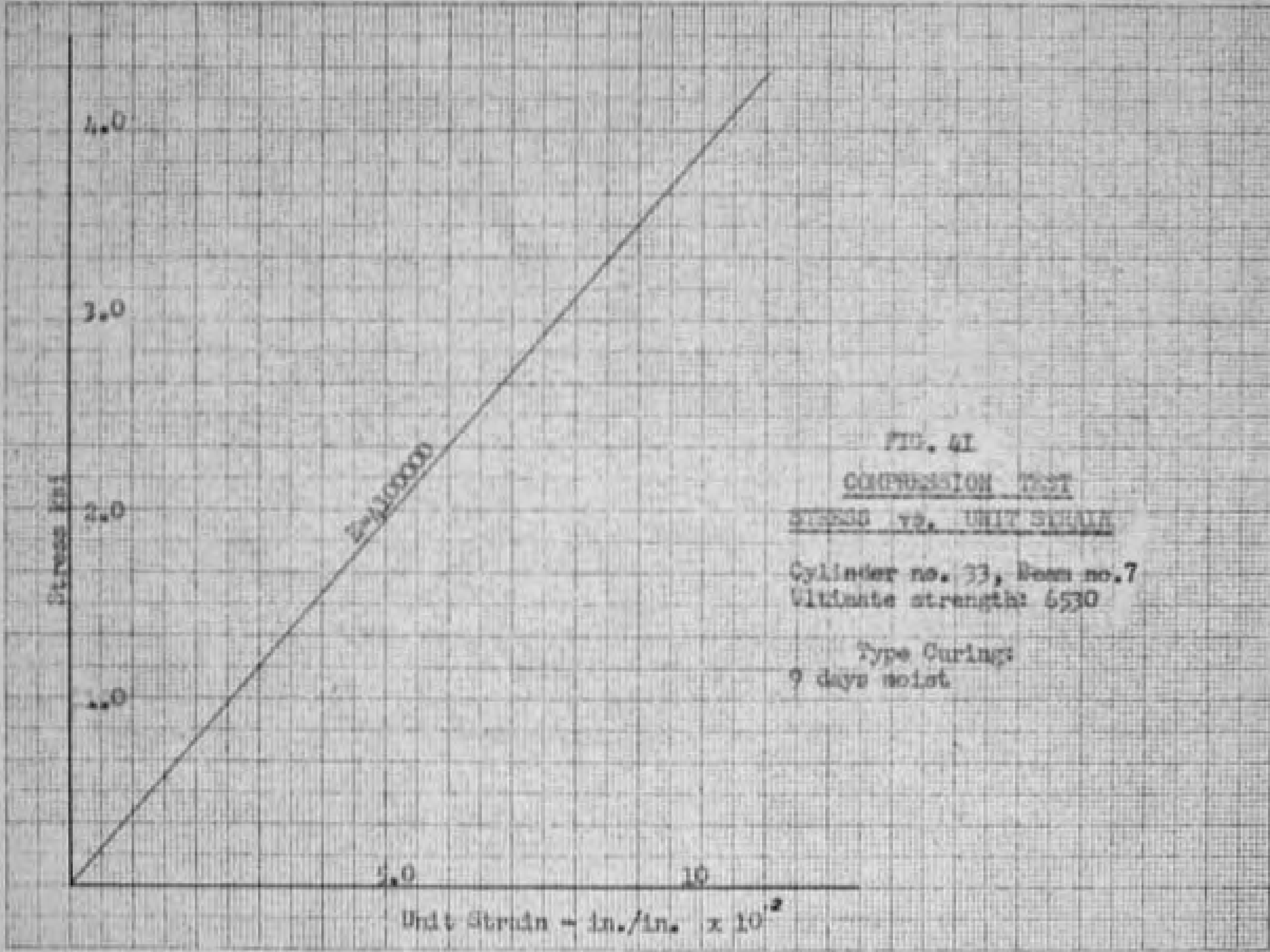


FIG. 41

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 33, Beam no.7
Ultimate strength: 6530

