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Determination of the Distribution of Stresses and Reactions in Wide Reinforced Concrete Beams

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DETERMINATION OF THE DISTRIBUTION OF STRESSES AND REACTIONS IN WIDE REINFORCED CONCRETE BEAMS

BY

OTTO COFFEEN FITZ RANDOLPH

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1913

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May 31, 1913.

This is to certify that the thesis prepared in the Department of Theoretical and Applied Mechanics by OTTO COFFEEN FITZ RANDOLPH entitled Determination of the Distribution of Stresses and Reactions in Wide Reinforced Concrete Beams is approved by me as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

A. N. Talbot
Instructor in Charge.

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A. N. Talbot
Professor of Municipal and Sanitary Engineering
In Charge of Theoretical and Applied Mechanics.

Approved:

Ira O. Baker
Professor of Civil Engineering

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CONTENTS

I. INTRODUCTION

This thesis is to form a part in the series of tests on broad beams being conducted under the general supervision of the Engineering Experiment Station of the University of Illinois. Tests have been made in the last two years to determine the distribution of stress across the beam. These results can not be fully interpreted, however, without some knowledge of the distribution of the reaction along the supports. No machine being available for tests of this kind, it became necessary for the department to design and build a machine in which the amount of reaction could be measured at short intervals across the end of beam. This thesis will describe as faithfully as possible the machine used and the methods pursued in preparing for the test. As is usually the case when entering new territory some minor difficulties were met and some of the initial plans had to be changed. The causes of these difficulties will be explained and the remedies used will be described in detail. The results obtained for a few tests made on one of the beams by this machine will be given and the author's analysis of these results and the conclusions that may be drawn from them. In order to indicate the stage at which the investigation is taken up, a review of the analytical status and of available data will be included.

II. A REVIEW OF EARLIER WORK

1. Former Theses.- The following three theses have been prepared on the general subject of broad beams: (1) Tests of Reinforced Concrete Beams: "Effect of Lateral Distribution of Concentrated Loads", presented by Mr. K. E. Robinson in 1910; (2) "The Effect of Lateral Distribution of Load over Wide Reinforced Concrete Beams", presented by Mr. E. J. Schell in 1911; (3) "An Investigation of the Distribution of Stress over Wide Reinforced Concrete Beams Under Concentrated Loads, presented by Mr. L. L. Livingstone in 1912.

2. Scope of Mr. K. E. Robinson's Thesis.- Mr. K. E. Robinson's thesis covered a test of 30 beams 24 ft. and 36 in. wide. One had a span of 72 in. All the rest were tested on a span of 48 in. The depth to steel of half the beams was 3 in. The remainder had a depth of approximately 6 in. to the steel. None of these beams had any transverse reinforcement. In testing beams, loads varying from one-tenth to one-half the span were used. His results showed that cross reinforcement was needed in beams having the ratio of their width to their span greater than one-half.

3. Scope of Mr. E. J. Schell's Thesis.- Mr. E. J. Schell's thesis covered the testing of 24 beams. The most of these were 24 and 36 in. slabs with spans of 48 in. There were two beams 96 in. wide having a span of 36 in. Two of the 36 in. slabs had spans of 72 in. Two-thirds of these beams were 3 in. thick, the remainder being 6 in. Four of them had transverse reinforcement. His results showed that a very small amount of

transverse reinforcement was needed to develop the full strength of the longitudinal steel.

4. Scope of Mr. Livingstone's Thesis.- Mr. Livingstone's thesis covered the testing of 36 beams all on a span of 48 in. and all but one having a depth of 6 in. This one was used as a standard for comparison and was but 3 in. thick. The beams were about evenly divided between 24 - 36 - 48 - 72 and 96 in. widths. All but twelve of the beams had some transverse reinforcement. The results obtained confirmed those of the former theses, that no transverse reinforcement was needed for widths less than half the span and that for any widths only a small amount of transverse reinforcement was necessary.

5. Efficiency Defined.- In order to compare the results of tests made on different beams, a basis of comparison must be established. A beam in which the full strength of all the longitudinal steel has been developed will be taken as the standard. In a beam loaded over a fraction of its width, if the lateral distribution of stress is not effectively made, the load carried will be smaller than that carried by the standard. The efficiency of such a beam has been defined as the ratio of the load carried to that carried by the standard or full strength beam.

6. Comparison of Previous Tests. Table I gives the efficiencies of similar beams tested in 1910, 1911 and 1912, together with the data of the width, span and reinforcement. The distance from the top of the beam to the center of the reinforcing bars in the first three lines of the table was 3 in. and in the remainder of the beams 6 in. It will be seen from this table that beams of the same span, dimensions, and reinforcement, do not always, fail

TABLE 1

Width in in.	Span in in.	Depth to Steel in in.	Longitudinal Reinforcement						Transverse Reinforcement						1910		1911		1912	
			1910		1911		1912		1910		1911		1912							
			Discription	per Cent	Discription.	per Cent	Driscription	per Cent	Discription	per cent	Discription	per cent	Discription	per cent	Method of Failure	Efficiency	Method of Failure	Efficiency	Method of Failure	Efficiency
24	48	3	7-3/8"-ϕ	1.07	7-3/8"-ϕ	1.07	7-3/8"-ϕ	1.07	—	—	—	—	—	—	T	.99	T	.99	T	.99
24	48	3	7-3/8"-ϕ	1.07	7-3/8"-ϕ	1.07	—	—	—	—	—	—	—	—	T	1.15	T.	.94	—	—
36	48	3	10-3/8"-ϕ	1.02	10-3/8"-ϕ	1.02	—	—	—	—	—	—	—	—	S	.86	S	.89	—	—
36	48	6	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	—	—	—	—	—	—	—	—	T	.82	T	.96	—	—
36	48	6	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	—	—	—	—	—	—	—	—	D	.83	D	.61	—	—
36	48	6	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	—	—	—		—	—	S	.57	S	.70	S	.52
36	48	6	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	12-1/4"-ϕ	.22	12-1/4"-ϕ	.22	4-3/8"-ϕ	.23	D	.87	D	.83	B_1	.69
36	48	6	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	20-3/8"-ϕ	1.02	12-3/8"-ϕ	.49	12-3/8"ϕ	.49	8-3/8"-ϕ	.46	D	.95	D	.83	B_1 & D	.94
36	72	3	10-3/8"ϕ	1.02	10-3/8"ϕ	1.02			—	—	—	—	—	—	T	1.05	T	.98		
96	30	6	—	—	24-1/4"ϕ	.052	39-7/16"ϕ	1.20	—	—	—	—	10-7/16"ϕ	0.78	—	—	T & D	.37	B_1	.43

T = Tension Failure.

D = Diagonal Tension or Shear.

B_1 = Bond Failure in Longitudinal Reinforcement.

S = Slab Failure.

in the same way. The five beams having a width one-half the length gave efficiencies averaging about unity. The thickness of these slabs was one-eighth of their breadth. Those beams that failed as a slab have an average efficiency lower than that of beams failing by any other means. The beams having a depth one-twelfth of their width failed as slabs at a much higher efficiency than those having a thickness of one-sixth their width. Those having transverse reinforcement failed in diagonal tension at efficiencies which average higher than the other beams of the same dimension.

7. Relation of Width of Span.- Mr. Livingstone made four diagrams in 1912 showing the relation of efficiency to the ratio of width to span for various degrees of transverse reinforcement. These would seem to show that after the ratio of width to span passed below a value of .5 the efficiency approaches one. As the ratio grows larger than .5 the efficiency decreases rapidly. The four curves each having a different ratio of transverse reinforcement are approximately parallel.

8. Transverse Reinforcement.- Mr. Livingstone also presents four diagrams which show the way in which the efficiency varies with the transverse reinforcement. These curves are surprisingly flat, the one for a beam 96 in. wide changing its efficiency only .1 for a change in the transverse reinforcement of .6%. These curves coincide very well with the conclusion reached by Mr. E. J. Schell in 1911, that a small amount of

cross reinforcement would prevent failure as a slab.

9. Livingstone's Analysis of Cross Reinforcement.- Mr. Livingstone in 1912 developed the following equation in order to determine the proper ratio of transverse reinforcement to the longitudinal reinforcement. The equation is based on the following assumptions: (1) The shear between any two adjacent strips is approximately proportional to the difference between the steel stresses in the two strips; (2) that for the same condition of loading, the load carried by any longitudinal strip of a beam is proportional to the stresses developed in the longitudinal steel in that strip; and (3) that the load is not distributed over a width greater than twice the span.

10. Load Line.- According to the above assumptions, the lateral distribution of the load is a straight line.

11. Equation Developed by Mr. Livingstone.-

Where K_2b = width of load

K_1 = coefficient depending on loading

d = effective depth of longitudinal reinforcing

d_t = effective depth of transverse reinforcing

p = steel ratio for longitudinal reinforcement

p_t = steel ratio for transverse reinforcement

$$\frac{p_t}{p} = \frac{K_2{}^2(1-1/3K_2)}{8K_1} \times \left(\frac{d}{d_t}\right)^2$$

12. Application of Livingstone's Formula.- There is not enough data available fully to test this equation. The experiments made by Mr. Livingstone gave results that approximately

agreed with it. The equation does not enable us to judge what load may be carried when no lateral reinforcement is used.

13. Survey of Work to be Accomplished.- Light will be thrown upon the action of such beams if we can determine the distribution of the reaction along the length of the support. Tests in which the intensity of the reaction along the support is measured as well as the stress in the longitudinal reinforcing bars at various points in the beam, should be of assistance in determining how the shear is distributed laterally. They will also help in judging of the correctness of the hypothesis used by Mr. Livingstone. The test plan for 1913 had these matters in view.

III. TESTING MACHINE

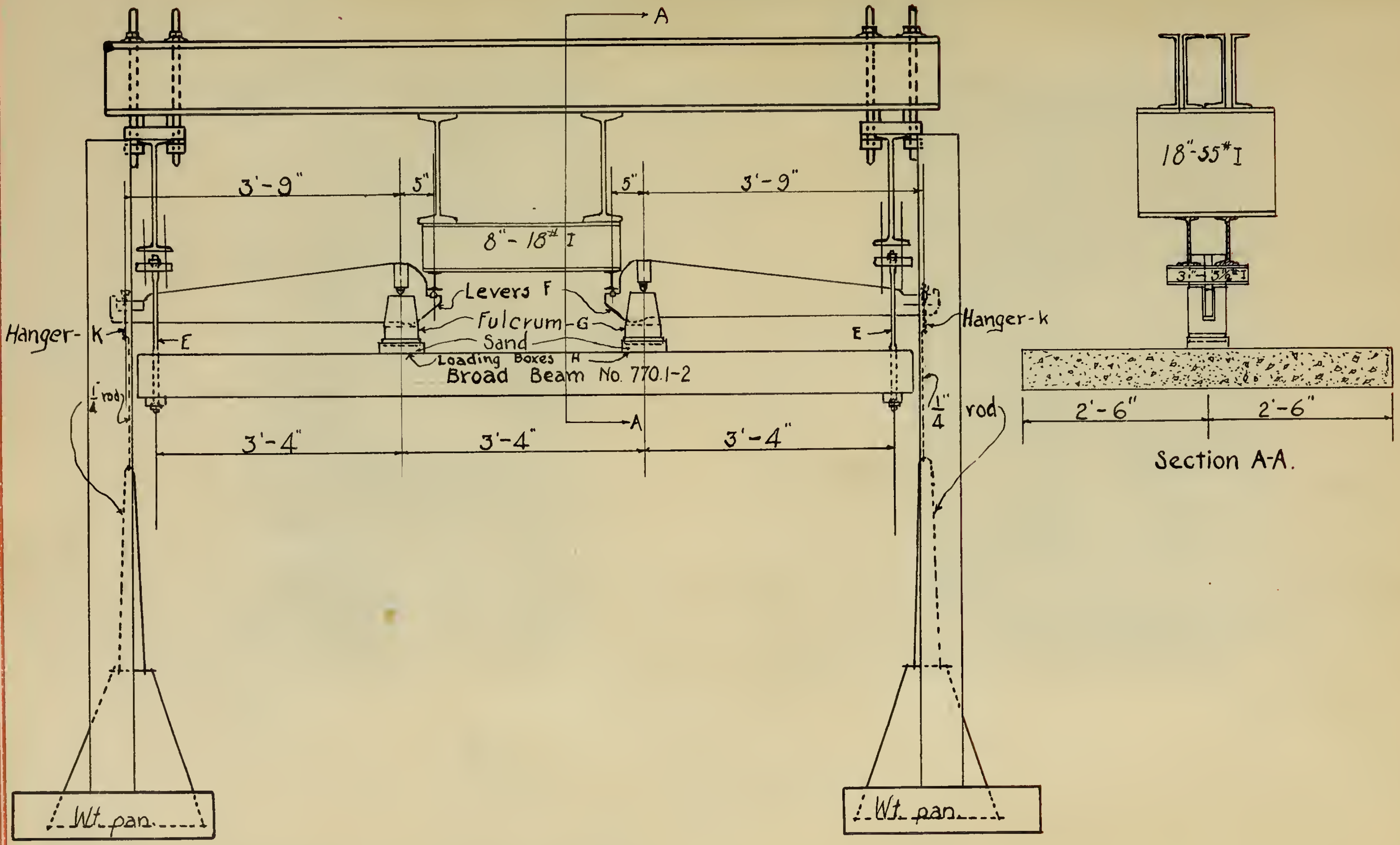
1. General Description.- The testing machine consists of four posts which hold two I-beams from which the specimen to be tested is suspended. The load is applied by exerting a pressure between the test piece and four channels which are clamped to the I-beam (Fig. 4).

