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Investigation of web buckling in steel beams, 1933

I. Lyse

H. J. Godfrey

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INVESTIGATION OF WEB BUCKLING IN STEEL BEAMS

by Inge Lyse* and H.J. Godfrey**

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1. SYNOPSIS

This report presents the results of an investigation on web failure of steel beams. Tests were made on rolled sections as well as on sections made up from plates by means of electric welding. The depth-thickness ratio of the web of the beams varied considerably, and all the beams gave indication of initial failure due to shear rather than buckling. The computed shearing stress in the web at the initial failure of the beam (the yield point) was found to correspond very well with the yield-point stress in shear as determined on coupons taken from the web. The conclusion is drawn that for depth-thickness ratios of 70 or less, the safety of the beam is determined by shear rather than by buckling.

2. INTRODUCTION

Since the question of web buckling of beams and girders has been a doubtful one and very little experimental data are available, an investigation of this subject was very much needed. This investigation was therefore undertaken to study the reliability of present design formulae for buckling of the web.

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* Research Assistant Professor of Engineering Materials, Lehigh University, Bethlehem, Pennsylvania

** Research Fellow in Civil Engineering, Lehigh University, Bethlehem, Pennsylvania

This study was made as a cooperative investigation by the Fritz Engineering Laboratory of Lehigh University and the Bethlehem Steel Company. The Bethlehem Steel Company designed and furnished all of the beams, and the Fritz Engineering Laboratory carried out the testing program. The great interest taken by the steel industry in this investigation is manifested by the number of representatives present during a part of the testing. The following representatives witnessed the testing on one or more days.

Jonathan Jones, Chief Engineer, McClintic-Marshall Corporation

C.H.Mercer, Consulting Engineer, McClintic-Marshall Corporation

V.E.Ellstrom, Manager, Sales Engineering, Bethlehem Steel Co.

C. E. Blank, Sales Engineer, Bethlehem Steel Co.

E. F. Kenny, Metallurgical Engineer, Bethlehem Steel Co.

N.J.Hittinger, Engineer, Bethlehem Steel Co.

Lee H. Miller, Chief Engineer, American Institute of Steel Construction

F.H.Frankland, Director of Engineering Service, American Institute of Steel Construction

B.G. Hastings, District Engineer, American Institute of Steel Construction

Magnus Gunderson, Chief Structural Engineer, Graham, Anderson, Probst and White

O. E. Hovey, Consulting Engineer, American Bridge Company

R. A. Marble, Structural Engineer, Carnegie Steel Company

F. C. Lucas, Structural Engineer, Illinois Steel Company

Fred Crane, Assistant Metallurgist, Illinois Steel Company

F. T. Llewellyn, United States Steel Corporation

Since the usefulness of a beam is determined by the maximum load it can carry without excessive deflection, the yield point of the beam is consequently the most important factor in the testing. Emphasis was therefore placed upon the securing of the actual yield-point strength instead of the ultimate. The ultimate load has no other significance than that of being a measure of the toughness of the beam after it has lost its usefulness. In the study of the data the yield-point strength of the beam was used as the criterion for its load-carrying capacity.

3. PROGRAM

In order to study the method of testing to be used in the major investigation, a preliminary series of tests was carried out. This preliminary series of tests included the testing of four Bethlehem B12-28 rolled sections. Two of the beams had free ends and the remaining two had steel plates welded to the end sections in order to prevent end twisting. A one-foot sample section of each beam was furnished for the preparation of test coupons, on which tensile and shear tests were made.

The major investigation consisted of two groups of beams designed to secure a definite failure in the web. The first group consisted of five beams, three of which were made up of steel plates welded together and the remaining two were rolled sections reinforced with cover plates welded to the flanges. The second group consisted of five beams, all of which were welded sections. Since the design formulae make the depth-thickness ratio of the web the criterion for the working stresses, beams having high ratios were included in these tests. The highest h/t ratio* for rolled sections was about 55, and for welded sections about 70.

4. PRELIMINARY INVESTIGATION

The information on the beams tested in the preliminary investigation is presented in Table 1. It is noted that these beams had an h/t ratio of about 40. These beams were tested in the 300,000-lb. capacity Olsen screw-power testing machine. The slowest speed of the head of this machine, which is about 0.05-in. per minute, was used in the testing of the beams. The dimensions of the sections were measured with micrometers on all beams, and they were found to be slightly different from the handbook section. The properties based on the actual dimensions are given in Table 1.

The beams were supported on a roller at one end and on a spherical bearing block at the other end. The load was applied to the beam at the quarterpoints of the span, through a roller and a spherical bearing block. At that side of the center line where a roller was used as support, a spherical bearing block was used for the application of the load, and

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* In this report, h represents the clear distance between the flanges, and t the thickness of the web.

vice versa. The loading arrangement for these beams is shown in Fig.1. The two beams having welded plates at the ends were whitewashed before testing, so that the appearance of strain lines could be studied. Lateral deflections of the web were observed at one end of the beams by means of a group of dial gages placed on both sides of the web along a line connecting the loading point and the support. A movement of 0.0001-inch could be read directly by these gages. On the first beam tested (No.1), the gages were supported on a frame attached to the table of the machine. This arrangement, however, did not give satisfactory results due to the relative movement between the beam and the table. For the remaining three beams the frame holding the gages was clamped directly to the flanges of the beams. Vertical deflections were observed only on the beams having fixed ends.

In the first beam having free ends the center of the web was 3/16-inch off center with respect to the top flange. This beam showed scaling of the web at a load of 110,000 lb., which was taken as the yield point of the beam. However, the beam continued to take load until a total load of 120,000 lb. had been applied. At this load one end of the beam twisted sideways in the same manner as that illustrated for beam No.2 in Fig. 2. The eccentricity of the top flange may have contributed to an earlier twisting than would otherwise have occurred.

The arrangement of the gages in the second beam having free ends, is shown in Fig. 3 and the observed lateral deflection is shown in Fig. 4. It is noted that the full depth of the web deflected to one side only. The greatest lateral deflection was near the top flange where, at a load of 110,000 lb. the deflection was about 0.075-inch. Up to a load of 60,000 lb. the increase in lateral deflection was nearly constant for each increment of load. Beyond 60,000 lb. the rate of increase in lateral deflection became greater for each additional increment of load. Strain gage measurements were also taken on this beam during the application of the load. The location of the strain gage points is illustrated in Fig.5. A 2-in. Olsen strain gage was used, and the average results obtained are shown in Fig. 6 and 7. It is noted that at a load of 110,000 lb. none of the gage lines showed strain near the yield-point strain of the material in the web, but there is a tendency for the strains to increase at a greater rate. The first scaling of the beam was observed at a load of 120,000 lb. which was taken as the yield point of the beam. This beam also continued to take load until a maximum of 129,000 lb. was reached. At this load the beam twisted sideways in the same manner as Beam No. 1, and the type of failure is illustrated in Fig. 2.

Table 1 shows that the test coupons gave yield-point stress in shear of web of 23,650 and 22,500 lb. per sq.in. for Beams 1 and 2 respectively. The average yield-point and ultimate strength in tension as determined on the coupons from the flanges of these beams, were 37,700 and 59,000 lb. per sq.in. The yield-point stress in shear, as determined from the coupons, was thus about 64 per cent of the yield-point stress in tension. The computed maximum shearing stresses in the web at the yield point of the beam were 18,550 and 20,250 lb.per sq.in. for the two beams. The maximum fibre stresses in the flange at the yield point were 29,000 and 31,600 lb. per sq. in. These figures show that the yield point of the material had not been reached, either in shear or in tension, at the yield point of the beam. This indicates that the first scaling off does not determine the true yield point of the beam. Furthermore, the failure was due to end twisting instead of web buckling. Steel plates were therefore welded to the ends of the remaining beams in order to prevent end twisting.

For the beams having end plates, vertical deflection measurements were made in addition to the lateral deflections. These beams were whitewashed before the testing so that strain lines could be observed more readily. The loading arrangement is shown in Fig. 8. The lateral deflections for Beams No. 3 and 4 are similar in shape, as is shown in Fig. 9 and 10. It is noted, however, that the lateral deflection of the web of Beam No. 4 became less after a load of 50,000 lb. had been applied, whereas in Beam No. 3 the deflection increased throughout the test. The similarity of the vertical deflection curves is very noticeable, as is shown by Fig. 11 and 12. The yield point for these beams has been taken as the point at which the slope of the tangent to the deflection curve is twice as large as the slope of the preceding straight portion of the curve. The loads at the yield points were 130,000 and 131,000 lb. The beams continued to take load beyond the yield point and until maximum loads of 141,500 and 144,450 lb. were reached. At maximum load, end twisting occurred.

On both beams, at a load of about 110,000 lb., the first strain lines appeared in the form of horizontal lines, on the root of the web near the support. With increased load, more strain lines appeared over the support at both bottom and top of the web, and also below the loading point. Fig.8 shows the appearance of the horizontal lines in Beam No. 3 after it had been loaded beyond its yield point. At loads of 138,500 and 134,550 lb. for Beams No. 3 and 4 respectively, the yielding was so great that it produced a drop in the beam of the testing machine. Further increase in the load produced also vertical strain lines which appeared in the web between the support and the point of loading. All these lines were evidently due to shear. An illustration of the appearance of the strain lines is shown in Fig. 13.

The computed maximum shearing stresses in the web of Beams No. 3 and 4 at the yield point, were 22,400 and 22,600 lb. per sq.in. These agree very closely with the yield-point stresses in shear (22,300 and 24,500 lb. per sq.in.) obtained on the coupons. The maximum fibre stresses in the flanges at the yield point of the beams were 35,100 and 35,400 lb. per sq.in. Since the tensile yield-point stresses of the coupons were 40,800 and 39,550 lb. per sq.in., the yield point of the beam was not caused by the flexural stresses. If the flexural stresses were computed for loads corresponding to the drop of the beam, they would still be less than the yield-point stres of the material. It may therefore be concluded that the yield point of these beams was determined by the yield point in shear of the material in the web. Furthermore, it may be said that the beams having free ends did not develop their full yield-point value of the material since twisting took place at lower loads. The tests demonstrated that no web buckling appeared for h/t ratios of 40. In the major series of tests the h/t ratios were therefore considerably above 40, and it was deemed advisable to restrain the ends of the beams in order to prevent end twisting below the yield point of the beam.

The tension specimens made from the outer edge of the flange usually were of higher strength than the specimens made from the center of the flange. Tension specimens were also made from the web, and were found to be uniform for all four beams. Shearing strengths of the material in the web were obtained on slotted plate specimens tested in a tension machine.

5. MAJOR INVESTIGATION

GROUP A - Group A consisted of five beams, three of which were all-welded sections, and two of which were reinforced rolled sections. The welded beams (WB-1, WB-2, WB-3) were made of 1/4-in. tank plates for the web, and 1-1/2 in. plates for the flanges. The h/t ratios for these three beams were 56.5; 54.9; and 58.9, respectively. All the beams had reinforcing plate stiffeners at the loading points and supports. Steel plates were welded to each end section to prevent end twisting. Beam WB-3 had, furthermore, two angle stiffners on each side of the web in one of the panels between the support and loading point. The make-up and the loading arrangement for these beams are shown in Fig. 14. Both vertical and lateral deflection measurements were taken during the testing of these beams. The location of the gages was similar to that of the beams in the preliminary tests. The beam designated in Table 2 as WB-2 was not whitewashed, and strain gage observations were taken with a 2-in. gage length at points of the web as indicated in Fig. 14. Both WB-1 and WB-3 were whitewashed and had no strain gage observations. The properties of the beams based on their actual dimensions, and the results of the tests are also given in Table 2. Records were taken of the appearance of strain lines in the web.

For Beam WB-1 the first local strain lines appeared at a load of only 50,000 lb. These strain lines on the web were quite small and appeared under the loading point near the junction of the web and top flange. At a load of 125,000 lb. horizontal and vertical strain lines appeared in the web in the panel between the loading point and support. The size and number of strain lines increased with an increase in load. The yield point at 155,000 lb. was determined from the deflection curve in the same manner as that used in the preliminary investigation. The beam continued to take load until a maximum of 218,500 lb. was attained and at this point the load fell off, accompanied with a gradual sagging of the beam.

The lateral deflections for WB-1 are given in Fig.15. It is noted that the center of the web deflected in opposite directions to the deflections at the top and bottom. No excessive lateral deflection took place at the yield point of the beam, indicating that buckling of the web was not the cause of yielding. The vertical deflections of this beam are given in Fig. 16. It is seen that a fairly sharp increase in the rate of deflection took place at a load of 155,000 lb., indicating the yield point of the beam. This load produced a computed maximum shearing stress of 20,400 lb. per sq.in. The coupon gave a yield-point stress in shear of 22,000 lb. per sq.in. which is not greatly different from the computed shearing stress at the yield point of the beam.

The appearance of WB-1 at the yield-point load is shown in Fig. 17. A large number of horizontal strain lines on the web were present at the yield-point load. The appearance of one-half of the beam after the yield-point load had been exceeded is shown in Fig. 18 and 19. It is noted that the horizontal and vertical strain lines are predominating. It is also seen that a number of local strain lines group themselves along the welds. After the beam had reached the maximum load a slight buckling of the web could easily be seen.

The beam WB-2 was tested in a manner similar to WB-1, except that strain gage observations were also taken. The location of the observation points is indicated in Fig. 14. The first strain lines on the web appeared at a load of 66,000 lb. and the yield point as determined from the deflection curve, was found to be at a load of 190,000 lb. The lateral deflections are given in Fig.20, from which it can be seen that the web deflected to one side only. The vertical deflections are shown in Fig. 21. The yield point of the beam was determined in the usual manner from the vertical deflection curves. The beam continued to take load beyond the yield point and reached a maximum load of 255,600 lb., at which time a slight buckle could be seen in the center of that web panel which contained strain gage holes. The computed maximum shearing stress at the yield point of the beam was 24,400 lb. per sq.in. This compared very well with a yield-point stress in shear of 24,500 lb. per sq.in., obtained on the coupons. The strains obtained by means of the 2-in. gage are plotted in Fig. 22. It is noted that certain gage lines showed strains indicating stresses approaching the yield point at a load of 175,000 lb.

Welded Beam WB-3 was tested in the same manner as WB-1. The first local strain lines appeared already at a load of 38,000 lb. After a load of 50,000 lb. had been applied the load was released to 1000 lb. and a set reading was observed. This was also done after every following increment of loading up to the maximum load. A complete set of deflection observations were taken at the release of the load, as well as at the loading increment. The lateral deflections as shown in Fig. 23 indicate that some permanent set had taken place even at low loads. The amount of set increased considerably after the yield point had been reached. The vertical deflections as given in Fig. 24 showed only very small sets at low loads, and that the yield point of the beam was reached at a load of 230,000 lb. The permanent set increased considerably as soon as the beam had been loaded beyond its yield point. The computed maximum shearing stress at the yield point of the beam was 24,700 lb. per sq.in. This compares favorably with a yield-point stress in shear of 26,470 obtained on the coupons. The appearance of the beam at the maximum load of 278,000 lb.

is shown in Fig. 25. While no indication of buckling of the web was present at the yield point of the beam, definite buckling of the web appeared at maximum load.

At the yield point of the beam the average shearing stresses as determined by dividing the reaction by the net area of the web, were 22,300, 26,600 and 26,300 lb.per sq. in. for WB-1, WB-2 and WB-3, respectively. These values also compare very closely with the shearing stress determined from the coupons. Although the h/t ratios of these beams were 56.5, 54.9 and 58.9, no indication of buckling was present up to the yield-point load.

These welded beams proved very satisfactory as no beam showed any indication of distress in the fillet welds.

The beam WB-4 was of a rolled Bethlehem B28-91 section. This beam had plate stiffeners welded to the web at points of support and loading and also had riveted angle stiffeners in one of the end panels. In order to prevent flexural failure, cover-plates extended to within a short distance of the supports. The make-up of the beam is shown in Fig. 14.

Both lateral and vertical deformation measurements were taken. Due to the size of this beam it had to be tested in the 800,000-lb. capacity Riehle testing machine. The beam was whitewashed before the application of the load. Fig. 26 shows the beam in the testing machine before the loading. As the load was applied the whole beam deflected sideways to some extent. When a load of 475,000 lb. was reached, that end of the beam which was supported by the spherical bearing block twisted sideways. The yield point of the beam had not been reached before this twisting took place. Fig. 27 shows that up to a load of 475,000 lb. there was no indication of yielding.

Small strain lines appeared on the stiffeners at a load of 100,000 lb. That these strain lines had no relation to the yield point of the beam is shown by the deflection curves in Fig. 27. Strain lines continued to occur to a slight extent during the increase of the load up to 475,000 lb. The lateral deflections for this beam are shown in Fig. 28. A photograph of the spherical bearing block and the gages for lateral deflections on the end of the beam which twisted, is presented in Fig. 29.

Since the twisting of one end of the beam evidently was due to the loading arrangement, it was deemed advisable to retest this beam under more favorable conditions. Consequently lateral restrains for the ends of the beam, in form of channels bolted to the supporting beam, was used in the retesting. The friction between the upper flange of the beam and the channels was kept at a minimum by the use of rollers. A roller was also substituted for the bearing block used as one of the supports in the previous test. The vertical and lateral deflections obtained during the retesting are shown in Fig. 30 and 31. From Fig. 30 the yield point of the beam was determined at the load of 560,000 lb. Set readings were observed at a load of 25,000 lb. and it is noted that in this test the lateral set of the web was very small in comparison to the lateral deflection of the web. This may be due either to the better restraining of the ends of the beam by the use of channels, or to the substitution of the roller for the spherical bearing block. This small set of the web indicates that lateral buckling of the web had not taken place. The beam continued to take load until a maximum load of 597,600 lb. was reached. The beam started to rotate at a load of only 325,000 lb. This rotation continued with the increase in load until at the maximum load the beam twisted so much that the whole loading rig was out of position and the beam did not take any greater load. At the maximum load the web did not scale except on the stiffeners and along the junction between the web and the flange.

The computed maximum shearing stress in the web at the yield-point load was 24,200 lb. per sq.in. in the section between the support and the cover-plates on the flanges, and 21,130 lb. per sq.in. in the web within the section having cover-plates. The yield-point stress in shear obtained from tests of coupons was 22,000 lb. per sq.in. For this beam also, the relation between the yield point in shear obtained from the material and the shearing stress computed for the web at the yield point of the beam was very good.

The beam WB-5 was a Bethlehem B22-62 reinforced with plate stiffeners and cover-plates, as indicated in Fig. 14. The beam which was whitewashed was also tested in the 800,000-lb. testing machine, and vertical and lateral deflection measurements were taken similarly to the previous tests. This beam, like all previous beams, had plates welded to the ends so as to prevent end twisting. The first appearance of scaling on the web was discovered at a load of 50,000 lb. As the load on the beam was increased, the scaling increased, especially at the stiffeners. The strain lines on the web were primarily horizontal, indicating shearing stress. Since the scaling off had no relation to the yield point of the beam, it was probably caused by high internal strains in the material. The vertical and lateral deflections and sets are given in Fig. 32 and 33. The lateral deflections increased until a load of 275,000 lb. was reached. For greater loads there was a tendency of the web to return to its original position. The yield point of the beam, as determined from the curves in Fig. 32, was 420,000 lb. The beam continued to take load until a maximum of 450,000 lb. was reached. At this load the beam continued to deflect vertically without any increase in the load. The beam showed no indication of web buckling, even at maximum load. At this load a large portion of the whitewash had flaked off in the portions between the support and the loading point, as can be seen quite clearly in Fig. 34.

The beam had rollers at both supports in order to secure more favorable conditions.

The computed maximum shearing stress at the yield point of the beam was 25,700 lb. per sq.in. at the section of the web between support and cover plate, and 23,250 lb. per sq.in. in the section having cover plates. These values are both somewhat lower than the yield point in shear of the coupons. The yield point in shear of the coupons was 29,200 lb. per sq.in. which is considerably above any of the values obtained from coupons of the other beams. All other beams showed yield point in shear of the coupons between 22,000 and 26,470 lb. per sq.in. It seems, therefore, that the value of 29,200 lb. per sq.in. may be somewhat in error. For Beams WB-1 to WB-4 the ratio between the yield point in shear and that in tension varied between 0.506 and 0.534. However, the ratio obtained for WB-5 was 0.569, which is considerably above the other ratios. Granting that the yield-point stress of the coupons was in error, Beam WB-5 also showed a fair agreement between the computed maximum shearing stress at yield point of the beam and yield point in shear of the material.

The computed maximum fibre stress in the flanges at the yield point of these beams was only 21,700 lb. per sq.in. as seen from Table 2. This is so far below the yield point of the material which had for a minimum 43,500 lb. per sq.in., as to make it quite evident that the beams were not damaged in flexure.

GROUP B - Group B consisted of five welded beams, (WB-6 - WB-10) all of which were whitewashed before being tested in the 300,000-lb. Olsen testing machine. The load was applied to the center of the beam through a spherical bearing block, and rollers were used at both supports. In order to prevent end twisting, plates were welded to each end section of the beams. Plate stiffeners were welded on the web of the beams at the supports and loading points so that local failure would not occur. The make-up and loading arrangement of these beams are shown in Fig. 35. Vertical and lateral deflections were observed, and strain gage readings were taken on various parts of the beams.

The lateral deflections of the web were observed by means of a group of 0.0001-inch dial gages placed along the web between the loading point and support. The vertical deflections were also measured by dial gages. A 10-inch Whittemore strain gage was used in measuring the deformations in the web and the flange. The load was applied in various increments, and observations and measurements were taken after each increment of load. The properties of these beams based on actual dimensions are given in Table III. The h/t ratios ranged from 49.4 to 70.0. The mild steel plates used for the web of these beams were found to be very ductile.

During the testing of these beams a decided drop of the beam of the testing machine was noted. The load at this point was considered the yield-point load of the beam.

Beam WB-6 had a web ratio of 70. The lateral gages were placed along the web in one of the panels, and strain gage readings were taken in the center of the other panel. The strain gage lines were at 45 degrees with the horizontal so that both compressive and tensile strains were measured. Strain gage readings were taken in the center of the bottom flange in order to determine the maximum flexural stresses developed. The position of the gages and strain gage holes are shown in Fig. 35.

The first flaking of the whitewash occurred at a load of 24,000 lb. The scaling was in the form of vertical lines, and appeared in the top corner of the web below the loading point. As the load increased, approximately vertical and horizontal strain lines extended across the web. At a load of 50,000 lb. strain lines appeared in the web near one of the supports. A gradual increase in the strain lines followed an increase in load, and the condition of the beam at a load of 120,000 lb. can be seen in Fig. 36. The general formation of strain lines as they appeared on the beams in this group of tests is shown in Fig. 37.

The net deflection of Beam WB-6 is shown in Fig.38 from which it is noted that an increase in the rate of deflection occurred at a load of 140,000 lb. At a load of 150,000 lb. the rate of deflection increased very sharply and a decided drop of the beam was observed at a load of 157,600 lb. No buckling was observed at this load, and the lateral deflections of the web are shown in Fig. 39. The maximum deflection was only about 0.008 in. at a load of 150,000 lb. and decreased as the load increased beyond this value. The strain curve for the flange, as shown in Fig. 40, indicates that the stress in the flange was very low at the yield point of the beam. The tensile strains in the web, as shown in Fig. 40, increased regularly until a load of 120,000 lb. was reached. At 140,000 lb. there was a decided increase in the tension strains. The compressive strains are shown in Fig. 41, in which the strains on both sides of the web have been plotted separately in order to bring out the buckling behavior. It is found that the strains on each side coincide almost exactly throughout the test, indicating that no buckling took place within the loads for which observations were taken. Had buckling occurred, the strains on one side of the web would have increased much faster than the strains on the opposite side, due to the bending effect. Fig. 41 is therefore a good illustration of the fact that no buckling took place at the yield point of the beam. The beam continued to take load until a maximum of 192,600 lb. was attained. With further motion of the head of the testing machine the web in one of the panels buckled considerably, as shown in Fig. 42. Table III shows a close agreement between the shearing stress in the web at the drop of the beam and the yield-point stress of the material, indicating that the yielding of the beam was due to the yielding of the material in shear. The computed maximum shearing stress at the yield point of the beam was 16,800 lb. per sq.in. This value agrees with the yield-point stress in shear of 17,450 lb. per sq.in. as determined by the test coupons. It is noted that the maximum shearing stress is less than the total shear divided by the net area of the web. This is due to the unusually thick flanges on these beams which tend to increase the moment of inertia of the beam relatively more than the statical moment. In Table IV the shearing stresses in the web are computed at the yield-point loads which were determined from the strain and deflection curves. These values are very compatible and still they do not agree very well with the yield point in shear as determined from the slotted plate coupons. However, it has been found that the values of the yield point in shear as determined from plate coupons, are about the same as those determined from solid torsion coupons. Seely and Putnam in Bulletin No.115 of the University of Illinois, say that the correct yield point in shear is about 85 per cent of the value obtained on solid coupons. If the values for the yield point in shear as found on the plate coupons, are reduced to 85 per cent of their original values, they will correspond fairly well with the shearing stress computed at the yield-point loads determined from strain and deflection curves.

The beam WB-7, of which the loading arrangement and make-up are shown in Fig. 35, had a web-ratio of 60.6. The first strain lines appeared on the web near the loading point at a load of 20,000 lb. Nearly vertical and horizontal strain lines continued to appear in the usual manner with an increase in the load. The deflection curve for WB-7 is shown in Fig.43. At a load of approximately 130,000 lb. an increase in the rate of deflection was noted and at a load of 148,200 lb. a decided drop of the beam took place. The lateral deflection, as shown in Fig. 44, reached a maximum of about 0.025-inch at a load of 150,000 lb. The tensile strains in the flange, and also in the web, are presented in Fig. 45. A decided increase in the tension strains was noted at a load of 120,000 lb., while the flange strains show no indication of yielding at the yield point of the beam. The compressive strains are shown in Fig. 46, in which the strains on both sides of the web are plotted. These strains were almost the same on both sides of the web throughout the test, indicating that no buckling occurred. A sharp increase in the compressive strains is noted at a load of 120,000 lb., which agrees with the behavior of the tensile strains. The beam continued to take load until one of the web panels began to buckle at a maximum load of 190,100 lb. A photograph of the beam after the maximum load had been reached is shown in Fig. 47. The computed stresses are given in Tables III and IV. The maximum shearing stress in the web at the drop of the beam was 17,800 lb. per sq.in. The yield point in shear as determined from the coupons, was 18,600 lb. per sq.in. which is in agreement with the computed stress at the drop of the beam. The shearing stresses computed at yield-point loads determined from the strain and deflection curves are somewhat lower as in the case of Beam WB-6.

The beam WB-8 was a companion of Beam WB-7 and had a web ratio of 59.7. This beam was the only one in Group B that had web material from a different steel plate. Instead of lateral gages being placed along one of the web panels, strain gage readings were taken in both panels to determine if the load was evenly distributed on both sides of the loading point. The loading arrangement and make-up are shown in Fig. 35. This beam behaved similarly to WB-7 throughout the loading, and the strain curves indicate a fairly even distribution of the load. An increase in the rate of deflection occurred at a load of 130,000 lb. as shown in Fig. 48, and a pronounced drop of the beam took place at a load of 143,100 lb. The tensile strains for both panels, as given in Fig.49, show an increase in strain at a load of 120,000 lb. The compressive strains for both sides of the web are shown in Fig. 50 and 51. These curves also indicate that buckling did not take place within the range of loading for which observations were made. The beam continued to take load after the yield point was reached, until a buckle in both panels began at a maximum load of 199,500 lb. The greatest buckle occurred in the east panel. The condition of the beam at the maximum load is shown in Fig. 52. The results as given in Table III, show maximum shearing stress in the web of 16,250 lb. per sq. in. at the drop of the beam. This value is less than that obtained for WB-7 and also less than the yield-point stress in shear as found by the test coupons. However, the shearing values found by the test coupons are somewhat doubtful. It is noted that the yield-point stress in tension for the specimens of this beam was 29,680 lb. per sq.in. as compared with 33,700 lb. per sq.in. for WB-7. Since Beam WB-7 had material with a yield-point stress in shear of 18,600 lb. per sq.in., the 19,920 lb. per sq.in. for WB-8 seems to be too high. Since the tensile yield point of the material for WB-8 was lower than the values for all the other beams in this group, and the yield-point stress in shear is the highest value, it seems that the high value of the shearing yield point of the coupon is in error. The shearing stresses computed in Table IV are also found to be relatively lower than the shearing stresses determined from the test coupons.

The beam WB-9, the loading arrangement and make-up of which are shown in Fig. 35, had a web ratio of 50. Strain lines were visible on the web near the loading point at a load of 20,000 lb. Both vertical and horizontal strain lines appeared with an increase in load in the same manner as described for the other beams in this group. The deflection curves as given in Fig.53, show an increase in the rate of deflection at a load of 100,000 lb. and a decided drop of the beam took place at a load of 118,900 lb. Fig. 54 gives a maximum lateral deflection of the web of 0.0045-inch at a load of 130,000 lb. which indicates that buckling had not occurred up to that point. The average compressive and tensile strains in the web and the tensile strains in the flange are given in Fig. 55. A decided increase in the web strains occurred at a load of 90,000 lb. The compression strains for both sides of the web are plotted in Fig. 56 and indicate that the strains at loads above the yield point of the beam are slightly greater on the north side than on the south side. Below the yield point, however, the strains on the north side were the smaller of the two, and this indicates that the small difference in strains was due to experimental errors. The maximum load carried was 184,000 lb. Both web panels buckled as shown in Fig. 57. The results which are given in Table III, show a good agreement between the shearing stresses in the web at the drop of the beam and the yield-point stress of the material. The maximum shearing stress was 17,250 lb. per sq.in., and the plate coupons gave 18,480 lb. per sq.in. The shearing stresses computed at the yield point of the beam as determined by strain and deflection curves, show the same relative results as in the other beams.

The beam WB-10 was a companion to Beam WB-9. The loading arrangement and make-up are shown in Fig. 35. The first strain lines appeared on the web at a load of 20,000 lb. near the loading point. Vertical and horizontal strain lines appeared in the usual manner as the load was increased. The deflection curve, Fig. 58, shows an increase in the rate of deflection at a load of about 120,000 lb. and a drop of the beam occurred at a load of 121,800 lb. The average tensile strains in the web and in the flange are shown in Fig. 59, from which it is seen that an increase in the tensile strains occurred at a load of 100,000 lb. The compressive strains for both sides of the web are plotted individually for the two web panels in Fig. 60 and 61. The curves show a sharp break at a load of 100,000 lb. The agreement of the strains on both sides of the web indicates that buckling did not take place. The beam continued to take load until a maximum of 188,000 lb. had been reached. The appearance of the beam at the completion of the test is shown in Fig. 62. The results which are given in Table III show a very good agreement between this beam and its companion, WB-9. The computed shearing stresses at the drop of the beam check the yield-point stress in shear as found by the test coupons.

In all these beams the maximum stresses in the flanges at the drop of the beam, were less than one-half the yield-point stress in tension of the material.

6. EFFECT OF h/t RATIO

In order to study the relation between the h/t ratio and the shearing stresses developed, Fig. 63 was prepared. The beams in Group B were the only ones which lent themselves to such a study. It is noted that the shearing stress at the drop of the beam of the testing machine was very nearly the same for all beams, regardless of their slenderness ratio. The shearing stress at the drop of the beam was about 17,000 lb. per sq.in. This is somewhat less than the average shearing stress at the yield point of the test coupons which was 18,600 lb. per sq.in., but it was in excess of the 85 percent of the coupon stress. This indicates that up to an h/t ratio of 70 the yield-point stress in shear of the web material determines the useful load-carrying capacity of the beam. For h/t ratios of 70 or less, there seems to be no reason for designing beams on the basis of web buckling.

The shearing stress at maximum load is also shown in Fig. 63. The stress developed in the web is greater for beams having a low slenderness ratio than for beams having a high ratio. The maximum shearing stress decreased quite reqularly with the increase in the h/t ratios, indicating that the toughness of the beam decreased with the increase in this ratio.

If buckling of the web had occurred in any of the beams, the maximum load would have been equal to, or less than the yield-point load.

7. SUMMARY

Although the number of beams tested in this investigation was too small on which to base final conclusions, the results obtained indicated that:

1. The beams which had free ends did not develop the full yield-point strength of the material, either in shear or in tension, due to failure in end twisting.

2. At the yield point of all the beams which had plates welded to the ends, the computed maximum shearing stress in the web corresponded very well with the yield point in shear of the material.

3. No beam showed any evidence of buckling at, or below its yield point, indicating that with h/t ratios up to 70 there is no danger of web buckling.

4. The yield point of the beam, rather than the maximum load, should be used as a criterion for the factor of safety. In general, the average shearing stress in the web should be based on net area, that is, h.t, rather than on gross area, D.t.

5. The first appearance of strain lines had no relation to the yield point of the beam.

6. The yield point of the beam was not affected by the h/t ratio of the web. The maximum load, however, decreased with an increase in the h/t ratio.

7. The yield-point strèss in shear was the important factor for all the beams included in this investigation. Further investigation of the shear properties of the materials is therefore of utmost importance.