Type Curing:
9 days moist

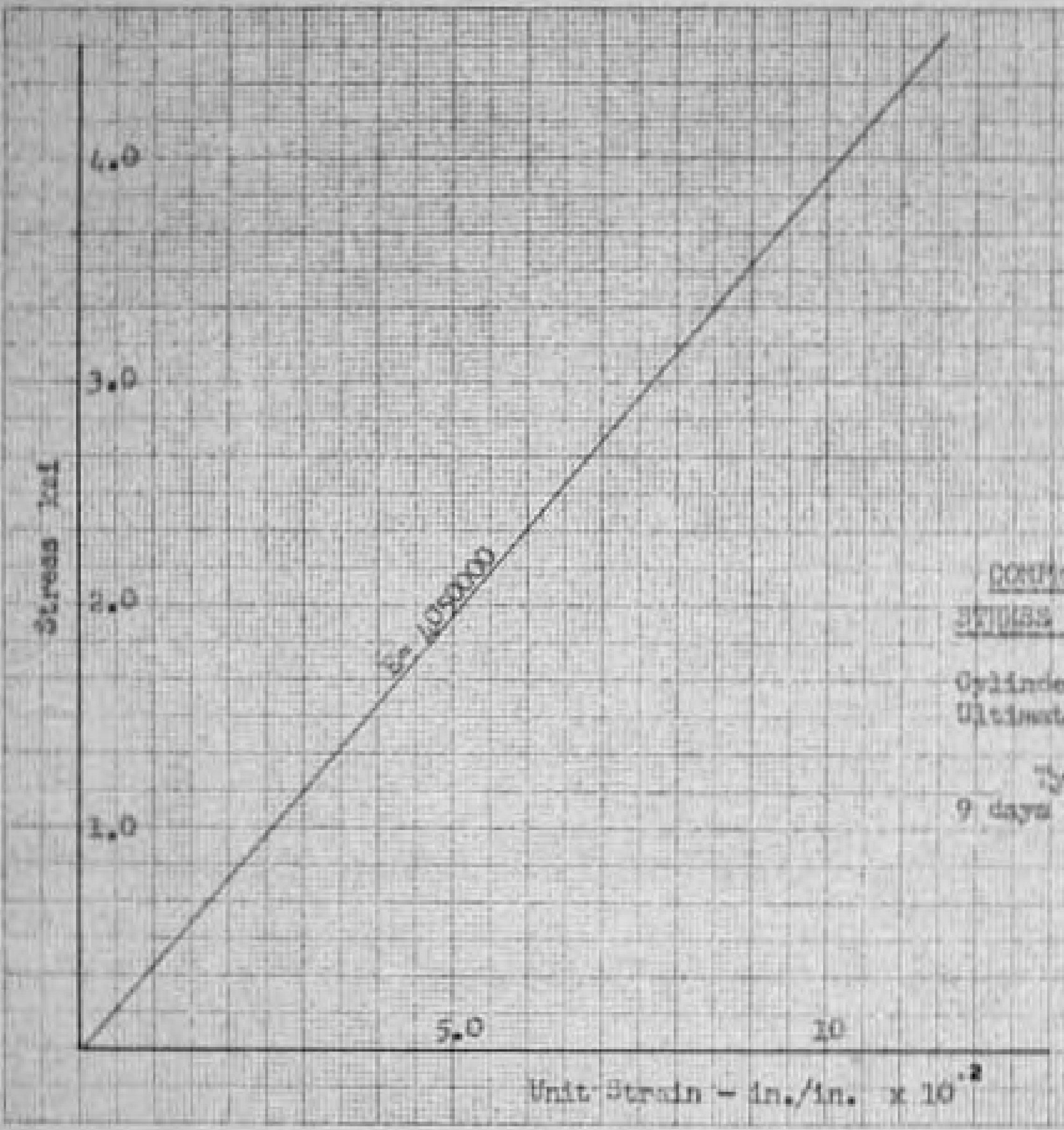


FIG. 42

COMPRESSION TEST
STRESS vs. UNIT STRAIN

Cylinder no. 34, Gage no.7
Ultimate strength: 6690

Type Curing:
9 days moist

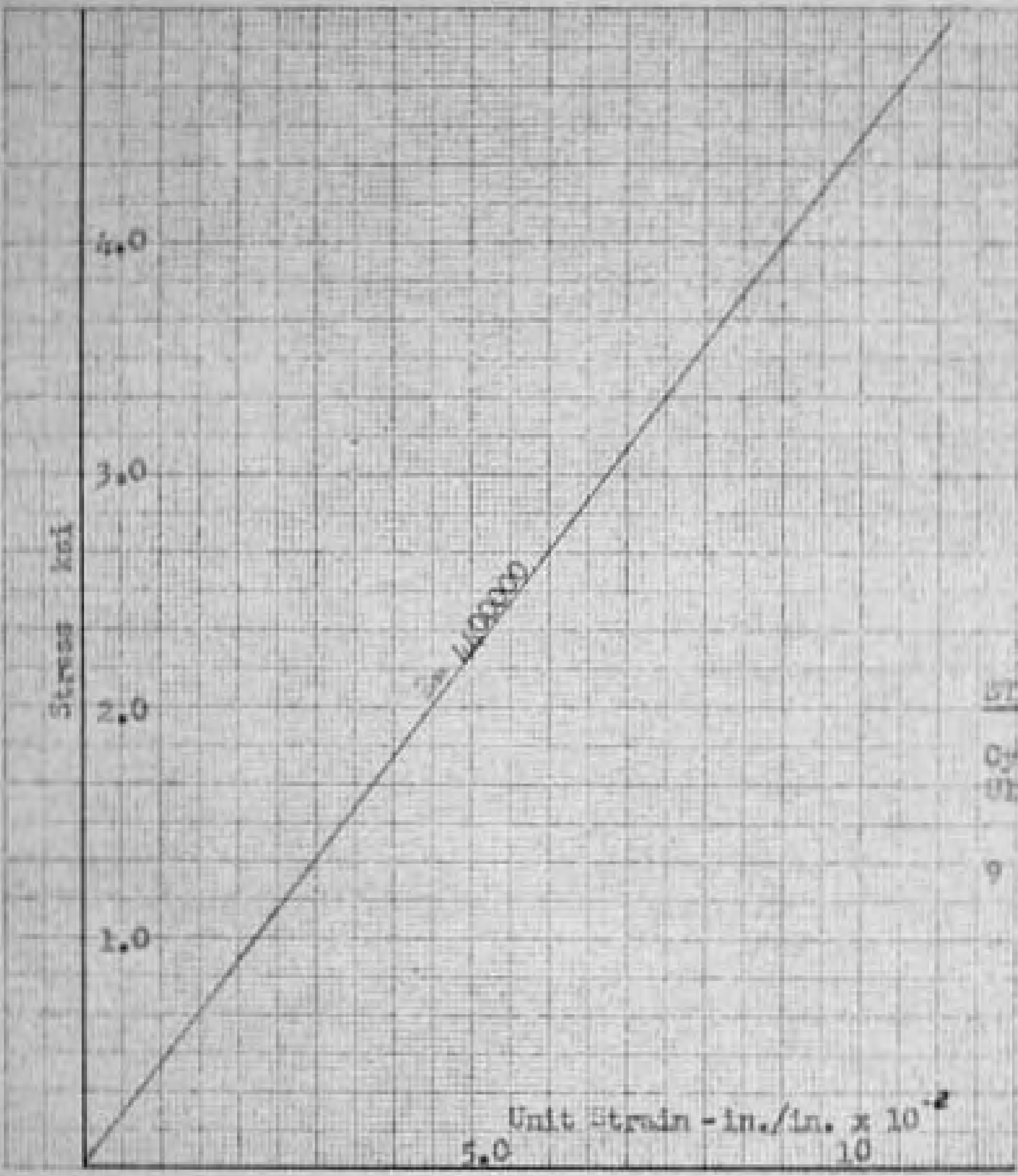


FIG. 43

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 83, Beam no. 8
Ultimate strength: 6385

