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TESTS OF LARGE BRIDGE COLUMNS

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Bureau of Standards

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By J. H. Griffith and J. G. Bragg

CONTENTS

	Page
I. Introduction	4
1. General	4
2. Description of apparatus	6
(a) Testing machine	6
(b) Preparation for tests	7
(c) Calibration and accuracy of the testing machine	8
3. Purpose and scope of the investigation	8
(a) Column strain in general	8
(b) End conditions	10
(c) Selection of observation stations—Extensometers	10
(d) Pin-plate observations	13
(e) Lattice-bar observations	13
(f) Repetitions of load—Initial strain	13
(g) Deflections	14
(h) Analysis of data	14
II. Original data of investigation	15
1. Description of test columns	15
2. Chemical and physical tests	16
III. Results of tests	16
1. Description of tables and curves	16
2. Explanatory note on tables and curves	18
IV. General discussion of results of tests	85
1. Behavior of columns as units	85
(a) Relation of loads at failure to yield points of steel	85
(b) Proportional limits of columns	88
(c) General remarks on maximum loads	89
(d) Maximum stress as a function of the slenderness ratio	91
(e) Deflections analyzed—Eccentricities	92
(f) Remarks on deflection curves	93
(g) Column moduli	94
(h) Ratio of lateral expansion to longitudinal compression	95
2. General laws of flexure	96
(a) Primary and secondary stresses discussed	96
(b) General types of eccentricity	97
(c) Law of stress distribution for the ideal case	98
(d) Method of calculation of maximum stresses	100
(e) Variations of stress shown graphically	101
(f) Components of eccentricity discussed	105

IV. General discussion of results of tests—Continued. Page
3. Initial strain in built-up columns 110
(a) Initial strain from punching and riveting 110
(b) Tests to show the effect of riveting on the proportional limit 113
(c) Elastic and inelastic strains due to texture of fabricated steel 114
4. Effect of repetitions of load 116
5. General action of lattice 117
(a) Behavior of ordinary lattice analyzed 118
(b) Action of lattice in deforming the column cross section 121
(c) Flexural stress in ordinary lattice 122
(d) Action of lattice in modifying the continuity of strain distribution 123
(e) Assymetrical action of single lattice 124
(f) General character of failures in columns with ordinary lattice 124
(g) Behavior of lattice with transverse bars 125
(h) Comparison of the two types of lattice 127
6. Behavior of pin plates 128
(a) Slipping of pin plates discussed 128
(b) Distribution of stress and strain near pin 129
(c) Splitting action of pin on web and plates 133
V. Summary and conclusions 134
1. Remarks 134
2. Recapitulation 135

I. INTRODUCTION

1. GENERAL

There are recorded in the following pages the results of an experimental investigation on 18 large bridge columns conducted by the Bureau of Standards at Pittsburgh. The specimens tested represented compression members designed for three bridges of long spans recently erected. The linear dimensions of the specimens were one-half those of the corresponding bridge members. Two of the columns, the first to be tested, were of nickel steel. The originals were designed for the Municipal Bridge at St. Louis. Ten of the columns were for the Chicago, Burlington & Quincy Railroad bridge over the Ohio River at Metropolis, Ill. They were constructed of various alloy and high-carbon steels. The remaining six members, test specimens for the new Memphis Bridge at Memphis, Tenn., were of nickel alloy and carbon steels. They possessed several essential points of difference in design from the other members.

The primary purpose of the tests was to determine the strength of such members under intended axial loads in relation to their general design and the grades of steel used in their construction.

The secondary purpose was to study the distribution of stress and strain throughout the shafts of the columns and their different details.

In the majority of cases the columns tested were essentially studies in design. It was one of the main purposes of the consulting engineers engaged in the design and construction of the bridges to supplement their knowledge obtained from preliminary calculations with that found from determinations of maximum stresses in the columns and their respective details. It thus became possible for the engineers to introduce such later modifications in the final construction of the actual columns of the bridges as appeared desirable after study of the tests.

Certain observations have been carried out in detail for all the columns. The scheme of observations has been varied in certain details when such changes promised to give valuable information and seemed practicable.

The Bureau of Standards acknowledges the cooperation of various engineers and manufacturers in this work. The firm of Boller, Hodge & Baird, consulting engineers and designers of the Municipal Bridge, was represented by H. C. Baird, who was present during a portion of the time of testing of the nickel-steel columns. C. H. Cartlidge, bridge engineer of the Chicago, Burlington & Quincy Railroad, in charge of the design and construction of the Metropolis Bridge, and Ralph Modjeski, consulting engineer in the design and construction of the Memphis Bridge, both took an active interest in the investigation and were present at a number of the tests.

The American Bridge Co. and the Pennsylvania Steel Co. did much to make the investigation full and complete. C. E. G. Larsson, assistant chief engineer of the American Bridge Co., and Thomas Earle and C. H. Mercer, superintendent and chief engineer, respectively, of the Pennsylvania Steel Co., represented their companies on the work. The preliminary reports of these officials covering their respective interests in the investigation were kindly placed at the disposal of the Bureau. They were consulted in the preparation of this paper, especially in the checking of results with the independent calculations of the Bureau.

The writers wish to make personal acknowledgments to C. E. Chase and F. M. Masters, inspecting engineers for Ralph Modjeski; Edward Godfrey, chief engineer of R. W. Hunt Co., in charge of inspection of the Municipal Bridge columns; and to R. T. Leipold,

representing C. E. G. Larsson, for assistance in their various capacities in making strain measurements, recording, or in preparation of certain portions of the data on those columns in which they were directly interested. The suggestions and criticisms of H. S. Prichard and O. H. Basquin in interpreting and discussing data were valuable in several instances. Thanks are due to the American Bridge Co., through G. F. Sparhawk, engineer at the Ambridge plant, and to the photographer of the Pennsylvania Steel Co. for a number of the cuts shown.

2. DESCRIPTION OF APPARATUS

(*a*) *Testing Machine.*—The columns were tested in the 10 000 000-pound Olsen compression machine of the Bureau. To make clear certain later steps in the experimentation, a brief description of this machine will be given. Reference may be made to the construction and accessories as shown in Figs. 1 and 2. This machine is capable of testing specimens whose length does not exceed 25 feet and whose width in the clear does not exceed 6 feet. The upper head during a test remains stationary, but for the purpose of adjusting the machine to the length of the specimen this head is moved by large nuts turning on the 13½ inch screws of the testing machine. These nuts are actuated by a gearing mechanism directly connected to and independently operated by a 15-horsepower electric motor placed on the head. The lower platen rests on a spherical base of 5 feet radius, which is mounted upon a vertically acting piston of 50 inches diameter and 2 foot stroke. There is attached to the bottom of this piston a 14 inch by 2 foot 5 inch guiding plunger, which fits into the large base casting of the testing machine and serves for proper centering of the parts.

The lower or straining platen is subject to the oil pressure in the cylinder, which at the capacity of the machine is approximately 5000 pounds per square inch. The oil pressure communicates through auxiliary piping with a smaller piston of $5\frac{9}{16}$ inches diameter. This piston has a "knife-edge" bearing on the main lever of an Olsen lever system. The ratio of the areas of the large and small piston is approximately 80:1, so that about 1/80 of the pressure upon the specimen is weighed, the weighing lever being graduated to read the actual load on the column.

The method of operating the machine during the tests was as follows: A triple plunger pump (see Fig. 1), having an air dome

FIG. 2.—*Bureau of Standards, 10 000 000 pound testing machine. Column 32S (silicon steel) under test*

for equalizing the pulsations of the three pistons and actuated by a 15-horsepower intrapole motor, is set in motion, the main valve of the mechanism being open and the relief valve closed. The function of the relief valve is to "short-circuit" the oil into the rectangular reservoir shown beneath the weighing mechanism. Speeds of the lower platen are varied in the following manner: Either by varying the lengths of strokes of pump piston through the hand wheel; by closing a by-pass needle valve and permitting only small quantities of oil to pass through into the main cylinder; or by varying the operating speed of the motor through the controller switch. The speeds of the main piston may in this manner be changed from 1/60 to 1/2 inch per minute.

For the purpose of uniformity in the application of loads, substantially the same piston speed was used in all tests. After the maximum load was recorded the column was strained at a higher speed in a manner to exaggerate its type of failure and the form of the deflection curve.

In reading the extensometers some 10 or 15 minutes were usually required by a single observer to take the strain measurements. In the earlier work the machine was stopped and held at the load with but little falling off. On a number of the later columns, however, as the time for observation was approached, the slowest possible speed was applied by means of the needle valve. The observer endeavored to so time his observation by beginning a few thousand pounds before and ending an equal interval later than the time at which the recorded load was reached that the mean time for the observations was the same as that of the recorded load. It is thought either of these methods gave satisfactory results.

(*b*) *Preparation for Tests.*—In preparing for a test the lower platen was leveled, the pin bearing was accurately centered on the platen, and the column was placed on the pin bearing by means of an overhead crane. The platen was lifted by applying pressure through the pump and tilted until a close parallelism of the upper column face was secured with the top head of the machine. The column was brought to bear in uniform contact with the upper head by such further adjustments of the lower platen as were found to be necessary.

The machine was "balanced" with the scale beam poise at zero, while there was an upward motion of the column during the working of the pump and before contact of the column with the upper head of testing machine was effected. This was done to eliminate

friction of the large piston as far as possible by substituting dynamic for static friction. With one exception, the initial load was taken at 1000 pounds per square inch of column cross section. Below loads of about this value stress strain readings are commonly found to be erratic, probably owing to nonuniform bearing in the testing machine. The loads applied were such as to give increments of stress of multiples of 1000 pounds per square inch. The strain readings were taken generally at closer intervals in the vicinity of the column yield point.

(*c*) *Calibration and Accuracy of the Testing Machine.*—On account of the capacity of the machine, the methods commonly used in calibration were impracticable, and the special apparatus for calibration was not finished at the time of tests. It was decided to use the test columns themselves as calibrating members, the method essentially being to compare the moduli of the columns with that of the mean moduli for steel as deduced from independent tests made on steel columns and small specimens. The mean axial modulus for the columns was found to be 29 600 000 pounds per square inch, with a probable error of the mean of ±120 000 pounds per square inch. This modulus is in close agreement with the mean modulus for steel as given from independent sources. The data and methods of calculation are fully described in the discussion in Part IV. Independent calibrations of this machine will be made later and the results published. It is believed that if any error exists in the constant of the machine, it is well within 1 per cent, judging from the values found in other large column tests.

3. PURPOSE AND SCOPE OF THE INVESTIGATION

(*a*) *Column Strain in General.*—It is recognized in the Mechanics of Materials[1] that there are but two determinate conditions under which a member in a testing machine may be tested: (1) Under such condition as will permit full freedom of motion at the ends, so that the column may accommodate itself to whatever flexure or unequal stress distributions occur during loading. Knife-edge bearings or ball joints have been used for this purpose in a number of tests. (2) Under the conditions where the bearings are fully restrained so that the terminal slopes of the strained axis may be maintained constant at that perpendicularity to the testing machine platens which existed before loading.

[1] Johnson's "Materials of Construction," 4th ed., p. 361.

If the terminal conditions are not definite, more or less indetermination is introduced in the mechanical status of the strained members.[2] This exists to a greater or less extent in the common type of end conditions where the members simply rest against the platens, however true the parallelism of end surfaces to platens may be. The volume of pressure (or surface integral) on the ends is indefinite, except as to the magnitude of its resultant. The shearing and normal stresses on the end surfaces cause bending and twisting of the column. These stresses are largely indeterminate experimentally with respect to their intensities and laws of variation. Moreover, the platens of the most perfectly constructed testing machines experience some elastic and inelastic displacements. When the column is otherwise mechanically defined as to its setting in the testing machine and its physical properties and coefficients as required in elastic analyses are known, it is believed the difference between the results actually obtained by tests and those that are deduced theoretically may still be sufficient by reason of the uncertainties enumerated to account for some of the discrepancies which are found to occur between theoretical and practical experimental determinations. The problem at hand is a practical one—to obtain the best possible results from the data by methods commonly employed among engineers, taking due account of such indetermination as exists as far as may be practicable, and keeping consistently in mind the general status of strain in the formation of judgments and the determination of laws.[3]

Although engineers have given various interesting analyses of those cases in which the variable tangent planes to the strained axis in general fall outside the principal planes of flexure, practical expediency makes it desirable to omit the consideration of a twisting torque, if its influence is sufficiently small that it may be neglected, and this was done with the following justifications. The columns are designed in such an ample manner that the twists which will occur in a more flexible member become relatively important. Evidence of this is shown in the cuts after failure. The deflection curves were always confined to one or the other of the principal planes for all the columns except those of nickel steel, and in these two the slight twists were purely

[2] See discussion of Ernest Jonson, Trans. A. S. C. E., 61, p. 200.

[3] J. O. Eckersley, in a parallel scheme, has shown the relation of the more rigorous theory of columns to the practical engineering theory. Trans. A. S. C. E., 76, p. 280. Love's Theory of Elasticity (2d ed., Cambridge, 1906), p. 370, gives an extended discussion of the equations of equilibrium which are to be satisfied as stated by Clebsch, Kirchhoff, and others. Reference may be made to Burr's Elasticity and Resistance of Materials, p. 447 et seq., for the engineering theory of the column, which has mainly been followed in this discussion.

local phenomena rather than actions of the members as a whole. These plane deflection curves were also always found in the tests of a long series of smaller columns conducted before the work of the present investigation was taken up. With the above distinctions in mind, the analysis of results has been conducted mainly from the more common engineering viewpoint. An effort has been made to state what appears to be the law of action of these columns in a way sufficient to meet the practical needs of designers in estimating the strengths of columns whose construction is similar to those discussed.

(*b*) *End Conditions.*—Column U_8U_9 of nickel steel had two square ends, the pin being omitted. Each of the others had a square upper end and a pin bearing at the lower end. The lower platform rests on a spherical surface of 5-foot radius, as has been described. Since the radius of curvature of the spherical bearing is relatively much greater than the radius of the pin, due account being taken of the coefficients of friction, motion must occur at the pin as the column deflects if it occurs at all. It seems, therefore, in the main proper to consider that the lower end of the columns, as is quite evident from the latter data, behaves practically as an ordinary square bearing up to a point near failure, when the friction is gradually overcome and the columns take the theoretical plane deflection curve corresponding to a pin and square ended member. (See Fig. 47.) When deflection takes place in the plane of axes of pin and column there is a reasonable "square-ended" restraint to the member. (See Fig. 50.)

(*c*) *Selection of Observation Stations—Extensometers.*—In view of the essentially heterogeneous nature of the steel it was deemed more appropriate to discuss, as a rule, the strain deportment of the members and details instead of their behavior in terms of stress, for the reason that the fundamental quantities directly measured are displacements and deformations. Although many engineers have expressed their experimental results in terms of stress, this seems inappropriate in the present analysis because stress as usually experimentally determined is a derivative quantity, obtained by multiplying the mean strain (deformation per inch of length), as measured with extensometers, by moduli of the material.

To observe the action of the members as a whole without special reference to the local distortions, which are always present to a greater or less extent in "built-up" columns, wooden rod extensometers were placed on the external flange angles, these being

placed symmetrically with reference to the mid length of the columns.[4] These extensometers were 80 inches long, having a gauge length conveniently taken as 5 and 10 multiples of that for the Berry portable gauges to be described later.[5] They had ball-joint connections at the top. At the bottom, through the medium of a sharp conical bearing, the rods rested on the spindles of Ames's dials. To prevent lost motion in these extensometers the spindles were held against the rods with rubber bands. The dial and ball-joint attachments were rigidly connected to small steel bars which screwed into ¼-inch tapped holes in the column. The Ames's dials read to 0.001 inch directly and by estimation to 0.0001 inch, the maximum error usually not exceeding 0.0002 or 0.0003 inch, as determined by the calibration. This error is insignificant in the platting of stress-strain curves. The common theory indicates that the extensometers should be placed symmetrically about the lower third point of the columns, so as to cover the commonly assumed region of greatest flexure, but, as has already been remarked, since the friction of the pin constrains the member for a considerable range of loading closely to its original perpendicularity with the base, it was deemed more justifiable to take readings at the mid-point, as has been described. The 20-inch Howard gauge was used on U_2U_3 and U_3U_9 for readings at the middle of column instead of 80-inch extensometers. The 8 or 16 inch Berry gauge was used for the remaining readings of these members and the other columns.

The outline of instructions for testing of the columns of the American Society of Civil Engineers' committee,[6] which was issued some time after this investigation was begun, recommended that four long-length gauge readings be taken on each member tested, two on the corners and one on the center of the flange and one on the center of the web. A departure from this suggestion was made for the purpose of having symmetry of observation stations about the axis, so that in all cases the mean axial strain could be found, special emphasis in this investigation being laid on the determination of axial moduli and stress-strain curves for a comparative study of behaviors of the various members.

Readings in 8-inch and 16-inch gauge lengths were taken in the majority of cases at the mid-lengths on the webs and flanges and at the ends, but sufficiently removed from pin plates and end

[4] The coefficient of expansion of wood is lower than in steel, but hygrometric effects are important when rods are not varnished.

[5] Longer lengths than 80 to 100 inches were found from previous investigations to be not as reliable in obtaining sets as the lengths chosen.

[6] Proc. Am. Soc. of Civil Engs., Jan. 20, 1915.

battens to avoid local effects due to the absorption of stress from the pin into the body of the column. Transverse readings in close proximity to the longitudinal stations were taken in a sufficient number of cases to determine the ratio of lateral to longitudinal strain; i. e., Poisson's ratio.

The Berry portable strain gauge was used for the shorter gauge readings. This gauge has "invar" steel side strips, the coefficient of expansion of this steel being nominally only about one-twenty-eighth that of carbon steel (but sometimes varying from this), so that it is affected little in handling. There is a fixed point at one end and a movable joint and bell-crank lever at the other end, the arm of this crank being connected to an Ame's gauge spindle, giving the possibility of reading directly to 0.0002 inch through the 1:5 multiplication of the bell-crank lever, and by estimation to 0.00002 inch. Two holes were drilled 8 inches apart, with a No. 55 drill, for setting the gauge. The holes were countersunk with a tool having an angle of 110 degrees, for the purpose of insuring a good shoulder in setting the gauge points.

The 8-inch Berry gauge was calibrated by means of an accurately machined apparatus consisting of fixed and movable steel heads mounted upon a substantial base. The gauge points were inserted in holes in the heads similar to those made with the No. 55 drill in the columns. The gauge was clamped in position by a "C" clamp similar to that used in clamping the instrument to a test piece. Small displacements of the movable head were measured with a screw micrometer attached to the apparatus and the corresponding readings on the dial taken. The micrometer read by vernier to 0.0001 inch, and its error was found by independent calibrations to be always less than 0.0001 inch for travel of the screw in either direction. The constant of the 8-inch gauge was found after a number of trials to be 0.956. The constants of 2-inch and 10-inch gauges were found to be 0.946 and 0.950, respectively. These values may be in error by one or two units in the last figure in each case. Observations made with the standard bar accompanying the gauges show that an observer can usually check his readings to within 0.0001 or 0.0002 inch. The chief sources of errors in the Berry gauge readings of the present tests may be attributed to imperfect holes for placing the gauge points. These holes are inserted with a portable drill and their axes are not always precisely normal to the surface of the column at the point measurements are taken. The gauges have been chiefly employed in the

tests for obtaining the behavior of the column details qualitatively. The results determined in this way may be in error to the extent of ± 5 or 10 per cent. Where quantitative results are desired for the purpose of making a closer comparison, the strains have been multiplied by 0.956 in the later discussions of the results.

The Ames' dials were found by similar calibrations, as a rule, to give readings correct within 0.0002 or 0.0003 inch. In certain dials it was observed that the reading at a particular point of the graduated arc may be at times as high as 0.0010 inch. The errors of the dials are largely compensated by the method of averaging results which was employed in the discussion of the observations.

(*d*) *Pin-Plate Observations.*—It was the original intention of the experimenters to determine data for platting the direction of the resultant strain and "lines of stress" in the flow of these from the pin, this being a "source" of strain energy from the lower platform, as the member is compressed. On account of the effects of initial strain, apparent lost motion and distortions of the plates themselves, it was difficult to draw precise conclusions in this respect. More attention was then given to a study of the amount of slip in the plates themselves, which is an important factor to be determined.

(*e*) *Lattice-Bar Observations.*—Two types of lattice bars are presented for discussion. One is the ordinary type in which double and single diagonal bars are used. The other employs in addition to the double diagonal members of the lattice panels a transverse bar between successive panels. This style of lattice will be of interest to engineers, because it was used throughout in the Hoangho Bridge[7] recently erected in the Province of Kaow Chow, China. The system is deserving of study in America as to certain possible mechanical advantages.[8] A sufficient study has been given to the two types to afford opportunities for comparison and the formation of judgments.

(*f*) *Repetitions of Load—Initial Strain.*—Attention is called to some behaviors obtained when various repetitions of the load were applied within the yield point of the column. The effect upon the various details, especially pin plates and lattice bars, is interesting and is indicated by the successive stress-strain curves.

The initial strain in columns has been discussed and supplementary tests have been given to show the influence of strains induced through fabrication operations.

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[7] Die Hoangho-Brücke, Zeit. des Vereines Deutscher Ingenieure, 58, pp. 374–377, Mar. 7, 1914.

[8] A rather slender and ineffective type of this lattice was used also on the first Quebec Bridge members.

(*g*) *Deflections.*—A number of measurements for deflection were taken in the coordinate directions parallel and perpendicular to the pin on columns U_2U_3 and U_8U_9 at the mid lengths. The results of these measurements are given to afford some idea of the local strains and twists to which the columns were subject. The method, however, is open to some criticism, it is believed, in the case of absolute deflections and was abandoned on the later columns. In several of the other columns a taut thin wire was stretched between points at the top and bottom. It has been remarked, however, by Buchanan [9] with reference to his own tests that "the deflection movements lateral and vertical were so irregular and eccentric that they were not thought to have any particular significance in determining the elastic value of the column" and perhaps the importance of the deflection has been overrated by those who are inclined to take the purely mathematical viewpoint in establishing a mechanics of the built-up member of small slenderness ratio.

(*h*) *Analysis of Data.*—In analyses of the experimental strain data of columns, engineers are often perplexed on account of the fact that while the platted results of calculation call for smooth curves without discontinuities, the experimental curves of laboratory practice are too often made up of many "zigzags" and tacks. This may be seen, for example, in a study of the numerous diagrams given by Talbot and Moore [10] in their investigation of built-up columns, or is instanced in the remark of Buchanan on the irregularity of the laws of column deflections obtained by him. It is quite inevitable that this must often be the case in any single analysis of a built-up member, even when the effect of minor variables have been taken into account and there has been an adequate compensation for accidental errors. An individual test is usually insufficient for the determination of correct conclusions on account of local action and readjustment in the pieces when strained. A recourse rather must be had to what Maxwell and Gibbs have called statistical methods in the interpretation of results. It is the general trend of the data that is of most importance and not the conclusion from a particular test in deriving laws of behavior. In the analysis of results given by the authors attention has been directed chiefly to determining the laws of the series of columns, rather than to those of individual cases. Comparisons of the stress-strain curves drawn for par-

[9] Engineering News, 58, p. 685; Dec. 26, 1907.

[10] Talbot and Moore, Investigation of Built-Up Columns Under Load, Univ. Ill. Exp. Station, Bull. 44, Figs. 8–15, p. 22 et seq.; also summary.

ticular stations of a column with the mean curves afford interesting data. To give all these curves, however, would extend the scope of the paper beyond the limits planned. Reference may be made to the tables for much matter bearing upon the behavior of individual columns for which time and space do not permit a discussion.

II. ORIGINAL DATA OF THE INVESTIGATION

1. DESCRIPTION OF TEST COLUMNS

There were seven distinct types of columns presented for test. In certain cases the design and dimensions of the columns were unvaried, but different grades of steel have been used in the construction, the steel being considered the essential variable. In other cases the designs and steels were both varied in accordance with the judgment of the designers, in such a way as to give the best performance as columns when the cost of construction was considered. The subdivision of the group was as follows (see Table 1): There were two columns of 3¼ per cent nickel steel, but of different dimensions corresponding to two separate members in the Municipal Bridge. There was but one member under consideration for the Metropolis Bridge. Two separate designs of this member were made and five grades of steel were used in the construction, viz, Mayari (1½ per cent nickel alloy from natural Cuban ore), silicon, chrome, and two grades of high-carbon steel, making 10 test members. In the case of the Memphis Bridge 3 different members of the bridge were considered. There were two designs corresponding to each of the originals, one for the use of high-carbon steel and the other for the Mayari steel, making 6 test members.

Table 1 gives the physical dimensions of the test columns, the different types of cross sections and their areas, the values of the radii of gyration and slenderness ratios used by the designers, with other essential data. The areas used by the designers were calculated from tables in the steel handbook and are recorded under the heading "Nominal area." They were used in calculating the total and unit loads at which observations were made. (See Tables 3 to 21.) After the completion of the experimentation, it was found that the scale weights of the different plates and angles used in the column shafts were available through the courtesy of the steel companies for most of the columns. The exact areas were calculated from these weights and are given under the heading "Actual areas." They were used in correcting the maximum stresses and moduli of the columns given in the discussion of Part IV.

2. CHEMICAL AND PHYSICAL TESTS

The chemical and physical properties of the steels used in the construction of the columns are given in Table 2. These are later discussed in connection with the behaviors of the columns.

III. RESULTS OF TESTS

1. DESCRIPTION OF TABLES AND CURVES

In Tables 3 to 21, inclusive, the numerals at the head of the columns give the station numbers at which strain readings were taken. A key to the position of these stations is found by reference to similar numerals placed upon diagrammatic sketches of the essential portion of the columns. One of these accompanies each table.

The initial-strain readings are taken, for reasons previously mentioned, at a load of 1000 pounds per square inch, which serves as the zero of the observations. Unusual care is required in taking the initial set of observations. Attention is drawn to a few results in the tables by footnote explanation where an error in the initial readings seemed probable.

The tabulated data given under the station numerals represent the cumulative positive or negative elongations in the gauge lengths between extensometer points corresponding to the load recorded by the beam of the testing machine. The plus sign (+) denotes extension in the steel; minus (−) denotes compression. These data are immediately derived from the original log sheets by taking the differences of initial and later readings of the extensometer dials at the particular loads under consideration. The results were finally checked with the original readings as recorded.[11] The strain data given in the table will suffice for many purposes in studying the behavior of columns under load. To make some of this information more readily apparent, stress-strain curves have been platted in a large number of instances, as shown in Figs. 22 to 41.

In the curves which show the action of the column as a whole the relative elongations in 80 inches, found by averaging readings

[11] It is common among observers to correct for considerable changes in temperature by compensating all readings with reference to a standard bar reading taken at the beginning and end of the interval of a series of observations. In the earlier experimentation time, temperature, and standard bar readings were taken for each load, but from practical considerations this was later abandoned, as the small variation in the surface temperature warranted the inference that the temperature within the body of column was uniform in the time range of test. In view of the fact that the probable error in a reading of the dial may be perhaps ±0.0001 inch or 0.0002 inch, no increase in accuracy attaches to a compensation of the results when the small range of temperature is considered, and the corrections being close to the above amounts, a slight error may even be introduced. Special attention was given to maintaining the temperature of the room nearly constant so that the variation of the mean temperature of the members was probably less than 1 degree in all cases.

at stations 1 to 4 and dividing by 80, are platted with the stresses. The slope of the resulting curve gives the mean modulus of the column. This modulus becomes variable and loses its ordinary meaning at a critical stress equal to approximately half the maximum load per square inch on the column. The critical point at which deviation of the curve from the tangent at the origin takes place has been called in this paper the proportional limit of the column by analogy with that found for a small test piece. It is, of course, understood that the proportional limit as above defined is not that of the component steel. It may be called that of the fabricated material. Readings have been reduced to strains and averaged, and stress-strain curves are platted in a number of cases for 8-inch gauge lengths at the middle of columns and adjacent to pin plates. While these curves are quite similar in form to the preceding, a slight difference in slope may in general be expected and is probably influenced by the lack of uniformity of stress distribution to some extent, also by the variable degrees in "fluting" or "waviness" of the different shapes brought about by the clamping action of high-power riveters.

The writers were unable to devise a very efficient graphical method for showing systematically the variation of strains in the latticing. This can be studied most expeditiously by direct observation and comparison of the elongations (±) as given in the table themselves for any group of lattice bars. The behaviors that are taking place are very apparent and easily analyzed. In Figs. 28 and 35 the axial strains in lattice bars are shown as functions of the average stress on the columns.

Curves have been drawn to illustrate the slipping of one pin plate over another. (See Figs. 26, 27, etc.) The principle involved consists in taking two gauge readings side by side for comparison as was done, for example, on column 31HC in Table 8 and Fig. 26. One gauge point was placed at the end of the lower outer pin plate. Another was placed on the plate next in order and on the same line as the first, but about one-half inch removed. Two other gauge points were similarly placed 16 inches away from the first. The horizontal difference between the abscissas of curves gives the relative movement of one plate over the other per inch of gauge length. The actual displacements are shown by the abscissas of the resultant curve multiplied by 16. Further evidence that this slip actually occurred was found by placing the fingers on the lower end of certain columns where

the movement was pronounced. The original planed surface at initial loading became "stepped" in some cases at an earlier stage than in others, the serrated action gradually increasing with the load. (See Fig. 43.)

In some cases scaling of the column was noted in proximity to the elastic limit of the steel and is recorded. In most cases but one maximum load was found, but in others, noticeably column MY, Table 7, after the column appreciably deflected higher loads were obtained with further deflections until a *maximum maximorum* was reached at some 4000 pounds per square inch higher than that of the primary failure.

The four columns US_2 MS_1 and U_2M_1 failed prematurely by sudden bending at the lower end for obvious reasons, and did not develop the full strength of their sections. Evidences of readjustments taking place as the loads advanced were apparent in many of the columns and have been recorded as "creaking sounds."

Systematic observations were made for deflections in the two coordinate directions of the cross section on columns U_2U_3 and U_8U_9. The apparatus used is illustrated in Fig. 5. The measurements were made at stations indicated with a B. & S. screw micrometer, and were read to 0.001 inch. The component displacements calculated from the data are given in Table 5. The plus sign signified an outward movement from the original axis. In the case of the other columns the measurements of deflections were made at the mid lengths of each column. A rule and taut wire attached to the ends of columns were used for this purpose.

2. EXPLANATORY NOTE ON TABLES AND CURVES

The following recapitulatory explanation will be found useful in a cursory examination of the data derived from the experiments.

Table 1 gives the structural and mechanical data for each column, its location in the bridge, etc.

Table 2 gives the results of chemical analyses and physical tests for the different steels used in the construction of columns.

Tables 3 to 21 give the cumulative deformations determined at the various stations of observation corresponding to the applied stresses in pounds per square inch of sectional area of the columns. Figs. 22 to 41, inclusive, show stress-strain curves platted from these data.

Remarks.—(*a*) Order: The tabular data, stress-strain curves, and photographs of members after failure follow the order of Table 1 throughout.

(*b*) Stations: The stations of observation are indicated by the numerals over the tabular data. The position and orientation of these stations on the respective members are given on the diagramatic or key sketch accompanying each table, Figs. 3 to 21, inclusive.

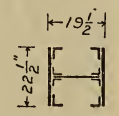
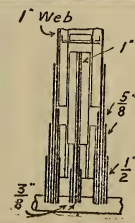
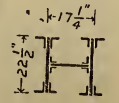
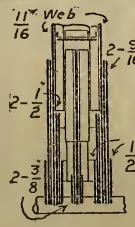
(*c*) Readings: The initial load as measured on the testing machine, unless noted, corresponds to 1000 pounds per square of nominal area of column. (See Table 1.) The increments of loading are taken in multiples of 1000 pounds per square inch of nominal section. The values recorded in the tables are the cumulative deformations in inches found by subtracting the reading on the extensometer at inital load from that at any load under consideration. The sign (+) signifies an elongation in the steel; (—) a contraction. The deformation recorded is the mean amount observed in whatever gauge length was used, 8 to 16 inches or 80 inches, as it happens to be.

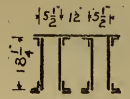
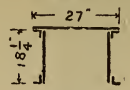
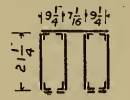
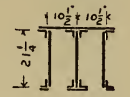
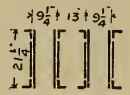
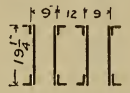
(*d*) Curves: To place all strain measurements on a mechanical parity for the group of columns, the deformations per inch—i. e., the mean strains—were calculated. To obtain these the tabular values are divided by the respective gauge legths, 8 to 16 inches or 80 inches, as the case may be. In the curves of Figs. 22 to 41, these mean strains are platted as abscissas; the mean stresses per square inch nominal area of column cross section are platted as ordinates. Other data are given on the figures themselves.

(*e*) The modulus of a column is taken (within the limits of proportionality) as the ratio of the mean stress over its cross section to the average strain as found from the readings upon the four 80-inch extensometers placed at mid length of the column and parallel to its axis. The "Proportional limit" for the column was taken as the stress at which there was a sensible deviation from constant proportionality of stress to strain. as shown by the 80-inch gauge lengths. Its value is about half the tensile yield stress of the column material as determined in the ordinary manner. The maximum loads, moduli, etc., given in Fig. 60 have been corrected to correspond with exact cross-sectional areas determined by weighing the columns, the results differing but slightly from those deduced from the nominal areas as found from the pocket-book shapes.

For further explanations, reference should be made to the descriptions and studies given in the paper, and to the figures and tables themselves.