TABLE 1 - PRELIMINARY INVESTIGATION

Date of Test	Size of Beam	Net Web Area ht	Web Ratio h/t	Moment of Inertia I	Sect. Modulus S	Stresses at Y. P. of Beam: Reaction V	V/ht	$\frac{Vay}{It}$	$f = \frac{M}{S}$	Web Specimens: Shear	Web Specimens: Tension	Flange Speci.: Tension	Ult. Reaction
1932	No.	in.^{2}		in.^{4}	in.^{3}	lb.	lb. per sq.in.			lb. per sq.in.			lb.
Apr. 1	B-12 28 1	3.06	$\frac{11.1}{.276}$ = 40.2	205.5	34.2	55,000	17,950	18,550	29,000	23,650*		37,340* 58,810°	60,000
Apr. 5	B-12 28 2	3.06	$\frac{11.1}{.276}$ = 40.2	205.5	34.2	60,000	19,600	20,250	31,600	22,500* 50,750°		37,970* 59,260°	64,500
May 27	B-12 28 3	3.02	$\frac{11.11}{.272}$ = 40.8	200.0	33.33	65,000	21,550	22,400	35,100	22,300* 50,200°	40,800* 59,370°	39,950* 58,150°	70,750
July 1	B-12 28 4	3.02	$\frac{11.11}{.272}$ = 40.8	200.0	33.33	65,600	21,700	22,600	35,400	24,500* 50,600°	39,550* 57,800°		72,225

* Yield-point stress
° Ultimate stress

TABLE 2 - MAJOR INVESTIGATION

Date of Test	Mark on Beam	Net Web Area ht	Web Ratio h/t	Moment of Inertia I	Sect. Modulus S	Stresses at Y.P. of Beam: Reaction V	V/ht	$\frac{Vay}{It}$	$f = \frac{M}{S}$	Web Specimens: Shear	Tension	Maximum Reaction
1932		in.^{2}		in.^{4}	in.^{3}	lb.	lb. per sq.in.			lb. per sq.in.		lb.
June 15	WB-1	3.47	56.5	1938	226	77,500	22,300	20,400	14,700	22,000* 53,500°	43,250* 62,000°	109,250
May 20	WB-2	3.57	54.9	1955	228	95,000	26,600	24,400	17,900	24,500* 51,300°	47,810* 60,900°	127,800
June 16	WB-3	4.36	58.9	2419	254	115,000	26,300	24,700	21,700	26,470* 51,600°	49,600* 61,425°	139,000
June 30	WB-4	12.41	54.7	3342ǂ 12,850ƒ	240ǂ 832ƒ	280,000	22,500	24,200ǂ 21,130ƒ	20,200	22,000* 53,400°	43,500* 61,600°	298,800
June 16	WB-5	8.15	52.2	1560ǂ 6922ƒ	141ǂ 550ƒ	210,000	25,800	25,700ǂ 23,250ƒ	21,150	29,200* 55,630°	51,300* 66,650°	225,000

* Yield-point stress
° Ultimate stress

ǂ At section having no cover plates
ƒ At section having cover plates

TABLE 3 - MAJOR INVESTIGATION

Date of Test 1932	Mark on Beam	Net Web Area ht in.^{2}	Web Ratio h/t	Moment of Inertia I in.^{4}	Sect. Modulus S in.^{3}	Yield Point at Drop of Beam: Reaction V lb.	Stresses-lb.per sq.in.: $s = \frac{V}{ht}$	$s = \frac{Vay}{It}$	$f = \frac{M}{S}$	Web Specimens lb.per sq.in.: Shear	Tension	Maximum Reaction lb.
Nov. 30	WB- 6	4.40	70.0	2866	278.5	78,800	17,900	16,800	14,100	17,450* 46,900°	33,080* 46,800°	96,300
Dec. 1	WB- 7	3.88	60.6	2216	241.8	74,100	19,100	17,800	13,800	18,600* 45,850°	33,700* 48,400°	95,050
Dec. 2	WB- 8	4.10	59.7	2326	249.2	71,550	17,500	16,250	12,930	19,920* 46,250°	29,680* 46,080°	99,750
Dec. 1	WB- 9	3.12	50.0	1523	196.5	59,450	19,050	17,250	12,100	18,480* 44,800°	30,280* 46,980°	92,000
Dec. 2	WB-10	3.14	49.4	1520	196.5	60,900	19,380	17,580	12,400	18,400* 46,250°	30,270* 47,150°	94,000

* Yield-Point Stress ° Ultimate Stress

TABLE 4 - MAJOR INVESTIGATION

Mark on Beam	Yield Point of Beam by Strain Curves			Yield Point of Beam by Deflection Curves		
	Reaction V lb.	Stress-lb. per sq. in.		Reaction V lb.	Stress-lb. per sq. in.	
		$s = \frac{V}{ht}$	$s = \frac{Vay}{It}$		$s = \frac{V}{ht}$	$s = \frac{Vay}{It}$
WB- 6	70,000	15,900	14,950	70,000	15,900	14,950
WB- 7	60,000	15,470	14,400	65,000	16,750	15,600
WB- 8	60,000	14,640	13,600	65,000	15,850	14,730
WB- 9	45,000	14,400	13,050	50,000	16,020	14,500
WB-10	50,000	15,900	14,430	50,000	15,900	14,430

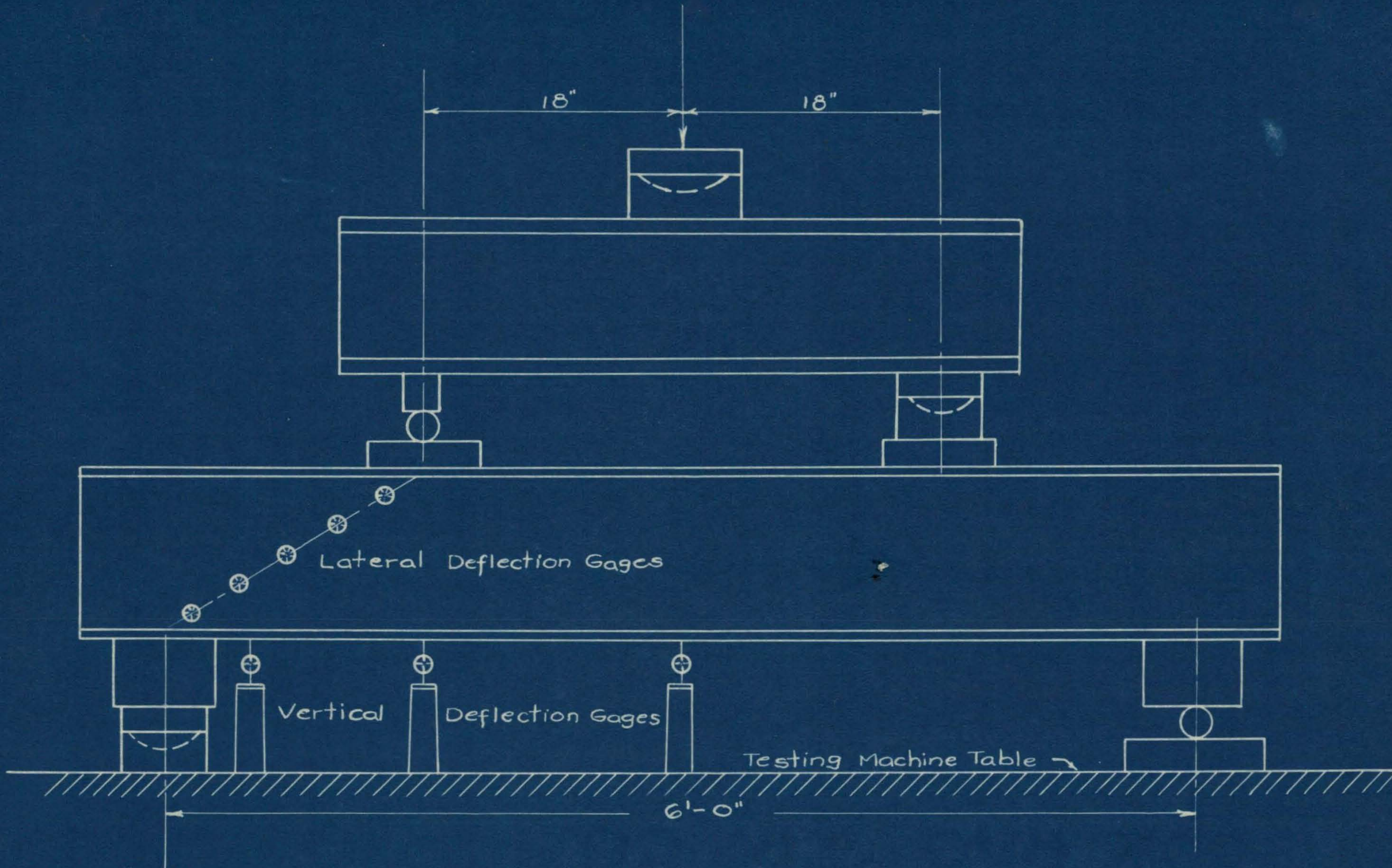

FIG. 1. LOADING ARRANGEMENT FOR PRELIMINARY INVESTIGATION

Fig. 2 - End View of Beam No.2, Bethlehem B12-28 Section

Showing Failure Due to End Twisting

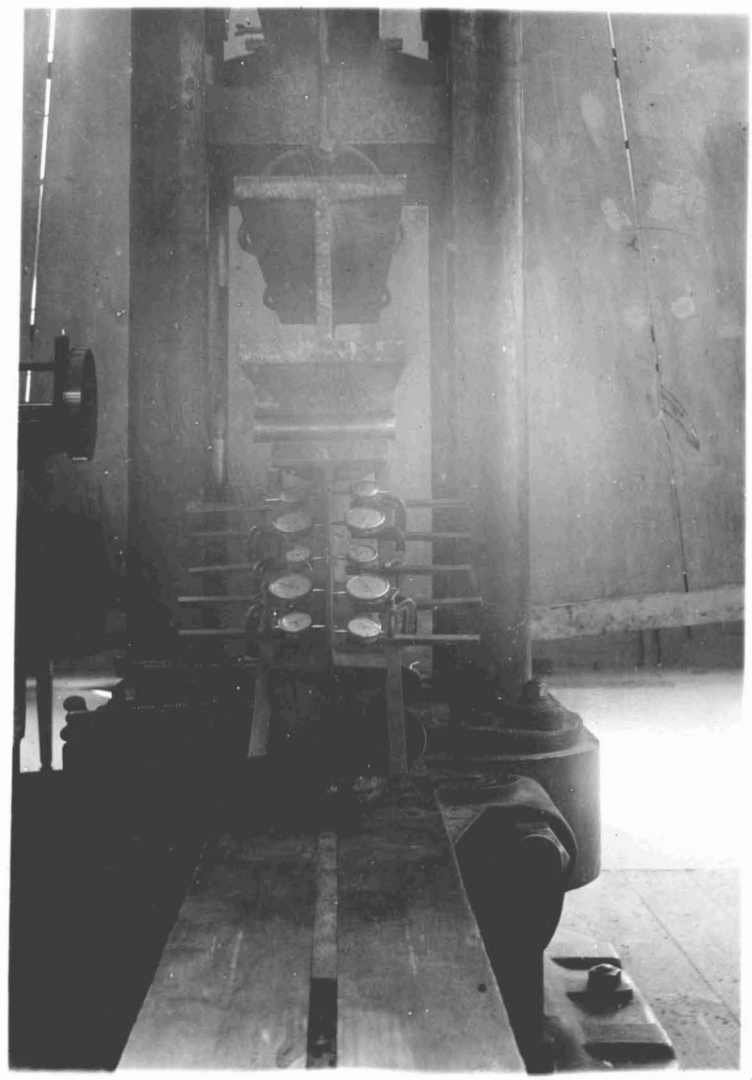

Fig. 3 - End View of Beam No. 2, Bethlehem B12-28 Section

Showing Arrangement of Gages

to Measure Web Buckling

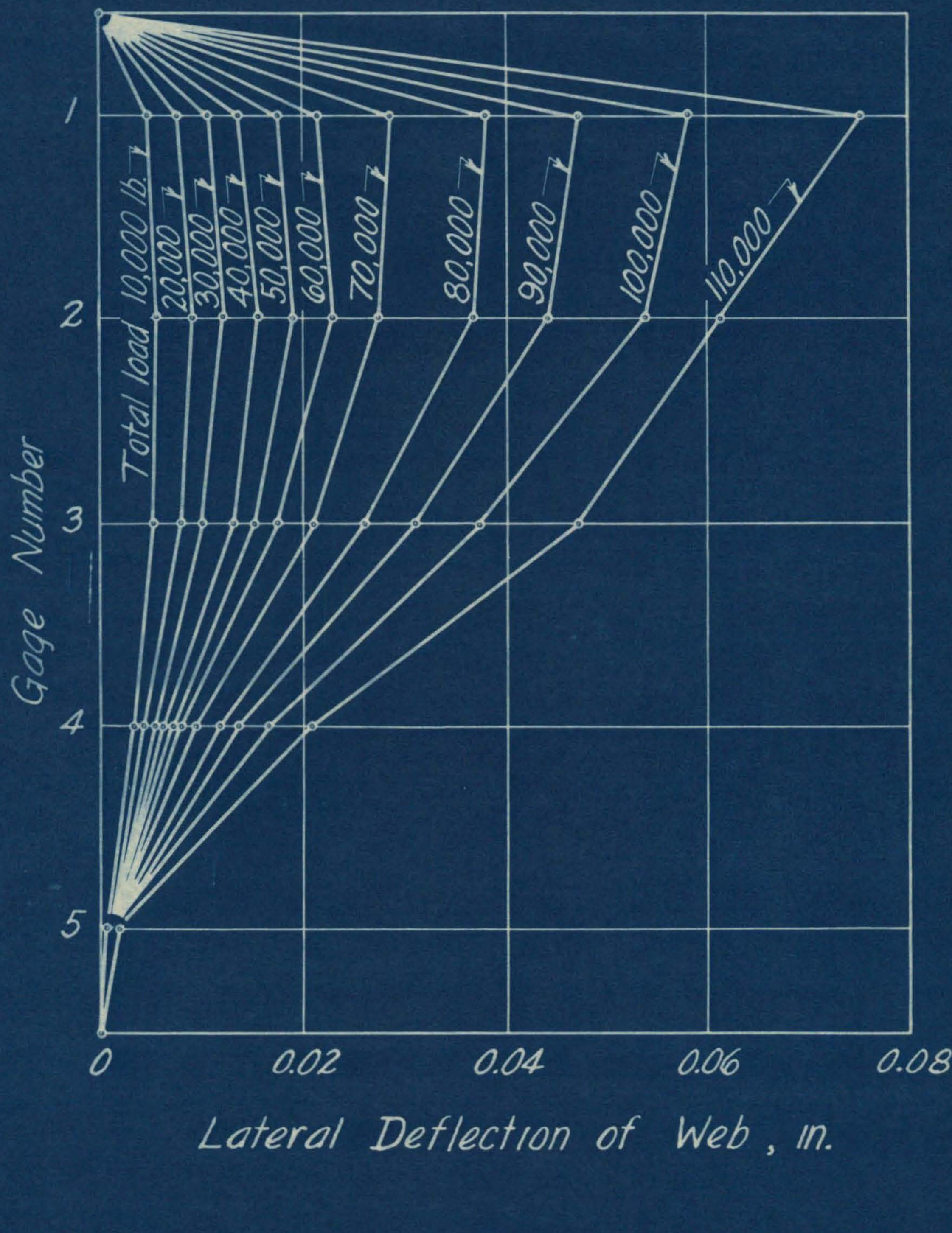

Fig. 4 Lateral Deflection of Web of Beam No. 2
B - 12 in. 28 lb. Bethlehem I-Beam.
Ultimate Load - 129,000 lbs. Tested April 5, 1932.

B12 X 28
No. 2.

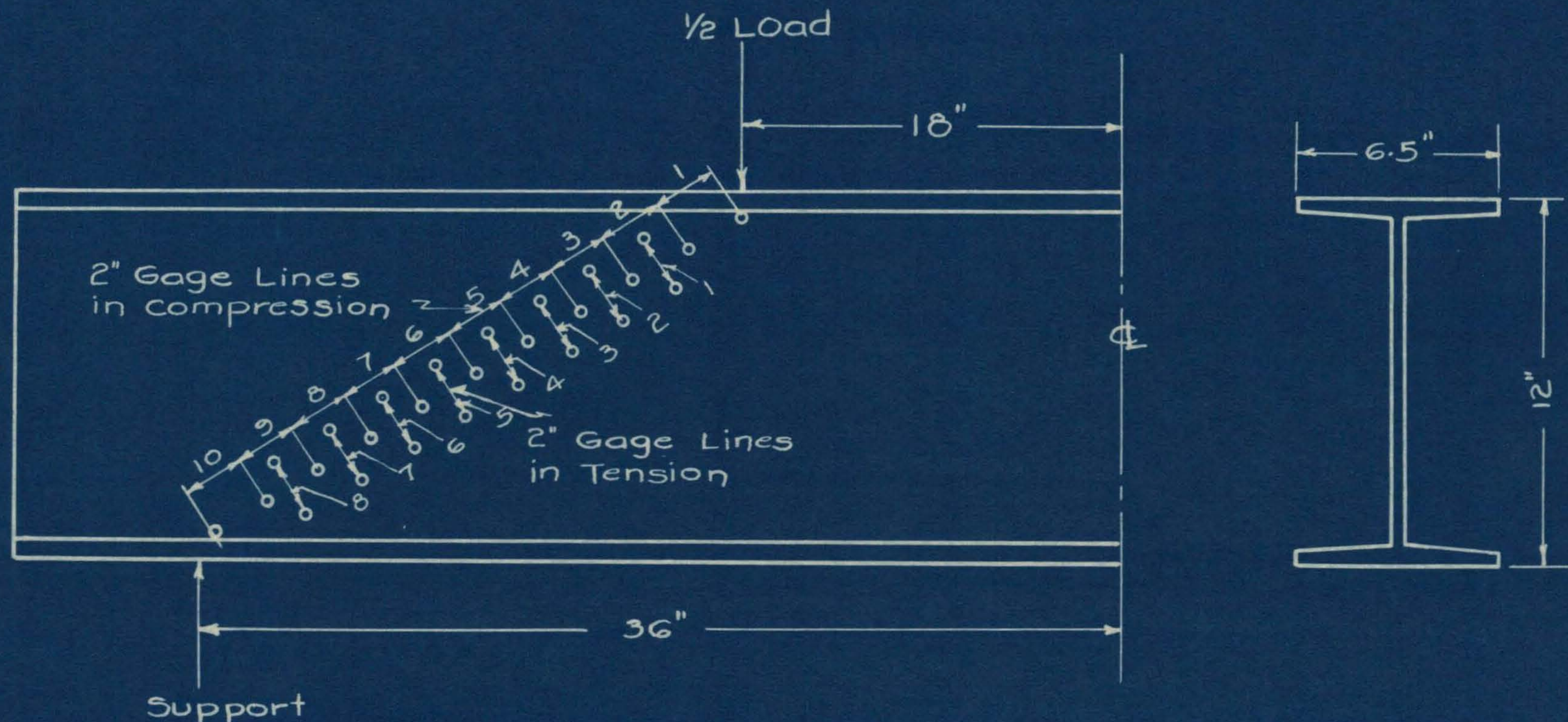

FIG. 5. LOCATION OF STRAIN GAGE POINTS ALONG WEB

No. 2 B12X28

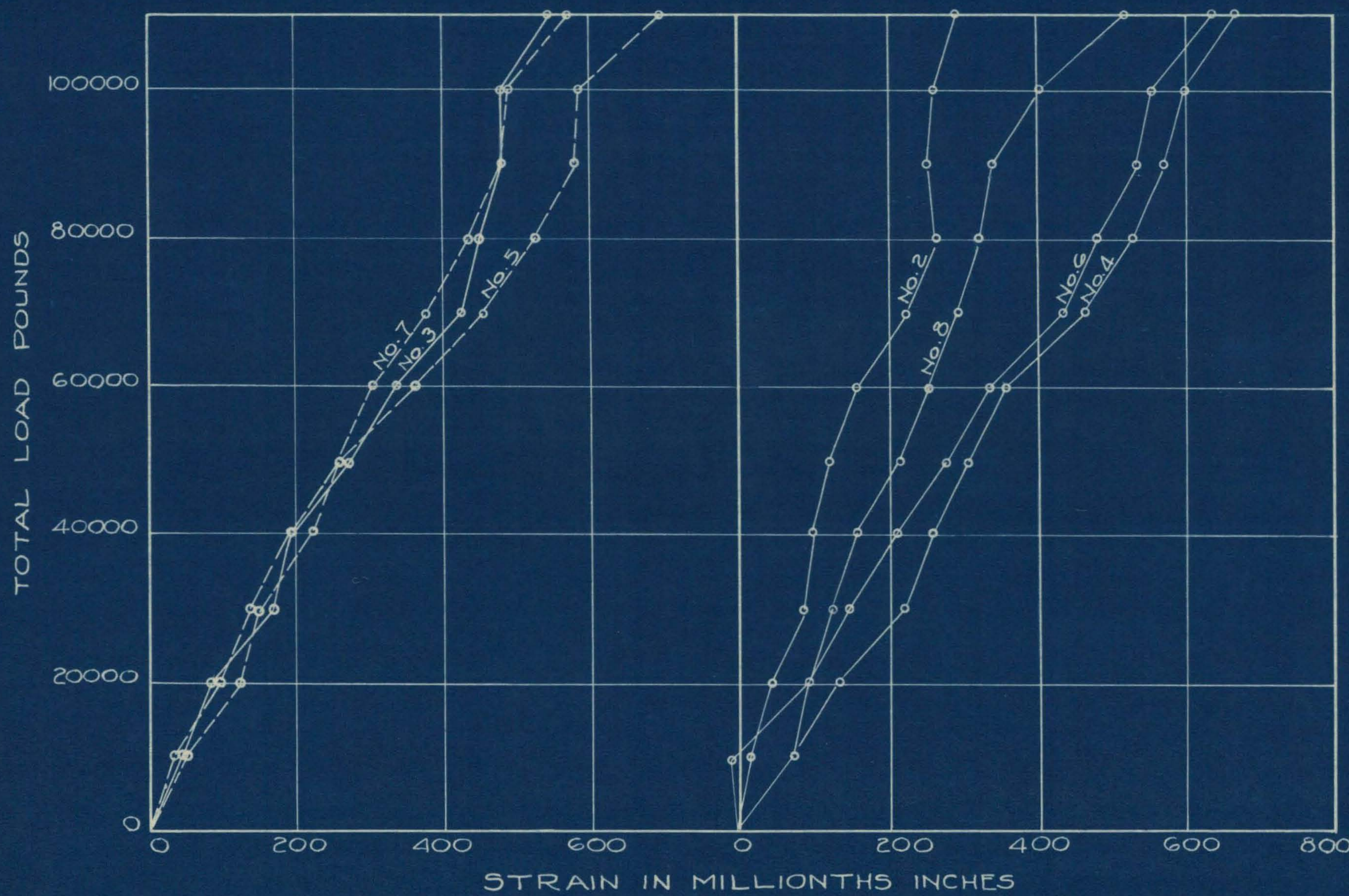

FIG. 6 AVERAGE STRAIN IN TENSION ALONG LINE CONNECTING LOADING POINT AND SUPPORT

No. 2 B12 X28

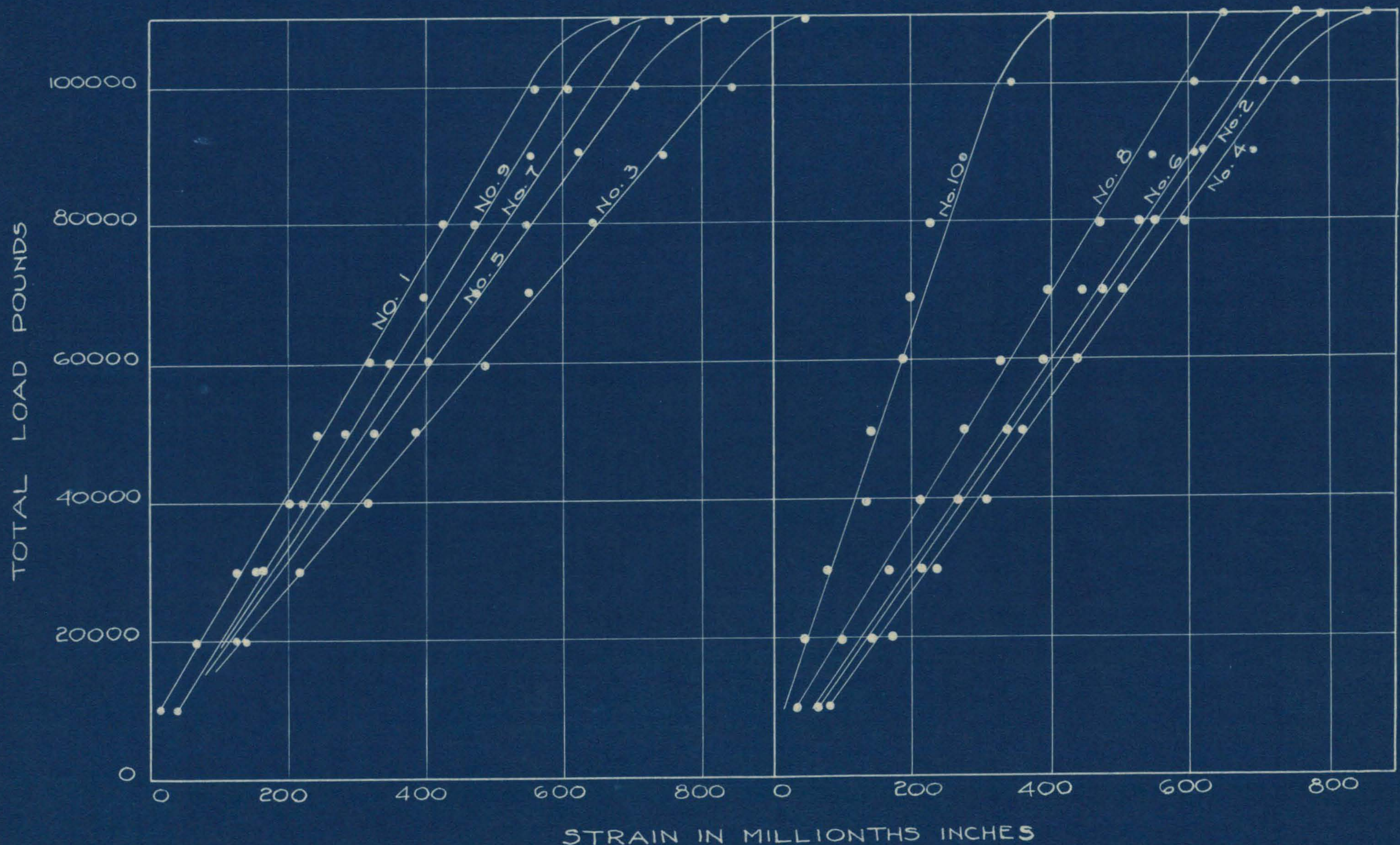

FIG. 7. AVERAGE STRAIN IN COMPRESSION ALONG LINE CONNECTING LOADING POINT AND SUPPORT

No. 2 B12 X 28

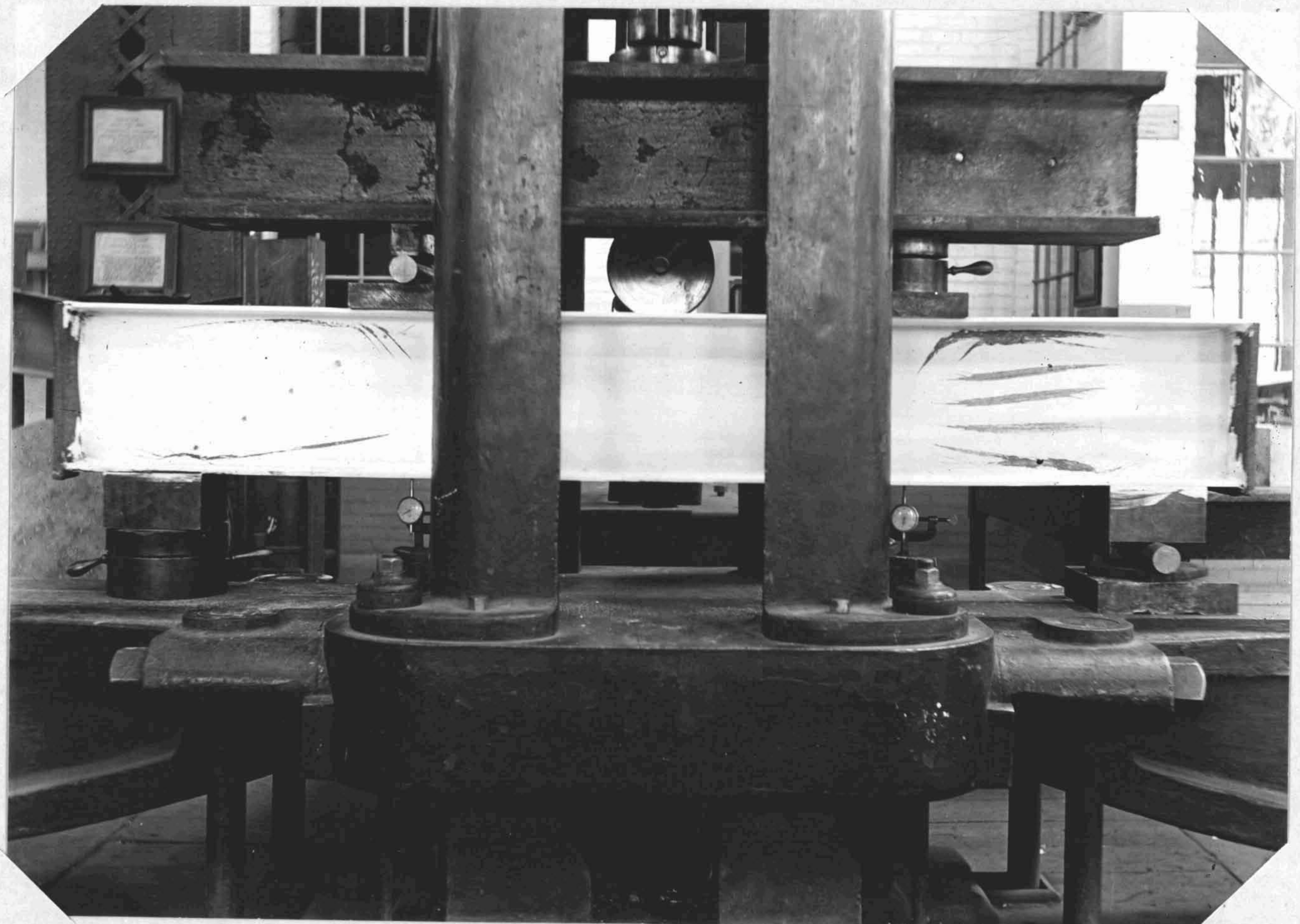

Fig. 8 - Loading Arrangement of Beam No.3, 12 x 28-lb. Bethlehem Section with Welded End Plates Showing Web Failure Due to Shear

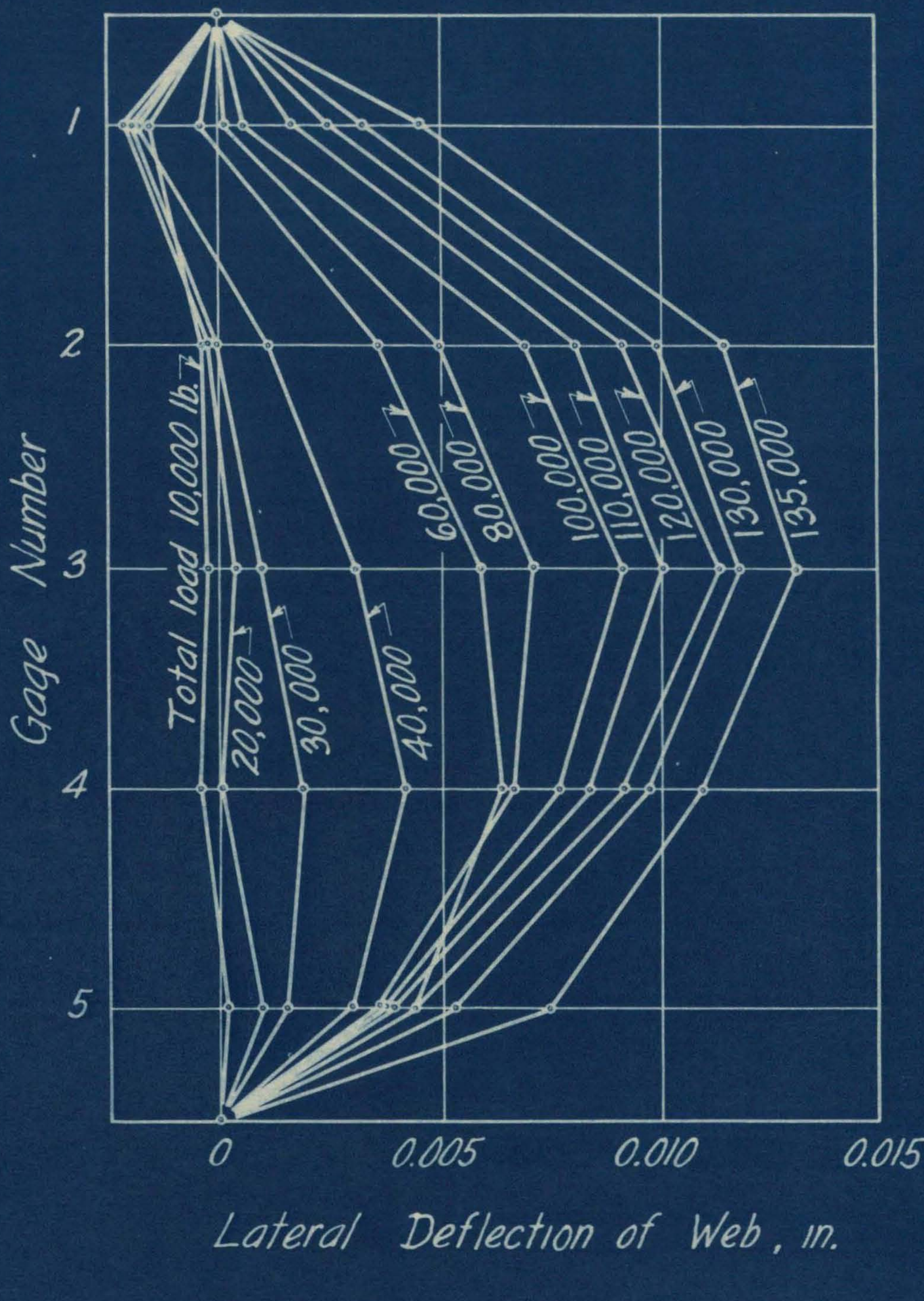

Fig. 9 Lateral Deflection of Web of Beam No. 3
B-12 in. 28 lb. Bethlehem I-Beam with End
Plates. Ultimate Load 141,500 lbs. Tested May 27, 1932.