Type Curing:
9 days moist

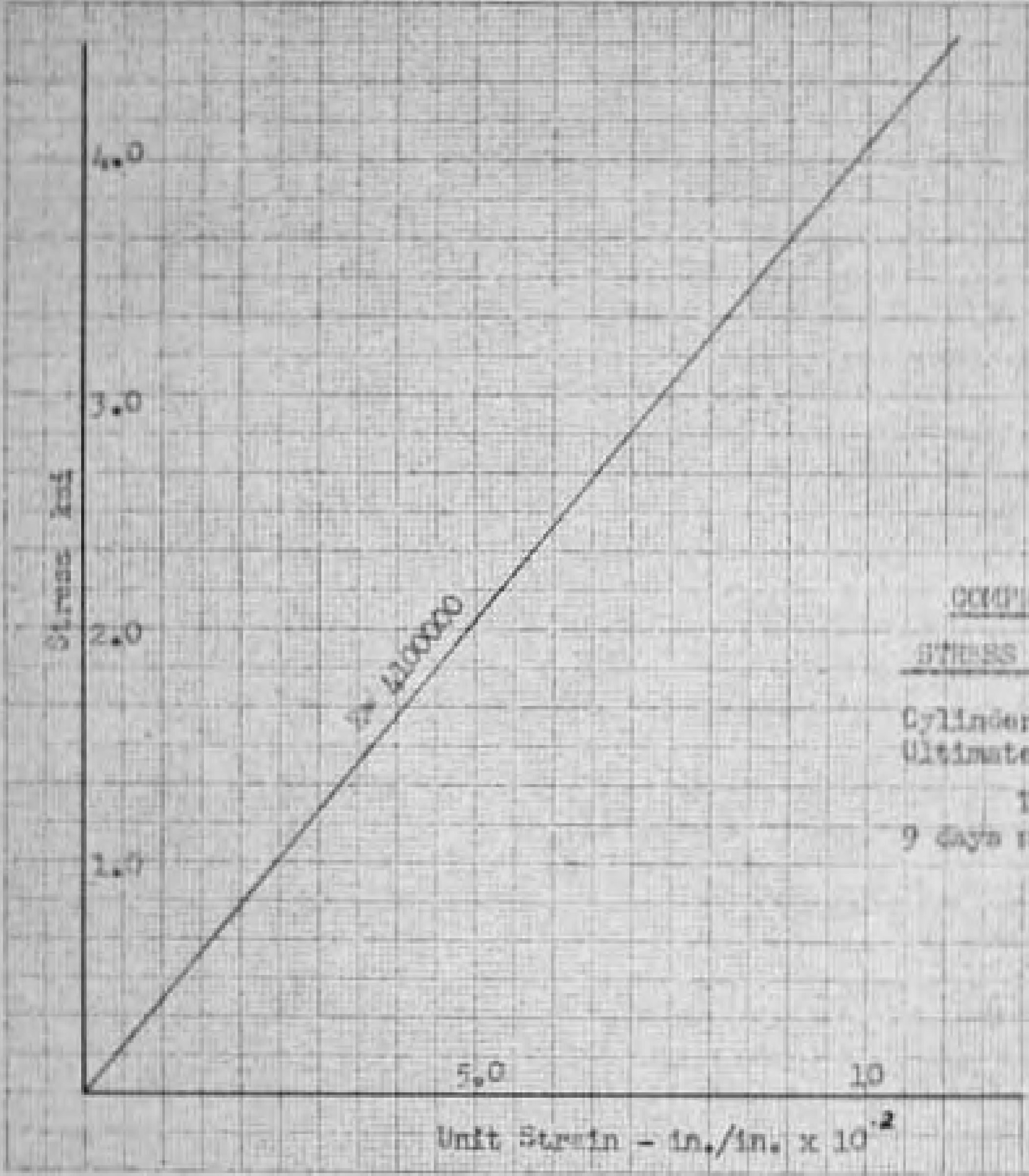


FIG. 44

COMPRESSION TEST

STRESS vs. UNIT STRAIN

Cylinder no. 64, Beam no. 8
Ultimate strength: 6385

Type Curing:
9 days moist

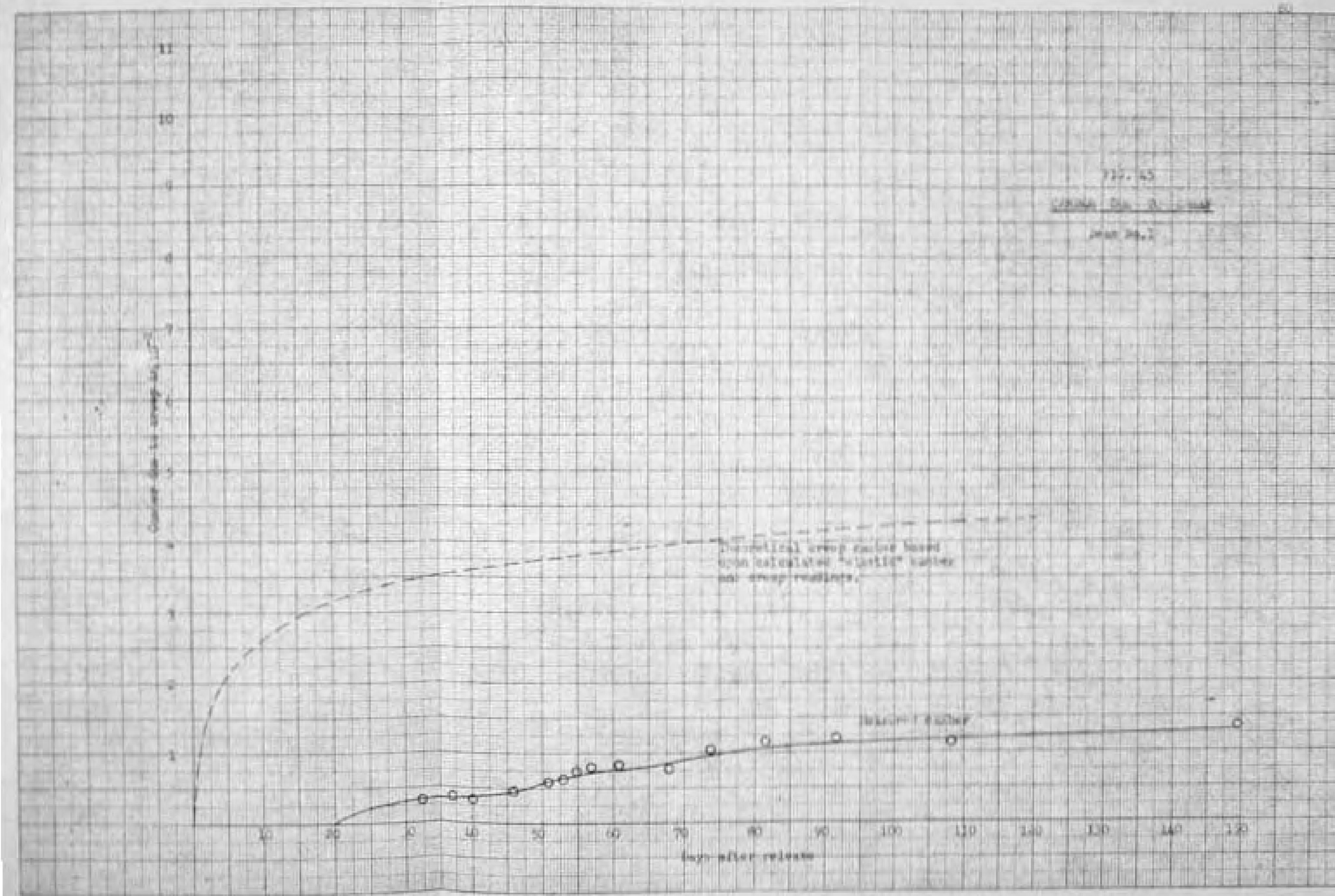
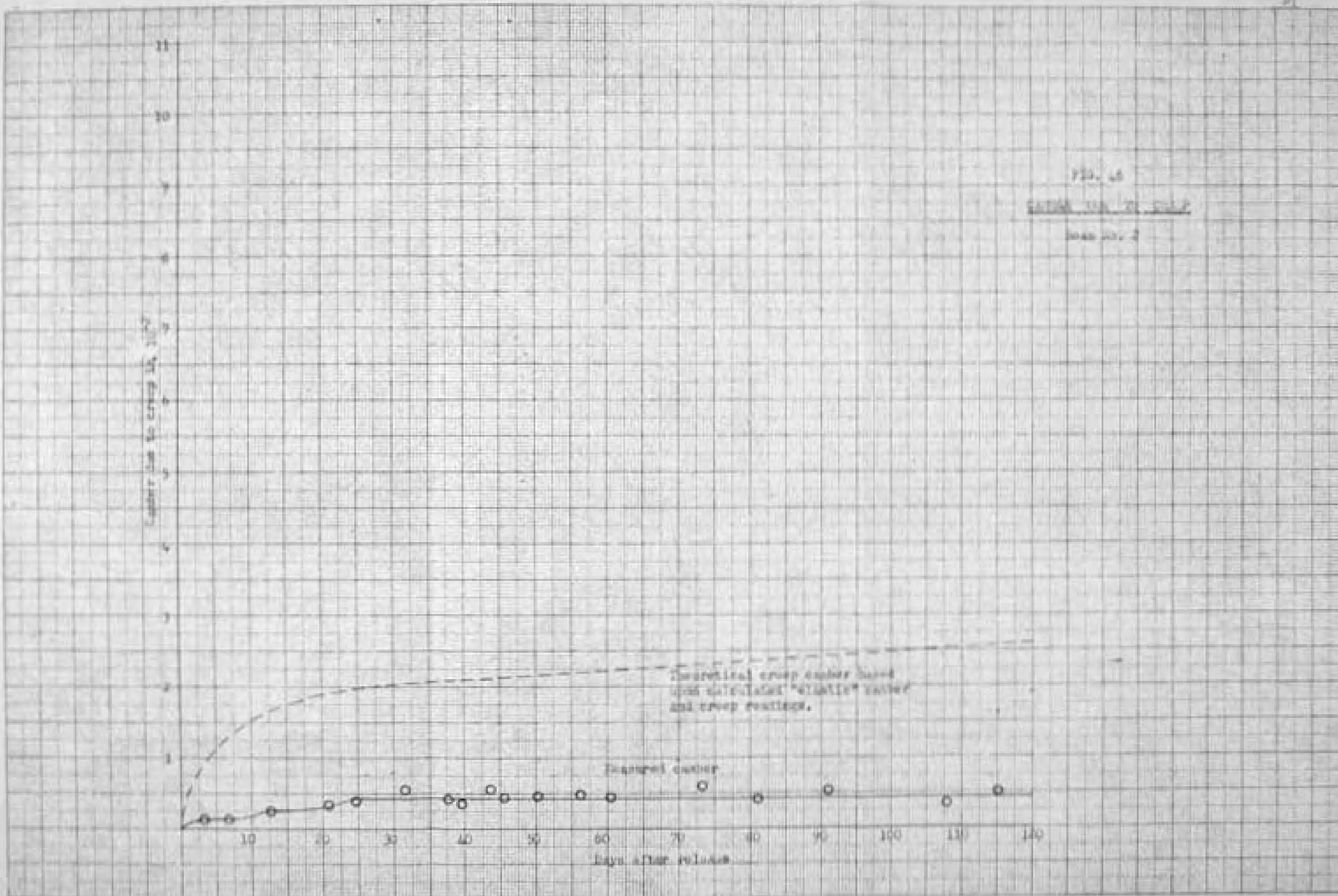