2. Method of Hanging Rod.- The basic element of the design for the machine was the suspension of the test piece by a series of hanger rods (Fig. 4 and Fig. 2-E). These rods were made large enough to support the beam and its largest loads, and at the same time, enough to give measurable deformations for small load increments. These rods fitted into holes (Fig. 5) in the test piece which were large enough to give the rods considerable play. This large clearance was given to prevent wedging of the rod and to so insure a direct tensil stress in it.

3. Bearing Plate.- A steel bearing plate 4 in. sq. x 1/4 in. (Fig. 5) thick was set in the bottom of the test piece around each hole.

4. Bottom Bearing Blocks.- Square bearing blocks (Fig.I-B) were provided for each rod to bear on the plates set in the concrete. These bottom bearing blocks were 3 in. sq. x 1/4 in. thick and had a machined bearing countersunk in their under side.

5. Bearing Nuts.- In these blocks, nuts machined to fit had bearing on a circular element. These bearing nuts were 1 3/4 in. in diameter and 1 in. thick. They had 1/4 in. holes drilled in their side 120° apart to give a grip for a spanner



Section A-A.

ASSEMBLY OF TESTING MACHINE

Fig. 4.

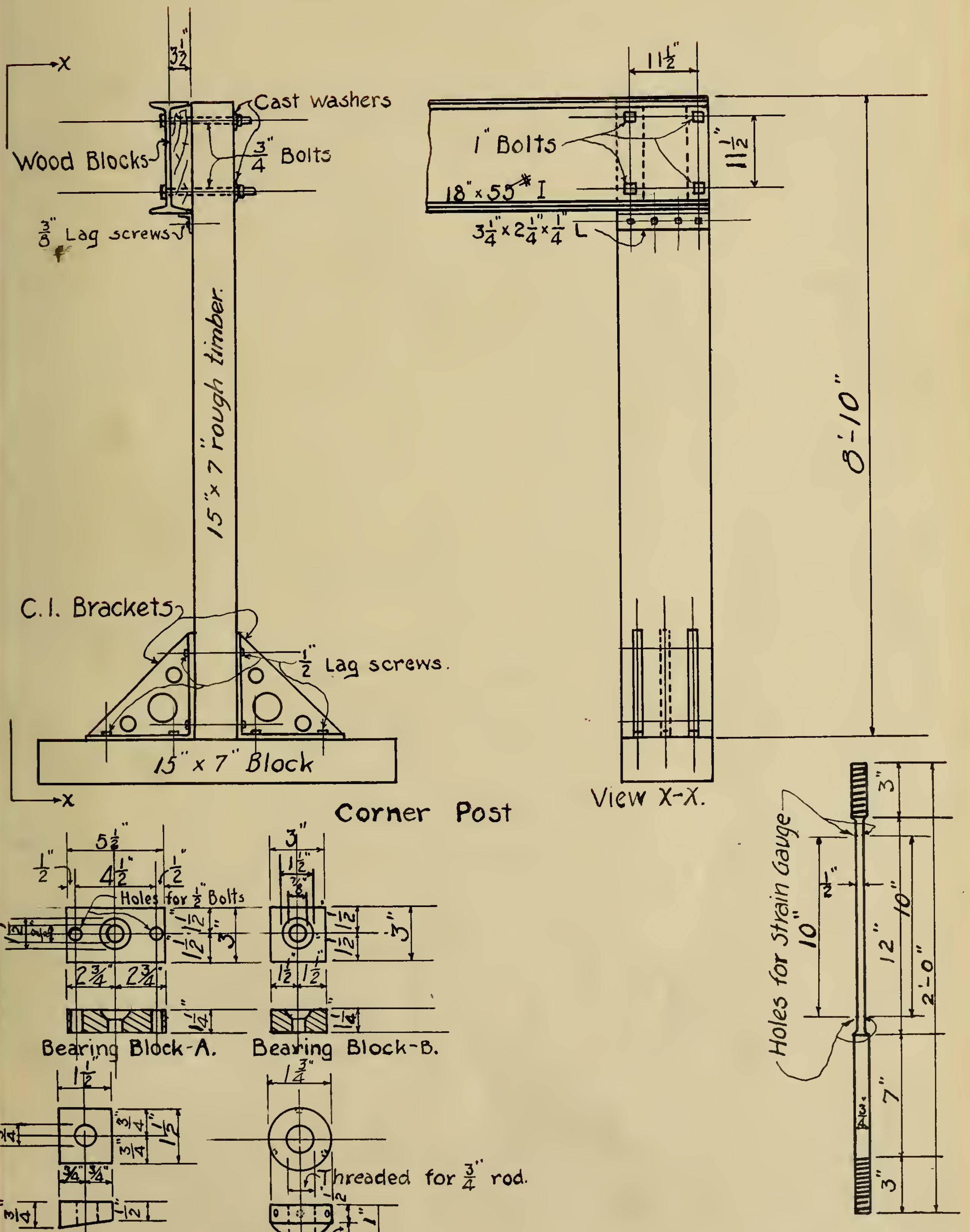
wrench.

6. Top Bearing Blocks.- Nuts on the top of rods had bearings in rectangular blocks (Fig. I-C) the same as those of the bottom nuts. These top bearing blocks (Fig. I-A) were 5 1/2 x 3 x 1 1/4 in. Holes were drilled on 4 1/2 in. centers for 5/8 in. bolts thru the blocks.

7. Supporting I-Beams.- The rods were supported overhead by means of the top bearing blocks bolted to the lower flange of an 18 in. - 55 lb. - I-beam. Each I-beam was supported by 15 x 7 in. corner posts (Fig. I) which were about 10 ft. apart. The I-beams were connected to each post by four 3/4 in. bolts and also rested on 3 1/4 x 2 1/4 x 1/4 in. clip angles which were fastened to the post by four 1/2 in. lag screws. (Fig. 7) For stability the posts rest on 15 x 7 in. sills. There were two frames as described, each frame supporting one end of the test piece.

8. Connection of Bearing Blocks to I Beam.- Holes for the bolts to connect the bearing blocks to the I-beam were drilled in the bottom flange of the I-beam, one hole on each side of the web. These holes were not opposite each other (Fig. 2), but were on 3 3/8 in. centers measured along the I-beam and 2 5/8 in. centers measured across the beam. This gave the long axis of the blocks an inclination of approximately 30° with the line of intersection of the I-beam web and bottom flange. These blocks were bolted to the flange by 5/8 in. bolts.

9. Special-Beveled Washers.- Special bevelled washers (Fig. 1-C) were used to give an even bearing on the I-beam flange.



Corner Post

View X-X.

Bearing Block-A.

Bearing Block-B.

Cast washer C.

Bearing Nut-D

Hanger Rod-E.

Fig. 1

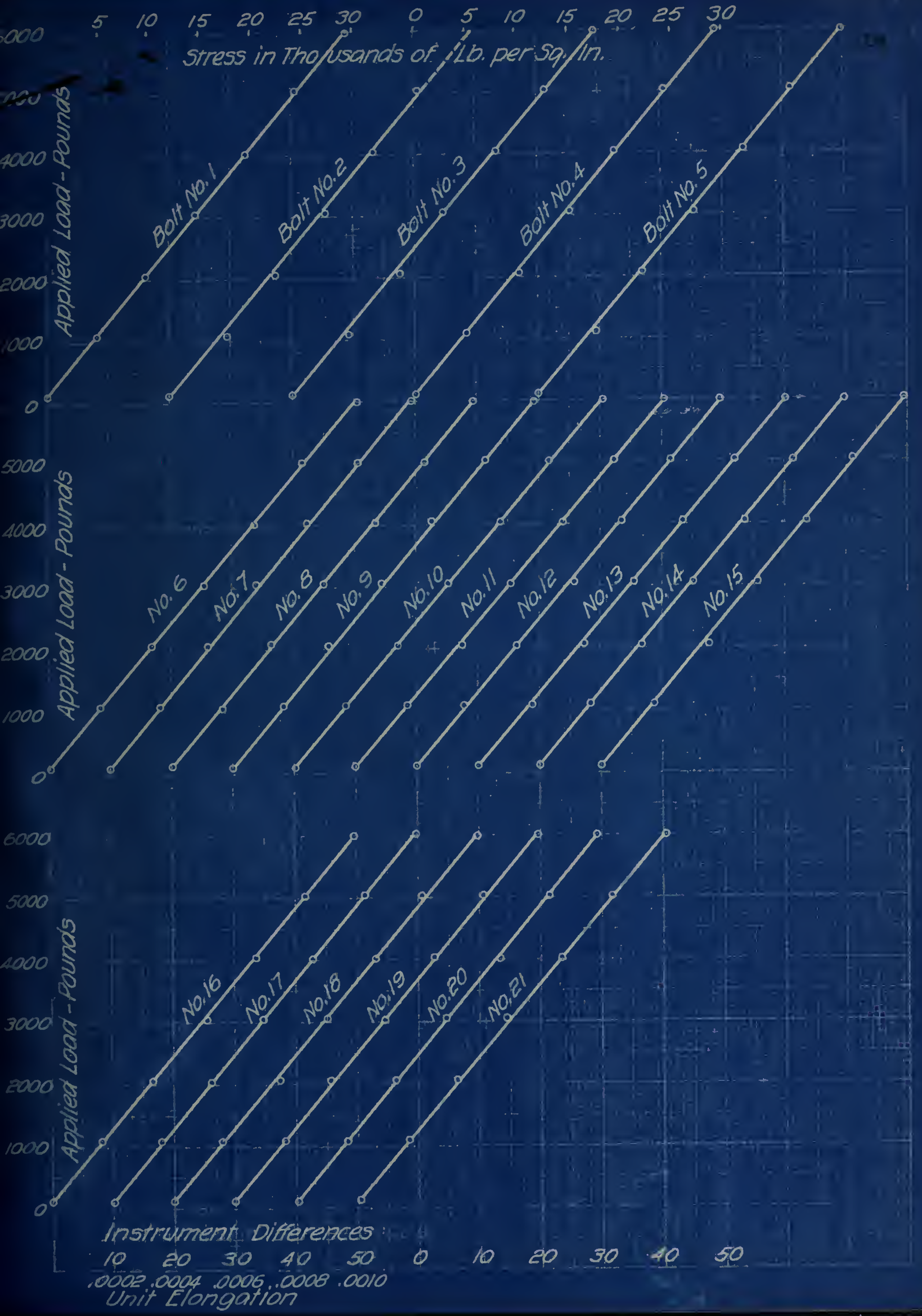
These washers were cast with one side having the same bevel as that of the I-beam flange, the opposite face being perpendicular to the other four sides. When placed with the bevelled side next the flange of the I-beam, the bolts when tightened between the bearing block and the flange always had a direct tensile stress along their vertical axis.