TABLE 1.—Structural and Mechanical Data Concerning Columns Tested

Bridge	Name of column	Steel	Manuf.	"L" length of column, center of pin to upper end.	Ends: Diam. of lower pin, in.	Ends: Upper	Cross section of shaft	Material of shaft	Material at pin end	Bracing of shaft: Lattice, in.	Bracing of shaft: Transv. diaphgms, in.	Bracing of shaft: Battens at pin, in.	Area, sq. in.: Nominal	Area, sq. in.: Actual	Radius of gyration R: ⊥ to pin	Radius of gyration R: ∥ to pin	Slenderness ratio=L÷R: ⊥ to pin	Slenderness ratio=L÷R: ∥ to pin
Municipal, St. Louis	U_2U_3	Ni	American Bridge Co.	15 ft. $8\frac{1}{2}$ in. end top chord	$7\frac{1}{2}$	Square		2 out. webs, $24''x\frac{3}{4}''$ 4 out. Ls, $4''x4''x\frac{7}{16}''$ 1 cen. web, $24''x\frac{11}{16}''$ 4 cen. Ls, $4''x4\frac{5}{16}''$ 2 dia. pls. $9\frac{1}{2}''x\frac{5}{16}''$ 8 dia. Ls, $2''x2''x\frac{1}{4}''$	In order from outside for U_2U_3 only 6 pls. $\frac{7}{16}''$ thick 2 pls. $\frac{1}{2}''$ thick 2 pls. $\frac{7}{16}''$ thick 2 webs, $\frac{3}{4}''$ thick out. 2 pls. $\frac{1}{2}''$ thick 8 pls. $\frac{3}{8}''$ thick 1 cen. web, $\frac{11}{16}''$ thick	$2\frac{1}{2}$ chan. ($\frac{1}{4}$ web)	$3\frac{1}{4}$ thick	$\frac{1}{4}$ pls.	88.8	no data	7.55	8.57	25.0	22.0
	U_8U_9	Ni		24 ft. 0 in. center top chord	Square Pin not used			2 out. webs, $24''x\frac{29}{32}''$ 2 side pls. $16''x\frac{7}{16}''$ 4 out. Ls, $4''x4''x\frac{7}{16}''$ 1 cen. web, $24''x\frac{11}{16}''$ 4 cen. Ls, $4''x4''x\frac{5}{16}''$ 2 dia. pls. $9\frac{1}{2}''x\frac{5}{16}''$ 8 dia. Ls, $2''x2''x\frac{1}{4}''$			$3\frac{5}{16}$ thick	$\frac{1}{4}$ pls.	110.3	no data	7.78	8.97	37.0	32.1
Metropolis	HC	C	Penna. Steel Co.	All 24 ft. 4 in. upper end post	7	Square		2 side pls. $22\frac{1}{2}''x1''$ 4 Ls, $4''x4''x\frac{1}{2}''$ 1 dia. pl. $17\frac{1}{2}''x\frac{7}{16}''$ 4 dia. Ls, $3''x3''x\frac{3}{8}''$		Transv. $4x3x\frac{5}{16}$ in. angles diag. $4x\frac{1}{4}$ bars	$3\frac{1}{4}$ thick	$\frac{1}{4}$ pls.	76.14	74.58	6.73	8.46 for all	43.5	34.5
	MY	My											76.14	74.90				
	31HC	C	American Bridge Co.										75.99	76.94				
	32S	Si											75.99	76.94				
	34CS	Cr											75.99	76.94				
	HCl	C	Penna. Steel Co.	All 24 ft. 4 in. upper end post	7	Square		2 side pls. $22\frac{1}{2}''x\frac{11}{16}''$ 8 Ls, $4''x4''x\frac{1}{2}''$ 1 dia. pl. $15\frac{3}{4}''x\frac{7}{16}''$ 4 Ls, $3''x3''x\frac{3}{8}''$		Transv. $4x3x\frac{5}{16}$ angles diag. $3x\frac{1}{4}$ bars	$3\frac{1}{4}$ thick	$\frac{1}{4}$ pls.	76.46	75.35	7.60	8.05 for all	38.5	36.4
	MYl	My											76.46	75.28				
	31HCl	C	American Bridge Co.										76.26	77.17				
	32Sl	Si											76.26	77.17				
	34CSl	Cr											76.26	77.17				

Memphis	US_2MS_1	C	Penna. Steel Co.	15 ft. 10 in. upper end post	7	Square		1 cov. pl. 27″x¼″ 4 web pls. 18″x$\frac{7}{16}$″ 8 Ls, 3″x2″x⅜″ 4 flats, 2″x⅜″	Each col. rib: 1 pl. ½″ outside 1 pl. ⅜″ inside 1 web, $\frac{7}{16}$″	Transv. tie pls. ¼, see cuts; diag. bar 3x¼	4 ¼ thick	$\frac{5}{16}$ pl.	55.17	55.97	6.61	9.05	28.7	21.0
Memphis	US_2MS_1	My		15 ft. 10 in. upper end post				1 cov. pl. 27″x$\frac{7}{16}$″ 2 web pls. 18″x½″ 6 Ls, 3″x3″x⅜″	Each col. rib: 2 pls. $\frac{9}{16}$″ outside 2 pls. ⅜″ inside 1 web pl. ½″	Diags. 3x1¾x¼ angles	None	⅜ pl.	42.47	41.66	6.87	9.78	27.7	19.4
Memphis	U_2M_1	C		23 ft. 6 in. upper end post	8½			1 cov. pl. 25″x¼″ 4 web pls. 21″x$\frac{11}{16}$″ 8 Ls, 4″x3″x½″ 4 flats, 3″x½″	Each col. rib: 1 pl. ¼″ outside 1 pl. ½″ inside 1 pl. ⅜″ inside 1 web, $\frac{11}{16}$″	Diags. 3x1¾x¼ angles	6 ¼ thick	$\frac{5}{16}$ pl.	96.01	95.77	7.80	9.25	36.1	30.5
Memphis	U_2M_1	My		23 ft. 6 in. upper end post				1 cov. pl. 27″x¼″ 2 out. webs, 21″x$\frac{17}{32}$″ 4 out. Ls, 4″x3″x½″ 2 flats, 3″x⅜″ 1 cen. web. 21″x$\frac{15}{32}$″ 4 cen. Ls, 4″x3″x⅜″ 1 flat, 6½″x⅜″	Outside ribs (2): 2 pls. ½″ 2 pls. ⅝″ 2 pls. $\frac{9}{16}$″ 2 pls. ⅜″ 2 pls. $\frac{9}{16}$″ } Cen. rib (1)	Diags. 3x1¾x¼ angles	5 ¼ thick	$\frac{5}{16}$ pl.	66.55	66.02	8.17	8.47	34.5	33.8
Memphis	LC_0LC_1	C		15 ft. 6 in. bot. chord at pier				4 webs, 21″x¾″ 4 side pls. 13″x½″ 8 Ls, 4″x4″x½″	Each col. rib: 3 pls. ⅜″ 1 web, ¾″ 1 side pl. ½″	Tie pls. ½ diag. 3x1¾x¼ angles	4 ¼ thick	$\frac{5}{16}$ pls.	119.00	118.18	6.74	12.14	27.6	15.3
Memphis	LC_0LC_1	My		15 ft. 6 in. bot. chord at pier				4 webs, 19″x$\frac{19}{32}$″ 8 Ls, 4″x4″x½″	Each col. rib: 3 pls. ⅜″ 1 pl. ½″ 1 web pl. $\frac{19}{32}$″	Tie pls. ¼ diag. 3x1¾x¼ angles	4 ¼ thick	$\frac{5}{16}$ pls.	75.12	76.47	6.87	11.59	27.1	16.0

Notes on rivets; The batten plates, lattice bars, and transverse diaphragms of columns U_2 U_3 and U_8 U_9 were of carbon steel, the other material being of nickel steel. All of this material was subpunched and reamed. The rivets in the main members were nickel steel of ½ inch diameter. The rivets in the battens and lattice were carbon steel of $\frac{7}{16}$ inch diameter. The rivets in all of the other columns were of soft steel ½ inch in diameter. The holes were drilled $\frac{9}{16}$ inch diameter in the case of the Metropolis Bridge and $\frac{17}{32}$ inch diameter in the case of the Memphis Bridge.

TABLE 2.—Properties of Steels

Name of column	Per cent of elements in chemical tests Averages								Tensile tests, pounds per square inch			
									Yield point			Break load, mean
	Ni	C	P	Mn	S	Si	Cr	Cu	Mean	High	Low	
Municipal Bridge:												
U_2 U_7	3.51	0.29	0.014	0.59	0.031	.....	.....	.....	55 500	63 100	50 500	93 300
U_8 U_9	3.59	.32	.015	.59	.033	.....	.....	.....	55 200	63 100	52 040	93 060
Metropolis Bridge:												
HC	.....	.36	.012	.43	.030	.....	.....	.....	40 700	50 000	37 590	76 660
HC1	.....	.36	.012	40	.030	.....	.....	.....	44 100	50 000	37 650	75 930
MY	1.12	.34	.010	.68	.037	0.10	0.53	0.10	59 900	68 380	55 890	100 610
MY1	1.12	.34	.010	.68	.037	.10	.53	.10	62 110	68 380	56 080	101 170
31HC	.....	.28	.017	.64	.031	.....	.....	.....	42 900	45 440	37 280	73 400
31HC1	.....	.28	.017	.64	.031	.....	.....	.....	42 700	45 440	37 280	74 400
32S	.....	.35	.017	.83	.039	.38	.....	.....	57 200	58 840	51 940	87 700
32S1	.....	.35	.017	.83	.039	.38	.....	.....	56 500	57 480	51 940	88 300
34CS	.....	.31	.017	.49	.028	.13	.66	.....	55 800	59 290	48 260	82 700
34CS1	.....	.31	.017	.49	.028	.13	.66	.....	54 200	58 010	48 260	83 600
Memphis Bridge:												
US_2MS_1 carbon	.....	.23	.012	.49	.027	.....	.....	.....	41 740	43 700	40 250	65 420
US_2MS_1 Mayari	1.58	.33	.016	.64	.027	.....	.37	.....	62 430	72 160	52 000	99 800
U_2M_1 carbon	.....	.23	.012	.49	.029	.....	.....	.....	41 740	43 700	40 250	65 420
U_2M_1 Mayari	1.58	.33	.016	.64	.027	.....	.37	.....	62 830	72 160	52 000	99 800
LC_6LC_1 carbon	.....	.23	.012	.49	.029	.....	.....	.....	41 740	43 700	40 250	65 420
LC_6LC_1 Mayari	1.58	.33	.016	.64	.027	.....	.37	.....	62 430	72 160	52 000	99 800

TABLE 3.—Log of Test, Column U_2U_3, of Nickel Steel

[Cumulative Elongations (±) at Stations Below. Tested Jan. 16–17, 1913. Time of Test, Jan. 16, 9 a. m. to 4.27 p. m.; Jan. 17, 10.30 a. m. to 4.30 p. m.]

Load, pounds	Load in pounds per square inch	19	20	21	22	23	24	Average 19 to 24	25	26	27	28
177 600	2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000				
355 200	4 000	—.0013	—.0014	—.0009	—.0017	—.0018	—.0012	—.0014				
532 800	6 000	—.0029	—.0029	—.0029	—.0032	—.0033	—.0027	—.0030				
888 000	10 000	—.0054	—.0055	—.0051	—.0057	—.0056	—.0049	—.0054				
1 243 200	14 000	—.0081	—.0078	—.0077	—.0084	—.0079	—.0073	—.0079				
1 598 400	18 000	—.0110	—.0107	—.0106	—.0114	—.0109	—.0106	—.0109				
1 953 600	22 000	—.0144	—.0138	—.0136	—.0145	—.0141	—.0139	—.0141				
2 308 800	26 000	—.0172	—.0167	—.0166	—.0174	—.0168	—.0165	—.0169				
2 841 600	32 000	—.0216	—.0209	—.0206	—.0218	—.0212	—.0209	—.0212				
3 374 400	38 000	—.0265	—.0256	—.0252	—.0262	—.0259	—.0256	—.0258				
[a]3 729 600	42 000	—.0301	—.0289	—.0285	—.0297	—.0292	—.0285	—.0292	([b])	([b])	([b])	([b])
2 600 000	29 300	—.0205	—.0203	—.0200	—.0207	—.0203	—.0199	—.0203	([c])	([c])	([c])	([c])
3 729 600	42 000	—.0303	—.0293	—.0290	—.0300	—.0294	—.0293	—.0296	0.0000	0.0000	0.0000	0.0000
3 907 200	44 000	—.0322	—.0312	—.0309	—.0319	—.0312	—.0313	—.0315	—.0007	+.0003	—.0018	+.0003
4 084 800	46 000	—.0340	—.0330	—.0328	—.0337	—.0330	—.0330	—.0333	—.0027	+.0007	—.0045	+.0005
4 173 600	47 000	—.0354	—.0343	—.0343	—.0354	—.0345	—.0344	—.0347	—.0037	+.0008	—.0067	+.0008
4 262 400	48 000	—.0363	—.0355	—.0355	—.0367	—.0357	—.0353	—.0358	—.0045	+.0009	—.0095	+.0010
4 351 200	49 000	—.0386	—.0378	—.0379	—.0394	—.0382	—.0378	—.0383	—.0065	.....	—.0139	+.0019
4 440 000	50 000	—.0397	—.0391	—.0396	—.0413	—.0399	—.0392	—.0398	—.0074	+.0012	—.0166	+.0025
4 528 800	51 000	—.0406	—.0404	—.0410	—.0431	—.0416	—.0402	—.0412	—.0086	+.0013	—.0203	+.0035
4 617 600	52 000	—.0423	—.0424	—.0435	—.0458	—.0437	—.0420	—.0433	—.0098	+.0016	—.0235	+.0044
4 706 400	53 000	—.0442	—.0448	—.0463	—.0490	—.0468	—.0441	—.0459	—.0114	+.0018	—.0277	+.0057
4 795 200	54 000	—.0461	—.0472	—.0489	—.0525	—.0492	—.0461	—.0483	—.0127	+.0021	—.0311	+.0071
4 884 000	55 000	—.0484	—.0506	—.0529	—.0574	—.0530	—.0483	—.0518	—.0146	+.0020	—.0326	+.0085
4 972 800	56 000	—.0506	—.0537	—.0567	—.0615	—.0566	—.0507	—.0550	—.0158	+.0024	.....	+.0096
5 061 600	57 000	—.0544	—.0589	—.0627	—.0698	—.0629	—.0546	—.0606	—.0181	+.0027	.....	+.0115
5 150 400	58 000	—.0585	—.0666	—.0723	—.0823	—.0725	—.0601	—.0687	—.0210	+.0029	.....	+.0140
5 239 000	59 000	—.0644	—.0784	—.0876	—.1050	—.0878	—.0687	—.0820	.....	.....	.....	.....
[d]5 480 000	61 700	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

[a] Test was stopped at 42 000 pounds per square inch Jan. 16, 4. 30 p. m. This load was left on the column overnight, and had dropped to 29 300 pounds per square inch at 9.30 a. m. Jan. 17. This drop was due to a leak in the inlet pipe, which was repaired and test resumed. Test was started at 10.30 a. m. Jan. 17 and finished at 4.30 p. m. Jan. 17.

[b] Stopped Jan. 16.

[c] Started Jan. 17.

[d] Maximum load.

NOTE.—Stations 19 to 24 give ± elongations in gauge length of 20 inches. All other stations given ± elongations in gauge length of 8 inches.

See continuation of table for lattice-bar readings.

TABLE 3.—Log of Test, Column U_2U_3, of Nickel Steel—Continued

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
177 600	2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
355 200	4 000	+.0002	−.0002	−.0001	.0000	+.0001	+.0003	+.0004	+.0001	−.0002	−.0002
532 800	6 000	.0000	−.0002	.0000	+.0001	+.0001	+.0001	+.0003	+.0004	−.0004	−.0002
888 000	10 000	−.0001	−.0005	.0000	+.0001	+.0001	+.0002	+.0003	+.0001	−.0002	−.0004
1 243 200	14 000	−.0002	−.0008	−.0001	+.0002	+.0001	+.0002	+.0003	+.0003	−.0007	−.0006
1 598 400	18 000	−.0003	−.0010	−.0003	+.0003	+.0001	+.0002	+.0003	+.0003	−.0007	−.0007
1 953 600	22 000	−.0004	−.0011	−.0002	+.0004	+.0001	+.0002	+.0002	+.0002	−.0010	−.0009
2 308 800	26 000	−.0008	−.0015	−.0002	+.0003	+.0001	+.0001	+.0003	+.0003	−.0012	−.0011
2 841 600	32 000	−.0011	−.0018	−.0004	+.0003	+.0002	+.0001	+.0002	+.0004	−.0014	−.0011
3 374 400	38 000	−.0015	−.0022	−.0004	+.0003	+.0001	.0000	.0000	+.0008	−.0016	−.0014
3 729 600	42 000	−.0016	−.0027	−.0005	+.0003	+.0004	−.0001	.0000	+.0005	−.0021	−.0018
2 600 000	29 300	−.0010	−.0017	−.0003	+.0002	+.0003	+.0001	+.0002	+.0004	−.0011	−.0011
3 729 600	42 000	−.0019	−.0027	−.0006	+.0001	+.0001	−.0001	−.0001	.0000	−.0023	−.0021
3 907 200	44 000	−.0021	−.0030	−.0006	+.0002	+.0001	−.0003	−.0002	+.0001	−.0024	−.0035
4 084 800	46 000	−.0021	−.0031	−.0006	+.0002	+.0002	−.0001	−.0002	+.0001	−.0027	−.0021
4 173 600	47 000	−.0021	−.0033	−.0005	+.0002	+.0003	−.0003	−.0004	+.0005	−.0028	−.0022
4 262 400	48 000	−.0022	−.0034	−.0006	+.0002	+.0003	−.0002	−.0003	+.0006	−.0031	−.0022
4 351 200	49 000	−.0023	−.0035	−.0006	+.0002	.....	.....	.....	.....	−.0030	−.0023
4 440 000	50 000	−.0025	−.0036	−.0005	+.0004	−.0001	−.0003	−.0005	+.0001	−.0036	−.0025
4 528 800	51 000	−.0020	−.0036	−.0004	+.0004	+.0005	−.0002	−.0006	+.0004	−.0035	−.0027
4 617 600	52 000	−.0021	−.0036	−.0006	+.0004	+.0005	−.0004	−.0007	+.0004	−.0038	−.0028
4 706 400	53 000	−.0022	−.0040	−.0005	+.0002	−.0003	−.0002	−.0007	+.0004	−.00-0	−.0032
4 795 200	54 000	−.0026	−.0042	−.0006	+.0003	+.0007	−.0002	−.0007	+.0005	−.0042	−.0035
4 884 000	55 000	−.0026	−.0046	−.0006	+.0004	+.0009	−.0002	−.0007	.0000	−.0044	−.0038
4 972 800	56 000	−.0028	−.0046	−.0006	+.0002	.....	.....	.....	.....	−.0044	−.0039
5 061 600	57 000	−.0032	−.0046	−.0006	+.0002	+.0011	.0000	−.0010	+.0003	−.0048	−.0043
5 150 400	58 000	−.0035	−.0050	−.0004	+.0002	+.0011	.0000	−.0014	−.0004	−.0050	−.0050
5 239 000	59 000	−.0043	−.0049	−.0021	+.0001	.....	.....	.....	.....	−.0052	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	15	16	17	18
177 600	2 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
355 200	4 000	.0000	.0000	+.0001	+.0002	+.0002	.0000	+.0003	+.0008
532 800	6 000	.0000	.0000	.0000	+.0004	+.0001	−.0002	+.0009	+.0016
888 000	10 000	.0000	.0000	−.0001	+.0002	.0000	−.0002	+.0016	+.0028
1 243 200	14 000	+.0001	+.0001	−.0001	+.0002	+.0002	−.0001	+.0026	+.0040
1 598 400	18 000	+.0001	+.0002	−.0001	+.0004	+.0002	−.0003	+.0035	+.0055
1 953 600	22 000	+.0001	+.0001	−.0001	+.0003	+.0002	−.0003	+.0043	+.0066
2 308 800	26 000	.0000	.0000	.0000	+.0004	+.0001	−.0002	+.0051	+.0079
2 841 600	32 000	+.0002	−.0001	.0000	+.0004	+.0002	−.0003	+.0067	+.0098
3 374 400	38 000	.0000	.0000	−.0001	+.0005	+.0001	−.0004	+.0083	+.0120
3 729 600	42 000	+.0002	−.0001	−.0003	+.0006	+.0002	−.0006	+.0093	+.0134
2 600 000	29 300	+.0001	.0000	.0000	+.0005	+.0002	−.0004	+.0073	+.0103
3 729 600	42 000	−.0001	−.0002	−.0005	+.0002	+.0001	−.0010	+.0098	+.0135
3 907 200	44 000	.0000	−.0001	−.0005	+.0004	+.0001	−.0019	+.0104	+.0143
4 084 800	46 000	+.0002	.0000	−.0005	+.0002	.0000	−.0008	+.0113	+.0150
4 173 600	47 000	+.0003	.0000	−.0005	+.0002	+.0001	−.0008	+.0118	+.0156
4 262 400	48 000	+.0004	+.0001	−.0005	+.0004	+.0002	−.0008	+.0124	+.0160
4 351 200	49 000	+.0004	+.0002	−.0005	+.0002	.....	.....	+.0133	+.0170
4 440 000	50 000	+.0006	+.0003	−.0005	+.0001	.0000	−.0007	+.0139	+.0176
4 528 800	51 000	+.0008	+.0005	−.0005	.0000	.0000	−.0009	+.0145	+.0186
4 617 600	52 000	+.0008	+.0005	−.0005	.0000	.0000	−.0007	+.0151	+.0194
4 706 400	53 000	+.0007	+.0005	−.0005	.0000	.0000	−.0008	+.0161	+.0203
4 795 200	54 000	+.0007	+.0006	−.0004	.0000	.0000	−.0007	+.0168	+.0212
4 884 000	55 000	+.0008	+.0008	−.0005	.0000	−.0001	−.0008	+.0177	+.0221
4 972 800	56 000	+.0007	+.0008	.....	.....	.....	.....	+.0206	+.0226
5 061 600	57 000	+.0008	+.0010	−.0011	.0000	−.0001	−.0008	+.0203	+.0230
5 150 400	58 000	+.0004	+.0010	−.0005	+.0001	.....	−.0008	+.0227	+.0233
5 239 000	59 000	.....	.....	.....	.....	.....	.....	.....	.....

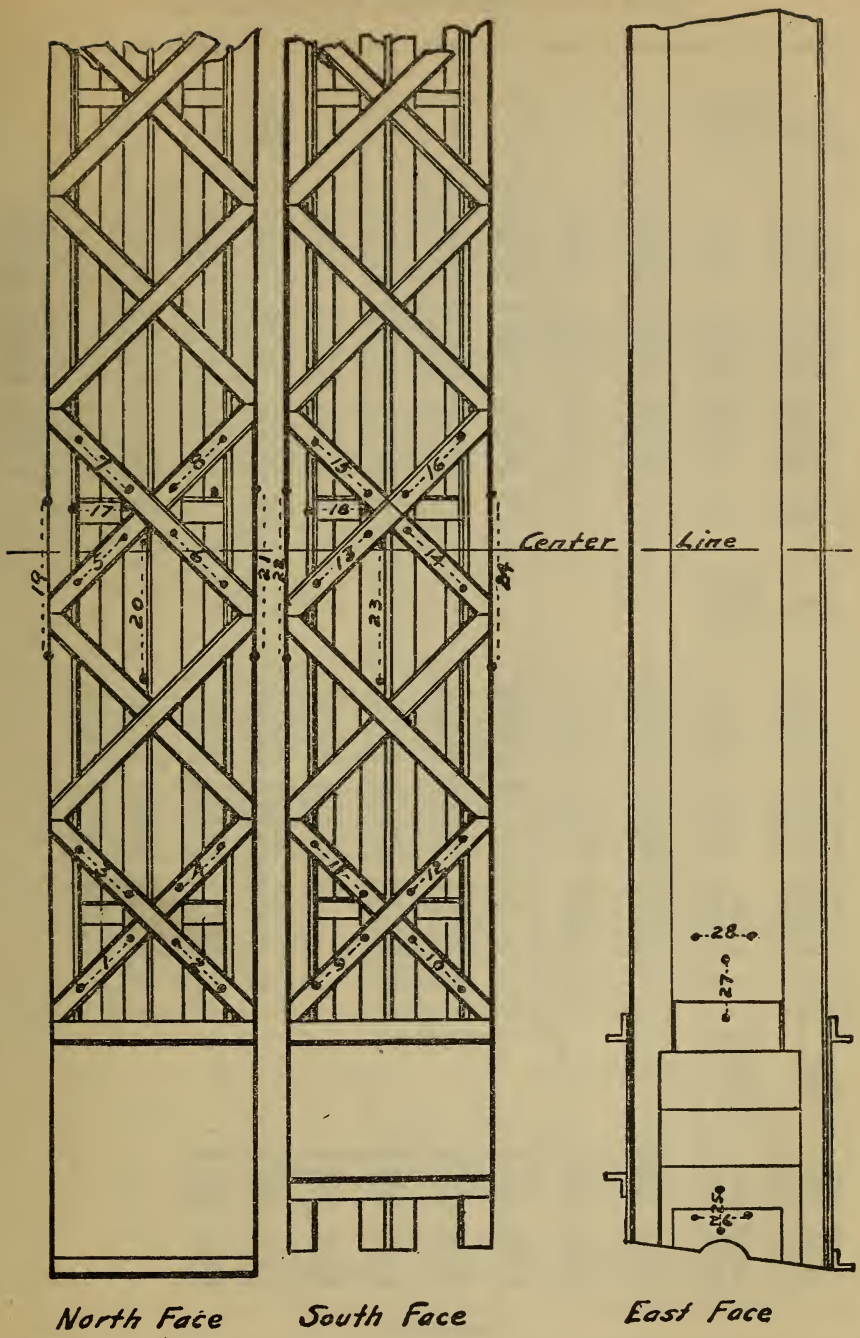


FIG. 3.—*Column U_2U_3 (nickel steel). Key to stations on Table 3*

TABLE 4.—Log of Test Column U_8U_9 of Nickel Steel

[Cumulative (±) Elongations at Stations Below. Tested Jan. 28, 1913. Time of Test, 9.30 a. m. to 4 p. m.]

Load, pounds	Load in pounds per square inch	47	48	49	50	51	52	Average, 47 to 52	53	54	55	56
110 300	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	Total compression not measured. Values below are total compression reckoned from "e."	0.0000
551 500	5 000	—.0030	—.0025	—.0028	—.0025	—.0026	—.0024	—.0026	—.0014	+.0004		+.0006
1 103 000	10 000	—.0062	—.0063	—.0061	—.0053	—.0057	—.0056	—.0059	—.0028	+.0008		+.0008
1 654 500	15 000	—.0094	—.0091	—.0091	—.0087	—.0090	—.0089	—.0090	—.0043	+.0013		+.0015
2 206 000	20 000	—.0128	—.0132	—.0123	—.0118	—.0125	—.0123	—.0125	—.0058	+.0017		+.0016
2 757 500	25 000	—.0160	—.0166	—.0154	—.0151	—.0160	—.0157	—.0158	—.0074	+.0022		+.0022
3 309 000	30 000	—.0196	—.0199	—.0188	—.0185	—.0195	—.0191	—.0192	—.0089	+.0025		+.0028
3 860 500	35 000	—.0232	—.0239	—.0225	—.0222	—.0236	—.0231	—.0231	—.0105	+.0029		+.0031
4 412 000	40 000	—.0280	—.0287	—.0272	—.0271	—.0287	—.0281	—.0280	—.0124	+.0035		+.0035
4 963 500	45 000	—.0316	—.0330	—.0308	—.0314	—.0333	—.0325	—.0321	—.0144	+.0041	e+o	+.0042
5 239 250	47 500	—.0338	—.0355	—.0329	—.0336	—.0362	—.0350	—.0345	—.0157	+.0045	0.0011	+.0046
5 515 000	50 000	—.0363	—.0384	—.0352	—.0364	—.0396	—.0378	—.0373	—.0171	+.0051	—.0028	+.0052
5 735 600	52 000	—.0384	—.0411	—.0373	—.0386	—.0431	—.0403	—.0398	—.0185	+.0055	—.0044	+.0062
5 956 200	54 000	—.0409	—.0446	—.0397	—.0416	—.0474	—.0432	—.0429	—.0202	+.0064	—.0064	+.0068
6 176 800	56 000	—.0441	—.0493	—.0431	—.0452	—.0536	—.0472	—.0471	—.0232	+.0077	—.0096	+.0080
6 397 400	58 000	—.0478	—.0557	—.0477	—.0504	—.0642	—.0535	—.0532	—.0271	+.0091	—.0142	+.0102
[a] 6 768 500	61 400	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	37	38	39	40	41	42	43	44	45	46
110 300	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
551 500	5 000	—.0010	—.0010	—.0012	—.0011	+.0014	+.0020	—.0010	—.0011	—.0012	—.0013
1 103 000	10 000	—.0025	—.0027	—.0026	—.0025	+.0027	+.0035	—.0026	—.0025	—.0026	—.0039
1 654 500	15 000	—.0041	—.0042	—.0040	—.0039	+.0042	+.0054	—.0040	—.0038	—.0041	—.0045
2 206 000	20 000	—.0056	—.0055	—.0055	—.0049	+.0058	+.0073	—.0054	—.0051	—.0056	—.0061
2 757 500	25 000	—.0071	—.0073	—.0069	—.0067	+.0073	+.0091	—.0069	—.0066	—.0071	—.0078
3 309 000	30 000	—.0088	—.0092	.....	—.0081	+.0089	+.0111	—.0083	—.0080	—.0087	—.0093
3 860 500	35 000	—.0106	—.0117	.....	—.0098	+.0104	+.0131	—.0097	—.0094	—.0104	—.0111
4 412 000	40 000	—.0127	—.0137	.....	—.0117	+.0121	+.0144	—.0112	—.0110	—.0122	—.0129
4 963 500	45 000	—.0151	—.0164	.....	—.0138	+.0137	+.0160	—.0128	—.0126	—.0145	—.0148
5 239 250	47 500	—.0165	—.0178	.....	—.0150	+.0147	+.0173	—.0137	—.0136	—.0156	—.0159
5 515 000	50 000	—.0178	—.0195	.....	—.0164	+.0159	.....	—.0146	—.0145	—.0172	—.0170
5 735 600	52 000	—.0191	—.0210	.....	—.0175	+.0195	.....	—.0153	—.0152	—.0185	—.0184
5 956 200	54 000	—.0206	—.0234	.....	—.0193	+.0197	.....	—.0166	—.0165	—.0204	—.0202
6 176 800	56 000	—.0234	—.0264	.....	—.0215	+.0197	.....	—.0182	—.0177	—.0230	—.0231
6 397 400	58 000	.....	—.0321	.....	—.0246	+.0197	.....	—.0206	—.0191	—.0271	.....

Load, pounds	Load in pounds per square inch	29	30	31	32	33	34	35	36
110 500	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
551 500	5 000	—.0002	.0000	.0000	—.0001	—.0001	.0000	—.0001	.0000
1 103 000	10 000	—.0004	—.0001	—.0002	—.0003	.0000	—.0002	.0000	—.0001
2 206 000	20 000	—.0006	—.0003	—.0006	—.0006	—.0001	—.0002	—.0001	—.0003
3 309 000	30 000	—.0010	—.0009	—.0005	—.0007	+.0003	—.0003	—.0001	—.0004
4 412 000	40 000	—.0014	—.0012	—.0006	—.0009	+.0001	.0000	.0000	—.0005
5 515 000	50 000	—.0021	—.0017	—.0005	—.0012	—.0002	—.0002	+.0005	—.0006
6 397 400	58 000	—.0026	—.0016	.0000	—.0011	—.0005	—.0002	—.0002	—.0006

[a] Maximum load.

NOTE.—Stations 47 to 52, inclusive, give ± elongations for gauge length of 20 inches. All other stations give ± elongations for gauge length of 8 inches.

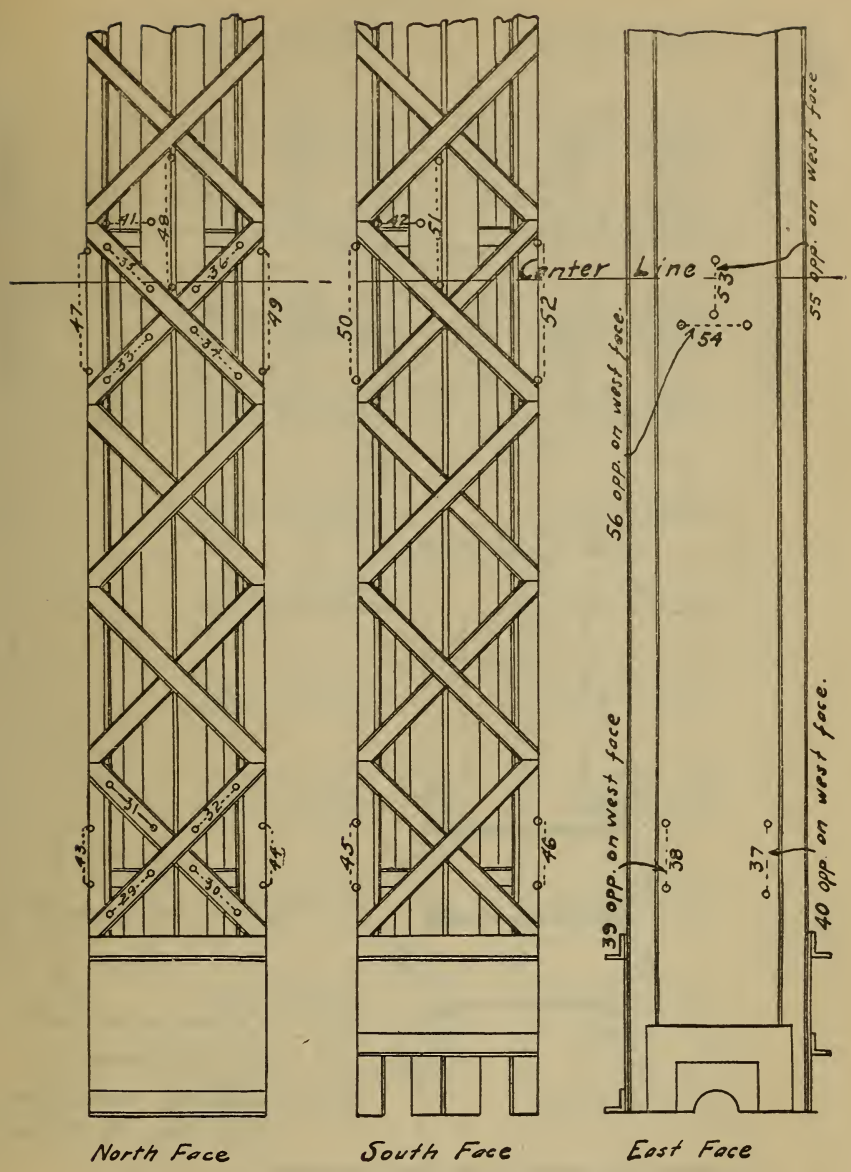


FIG. 4.—*Column U_8U_9 (nickel steel). Key to stations on Table 4*

TABLE 5.—Cumulative Components of Deflection at Mid-Length in Inches

[+ sign denotes displacement from unstrained axis: — sign denotes displacement toward unstrained axis.]