Fig. 45
11
10
1
Theoretical creep curve based upon calculated "elastic" camber and creep readings.
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Days after release



Theoretical creep camber based upon calculated "elastic" camber and creep readings.
Measured camber
Days after release
10
20
30
40
50
60
70
80
90
100
110
120
130
140
1
2
3
4
5
6
7
8
9
10
11

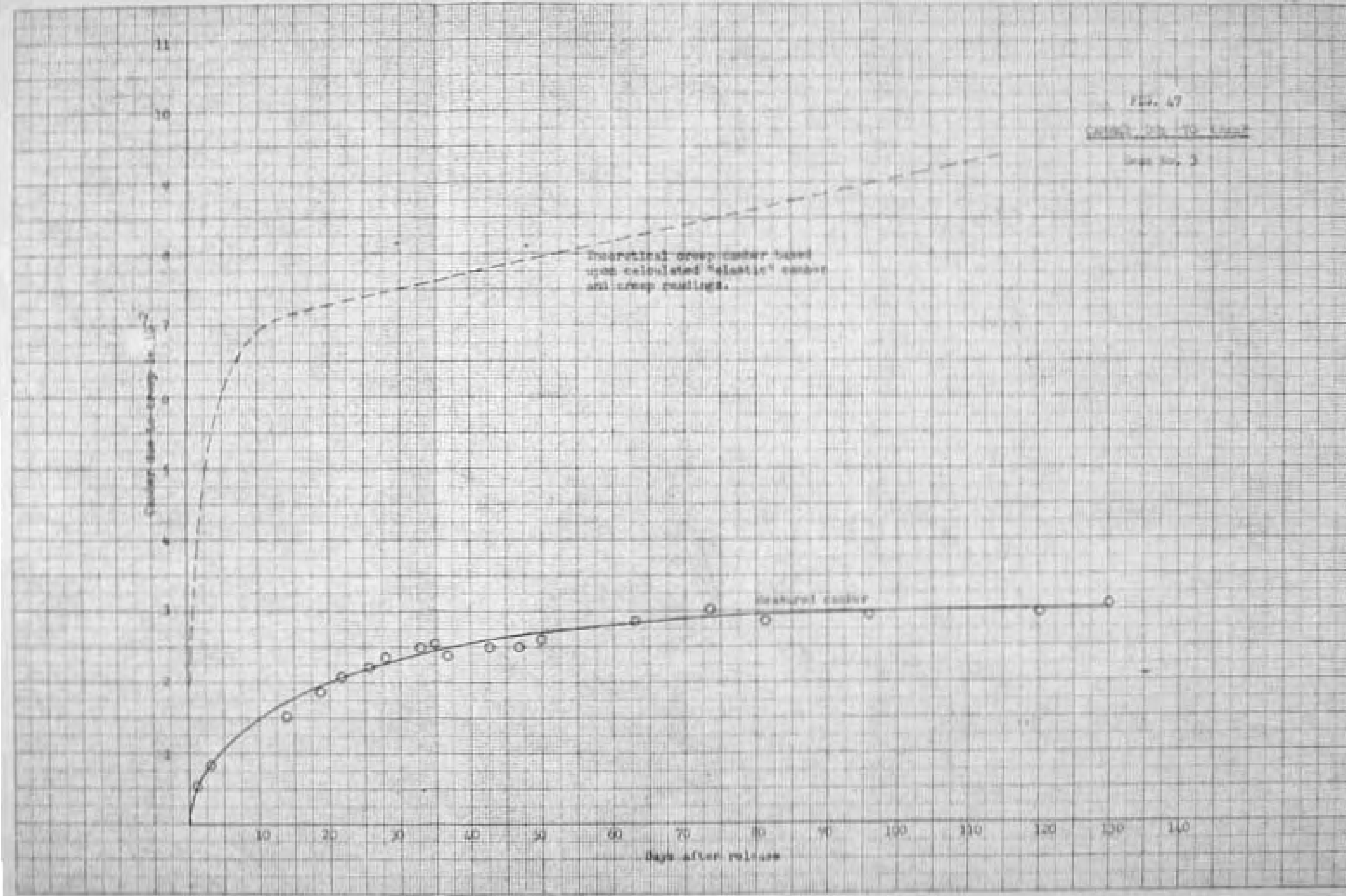


Fig. 47
Beam No. 3
Theoretical creep camber based upon calculated "elastic" camber and creep readings.
Measured camber
Days after release
11
10
9
8
7
6
5
4
3
2
1
10
20
30
40
50
60
70
80
90
100
110
120
130
140