10. Pipe Spacers.- There was a 1 3/4 in. space (Fig. 2) left between the bottom of the I-beam and the top of the bearing block. This clearance was maintained by pipe washers. The clearance was necessary to provide access to the nut D (Fig. 1) on the top of the hanger rod.

11. Hanger Rods.- The hanger rods (Fig. 1-E) were 3/4 in. steel, 24 1/2 in. long with 3 in. of thread on each end. One end was left rough for a distance of 3 in. and the other for a distance of 10 in. The space between these two sections was machined smooth to a true diameter of 1/2 inch.

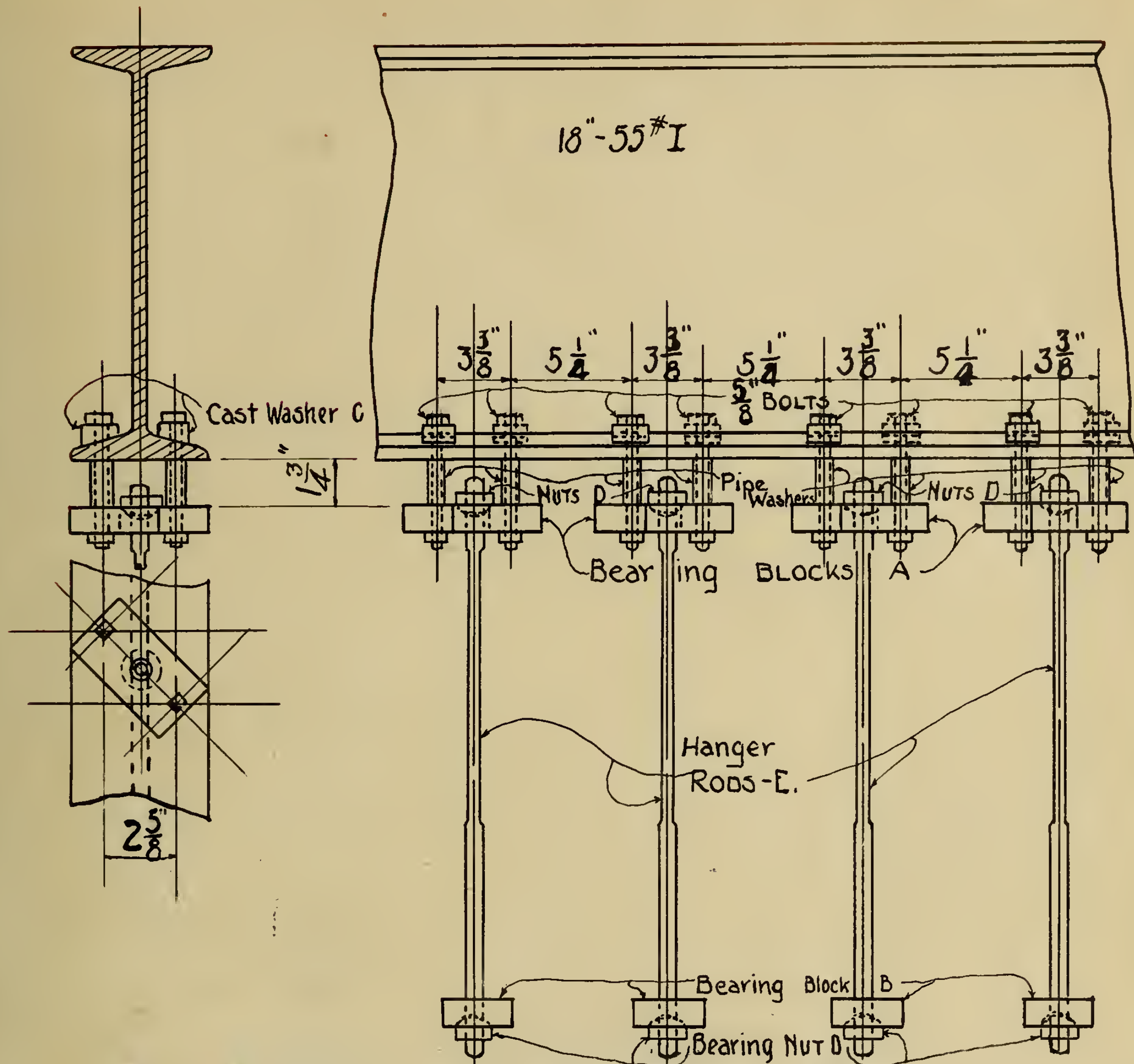
12. Bearing Nuts.- Each of these rods is provided with two bearing nuts (Fig. 1-D).

13. Calibration of Hanger Rods.- The hanger rods from which the beams were suspended were calibrated by placing them in a Riehle testing machine. The rods were placed in the machine in a vertical position and stressed in direct tension. Load increments of 5000 lbs. per sq. in. were made until the rod was stressed to 30,000 lbs. per sq. in. A reading on a gage line on one side of the bar only was taken with a 10 in. strain gage at each load increment. Curves drawn from this series of tests for all the rods varied from the theoretical curve drawn

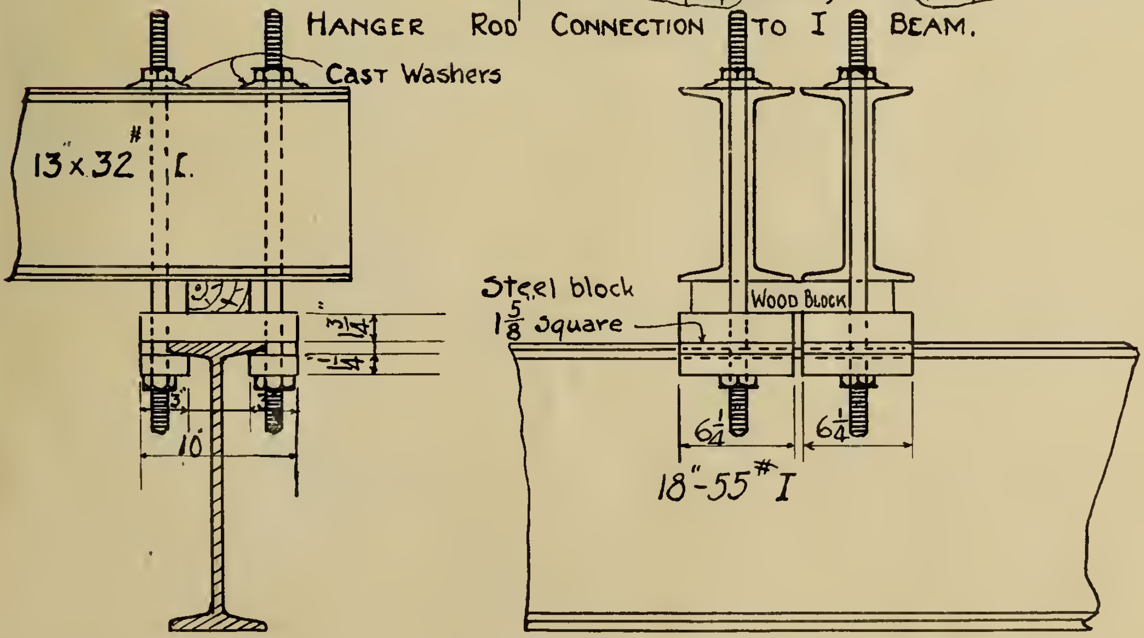

5
10
15
20
25
30
0
5
10
15
20
25
30
Stress in Thousands of Lb. per Sq. In.
000
000
4000
3000
2000
1000
0
Applied Load - Pounds
Bolt No. 1
Bolt No. 2
Bolt No. 3
Bolt No. 4
Bolt No. 5
5000
4000
3000
2000
1000
0
Applied Load - Pounds
No. 6
No. 7
No. 8
No. 9
No. 10
No. 11
No. 12
No. 13
No. 14
No. 15
6000
5000
4000
3000
2000
1000
0
Applied Load - Pounds
No. 16
No. 17
No. 18
No. 19
No. 20
No. 21
Instrument Differences
10
20
30
40
50
0
10
20
30
40
50
.0002
.0004
.0006
.0008
.0010
Unit Elongation

under the assumption that Young's modulus for the material was 30,000,000 lb. per sq. in. The curves also varied from each other. This variation was thought to be caused by a slight eccentricity in the loading of the bar. Holes for gage readings were accordingly drilled on the opposite side from that on which the first set of readings had been taken. Now if the rods were stressed again, a mean of the readings from the two sets of holes opposite each other should give the true change in length. The rods were accordingly stressed once more and a new set of readings taken in the same manner as before, except that in this instance the readings were taken for both sides of the bar. The mean differences as calculated from these readings were now plotted against the loads. These curves checked with each other as closely as the scale could record and also checked with the curve drawn assuming Young's modulus to be 30,000,000 lb. per sq. in. This set of curves accompanies this thesis and was used in determining the loads in the rods, the first set of curves being rejected.

14. Channel Connections.- It was planned to apply the load by exerting a pressure between the beam and four 13-in. x 32 lb. channels which rested on the I-beams and was clamped to the top flange of the I-beams (Fig. 2). The clamp consisted of two steel blocks, one on each side of the I-beam flange thru which 1 in. bolts extended. Each bolt led between a pair of channels which were placed back to bask. The bolts were tightened on top by a nut on cast washers. There were thus four bolts on each end of the channels (Fig. 2). The bolts were drawn up tight so there was a bearing between the channels and the I-beam. This arrangement was to secure a direct stress in the bolts without eccen-



HANGER ROD CONNECTION TO I BEAM.



DETAIL OF CHANNEL CONNECTION TO I-BEAM.

FIG. 2.

tricity. The beam in testing position (Fig. 4) was suspended from the rods, which were on 7 5/8 centers and the lower nuts were tightened till all the rods had an equal stress as nearly as could be judged from the tightness of the nuts.

15. Loading Boxes.- The load was applied at the one-third points of the span thru a sand cushion which was used to secure an even bearing. This sand cushion was obtained by using specially made boxes (Fig. 3-11) which were 7 in. square by 1 1/2 in.deep. The boxes were centered over the points to be loaded and filled three quarters full of ordinary dry river sand. Steel blocks 6 1/2 x 6 x 1 in. were then placed on the sand and the load applied on these blocks.

16. Method of Applying the Load.- In the first tests the load was applied by means of a hydraulic jack, placed upon an I-beam (whose ends rested on the two loading blocks) pushing against the channels. A dynamometer was used in order to determine the loads applied by the jack.

17. Calibration of Dynamometer.- A calibration was made by loading the dynamometer in a 200,000 Olsen machine and noting its deflection. A curve drawn from these observations with the deflections plotted against the load increments is given in Fig. 7.