B12 X 28
No. 3

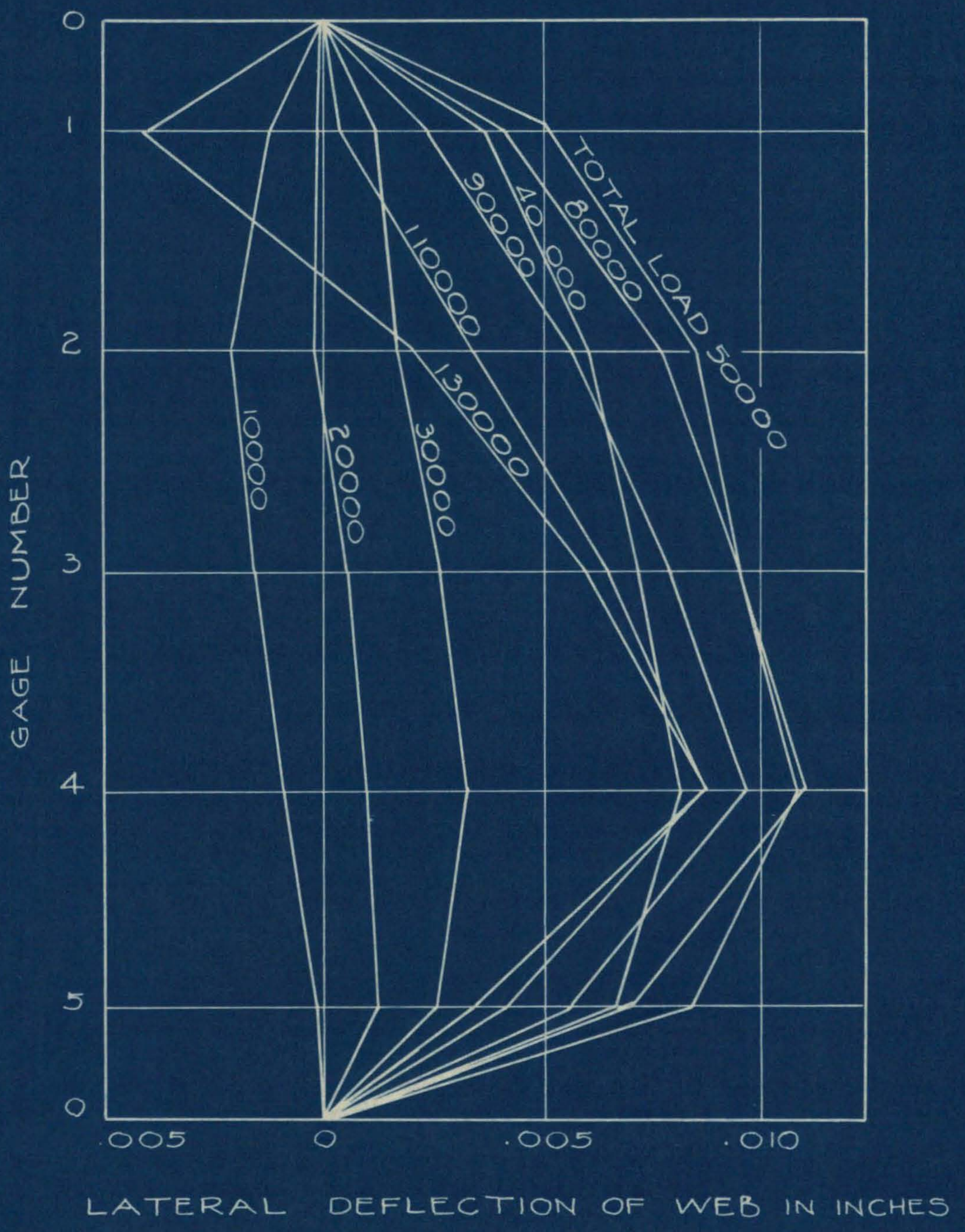

FIG. 10. LATERAL DEFLECTION OF WEB OF BEAM No 4. B12" x 28 BETHLEHEM I-BEAM. ULTIMATE LOAD 144,450. TESTED. JULY 1, 1932.

No. B12 X 28

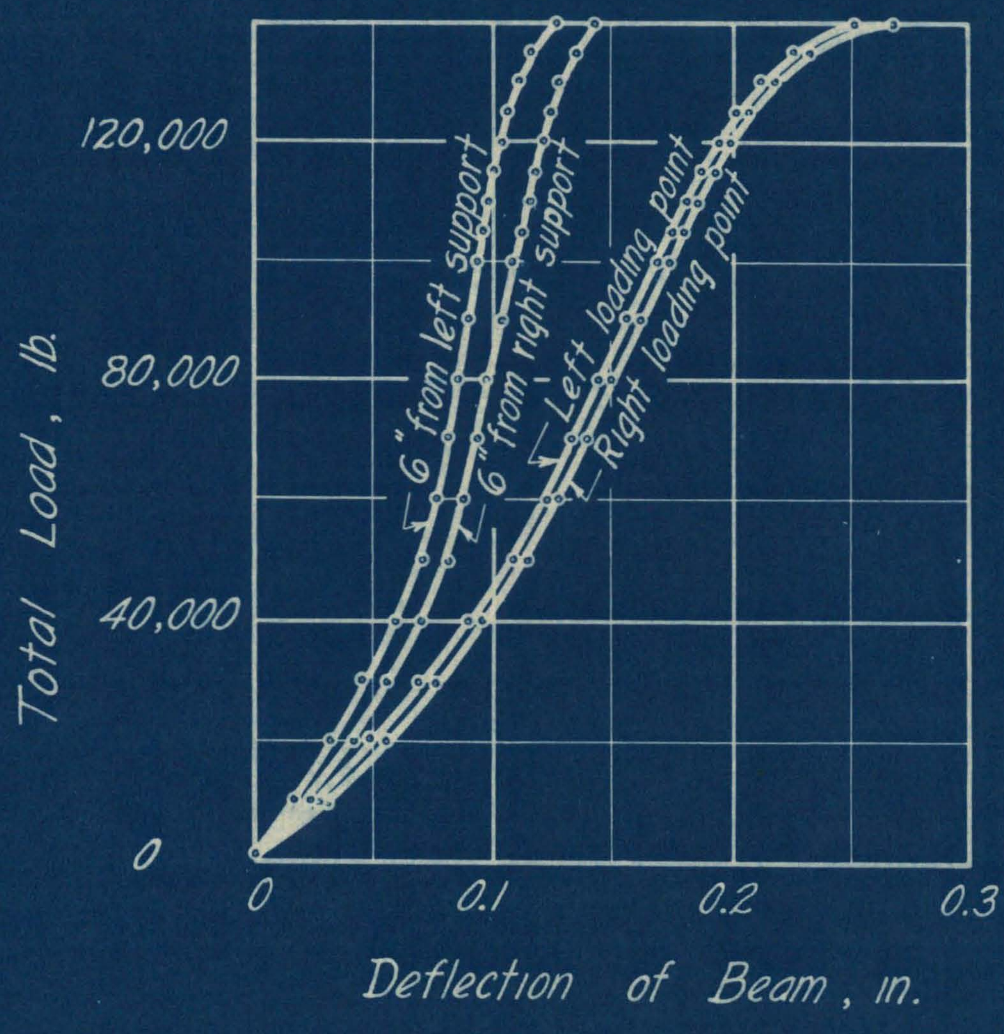

Fig 11 Deflection of a B-12 in., 28 lb. Bethlehem I-Beam with End Plates. Beam No. 3 Ultimate Load-141,500 lbs. Tested May 27, 1932.

B12 X 28
No. 3

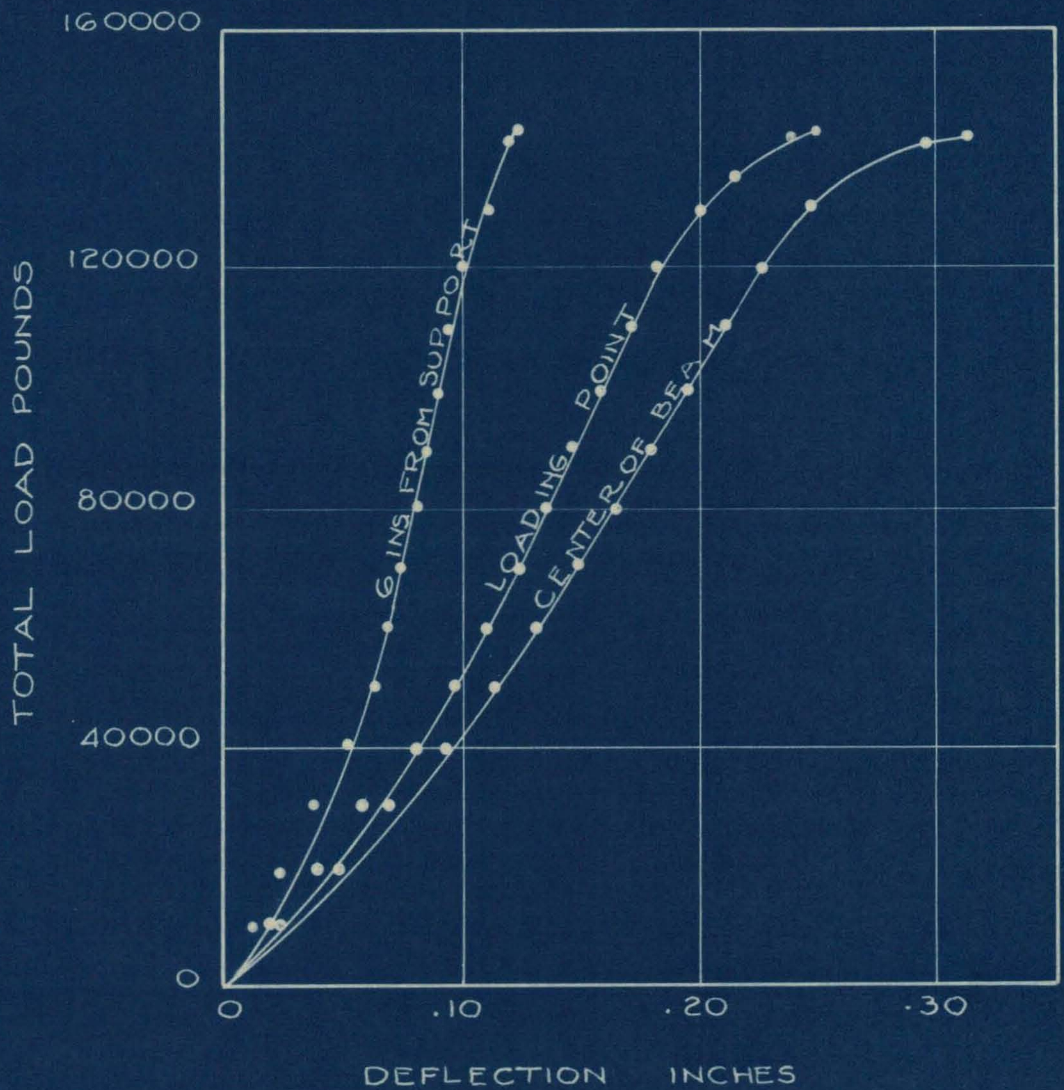

FIG. 12. DEFLECTION CURVES FOR No. 4 B12 X 28 LB. BETHLEHEM BEAM ULTIMATE LOAD 144,450. TESTED JULY 1, 1932.

No. 4 B12 X 28

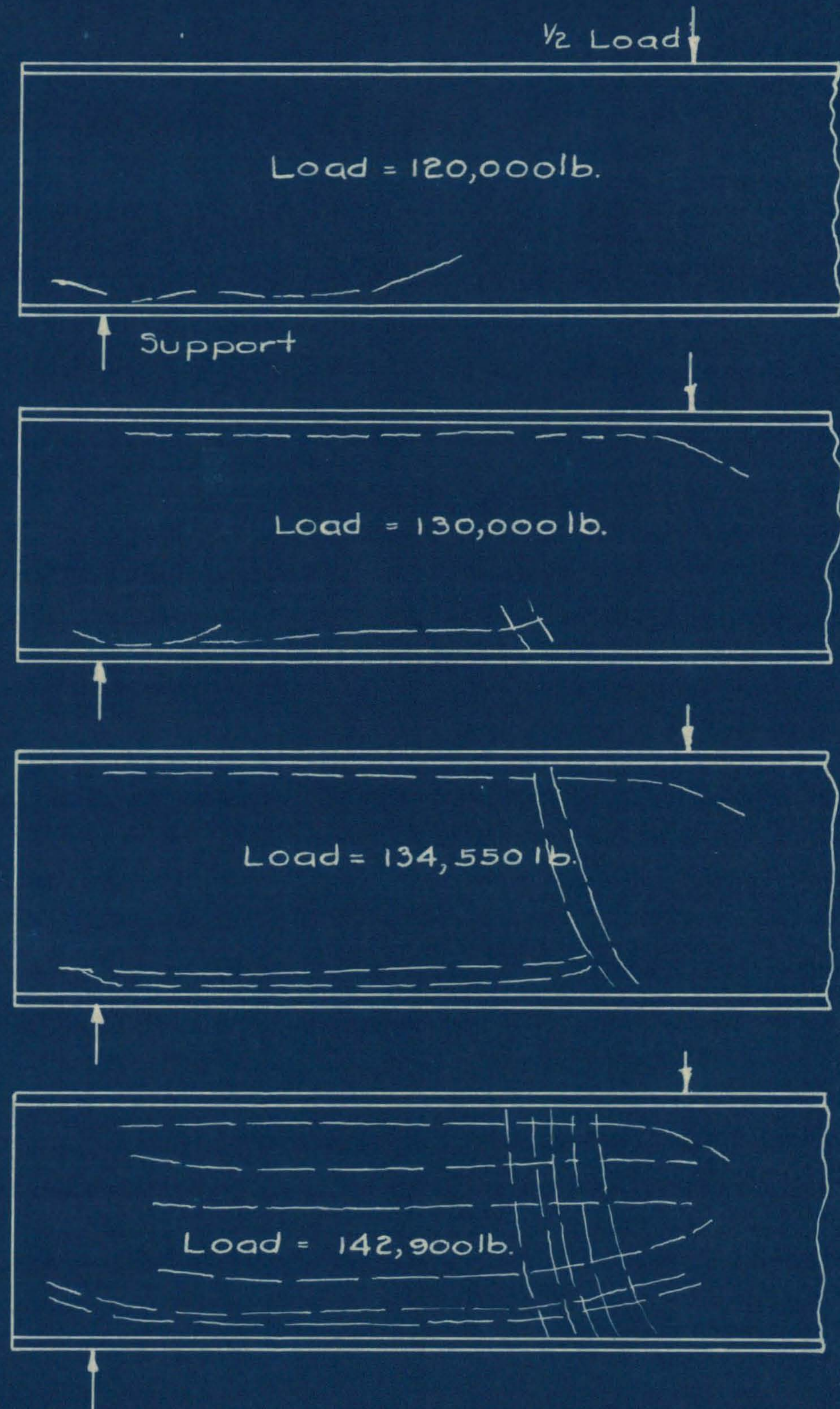

FIG. 13 FLAKING OF WHITE-WASH ON BEAM No. 4 B12X28

No. 4. B12X28

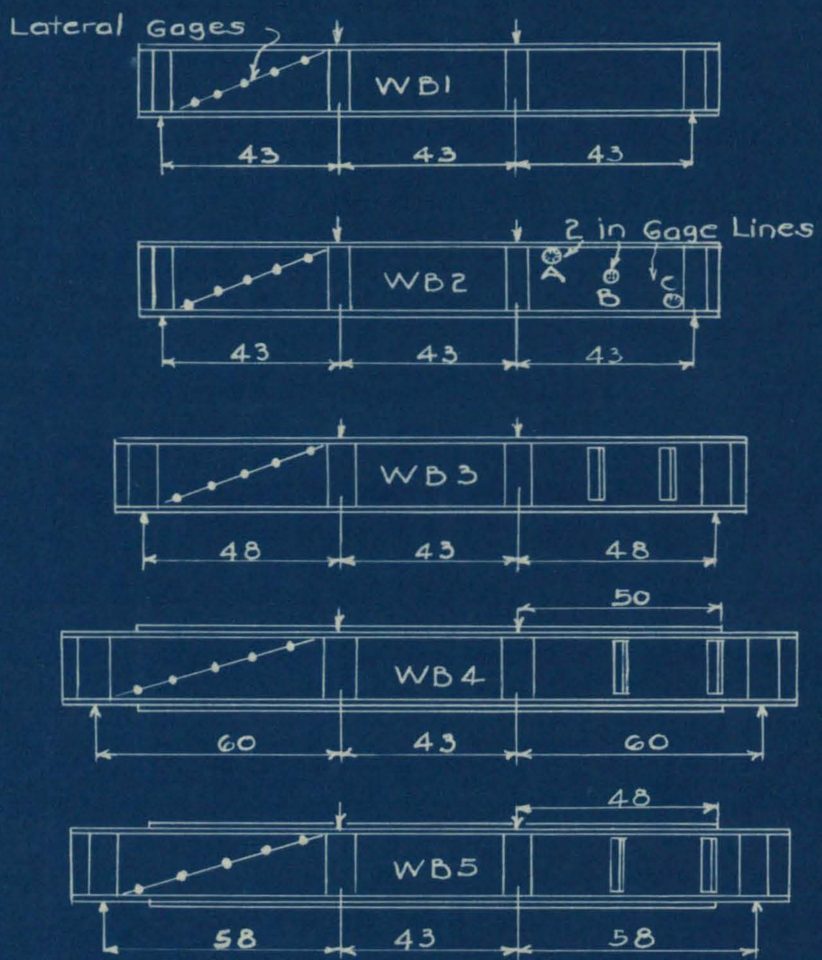

FIG. 14. MAKE-UP AND LOADING ARRANGEMENT FOR MAJOR INVESTIGATION GROUP A. BEAMS WB-1 – WB-5.

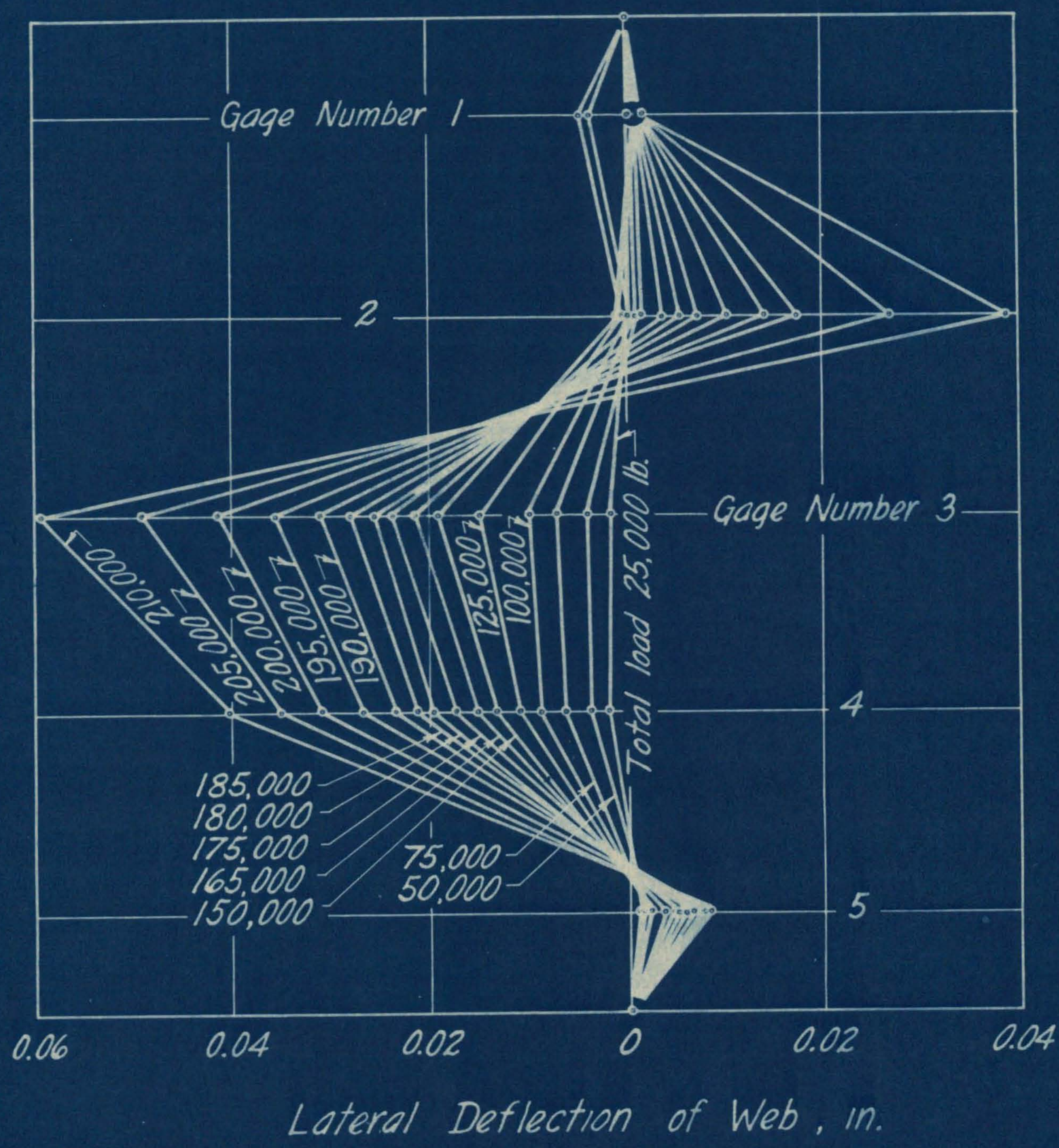

Fig. 15 Lateral Deflection of Web of 17 in. Welded Beam No. WB-1
Ultimate Load 218,500 lbs. Tested June 15, 1932.

WB-1

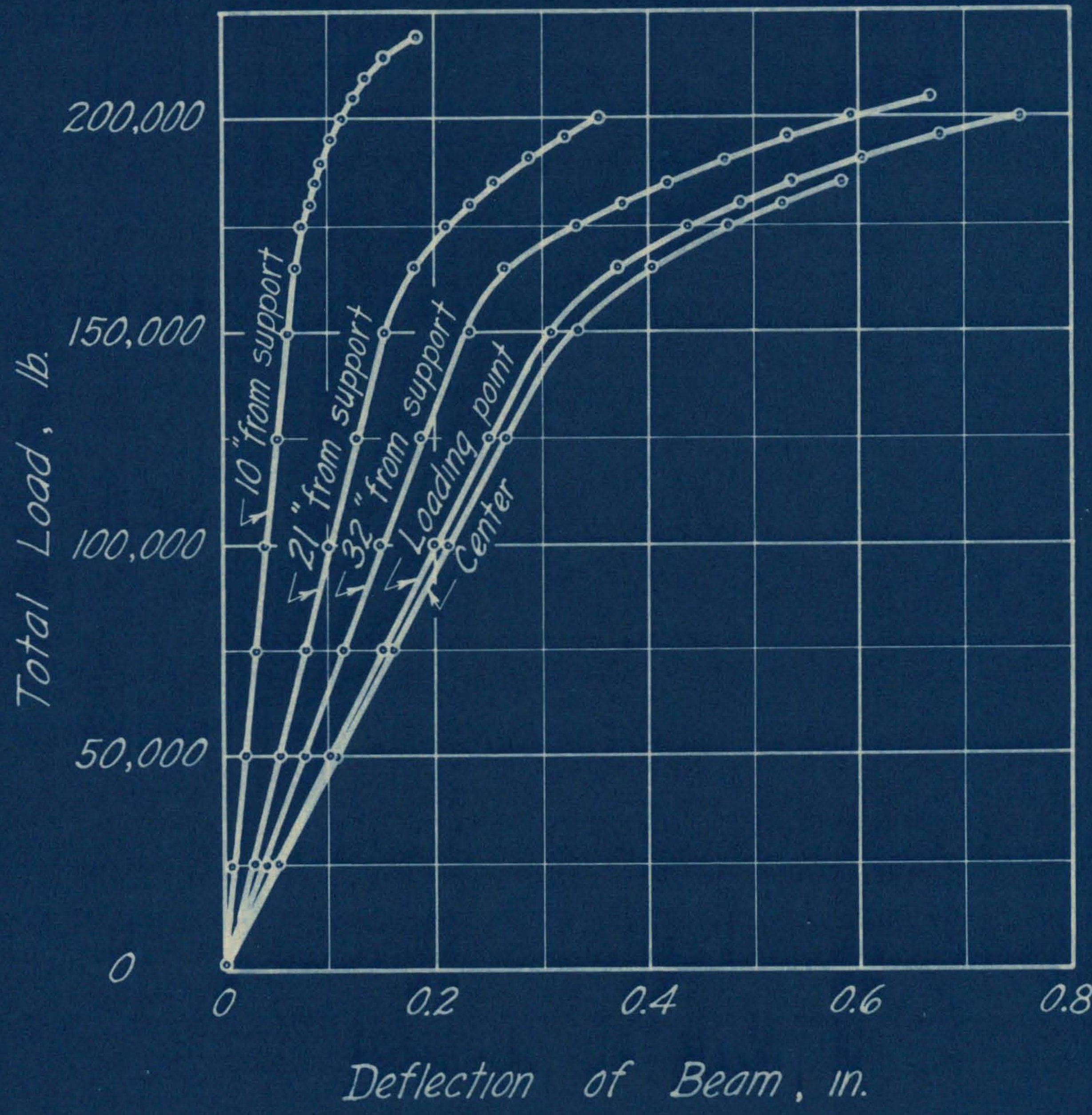

Fig 16. Deflection of 17 in Welded Beam No. WB-1
Ultimate Load 218,500 lbs. Tested June 15, 1932.