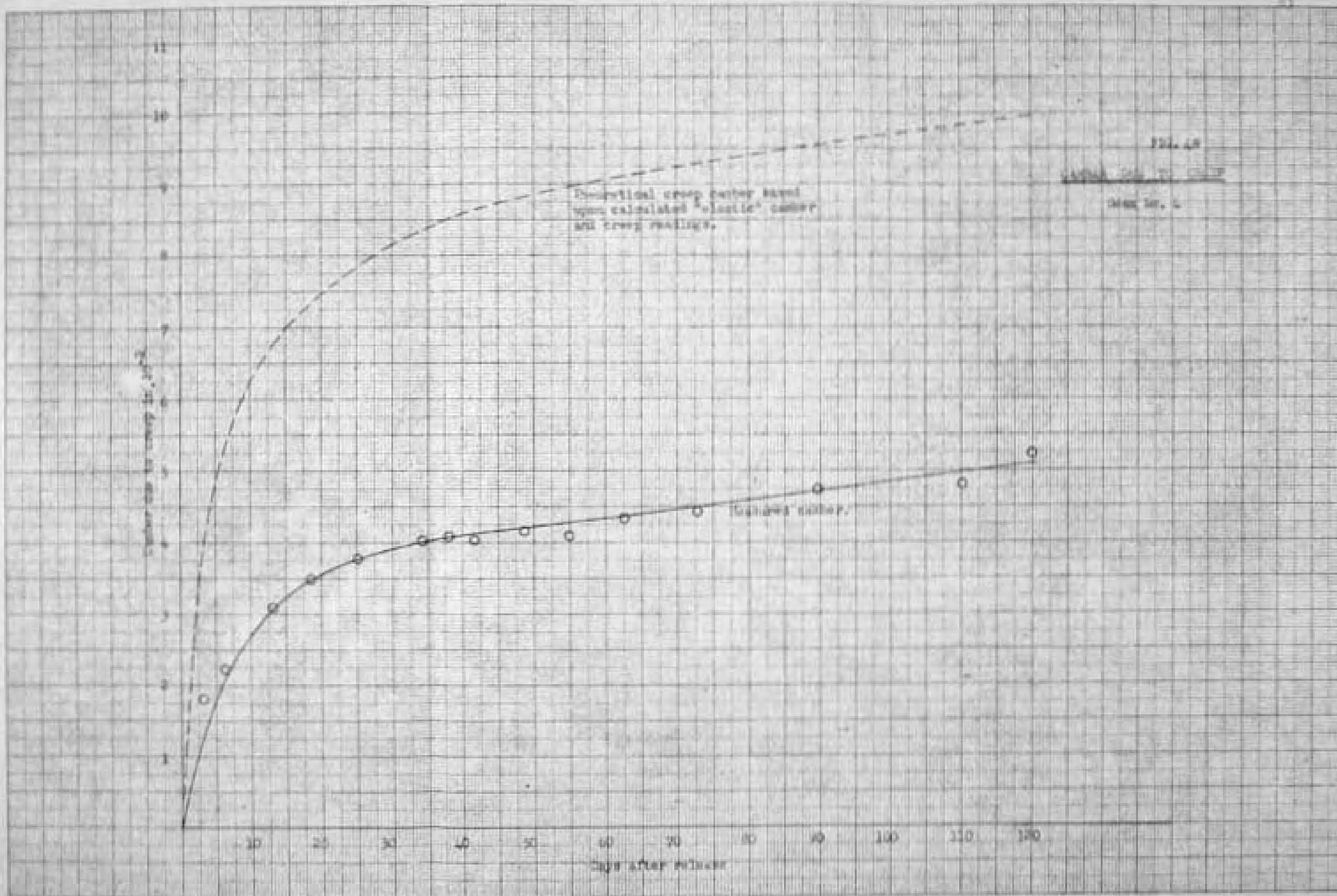


Fig. 48

CAMBER DUE TO CREEP

Beam No. 1

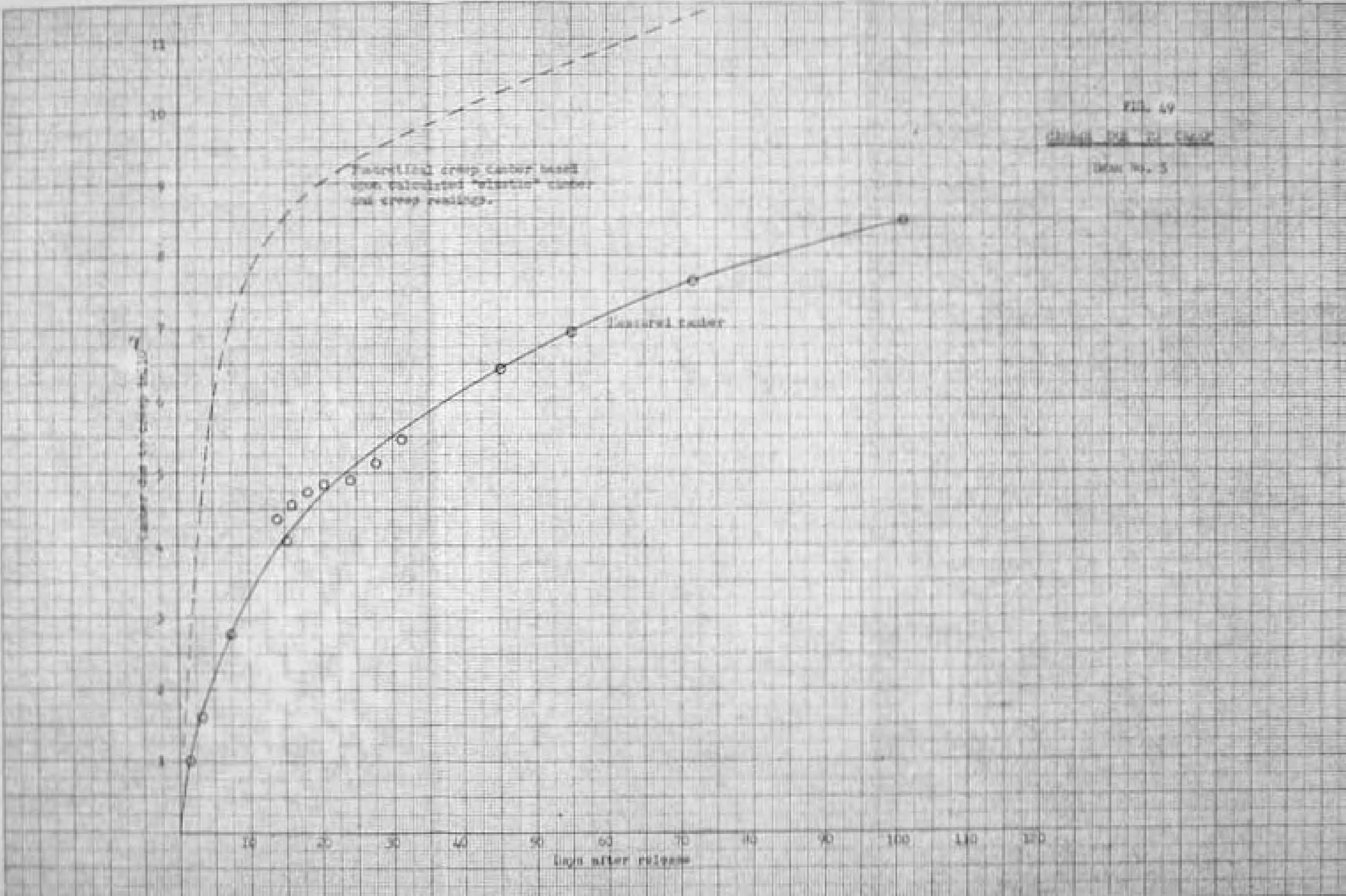


Fig. 49
Theoretical creep camber based
upon calculated "elastic" camber
and creep readings.
Measured camber
Days after release
10
20
30
40
50
60
70
80
90
100
110
120
1
2
3
4
5
6
7
8
9
10
11