18. Failure of Jack.- When the jack was tried, it was found that a load placed upon the beam fell off so rapidly that it was impossible to maintain a constant load during the time that readings were being taken. It was known that the jack leaked, but it was thought probable that part of this falling off of load was

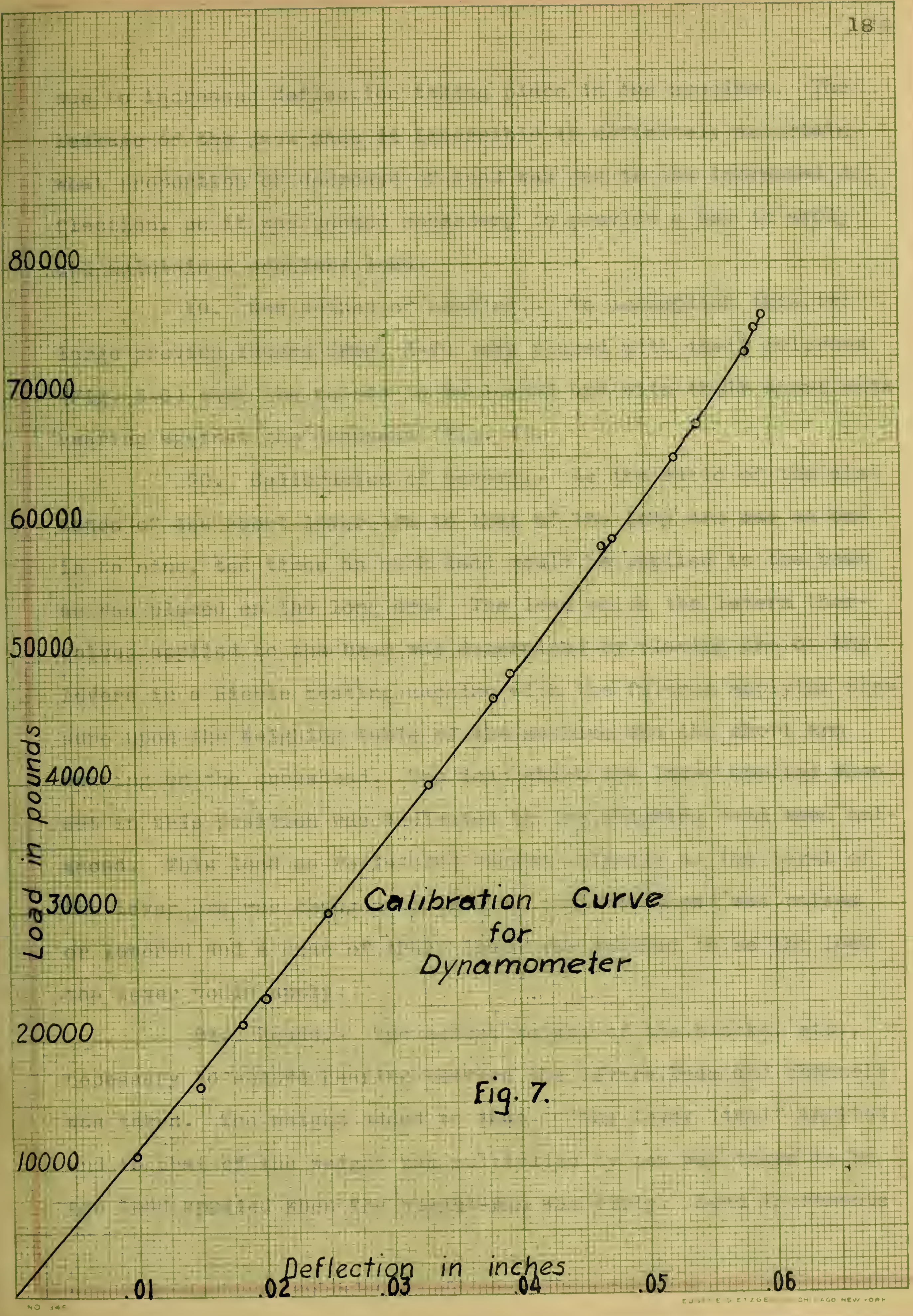

80000
70000
60000
50000
40000
30000
20000
10000
Load in pounds
.01
.02
.03
.04
.05
.06
Deflection in inches
Calibration Curve
for
Dynamometer

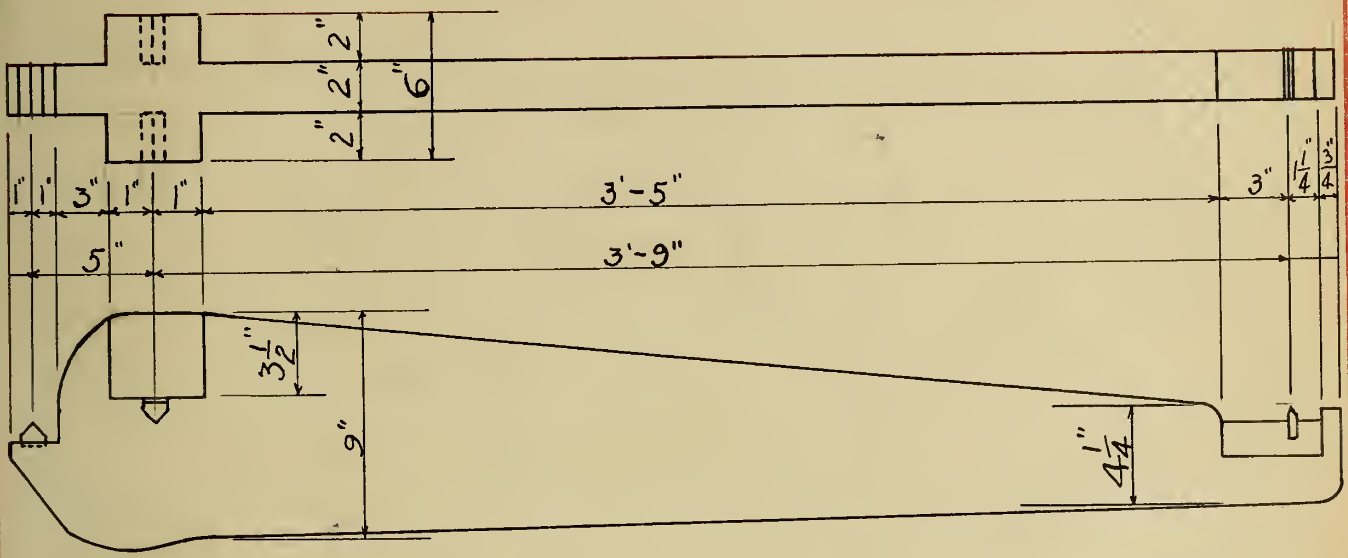
Fig. 7.

due to increased deflection taking place in the specimen. The leakage of the jack made it impossible to definitely ascertain what proportion of decrease of load was due to the increased deflection, so it was deemed necessary to provide a way to apply and maintain a constant load.

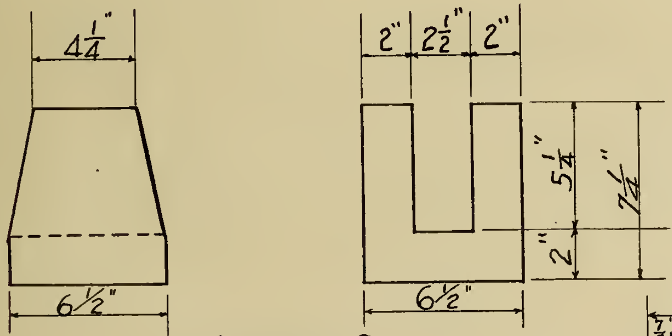
19. New Method of Loading.- To accomplish this, two large proving levers (Fig. 3-F) were placed with their fulcrums (Fig. 3-G) over the points to be loaded and with their short ends bearing against the channels (Fig. 4).

20. Calibration of Levers.- As the ratio of the distance of the short lever arm to that of the long arm was as one is to nine, ten times as much load would be applied to the beam as was placed on the long arm. The load which the levers themselves applied to the beam was determined by placing one of the levers in a Riehle testing machine with the fulcrum applying pressure upon the weighing table of the machine and the short arm bearing on the crosshead. The load which the lever applied when set in this position was indicated by the weighing beam when balanced. This load as registered varied slightly as the level of the lever arm was changed, i. e., as the cross-head was raised or lowered and a mean of these loads was assumed to be the load the lever would apply.

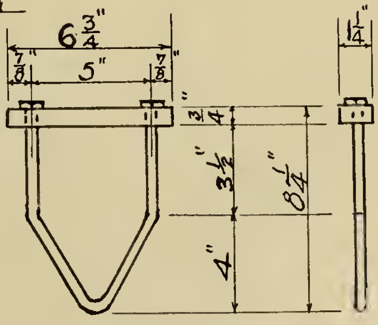
21. Loads.- The actual weight of the blocks, etc., necessary to secure bearing between the levers, beam and channels was taken. The weight added to that the lever itself applied and to that of the weight pan multiplied by ten was taken to be the load applied when the weight-pan was empty. Load increments



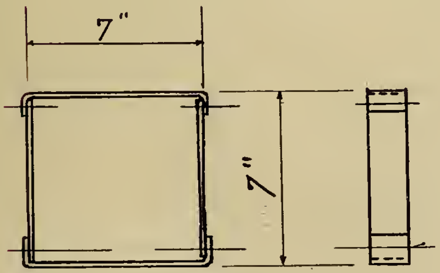
PROVING LEVERS. - F



Fulcrum.- G



Hanger.-K.



Loading Box.-H

Fig. 3.

were made by placing standard weights in the weight-pan.

22. Method of Removing Load.- In order to facilitate the removal of the load from the proving levers, two hooks corresponding to those used to suspend the weight pans from the levers were made. These hooks were suspended from the crane and so enabled the pans and their load to be easily lifted off the levers and set upon the floor from which position they could be readily lifted again and replaced.

23. Strain Gages.- Measurements of deformations were made by strain gages. Four of these gages were used, viz; two 4 in. and one 8 in. and one 10 in. The 8 and 10 in. gages are fully described in Bulletin No. 64 of the Engineering Experiment Station of the University of Illinois. The 4 in. strain gages are described in a paper prepared for the American Society for Testing Materials by Mr. W. A. Slater and Professor H. F. Moore in 1913. Photographs of these instruments are shown on plates 1 and 2.

IV. MATERIALS

1. Materials.- The sand and stone were ordered under the same specifications as those given in the bulletings of the Engineering Experiment Station.

2. Stone.- The stone was a good quality of limestone from Kankakee, Illinois, ordered screened thru a 1-inch and over a 1/4-inch screen and was graded as shown in Table 2 copied from L. L. Livingston's thesis.

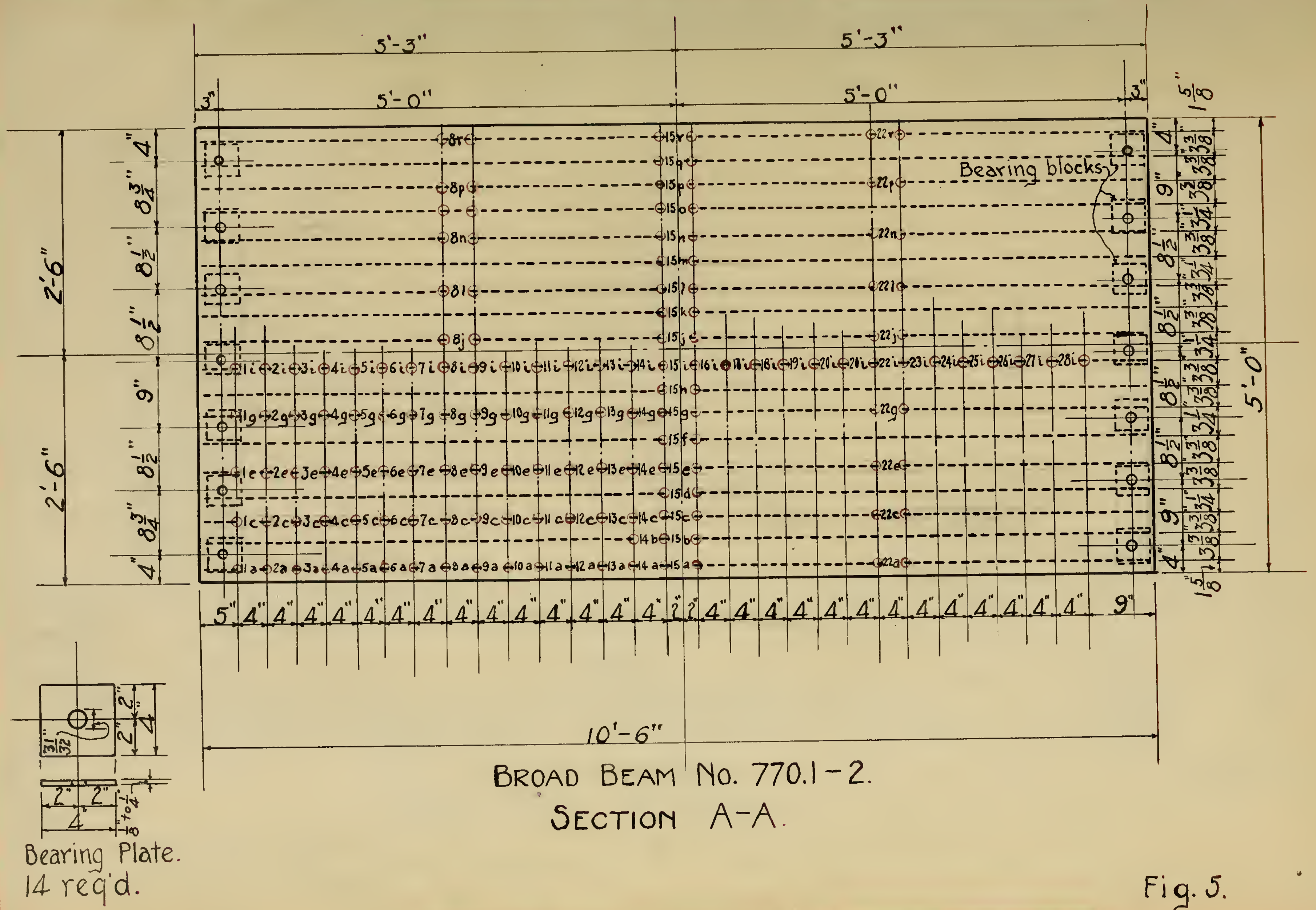
3. Sand.- The sand came from near the Wabash River at Attica, Indiana. It was relatively free from dirt and was graded as shown in Table 3.