WB-1

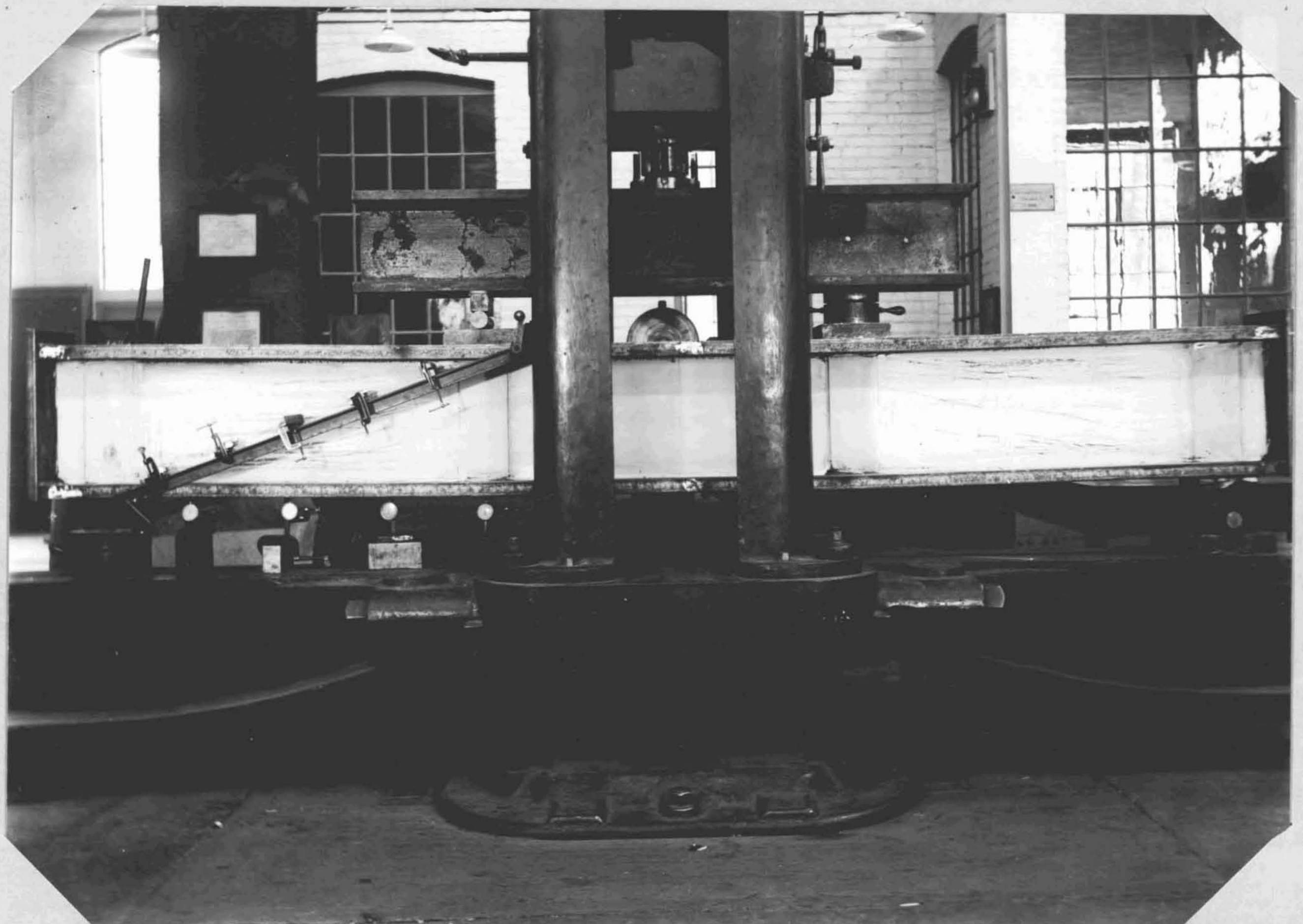

Fig. 17 - Welded Beam, WB-1, Showing Web Failure Due to Shear

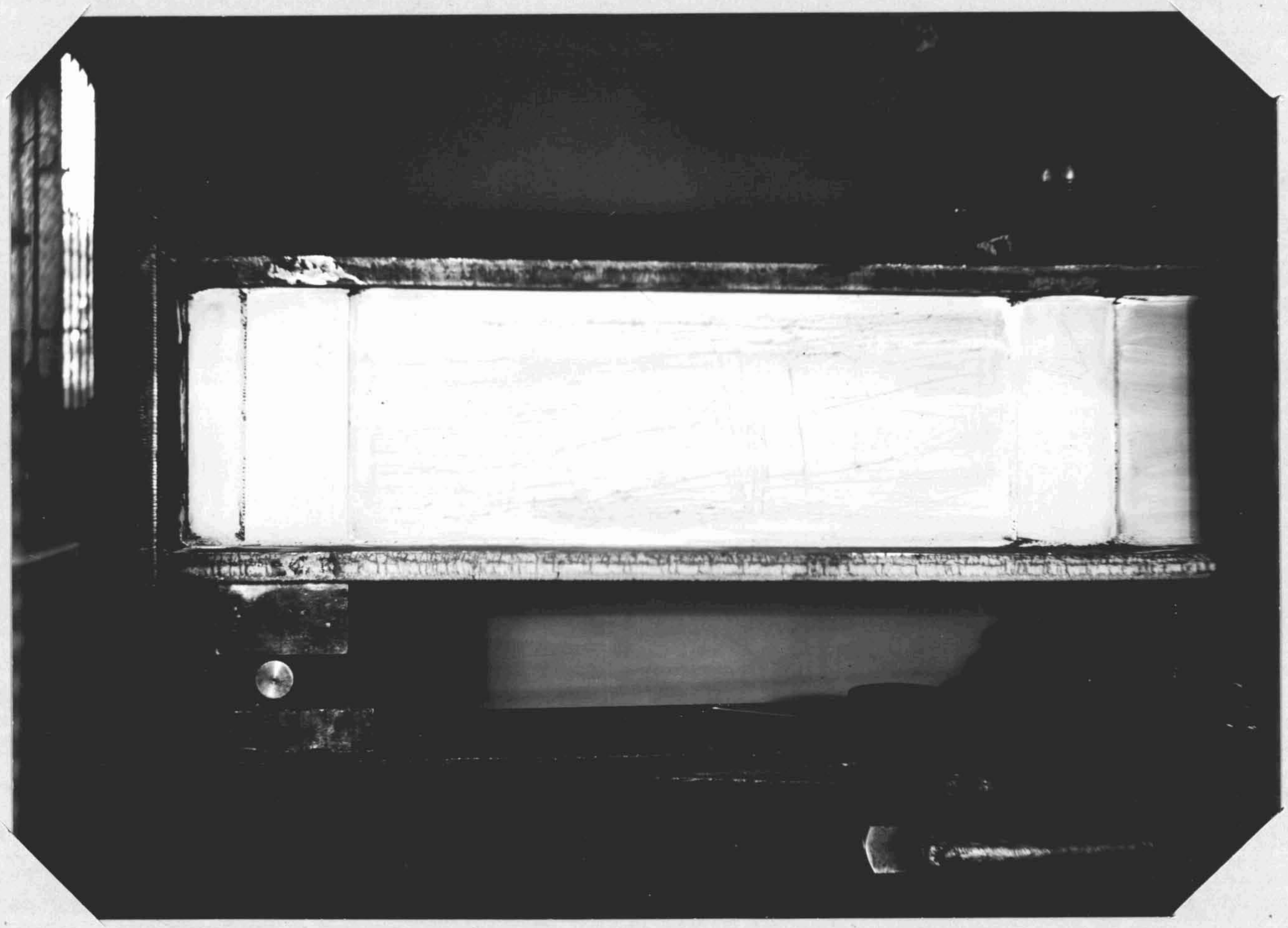

Fig. 18 - Welded Beam, WB-1, Showing Strain Lines on Web Due to Shear

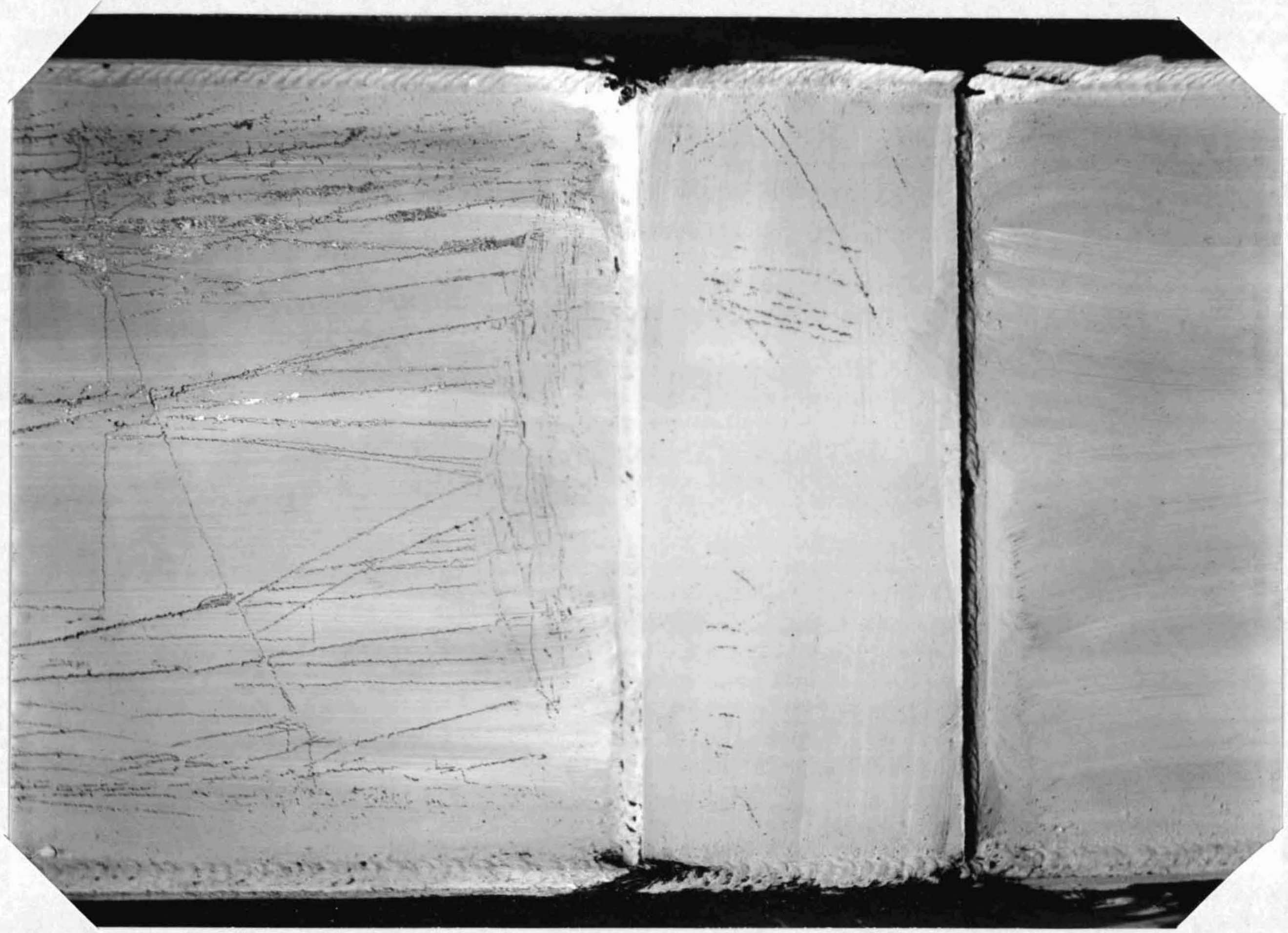

Fig. 19 - Welded Beam WB-1 Showing Strain Lines on Web and Stiffeners

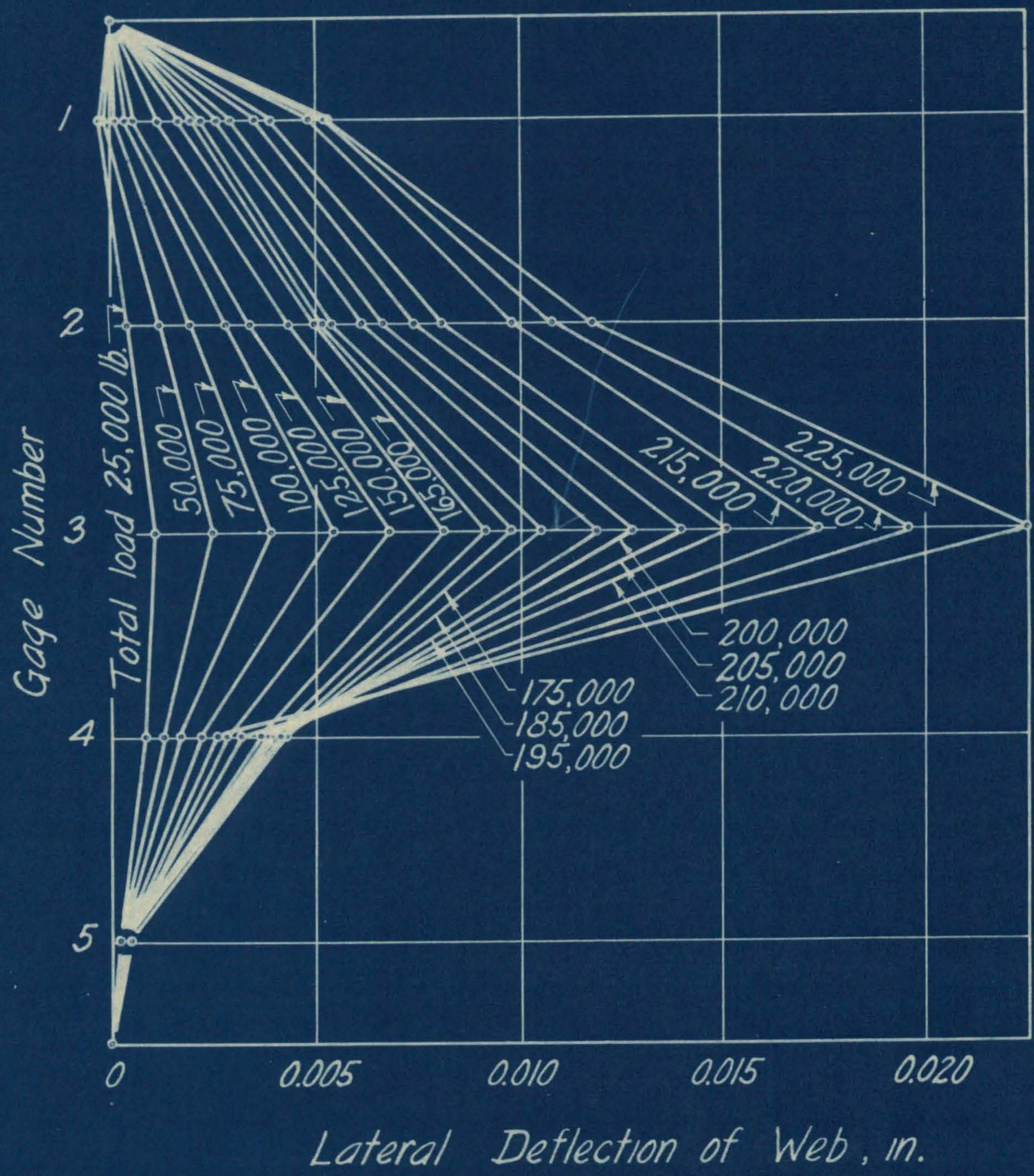

Fig. 20 Lateral Deflection of Web of 17 in Welded Beam No. WB-2.
Ultimate Load - 255,600 lbs. Tested May 20, 1932

WB-2

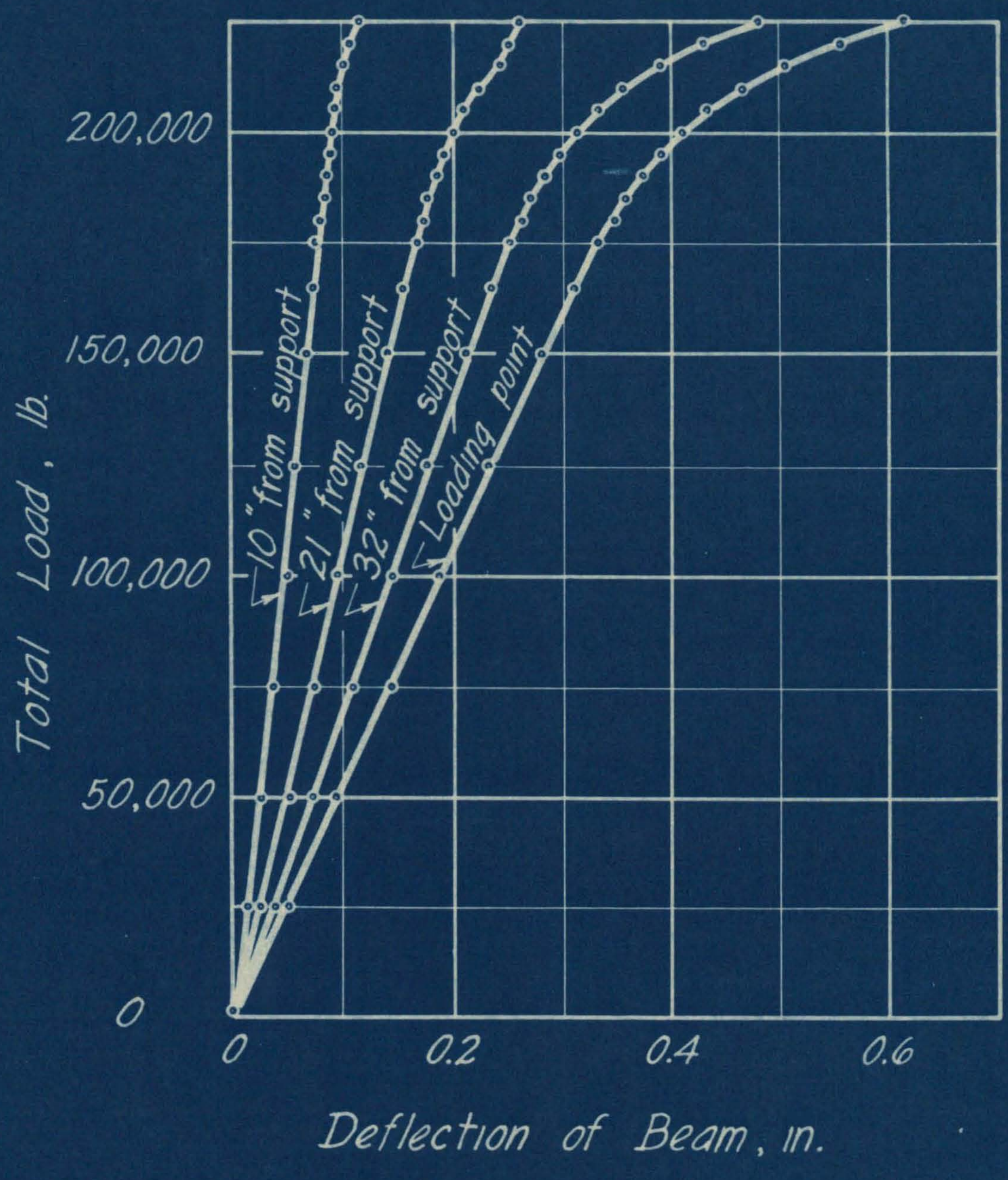

Fig. 21 Deflection of 17 in Welded Beam No. WB-2.
Ultimate Load - 255,600 lbs. Tested May 20, 1932.

WB-2

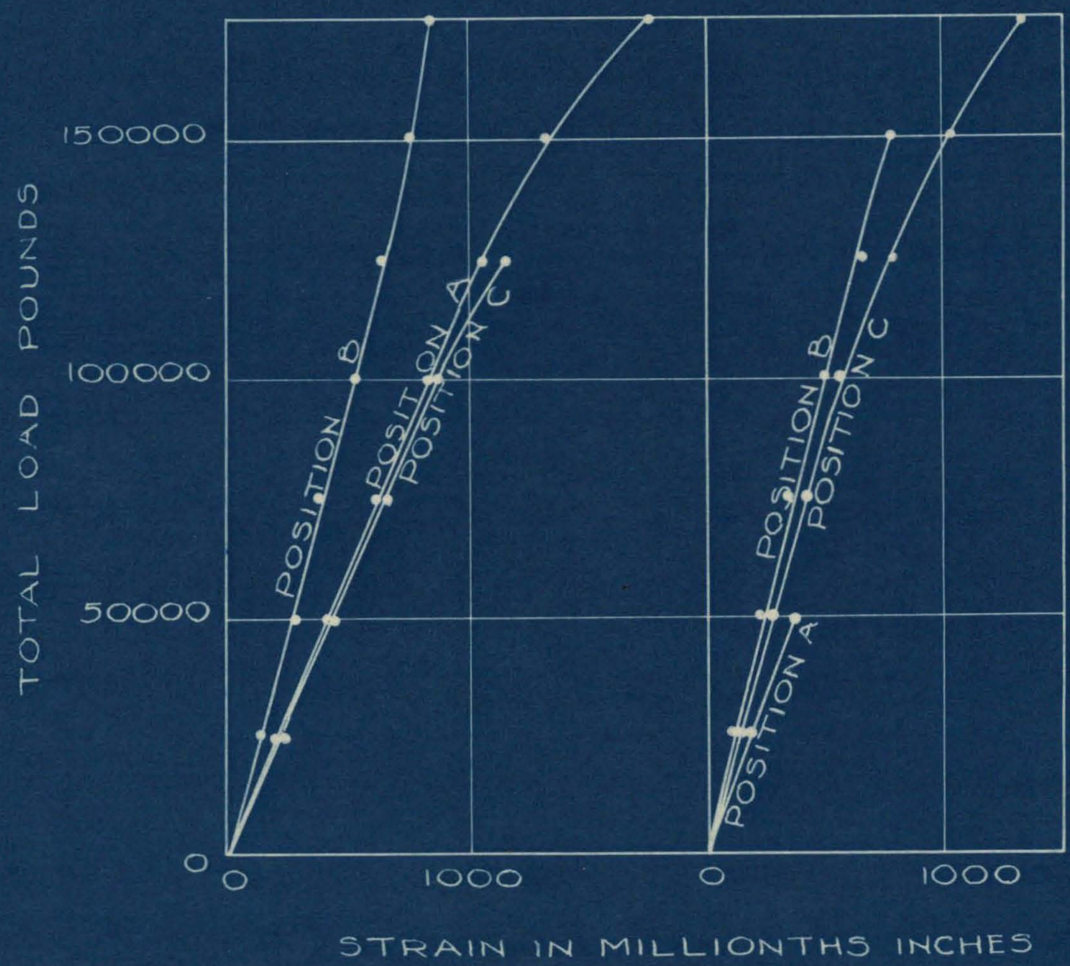

FIG. 22. AVERAGE TENSION AND COMPRESSION STRAINS ON WEB OF BEAM WB-2. TESTED-MAY. 20, 1932

WB-2

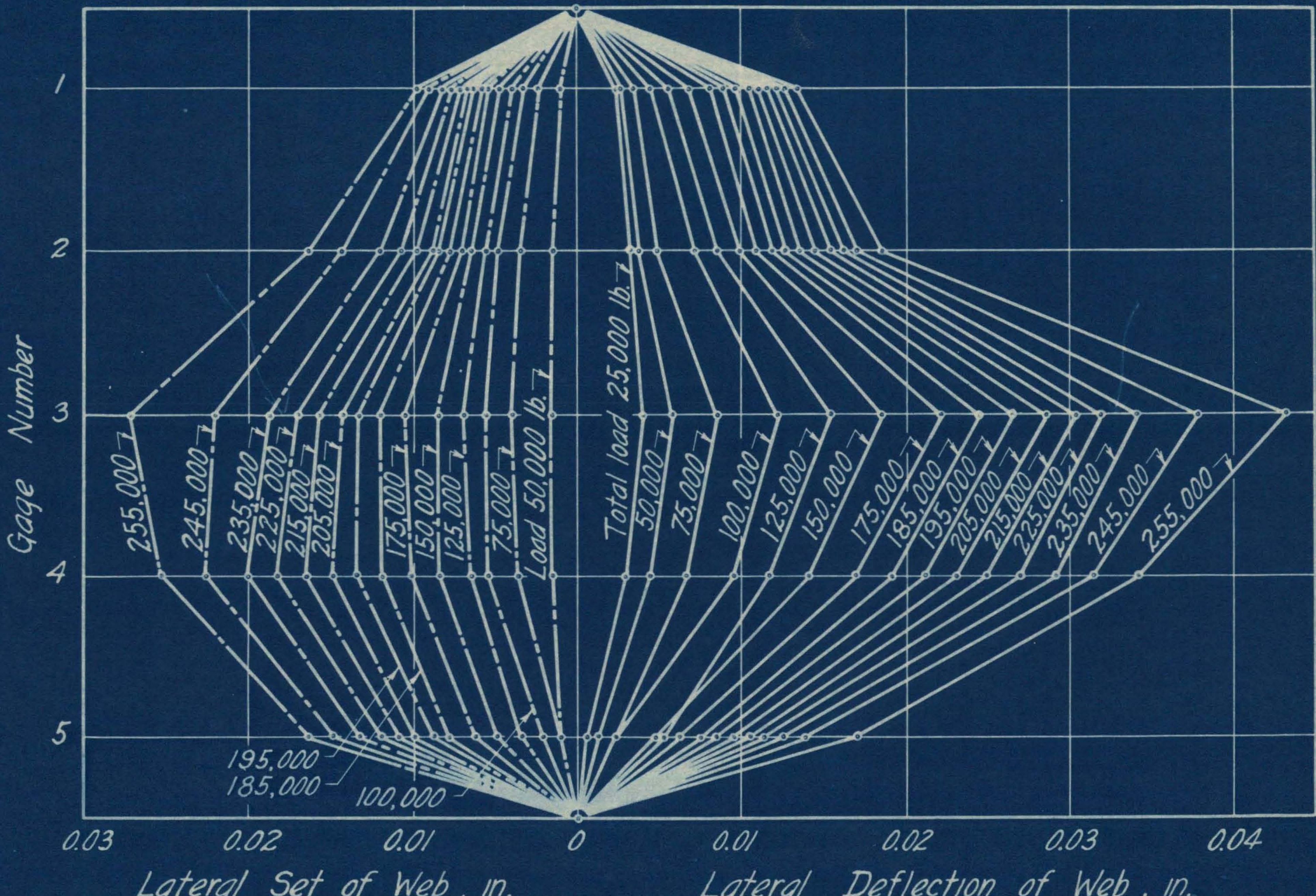

Fig.23 Lateral Set and Deflection of Web of 19 in Welded Beam No. WB-3
Ultimate Load 278,000 lbs. Tested June 16, 1932. WB-3

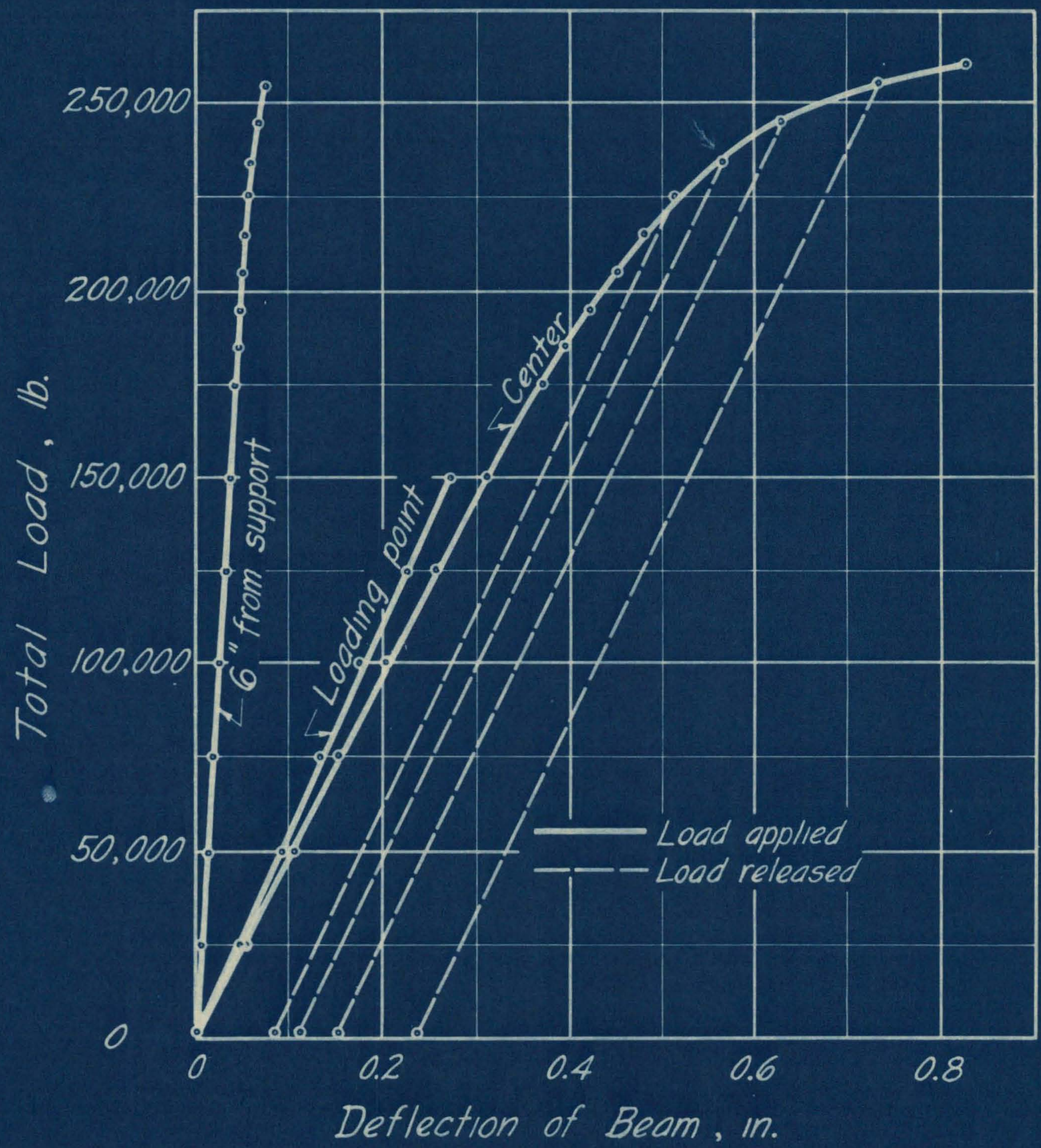

Fig. 24 Deflection of 19 in Welded Beam No. WB-3.
Ultimate Load 278,000 lbs. Tested June 16, 1932.

WB-3

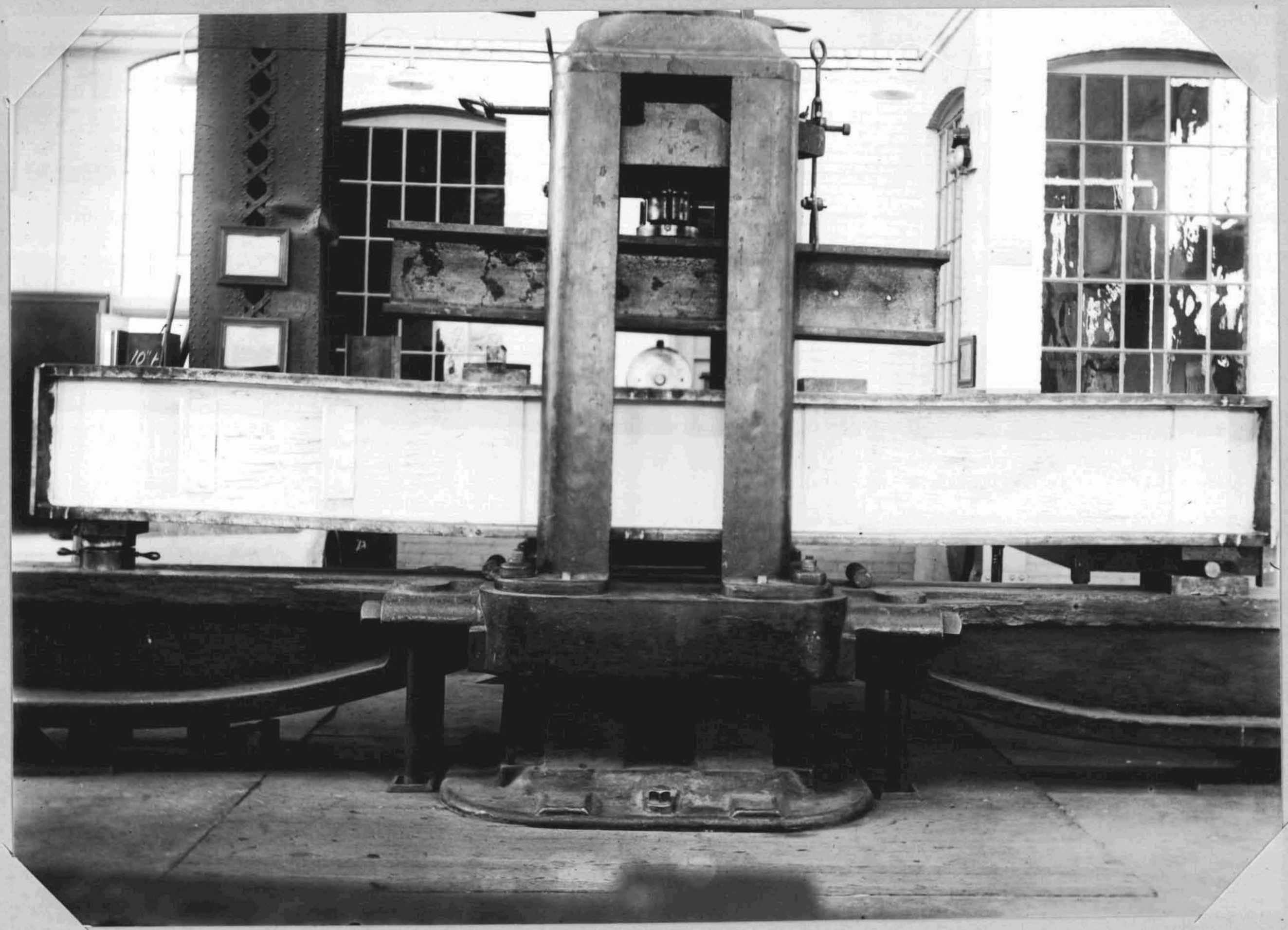

Fig. 25 - Welded Beam WB-3 Showing Failure at Ultimate Load

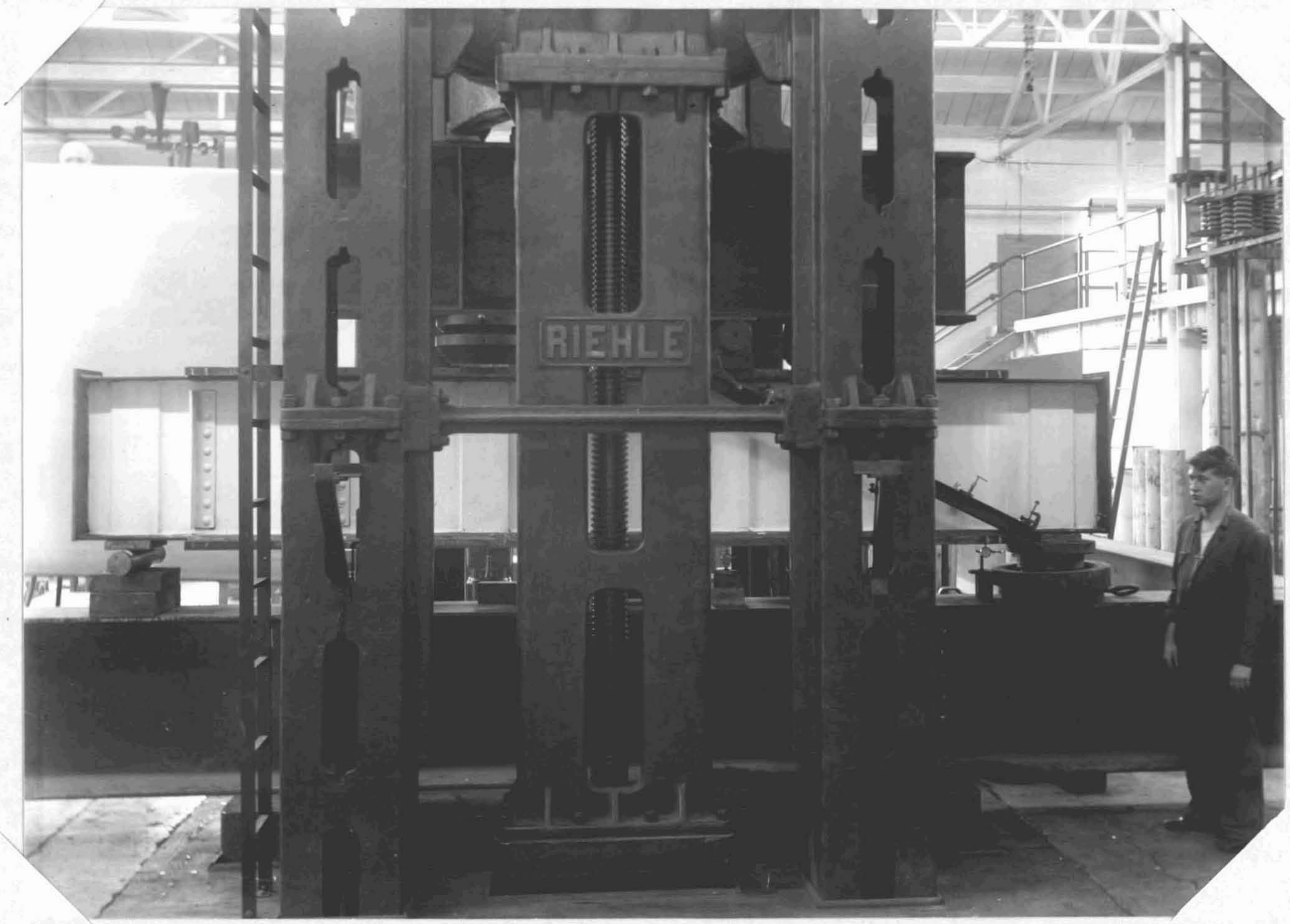

Fig. 26 - Loading Arrangement of Beam WB-4, A Reinforced Bethlehem B23-91 Section

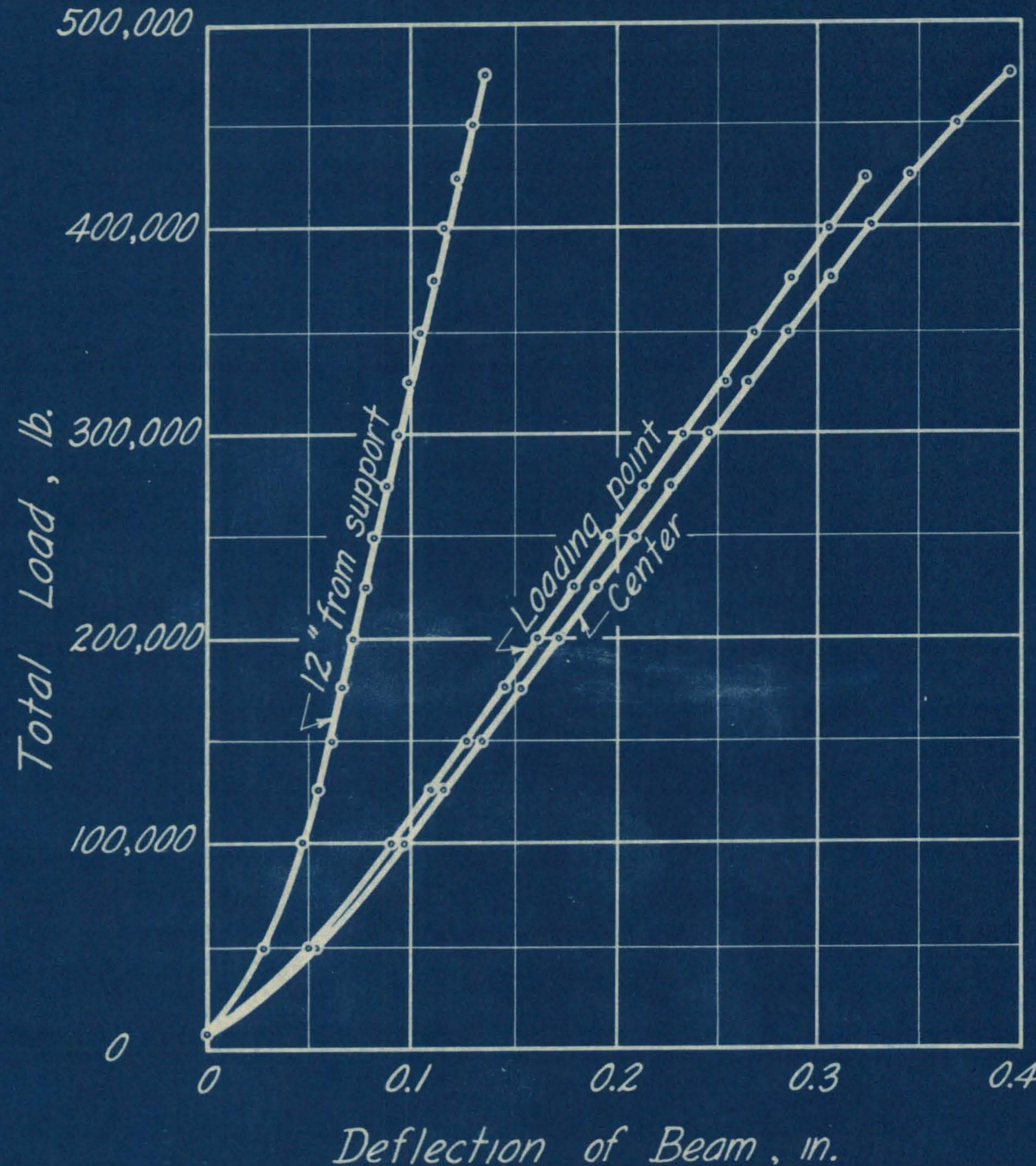

Fig. 27 Deflection of a Reinforced B-28 in, 91 lb.
Bethlehem I-Beam No. WB-4
Ultimate Load 475,000 lbs. Tested June 15, 1932.