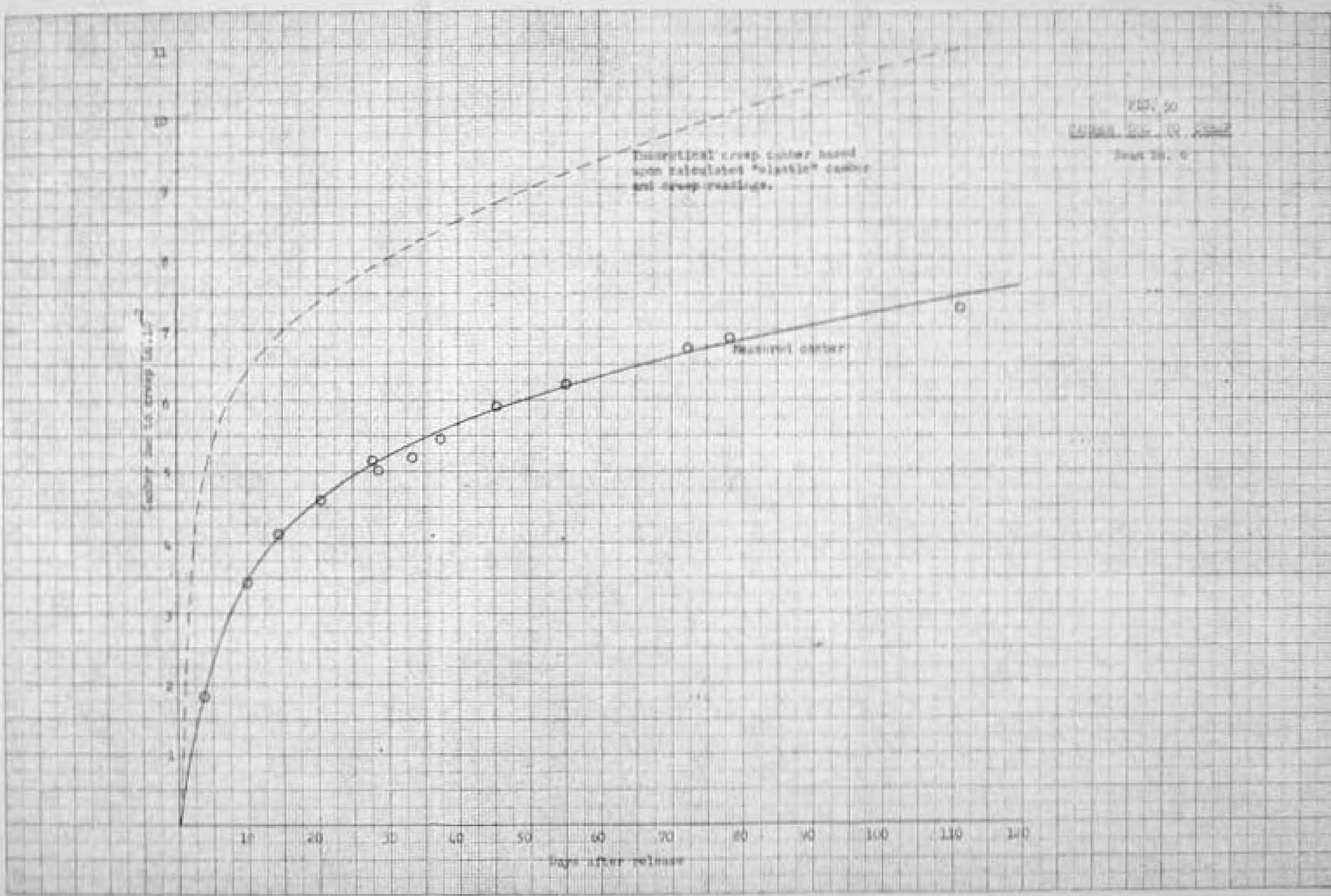


FIG. 30
CAMBER DUE TO CREEP
Beam No. 6

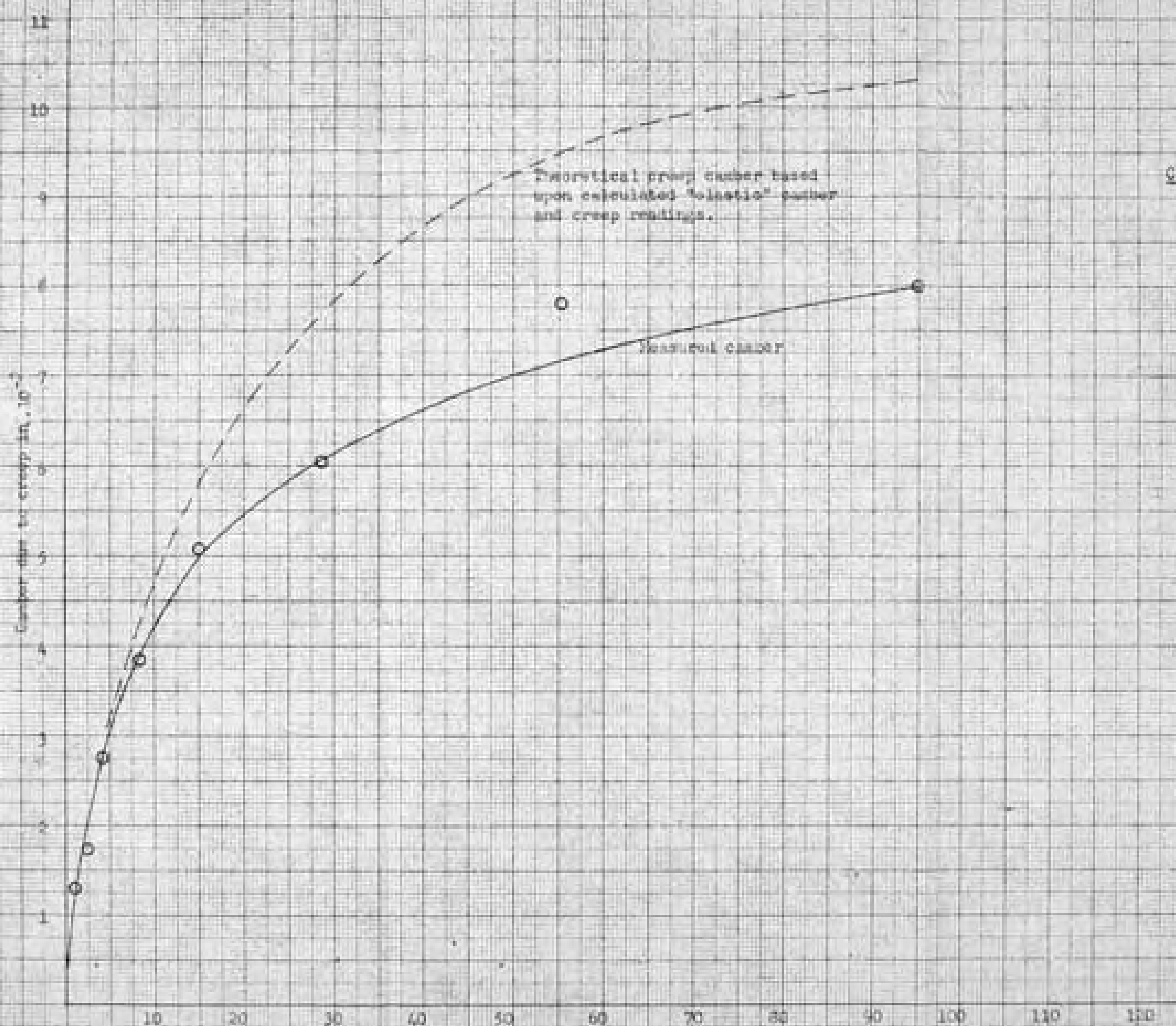
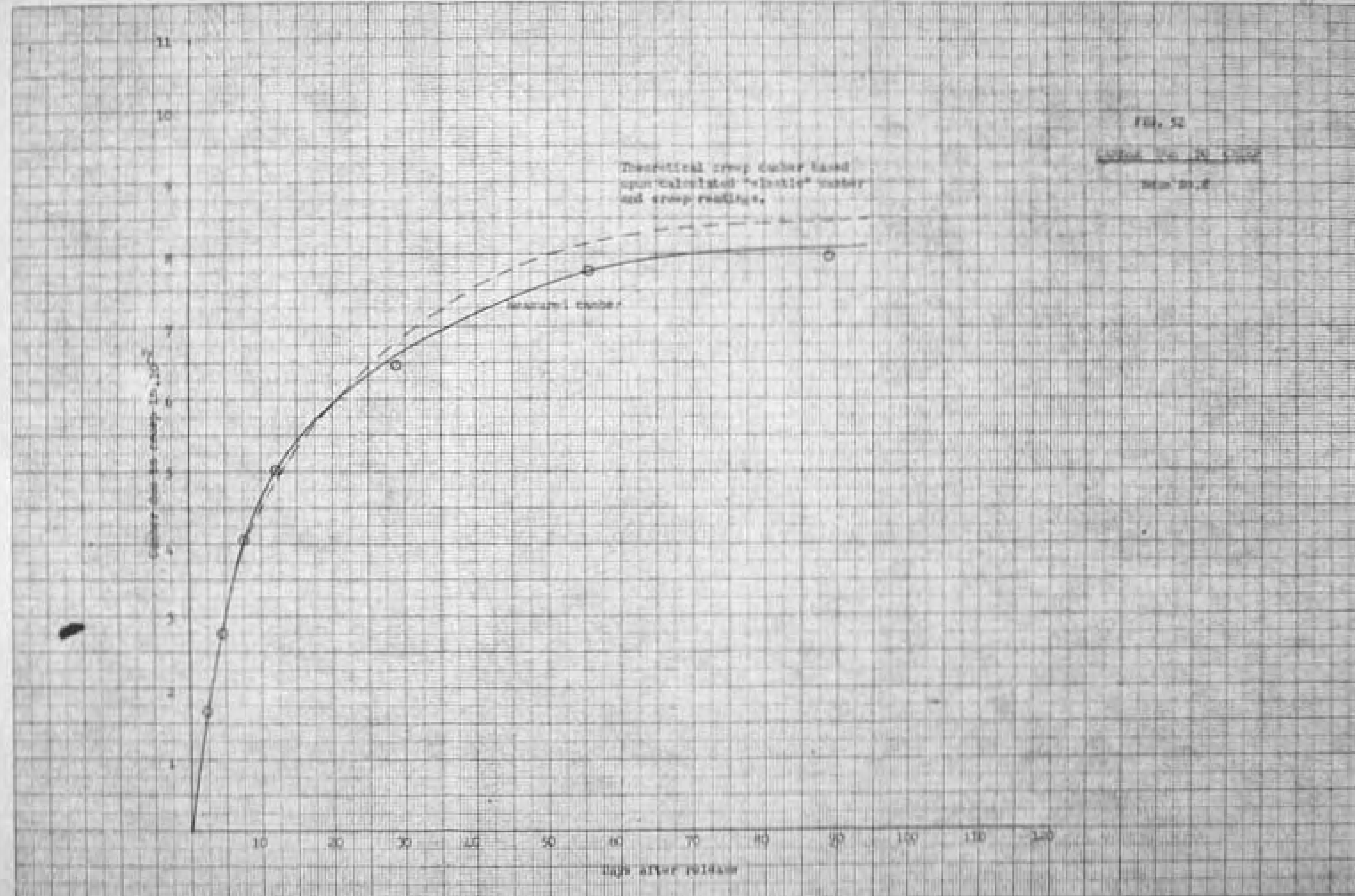


FIG. 51

CAMBER DUE TO CREEP

Beam No.7



Theoretical creep camber based
upon calculated "elastic" camber
and creep readings.
Measured camber
Days after release
10
20
30
40
50
60
70
80
90
100
110
120
1
2
3
4
5
6
7
8
9
10
11

FIG. 52

DISCUSSION OF RESULTS

For all four sets of beams the limestone concrete show more creep than the gravel concrete (Table 5, Figures 25-28). This in spite of the fact that the limestone has a much better record what other concrete properties are concerned. Considering the more plate-shaped structure of the limestone compared with the crystalin structure of the gravel this result might not be surprising. The gravel is no doubt more brittle and will take load to a certain point and then break. The limestone, on the other hand, will have a tendency to yield more and be able to pick up additional load. The cylinder tests on Figures 33-44 show that the limestone concrete always had the highest strength, but the lowest modulus of elasticity. As the aggregate was the only variable in this case, the mentioned properties of the aggregates may be a good explanation for the results. The results may also indicate that the creep is dependent upon the modulus of elasticity and only upon the strength when a higher strength gives a higher modulus of elasticity. We may say that creep is decreasing with increasing modulus of elasticity. Tests performed at Ohio State University (13) show the same results as in this experiment. The tests at Ohio State were performed on concrete cylinders using Ohio Limestone and a Silica gravel. In all tests the limestone concrete showed more creep even when the limestone concrete had a higher strength.