4. Cement.- Universal Portland Cement was used. Tests in this cement showed the initial set to occur at 3 hr. 5 min., and final set at 6 hr. 32 min. after mixing. The tests of the briquettes are shown in Table 4. These tests were made according to standard methods by Mr. B. L. Bowling at the Cement Testing Laboratory, University of Illinois.

5. Concrete.- A 1-2-4 mixture was used for all the specimens. Men in the employ of the Experiment Station who were accustomed to the work made the test beams. The material was mixed in a rotary mixer.

6. Steel.- All the steel used was mild steel, and came from the Illinois Steel Company. The yield point, as given by the average of 20 tests, was at a stress of 39,400 lb. per sq. in.

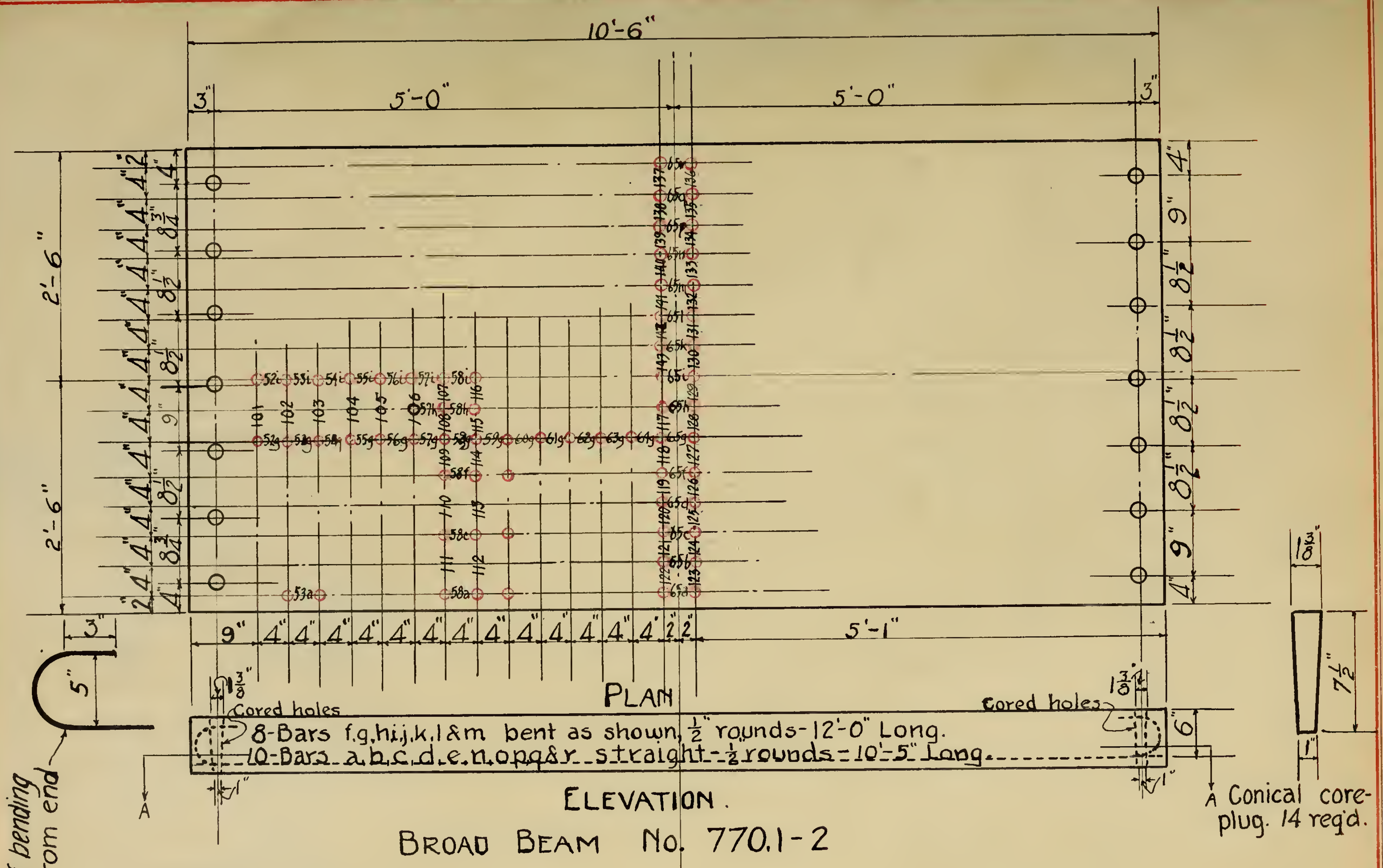
7. Test Piece.- Fig. 5 and 6 show the test piece and the position of the reinforcement in detail. The percentage of



Bearing Plate.
14 req'd.

BROAD BEAM No. 770.1-2.
SECTION A-A.

Fig. 5.



BROAD BEAM No. 770.1-2

○ indicates holes for Strain gage readings.
Numbers are to distinguish gage lines.

Fig. 6.

reinforcement was 1.0.

8. Method of Making the Beams.- The beams were made directly on the concrete floor of the laboratory. A strip of building paper was laid on the forms to prevent the concrete adhering to the floor. The forms used were the ordinary knock down wooden type. Corks were placed next the reinforcing bars where holes for gage readings were to be cored. After the concrete had set these corks were withdrawn, thus exposing the bars.

9. Storage.- The beam was stored in a room, the temperature of which was from 60°F to 70°F. The age of the beam when tested was 103 days.

TABLE 2

MECHANICAL ANALYSIS OF STONE

AVERAGE OF 5 SAMPLES

Size of Square Opening	Separation Size Inches	Per Cent Passing
1 in.	---	100
3/4 in.	---	95.5
1/2 in.	---	66.7
3/8 in.	---	46.3
No. 3	0.28	25.9
No. 5	0.174	8.1
No. 10	0.071	3.4

TABLE 3

MECHANICAL ANALYSIS OF SAND

AVERAGE OF 5 SAMPLES

Screen No.	Per Cent Passing Thru
3	100
5	90.9
10	69.1
12	63.8
16	58.3
18	48.4
30	31.1
40	19.5
50	6.5
74	2.9
150	.9

TABLE 4

BRIQUETTE TEST OF UNIVERSAL PORTLAND CEMENT

Each value is the average of 15 tests.
Loads are in lb. per sq. in.

Neat Cement		1 - 3 Mortar	
7 days	28 days	7 days	28 days
595	207	740	301

V. EXPERIMENTAL DATA AND DISCUSSION

1. Records of data. The original data in this test were recorded on data sheets kept by the Engineering Experiment Station of the University of Illinois and are too voluminous to be copied into this thesis.

2. Description of Graphs.- The results of the tests made are shown by graphs on Fig. 8, 9, 10, 11, 12 and 13. These graphs have unit stresses as ordinates and some dimension of the beam as abscissae. They show the distribution of the stress across the beam at the third points and at the center and also the distribution of stress along the span. The distribution of the reaction across the ends is also shown.

3. Reaction Distribution.- There is some difference in the amount of stress occurring in the various hanger rods, but this difference does not follow any recognizable law until the load of 20,000 lb. is applied. Under the 20,000 lb. load the load deflection curve assumed the shape of a circular segment. The amount of the load transmitted to any section at this load was a function of the distance the section considered was from the center line of the beam where the reaction reached a maximum. The above is deduced from the general shape of the curve. There were large variations from it in one or two bars at each end of the specimen. Returning to a consideration of the reaction distribution under the smaller loads. The distribution of these loads across the reaction when a few inharmonious results are neglected is found to be approximately uniform. This fact, coupled with a consideration of the results of former tests and

the distribution of the stresses in the steel in this test tend to justify the assumption that the distribution of the reaction is nearly uniform for small laods. It seems possible that the load actually was distributed in the peculiar manner indicated by some of the diagrams, but to anaylze the cause of the apparent discrepancies mentioned above is most difficult. Concluding that the reaction was evenly distributed for small loads and that for large loads the distribution of the load to a section would vary inversely as the distance of the section from the center of the load still leaves the necessity of explaining the inconsitencies of the data. One possible explanation of the cause of these discrepancies in the reaction distribution is the probability that the rods did not all have the same stress at first. The nuts could not have had exactly the same stress tightened as they were by touch and the variation may have been large in some cases. Another possible cause would be the presence of particles of grit, etc., between the bearing blocks and the bearing plates in the concrete. Great care was taken to remove all such material, but in spite of this it is possible that a small amount of it was present. The application of the load would cause these particles to be crushed and so leave a loose bearing reducing the stress in one of the hanger rods.

4. Steel Stresses.- The steel stress diagrams (Fig. 8) which show the distribution of stress in the longitudinal bars at the center of the span are peculiar in that the steel having the least stress appears to be in the center of the beam. This is true of both the 15,000 lb. and 20,000 lb. loads. It would appear that this rod j must be slipping in the concrete. This is probable

as it is over this rod that the first longitudinal crack occurred. Outside of this item of apparent slippage, the stresses are uniform all across the beam. These results should be very reliable as in every case the load was removed and reapplied at least once. The second and third applications showed stresses checking approximately with those of the first application. The diagrams shown in Fig. 9 show that the longitudinal distribution of stress was very close to that which would be expected from a theoretical consideration. The curves for the 10,000 lb. load was drawn from the mean of the stresses in five bars. The stress appeared fairly constant between the two loads falling off from them towards the ends of the beam. The diagrams drawn to show the stress distribution under the loads were so uneven that no conclusions could be drawn from them.

5. Concrete Stresses.- The concrete diagrams were too irregular to permit any definite conclusions being drawn from them. There was one more interesting feature in the cracking of the beam. When the first load of 5000 lbs. was applied to the beam, a longitudinal crack occurred along the center line. This crack was directly over one of the reinforcing bars. As the load was increased, the longitudinal crack did not open any wider, but transverse cracks appeared under the loads and these cracks steadily grew larger and more numerous as the load was increased. If slab failure had followed, it would have called for no particular comment. How the slab can continue to bear load and give every indication of standing up until the full strength of the steel is developed, is difficult to explain. It may be that the crack

only goes in as far as the steel and possibly was caused by the bar not being properly embedded in the concrete. The bar began slipping upon the application of the 15,000 lb. load. The crack probably caused this weakness in the bond. The fact that the load is in the center of the beam makes the transverse moment in the center greatest. It then follows that the radius of curvature of the transverse deflection curve is a minimum at the center. This would explain the tendency to crack, but would not explain the reason for the cracks not causing failure.

6. Bearing of Results on Assumption made by Mr. Livingstone.- These results would have little bearing on Mr. Livingstone's first assumption, i.e. that the shear between any two adjacent sections is approximately proportioned to the difference between the steel stresses in the two strips. Mr. Livingstone unquestionably had in mind a beam whose width bore a larger relation to its length than did that of the beam tested. In the beam tested during the course of the tests, the width was half the span and the difference between steel stresses in adjacent strips was approximately zero. This then would lead to the conclusion that there was no shear in a transverse section which is palpably absurd. The second assumption that for the same condition of loading the load carried by a longitudinal strip of a beam is proportional to the stresses developed in the longitudinal strip was, in a way, confirmed by the results obtained. The stresses were evenly distributed in the steel and the reactions evenly distributed along the ends. This can not be taken as meaning much tho, as the real test of the truth of this assumption must come when the load and stress are not evenly distributed, as in the

case of a broader beam. What we would do with the assumption when the steel was slipping is hard to comprehend. It is hardly to be conceived that the load carried by a longitudinal strip decreased because reinforcement under that strip fails in carrying stress. There is no means of ascertaining the status of the Mr. Livingstone's third assumption by the tests made this year. The assumption was that "the load is not distributed over a width greater than twice the span", but no beam of such dimensions was tested.