WB-4

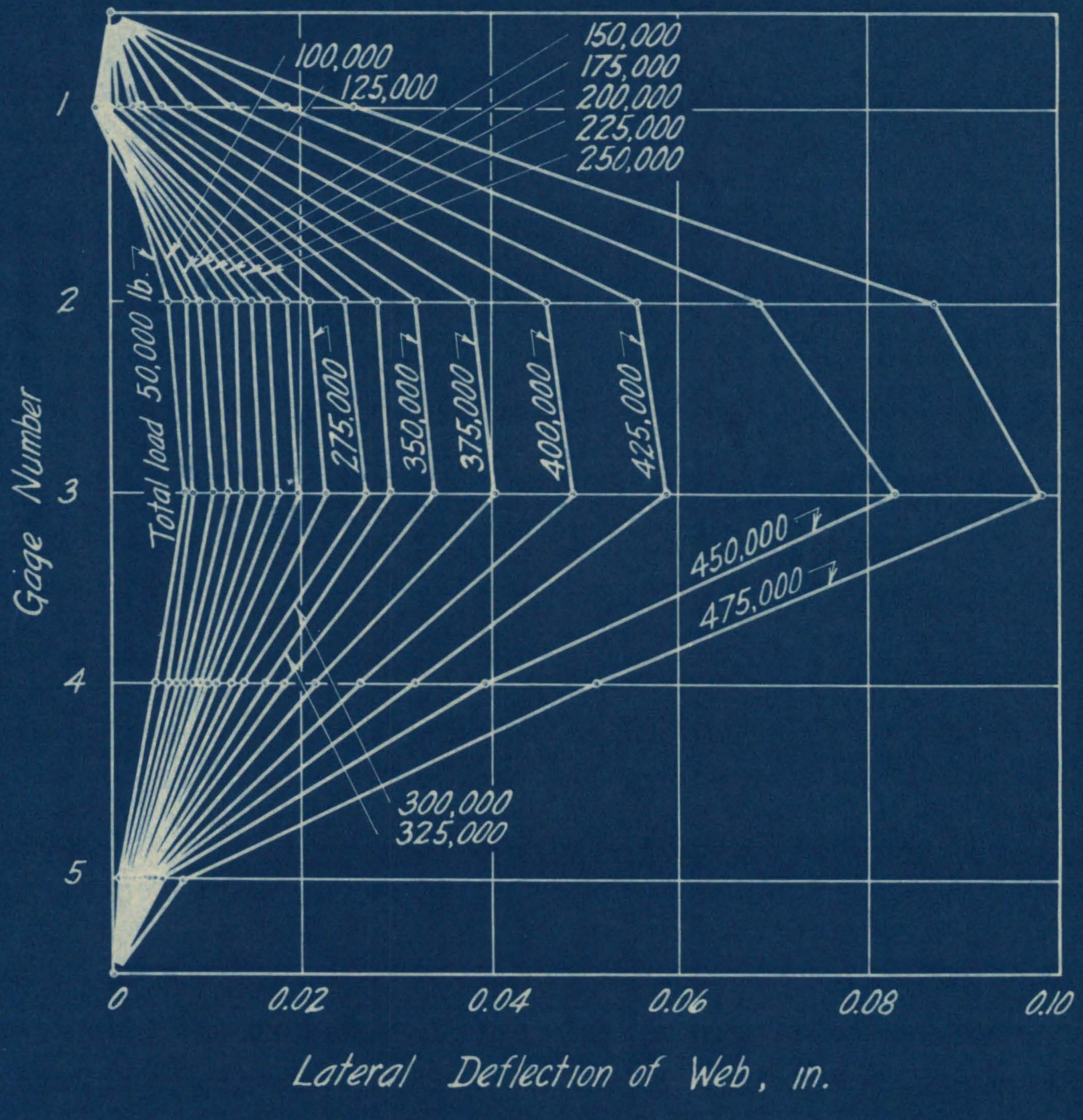

Fig. 28 Lateral Deflection of Web of Beam No. WB-4, a Reinforced B-28 in, 91 lb. Bethlehem I-Beam Ultimate Load 475,000 lbs. Tested June 15, 1932.

WB-4

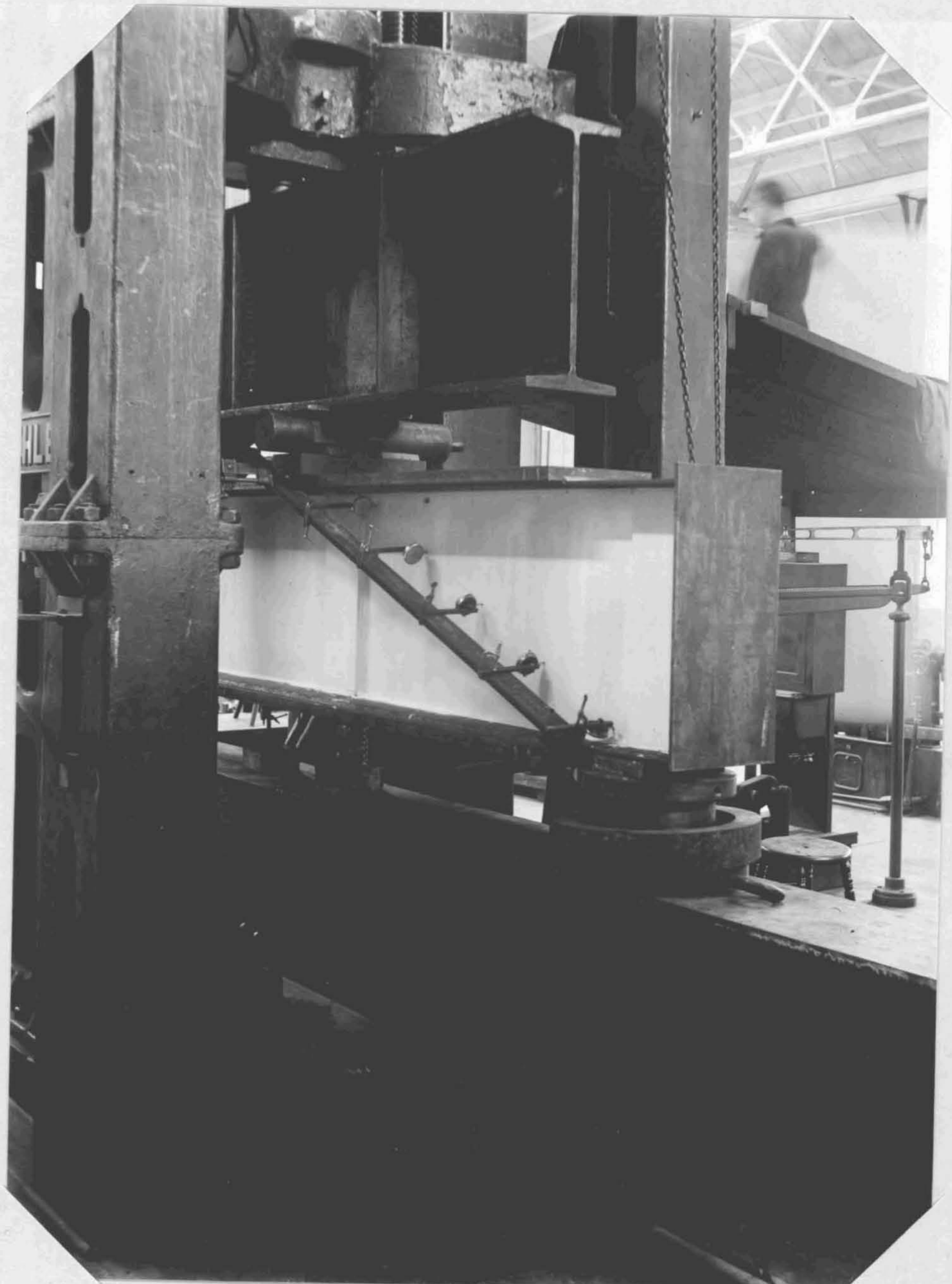

Fig. 29 - Loading Arrangement for Beam WB-4,
A Reinforced Bethlehem B28-91 Section

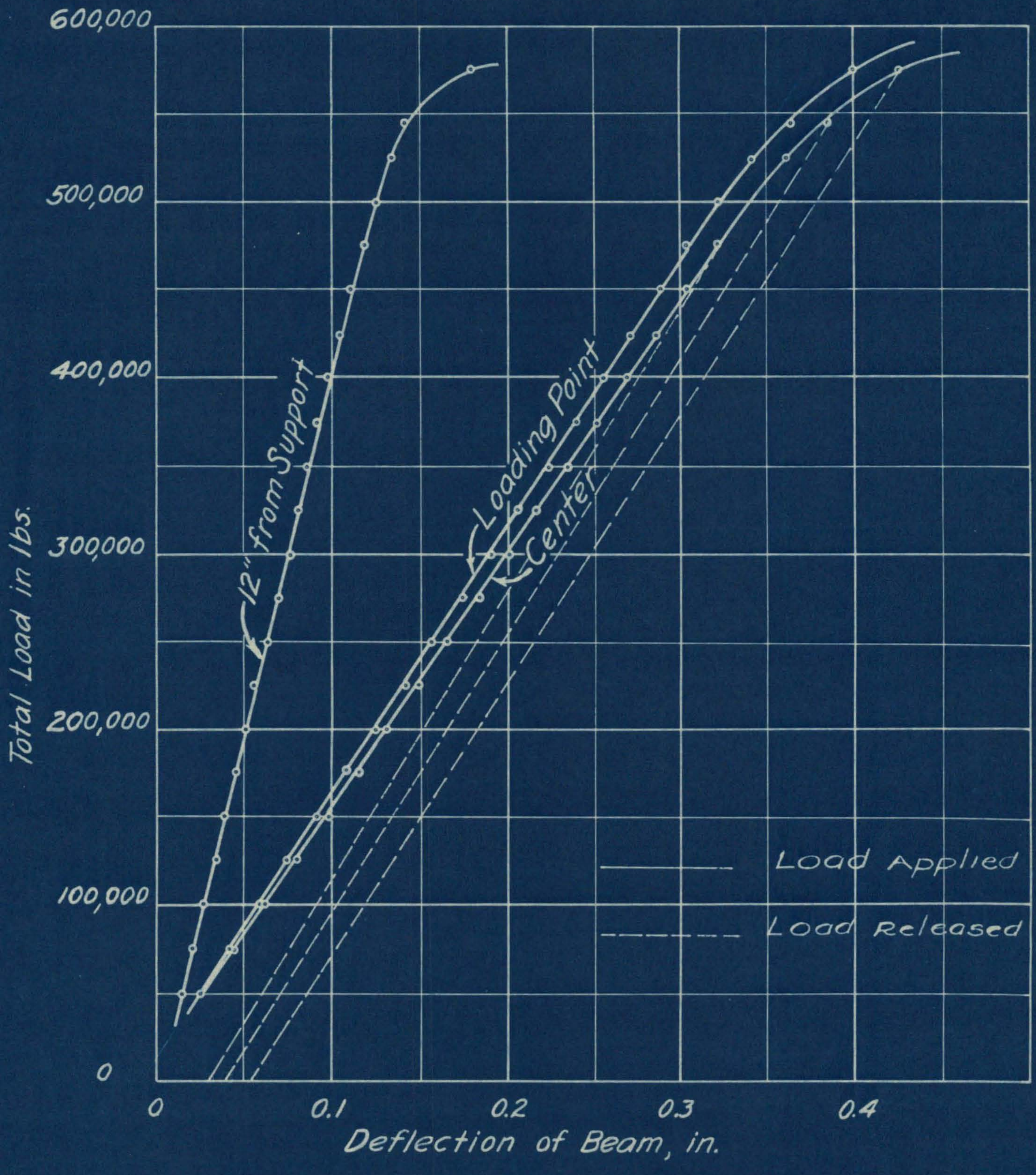

Fig 30 Deflection of a Reinforced B-28 in. 91 lb. Bethlehem I-Beam No. W.B-4 with ends Restrained Ultimate Load = 597600 lb. Retested June 30, 1932.

WB-4-2ND

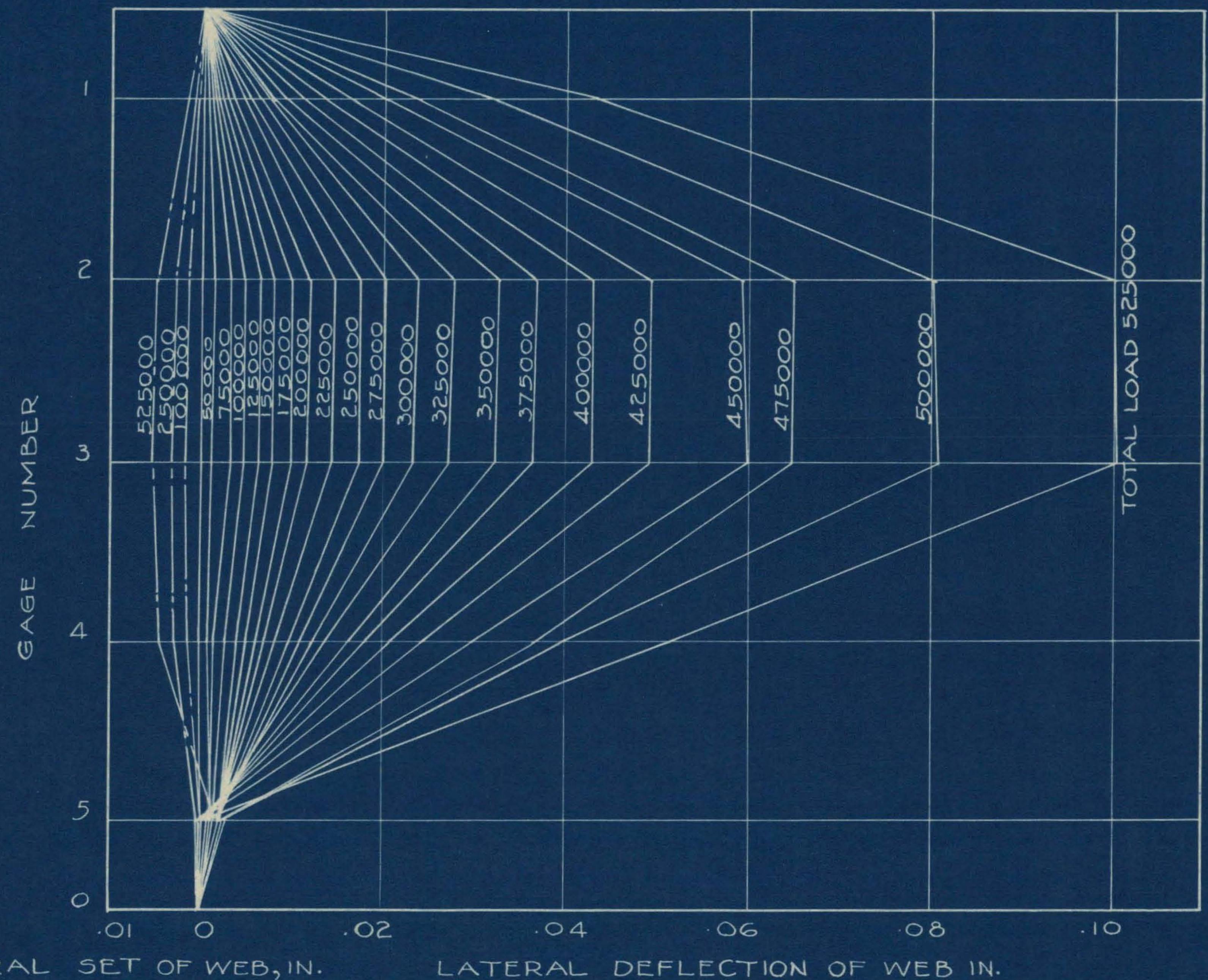

FIG. 31. LATERAL DEFLECTION AND SET OF WEB OF BEAM WB-4, A REINFORCED B-28 IN., 91-LB. BETHLEHEM, ULTIMATE LOAD 597,600 LBS. -TESTED - JUNE 30, 1932.

WB 4

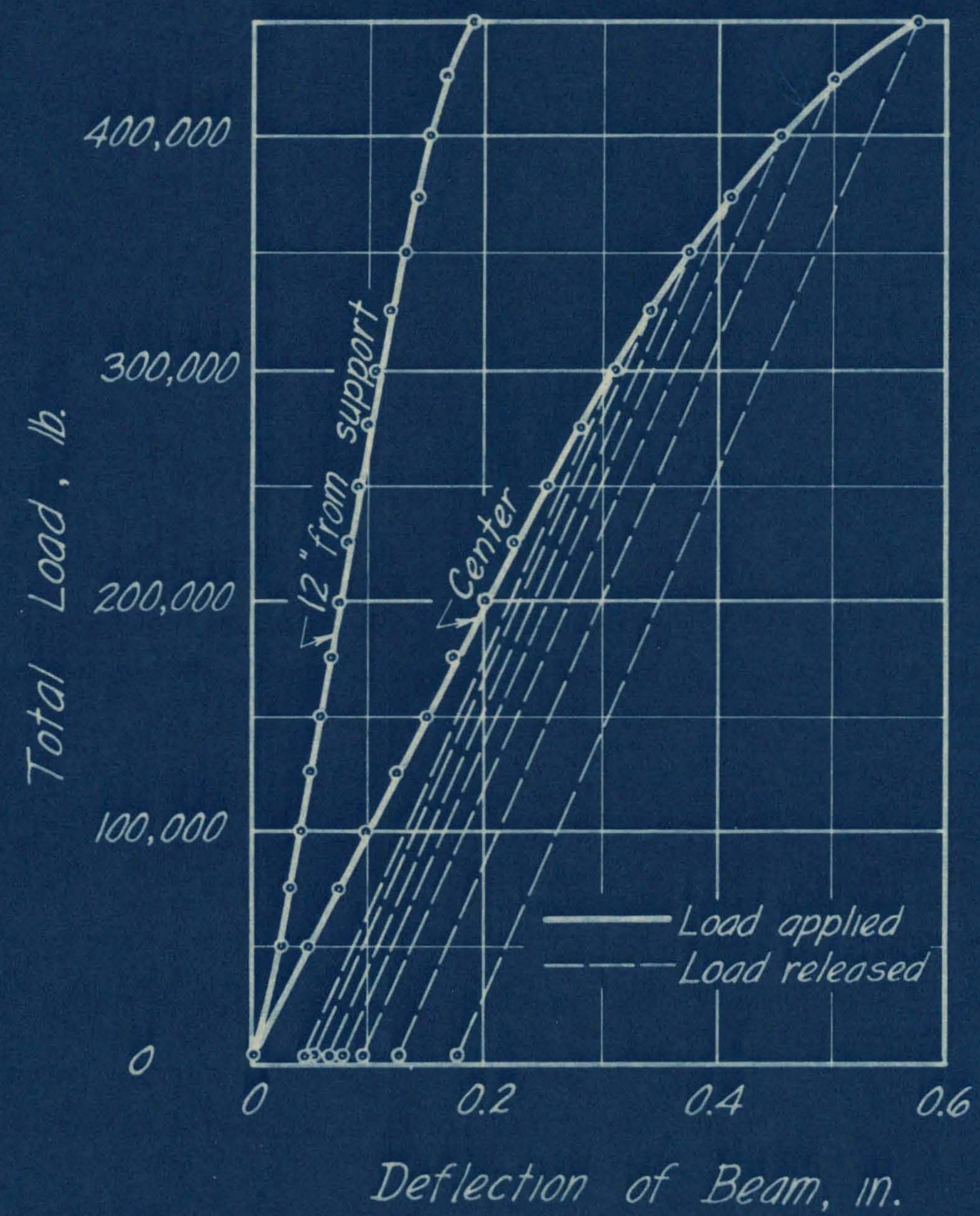

Fig. 32 Deflection of a Reinforced B-22 in, 62-lb. Bethlehem I-Beam No. WB-5.
Ultimate Load 450,000 lbs. Tested June 16, 1932.

WB-5

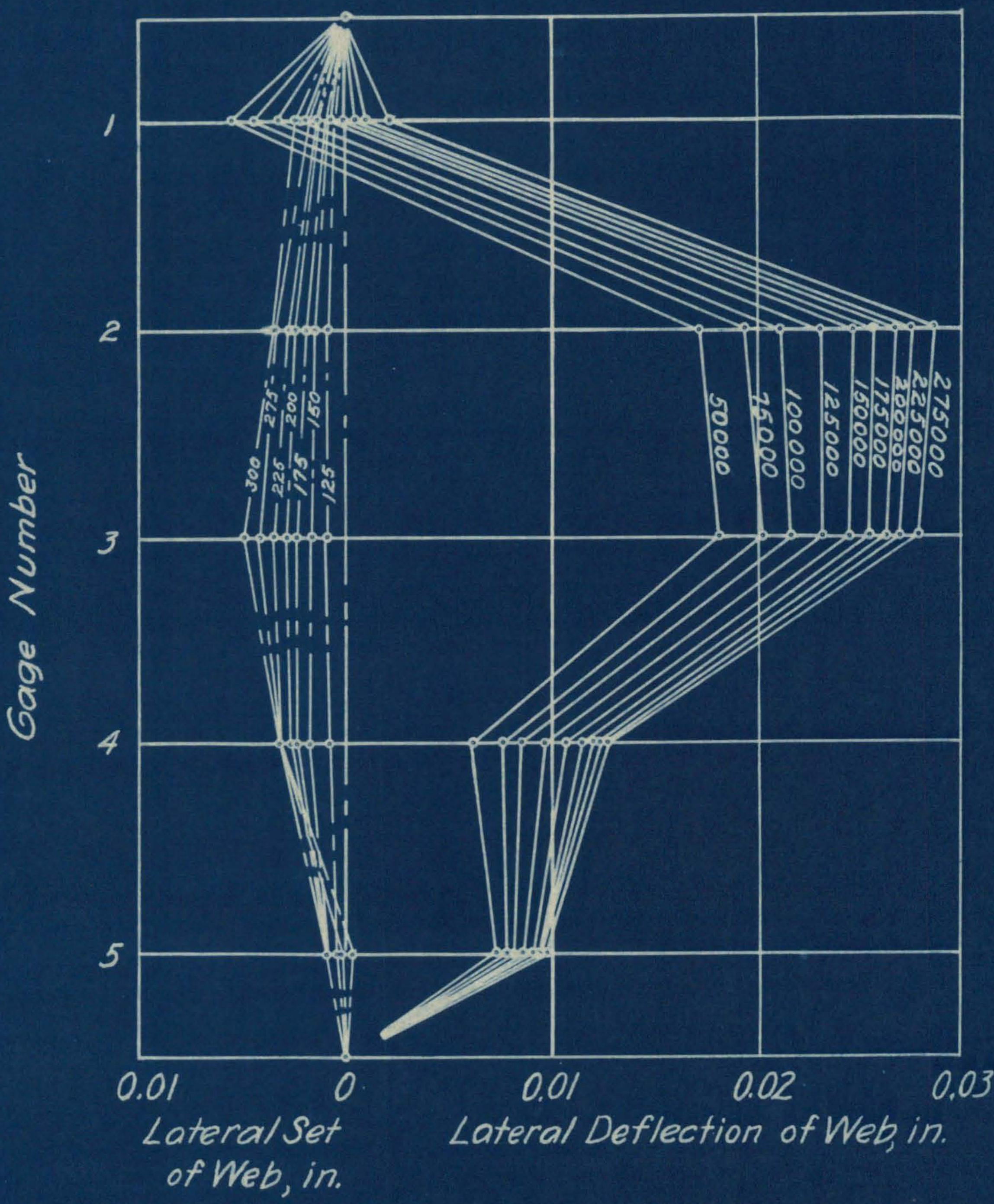

Fig. 33 Lateral Set and Deflection of Web of a Reinforced B-22 in., 62 lb Bethlehem I-Beam
Ultimate Load = 450,000 lbs. Tested June 16, 1932.

WB-5

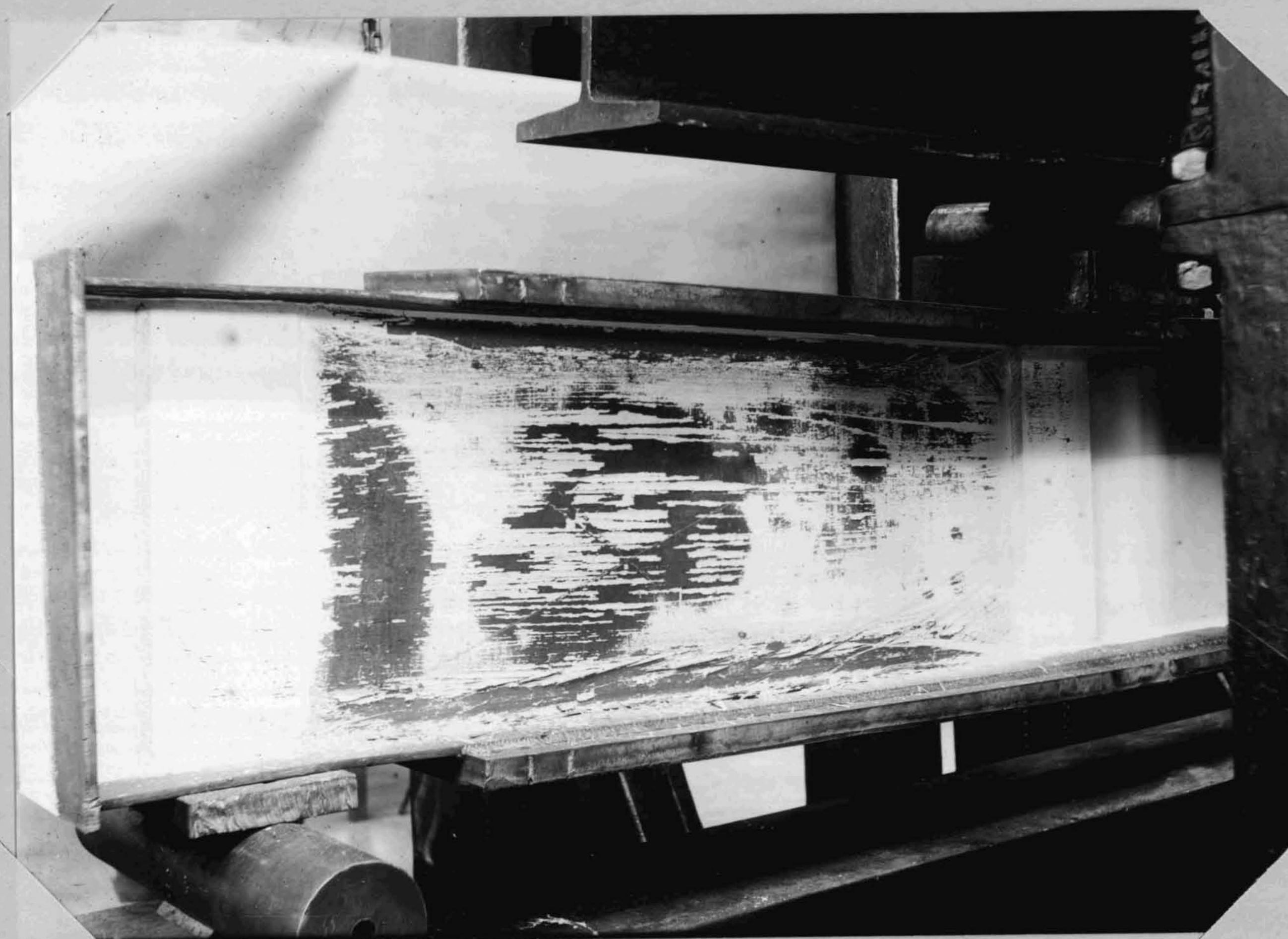

Fig. 34 - Condition of Beam WB-5, Bethlehem B22-62 Section

at Ultimate Load

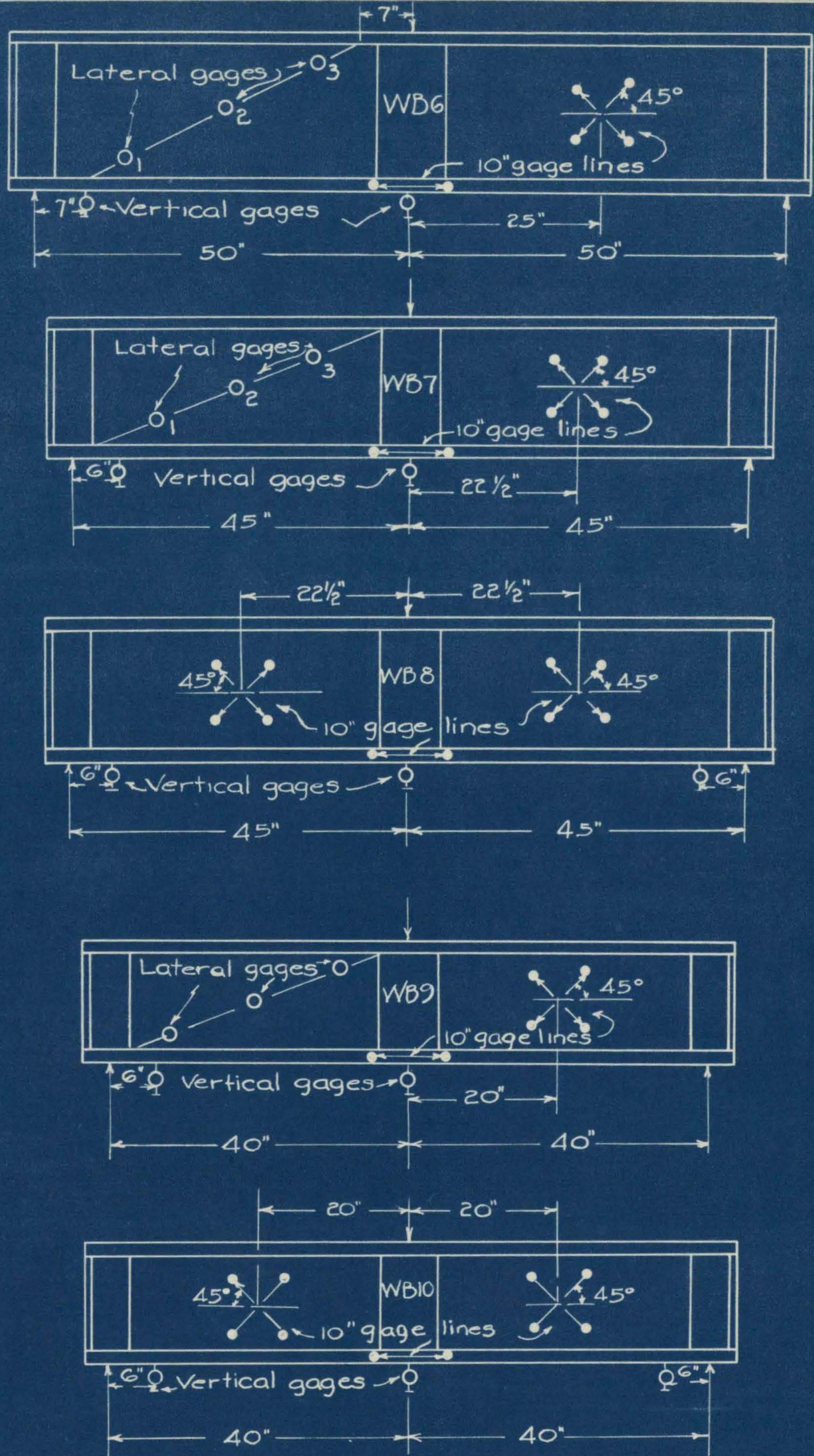

FIG. 35. MAKE-UP AND LOADING ARRANGEMENT FOR MAJOR INVESTIGATION - GROUP B - BEAMS WB-6 - WB-10.