The average modulus of elasticity was $4.2.10^6$ psi with a variation of + 10.7 percent and - 8.3 percent. The value stayed below the usual value of 1000f'c.

We further see that the curves conform very close to Cacout's curve. This can be taken as a confirmation of Cacout's theory or, if we accept Cacout's theory, as a proof of the dependability of the method using gage points.

Looking at Table 7 we see that the elastic shortening of the concrete is smaller for the gravel concrete than for limestone concrete which confirms the higher modulus of elasticity found for the gravel concrete. The calculated elastic shortenings stay pretty close to the measured ones with an average discrepancy of ten percent, the largest being fourteen percent. When calculating the elastic shortening, the prestress force was reduced for elastic shortening.

In spite of all efforts the prestress force was no doubt never exact and in all probability all strands did not carry exactly the same load. This may cause the theoretical resultant to shift from its D/3 position from the bottom. Settlement of the forms under the load of the wet concrete would have had the same effect. On that background the discrepancies must be said to be very small.

Except for the first few days there is a good linear relationship between the stress and creep. (Figs. 29-32) Beam No. 3 is the one deviating most. For the first few days it is hard to get a linear relationship even when using the somewhat idealized curve. Nevertheless the results can be taken as a confirmation of the theory that creep is proportional to stress. In both the elastic and plastic theory it is assumed that plane sections remain plane. With the close correspondence between the measured and calculated strains we may conclude that stress is proportional to strain at least for a distance of 2/3 of the depth.

The expected losses of prestress in the strands are given in Table 6. When using the recommended tensioning and design loads given in the Roebling Company's Catalog a loss of 20 percent is assumed. We see that beams 1-4 keep below this value and that beams 5-8 do not. Beams 5-8 have a design stress higher than the value that is ordinarily used in design. The given allowable design stresses are not reduced for elastic shortening as has been done in this case. The losses are based upon the surface shrinkage and are surely higher than the losses in the inner part of the beam.

Looking back on Table 3 we can study the slump for the mixes. A substantial reduction in the slump could be used; in particular for the gravel concrete as the gravel concrete gives a more workable mix than the limestone concrete, both having the same slump. A reduction of the water content will reduce shrinkage and creep and augment the modulus of elasticity. The concrete used was satisfactory but it is possible to improve its quality, and thus reduce the losses. Steam curing would make it still more favorable. The results show the importance of close control of the mixing of concrete for prestressed concrete.

The measured camber when using a level or the device shown in Fig. 21 did not check too good with the calculated camber. The average difference between the calculated and measured camber using the wire was 15%. With a more exact device a closer correspondence might have been obtained but a 100% correspondence would never be obtained as the assumptions the calculated camber is based upon are not factual. It might be recommended for future projects to have a device which would measure the exact camber and find a correction factor for the calculated camber.

Creep camber measurements were not part of the original plan and measurements for the first two sets of beams must be described as improvised and approximate. The camber readings from beams 1 and 2 were not too accurate as it took some time before the pertinent people learned to move with some care in the vicinity of the gages. The readings for beam 1 showed the beam to deflect downward the first few days as a result of this carelessness and for this reason were not plotted.

It would have been desirable to have stronger springs in the gages. Some of the springs were weak, but they were the only ones available. Since all readings had to be corrected for the movement of the other beams some error might have been introduced except for beam 1 and 2. The method of stacking the beams may have given a constraint to the movement of the beams. The fact that the last beams give a better check between the calculated and measured creep camber might confirm this. Perhaps it can be concluded from the graphs that the factual creep camber will keep below the calculated camber.

Little can be concluded from Table 9 as the top and bottom readings cannot be taken as 100 percent reliable. Comparing the two bottom lines for each beam we might say that it confirms the assumption that plane sections remain plane.

The nature of the bond in the anchorage zone is a disputed question. A slippage may or may not take place. But whatever happens it does not seem likely that there is any significant difference between the nature of the bond for the 1/2" strand and the other smaller sizes. Considering that the cross section of the 1/2" strand is 2.5 times the cross section of the 5/16" strand and that the prestress was almost double one should expect slip-in of the magnitude we got. If the slip-in is proportional to the load we should have expected a slip-in of $17.6 \times 2.5 \times 1.9 = 84 \times 10^{-3}$ inches. This is larger than any of the observed slip-ins. The anchorage length is more than double of that for the 3/8" strand (Table 12) but it must be recalled that the ratio between the cross sections is 1.8 while the ratio between the circumferences is 1:33.

Increase of the slip-in (Table 10) and anchorage length (Table 11) with time checks well with the theory that plastic flow is taking place in the concrete around the strand, thus increasing the anchorage length. The anchorage length was different for the two faces of the beams (west side and east side) because of the difference in surface conditions of the strands. The importance of the surface condition of the strand in developing bond can be clearly seen in Table 11. Guyon's formula for elastic bond gave only reasonable results when the strand was rusty or, in other words, when high bond stresses could be developed.

More research should be done with the 1/2" strand before any definite conclusions can be made.

Electric strain-gages

In general the electric strain gages were not successful. The best results obtained when reading at full load was about 87 percent (1 and 2), 75 percent (5 and 6) and 90 percent (7 and 8, one strand only) of the expected readings. For the first four beams Duco Cement was used, for the last four Armstrong Cement. No significant difference was observed. The results with the bigger strand indicates that the trouble was to get a good clamping pressure all over the gage. A reduced bond will of course give lower readings.

Another possibility is that the hot asphalt used for waterproofing might have affected the gages. This seems possible as the gages put on the strands outside the forms, and therefore not waterproofed, with hot asphalt gave readings up to 93 percent. On beams 7 and 8 the entire gage was covered by a layer of Armstrong Cement before any tar was applied.

The waterproofing of the gages was not satisfactory. After release, the readings were erratic. The quite good correspondence with the measured strain in Table 7 is just a coincidence.

It is likely, that the strain and twisting that takes place when a strand is loaded from no load to full load will tend to impair the waterproofing. To the author's knowledge there is only one case where the use of electric strain gage on strand in concrete was a success (10). In that case the gage was put on when the strand was fully tensioned as only the elastic shortening was wanted. Further, the more favorable A-12 gage was used.

It seems that there are so many factors that could cause the gages to give bad results that further use is not recommended.

CONCLUSIONS

a) The results obtained confirm the theory of Cacout about the rate of creep and that creep is proportional to stress.

b) Concrete with the glacial gravel as aggregate gives less creep under sustained load than concrete with the limestone as aggregate; all other factors being the same. The difference is small but the gravel concrete can be improved more than the limestone.

c) Neither of the two aggregates used in concrete will give excessive prestress losses when using proper mix design and curing conditions.

d) Creep camber seems to be smaller than the theoretically calculated camber.

e) The nature of the bond when using 1/2" strand does not appear to be different from the nature of bond when using smaller size. More investigation is necessary before any definite conclusion can be made on the nature of bond on 1/2" strand as this conclusion is based on the results of two beams only.

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