FOR COLUMN U_2U_3

Loads, pounds per square inch	Stations							
	a	b	c	d	e	f	g	h
10 000	+0.040	−0.007	+0.018	+0.018	+0.026	−0.015	+0.045	−0.002
22 000	+ .065	− .014	+ .030	+ .015	+ .033	− .021	+ .074	+ .001
32 000	+ .057	− .011	+ .041	+ .012	+ .042	− .019	+ .103	+ .002
47 000	+ .114	− .013	+ .050	+ .026	+ .052	− .020	+ .159	− .004
50 000	+ .136	− .010	+ .046	+ .030	+ .052	− .015	+ .170	− .009
55 000	+ .183	+ .002	+ .022	+ .046	+ .033	− .025	+ .234	− .017
59 000	+ .346	+ .048	− .091	+ .097	− .087	− .032	+ .416	− .054
FOR COLUMN U_8U_9								
1000	0	0	0	0	0	0	0	0
10 000	−0.005	0	+0.026	−0.007	+0.038	+0.017	+0.022	+0.005
20 000	+ .004	+0.002	+ .048	− .004	+ .061	+ .016	+ .042	+ .005
30 000	+ .015	+ .005	+ .065	+ .006	+ .083	+ .011	+ .059	+ .001
40 000	+ .027	+ .075	+ .081	+ .014	+ .105	− .003	+ .077	− .007
50 000	+ .045	+ .028	+ .093	+ .051	+ .119	− .030	+ .085	+ .025
54 000	+ .055	+ .049	+ .094	+ .073	+ .118	− .065	+ .109	− .047
58 000	+ .100	+ .128	+ .068	+ .166	+ .113	− .160	+ .159	+ .121

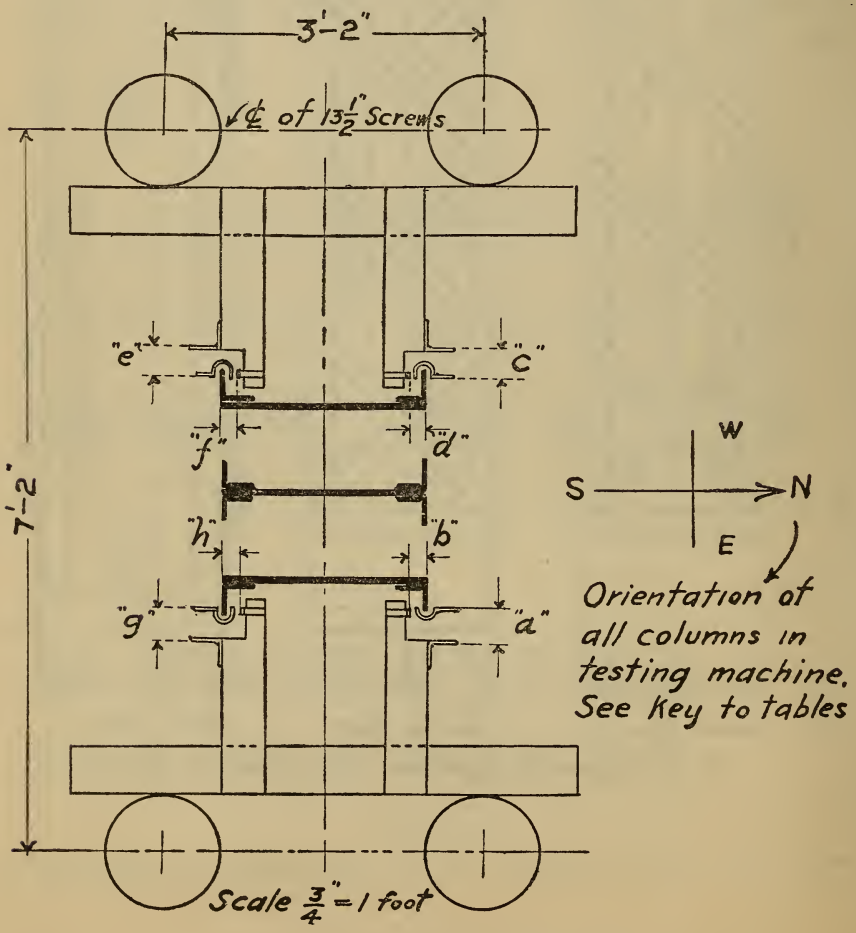


FIG. 5.—*Key to deflection stations above*

TABLE 6.—Log of Test, Column HC, of High-Carbon Steel

[Area=76.14. Cumulative Elongations (±) at Stations Below, Tested Jan. 12, 1914. Time of Test, 9 a. m. to 3.30 p. m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10	11	12	13
76 140	1000	0	0	0	0	0	0	0	0	0	0	0	0	0
152 280	2000	−0.0054	−0.0004	+0.0001	+0.0009	−0.0002	−0.0044	−0.0007	+0.0004	+0.0002	−0.0006	−0.0007	−0.0004	−0.0005
228 420	3000	− .0096	− .0006	+ .0001	+ .0012	− .0004	− .0089	− .0011	+ .0022	+ .0003	− .0009	− .0010	− .0008	− .0009
320 000	4200	− .0151	− .0011	− .0012	+ .0006	− .0009	− .0143	− .0016	+ .0018	0	− .0013	− .0011	− .0014	− .0015
380 700	5000	− .0176	− .0012	− .0021	+ .0002	− .0012	− .0165	− .0020	+ .0017	+ .0002	− .0016	− .0016	− .0018	− .0019
761 400	10 000	− .0319	− .0027	− .0139	− .0102	− .0027	− .0351	− .0038	+ .0003	− .0010	− .0033	− .0031	− .0051	− .0049
1 142 100	15 000	− .0529	− .0045	− .0272	− .0236	− .0042	− .0529	− .0054	− .0011	− .0025	− .0049	− .0044	− .0091	− .0090
1 522 800	20 000	− .0702	− .0057	− .0410	− .0389	− .0058	− .0699	− .0074	− .0023	− .0040	− .0064	− 0056	− 0125	− 0127
1 903 500	25 000	− 0909	− 0075	− 0582	− 0570	− .0076	− .0897	− .0094	− .0046	− .0058	− .0082	− .0078	− .0177	− .0180
2 284 200	30 000	− .1162	− .0097	− .0802	− .0791	− .0098	− .1154	− .0124	− .0069	− .0082	− .0102	− .0101	− .0236	− .0250
2 664 900	35 000	− .2474	− .0249	− .1769	− .1631	− .0245	− .2319	− .0139	− .0162	− .0172	− .0198	− .0122	.....	.....
a 2 683 500	35 300	.....	.....	.....	.....	.....	.....	.....	(c)	.....	.....	.....	.....	.....
b 2 920 000	38 400	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	14	15	16	17	18	19	20	21	22	23	24	25
76 140	1000	0	0	0	0	0	0	0	0	0	0	0	0
152 280	2000	−0.0008	−0.0032	+0.0001	0	−0.0003	−0.0002	−0.0003	0	+0.0078	−0.0003	−0.0004	−0.0003
228 420	3000	− .0007	− .0034	+ .0002	−0.0002	− .0003	− .0004	− .0009	−0.0007	+ .0062	− .0009	− .0010	− .0007
320 000	4200	− .0010	− .0033	− .0005	− .0003	− .0005	− .0012	− .0007	− .0007	+ .0072	− .0013	− .0011	− .0005
380 700	5000	− .0013	− .0037	− .0001	− .0002	− .0006	− .0009	− .0006	− .0008	+ .0074	− .0014	− .0008	− .0013
761 400	10 000	− .0029	− .0051	− .0005	− .0005	− .0021	− .0019	− .0012	− .0014	+ .0065	− .0030	− .0029	− .0028
1 142 100	15 000	− .0046	− .0068	− .0005	− .0009	− .0036	− .0035	− .0020	− .0021	+ .0042	− .0048	− .0045	− .0043
1 522 800	20 000	− .0066	− .0085	− .0007	− .0009	− .0057	− .0051	− .0026	− .0025	+ .0026	− .0069	− .0067	− .0071
1 903 500	25 000	− .0089	− .0100	− .0008	− .0010	− .0079	− .0066	− .0033	− .0034	+ .0002	− .0092	− .0087	− .0085
2 284 200	30 000	− .0145	− .0120	− .0009	− .0011	− .0113	− .0086	− .0038	− .0042	− .0038	− .0113	− .0111	− .0109
2 664 900	35 000	− .0334	− .0160	− .0013	− .0021	− .0283	.....	.....	.....	(c)	− .0127	− .0136	− .0138

a Maximum load. Buckled to north 2 inches at center. Lattice buckled at center.
b Maximum load after buckling=secondary failure.
c The initial reading was probably in error in taking measurements.

TABLE 6.—Log of Test, Column HC, of High-Carbon Steel—Continued

Load, pounds	Load in pounds per square inch	26	27	28	29	30	31	32	33	34	35	36	37
76 140	1000	0	0	0	0	0	0	0	0	0	0	0	0
152 280	2000	−0.0002	−0.0010	−0.0002	−0.0001	0	0	−0.0003	−0.0001	−0.0003	0	−0.0002	−0.0008
228 420	3000	− .0010	+ .0002	− .0002	− .0002	−0.0001	−0.0030	+ .0001	+ .0002	− .0002	0	0	0
320 000	4200	− .0005	0	− .0002	− .0002	0	− .0031	+ .0001	− .0001	− .0005	−0.0002	− .0001	− .0011
380 700	5000	− .0007	0	0	− .0002	− .0006	− .0034	− .0001	− .0003	− .0007	+ .0002	− .0002	− .0008
761 400	10 000	− .0024	− .0002	− .0006	− .0007	− .0003	− .0042	− .0013	− .0020	− .0012	+ .0002	− .0004	− .0012
1 142 100	15 000	− .0045	− .0005	− .0006	− .0010	− .0010	− .0047	− .0027	− .0029	− .0019	+ .0001	− .0005	− .0019
1 522 800	20 000	− .0057	− .0006	− .0009	− .0012	− .0011	− .0053	− .0037	− .0041	− .0024	0	− .0009	− .0020
1 903 500	25 000	− .0075	− .0007	− .0011	− .0014	− .0012	− .0063	− .0044	− .0052	− .0032	0	− .0011	− .0024
2 284 200	30 000	− .0103	− .0007	− .0013	− .0016	− .0013	− .0068	− .0052	− .0057	− .0044	0	− .0011	− .0027
2 664 900	35 000	− .0144	.....	.....	.....	.....	− .0080	− .0063	− .0085	− .0053	+ .0003	− .0013	− .0028

Load, pounds	Load in pounds per square inch	38	39	40	41	42	43	44	45	46	47	48	49
76 140	1000	0	0	0	0	0	0	0	0	0	0	0	0
152 280	2000	−0.0002	−0.0002	−0.0001	−0.0003	−0.0004	+0.0002	+0.0005	−0.0002	0	−0.0001	−0.0006	−0.0003
228 420	3000	− .0003	0	− .0001	− .0002	0	+ .0001	+ .0003	− .0001	+0.0001	+ .0002	− .0010	− .0005
320 000	4200	− .0004	− .0002	− .0001	− .0002	+ .0001	− .0001	+ .0023	+ .0010	+ .0003	+ .0005	− .0015	− .0007
380 700	5000	− .0004	− .0003	− .0001	− .0002	+ .0002	+ .0004	+ .0023	+ .0011	+ .0010	+ .0006	− .0016	− .0009
761 400	10 000	− .0011	− .0009	− .0004	− .0003	− .0008	+ .0004	+ .0023	+ .0011	+ .0006	+ .0008	− .0033	− .0015
1 142 100	15 000	− .0015	− .0013	− .0007	− .0003	0	+ .0005	+ .0023	+ .0011	+ .0004	+ .0014	− .0050	− .0033
1 522 800	20 000	− .0019	− .0016	− .0010	− .0004	+ .0006	+ .0006	+ .0027	+ .0015	+ .0008	.....	− .0066	− .0061
1 903 500	25 000	− .0023	− .0021	− .0012	− .0007	+ .0006	+ .0009	+ .0023	+ .0013	+ .0011	.....	− .0080	− .0073
2 284 200	30 000	− .0026	− .0024	− .0015	− .0009	+ .0005	+ .0007	+ .0023	+ .0011	+ .0009	.....	− .0099	− .0123
2 664 900	35 000	− .0028	− .0026	− .0018	− .0009	+ .0005	+ .0007	+ .0037	+ .0016	.....	.....	− .0156	− .0359

NOTE.—Stations 1, 3, 4, and 6 give compressions in inches in gauge length of 80 inches. Other stations give (±) elongations in inches in gauge length of 8 inches.

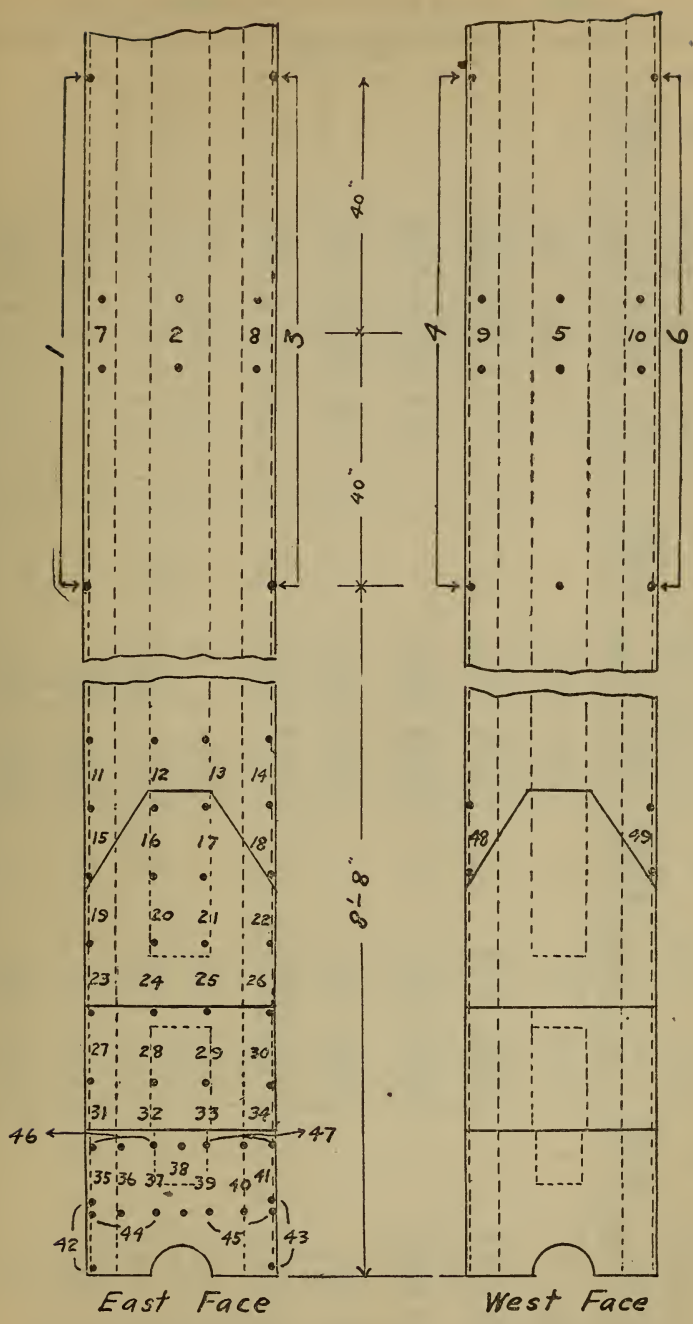


FIG. 6.—*Column HC (carbon steel). Key to stations on Table 6*

TABLE 7.—Log of Test, Column MY, of Mayari Steel

[Area=76.14. Cumulative (±) Elongations at Stations Below. Tested Jan. 13, 1914. Time of Test, 1.30 to 4.45 p. m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
76 140	1000	0	0	0	0	0	0	0	0	0	0
152 280	2000	−.0044	−.0002	−.0039	−.0011	−.0004	−.0071	−.0006	0	−.0001	−.0007
380 700	5000	−.0150	−.0010	−.0050	−.0073	−.0013	−.0170	−.0016	−.0006	−.0003	−.0017
761 400	10 000	−.0297	−.0024	−.0178	−.0201	−.0027	−.0380	−.0031	−.0020	−.0017	−.0032
1 142 100	15 000	−.0437	−.0040	−.0306	−.0339	−.0041	−.0460	−.0047	−.0034	−.0031	−.0047
1 522 800	20 000	−.0572	−.0054	−.0437	−.0474	−.0054	−.0598	−.0061	−.0046	−.0045	−.0062
1 903 500	25 000	−.0718	.....	−.0569	−.0613	.....	−.0740	.....	.....	.....	.....
2 284 200	30 000	−.0874	−.0083	−.0715	−.0760	−.0080	−.0884	−.0092	−.0076	−.0072	−.0090
2 664 900	35 000	−.1039	.....	−.0868	−.0903	.....	−.1037	.....	.....	.....	.....
3 045 600	40 000	−.1233	−.0118	−.1042	−.1061	−.0113	−.1193	−.0126	−.0112	−.0104	−.0120
3 426 300	45 000	−.1479	−.0142	−.1245	−.1245	−.0131	−.1387	−.0150	−.0133	−.0121	−.0140
3 578 600	47 000	−.1595	.....	−.1364	−.1335	.....	−.1485	.....	.....	.....	.....
[a]3 740 000	49 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[b]4 350 000	57 300	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[c]4 500 000	59 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[d]4 682 500	61 500	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	15	18	46	47	48	49	50	51	52	53
76 140	1000	0	0	0	0	0	0	0	0	0	0
152 280	2000	−.0002	+.0006	−.0002	−.0008	+.0005	+.0002	−.0003	0	−.0018	0
380 700	5000	−.0015	0	−.0010	−.0017	+.0002	0	−.0001	−.0002	−.0006	−.0001
761 400	10 000	−.0030	−.0014	−.0022	−.0034	+.0004	0	−.0008	−.0006	−.0016	−.0004
1 142 100	15 000	−.0044	−.0028	−.0036	−.0048	+.0004	−.0004	−.0010	−.0008	−.0020	−.0010
1 522 800	20 000	−.0062	−.0042	−.0049	−.0062	−.0004	−.0007	−.0007	−.0010	−.0018	−.0013
2 284 200	30 000	−.0092	−.0073	−.0076	−.0090	−.0010	−.0014	−.0012	−.0013	−.0021	−.0018
3 045 600	40 000	−.0128	−.0116	−.0104	−.0116	−.0017	−.0022	−.0015	−.0020	−.0022	−.0026
3 426 300	45 000	−.0156	−.0148	−.0122	−.0134	−.0022	−.0026	−.0014	−.0024	−.0032	−.0032

Load, pounds	Load in pounds per square inch	54	55	56	57	58	59	60	61	62	63
76 140	1000	0	0	0	0	0	0	0	0	0	0
152 280	2000	0	+.0004	+.0007	−.0002	+.0003	+.0001	+.0003	0	−.0007	−.0023
380 700	5000	+.0002	+.0001	+.0008	−.0004	+.0001	+.0006	+.0015	−.0006	−.0004	−.0019
761 400	10 000	+.0004	−.0003	+.0012	−.0002	−.0012	+.0010	+.0013	−.0008	−.0020	−.0022
1 142 100	15 000	0	−.0007	+.0014	0	+.0003	+.0015	+.0017	−.0006	−.0008	−.0025
1 522 800	20 000	−.0006	−.0010	+.0015	−.0006	+.0007	+.0012	+.0017	−.0004	−.0007	−.0037
2 284 200	30 000	−.0014	−.0016	+.0027	+.0002	+.0007	+.0019	+.0021	−.0002	−.0014	−.0033
3 045 600	40 000	−.0020	−.0024	+.0027	+.0016	+.0005	+.0023	+.0026	+.0002	−.0018	−.0033
3 426 300	45 000	−.0024	−.0028	+.0028	+.0020	+.0011	+.0031	+.0029	+.0004	−.0022	−.0037

Load, pounds	Load in pounds per square inch	64	65	66	67	68	69	70	71
76 140	1000	0	0	0	0	0	0	0	0
152 280	2000	0	−.0001	+.0024	0	−.0001	−.0006	+.0002	0
380 700	5000	+.0001	−.0003	+.0024	0	−.0002	−.0006	+.0003	+.0006
761 400	10 000	0	−.0027	+.0016	−.0001	−.0012	−.0006	+.0011	+.0011
1 142 100	15 000	−.0001	−.0031	+.0017	−.0005	−.0008	−.0012	+.0011	+.0016
1 522 800	20 000	−.0002	−.0029	+.0014	−.0007	−.0012	−.0014	+.0015	+.0022
2 284 200	30 000	−.0005	−.0037	+.0011	−.0014	−.0014	−.0008	+.0035	+.0032
3 045 600	40 000	−.0006	−.0035	+.0007	−.0022	−.0018	−.0001	+.0039	+.0042
3 426 300	45 000	−.0010	−.0041	+.0002	−.0028	−.0018	.0000	+.0055	+.0045

[a] Scaling of steel.
[b] Buckling north about 1 inch.
[c] Buckling north about 1½ inches.
[d] Buckling north about 2½ inches. Maximum load. Load fell off.

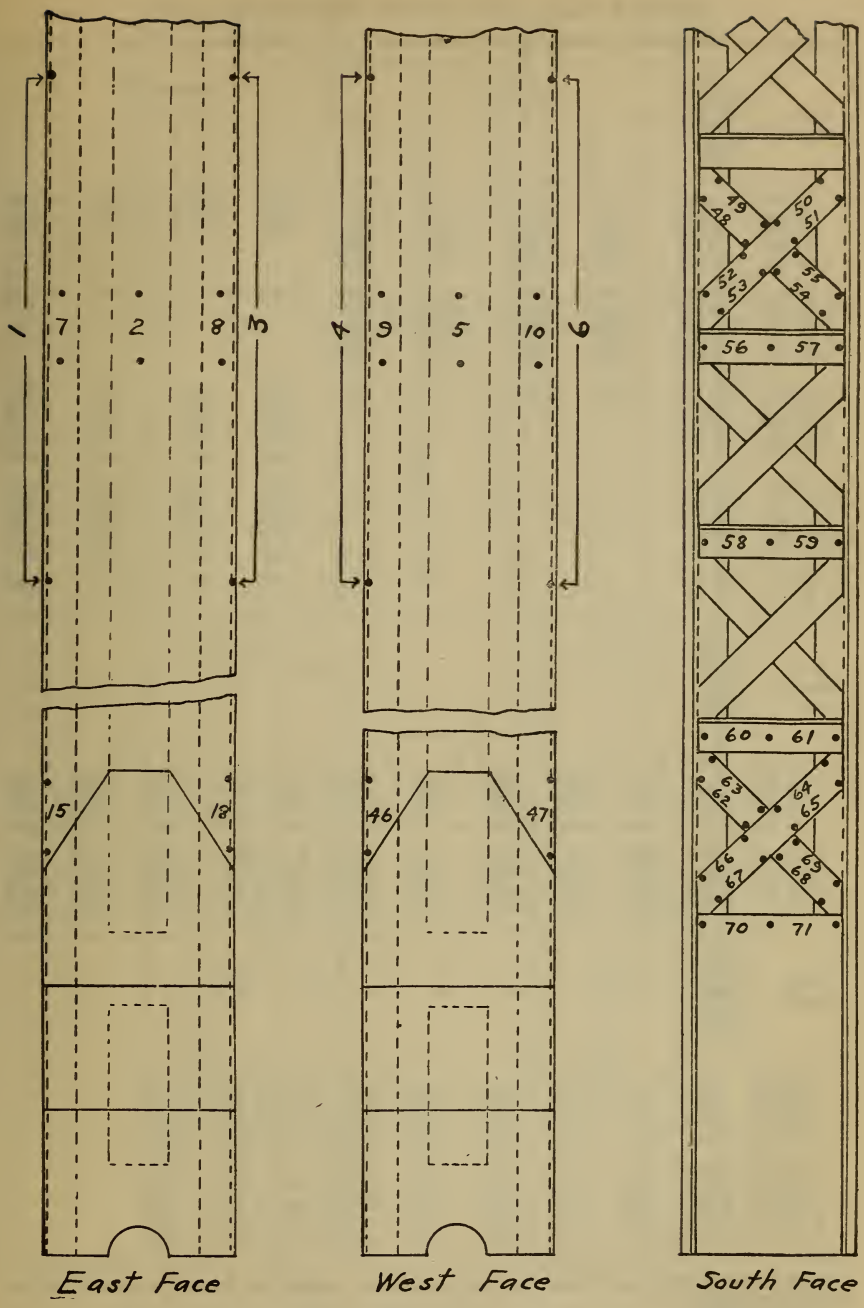


FIG. 7.—*Column MY (Mayari steel). Key to stations on Table 7*

TABLE 8.—Log of Test, Column 31HC, Carbon Steel

[Cumulative (±) Elongations at Stations Below. Tested July 7, 1914. Time of Test, 9.35 a. m. to 12.17 p. m.]

Load, pounds	Load in pounds per square inch	First run [a]				Second run			
		1	2	3	4	1	2	3	4
75 990	1000	0	0	0	0	−0.0070	−0.0028	−0.0019	−0.0061
379 950	5000	−.0145	−.0072	−.0071	−.0144	−.0203	−.0106	−.0090	−.0195
759 900	10 000	−.0291	−.0200	−.0200	−.0293	−.0345	−.0230	−.0219	−.0335
987 870	13 000	−.0390	−.0280	−.0276	−.0385	.....	.....	.....	.....
1 063 860	14 000	−.0417	−.0309	−.0304	−.0414	.....	.....	.....	.....
1 139 850	15 000	−.0440	−.0332	−.0330	−.0440	−.0481	−.0364	−.0350	−.0470
1 215 840	16 000	−.0477	−.0368	−.0360	−.0474	.....	.....	.....	.....
1 291 830	17 000	−.0508	−.0393	−.0386	−.0505	.....	.....	.....	.....
1 367 820	18 000	−.0542	−.0420	−.0410	−.0535	.....	.....	.....	.....
1 443 810	19 000	−.0573	−.0453	−.0439	−.0566	.....	.....	.....	.....
1 519 800	20 000	−.0608	−.0483	−.0470	−.0599	−.0618	−.0495	−.0475	−.0607
1 671 780	22 000	.....	.....	.....	.....	−.0688	−.0552	−.0536	−.0675
1 823 760	24 000	.....	.....	.....	.....	−.0760	−.0625	−.0598	−.0750
1 899 750	25 000	.....	.....	.....	.....	−.0807	−.0662	−.0631	−.0783
1 975 740	26 000	.....	.....	.....	.....	−.0852	−.0703	−.0670	−.0832
2 127 720	28 000	.....	.....	.....	.....	−.0951	−.0792	−.0749	−.0923
2 279 700	30 000	.....	.....	.....	.....	−.1063	−.0884	−.0840	−.1025
[b] 2 431 680	32 000	.....	.....	.....	.....	−.1210	−.1009	−.0942	−.1156
[c] 2 583 660	34 000	.....	.....	.....	.....	−.1400	−.1166	−.1083	−.1334
[d] 2 659 650	35 000	.....	.....	.....	.....	−.1545	−.1267	−.1180	−.1457
[e,f] 2 887 620	38 000	.....	.....	.....	.....	−.1902	−.1516	−.1418	−.1813
[g] 3 172 000	41 740	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	5	6	7	8	9	10	11	12	13	14
75 990	1000	0	0	0	0	0	0	0	0	0	0
[h] 75 990	1000	.0000	.0000	.0000	.0000	+.0001	−.0001	.0000	−.0001	.0000	+.0001
379 950	5000	−.0015	−.0012	−.0009	−.0008	−.0010	−.0016	+.0004	+.0003	+.0002	+.0005
759 900	10 000	−.0031	−.0026	−.0021	−.0022	−.0025	−.0031	+.0008	+.0008	+.0008	+.0009
1 139 850	15 000	−.0046	−.0042	−.0037	−.0035	−.0038	−.0048	+.0014	+.0012	+.0011	+.0013
1 519 800	20 000	−.0063	−.0057	−.0052	−.0051	−.0054	−.0064	+.0019	+.0018	+.0014	+.0019
1 899 750	25 000	−.0083	−.0078	−.0068	−.0068	−.0071	−.0082	+.0027	+.0025	+.0021	+.0024
2 279 700	30 000	−.0110	−.0107	−.0090	−.0093	−.0093	−.0108	+.0039	+.0035	+.0029	+.0034
2 659 650	35 000	−.0157	−.0156	−.0125	−.0133	−.0133	−.0161	+.0064	+.0058	+.0045	+.0057

Load, pounds	Load in pounds per square inch	15	16	17	18	19	20	21	22
75 990	1000	0	0	0	0	0	0	0	0
[h] 75 990	1000	+.0002	−.0004	+.0001	−.0003	−.0003	.0000	+.0001	+.0001
379 950	5000	−.0004	−.0014	−.0019	−.0014	−.0016	−.0013	−.0018	−.0009
759 900	10 000	−.0026	−.0037	−.0046	−.0033	−.0034	−.0032	−.0038	−.0028
1 139 850	15 000	−.0044	−.0056	−.0074	−.0050	−.0055	−.0049	−.0067	−.0046
1 519 800	20 000	−.0075	−.0078	−.0104	−.0065	−.0085	−.0076	−.0093	−.0058
75 990	1000	−.0001	−.0011	−.0011	+.0004	−.0010	−.0009	−.0001	+.0010
1 899 750	25 000	−.0100	−.0100	−.0134	−.0080	−.0109	−.0096	−.0120	−.0073
2 279 700	30 000	−.0129	−.0124	−.0166	−.0094	−.0141	−.0118	−.0153	−.0085
2 659 650	35 000	−.0162	−.0148	−.0200	−.0111	−.0195	−.0165	−.0207	−.0099

[a] First run taken from 1000 to 20 000, then back to 1000, and readings continued to maximum load on second run as shown.
[b] Scaling of steel.
[c] Loud "creaking."
[d] Load dropped to 2 600 000.
[e] Scaling freely at 2 730 000.
[f] This load was probably 2 787 620.
[g] Maximum load.
[h] Repetition for check readings.

NOTE.—Stations 1 to 4 give compression in inches in gauge length of 80 inches. Stations 5 to 14, inclusive, give (±) elongations in inches in gauge length of 8 inches, while stations 15 to 22, inclusive, are for 16 inches.

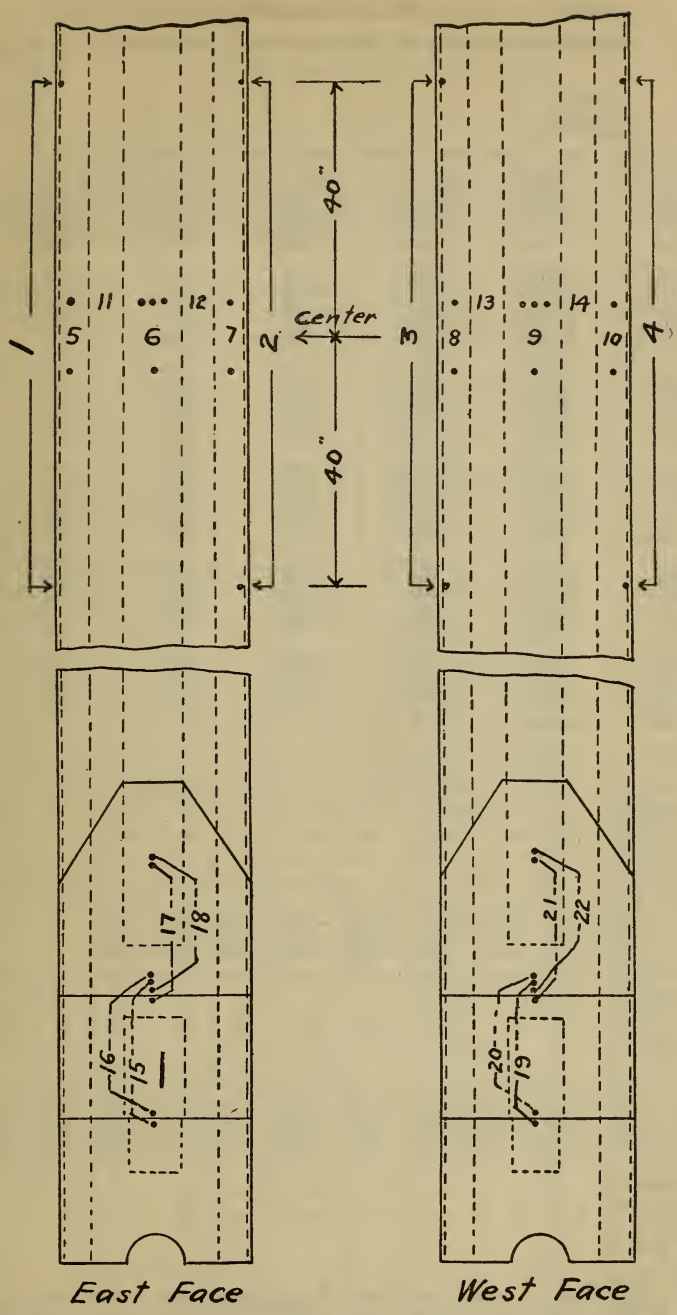


FIG. 8.—*Column 31HC (carbon steel). Key to stations on Table 8*

TABLE 9.—Log of Test, Column 32S, of Silicon Steel

[Cumulative (±) Elongations at Stations Below. Tested July 8, 1914. Time of Test, 10.15 a. m. to 12.30 p. m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	Average, 1 to 4
75 990	1000	0	0	0	0	0
379 950	5000	— .0065	— .0060	— .0150	— .0144	— .0105
759 900	10 000	— .0190	— .0192	— .0285	— .0276	— .0236
1 139 850	15 000	— .0321	— .0322	— .0414	— .0403	— .0365
1 215 840	16 000	— .0351	— .0352	— .0443	— .0430	— .0394
1 291 820	17 000	— .0380	— .0380	— .0469	— .0458	— .0422
1 367 820	18 000	— .0407	— .0410	— .0500	— .0486	— .0451
1 443 810	19 000	— .0430	— .0437	— .0527	— .0510	— .0476
1 519 800	20 000	— .0460	— .0463	— .0554	— .0537	— .0504
1 595 790	21 000	— .0480	— .0487	— .0578	— .0559	— .0526
1 671 780	22 000	— .0507	— .0519	— .0604	— .0588	— .0555
[a] 1 747 770	23 000	— .0537	— .0550	— .0635	— .0618	— .0585
1 823 760	24 000	— .0563	— .0578	— .0665	— .0646	— .0613
1 899 750	25 000	— .0592	— .0608	— .0697	— .0677	— .0644
1 975 740	26 000	— .0620	— .0637	— .0725	— .0704	— .0672
2 051 730	27 000	— .0649	— .0672	— .0757	— .0740	— .0705
2 127 720	28 000	— .0680	— .0706	— .0790	— .0771	— .0737
2 203 710	29 000	— .0714	— .0742	— .0829	— .0804	— .0772
2 279 700	30 000	— .0750	— .0772	— .0861	— .0837	— .0805
[b] 2 279 700	30 000	— .0750	— .0772	— .0863	— .0834	— .0805
[c] 2 431 680	32 000	— .0810	— .0840	— .0929	— .0900	— .0870
2 583 660	34 000	— .0887	— .0919	— .1013	— .0983	— .0951
2 735 640	36 000	— .0964	— .1003	— .1100	— .1067	— .1034
2 887 620	38 000	— .1057	— .1094	— .1203	— .1170	— .1131
3 039 600	40 000	— .1165	— .1208	— .1337	— .1286	— .1249
3 191 580	42 000	— .1309	— .1354	— .1513	— .1454	— .1408
[d] 3 343 560	44 000	— .1515	— .1575	— .1790	— .1725	— .1651
[e] 3 495 540	46 000	— .1945	— .2000	— .2370	— .2325	— .2160
[f] 3 700 700	48 700	.....	.....	.....	.....	[g] — .2986
[h] 3 829 900	50 400	.....	.....	.....	.....	[g] — .3990
[i] 4 020 000	52 800	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	15	16	17	18	19	20	21	22
75 990	1 000	0	0	0	0	0	0	0	0
759 900	10 000	—.0026	—.0027	—.0035	—.0026	—.0056	—.0039	—.0044	—.0029
1 519 800	20 000	—.0073	—.0063	—.0083	—.0051	—.0116	—.0081	—.0097	—.0049
2 279 700	30 000	—.0139	—.0121	—.0147	—.0069	—.0180	—.0130	—.0156	—.0067

Load, pounds	Load in pounds per square inch	23	24	25	26	27	28	29	30	31	32	33
75 990	1000	0	0	0	0	0	0	0	0	0	0	0
759 900	10 000	+.0012	—.0010	—.0005	+.0010	—.0008	—.0004	+.0007	—.0014	—.0003	+.0006	—.0015
1 519 800	20 000	+.0016	—.0018	—.0014	+.0017	—.0018	—.0014	+.0016	—.0026	—.0010	+.0018	—.0021
2 279 700	30 000	+.0026	—.0029	—.0021	+.0031	—.0028	—.0018	+.0032	—.0034	—.0016	+.0025	—.0030

Load, pounds	Load in pounds per square inch	34	35	36	37	37A	38	39	40	41	42	43
75 990	1000	0	0	0	0	0	0	0	0	0	0	0
759 900	10 000	—.0006	+.0001	—.0033	—.0002	+.0020	+.0006	—.0004	—.0007	+.0006	+.0001	—.0003
1 519 800	20 000	—.0012	+.0011	—.0029	—.0003	+.0039	+.0017	—.0009	—.0019	+.0013	—.0006	—.0009
2 279 700	30 000	—.0022	+.0017	—.0035	—.0014	+.0061	+.0022	—.0015	—.0026	+.0023	—.0013	—.0013

[a] Slight "creaking" sound.
[b] Check readings 17 minutes later.
[c] Pronounced scaling of steel.
[d] Creaking sounds.
[e] Scaling heavily; no deflection.
[f] Column deflects to east $\frac{3}{16}$ inch.
[g] Approximate as read with a scale.
[h] Center web buckling.
[i] Maximum load. Falls off to 3 980 000 and then to 3 910 000. Maximum deflection 2⅞ inches to east.