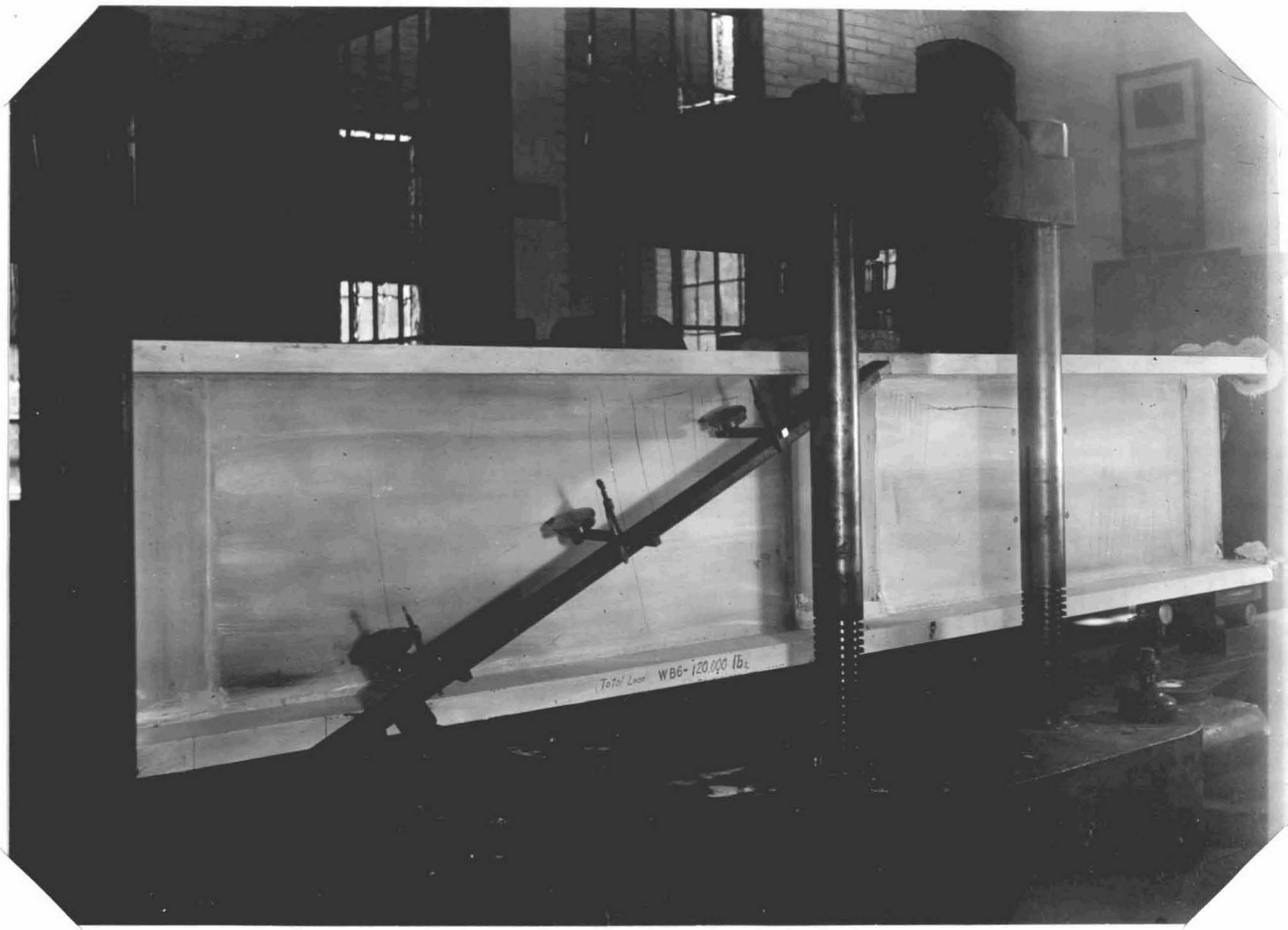

Fig. 36 - Appearance of Beam WB-6 Under Load of 120,000 lb.

Tested November 30, 1932

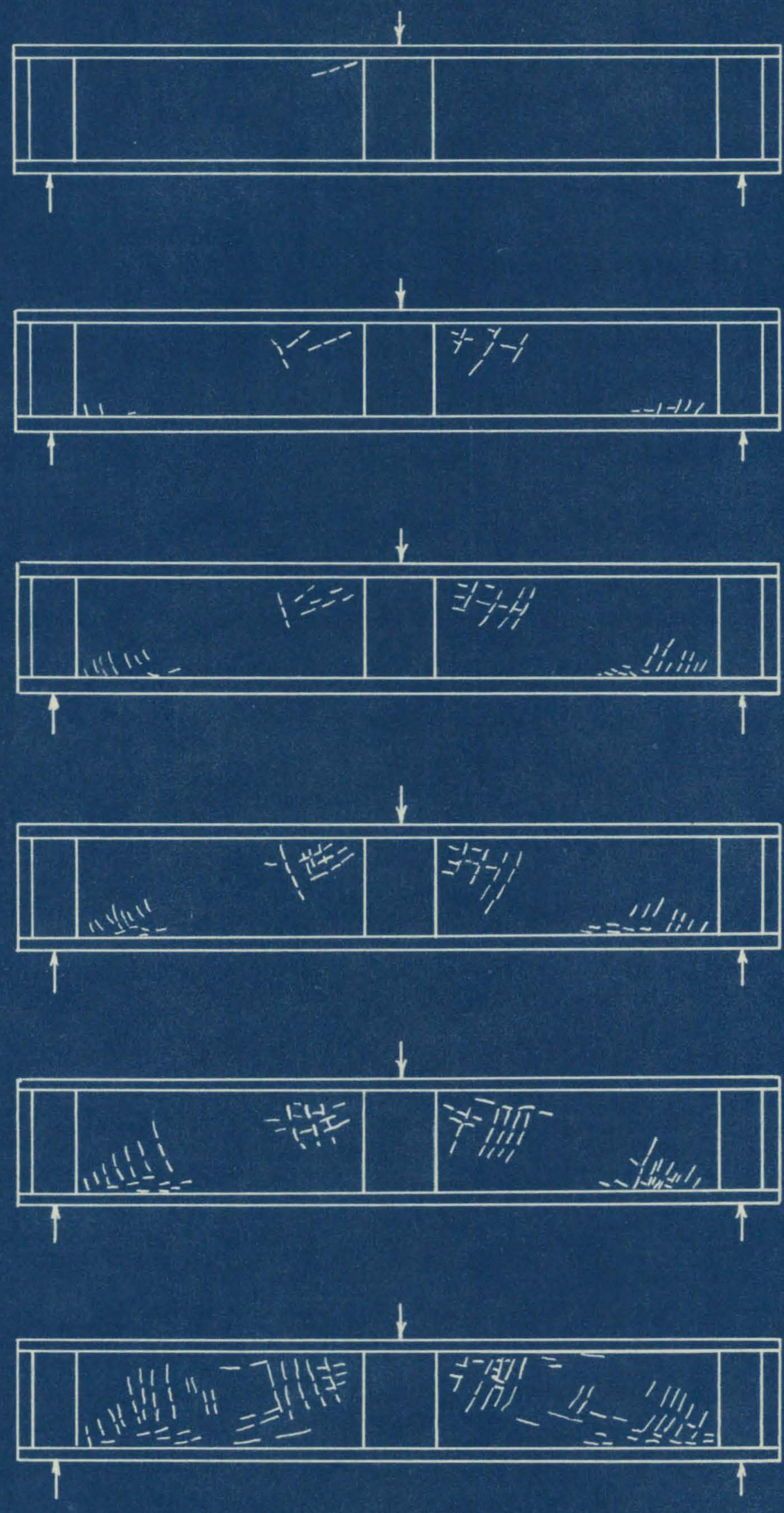

FIG. 37. GENERAL FORMATION OF STRAIN LINES ON WEB OF BEAMS.

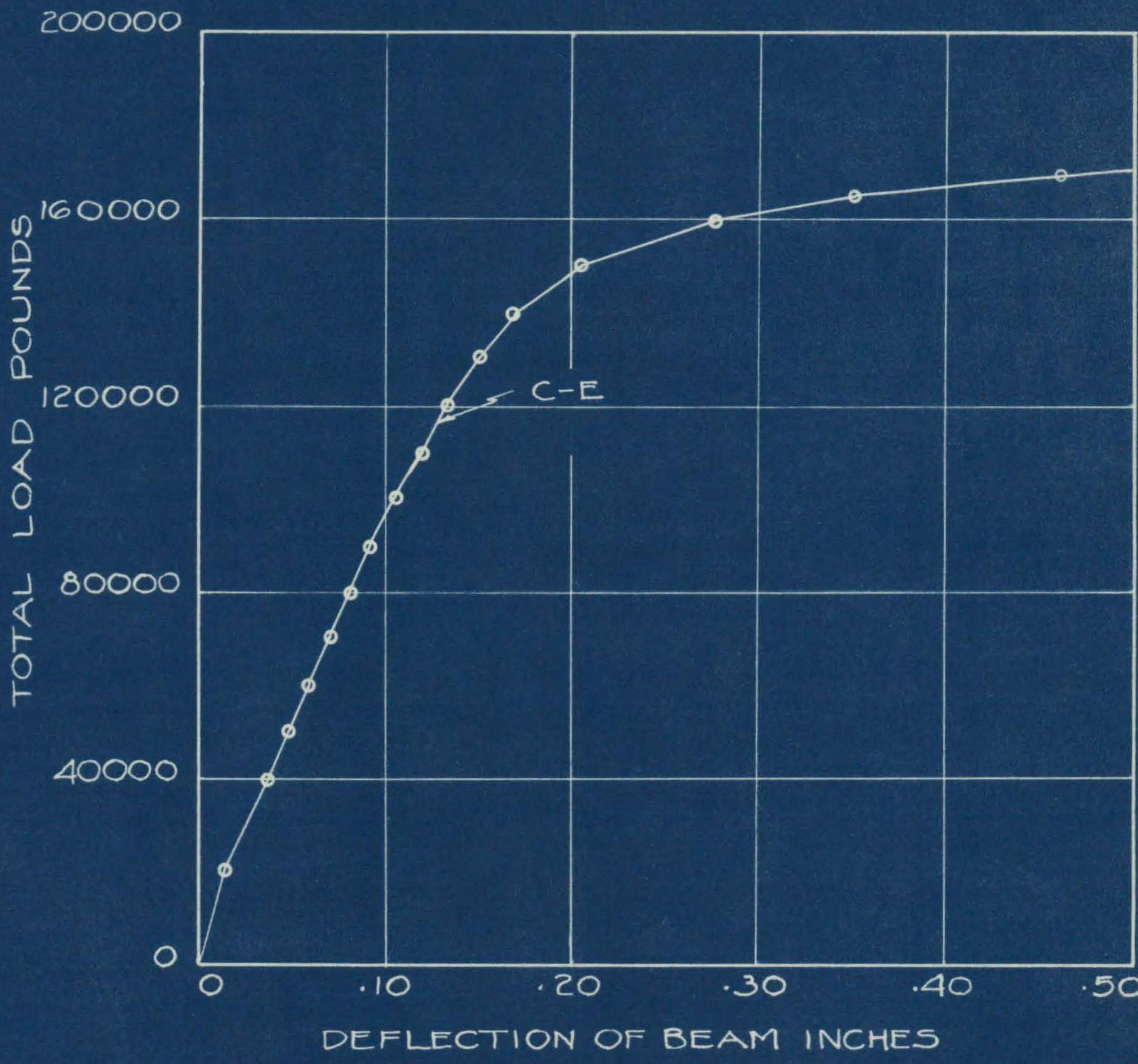

FIG. 38. NET CENTER DEFLECTION OF BEAM WB-6
ULTIMATE LOAD 192,600 LBS. TESTED NOV. 30, 1932.

WB-6

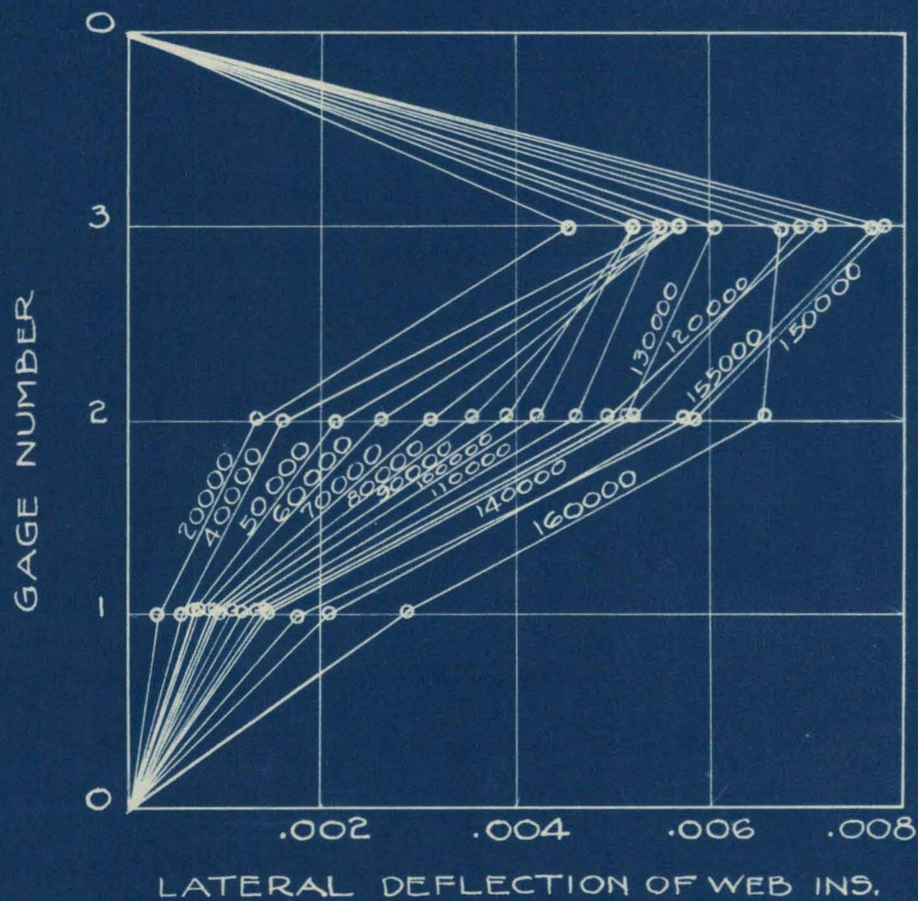

FIG. 39. LATERAL DEFLECTION OF WEB OF BEAM WB-6
ULTIMATE LOAD 192,600 LBS. TESTED NOV. 30, 1932.

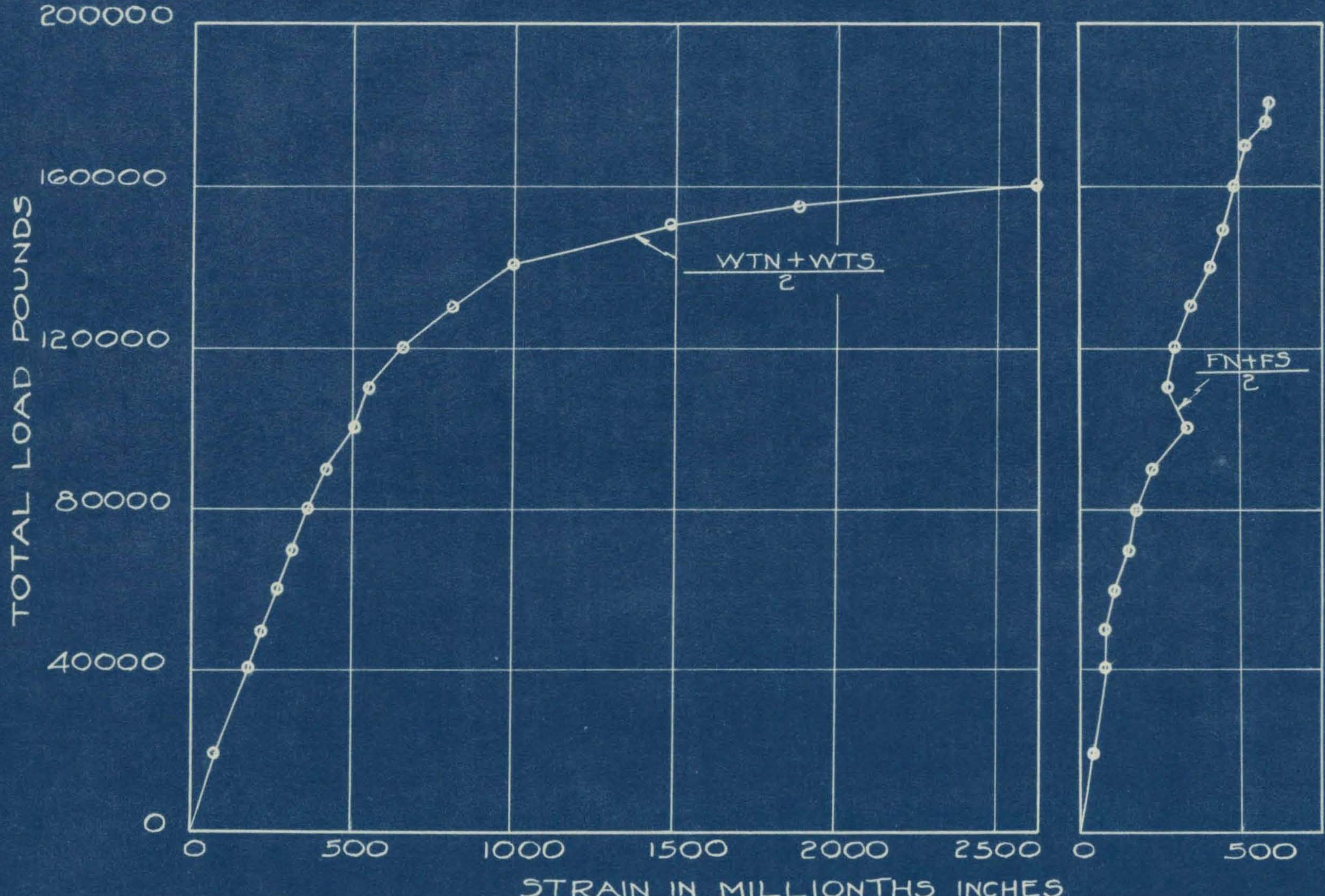

FIG. 40. AVERAGE TENSION STRAINS IN WEB AND IN FLANGE OF BEAM WB-6 - TESTED NOV. 30, 1932

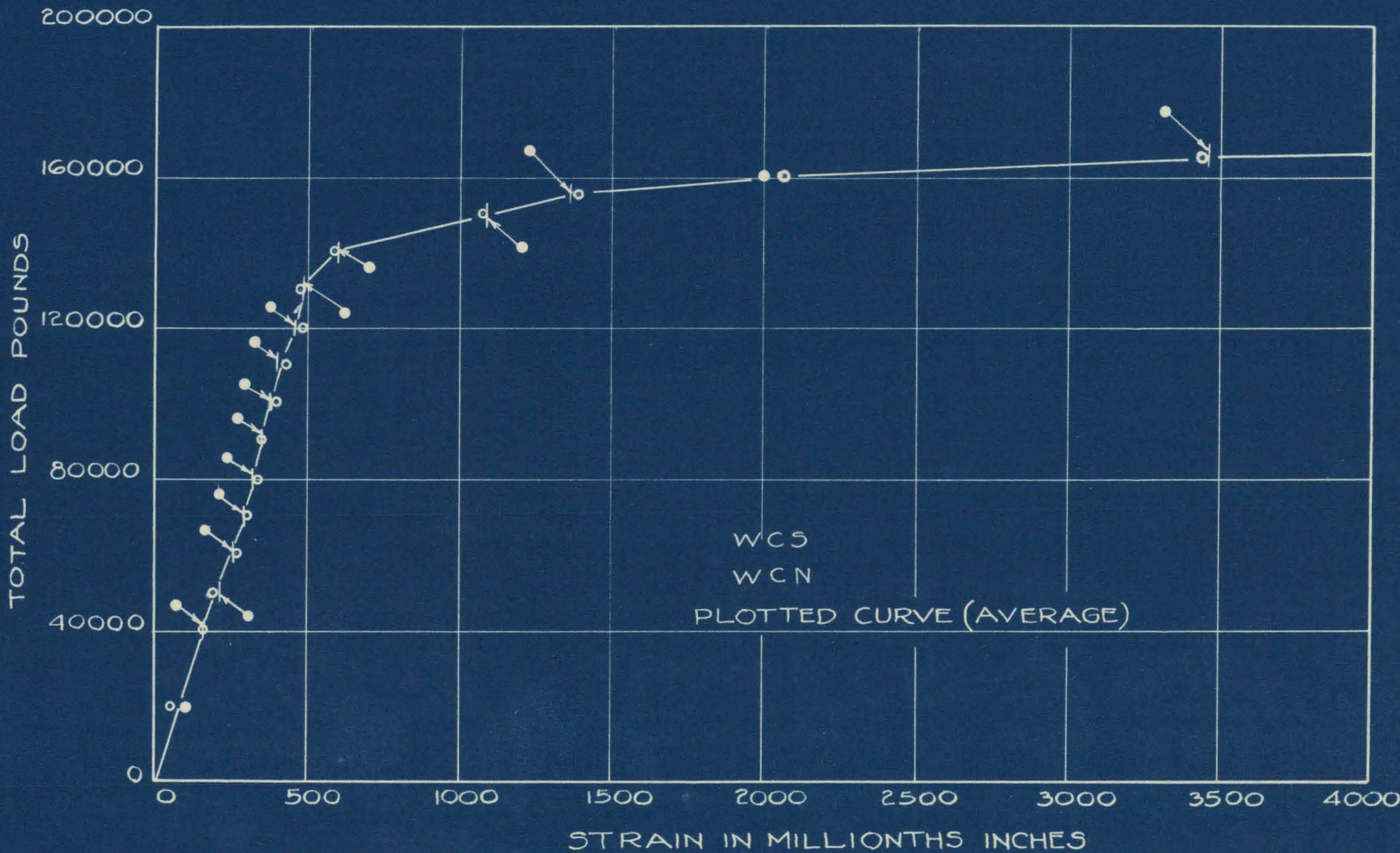

FIG. 41. COMPRESSIVE STRAINS ON BOTH SIDES OF WEB OF BEAM WB-6
TESTED NOV. 30, 1932.