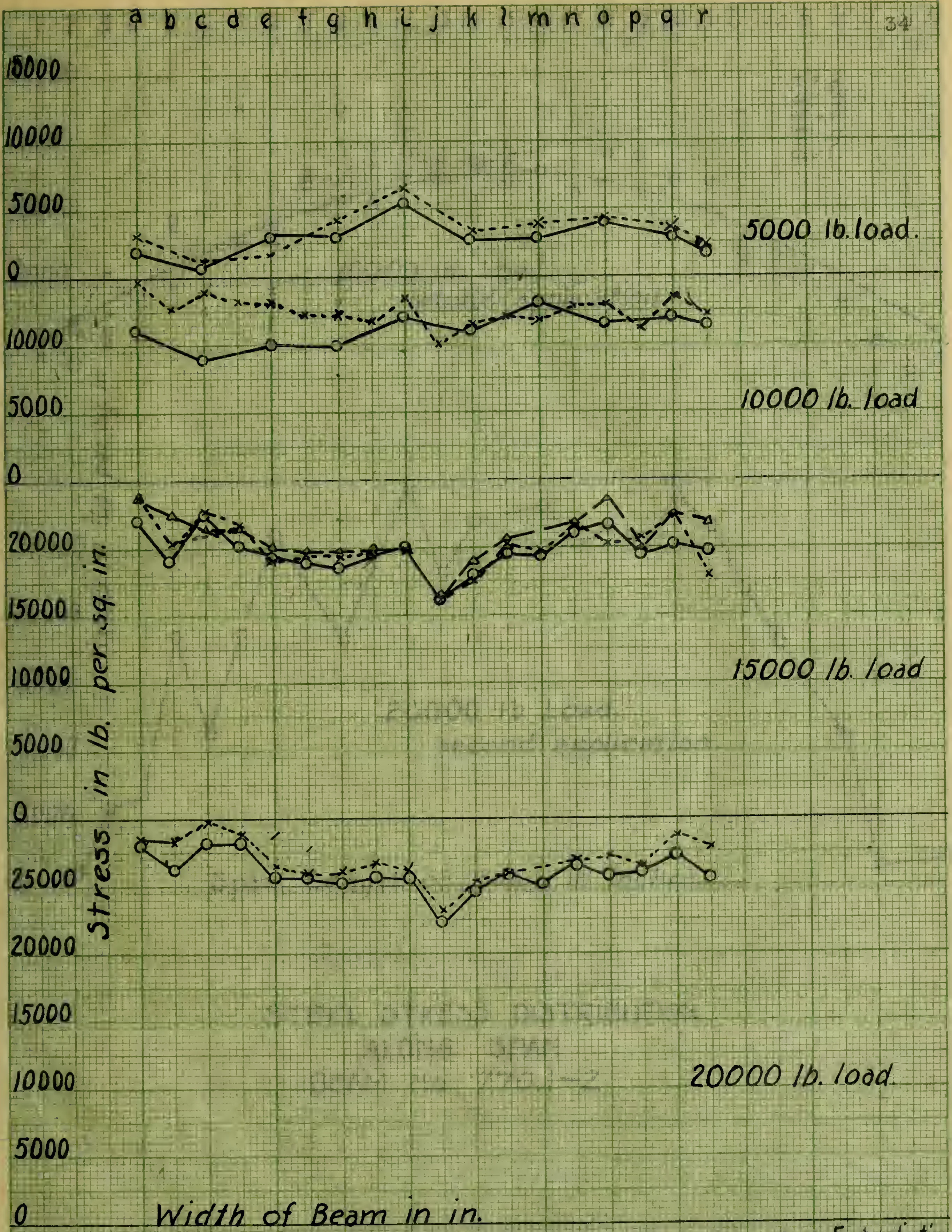
7. Table 5.- Table [illegible] shows that the steel stresses were evenly distributed. This table gives the stresses in the steel as found in each rod at the center of the span, it also shows as a basis of comparison the stress that the steel should have carried assuming the stress evenly distributed across the beam.

TABLE 5

STRESS IN STEEL AT CENTER LINE OF SPAN

Stress in Lbs. Per Sq. In.

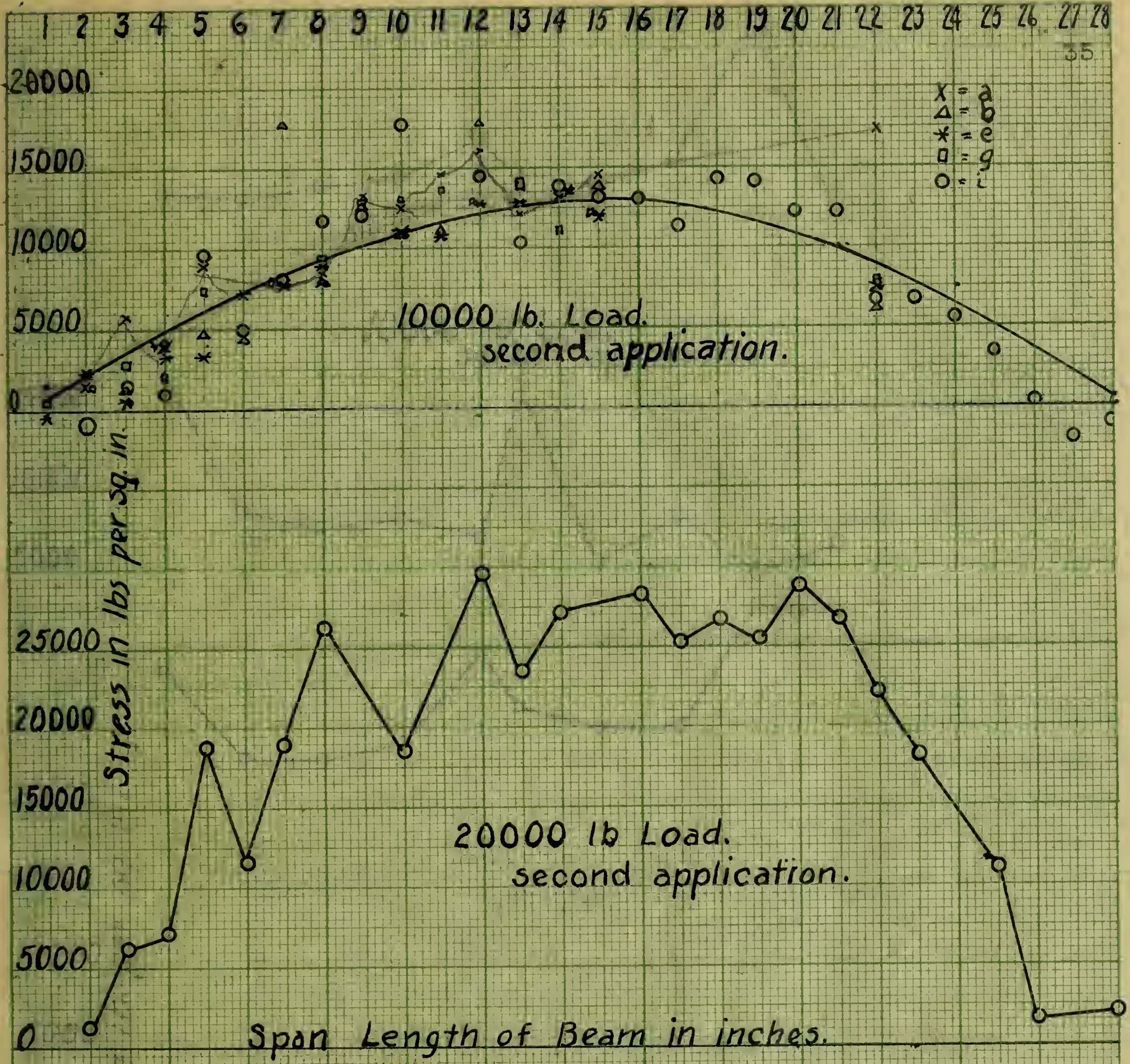
BAR	LOAD			
	5,000	10,000	15,000	20,000
a	3,000	15,300	23,800	28,500
b		12,700	20,400	28,200
c	1,200	14,000	22,600	30,000
d		13,300	21,700	28,800
e	1,500	12,400	19,100	26,400
f		12,200	19,500	25,900
g	4,000	11,900	19,300	26,200
h		11,700	19,600	26,700
i	6,600	13,500	19,800	26,100
j		10,100	16,200	23,200
k	3,000	11,500	17,500	25,200
l		12,400	20,200	25,900
m	3,900	12,700	19,800	12,000
w		13,400	21,500	26,900
o	4,100	13,400	20,300	27,100
p		11,500	20,700	26,400
q	3,700	14,100	22,200	28,700
r	2,400	12,500	17,700	27,700
Average	3,340	12,600	19,800	25,800
Theoretical Value	5,400	10,800	16,400	21,600



STEEL STRESSES AT CENTER OF SPAN.
BEAM No. 770.1-2.

FIG. 8.

NO 345
EUGENE DIETZGEN CO. CHICAGO NEW YORK

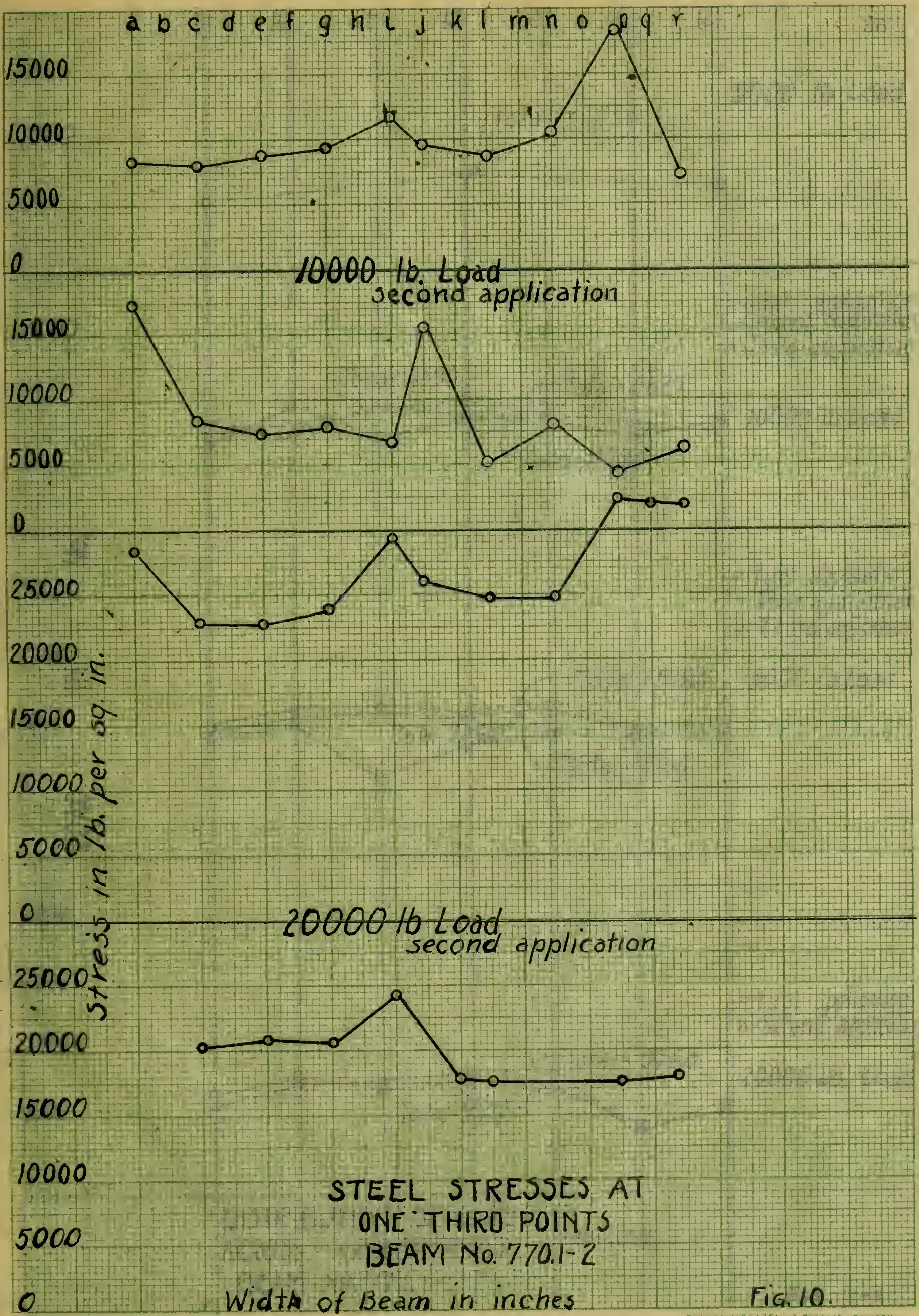


STEEL STRESS DISTRIBUTION
ALONG SPAN
BEAM No 770.1-2

FIG. 9.