TABLE 9.—Log of Test, Column 32S, of Silicon Steel—Continued

Load, pounds	Load in pounds per square inch	44	45	46	47	48	49	50	51	52	52A
75 990	1000	0	0	0	0	0	0	0	0	0	0
759 900	10 000	+.0015	−.0002	−.0008	+.0005	−.0006	−.0009	+.0005	.0000	−.0006	+.0022
1 519 800	20 000	+.0020	−.0012	−.0021	+.0019	−.0009	−.0017	+.0012	.0000	−.0009	+.0044
2 279 700	30 000	+.0031	−.0022	−.0024	+.0034	−.0017	−.0033	+.0019	+.0002	−.0023	+.0059

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Other stations give (±) elongations in inches in gauge length of 16 inches.

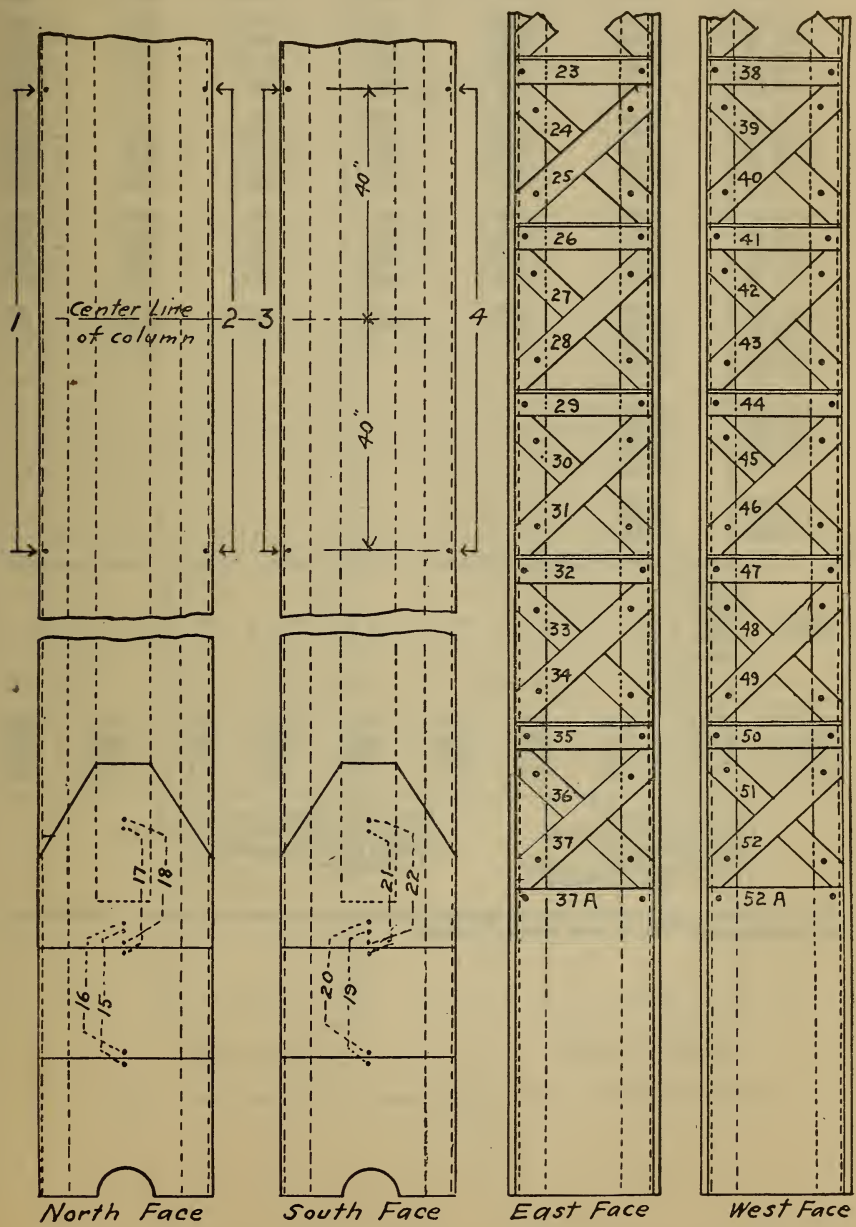


FIG. 9.—*Column 32S (silicon steel). Key to stations on Table 9*

TABLE 10.—Log of Test, Column 34CS, of Chrome Steel

[Cumulative (±) Elongations at Stations Below. Tested July 11, 1914. Time of Test, 9.30 a. m. to 11.12 a. m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	Average, 1 to 4
75 990	1 000	0	0	0	0	0
379 950	5 000	− .0113	− .0087	− .0103	− .0113	− .0104
759 900	10 000	− .0249	− .0225	− .0230	− .0250	− .0239
1 139 850	15 000	− .0387	− .0360	− .0364	− .0390	− .0375
1 519 800	20 000	− .0537	− .0507	− .0505	− .0530	− .0520
1 595 790	21 000	− .0565	− .0537	− .0538	− .0560	− .0550
1 671 780	22 000	− .0600	− .0567	− .0568	− .0588	− .0581
1 747 770	23 000	− .0633	− .0600	− .0600	− .0622	− .0614
1 823 760	24 000	− .0666	− .0630	− .0630	− .0658	− .0646
1 899 750	25 000	− .0698	− .0669	− .0668	− .0694	− .0682
1 975 740	26 000	− .0734	− .0700	− .0701	− .0730	− .0716
2 051 730	27 000	− .0774	− .0737	− .0739	− .0765	− .0754
[a] 2 127 720	28 000	− .0814	− .0773	− .0776	− .0805	− .0792
2 203 710	29 000	− .0853	− .0808	− .0813	− .0847	− .0830
2 279 700	30 000	− .0891	− .0850	− .0856	− .0890	− .0872
[b] 2 431 680	32 000	− .0986	− .0937	− .0944	− .0980	− .0962
2 583 660	34 000	− .1090	− .1034	− .1050	− .1084	− .1065
2 735 640	36 000	− .1220	− .1149	− .1158	− .1222	− .1187
2 887 620	38 000	− .1387	− .1281	− .1310	− .1396	− .1344
3 039 600	40 000	− .1620	− .1466	− .1513	− .1636	− .1559
[c] 3 191 580	42 000	− .2005	− .1763	− .1826	− .2040	− .1909
[d] 3 267 570	43 000	− .2300	− .1963	− .2035	− .2330	− .2157
3 495 540	46 000	.....	.....	.....	.....	.....
[e] 3 527 000	46 400	.....	.....	.....	.....	.....
3 577 000	47 100	.....	.....	.....	.....	.....
3 647 520	48 000	.....	.....	.....	.....	.....
[f] 3 670 000	48 300	.....	.....	.....	.....	.....
3 690 000	48 600	.....	.....	.....	.....	.....
3 715 000	48 900	.....	.....	.....	.....	.....
[g] 3 741 000	49 200	.....	.....	.....	.....	.....
[h] 3 680 000	48 500	.....	.....	.....	.....	.....
3 634 000	47 700	.....	.....	.....	.....	.....
[i] 3 400 000	44 700	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	15	16	17	18	19	20	21	22
75 990	1 000	0	0	0	0	0	0	0	0
759 900	10 000	− .0028	− .0024	− .0038	− .0028	− .0040	− .0026	− .0045	− .0035
1 519 800	20 000	− .0075	− .0063	− .0093	− .0059	− .0088	− .0071	− .0102	− .0065
2 279 700	30 000	− .0139	− .0110	− .0147	− .0083	− .0144	− .0122	− .0169	− .0090
3 039 600	40 000	− .0225	− .0180	− .0234	− .0109	− .0214	− .0187	− .0243	− .0119

[a] Creaking sounds.
[b] Center diaphragm scaling.
[c] Steel scaling freely.
[d] Center diaphragm buckling.
[e] Lacing begins to buckle.
[f] Center web buckles about ¼ inch.
[g] Load begins to fall off. Maximum load.
[h] Lacing buckles considerably.
[i] Last reading.

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Other stations give compressions in inches in gauge length of 16 inches.

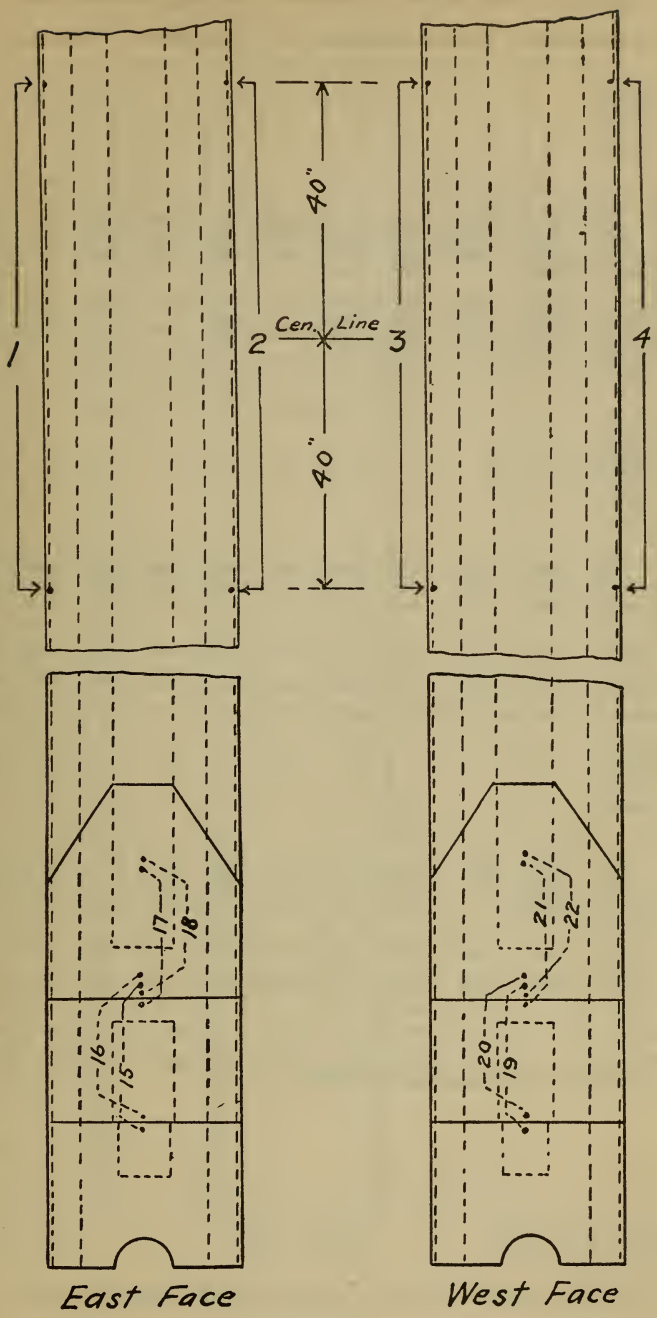


FIG. 10.—*Column 34CS (chrome steel). Key to stations on Table 10*

TABLE 11.—Log of Test, Column HC1, of High Carbon Steel

[Area=76.46 square inches. Cumulative Elongations (±) at Stations Below. Tested Jan. 17, 1914.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10	11
76 460	1000	0	0	0	0	0	0	0	0	0	0	0
152 920	2000	−.0022	−.0004	−.0029	−.0035	−.0002	−.0024	−.0004	−.0002	−.0007	−.0006	+.0001
382 300	5000	−.0112	−.0014	−.0118	−.0108	−.0008	−.0095	−.0010	−.0033	−.0011	−.0013	−.0019
764 600	10 000	−.0252	−.0028	−.0258	−.0247	−.0022	−.0225	−.0022	−.0044	−.0027	−.0024	−.0042
1 146 900	15 000	−.0389	.....	−.0403	−.0386	.....	−.0363	.....	.....	.....	.....	.....
1 529 200	20 000	−.0535	−.0054	−.0557	−.0537	−.0050	−.0506	−.0047	−.0069	−.0048	−.0057	−.0121
1 928 000	25 200	−.0702	.....	−.0733	−.0699	.....	−.0663	.....	.....	.....	.....	.....
2 293 800	30 000	−.0876	−.0090	−.0915	−.0868	−.0086	−.0825	−.0082	−.0091	−.0081	−.0080	−.0227
2 676 100	35 000	−.1193	.....	−.1197	−.1111	.....	−.1065	.....	.....	.....	.....	.....
3 460 000	[a] 45 300	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
3 220 000	([b])	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	12	13	14	15	16	17	18	19	20	21	22
76 460	1000	0	0	0	0	0	0	0	0	0	0	0
152 920	2000	−.0008	−.0004	−.0024	−.0002	−.0012	−.0003	−.0006	−.0014	+.0006	+.0008	+.0024
382 300	5000	−.0028	−.0029	−.0040	−.0008	−.0014	+.0002	−.0004	−.0016	−.0004	+.0002	+.0022
764 600	10 000	−.0061	−.0070	−.0069	−.0019	−.0022	−.0002	−.0012	−.0014	−.0016	−.0011	+.0002
1 146 900	15 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1 529 200	20 000	−.0144	−.0141	−.0139	−.0043	−.0020	−.0008	−.0012	−.0018	−.0043	−.0036	−.0038
1 928 000	25 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2 293 800	30 000	−.0240	−.0248	−.0238	−.0264	−.0014	−.0013	−.0016	−.0020	−.0066	−.0074	−.0056

Load, pounds	Load in pounds per square inch	23	24	25	26	27	28	29	30	31	32
76 460	1000	0	0	0	0	0	0	0	0	0	0
152 920	2000	+.0033	+.0001	+.0002	+.0004	−.0003	−.0005	−.0006	0	−.0011	−.0004
382 300	5000	+.0008	−.0001	0	+.0004	−.0001	−.0002	−.0002	0	−.0017	−.0005
764 600	10 000	−.0012	0	−.0004	+.0005	+.0006	−.0002	+.0002	+.0004	−.0027	−.0013
1 146 900	15 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1 529 200	20 000	−.0026	−.0010	−.0016	+.0001	+.0014	+.0007	−.0008	+.0010	−.0037	−.0041
1 928 000	25 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2 293 800	30 000	−.0062	−.0020	−.0020	−.0004	+.0008	+.0004	+.0018	+.0016	−.0055	−.0063

[a] Maximum load. Column buckled to west 4 inches. [b] Load fell off to 3 220 000 pounds.

NOTE.—Stations 1, 3, 4, and 6 give compressions in inches in gauge length of 80 inches. Other stations give (±) elongations in inches in gauge length of 8 inches.

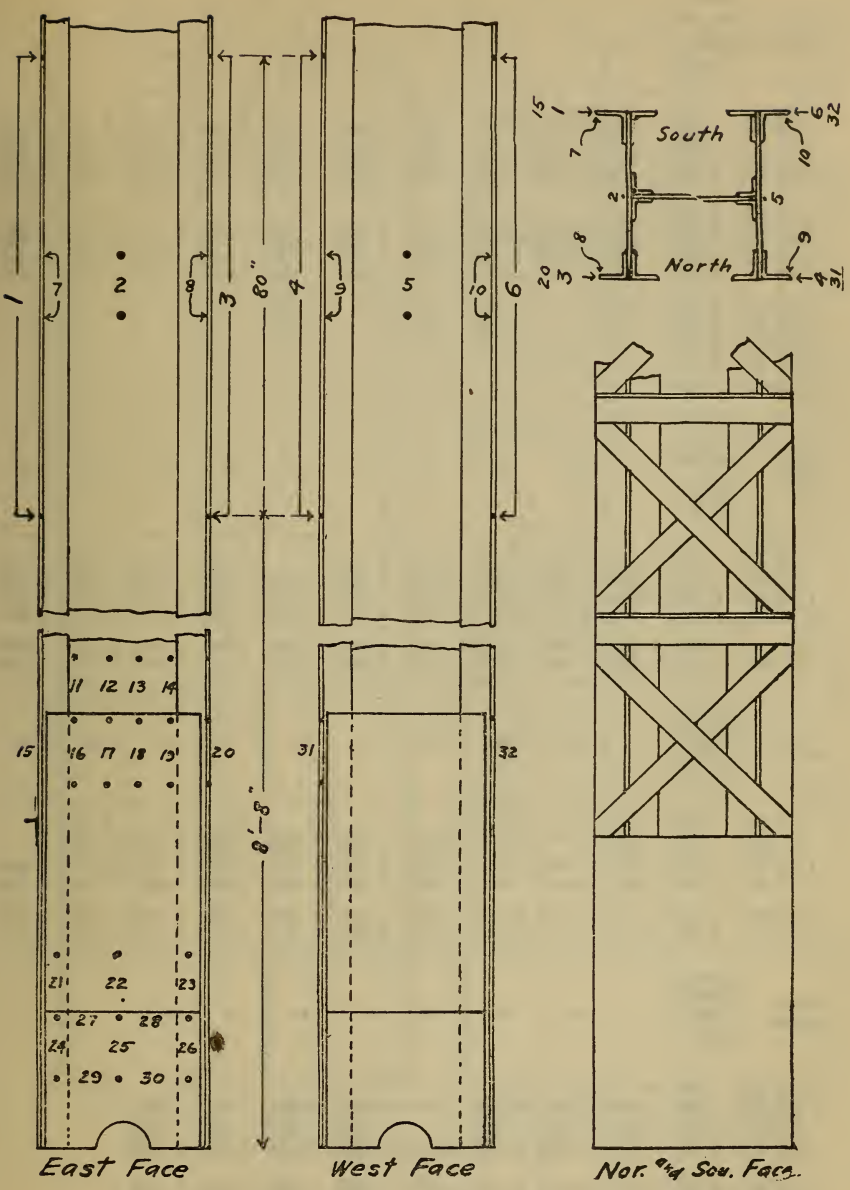


FIG. 11.—*Column HC1 (carbon steel). Key to stations on Table 11*

TABLE 12.—Log of Test, Column MY1, of Mayari Steel

[Area=76.46. Cumulative (±) Elongations at Stations Below. Tested Jan. 15, 1914.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
76 460	1000	0	0	0	0	0	0	0	0	0	0
382 300	5000	−.0112	−.0012	−.0100	−.0083	−.0012	−.0111	+.0022	−.0021	−.0009	−.0004
764 600	10 000	−.0252	−.0026	−.0227	−.0211	−.0021	−.0241	−.0018	−.0036	−.0020	−.0008
1 146 900	15 000	−.0391	−.0042	−.0360	−.0338	−.0039	−.0374	−.0016	−.0050	−.0032	−.0032
1 529 200	20 000	−.0519	−.0052	−.0486	−.0456	−.0046	−.0502	−.0018	−.0060	−.0046	−.0046
1 911 500	25 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2 293 800	30 000	−.0794	−.0084	−.0758	−.0722	−.0081	−.0786	−.0063	−.0087	−.0074	−.0072
2 676 100	35 000	−.0937	−.0100	−.0909	−.0854	−.0094	−.0932	−.0098	−.0100	−.0088	−.0082
3 058 400	40 000	−.1085	−.0116	−.1055	−.0987	−.0110	−.1081	−.0080	−.0112	−.0102	−.0096
3 440 700	45 000	−.1252	−.0134	−.1219	−.1127	−.0128	−.1239	−.0098	−.0128	−.0117	−.0118
3 823 000	50 000	−.1460	.....	−.1423	−.1279	.....	−.1428	(a)	.....	.....	.....
4 205 300	55 000	−.2011	.....	−.1895	−.1566	.....	−.1811	.....	.....	.....	.....
[b] 4 700 000	61 400	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4 800 000	62 800	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[c] 4 934 100	64 700	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[d] 2 520 000	33 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	37	38	39	40	41	42	43	44	45	46
76 460	1000	0	0	0	0	0	0	0	0	0	0
382 300	5000	+.0004	+.0010	+.0002	+.0010	+.0008	0	−.0001	−.0016	−.0002	+.0001
764 600	10 000	+.0004	+.0006	+.0008	+.0008	−.0016	+.0001	+.0002	−.0003	+.0002	+.0006
1 146 900	15 000	+.0008	+.0008	+.0006	+.0022	−.0008	+.0002	+.0006	−.0004	+.0009	+.0005
1 529 200	20 000	+.0010	+.0011	+.0008	+.0031	−.0002	+.0004	+.0005	−.0003	+.0010	+.0007
2 293 800	30 000	+.0014	+.0018	+.0008	+.0034	+.0008	+.0006	+.0011	−.0002	+.0014	+.0008
3 058 400	40 000	+.0016	+.0020	+.0004	+.0034	+.0010	+.0012	+.0014	−.0002	+.0018	+.0009

Load, pounds	Load in pounds per square inch	47	48	49	50	51	52	53	54	55	56
76 140	1000	0	0	0	0	0	0	0	0	0	0
382 300	5000	+.0008	+.0007	+.0019	+.0011	−.0001	+.0006	+.0007	+.0008	+.0005	0
764 600	10 000	+.0012	.....	+.0015	+.0005	+.0023	+.0007	+.0005	+.0013	+.0007	0
1 146 900	15 000	+.0016	+.0008	+.0015	+.0012	+.0011	+.0010	+.0007	+.0022	+.0006	+.0002
1 529 200	20 000	+.0022	+.0010	+.0017	+.0012	+.0015	+.0014	+.0009	+.0014	+.0012	+.0003
2 293 800	30 000	+.0032	+.0013	+.0031	+.0014	+.0021	+.0020	+.0009	+.0016	+.0017	+.0013
3 058 400	40 000	+.0046	+.0015	+.0031	+.0017	+.0025	+.0022	+.0017	+.0016	+.0022	+.0014

Load, pounds	Load in pounds per square inch	57	58	59	60	61	62	63	64
76 140	1000	0	0	0	0	0	0	0	0
382 300	5000	+.0003	+.0001	+.0004	+.0002	−.0010	−.0012	−.0008	−.0010
764 600	10 000	+.0004	+.0004	+.0008	.....	−.0022	−.0020	−.0030	−.0020
1 146 900	15 000	+.0007	+.0006	+.0012	.....	−.0030	−.0030	−.0041	−.0032
1 529 200	20 000	+.0011	+.0010	+.0018	+.0005	−.0039	−.0040	−.0051	−.0033
2 293 800	30 000	+.0017	+.0018	+.0030	+.0008	−.0054	−.0060	−.0074	−.0066
2 676 100	35 000	.....	.....	.....	.....	−.0072	−.0073	−.0085	−.0077
3 058 400	40 000	+.0022	+.0028	+.0048	+.0008	−.0082	−.0086	−.0097	−.0089
3 440 700	45 000	.....	.....	.....	.....	−.0094	−.0099	−.0110	−.0103

[a] The initial reading may have been in error. The observer complained of bad holes for gauge points.
[b] Primary failure. Buckling to west 2 inches.
[c] Maximum load.
[d] Load fell to 2 520 000. Total deflection to west, 7⅜ inches; to north, 1½ inches.

NOTE.—Stations 1, 3, 4, and 6 give compressions in inches in gauge lengths of 80 inches. Other stations give (±) elongations in inches in gauge lengths of 8 inches.

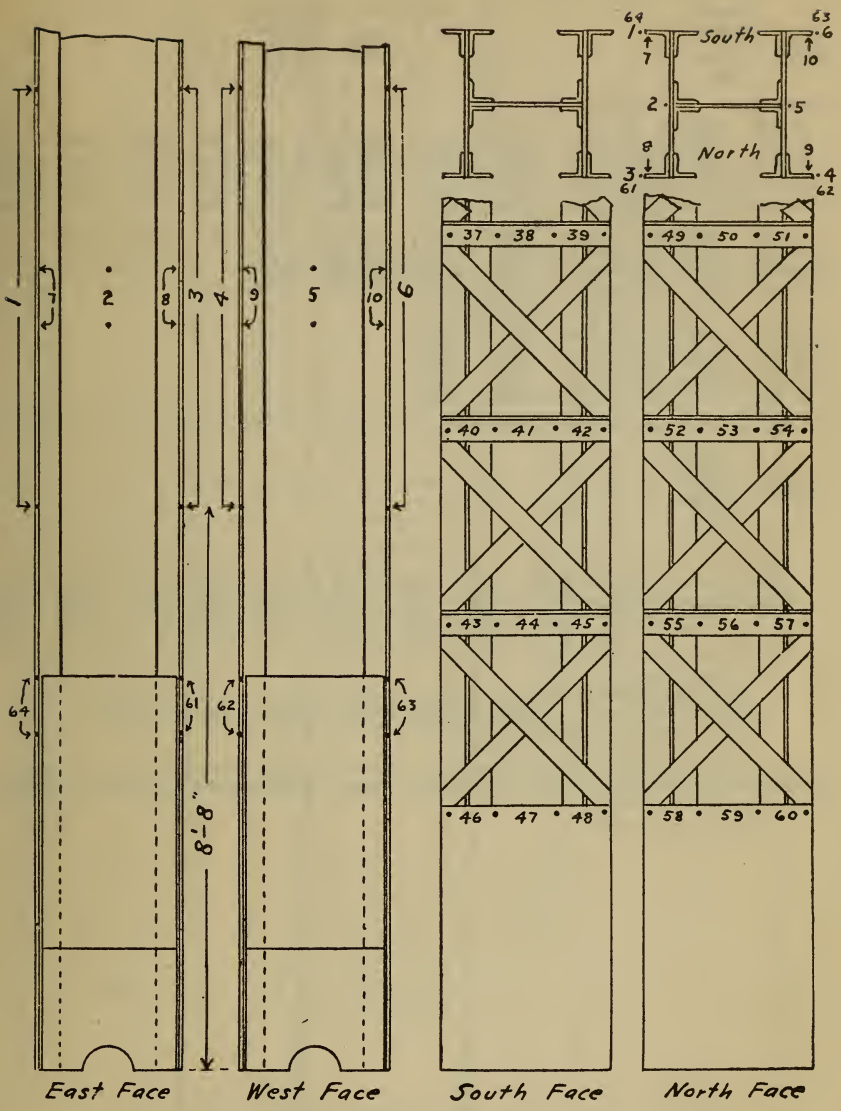


FIG. 12.—*Column MY1 (Mayari steel). Key to stations on Table 12*

TABLE 13.—Log of Test, Column 31HC1, Carbon Steel

[Cumulative (±) Elongation at Stations Below. Tested July 10, 1914. Time of test, 10.02 a. m. to 12 m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	Load, pounds	Load in pounds per square inch	1	2	3	4
76 260	1000	0	0	0	0			0	0	0	0
381 300	5000	−.0084	−.0077	−.0127	−.0143	2 440 320	32 000	−.0935	−.0927	−.0995	−.1003
762 600	10 000	−.0211	−.0212	−.0268	−.0283	[a] 2 592 840	34 000	−.1040	−.1027	−.1109	−.1111
991 380	13 000	−.0293	−.0296	−.0350	−.0365	2 745 360	36 000	−.1169	−.1152	−.1247	−.1250
1 067 640	14 000	−.0320	−.0320	−.0377	−.0391	2 897 880	38 000	−.1350	−.1330	−.1446	−.1447
1 143 900	15 000	−.0344	−.0350	−.0408	−.0422	3 050 400	40 000	−.1632	−.1599	−.1753	−.1754
1 220 160	16 000	−.0374	−.0381	−.0434	−.0451	3 202 920	42 000	−.2004	−.1987	−.2539	.....
1 296 420	17 000	−.0403	−.0407	−.0463	−.0477	[b] 3 355 440	44 000	.....	.....	.....	.....
1 372 680	18 000	−.0430	−.0437	−.0490	−.0505	[c] 3 507 960	46 000	.....	.....	.....	.....
1 448 940	19 000	−.0460	−.0462	−.0520	−.0533	[d] 3 550 000	46 550	.....	.....	.....	.....
1 525 200	20 000	−.0490	−.0490	−.0553	−.0564						
1 677 720	22 000	−.0550	−.0553	−.0613	−.0627						
1 830 240	24 000	−.0618	−.0620	−.0675	−.0687						
1 982 760	26 000	−.0684	−.0685	−.0750	−.0757						
2 135 280	28 000	−.0769	−.0761	−.0823	−.0835						
2 287 800	30 000	−.0843	−.0836	−.0906	−.0918						

Load, pounds	Load in pounds per square inch	15	16	17	18	19	20	21	22	23	24
76 260	1000	0	0	0	0	0	0	0	0	0	0
762 600	10 000	−.0023	−.0017	−.0066	−.0043	−.0024	−.0021	−.0078	−.0051	−.0022	+.0011
1 525 200	20 000	−.0060	−.0042	−.0164	−.0102	−.0068	−.0050	−.0186	−.0111	−.0052	+.0020
2 287 800	30 000	−.0103	−.0071	−.0308	−.0181	−.0114	−.0078	−.0332	([e])	−.0089	+.0030
3 050 400	40 000	−.0157	−.0098	−.0819	−.0374	−.0180	−.0106	−.0862	.....	−.0142	+.0045

[a] Scaling freely.
[b] Maximum deflection ⅛ inch to east.
[c] Center web buckling. Column deflects to north. Deflection to east disappearing.
[d] Maximum load. Deflection, 5 inches to north at 3 430 000.
[e] Movement beyond range of gauge.

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Stations 15 to 22 give compressions in inches in gauge length of 16 inches. Stations 23 and 24 give (±) elongations in inches in gauge length of 8 inches.