WB-6

Fig. 42 - Appearance of Beam WB-6 at the Maximum Load

of 192,600 lb. Tested November 30, 1932

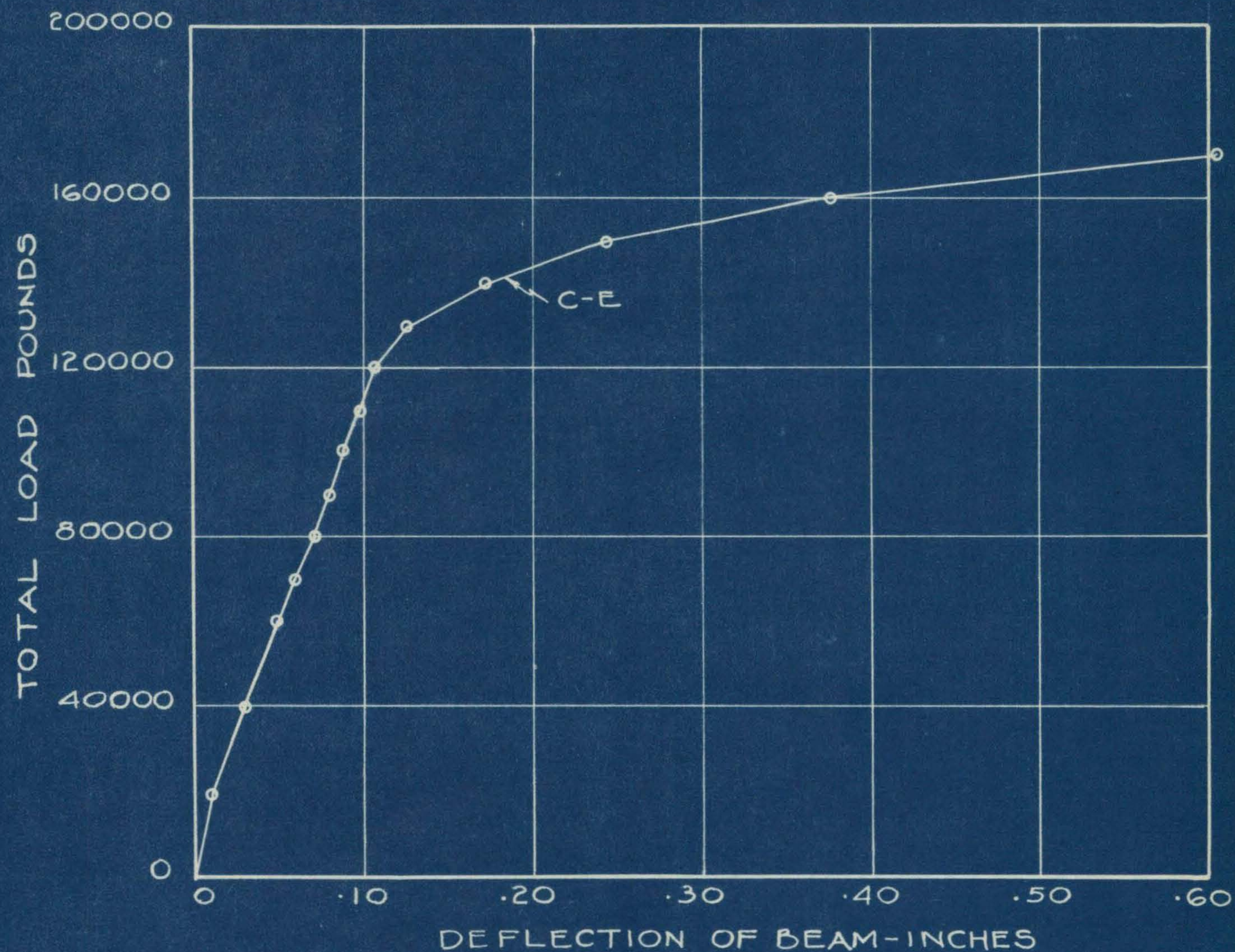

FIG. 43. NET CENTER DEFLECTION OF BEAM WB-7
ULTIMATE LOAD 190,100 LBS. TESTED DEC. 1, 1932

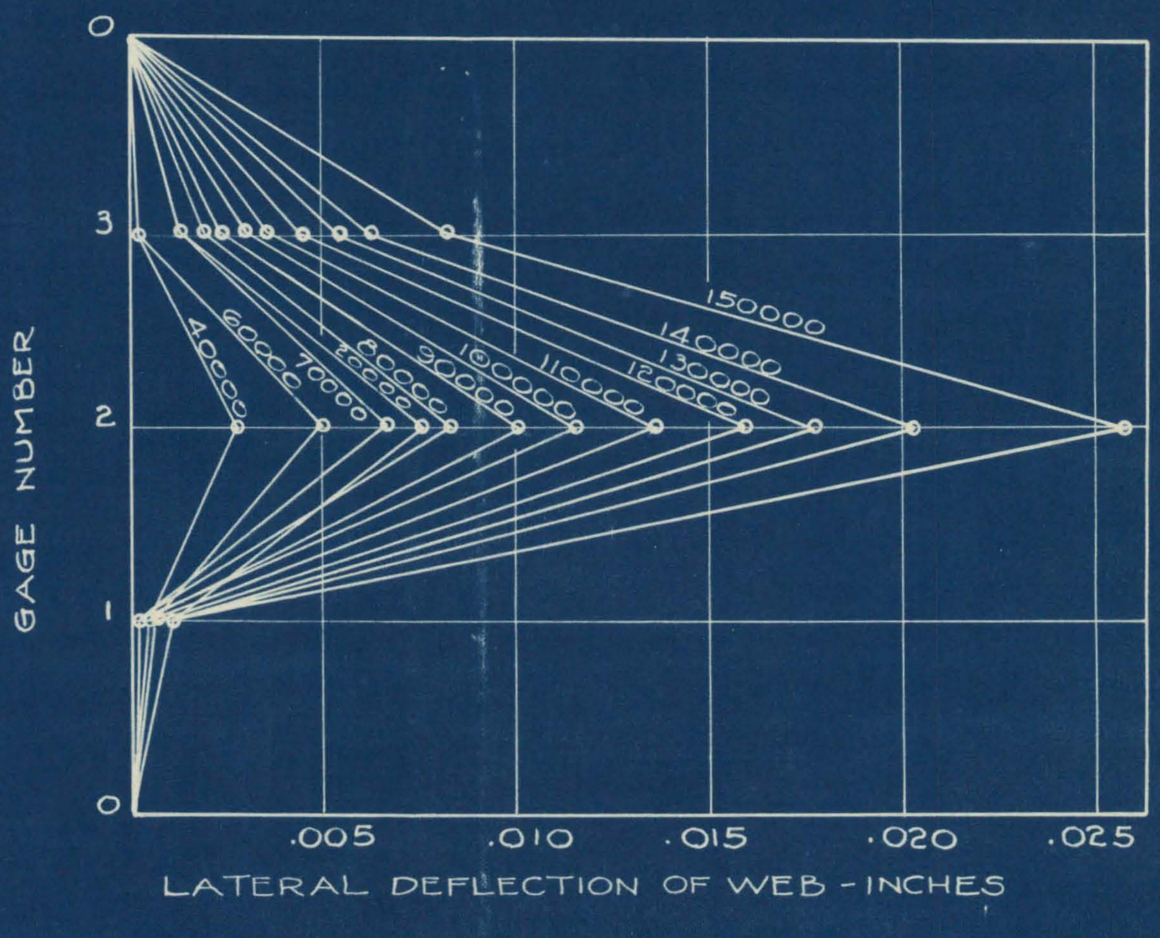

FIG. 44. LATERAL DEFLECTION OF WEB OF BEAM WB-7
ULTIMATE LOAD 190,100 LBS. TESTED DEC. 1, 1932

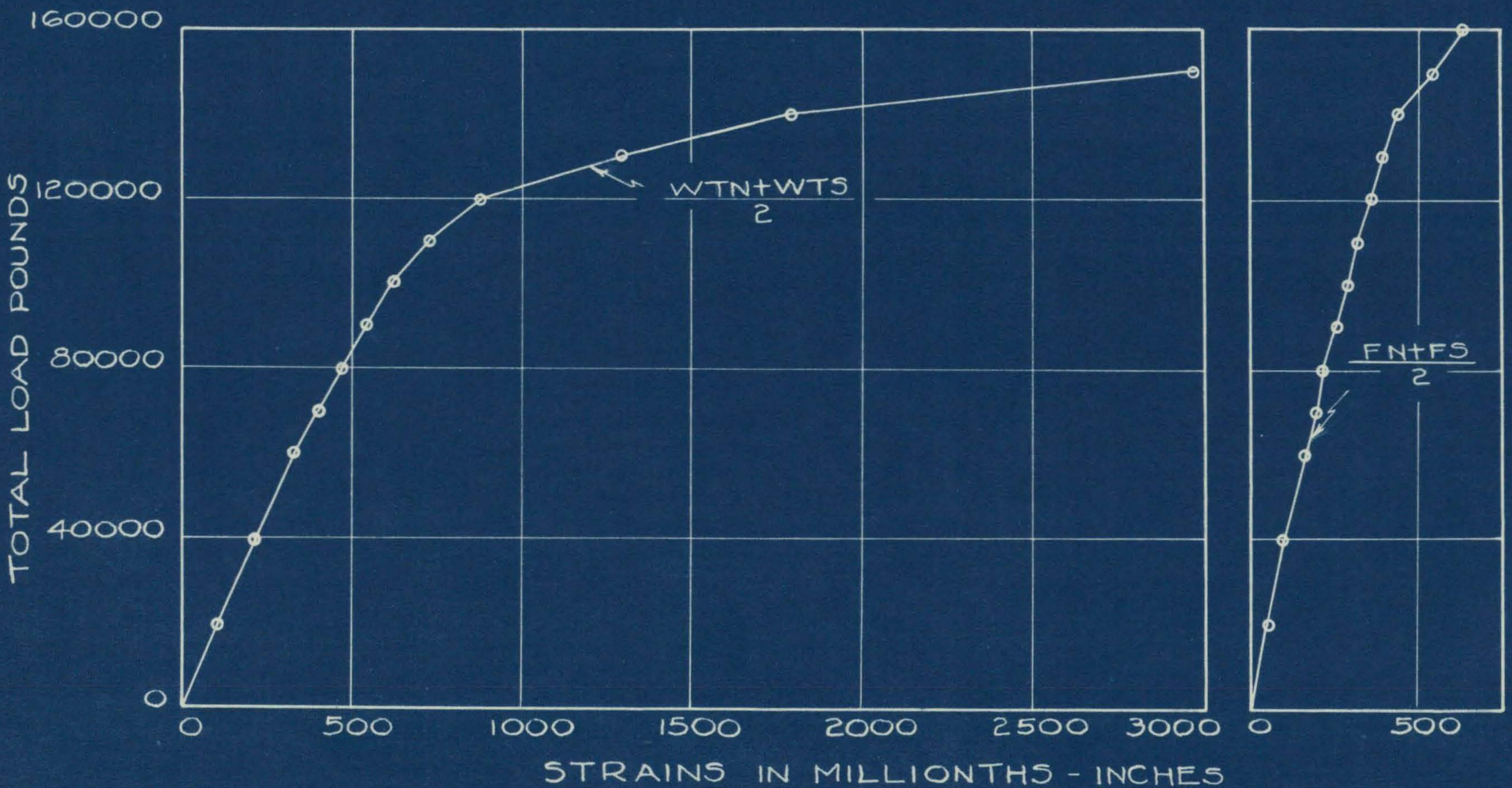

FIG. 45. AVERAGE TENSION STRAINS IN WEB AND FLANGE OF BEAM WB-7

TESTED DEC. 1, 1932

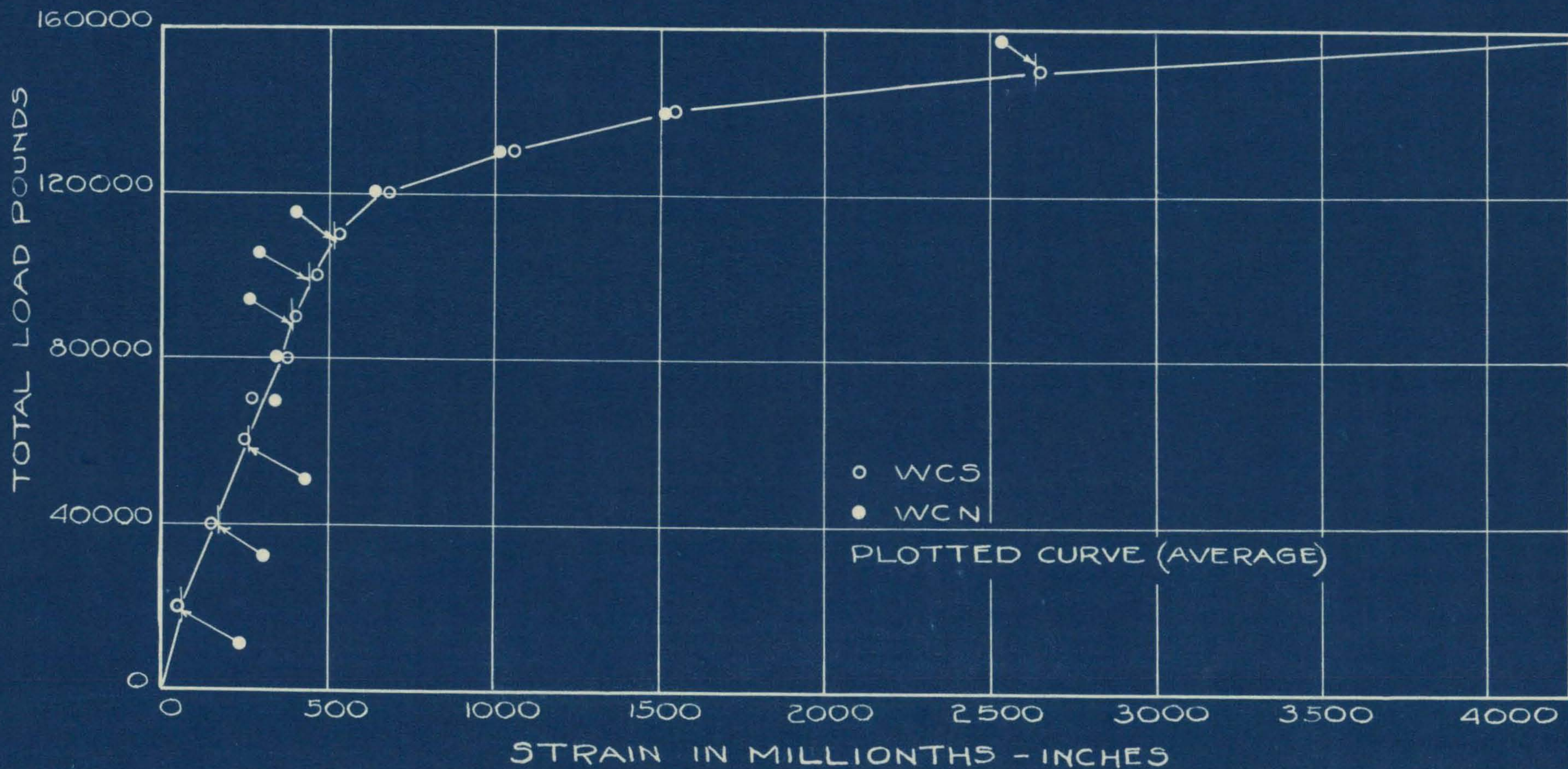

FIG. 46. COMPRESSIVE STRAINS ON BOTH SIDES OF THE WEB OF BEAM WB-7
TESTED DEC. 1, 1932

WB-7

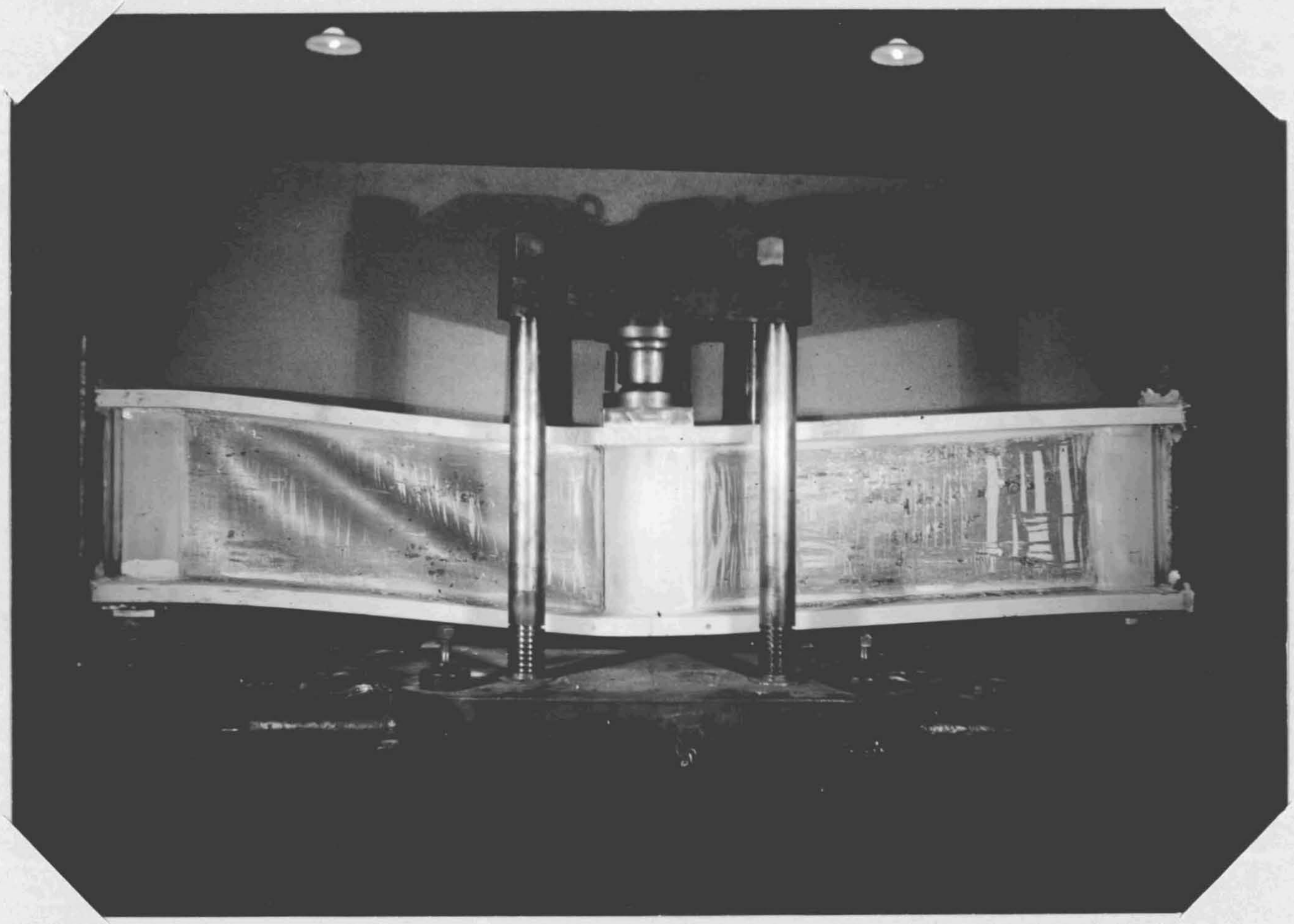

Fig. 47 - Appearance of Beam WB-7 at the Maximum Load
of 190,100 lb. Tested December 1, 1932

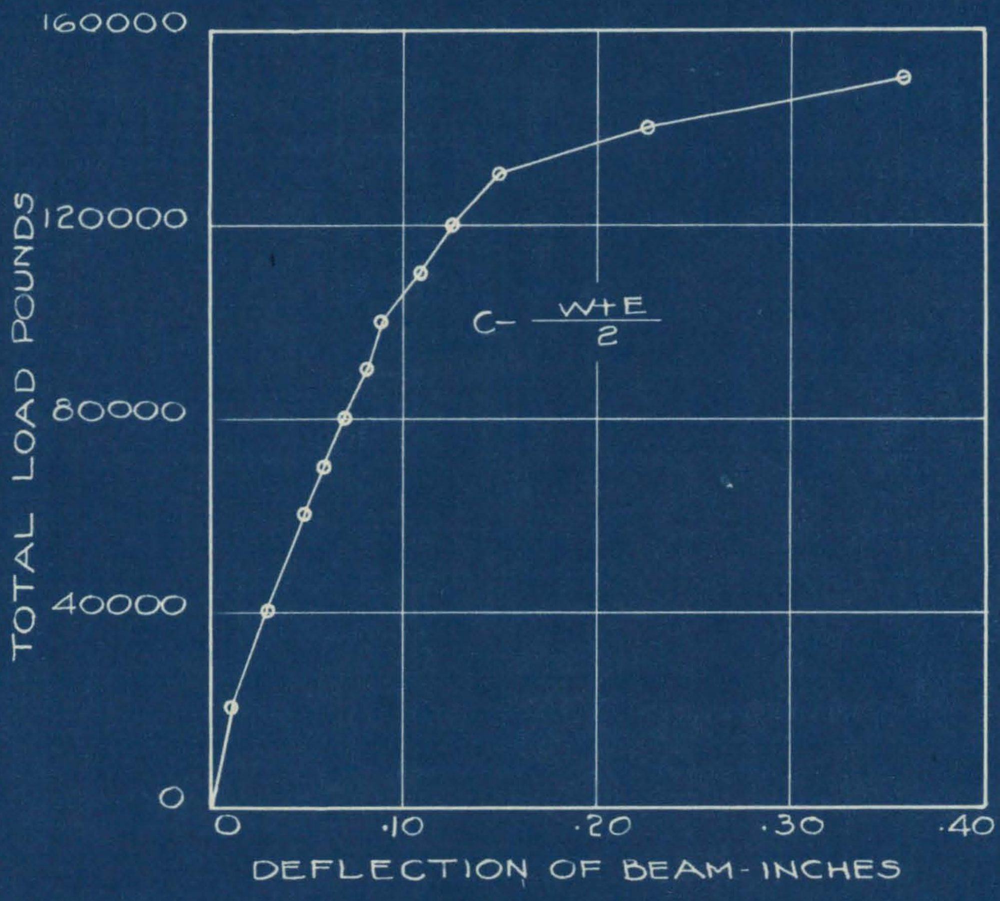

FIG. 48. NET CENTER DEFLECTION OF BEAM WB-8
ULTIMATE LOAD 199,500 LBS. TESTED DEC. 2, 1932

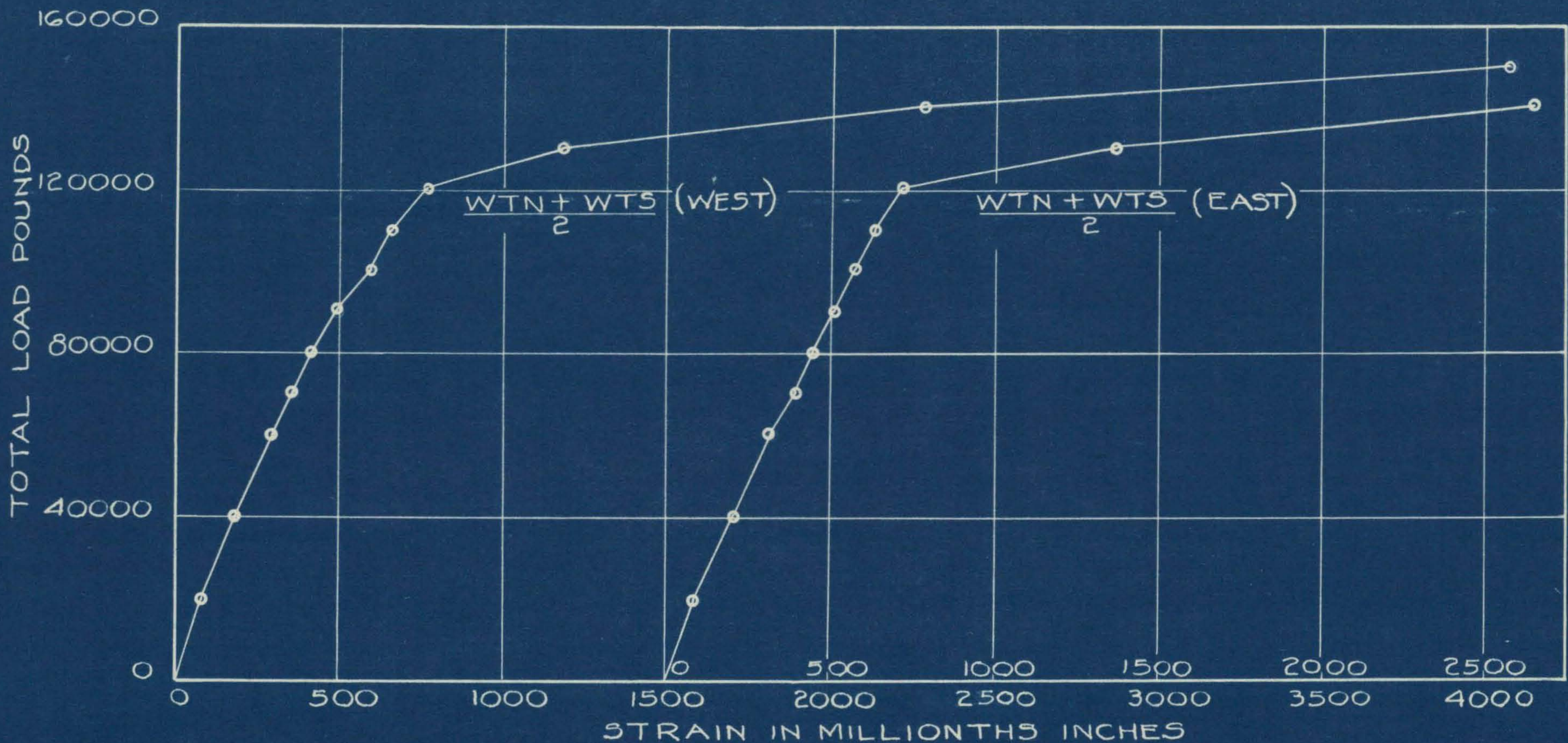

FIG. 49. - AVERAGE TENSION STRAINS IN WEB OF BOTH PANELS OF BEAM WB-8

TESTED DEC. 2, 1932

WB-8

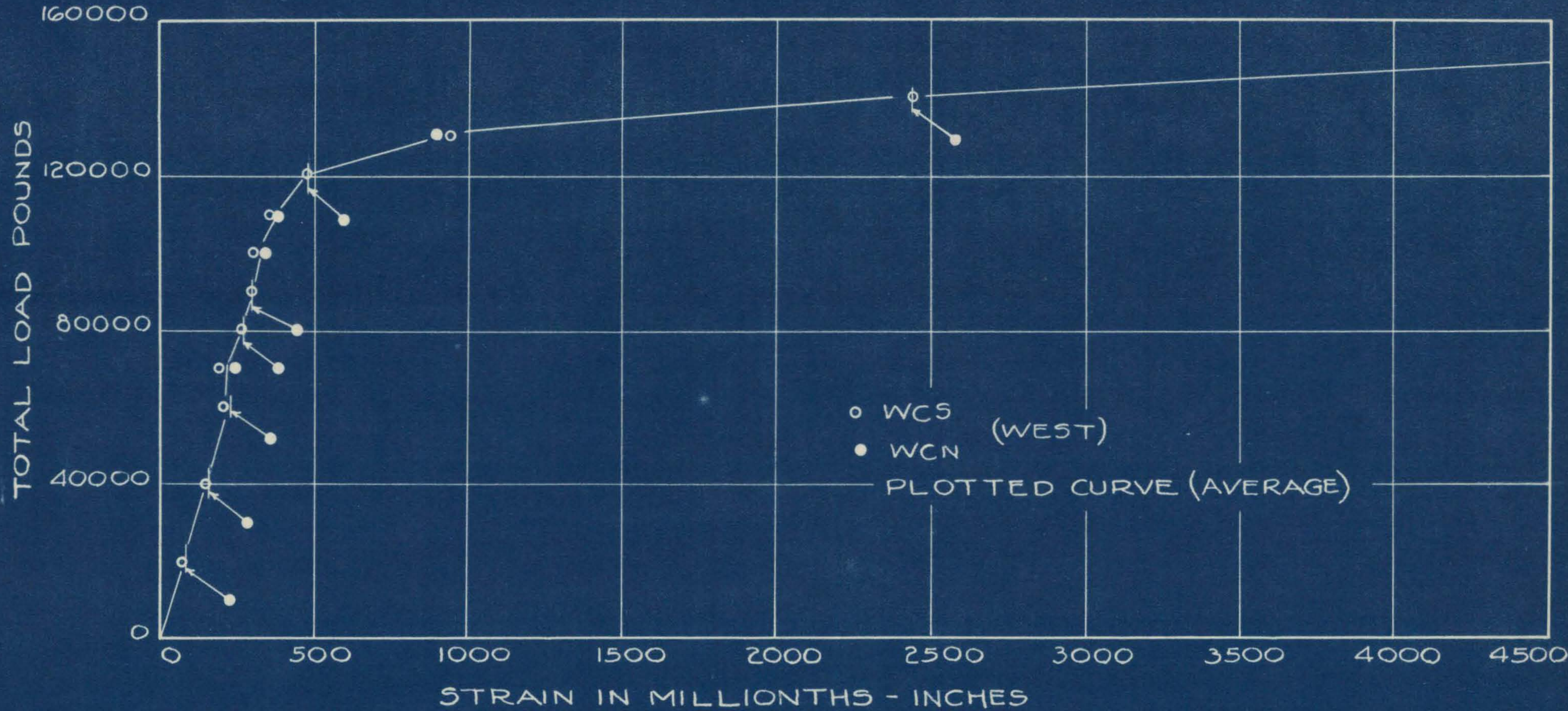

FIG 50 COMPRESSIVE STRAINS ON BOTH SIDES OF THE WEB OF BEAM WB-8

TESTED DEC. 2, 1932

WB-8

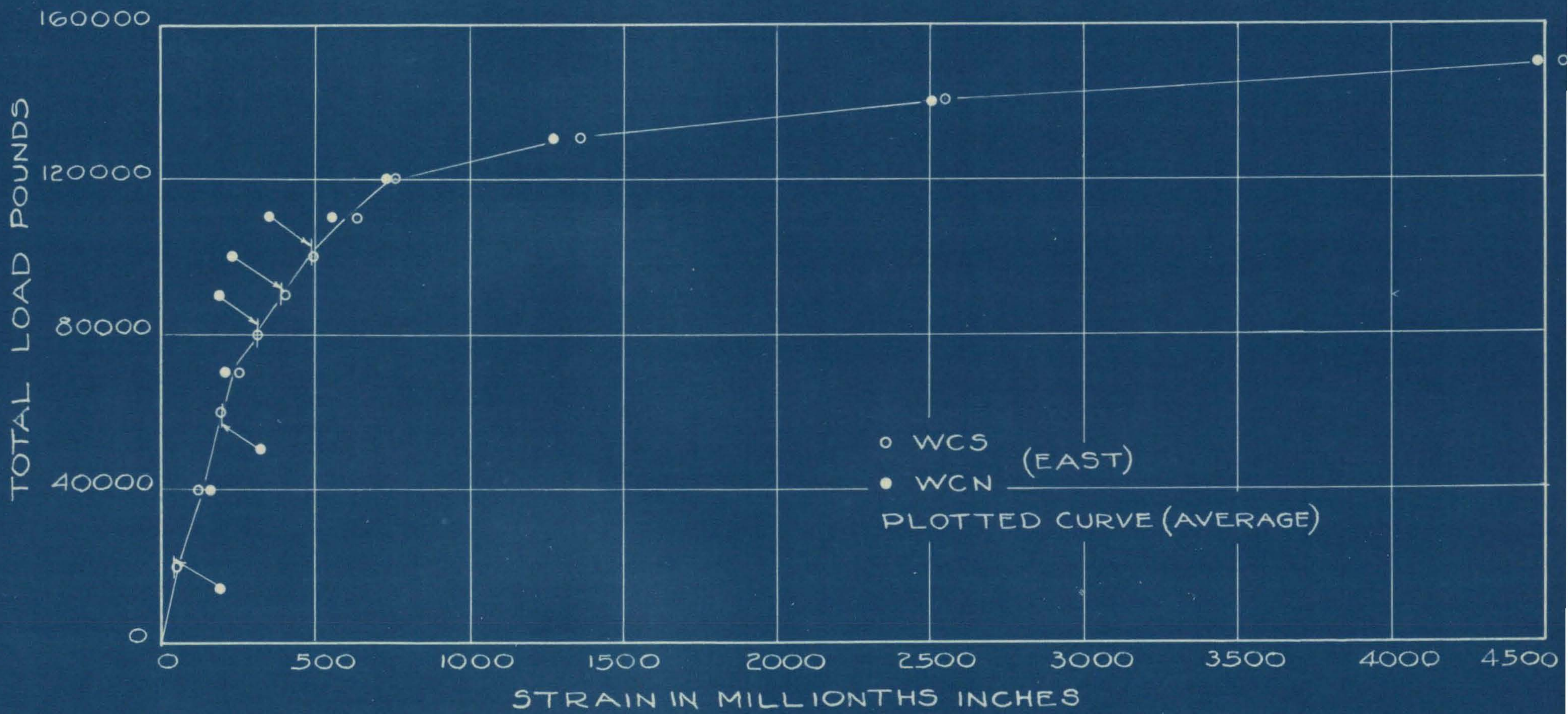

FIG. 51. COMPRESSIVE STRAINS ON BOTH SIDES OF THE WEB OF BEAM WB-8

TESTED DEC. 2, 1932

WB 8

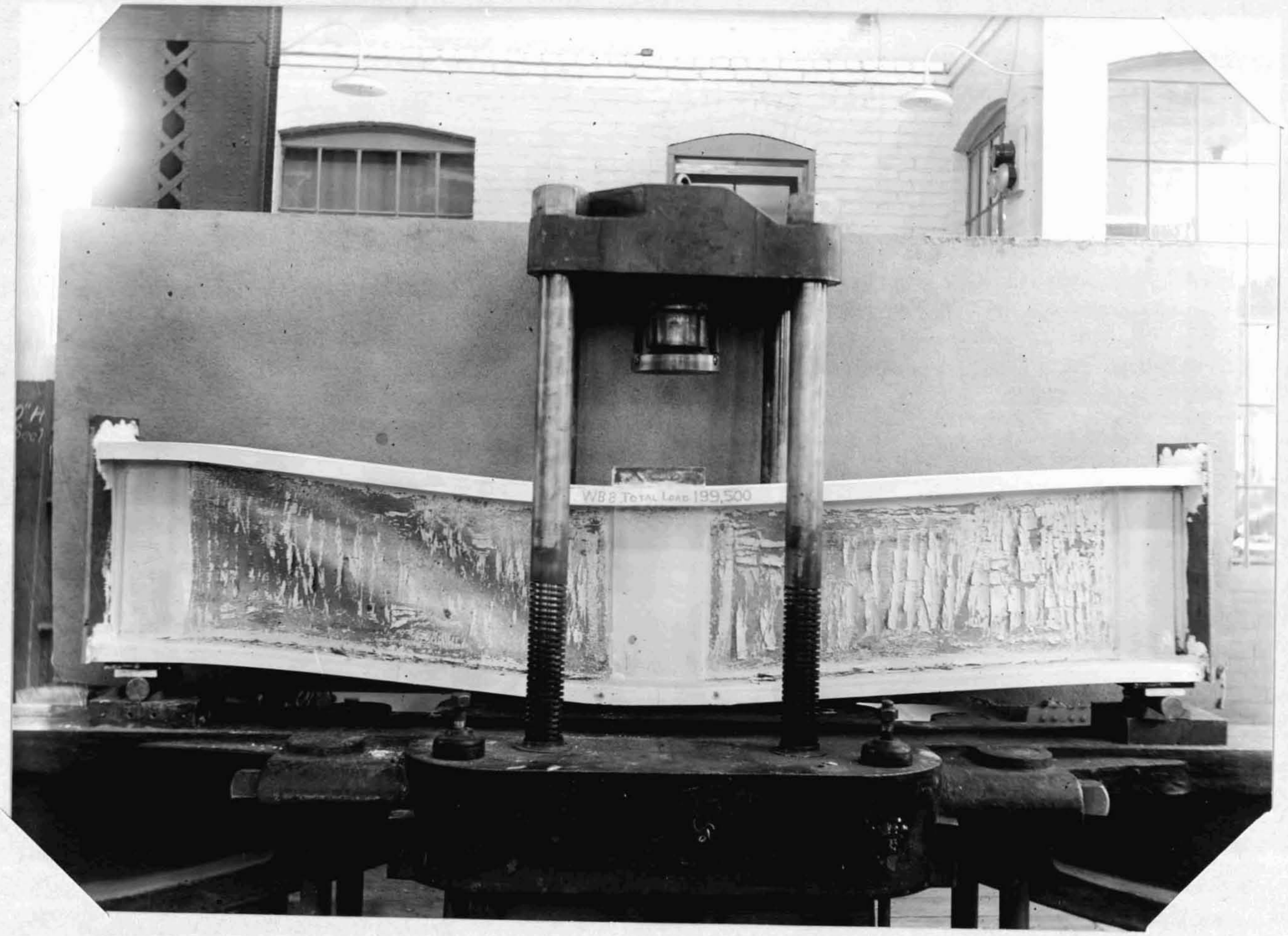

Fig. 52 - Appearance of Bean WB-8 at the Maximum Load of 199,500 lb. Tested December 2, 1932

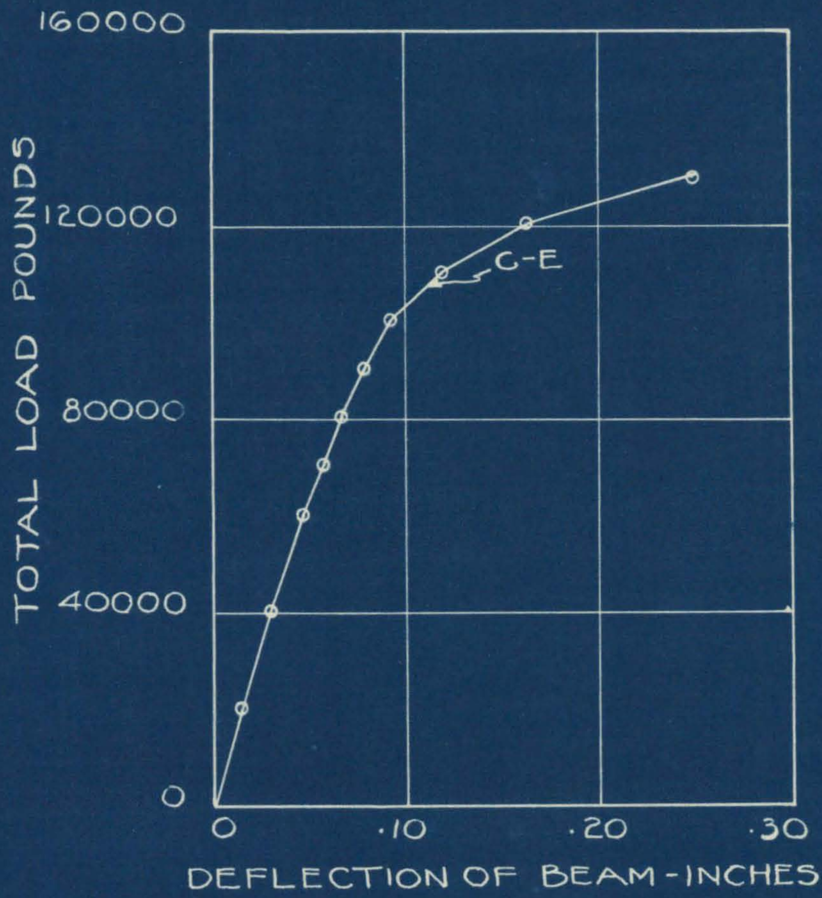

FIG. 53. NET CENTER DEFLECTION OF BEAM WB-9
ULTIMATE LOAD - 184,000. TESTED DEC. 1, 1932.

WB-9

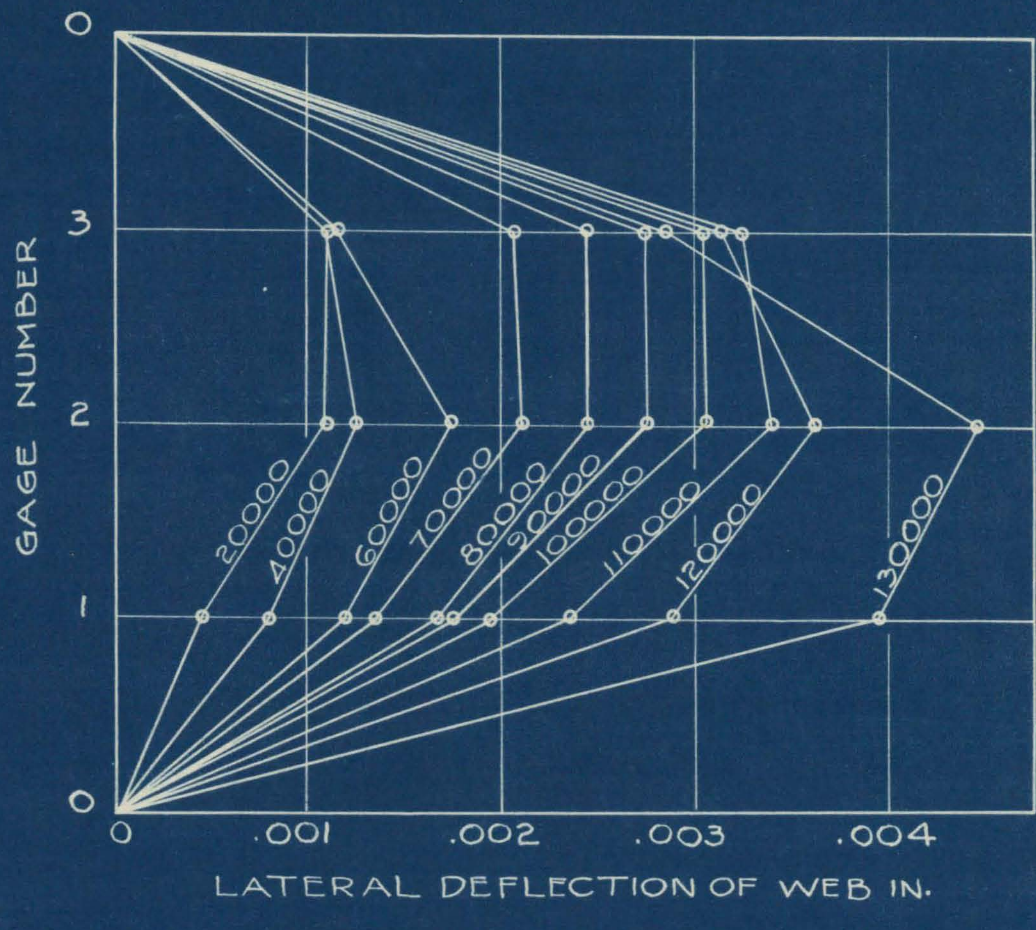

FIG. 54. LATERAL DEFLECTION OF WEB OF BEAM WB-9
ULTIMATE LOAD 184,000 LB. TESTED DEC. 1, 1932.