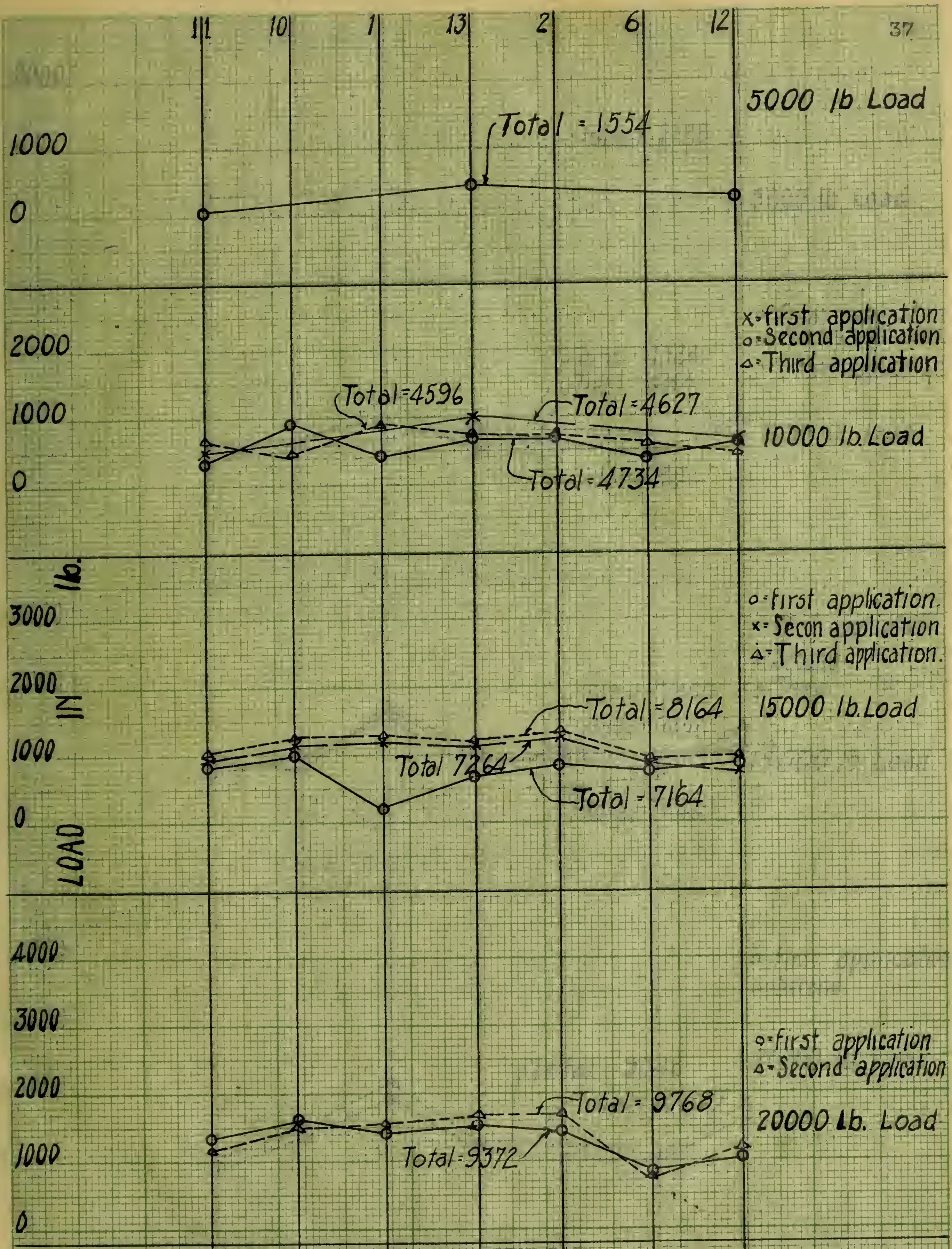
NO 346

EUGENE DIETZGEN CO CHICAGO NEW YORK



STEEL STRESSES AT
ONE THIRD POINTS
BEAM No. 770.1-2

Fig. 10.



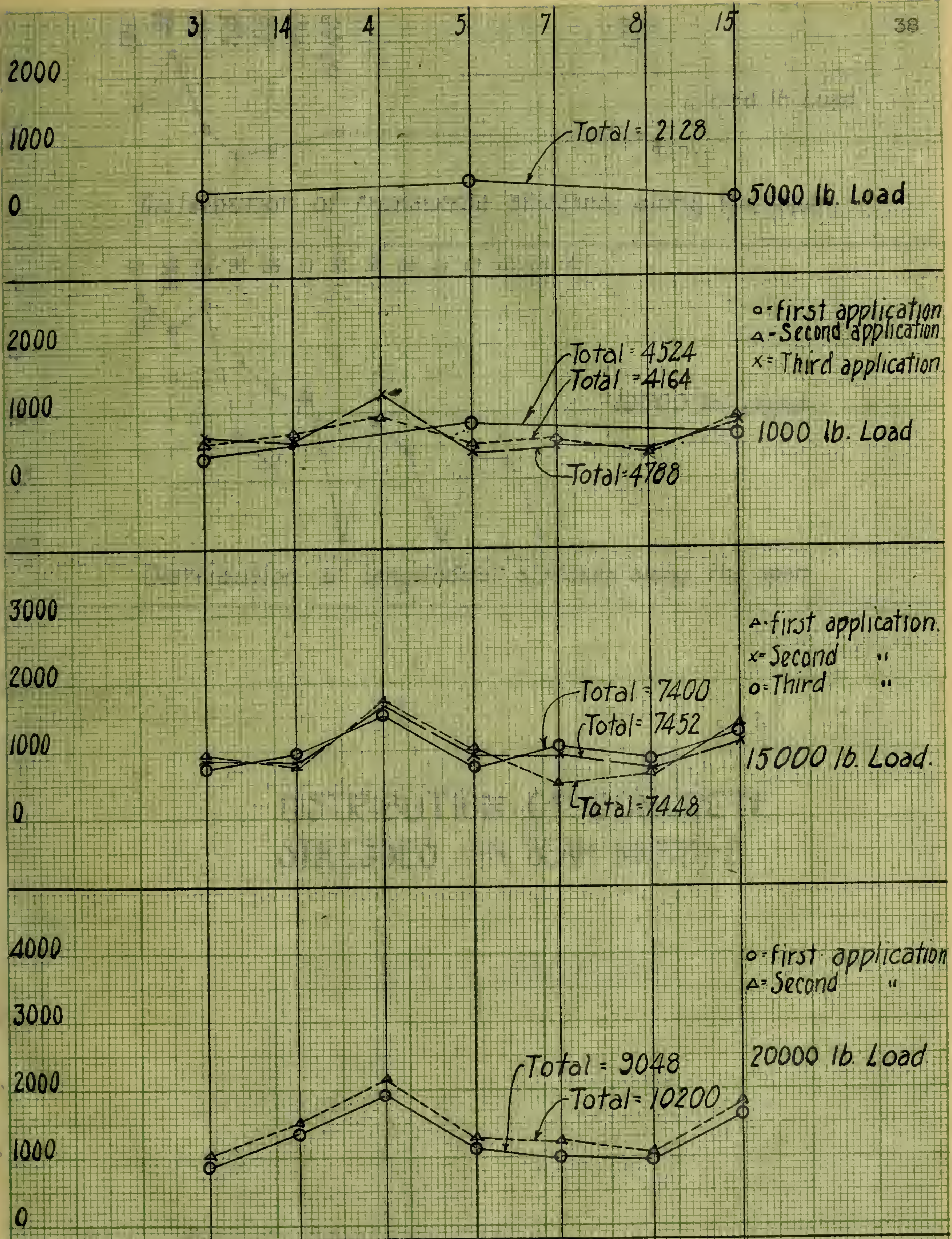
DISTRIBUTION OF LOAD
ACROSS HANGER RODS
BEAM NO. 770.1-2

East End

FIG 11-A.

NO 345

EUGENE DIETZGEN CO CHICAGO-NEW YORK



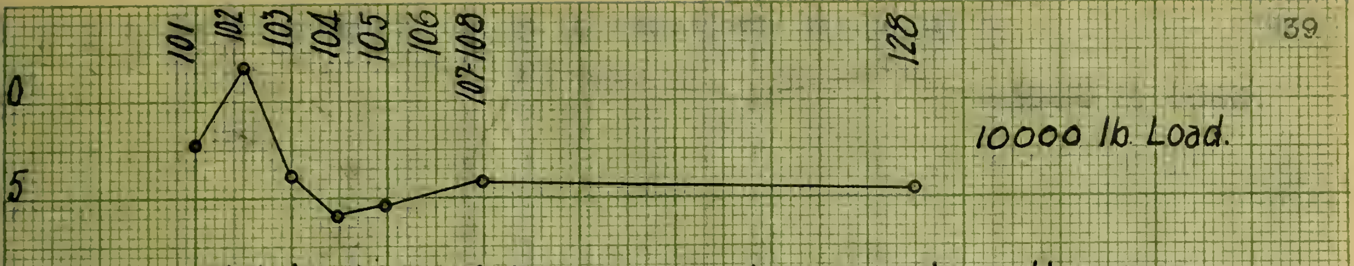
DISTRIBUTION OF LOAD
ACROSS HANGER RODS
BEAM NO. 770.1-2

West End.

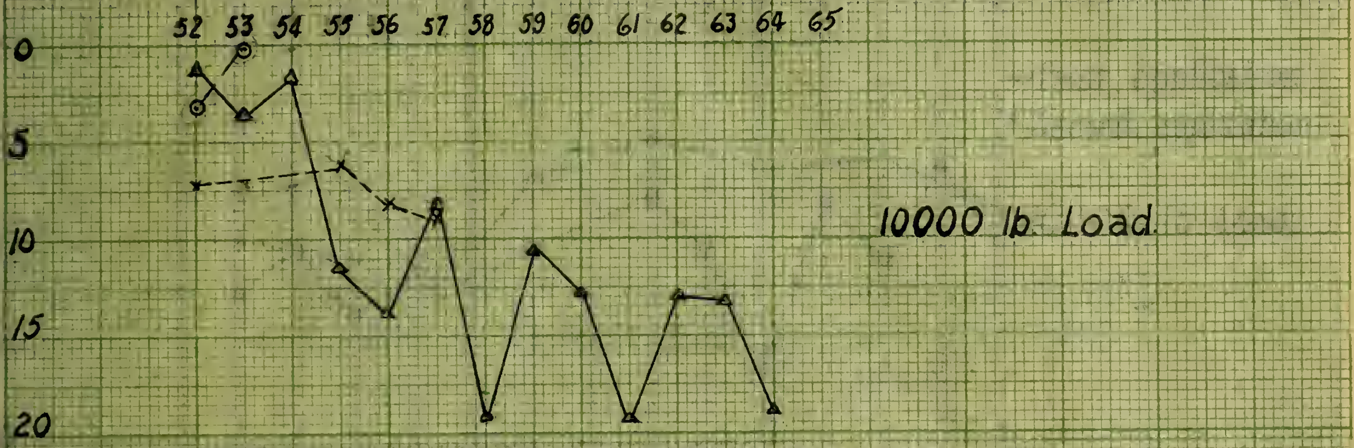
FIG. 11-B.