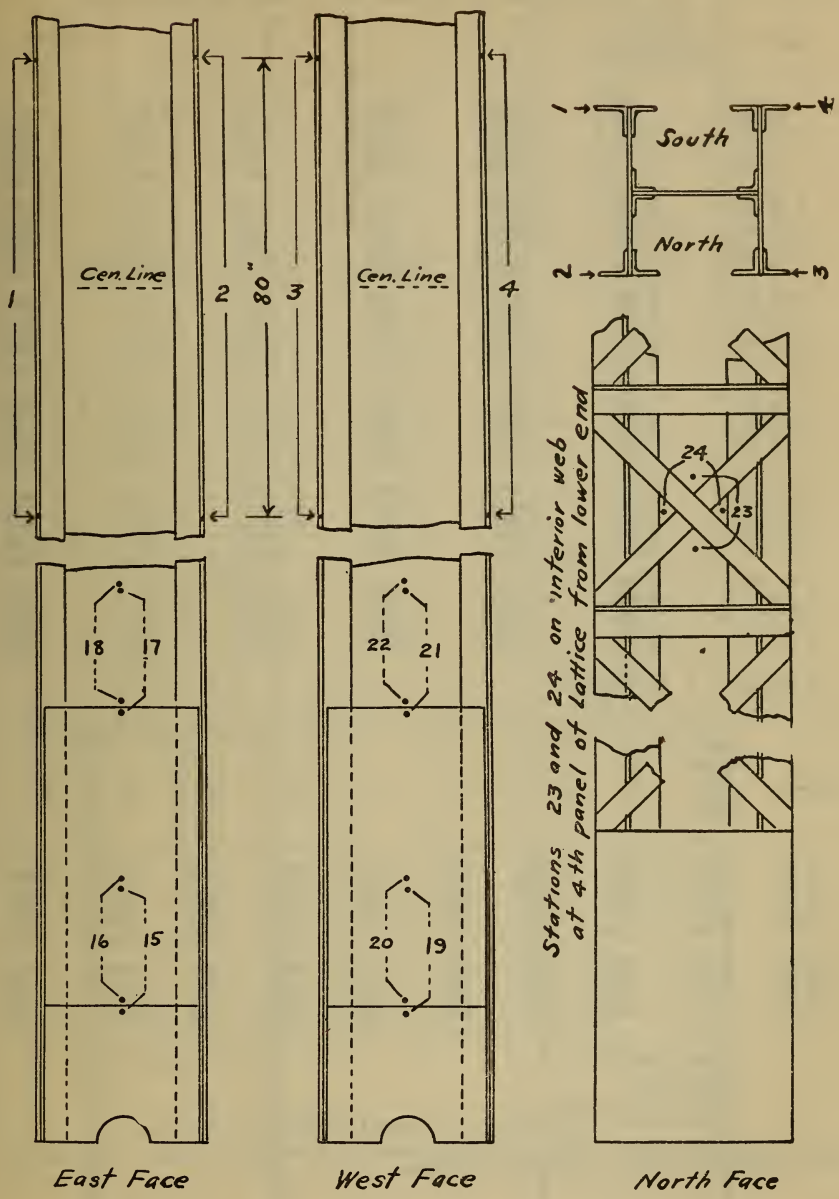


FIG. 13.—*Column 31HC1 (carbon steel). Key to stations on Table 13*

TABLE 14.—Log of Test, Column 32S1, of Silicon Steel

[Cumulative (±) Elongations at Stations Below. Tested July 9, 1914. Time of Test, 10.29 a. m. to 12.16 p. m.]

oad, pounds	Load in pounds per square inch	1	2	3	4	Average, 1 to 4
76 260	1000	0.0000	—0.0000	0.0000	0.0000	0.0000
381 300	5000	— .0084	— .0156	— .0119	— .0061	— .0105
762 600	10 000	— .0208	— .0319	— .0261	— .0168	— .0239
76 260	1000	— .0002	+ .0002	+ .0002	— .0002	.0000
762 600	10 000	— .0213	— .0323	— .0264	— .0172	— .0243
1 143 900	15 000	— .0342	— .0470	— .0404	— .0285	— .0375
1 220 160	16 000	— .0366	— .0500	— .0430	— .0313	— .0402
1 296 400	17 000	— .0395	— .0530	— .0456	— .0337	— .0430
1 372 700	18 000	— .0423	— .0557	— .0483	— .0363	— .0457
1 448 940	19 000	— .0450	— .0588	— .0513	— .0388	— .0485
1 525 200	20 000	— .0478	— .0620	— .0545	— .0413	— .0514
1 601 460	21 000	— .0503	— .0653	— .0574	— .0437	— .0542
1 677 720	22 000	— .0532	— .0683	— .0600	— .0463	— .0570
[a] 1 753 980	23 000	— .0563	— .0712	— .0630	— .0492	— .0599
1 830 240	24 000	— .0593	— .0747	— .0660	— .0523	— .0631
1 906 500	25 000	— .0623	— .0784	— .0695	— .0549	— .0663
76 260	1000	— .0012	— .0049	— .0027	— .0004	— .0023
[b] 1 906 500	25 000	— .0623	— .0787	— .0693	— .0549	— .0663
1 982 760	26 000	— .0650	— .0819	— .0721	— .0573	— .0691
2 135 280	28 000	— .0713	— .0884	— .0785	— .0629	— .0753
2 287 800	30 000	— .0780	— .0963	— .0858	— .0694	— .0824
2 440 320	32 000	— .0850	— .1043	— .0934	— .0755	— .0896
2 592 840	34 000	— .0930	— .1133	— .1014	— .0827	— .0976
2 745 360	36 000	— .1005	— .1230	— .1100	— .0904	— .1060
2 897 880	38 000	— .1090	— .1345	— .1198	— .0985	— .1155
3 050 400	40 000	— .1190	— .1464	— .1308	— .1072	— .1259
[c] 3 202 900	42 000	— .1308	— .1617	— .1460	— .1185	— .1393
3 355 440	44 000	— .1458	— .1812	— .1634	— .1330	— .1559
3 507 960	46 000	— .1670	— .2089	— .1890	— .1542	— .1798
3 660 480	48 000	— .1992	— .2524	— .2354	— .1830	— .2175
[d] 3 813 000	50 000	.....	.....	.....	.....	.....
[e] 3 949 000	51 700	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	15	16	17	18	19	20	21	22
76 260	1000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
762 600	10 000	— .0032	— .0023	— .0073	— .0060	— .0029	— .0027	— .0077	— .0050
76 260	1000	— .0000	+ .0003	— .0001	+ .0001	+ .0005	+ .0002	— .0004	+ .0003
762 600	10 000	— .0032	— .0022	— .0069	— .0052	— .0025	— .0025	— .0073	— .0044
1 143 900	15 000	— .0048	— .0035	— .0112	— .0079	— .0040	— .0035	— .0121	— .0072
1 525 200	20 000	— .0070	— .0050	— .0165	— .0108	— .0060	— .0046	— .0178	— .0102
1 906 500	25 000	— .0090	— .0064	— .0225	— .0139	— .0084	— .0061	— .0242	— .0134
76 260	1000	— .0003	+ .0005	— .0031	— .0004	+ .0004	+ .0007	— .0036	.0000
1 906 500	25 000	— .0093	— .0064	— .0227	— .0136	— .0083	— .0060	— .0243	— .0133

[a] Scaling slightly.
[b] Creaking.
[c] Scaling freely.
[d] Center web buckling.
[e] Maximum load. Load falls off. 4 by 4 inch L's buckling at lacing points.

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Other stations give (±) elongations in inches in gauge length of 16 inches.

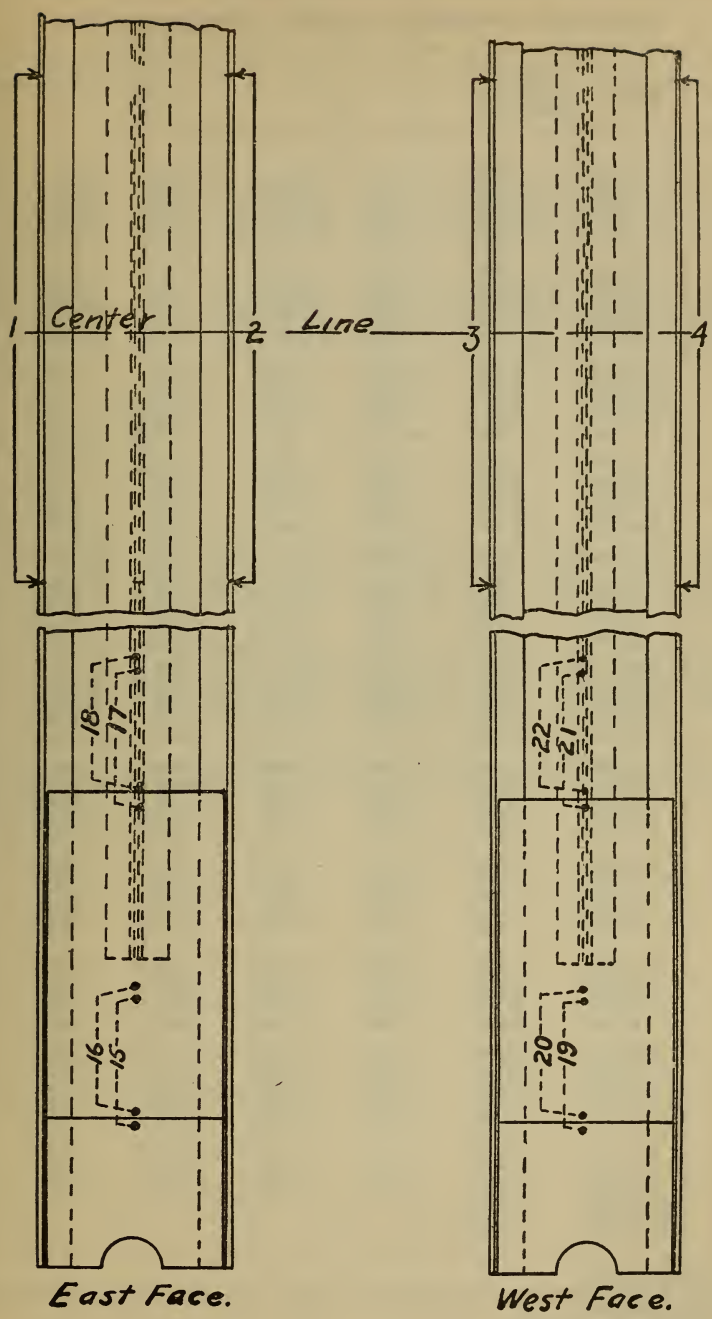


FIG. 14.—*Column 32S1 (silicon steel). Key to stations on Table 14*

TABLE 15.—Log of Test, Column 34CS1, of Chrome Steel

[Cumulative (±) Elongations at Stations Below. Tested July 13–14, 1914]

Time	Load, pounds	Load in pounds per square inch	1	2	3	4	Average, 1 to 4
July 13:							
2.20	76 260	1000	0	0	0	0	0
2.25	381 300	5000	−.0124	−.0071	−.0083	−.0130	−.0102
2.30	762 600	10 000	−.0262	−.0196	−.0208	−.0273	−.0235
	1 143 900	15 000	−.0400	−.0330	−.0343	−.0410	−.0371
2.46	76 260	1000	−.0007	−.0001	−.0009	−.0018	−.0009
3.00	381 300	5000	−.0132	−.0077	−.0094	−.0142	−.0111
3.03	762 600	10 000	−.0272	−.0206	−.0217	−.0280	−.0244
3.14	1 143 900	15 000	−.0408	−.0337	−.0348	−.0413	−.0377
	1 525 200	20 000	−.0557	−.0477	−.0487	−.0556	−.0519
	1 601 460	21 000	−.0592	−.0510	−.0517	−.0586	−.0551
	1 677 720	22 000	−.0623	−.0540	−.0543	−.0610	−.0579
	1 753 980	23 000	−.0657	−.0570	−.0576	−.0644	−.0612
	1 830 240	24 000	−.0690	−.0597	−.0608	−.0680	−.0644
	[a] 1 906 500	25 000	−.0730	−.0630	−.0640	−.0714	−.0679
3.41	76 260	1000	−.0060	−.0032	−.0043	−.0064	−.0050
3.43	381 300	5000	−.0182	−.0112	−.0133	−.0189	−.0154
	762 600	10 000	−.0321	−.0244	−.0257	−.0324	−.0287
3.50	1 143 900	15 000	−.0459	−.0377	−.0384	−.0450	−.0418
	1 525 200	20 000	−.0594	−.0505	−.0510	−.0582	−.0548
4.00	1 906 500	25 000	−.0733	−.0633	−.0642	−.0717	−.0681
4.12	[b] 76 260	1000	−.0064	−.0033	−.0047	−.0068	−.0053
July 14:							
9.36	86 000	1130	−.0104	−.0079	−.0098	−.0108	−.0097
9.45	76 260	1000	−.0100	−.0082	−.0102	−.0105	−.0097
10.18	381 300	5000	−.0229	−.0163	−.0177	−.0234	−.0201
10.22	762 600	10 000	−.0360	−.0290	−.0306	−.0370	−.0332
	1 143 900	15 000	−.0498	−.0424	−.0434	−.0497	−.0463
	1 525 200	20 000	−.0635	−.0557	−.0559	−.0623	−.0594
10.53	1 601 460	21 000	−.0662	−.0580	−.0586	−.0650	−.0620
	1 677 720	22 000	−.0689	−.0604	−.0613	−.0680	−.0647
10.55	1 753 980	23 000	−.0717	−.0631	−.0637	−.0707	−.0673
10.56	1 830 240	24 000	−.0744	−.0659	−.0662	−.0732	−.0699
10.57	1 906 500	25 000	−.0773	−.0687	−.0686	−.0760	−.0727
10.58	1 982 760	26 000	−.0802	−.0706	−.0713	−.0785	−.0752
11.00	2 059 020	27 000	−.0834	−.0750	−.0749	−.0821	−.0789
11.02	2 135 280	28 000	−.0873	−.0781	−.0782	−.0858	−.0824
11.04	2 211 540	29 000	−.0914	−.0820	−.0817	−.0898	−.0862
11.06	[c] 2 287 800	30 000	−.0955	−.0857	−.0853	−.0934	−.0900
11.20	76 260	1000	−.0153	−.0119	−.0140	−.0157	−.0142
	381 300	5000	−.0281	−.0205	−.0214	−.0280	−.0245
	762 600	10 000	−.0417	−.0336	−.0347	−.0418	−.0380
	1 143 900	15 000	−.0552	−.0464	−.0476	−.0545	−.0509
11.33	1 525 200	20 000	−.0688	−.0594	−.0604	−.0673	−.0640
11.40	1 906 500	25 000	−.0825	−.0730	−.0731	−.0807	−.0773
11.46	2 287 800	30 000	−.0967	−.0866	−.0861	−.0942	−.0909
11.50	[d] 2 440 320	32 000	−.1038	−.0931	−.0932	−.1018	−.0980
11.53	2 592 840	34 000	−.1131	−.1012	−.1011	−.1104	−.1065
11.56	[e] 2 745 360	36 000	−.1248	−.1113	−.1111	−.1214	−.1172
	2 897 880	38 000	−.1376	−.1230	−.1206	−.1337	−.1287
	[f] 3 050 400	40 000	−.1560	−.1380	−.1338	−.1499	−.1444
12.20	3 202 900	42 000	−.1780	−.1559	−.1507	−.1697	−.1636
	3 279 160	43 000	−.1917	−.1673	−.1619	−.1833	−.1761
	3 355 440	44 000	−.2115	−.1824	−.1749	−.2011	−.1925
12.30	3 431 680	45 000	−.2341	−.1999	−.1909	−.2237	−.2122
	[g] 3 876 000	50 800	.....	.....	.....	.....	.....
	[h] 3 948 000	51 800	.....	.....	.....	.....	.....
	[i] 3 920 000	51 400	.....	.....	.....	.....	.....

[a] No deflection visible.
[b] This load was left overnight and on the following morning corresponding readings were as below.
[c] Scaling of center web.
[d] Deflection of 0.005 inch north.
[e] Column scaling.
[f] Center web buckling. Deflection, 0.05 inch north.
[g] Lacing buckles.
[h] Maximum load. Load falls off.
[i] Last reading.

TABLE 15.—Log of Test, Column 34CS1, of Chrome Steel—Continued

Time	Load, pounds	Load in pounds per square inch	15	16	17	18	19	20	21	22
July 13:										
2.10....	76 260	1000	0	0	0	0	0	0	0	0
2.35....	762 600	10 000	−.0042	−.0027	−.0074	−.0049	−.0036	−.0024	−.0076	−.0054
2.50....	76 260	1000	−.0011	−.0001	−.0011	−.0002	−.0009	−.0002	−.0021	−.0004
3.10....	762 600	10 000	−.0042	−.0029	−.0085	−.0052	−.0042	−.0024	−.0091	−.0053
	1 525 200	20 000	−.0086	−.0059	−.0184	−.0110	−.0083	−.0059	−.0192	−.0110
July 14:										
10.14...	76 260	1000	−.0010	+.0004	−.0042	−.0007	−.0010	.0000	−.0051	−.0007
10.30...	762 600	10 000	−.0043	−.0022	−.0108	−.0054	−.0034	−.0026	−.0124	−.0057
10.48...	1 525 200	20 000	−.0079	−.0050	−.0199	−.0109	−.0079	−.0057	−.0213	−.0111
11.14...	2 287 800	30 000	−.0120	−.0077	−.0311	−.0175	−.0118	−.0076	−.0326	−.0175
12.11...	3 050 400	40 000	−.0174	−.0100	−.0509	−.0290	−.0181	−.0107	−.0562	−.0292

Time	Load, pounds	Load in pounds per square inch	26	27	28	29	30	31	32	33	34
July 13:											
2.13.....	76 260	1000	0	0	0	0	0	0	0	0	0
2.30.....	762 600	10 000	+.0006	−.0018	−.0016	+.0008	−.0022	−.0013	+.0005	−.0020	−.0015
2.48.....	76 260	1000	−.0006	+.0003	.0000	+.0002	.0000	.0000	+.0002	.0000	−.0001
3.04.....	762 600	10 000	+.0003	−.0016	−.0018	+.0008	−.0021	−.0018	+.0014	−.0021	−.0013
3.20.....	1 525 200	20 000	+.0013	−.0036	−.0035	+.0013	−.0044	−.0034	+.0014	−.0043	−.0032
July 14:											
10.06....	76 260	1000	−.0003	−.0004	−.0005	−.0003	−.0004	−.0003	.0000	+.0009	+.0010
10.24....	762 600	10 000	+.0012	−.0016	−.0015	+.0009	−.0018	−.0014	+.0012	−.0012	−.0006
10.40....	1 525 200	20 000	+.0023	−.0031	−.0029	+.0020	−.0035	−.0027	+.0021	−.0037	−.0023
11.08....	2 287 800	30 000	+.0031	−.0049	−.0042	+.0033	−.0053	−.0039	+.0034	−.0054	−.0036
12.09....	3 050 400	40 000	+.0056	−.0067	−.0054	+.0041	−.0072	−.0056	+.0046	−.0068	−.0051

NOTE.—Stations 1 to 4 give compressions in inches in gauge length of 80 inches. Stations 15 to 22, inclusive give relative compressions in gauge lengths of 16 inches for various repetitions of loadings and referred to a common origin.

2820°—18——4

TABLE 15.—Log of Test, Column 34CS1, of Chrome Steel—Continued

Time	Load, pounds	Load in pounds per square inch	35	36	37	37A	41	42	43	44	45
July 13 [a]	76 260	1000	0	0	0	0	0	0	0	0	0
	762 600	10 000	+.0003	−.0017	−.0012	+.0016	+.0008	−.0016	−.0012	+.0011	−.0016
	76 260	1000	−.0002	.0000	−.0004	+.0004	+.0004	.0000	+.0001	+.0001	−.0002
	762 600	10 000	−.0001	−.0014	−.0016	+.0016	+.0007	−.0013	−.0013	+.0009	−.0018
	1 525 200	20 000	+.0016	−.0029	−.0026	+.0031	+.0015	−.0036	−.0030	+.0019	−.0037
July 14 [a]	76 260	1000	+.0004	+.0004	+.0003	+.0015	−.0002	−.0004	.0000	−.0001	−.0004
	762 600	10 000	+.0006	−.0007	−.0007	+.0025	+.0018	−.0012	−.0007	+.0017	−.0016
	1 525 200	20 000	+.0018	−.0020	−.0019	+.0041	+.0024	−.0029	−.0023	+.0027	−.0034
	2 287 800	30 000	+.0029	−.0035	−.0030	+.0058	+.0034	−.0046	−.0037	+.0039	−.0053
	3 050 400	40 000	+.0037	−.0051	−.0035	+.0082	+.0048	−.0058	−.0051	+.0048	−.0072

Time	Load, pounds	Load in pounds per square inch	46	47	48	49	50	51	52	52A
July 13 [a]	76 260	1000	0	0	0	0	0	0	0	0
	762 600	10 000	−.0009	+.0015	−.0017	−.0014	+.0001	−.0006	−.0011	+.0013
	76 260	1000	−.0001	+.0001	−.0002	−.0006	−.0001	+.0005	−.0006	−.0002
	762 600	10 000	−.0018	+.0008	−.0016	−.0016	+.0003	−.0004	−.0016	+.0012
	1 525 200	20 000	−.0032	+.0014	−.0040	−.0030	+.0018	−.0014	−.0029	+.0028
July 14 [a]	76 260	1000	−.0001	−.0002	(b)	+.0008	+.0003	+.0014	−.0003	+.0004
	762 600	10 000	−.0010	+.0018	.....	−.0010	+.0010	+.0003	−.0012	+.0018
	1 525 200	20 000	−.0028	+.0024	.....	−.0023	+.0020	−.0006	−.0023	+.0036
	2 287 800	30 000	−.0039	+.0037	.....	−.0035	+.0033	−.0022	−.0029	+.0053
	3 050 400	40 000	−.0051	+.0046	.....	−.0045	+.0041	−.0034	−.0040	+.0079

[a] Time approximately the same as for stations 26–34.
[b] Last run for bar 48 rejected on account of doubtful initial reading.

NOTE.—Stations 26 to 52A on lattice bars give (±) elongations in inches in gauge length of 16 inches for repetitions of loadings as shown in order to show effect on lattice bars.

CHECK READINGS, STATIONS 1 TO 4

[8-inch Berry Guage on Outer Point of Angles at Center of 80-inch Rods]

Time	Load, pounds	Load in pounds per square inch	1	2	3	4	
July 13:							
4.45	76 260	1000	0.0198	0.0276	0.0184	0.0142	Standard bar 0.0288
July 14:							
9.35	86 000	1130	.0198	.0276	.0185	.0143	Standard bar 0.0289
9.45	76 260	1000	.0198	.0277	.0186	.0141	Standard bar 0.0289

NOTE.—Stations 1 to 4 give compressions in inches in gauge length of 80 inches for various repetitions of loadings.

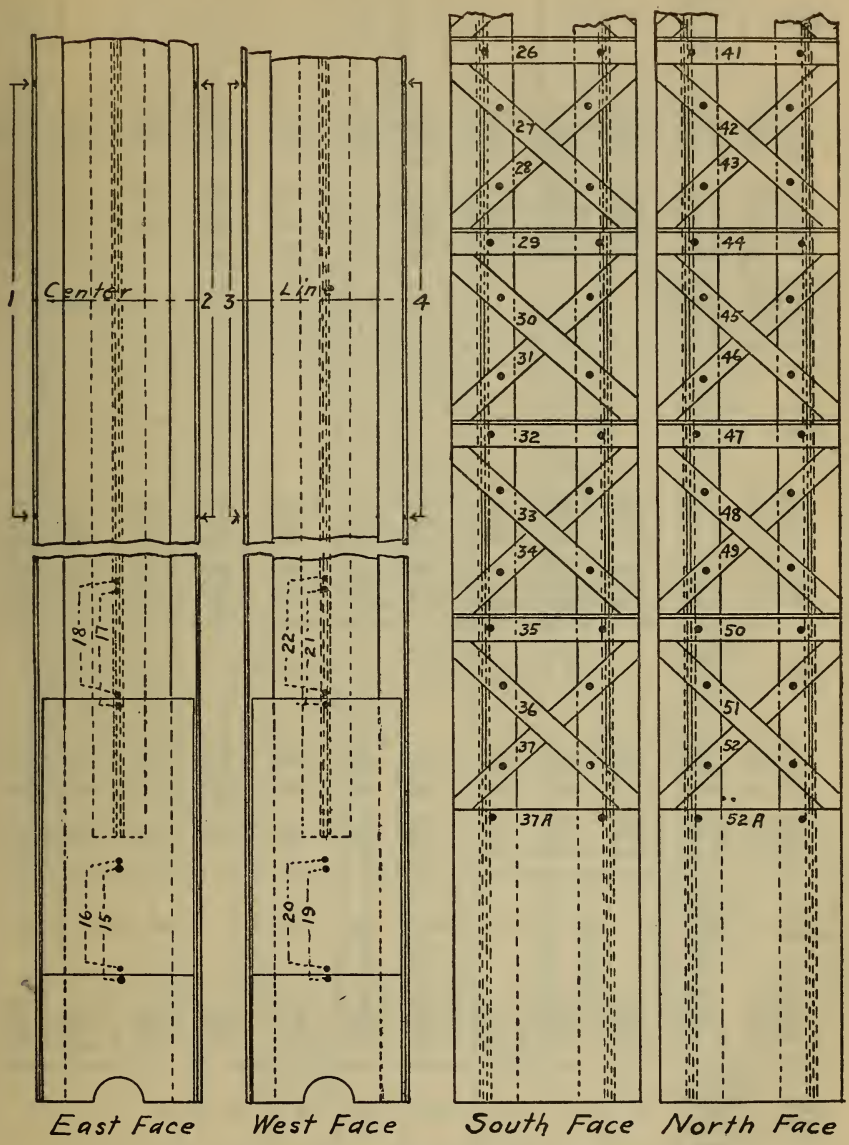


FIG. 15.—*Column 34CS1 (chrome steel). Key to stations on Table 15*

TABLE 16.—Log of Test, Column US_2MS_1, Carbon Steel

[Cumulative (±) Elongations at Stations Below. Tested Jan. 27, 1914. Time of Test, 10 a. m. to 12 m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
55 170	1000	0	0	0	0	0	0	0	0	0	0
275 850	5000	−.0110	−.0045	−.0087	−.0163	−.0014	−.0007	−.0011	−.0014	−.0022	−.0012
551 700	10 000	−.0263	−.0145	−.0207	−.0335	−.0029	−.0021	−.0024	−.0030	−.0032	−.0027
827 550	15 000	−.0429	−.0270	−.0332	−.0507	−.0051	−.0035	−.0033	−.0046	−.0050	−.0047
965 500	17 500	−.0525	−.0331	−.0396	−.0600	.....	.....	.....	.....	.....	.....
1 103 400	20 000	−.0620	−.0400	−.0464	−.0699	−.0072	−.0051	−.0050	−.0068	−.0076	−.0061
1 241 300	22 500	−.0730	−.0467	−.0538	−.0807	.....	.....	.....	.....	.....	.....
1 379 250	25 000	−.0860	−.0546	−.0609	−.0934	−.0103	−.0065	−.0062	−.0090	−.0100	−.0084
1 517 200	27 500	−.1000	−.0614	−.0686	−.1071	.....	.....	.....	.....	.....	.....
1 655 100	30 000	−.1190	−.0690	−.0771	−.1281	−.0159	−.0082	−.0076	−.0118	−.0125	−.0115
[a] 1 853 000	33 600	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[b] 1 894 000	34 300	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[c] 1 801 000	32 700	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[d] 1 848 000	33 500	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[e] 1 280 000	23 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	15	16	17	18	19	20
55 170	1000	0	0	0	0	0	0	0	0	0	0
275 850	5000	+.0006	+.0004	−.0010	−.0007	−.0006	+.0003	+.0004	−.0009	−.0010	−.0011
551 700	10 000	+.0008	+.0012	−.0019	−.0021	−.0018	+.0008	+.0008	−.0019	−.0023	−.0024
827 550	15 000	+.0014	+.0016	−.0038	−.0029	−.0032	+.0012	+.0014	−.0032	−.0035	−.0038
1 103 400	20 000	+.0020	+.0024	−.0052	−.0042	−.0043	+.0018	+.0018	−.0050	−.0050	−.0055
1 379 250	25 000	+.0028	+.0032	−.0070	−.0061	−.0060	+.0021	+.0024	−.0061	−.0063	−.0071
1 655 100	30 000	+.0035	+.0042	−.0088	−.0076	−.0073	+.0028	+.0031	−.0079	−.0079	−.0094

Load, pounds	Load in pounds per square inch	21	22	23	24	25	26	27	28	29	30
55 170	1000	0	0	0	0	0	0	0	0	0	0
275 850	5000	+.0007	+.0002	−.0015	−.0020	−.0019	−.0006	+.0001	.0000	+.0001	+.0007
551 700	10 000	+.0005	+.0006	−.0034	−.0036	.....	.....	.....	.....	.....	.....
827 550	15 000	+.0018	+.0011	−.0050	−.0052	−.0021	−.0014	−.0001	+.0002	.0000	+.0007
1 103 400	20 000	+.0021	+.0016	−.0068	−.0074	.....	.....	.....	.....	.....	.....
1 379 250	25 000	+.0028	+.0023	−.0090	−.0098	−.0019	−.0014	−.0029	+.0014	+.0002	+.0009
1 655 100	30 000	+.0036	+.0028	−.0112	−.0124	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	31	32	33	34	35	36	37	38	39	40
55 170	1000	0	0	0	0	0	0	0	0	0	0
275 850	5000	+.0001	−.0002	+.0002	+.0001	+.0004	+.0002	+.0002	+.0004	−.0001	−.0001
827 550	15 000	−.0006	−.0005	+.0002	+.0004	+.0006	+.0003	+.0004	+.0002	+.0002	−.0002
1 379 250	25 000	−.0004	−.0006	+.0002	+.0006	+.0007	+.0002	+.0009	+.0003	+.0008	+.0001

Load, pounds	Load in pounds per square inch	41	42	43	44	45	46	47	48
55 170	1000	0	0	0	0	0	0	0	0
275 850	5000	−.0007	.0000	+.0004	+.0003	+.0002	+.0001	−.0001	−.0002
827 550	15 000	−.0012	+.0002	+.0004	+.0004	+.0003	.0000	.0000	.0000
1 379 250	25 000	−.0012	+.0008	−.0002	+.0004	+.0007	.0000	+.0007	+.0008

[a] Slight scaling of steel.
[b] Maximum load.
[c] Load fell off to this value.
[d] Began to show pronounced failure at end (see photograph).
[e] Load fell off to this value.

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Other stations give (±) elongations in a gauge length of 8 inches.

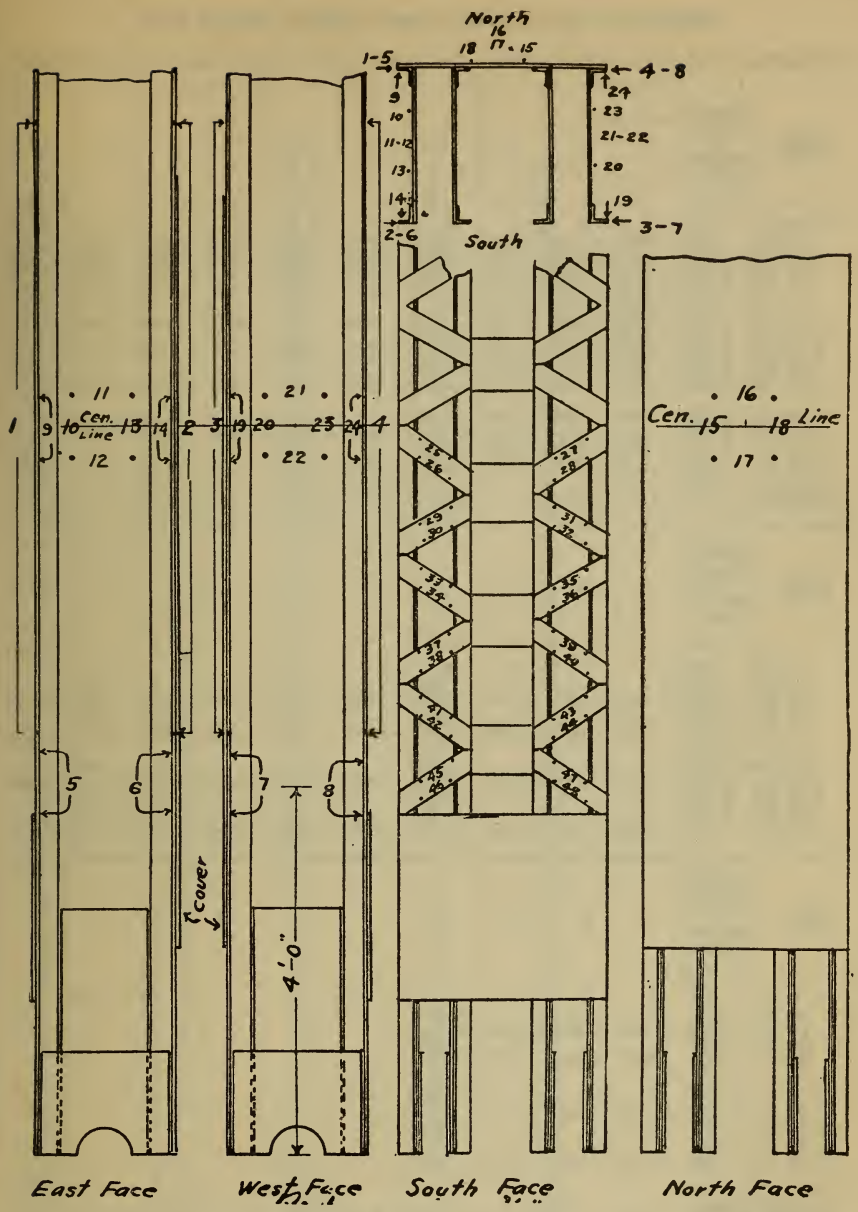


FIG. 16.—*Column* US_2MS_1 (*carbon steel*). *Key to stations on Table 16*

TABLE 17.—Log of Test, Column US_2MS_1, Mayari Steel

[Cumulative (±) Elongations at Stations Below. Tested Jan. 22, 1914. Time of Test, 2 p. m. to 4.15 p. m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
42 470	1000	0	0	0	0	0	0	0	0	0	0
212 350	5000	−.0131	−.0105	−.0085	−.0102	−.0004	−.0008	−.0012	−.0007	−.0014	.....
424 700	10 000	−.0297	−.0221	−.0190	−.0263	−.0007	−.0018	−.0026	−.0014	−.0031	−.0026
637 050	15 000	−.0457	−.0304	−.0324	−.0421	−.0012	−.0032	−.0041	−.0022	−.0048	.....
849 400	20 000	−.0628	−.0451	−.0414	−.0594	−.0017	−.0046	−.0057	−.0031	−.0068	−.0059
1 061 750	25 000	−.0801	−.0559	−.0525	−.0765	−.0020	−.0058	−.0072	−.0038	−.0086	.....
1 167 920	27 500	−.0888	−.0618	−.0583	−.0857	.....	.....	.....	.....	.....	.....
1 274 100	30 000	−.0983	−.0678	−.0635	−.0944	−.0022	−.0069	−.0083	−.0048	−.0108	−.0098
1 380 270	32 500	−.1074	−.0730	−.0697	−.1041	.....	.....	.....	.....	.....	.....
1 486 450	35 000	−.1176	−.0784	−.0754	−.1141	−.0028	−.0078	−.0098	−.0061	−.0133	.....
1 592 630	37 500	−.1279	−.0836	−.0806	−.1250	.....	.....	.....	.....	.....	.....
1 698 800	40 000	−.1390	−.0881	−.0865	−.1361	−.0027	−.0096	−.0108	−.0076	−.0219	.....
1 804 970	42 500	−.1520	−.0924	−.0914	−.1487	.....	.....	.....	.....	.....	.....
1 911 150	45 000	−.1686	−.0971	−.0949	−.1670	−.0030	−.0112	−.0114	−.0103	.....	−.0157
[a] 2 000 000	47 200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[b] 1 000 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	15	16	17	18	19	20
42 470	1000	0	0	0	0	0	0	0	0	0	0
212 350	5000	.....	.....	.....	−.0012	.....	.....	.....	.....	−.0008	.....
424 700	10 000	+.0009	+.0007	−.0024	−.0025	−.0018	+.0007	+.0007	−.0021	−.0018	−.0024
637 050	15 000	.....	.....	.....	−.0032	.....	.....	.....	.....	−.0032	.....
849 400	20 000	+.0020	+.0019	−.0050	−.0049	−.0040	+.0017	+.0014	−.0043	−.0043	−.0052
1 061 750	25 000	.....	.....	.....	−.0061	.....	.....	.....	.....	−.0056	.....
1 274 100	30 000	+.0025	+.0028	−.0077	−.0073	−.0062	+.0027	+.0024	−.0065	−.0067	−.0080
1 486 450	35 000	.....	.....	.....	−.0085	.....	.....	.....	.....	−.0082	.....
1 698 800	40 000	.....	.....	.....	−.0098	.....	.....	.....	.....	−.0094	.....
1 911 150	45 000	+.0042	+.0048	−.0120	−.0107	−.0088	+.0043	+.0032	−.0091	−.0104	−.0124

Load, pounds	Load in pounds per square inch	21	22	23	24
42 470	1000	0	0	0	0
212 350	5000	.....	.....	.....	−.0014
424 700	10 000	+.0008	+.0009	−.0023	−.0026
637 050	15 000	.....	.....	.....	−.0048
849 400	20 000	+.0016	+.0022	−.0059	−.0066
1 061 750	25 000	.....	.....	.....	−.0084
1 274 100	30 000	+.0026	+.0032	−.0094	−.0104
1 486 450	35 000	.....	.....	.....	−.0123
1 698 800	40 000	.....	.....	.....	−.0147
1 911 150	45 000	+.0044	+.0056	−.0154	−.0173

[a] Maximum load.