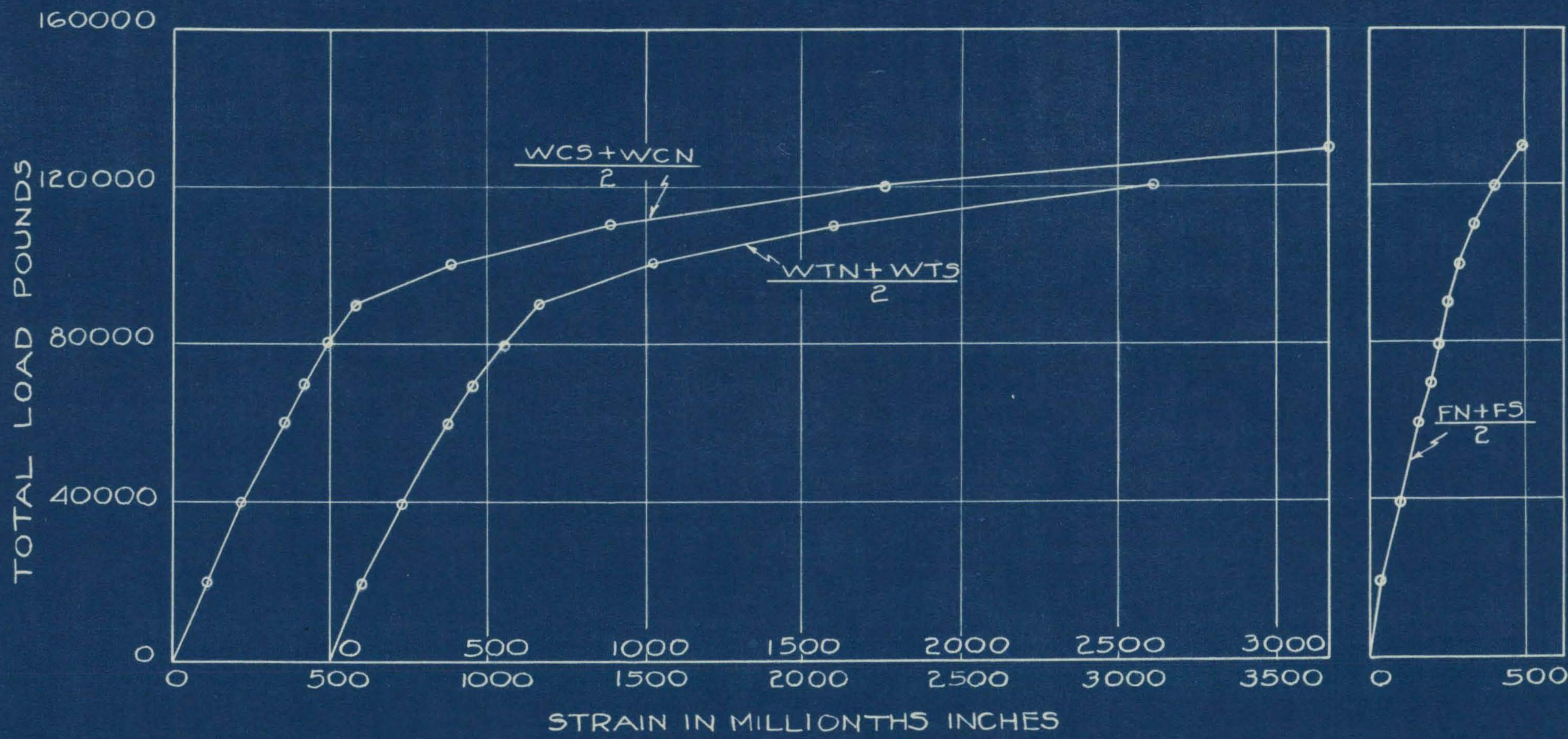

FIG. 55. AVERAGE COMPRESSION AND TENSION STRAINS IN WEB AND TENSION STRAINS IN FLANGE OF BEAM WB-9 — TESTED DEC. 1, 1932.

WB-9

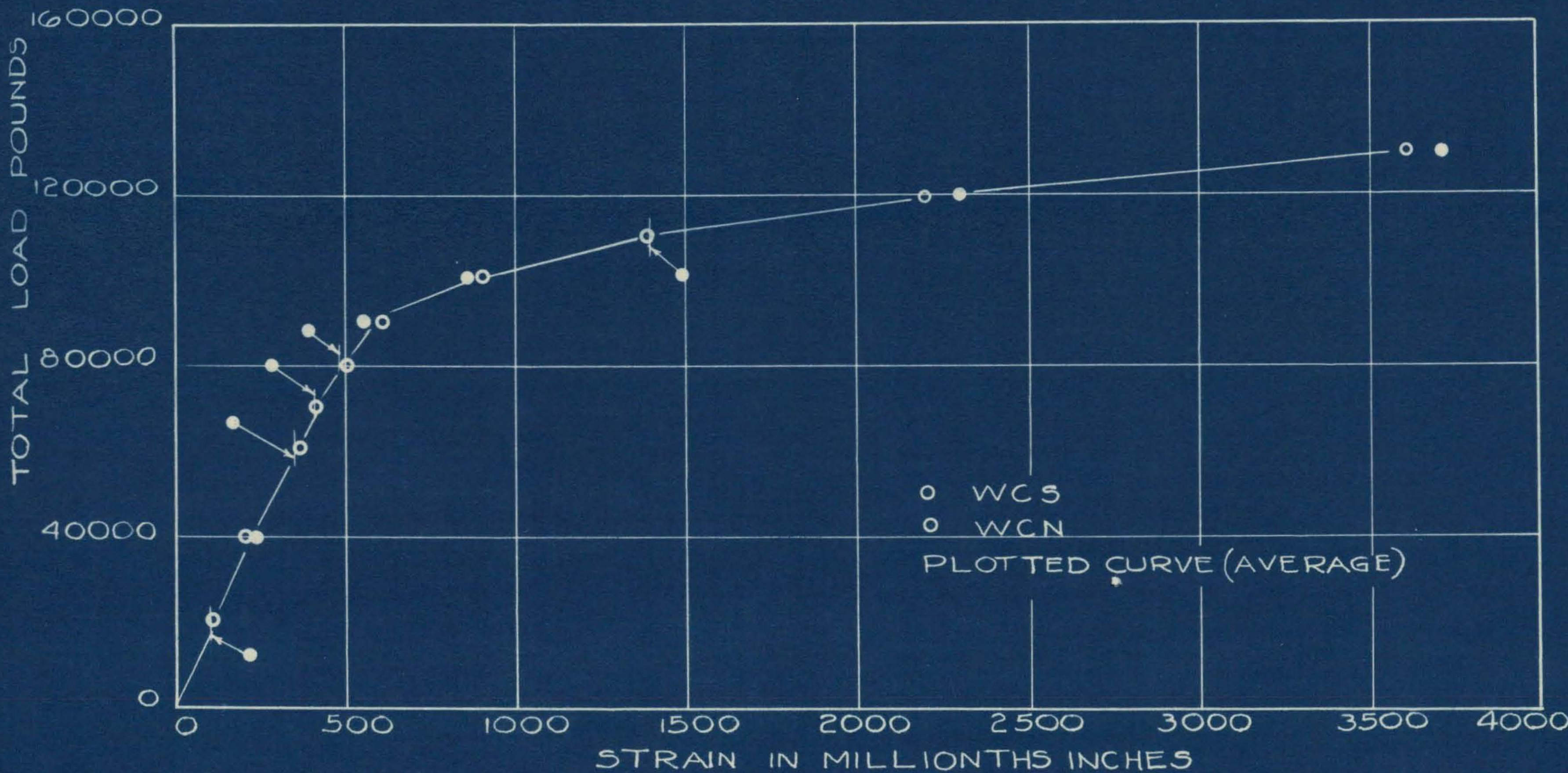

FIG. 56. COMPRESSION STRAINS ON BOTH SIDES OF WEB OF BEAM WB-9

WB-9.

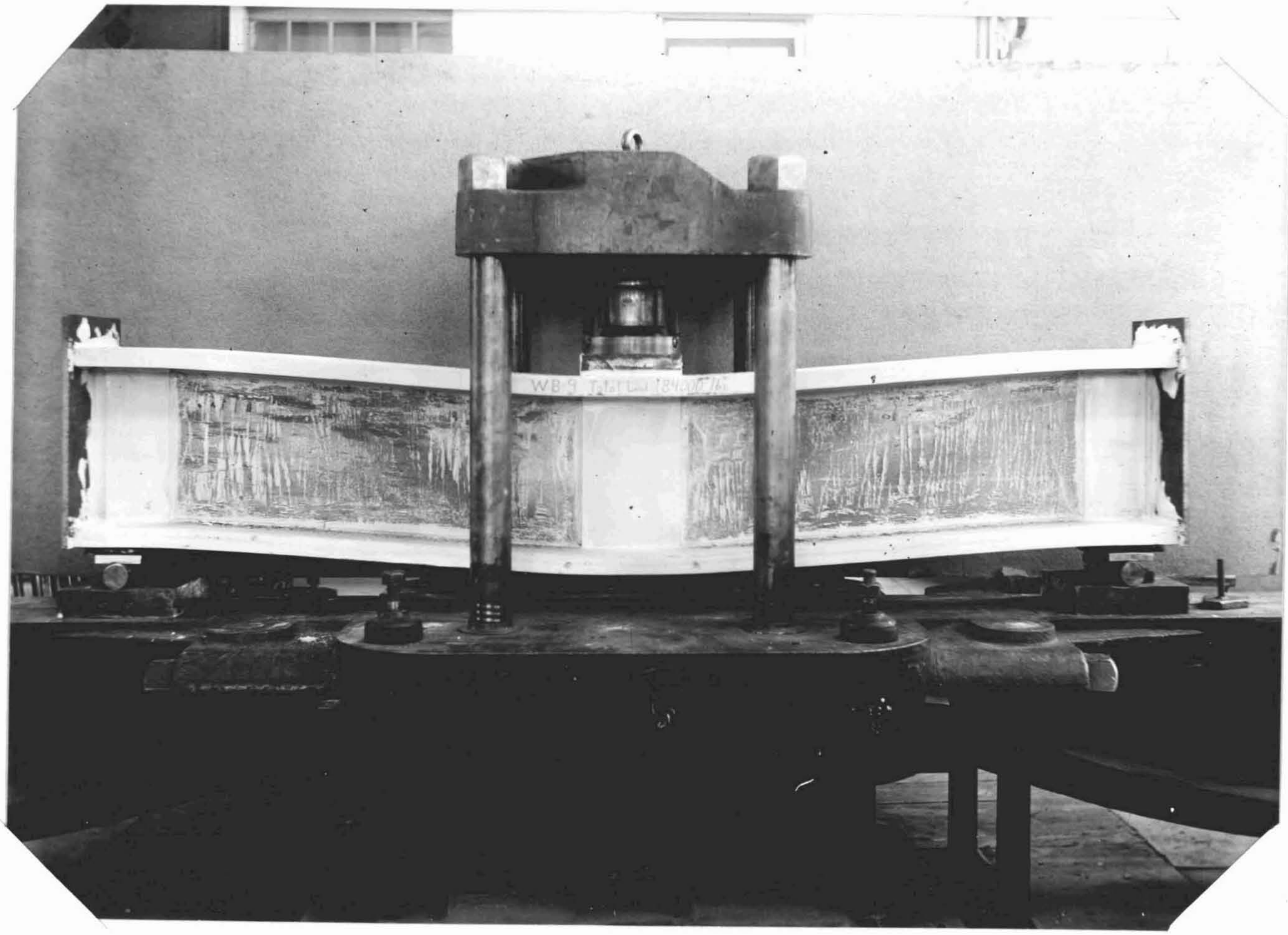

Fig. 57 - Appearance of Bean WB-9 at the Maximum Load of 184,000 lb. Tested December 1, 1932

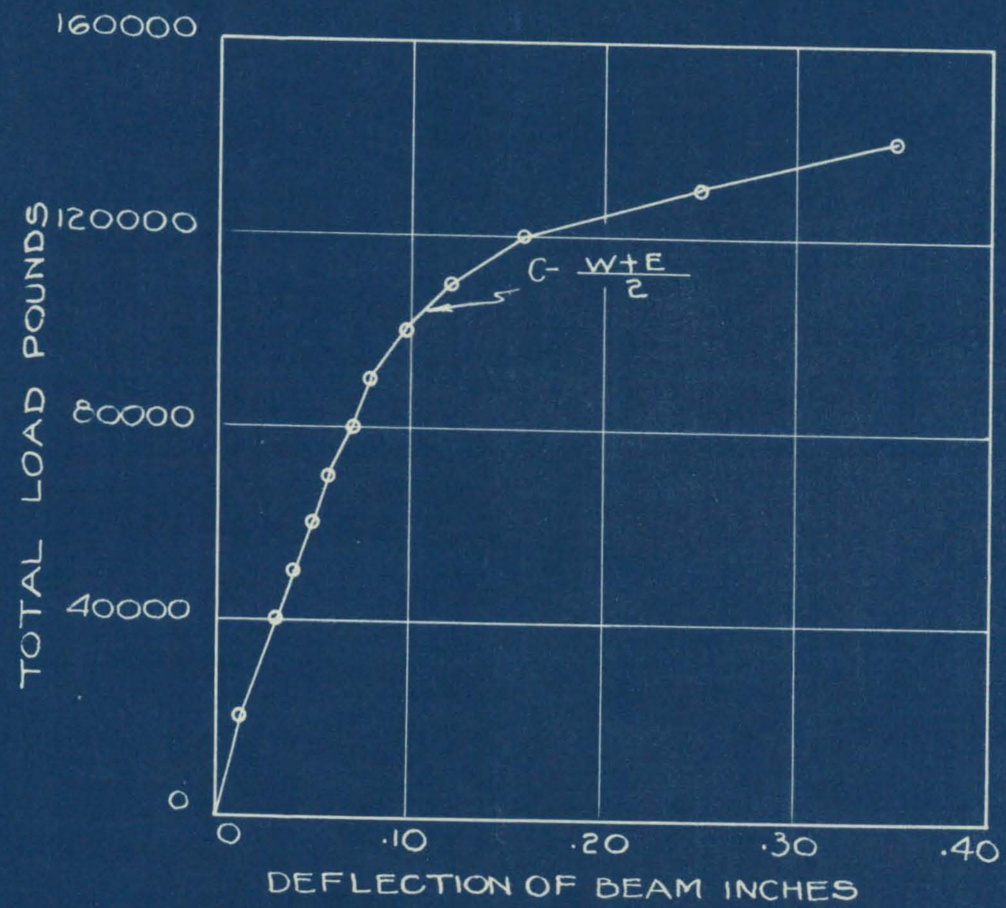

FIG. 58 - NET CENTER DEFLECTION OF BEAM WB-10
ULTIMATE LOAD 188,000 LBS. - TESTED DEC. 2, 1932

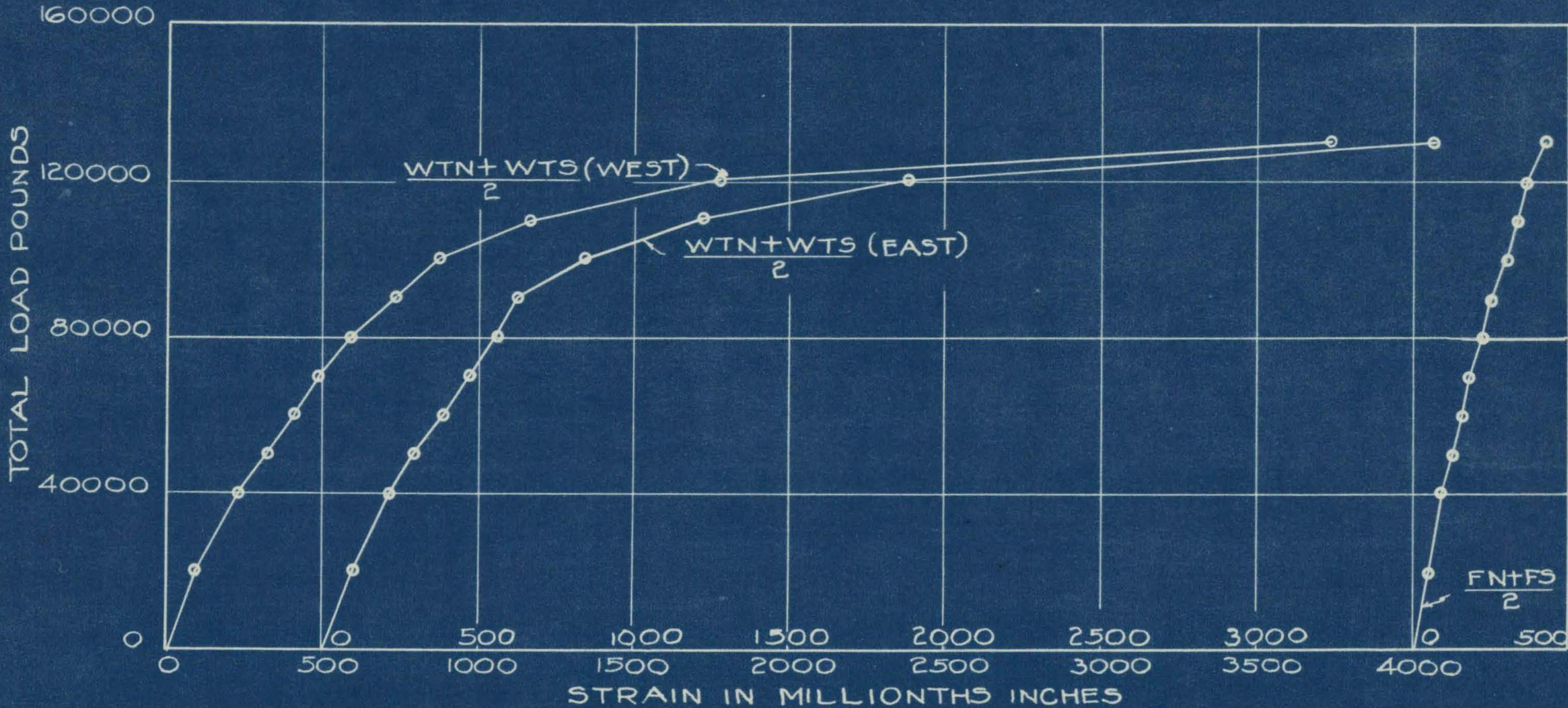

FIG. 59. AVERAGE TENSION STRAINS IN WEB AND FLANGE OF BEAM WB-10
TESTED DEC. 2, 1932

WB-10

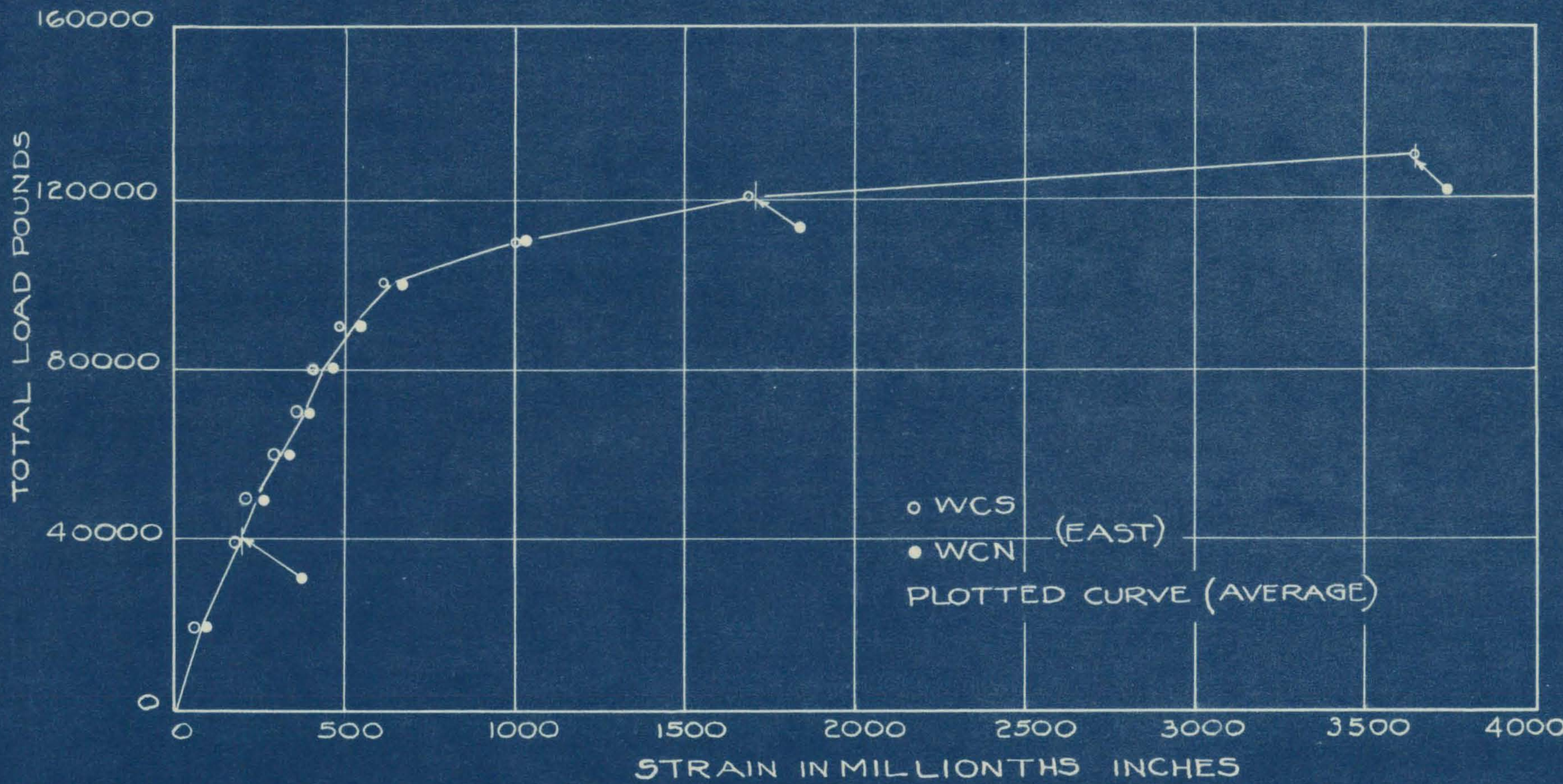

FIG. 60. COMPRESSIVE STRAINS ON BOTH SIDES OF WEB OF BEAM WB-10
TESTED DEC. 2, 1932

WB-10

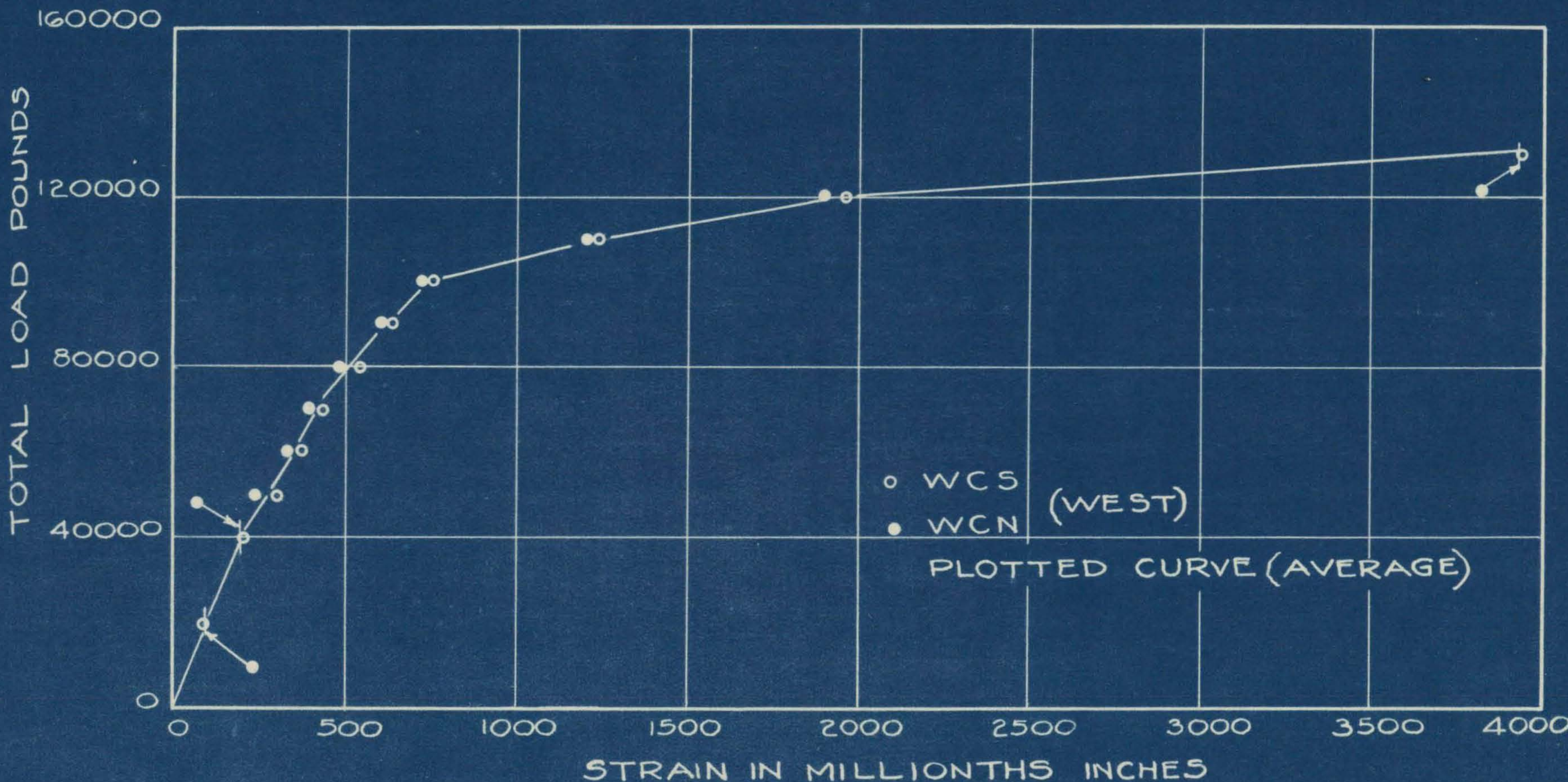

FIG. 61. COMPRESSIVE STRAINS ON BOTH SIDES OF WEB OF BEAM WB-10
TESTED DEC. 2, 1932

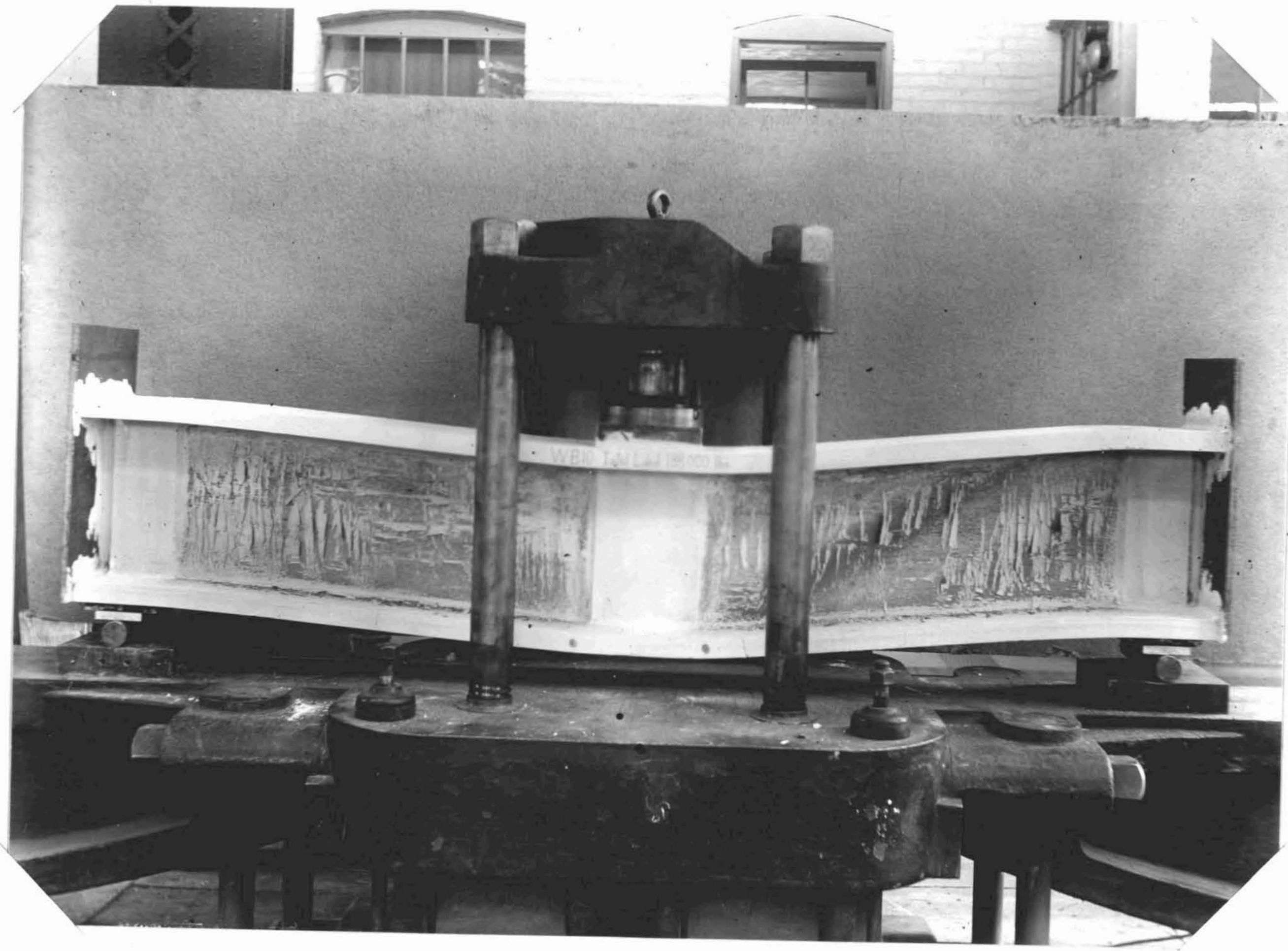

Fig. 62 - Appearance of Beam WI-10 at the Maximum Load of 188,000 lb. Tested December 2, 1932

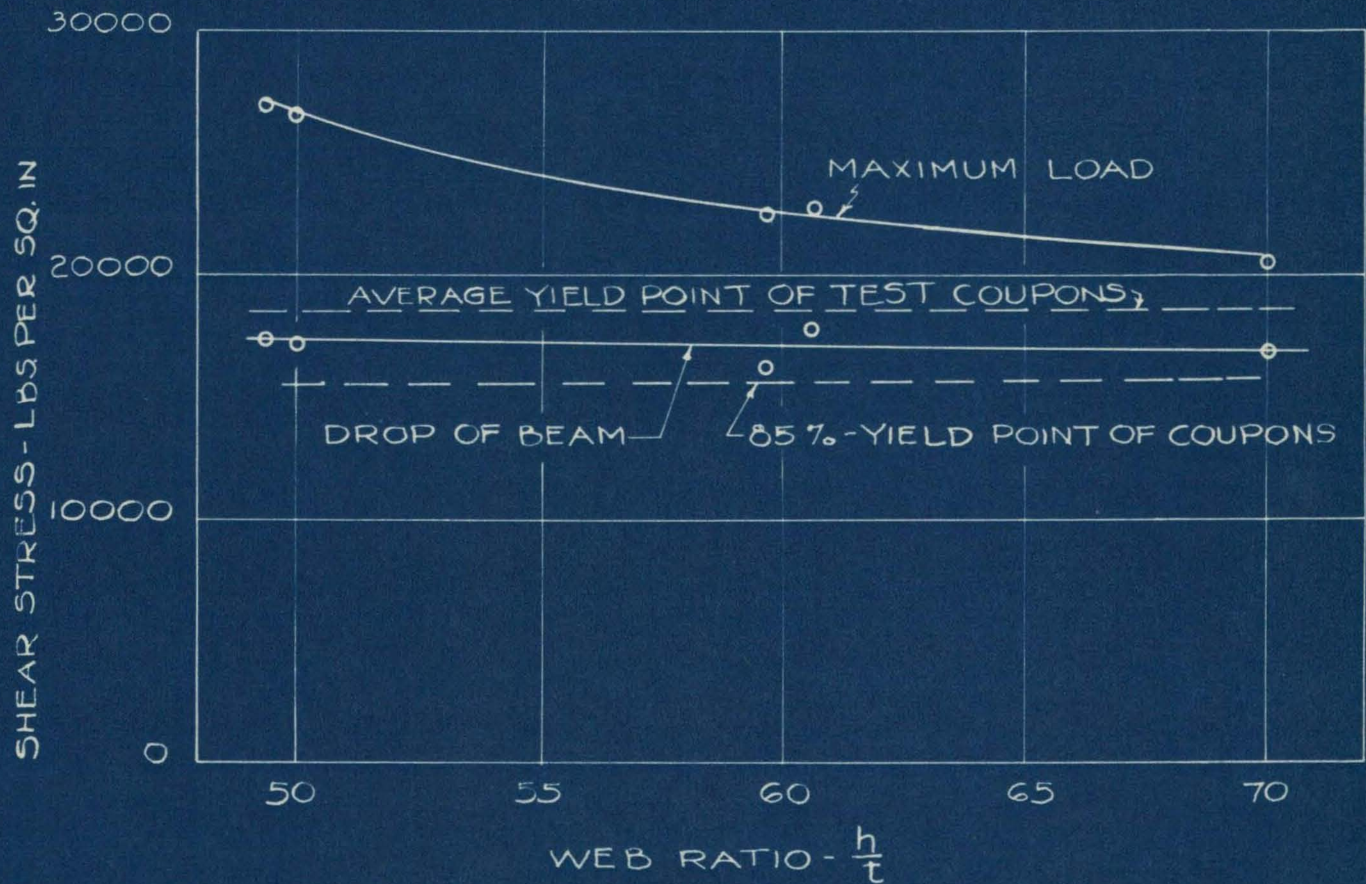

FIG. 63. COMPUTED MAXIMUM SHEARING STRESS IN THE WEB OF THE BEAMS AND YIELD-POINT STRESS IN SHEAR OF TEST COUPONS.