NO 345

EUGENE DIETZGEN CO CHICAGO-NEW YORK



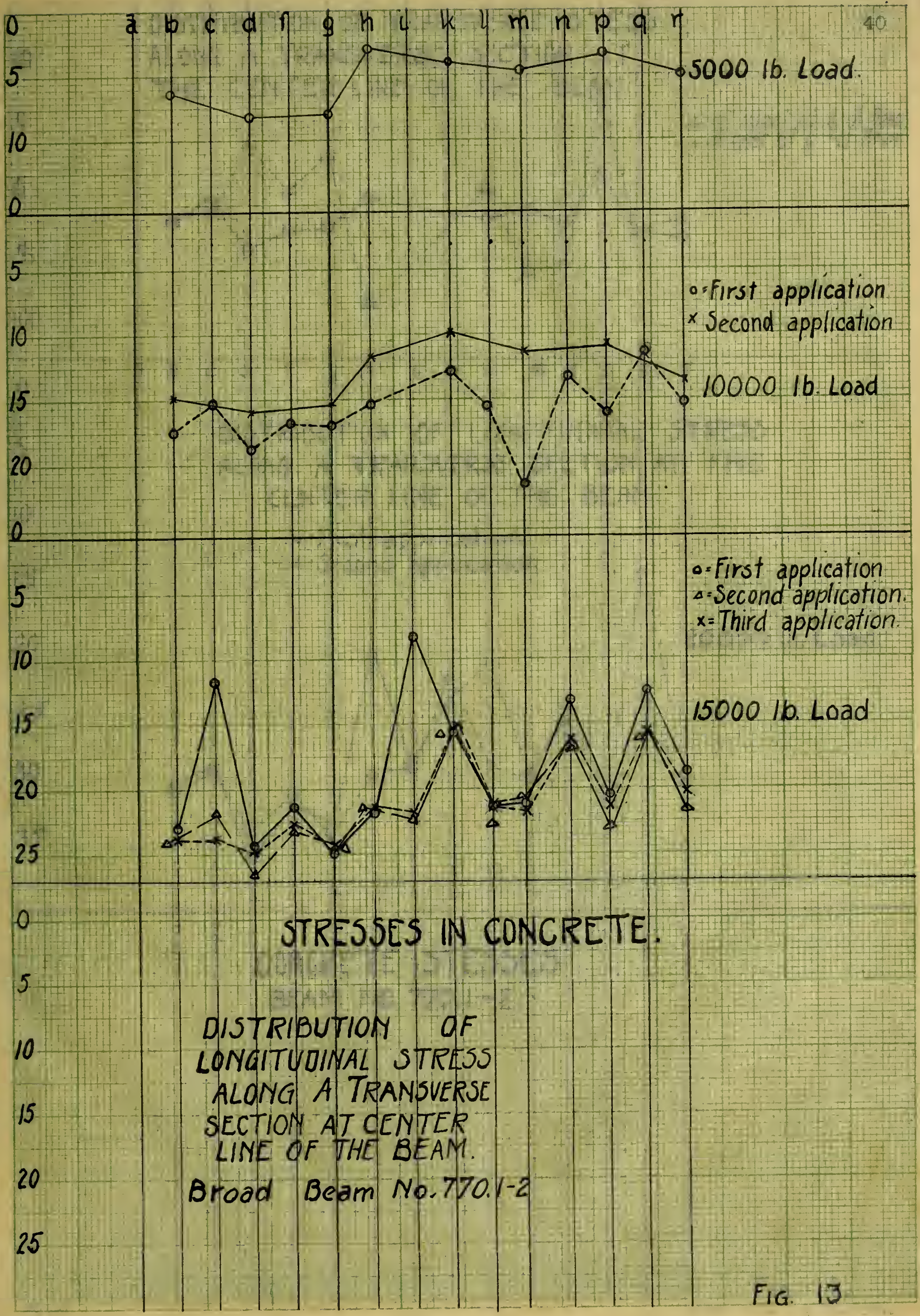
Distribution of transverse stresses along the span.



Distribution of longitudinal stresses along the span

DISTRIBUTION OF CONCRETE STRESSES IN BEAM NO. 770.1-2

FIG. 12



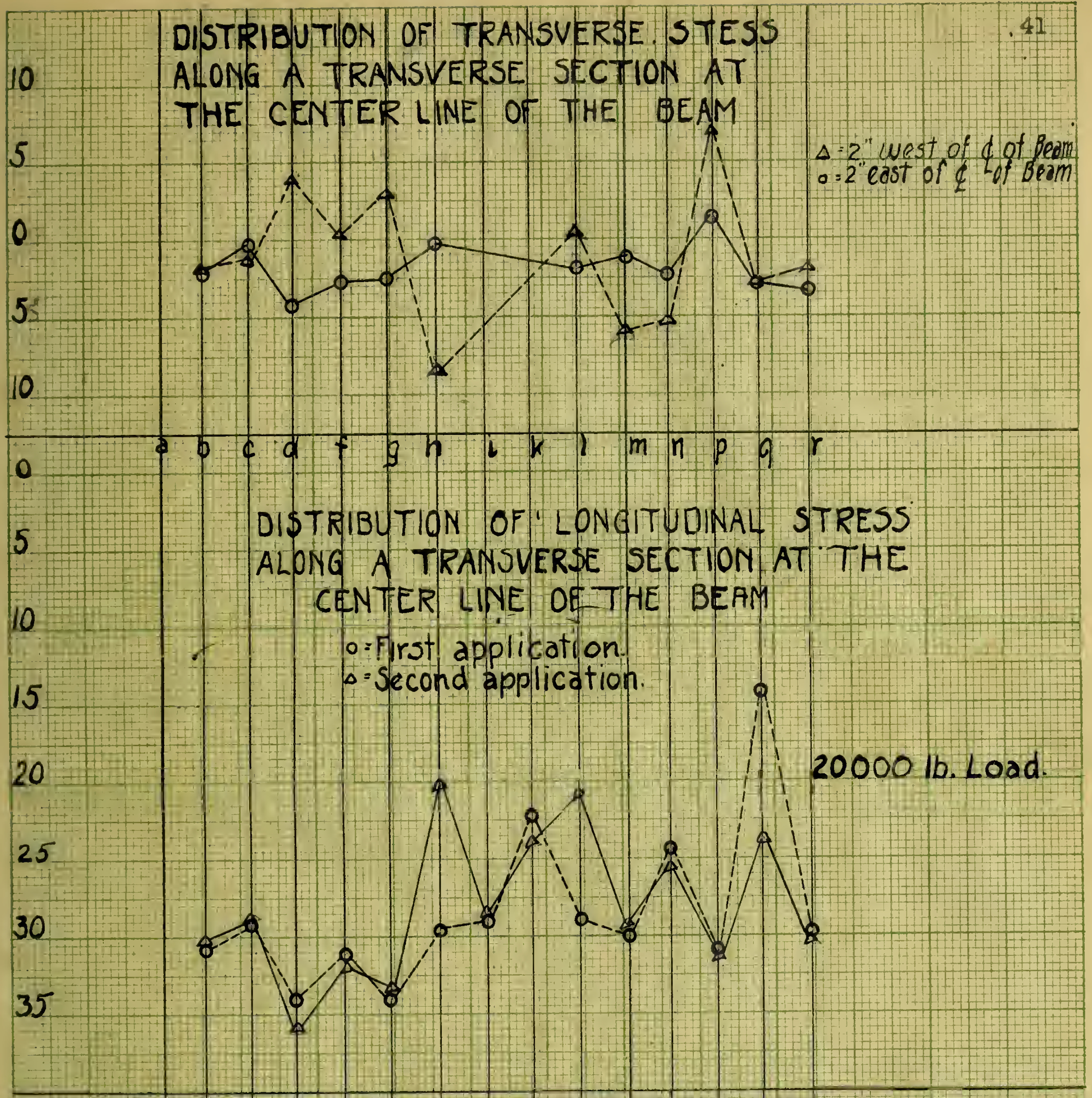
STRESSES IN CONCRETE.

DISTRIBUTION OF
LONGITUDINAL STRESS
ALONG A TRANSVERSE
SECTION AT CENTER
LINE OF THE BEAM.

Broad Beam No. 770.1-2

FIG. 13

DISTRIBUTION OF TRANSVERSE STESS ALONG A TRANSVERSE SECTION AT THE CENTER LINE OF THE BEAM



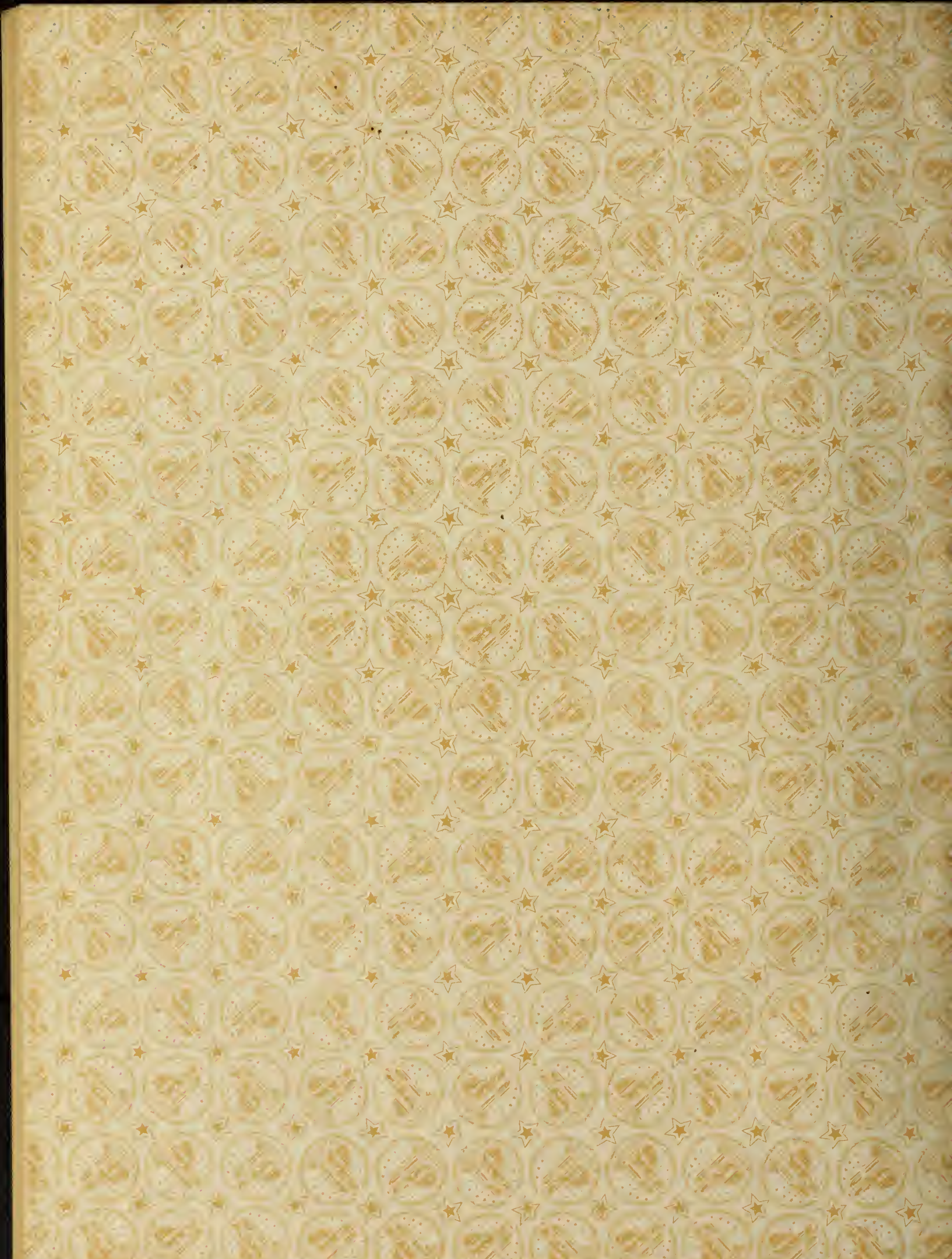
DISTRIBUTION OF LONGITUDINAL STRESS ALONG A TRANSVERSE SECTION AT THE CENTER LINE OF THE BEAM

CONCRETE STESSES
BEAM NO. 770.1-2.

FIG. 14.

NO 345

EUGENE DIETZGEN CO. NEW YORK



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