[b] Fell off to 1 000 000 pounds. Wrinkles in web and angles west face above pin plates. Same on east face to less extent.

NOTE.—Stations 1, 2, 3, and 4 give compressions in inches in gauge lengths of 80 inches. Other stations give (±) elongations in inches in gauge length of 8 inches.

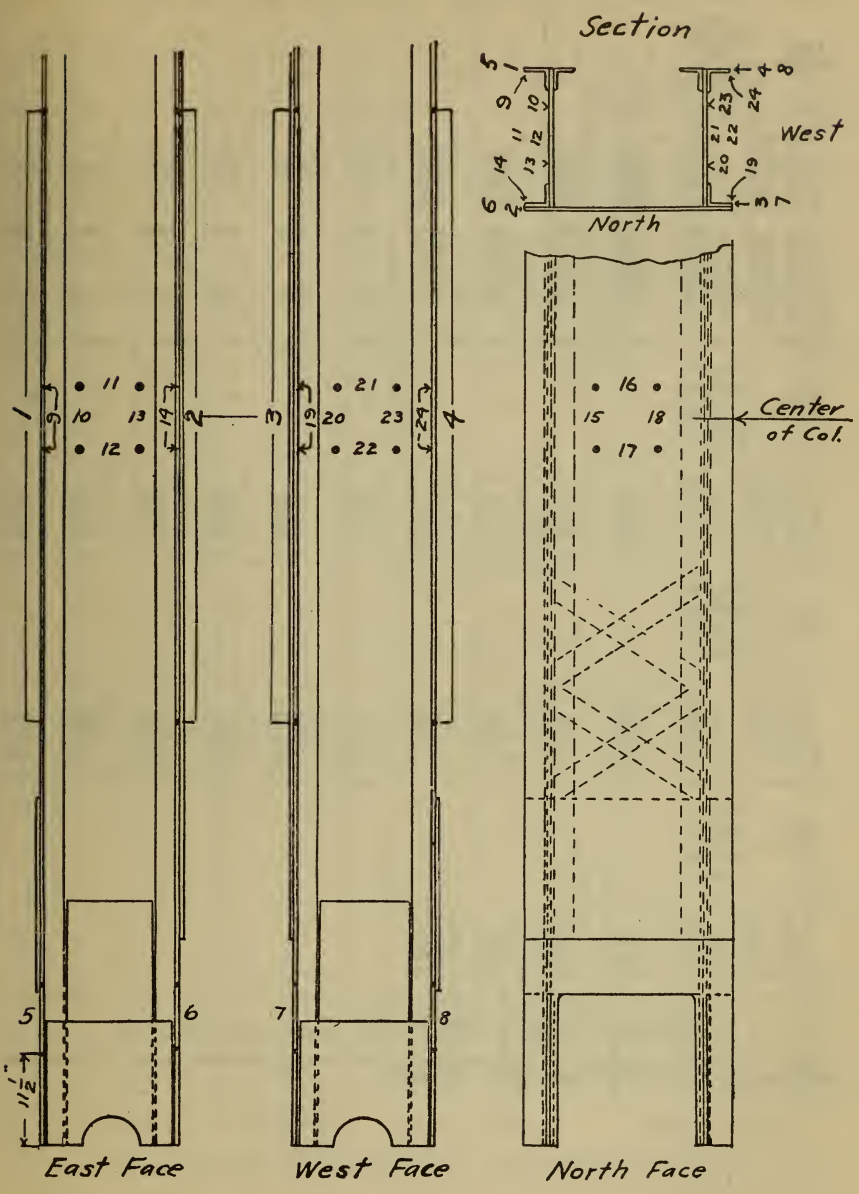


FIG. 17.—*Column US_2MS_1 (Mayari steel). Key to stations on Table 17*

TABLE 18.—Log of Test, Column U_2M_1, Carbon Steel

[Area=96.01. Cumulative (±) Elongations at Stations Below. Tested Jan. 28, 1914.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6[a]	7	8	9	10
96 010	1000	0	0	0	0	0	0	0	0	0	0
480 050	5000	−.0140	−.0125	−.0068	−.0074	.....	.....	.....	.....	.....	.....
960 100	10 000	−.0290	−.0281	−.0204	−.0205	−.0029	+.0023	−.0022	−.0028	−.0048	−.0044
1 440 150	15 000	−.0448	−.0444	−.0350	−.0351	.....	.....	.....	.....	.....	.....
1 680 200	17 500	−.0524	−.0533	−.0428	−.0433	.....	.....	.....	.....	.....	.....
1 920 200	20 000	−.0623	−.0635	−.0517	−.0516	−.0067	−.0016	−.0056	−.0063	−.0082	−.0082
2 160 200	22 500	−.0721	−.0745	−.0616	−.0608	.....	.....	.....	.....	.....	.....
2 400 250	25 000	−.0840	−.0874	−.0727	−.0703	.....	.....	.....	.....	.....	.....
2 640 300	27 500	−.0985	−.1040	−.0867	−.0819	.....	.....	.....	.....	.....	.....
2 880 300	30 000	−.1200	−.1311	−.1059	−.0950	−.0192	−.0139	−.0182	−.0130	−.0135	−.0146
[b] 2 980 000	31 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	15	16	17[a]	18	19	20
96 010	1000	0	0	0	0	0	0	0	0	0	0
960 100	10 000	+.0013	+.0007	−.0036	−.0036	−.0033	−.0024	−.0024	+.0003	−.0016	−.0020
1 920 200	20 000	+.0023	+.0017	−.0071	−.0071	−.0065	−.0056	−.0016	+.0020	−.0048	−.0048
2 880 300	30 000	+.0045	+.0037	−.0121	−.0126	−.0121	−.0108	+.0003	+.0037	−.0094	−.0091

Load, pounds	Load in pounds per square inch	21	22	23	24	25	26	27	28	29	30
96 010	1000	0	0	0	0	0	0	0	0	0	0
960 100	10 000	−.0025	−.0025	−.0028	−.0029	+.0008	+.0008	+.0008	−.0016	−.0005	−.0007
1 920 200	20 000	−.0060	−.0058	−.0060	−.0066	.....	+.0028	+.0016	−.0031	−.0011	−.0015
2 880 300	30 000	−.0104	−.0106	−.0113	−.0118	+.0032	+.0036	+.0037	−.0046	−.0026	−.0032

Load, pounds	Load in pounds per square inch	31	32	33	34
96 010	1000	0	0	0	0
960 100	10 000	−.0003	+.0009	+.0001	+.0005
1 920 200	20 000	−.0012	+.0015	+.0005	+.0007
2 880 300	30 000	−.0041	+.0015	+.0016	+.0013

[a] Probably the initial gauge reading was in error.
[b] Maximum load. Scaling at 2 960 000 pounds. Sudden failure at lower end.

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Other stations give elongations (±) in inches in gauge length of 8 inches.

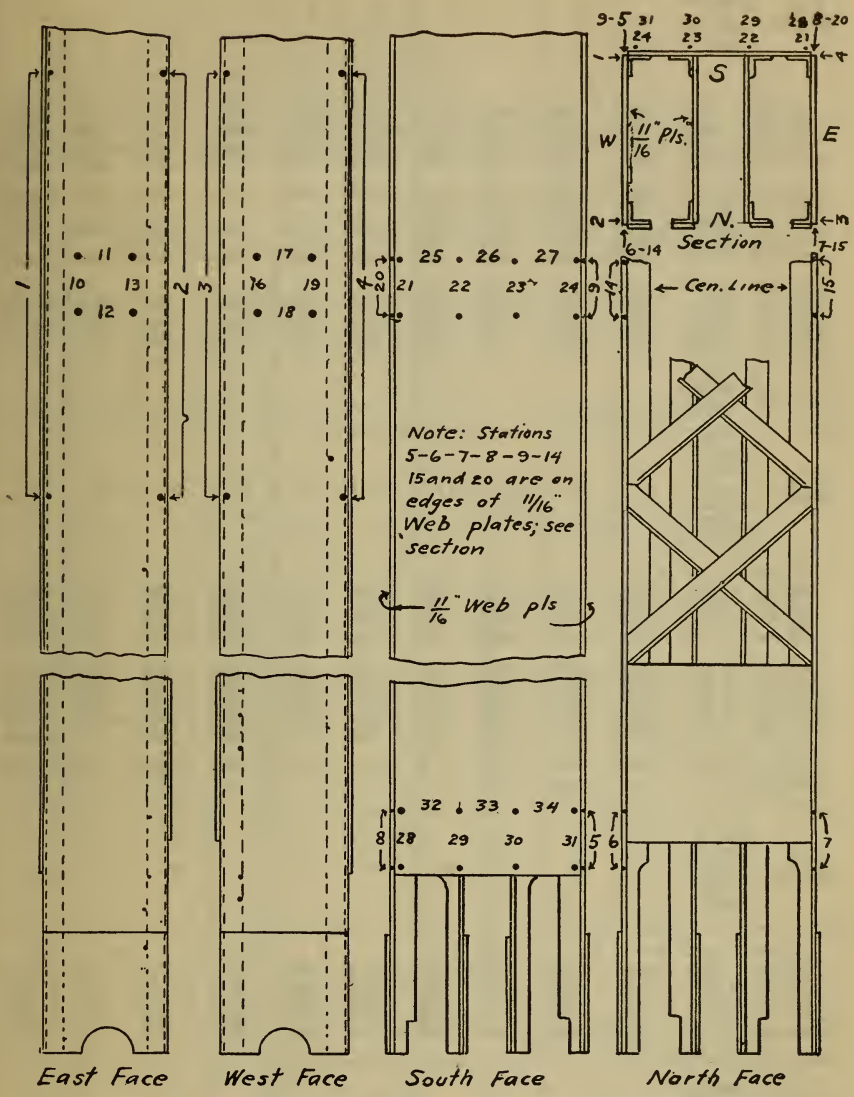


FIG. 18.—*Column $U_2 M_1$ (carbon steel). Key to stations on Table 18*

TABLE 19.—Log of Test, Column U_2M_1, Mayari Steel

[Cumulative (±) Elongations at Stations Below. Tested Jan. 26, 1914. Time of Test, 9.35 a. m. to 12 m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
59 800	900	0	0	0	0	0	0	0	0	0	0
299 000	4490	−.0154	−.0062	−.0041	−.0126	−.0015	−.0013	−.0007	−.0013	−.0016	−.0016
598 000	8990	−.0294	−.0172	−.0140	−.0264	−.0025	−.0025	−.0018	−.0024	−.0031	−.0028
897 000	13 480	−.0437	−.0293	−.0259	−.0403	−.0043	−.0037	−.0028	−.0039	−.0044	−.0042
1 196 000	17 970	−.0576	−.0417	−.0371	−.0532	−.0059	−.0049	−.0042	−.0057	−.0058	−.0057
1 495 000	22 460	−.0719	−.0532	−.0491	−.0663	−.0076	−.0062	−.0054	−.0072	−.0073	−.0070
1 644 500	24 700	−.0797	−.0600	−.0551	−.0742	.....	.....	.....	.....	.....	.....
1 794 000	26 960	−.0870	−.0657	−.0615	−.0812	−.0091	−.0076	−.0066	−.0090	−.0088	−.0085
1 943 500	29 200	−.0951	−.0723	−.0678	−.0887	.....	.....	.....	.....	.....	.....
2 093 000	31 450	−.1034	−.0789	−.0741	−.0960	−.0109	−.0092	−.0079	−.0106	−.0104	−.0100
2 242 500	33 700	−.1119	−.0854	−.0804	−.1040	.....	.....	.....	.....	.....	.....
2 392 000	35 940	−.1205	−.0923	−.0868	−.1122	−.0124	−.0109	−.0093	−.0126	−.0126	−.0118
2 691 000	40 430	−.1400	−.1057	−.1000	−.1301	−.0151	−.0125	−.0104	−.0145	−.0157	−.0132
2 990 000	44 930	−.1619	−.1206	−.1135	−.1500	−.0184	−.0146	−.0116	−.0169	−.0196	−.0155
[a] 3 289 000	49 420	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[b] 3 103 500	46 600	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[c] 200 000	3010	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	19	20	21	22	23	24
59 800	900	0	0	0	0	0	0	0	0	0	0
299 000	4490	+.0003	+.0006	−.0010	−.0006	−.0004	−.0008	+.0004	+.0003	−.0011	−.0015
598 000	8990	+.0007	+.0009	−.0022	−.0019	−.0016	−.0019	+.0008	+.0009	−.0023	−.0027
897 000	13 480	+.0011	+.0012	−.0035	−.0028	−.0028	−.0030	+.0012	+.0013	−.0037	−.0042
1 196 000	17 970	+.0013	+.0014	−.0048	−.0040	−.0040	−.0042	+.0015	+.0014	−.0051	−.0057
1 495 000	22 460	+.0017	+.0019	−.0063	−.0054	−.0052	−.0056	+.0019	+.0016	−.0061	−.0071
1 794 000	26 960	+.0021	+.0022	−.0076	−.0066	−.0065	−.0068	+.0024	+.0023	−.0075	−.0085
2 093 000	31 450	+.0025	+.0026	−.0090	−.0079	−.0078	−.0080	+.0028	+.0027	−.0089	−.0102
2 392 000	35 940	+.0029	+.0031	−.0105	−.0093	−.0092	−.0094	+.0032	+.0031	−.0105	−.0123
2 691 000	40 430	+.0031	+.0035	−.0121	−.0106	−.0103	−.0108	+.0038	+.0036	−.0122	−.0148
2 990 000	44 930	+.0033	+.0040	−.0137	−.0119	−.0114	−.0122	+.0043	+.0039	−.0139	−.0183
3 289 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	25	26	27	28	29	30	31	32	33	34
59 800	900	0	0	0	0	0	0	0	0	0	0
598 000	8990	−.0006	−.0008	−.0012	−.0017	−.0012	−.0008	−.0004	−.0002	−.0001	−.0007
1 196 000	17 970	−.0014	−.0017	−.0025	−.0031	−.0030	−.0018	−.0010	−.0006	−.0005	−.0016
1 794 000	26 960	−.0023	−.0027	−.0030	−.0042	−.0040	−.0028	−.0014	−.0009	−.0008	−.0030
2 392 000	35 940	−.0030	−.0031	−.0043	−.0053	−.0051	−.0035	−.0022	−.0016	−.0016	−.0045
2 990 000	44 930	−.0040	−.0041	−.0050	−.0059	−.0056	−.0042	−.0033	−.0028	−.0036	−.0078

Load, pounds	Load in pounds per square inch	35	36	37	38	39	40	41	42	43	44
59 800	900	0	0	0	0	0	0	0	0	0	0
598 000	8990	.....	−.0034	−.0025	−.0007	−.0004	−.0006	+.0003	+.0004	+.0006	+.0004
1 196 000	17 970	−.0019	−.0076	−.0055	−.0017	−.0010	−.0006	+.0006	+.0006	+.0010	+.0010
1 794 000	26 960	−.0046	−.0112	−.0084	−.0031	−.0016	−.0003	+.0008	+.0008	+.0017	+.0015
2 392 000	35 940	−.0075	−.0156	−.0127	−.0054	−.0028	−.0006	+.0010	+.0010	+.0024	+.0022
2 990 000	44 930	−.0132	−.0274	−.0215	−.0101	−.0054	−.0021	+.0012	+.0010	+.0032	+.0030

[a] Maximum load. [b] Sudden failure at lower end. [c] Final reading after load fell off.

TABLE 19.—Log of Test, Column U_2M_1, Mayari Steel—Continued

Load, pounds	Load in pounds per square inch	45	46	47	48
59 800	900	0	0	0	0
598 000	8990	−.0004	−.0003	−.0003	+.0008
1 196 000	17 970	−.0004	−.0002	−.0009	+.0016
1 794 000	26 960	0	+.0007	−.0003	+.0026
2 392 000	35 940	+.0010	+.0032	−.0011	+.0035
2 990 000	44 930	+.0047	+.0122	+.0061	+.0042

NOTE.—Stations 1 to 4, inclusive, give compression in inches in gauge length of 80 inches. Other stations give elongation (±) in inches in gauge length of 8 inches.

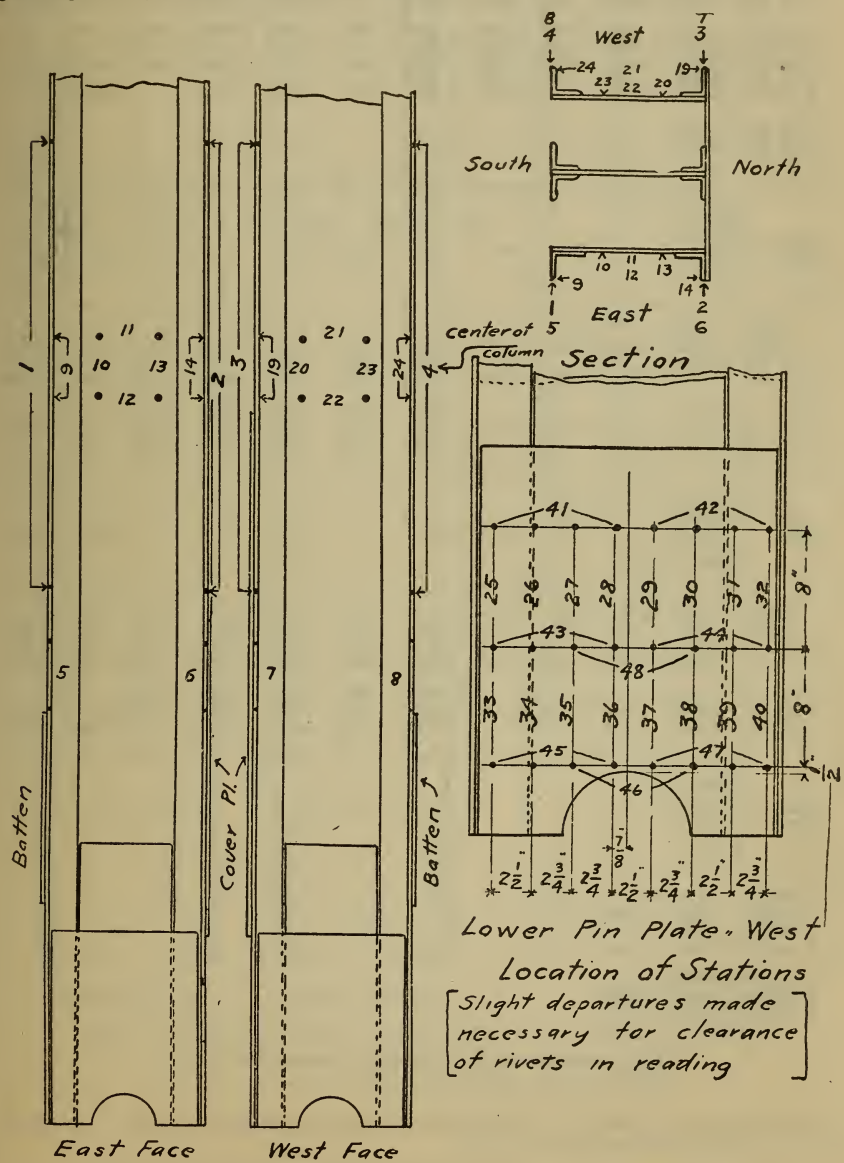


FIG. 19.—*Column U_2 M_1 (Mayari steel). Key to stations on Table 19*

TABLE 20.—Log of Test, Column LC_0LC_1, Carbon Steel

[Cumulative (±) Elongations at Stations Below. Tested Jan. 28, 1914. Time of Test, 9.32 a. m. to 12 m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
119 000	1000	0	0	0	0	0	0	0	0	0	0
595 000	5000	−.0128	−.0082	−.0097	−.0122	−.0012	−.0013	−.0008	−.0014	−.0015	−.0014
1 190 000	10 000	−.0271	−.0217	−.0238	−.0267	−.0028	−.0031	−.0027	−.0031	−.0033	.....
1 785 000	15 000	−.0420	−.0367	−.0388	−.0422	−.0047	−.0049	−.0044	−.0046	−.0051	−.0047
2 082 500	17 500	−.0510	−.0446	−.0475	−.0510	.....	.....	.....	.....	.....	.....
2 380 000	20 000	−.0600	−.0543	−.0566	−.0600	−.0071	−.0075	−.0068	−.0068	−.0079	.....
2 677 500	22 500	−.0714	−.0648	−.0680	−.0720	.....	.....	.....	.....	.....	.....
2 975 000	25 000	−.0859	−.0779	−.0827	−.0880	−.0104	−.0117	−.0106	−.0106	−.0120	−.0105
3 272 500	27 500	−.1029	−.0951	−.1034	−.1090	.....	.....	.....	.....	.....	.....
3 570 000	30 000	−.1248	−.1201	−.1390	−.1454	−.0170	−.0195	−.0216	.....	−.0204	.....
[a] 3 820 000	32 100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[b] 3 858 000	32 400	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[c] 2 100 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	15	16	17	18	19	20
119 000	1000	0	0	0	0	0	0	0	0	0	0
595 000	5000	+.0005	+.0003	−.0008	−.0010	−.0010	−.0009	−.0009	−.0008	−.0007	−.0010
1 190 000	10 000	.....	.....	.....	−.0025	−.0024	−.0025	−.0020	−.0024	−.0022	.....
1 785 000	15 000	+.0015	+.0013	−.0041	−.0039	−.0040	−.0040	−.0036	.−0040	−.0037	−.0036
2 380 000	20 000	.....	.....	.....	−.0057	−.0059	−.0061	−.0050	−.0057	−.0055	.....
2 975 000	25 000	+.0029	+.0028	−.0095	−.0081	−.0084	−.0085	−.0070	−.0084	−.0085	−.0085
3 570 000	30 000	.....	.....	.....	−.0135	−.0130	−.0124	−.0120	−.0140	−.0114	.....

Load, pounds	Load in pounds per square inch	21	22	23	24	25	26	27	28	29	30
119 000	1000	0	0	0	0	0	0	0	0	0	0
595 000	5000	+.0003	+.0004	−.0011	−.0015	−.0012	−.0012	−.0012	−.0011	.0000	+.0001
1 190 000	10 000	.....	.....	.....	−.0030	−.0027	−.0027	−.0027	−.0026	.....	.....
1 785 000	15 000	+.0013	+.0013	−.0040	−.0046	−.0044	−.0046	−.0044	−.0042	+.0003	+.0001
2 380 000	20 000	.....	.....	.....	−.0063	−.0063	−.0074	−.0067	−.0062	.....	.....
2 975 000	25 000	+.0030	+.0027	−.0091	−.0080	−.0094	−.0109	−.0105	−.0093	+.0007	+.0004
3 570 000	30 000	.....	.....	.....	−.0087	−.0149	−.0146	−.0184	−.0146	+.0008	+.0006

Load, pounds	Load in pounds per square inch	31
119 000	1000	0
595 000	5000	−.0001
1 190 000	10 000	.....
1 785 000	15 000	−.0003
2 380 000	20 000	.....
2 975 000	25 000	−.0004
3 570 000	30 000	−.0007

[a] Failure commenced. First deflection. Buckled east 4 inches.
[b] Maximum load. Held for several minutes.
[c] Load fell off gradually to this value.

NOTE.—Stations 1 to 4, inclusive, give compressions in inches in gauge length of 80 inches. Other stations give (±) elongations in a gauge length of 8 inches.

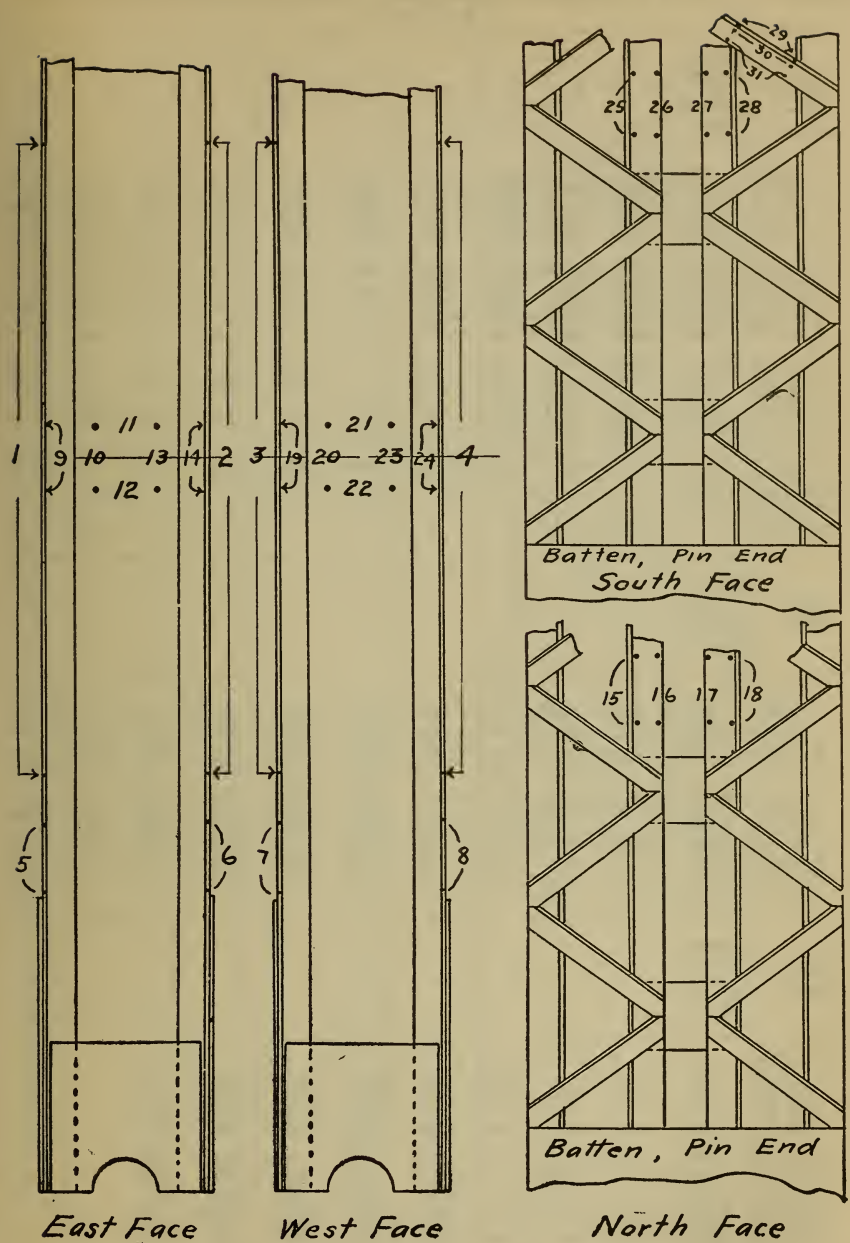


FIG. 20.—*Column LC_0 LC_1 (carbon steel). Key to stations on Table 20*

TABLE 21.—Log of Test, Column LC_0LC_1, Mayari Steel

[Area=75.1. Cumulative (±) Elongations at Stations Below. Tested Jan. 23, 1914. Time of test, 1.30 p. m. to 4.15 p. m.]

Load, pounds	Load in pounds per square inch	1	2	3	4	5	6	7	8	9	10
75 100	1000	0	0	0	0	0	0	0	0	0	0
375 600	5000	−.0120	−.0083	−.0084	−.0116	−.0020	−.0013	−.0008	−.0010	−.0014	−.0014
751 300	10 000	−.0259	−.0217	−.0200	−.0248	−.0037	−.0027	−.0027	−.0028	−.0029	−.0030
1 126 900	15 000	−.0396	−.0349	−.0328	−.0377	−.0053	−.0041	−.0042	.....	−.0044	.....
1 502 500	20 000	−.0537	−.0483	−.0457	−.0509	−.0070	−.0055	−.0060	−.0054	−.0058	−.0058
1 878 100	25 000	−.0676	−.0618	−.0590	−.0635	−.0082	−.0075	−.0079	−.0074	−.0072	.....
2 066 000	27 500	−.0749	−.0690	−.0655	−.0707	.....	.....	.....	.....	.....	.....
2 253 800	30 000	−.0820	−.0754	−.0723	−.0771	−.0105	−.0093	−.0097	−.0090	−.0085	−.0088
2 441 600	32 500	−.0887	−.0830	−.0793	−.0843	.....	.....	.....	.....	.....	.....
2 629 400	35 000	−.0961	−.0900	−.0862	−.0915	−.0125	−.0111	−.0115	−.0109	−.0098	.....
2 817 200	37 500	−.1034	−.0973	−.0941	−.0998	.....	.....	.....	.....	.....	.....
3 005 000	40 000	−.1109	−.1045	−.1014	−.1072	.....	−.0125	−.0137	−.0130	−.0112	−.0116
3 192 800	42 500	−.1189	−.1120	−.1093	−.1152	.....	.....	.....	.....	.....	.....
3 380 600	45 000	−.1269	−.1200	−.1171	−.1240	−.0169	−.0148	−.0164	−.0148	−.0126	.....
3 568 400	47 500	−.1360	−.1287	−.1257	−.1335	.....	.....	.....	.....	.....	.....
3 756 200	50 000	−.1465	−.1380	−.1339	−.1431	−.0205	−.0169	−.0199	−.0178	−.0140	−.0153
4 131 800	55 000	−.1740	−.1610	−.1550	−.1678	.....	−.0207	−.0257	.....	.....	.....
[a] 4 443 000	57 700	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[b] 3 720 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
[c] 2 900 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	11	12	13	14	15	16	17	18	19	20
75 100	1000	0	0	0	0	0	0	0	0	0	0
375 600	5000	+.0004	+.0004	−.0011	−.0009	−.0012	−.0011	−.0007	−.0011	−.0008	−.0015
751 300	10 000	+.0010	+.0014	−.0026	−.0022	−.0027	−.0022	−.0022	−.0024	−.0019	−.0032
1 126 900	15 000	.....	.....	.....	−.0037	−.0041	−.0036	−.0035	−.0037	−.0032	.....
1 502 500	20 000	+.0018	+.0026	−.0056	−.0051	−.0054	−.0050	−.0050	−.0052	−.0050	−.0053
1 878 100	25 000	.....	.....	.....	−.0065	−.0068	−.0064	−.0071	−.0066	−.0062	.....
2 253 800	30 000	+.0028	+.0035	−.0084	−.0079	−.0083	−.0081	−.0079	−.0080	−.0078	−.0065
2 629 400	35 000	.....	.....	.....	−.0093	−.0098	−.0098	−.0095	−.0088	−.0092	.....
3 005 000	40 000	+.0038	+.0046	−.0116	−.0108	−.0110	−.0113	−.0111	−.0104	−.0103	−.0117
3 380 600	45 000	.....	.....	.....	−.0125	−.0122	−.0130	−.0125	−.0121	−.0122	.....
3 756 200	50 000	+.0054	+.0058	−.0149	−.0154	−.0142	−.0145	−.0140	−.0147	−.0138	−.0149
4 131 800	55 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Load, pounds	Load in pounds per square inch	21	22	23	24	25	26	27	28
75 100	1000	0	0	0	0	0	0	0	0
375 600	5000	+.0005	0	−.0006	−.0013	−.0013	−.0012	−.0012	−.0014
751 300	10 000	+.0007	+.0009	−.0025	−.0026	−.0029	−.0026	−.0023	−.0026
1 126 900	15 000	.....	.....	.....	−.0040	−.0042	−.0038	−.0038	−.0040
1 502 500	20 000	+.0015	+.0017	−.0051	−.0054	−.0053	−.0056	−.0052	−.0054
1 878 100	25 000	.....	.....	.....	−.0069	−.0070	−.0073	−.0068	−.0068
2 253 800	30 000	+.0025	+.0028	−.0079	−.0082	−.0084	−.0088	−.0084	−.0083
2 629 400	35 000	.....	.....	.....	−.0098	−.0100	−.0104	−.0096	−.0101
3 005 000	40 000	+.0034	+.0038	−.0091	−.0115	−.0114	−.0125	−.0108	−.0116
3 380 600	45 000	.....	.....	.....	−.0134	−.0134	−.0142	−.0118	−.0132
3 756 200	50 000	+.0046	+.0051	−.0145	−.0153	−.0154	−.0155	−.0126	−.0152
4 131 800	55 000	.....	.....	.....	.....	.....	.....	.....	.....

[a] Maximum load. Southwest angle had a slight initial dent or wrinkle at center of column which did not seem to appreciably affect strength. Reports were heard at 3 100 000, 3 460 000, and 3 850 000. Slight scaling at 3 930 000.

[b] Wrinkling.

[c] Pronounced wrinkling.

NOTE.—Stations 1 to 4, inclusive, give compression in inches in gauge length of 80 inches. Other stations give elongations (±) in inches in gauge length of 8 inches.

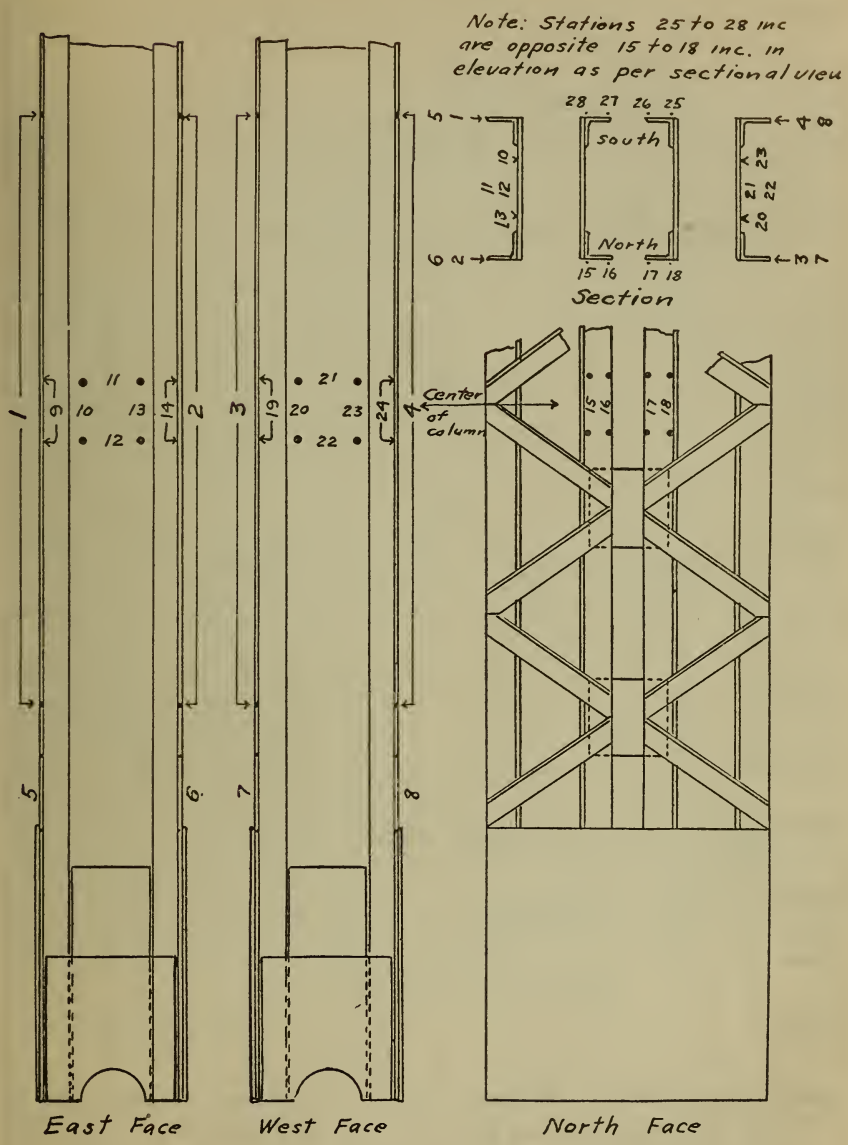


FIG. 21.—*Column LC_0 LC_1 (Mayari steel). Key to stations on Table 21*

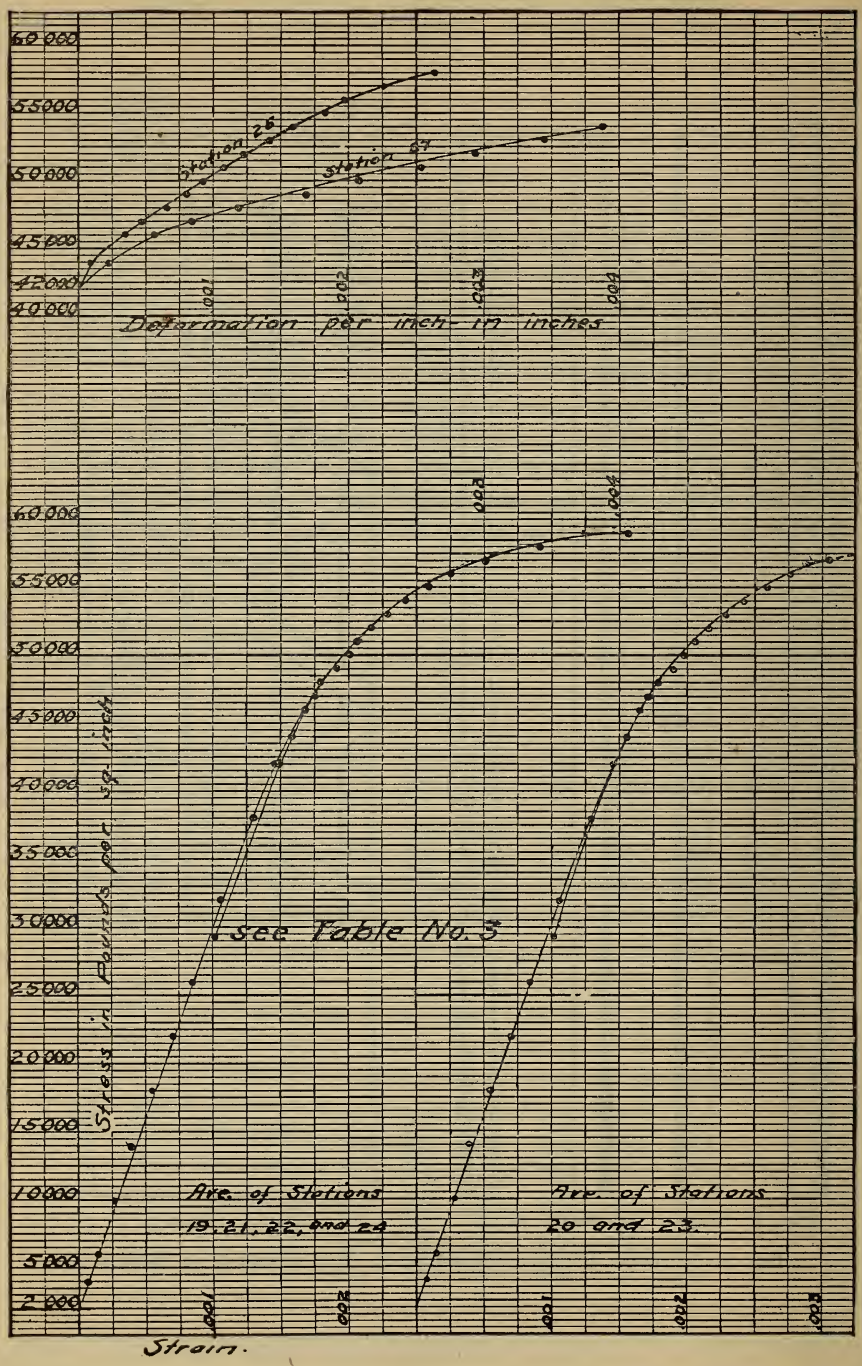


FIG. 22.—*Column U_2U_3 (nickel steel)*

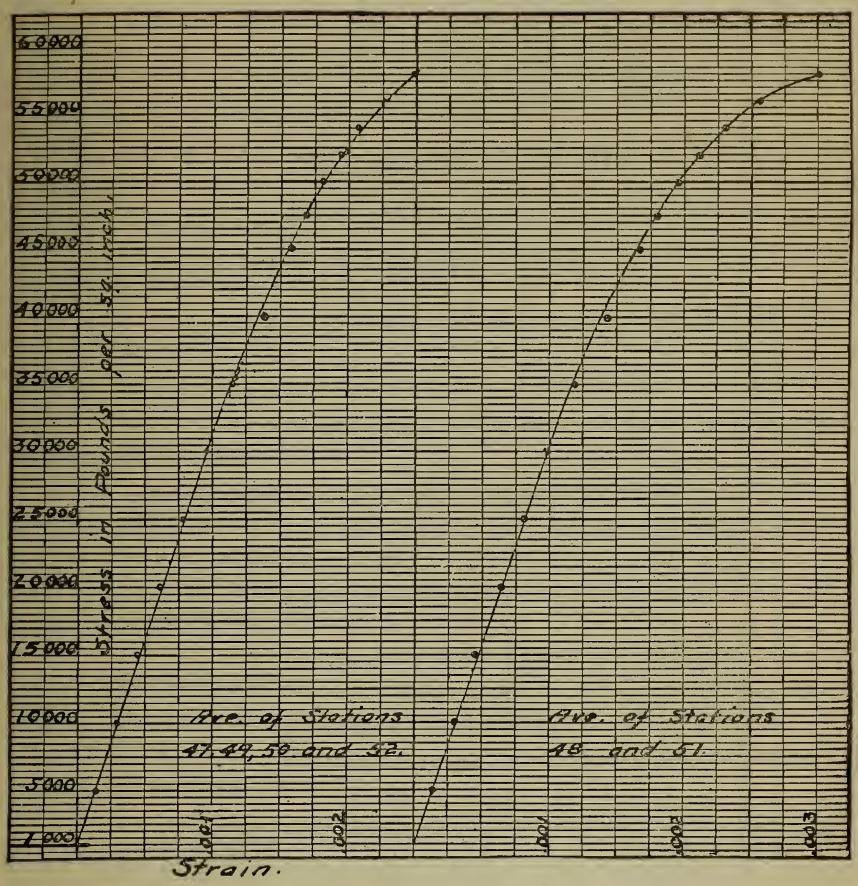


FIG. 23.—*Column U_8U_9 (nickel steel)*

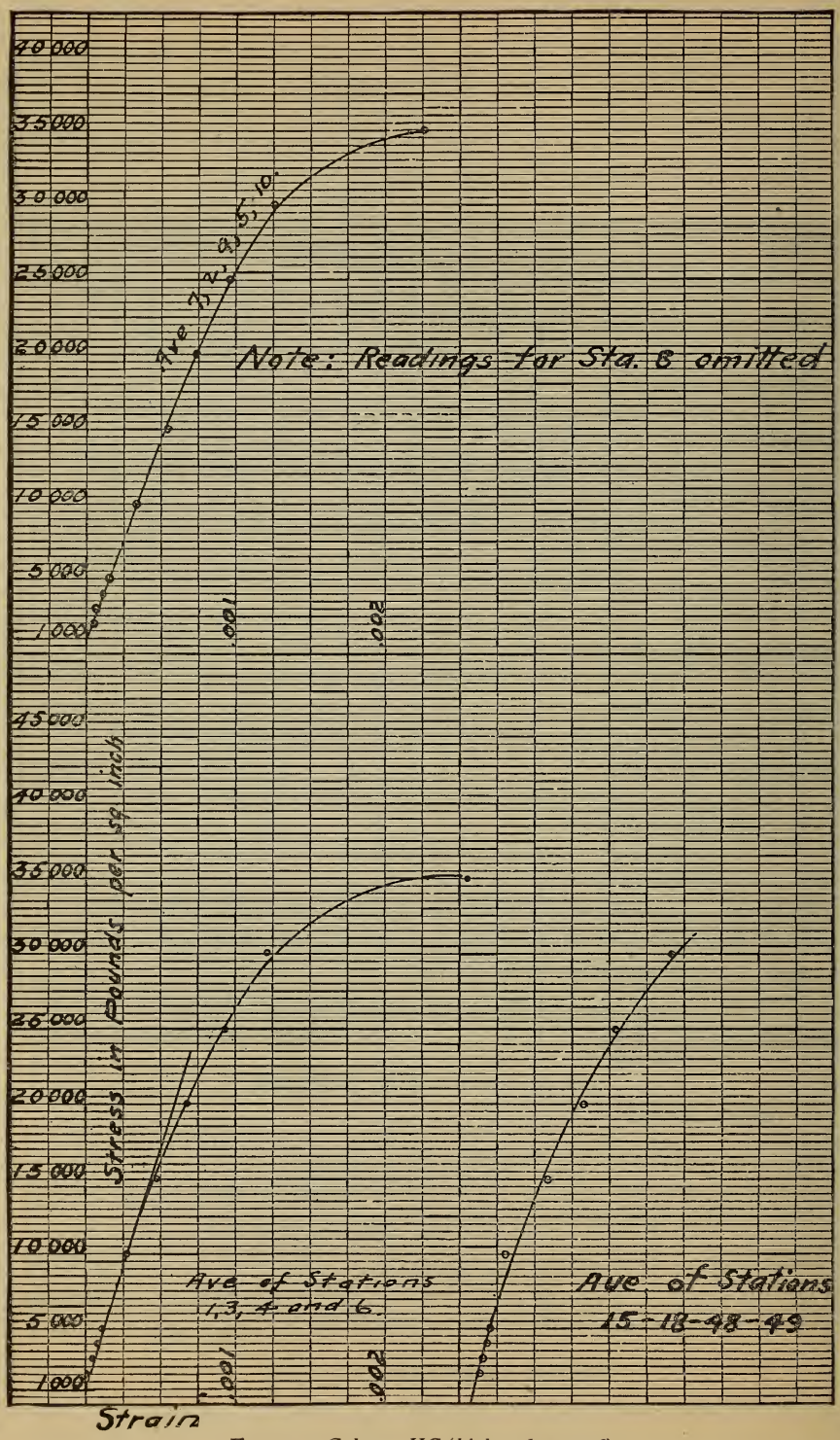


FIG. 24.—*Column HC (high carbon steel)*

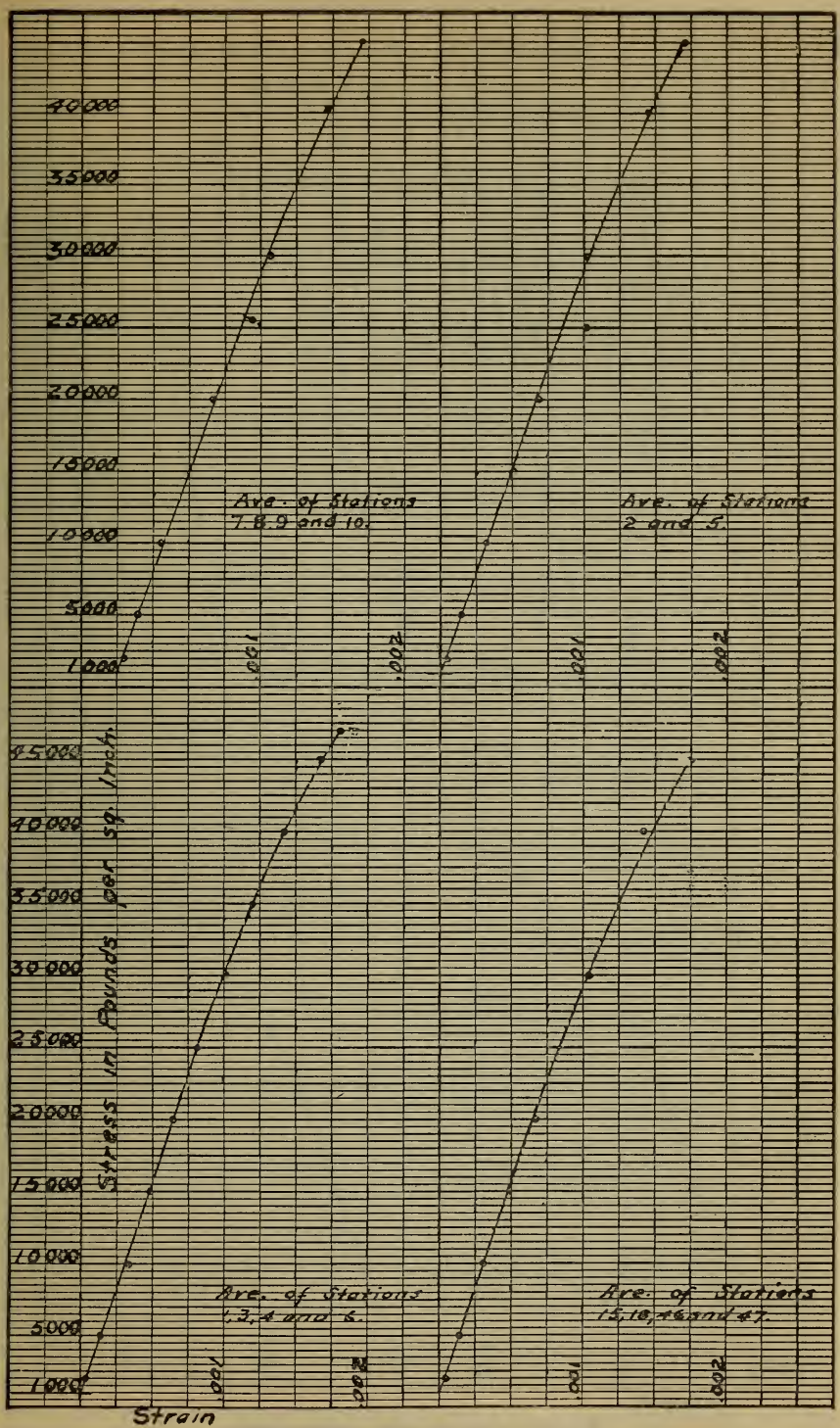


FIG. 25.—*Column MY (Mayari steel)*

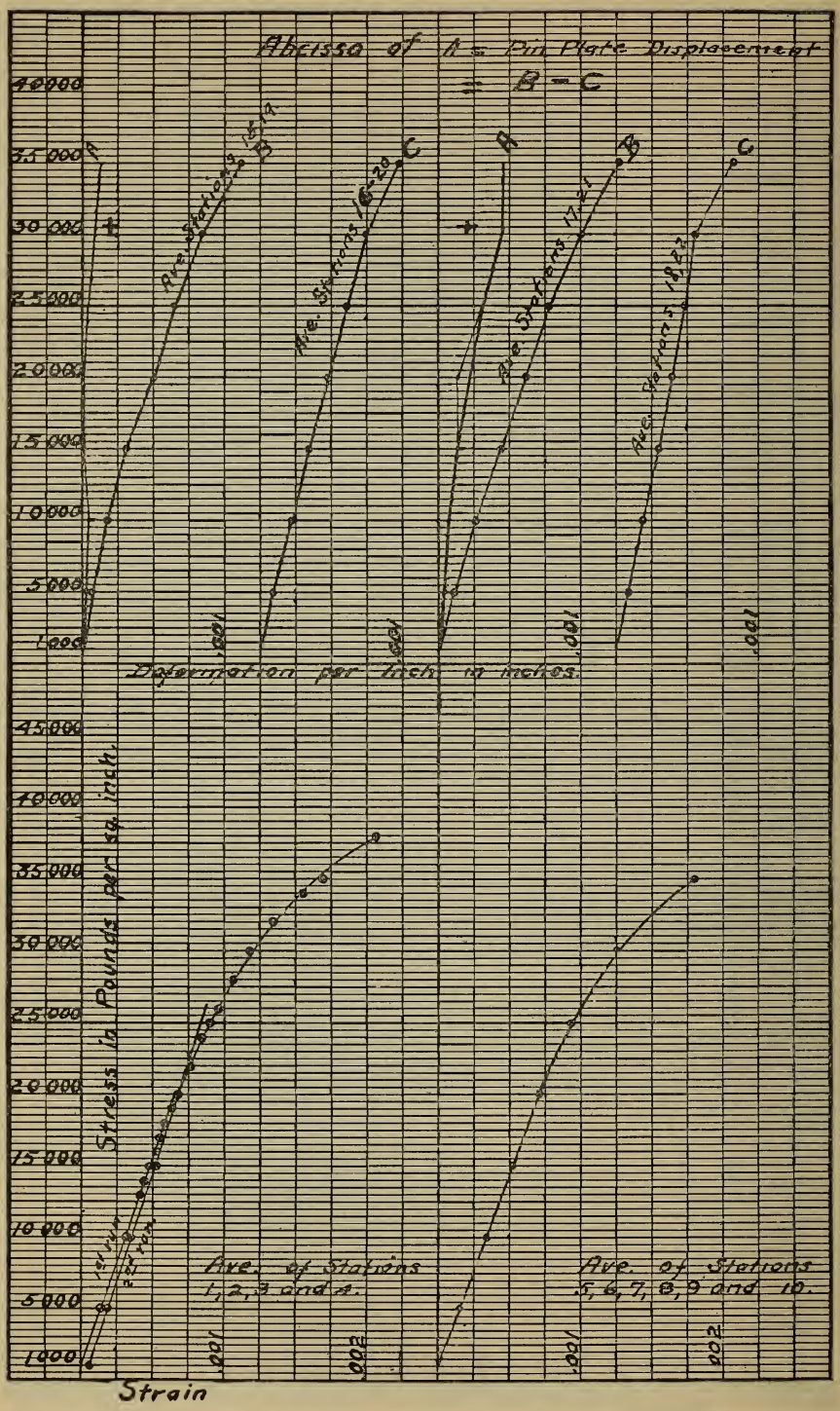


FIG. 26.—*Column 31HC (carbon steel)*

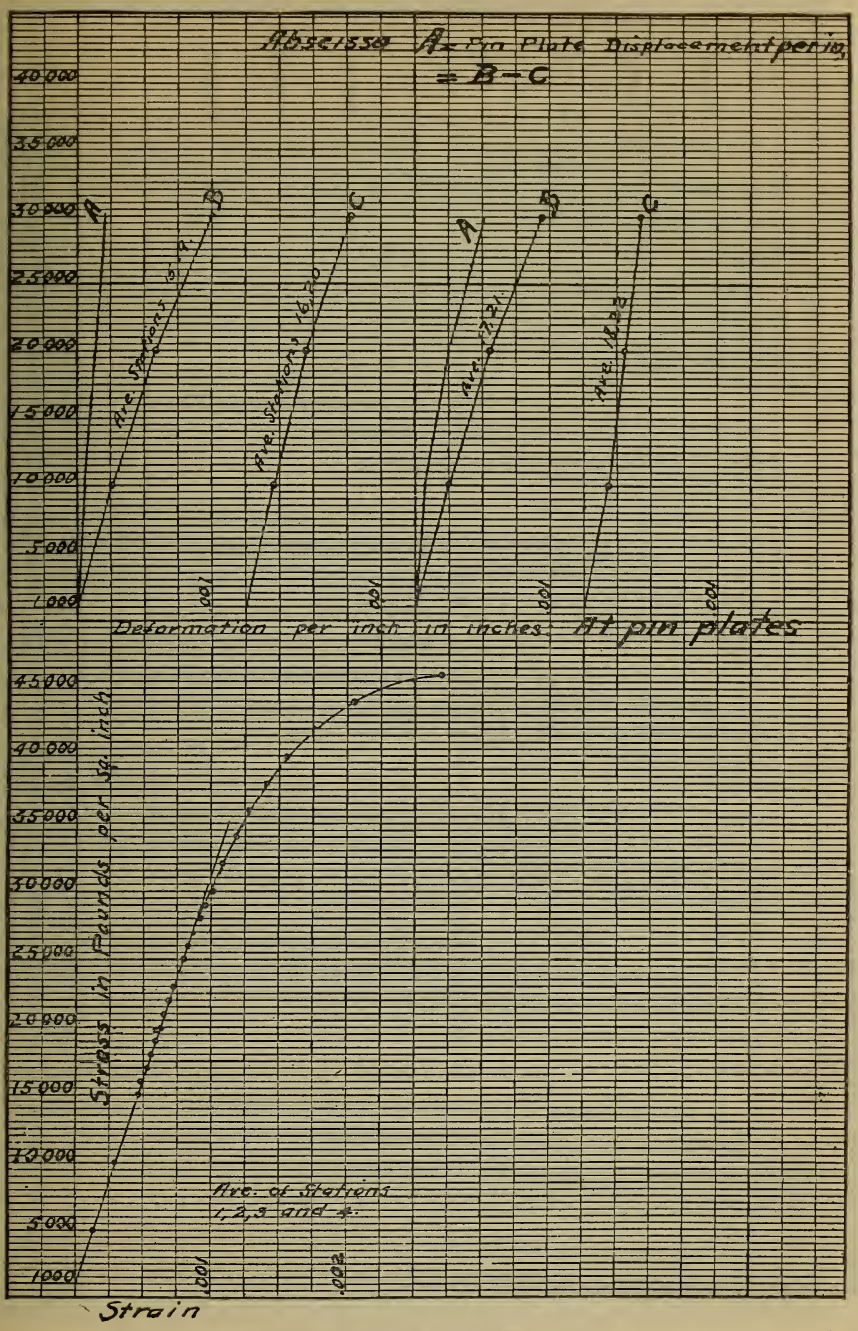


FIG. 27.—*Column 32S (silicon steel)*

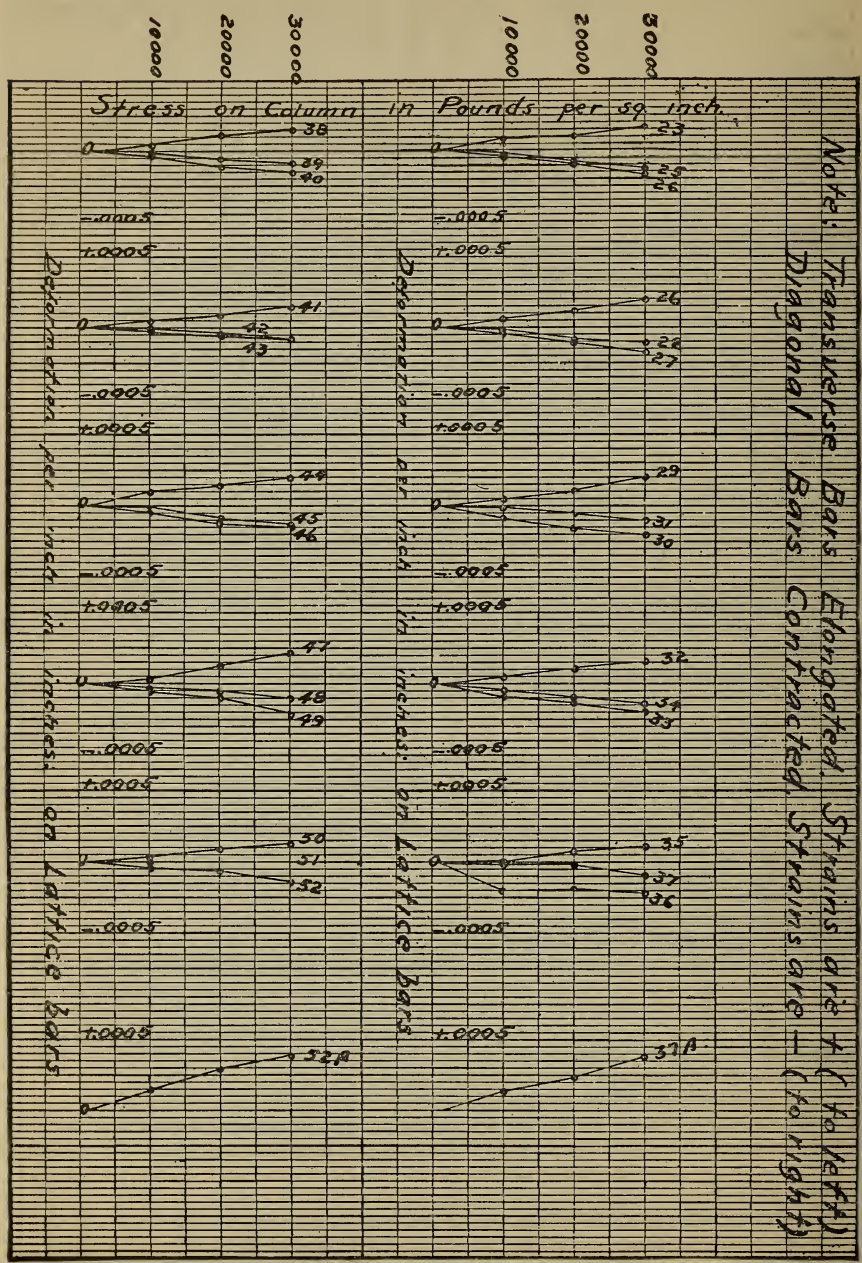


FIG. 28.—*Column 32S (silicon steel)*

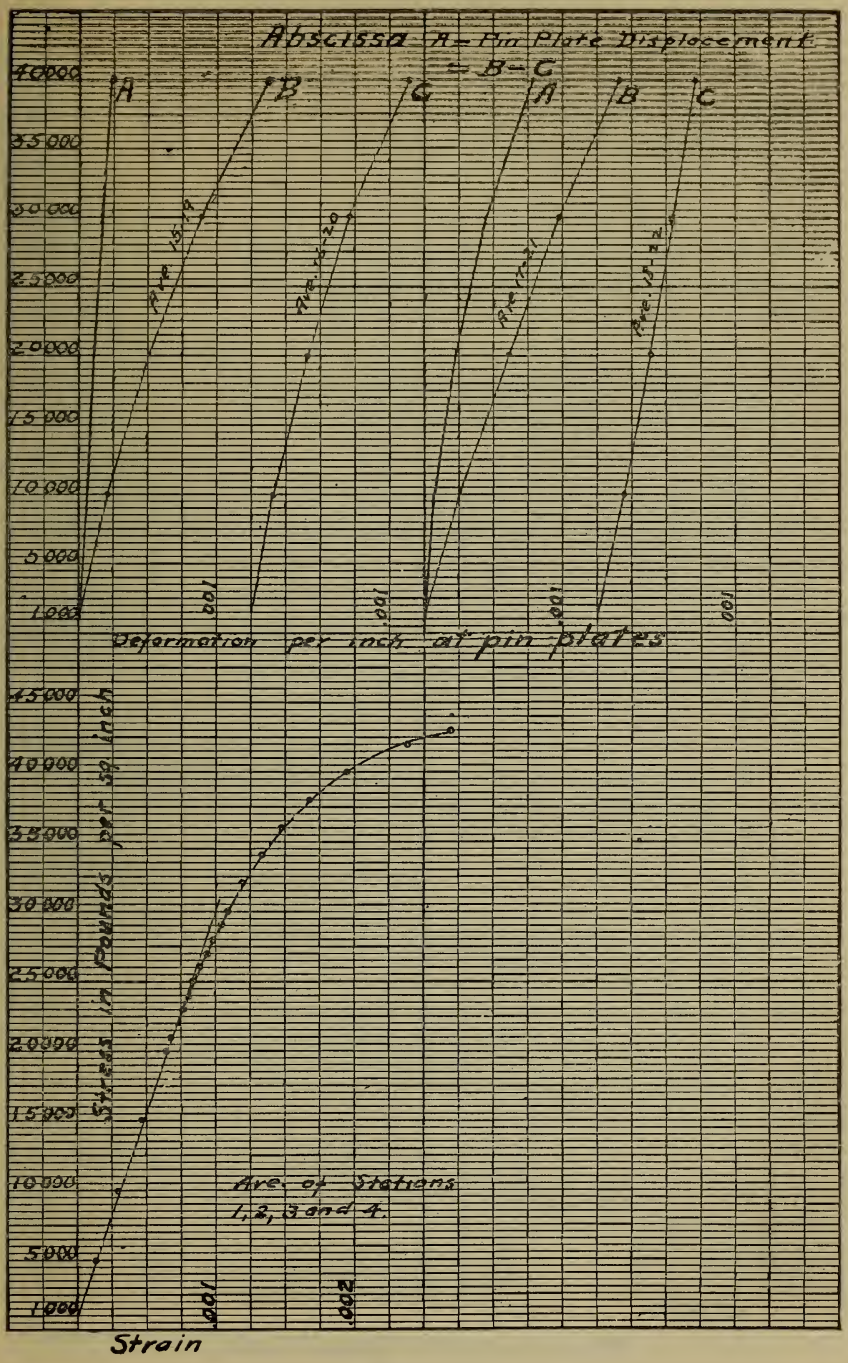


FIG. 29.—*Column 34CS (chrome steel)*

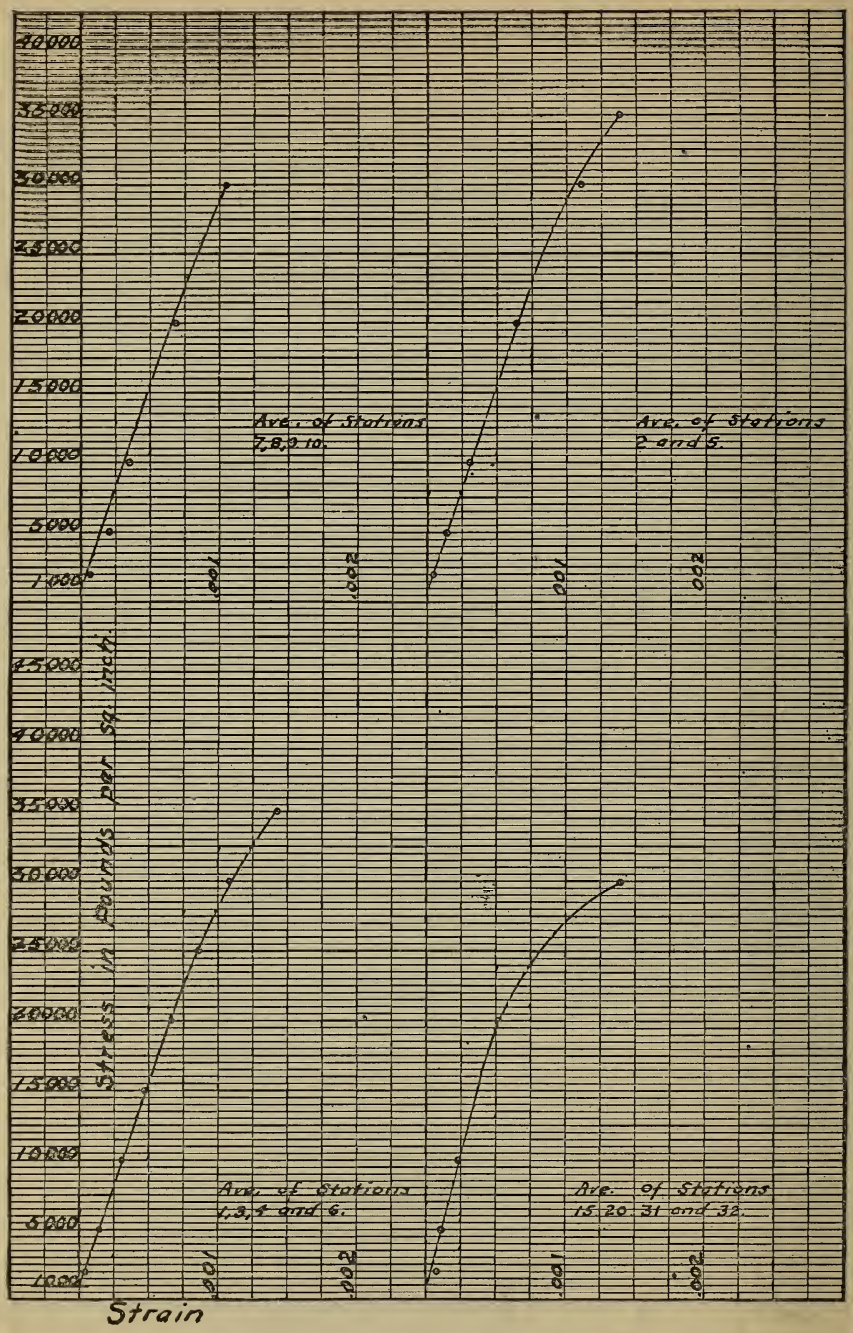


FIG. 30.—*Column HC1 (high carbon steel)*

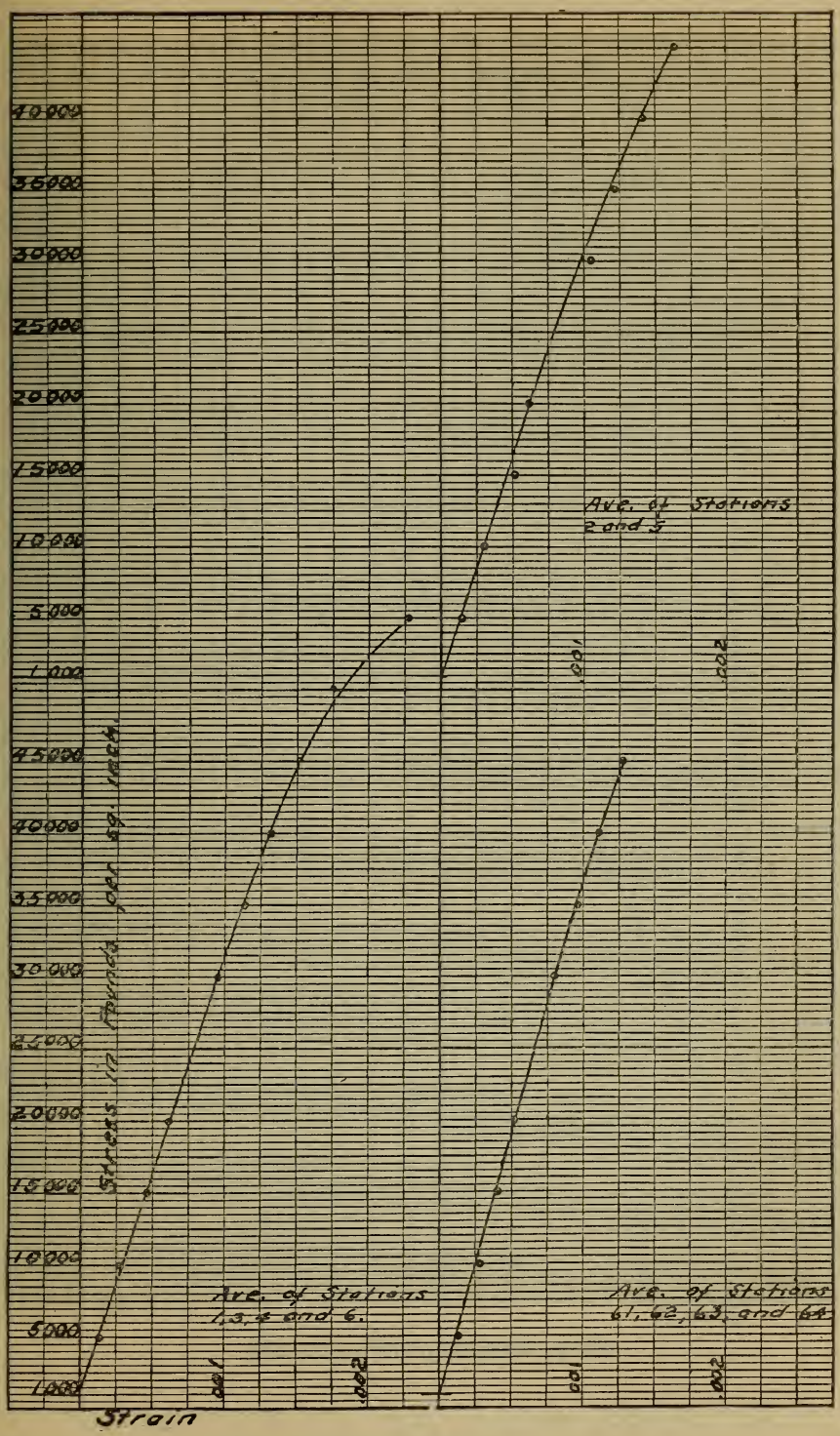


FIG. 31.—*Column MY1 (Mayari steel)*

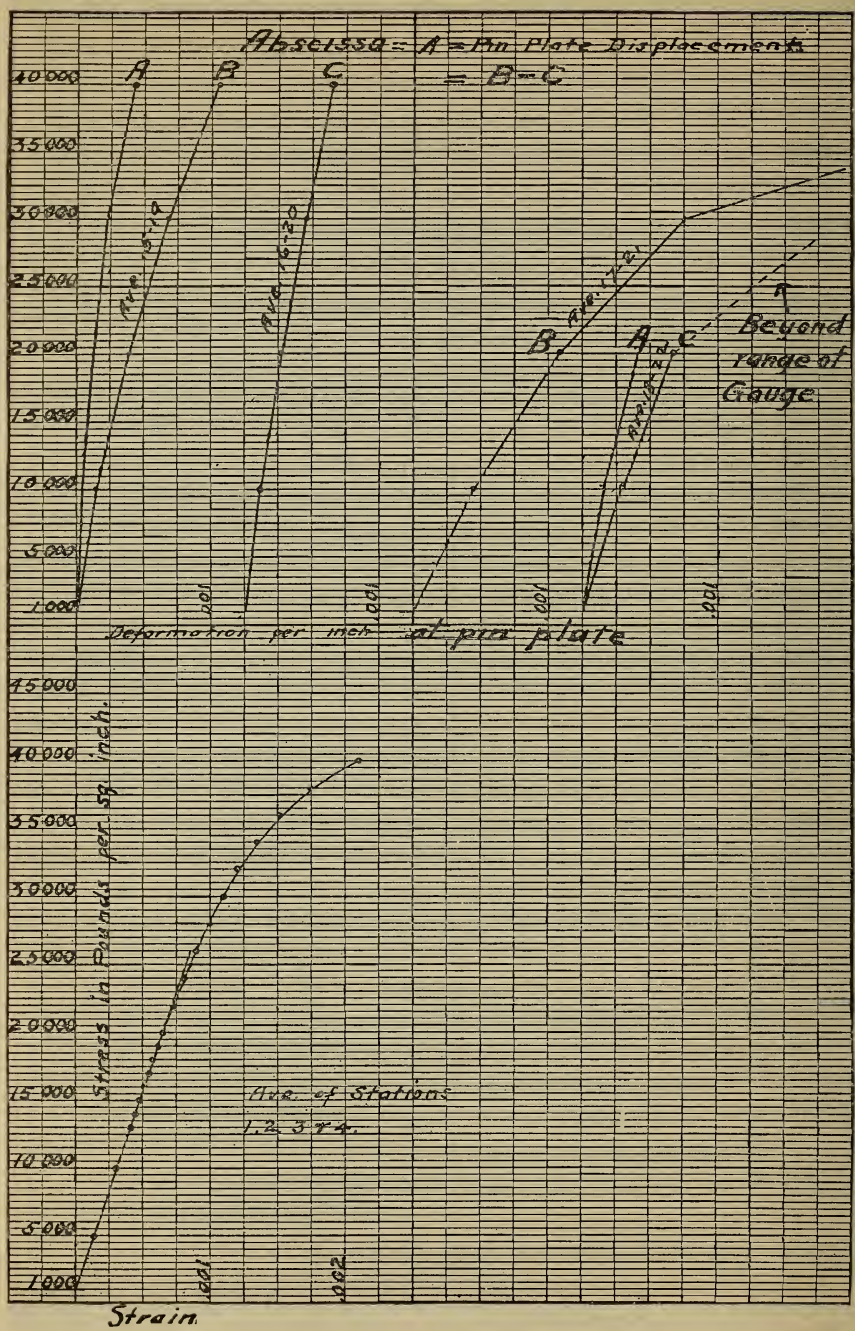


FIG. 32.—*Column 31HC1 (carbon steel)*

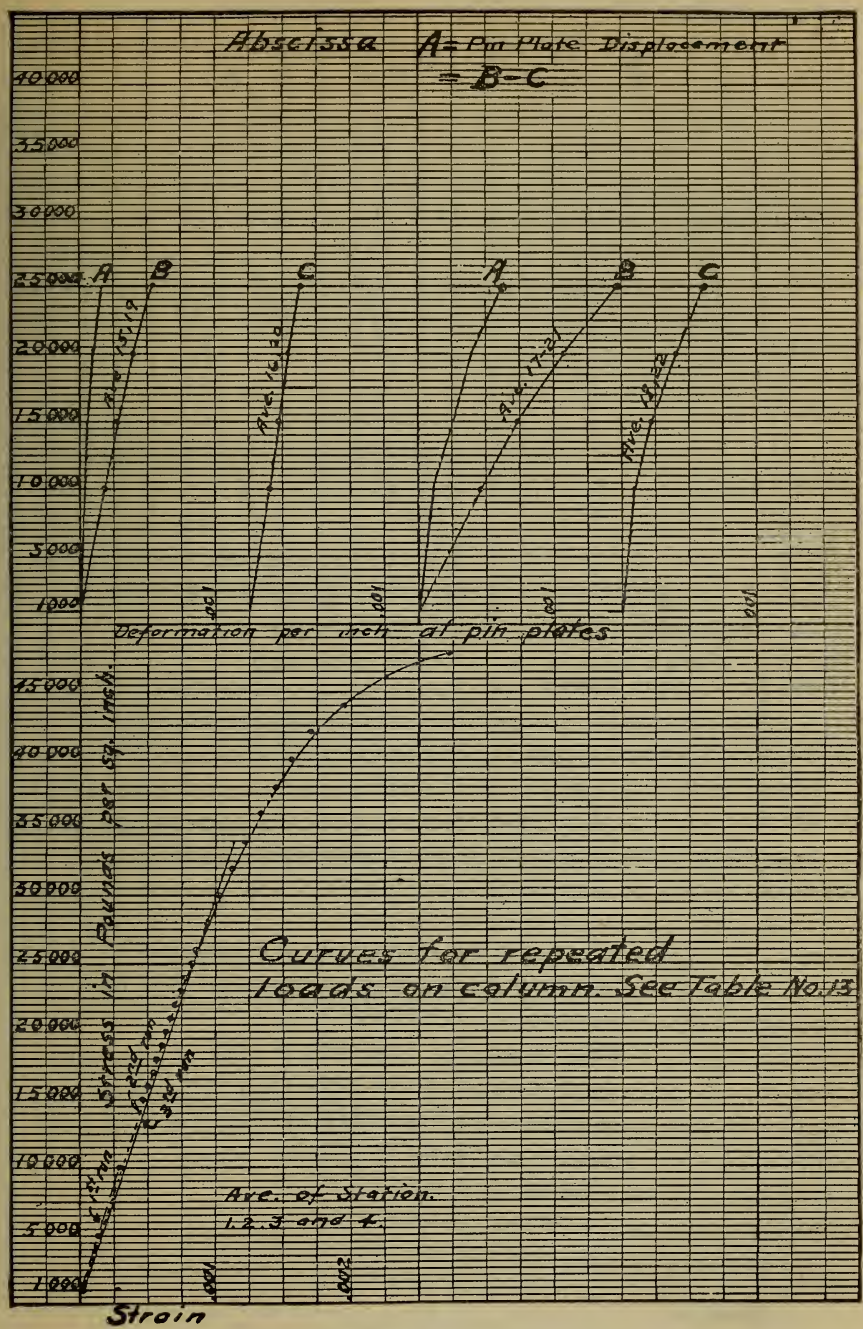


FIG. 33.—*Column 32S1 (silicon steel)*

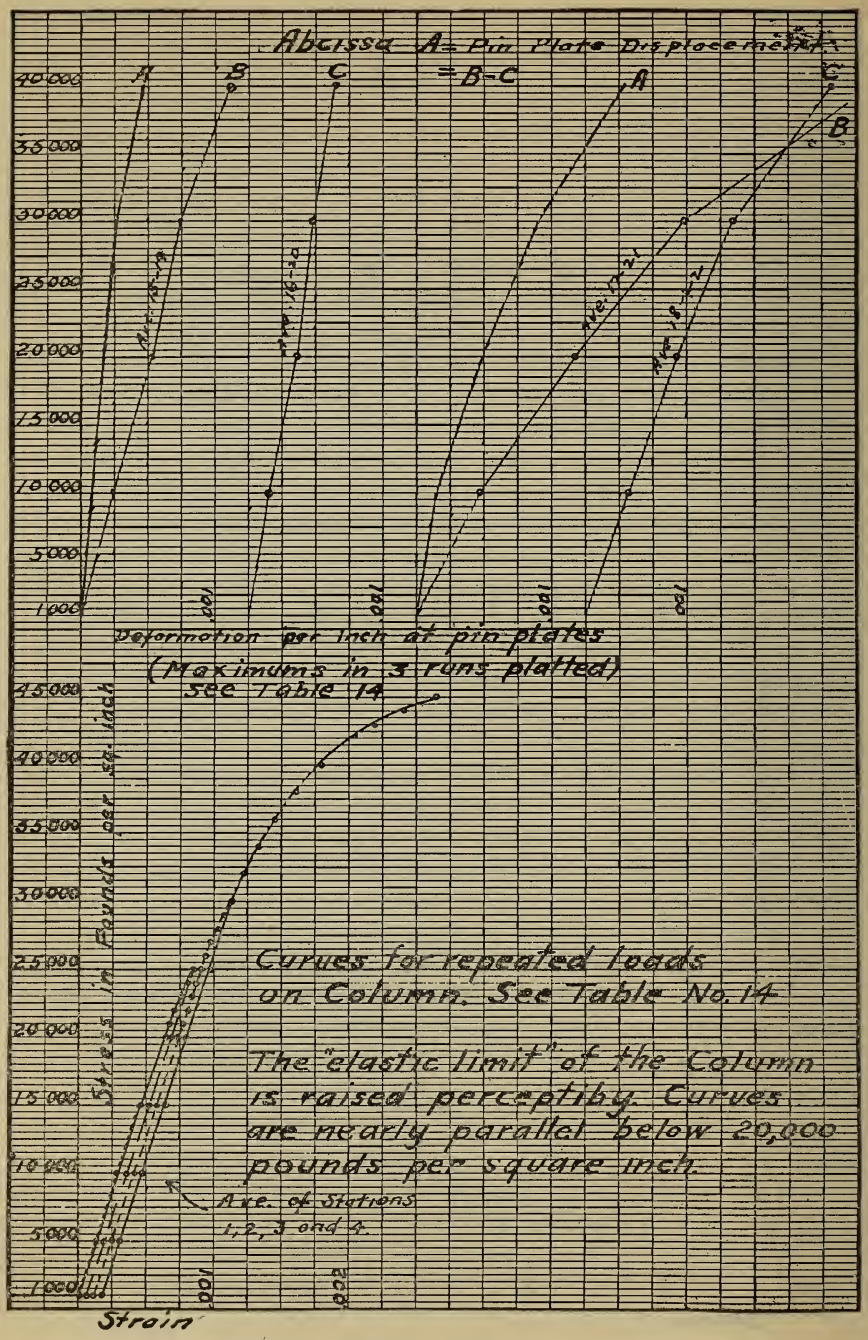


FIG. 34.—*Column 34CS1 (chrome steel)*

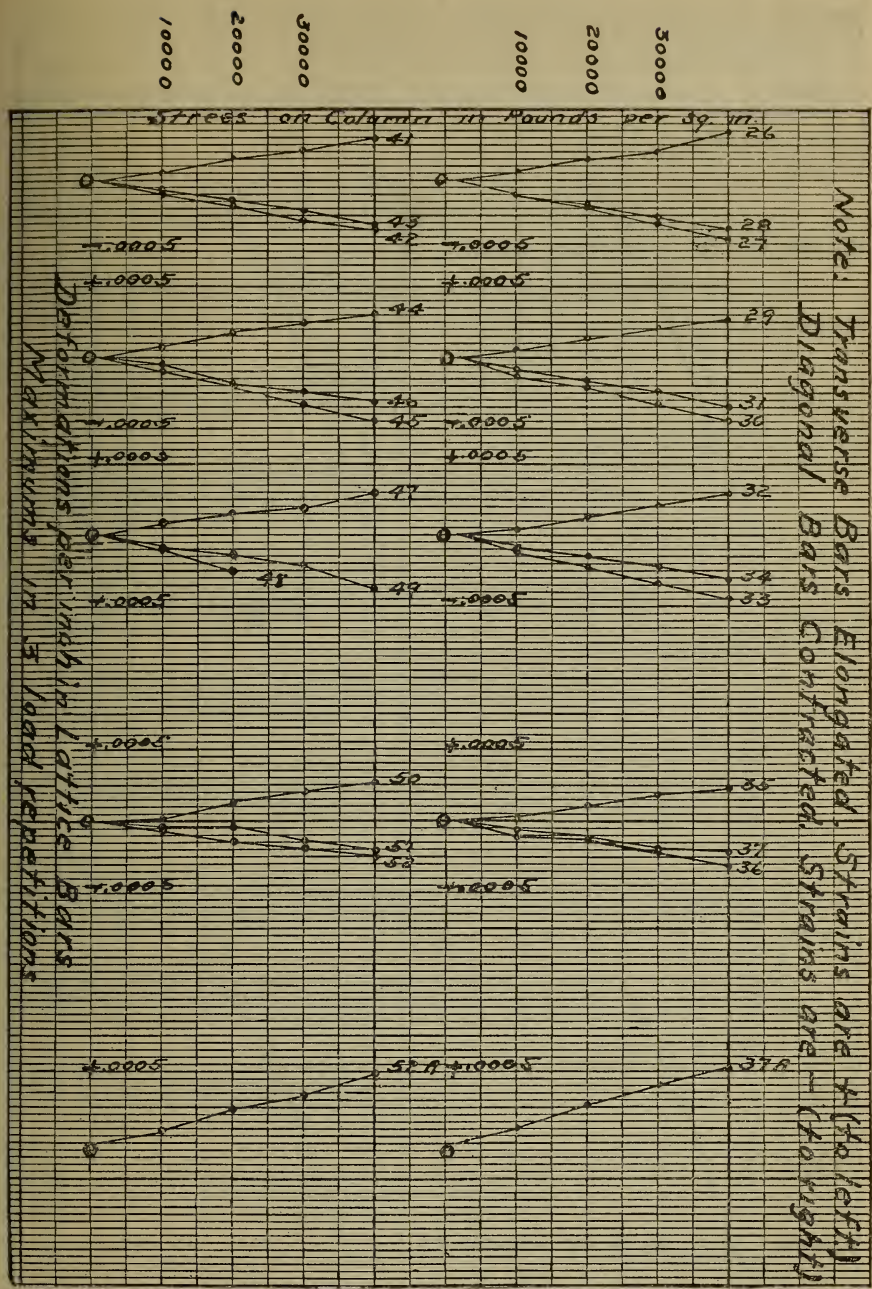


FIG. 35.—*Column 34CS1 (chrome steel)*

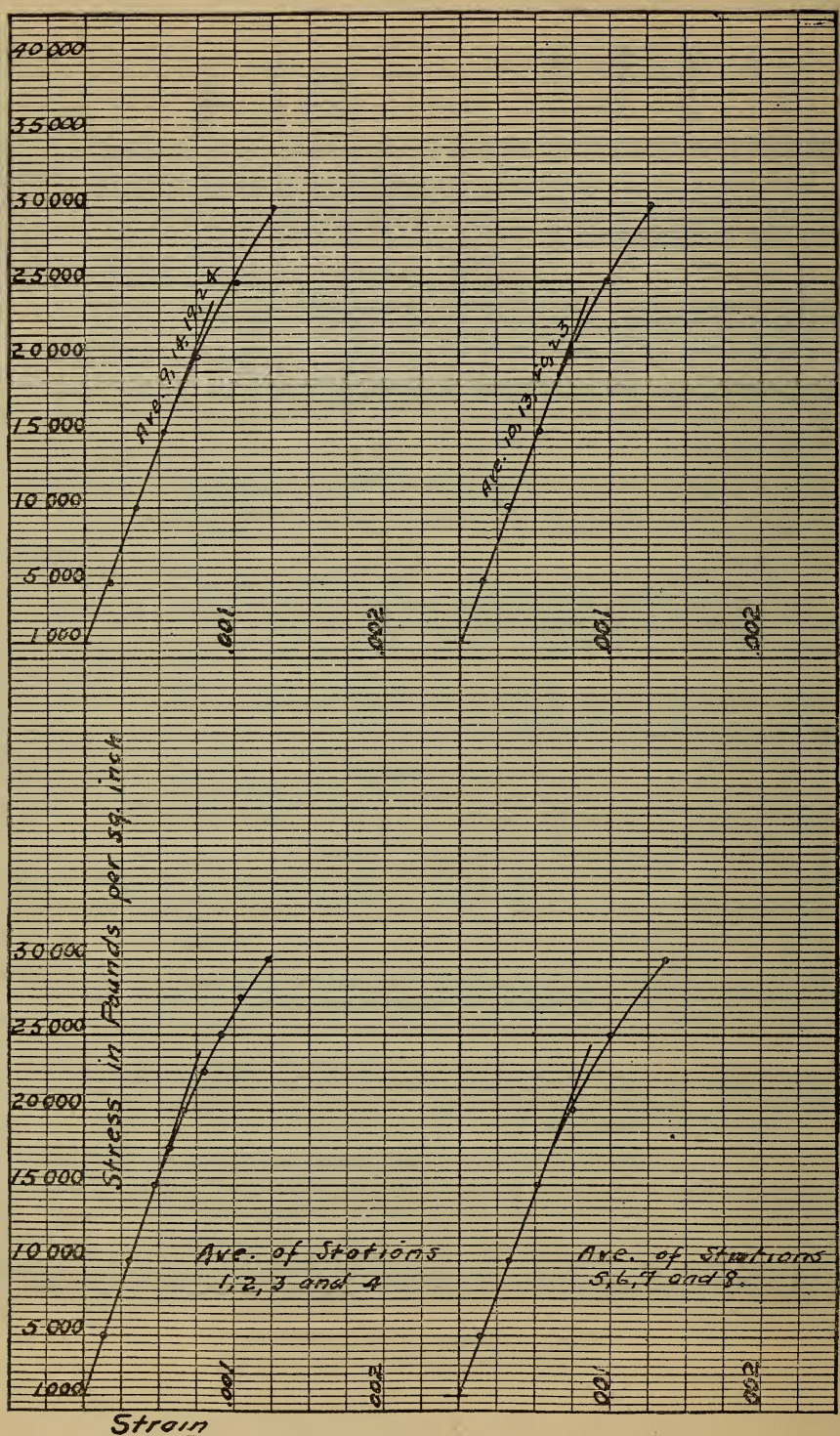


FIG. 36.—*Column US_2 MS_1 (carbon steel)*

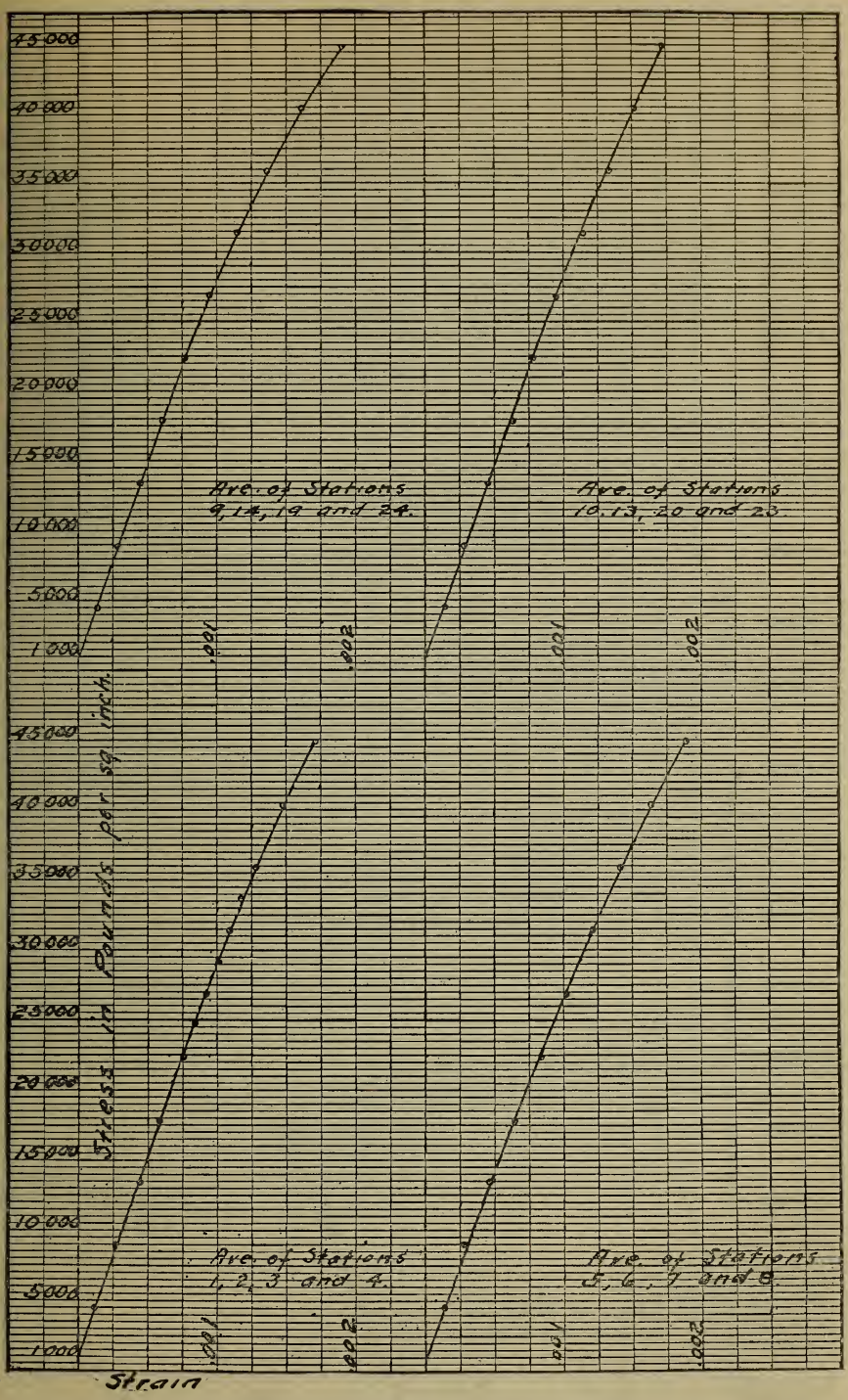


FIG. 37.—*Column US_2 MS_1 (Mayari steel)*

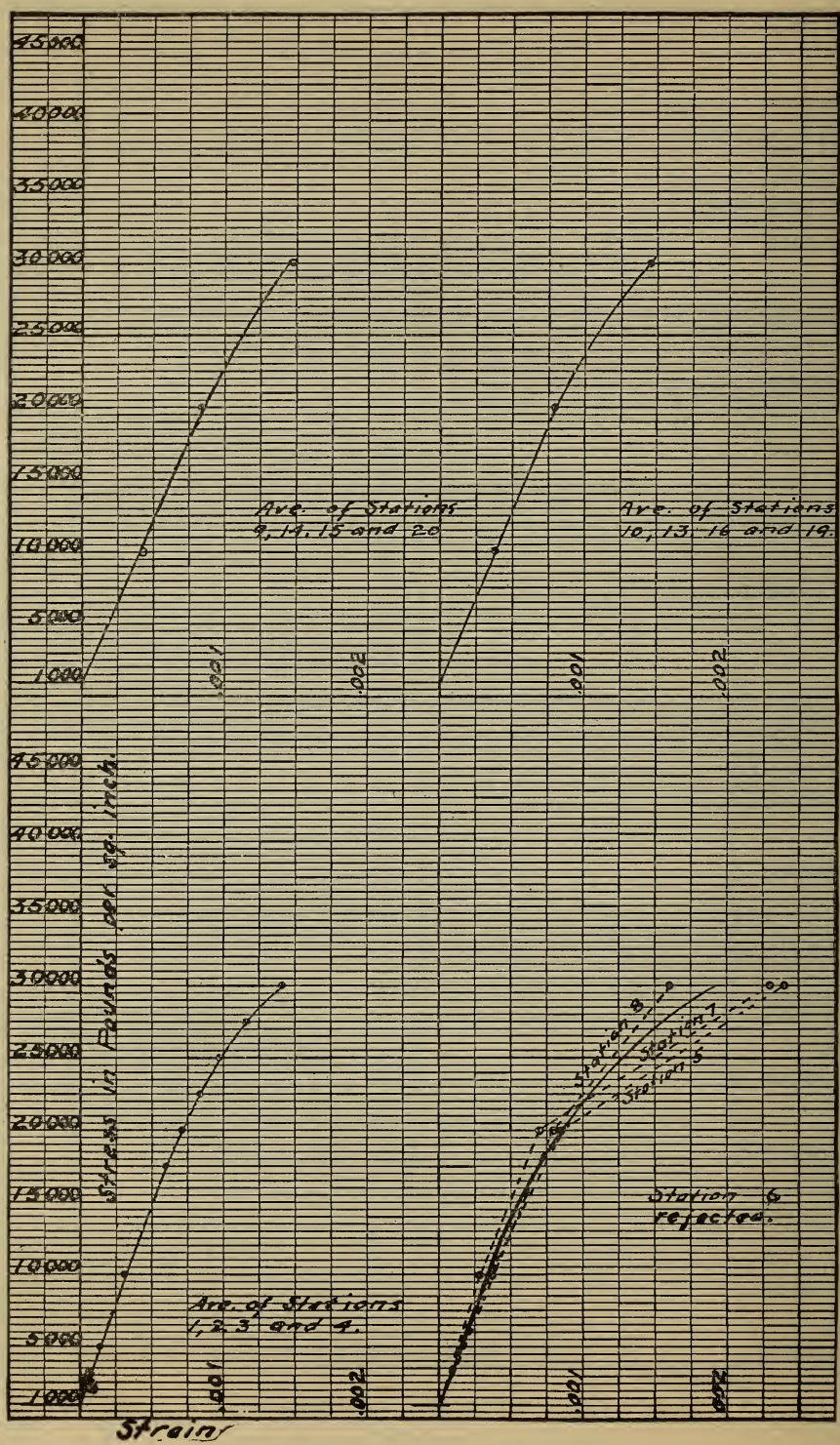


FIG. 38.—*Column U_2 M_1 (carbon steel)*

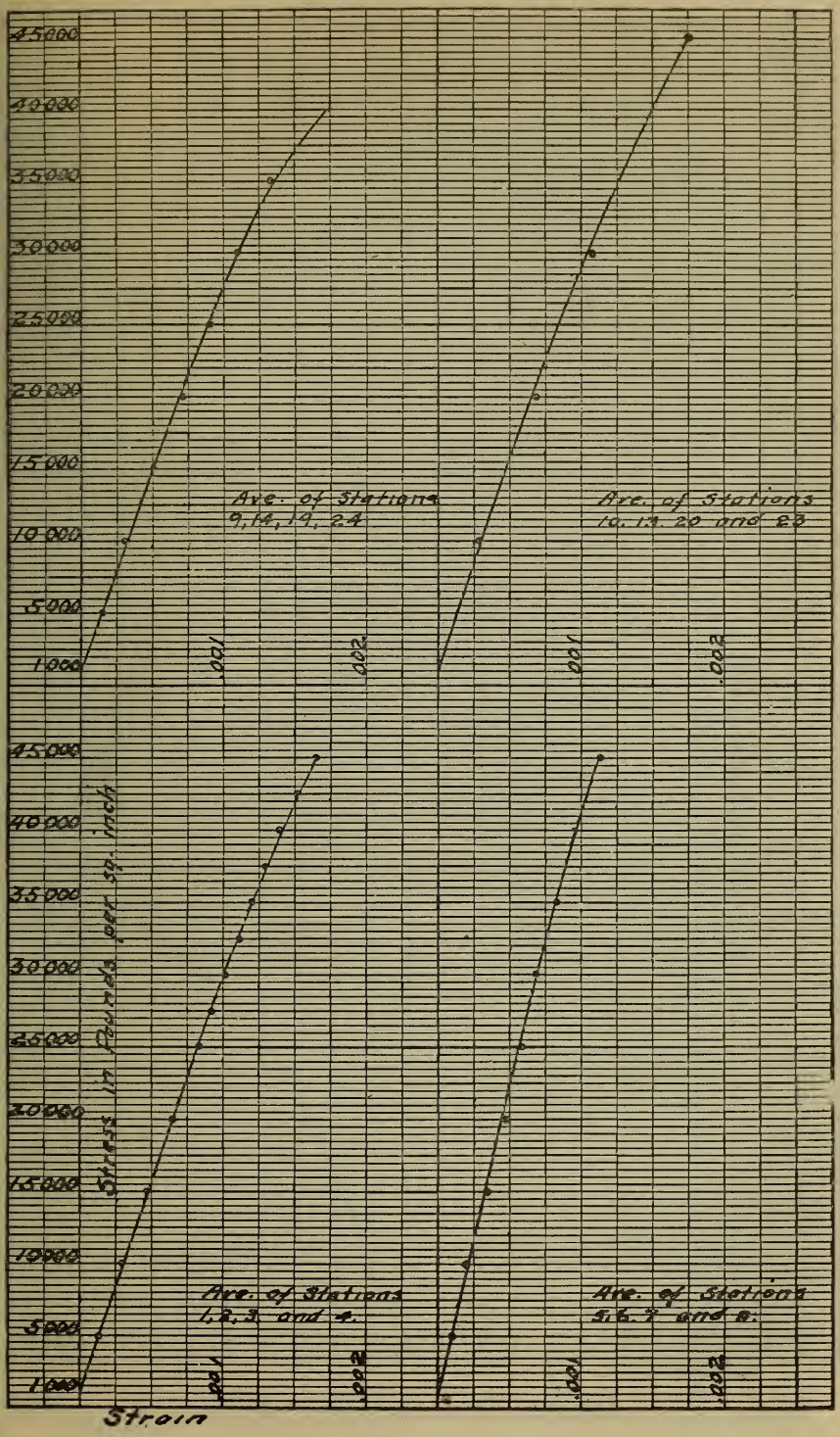


FIG. 39.—*Column U_2 M_1 (Mayari steel)*

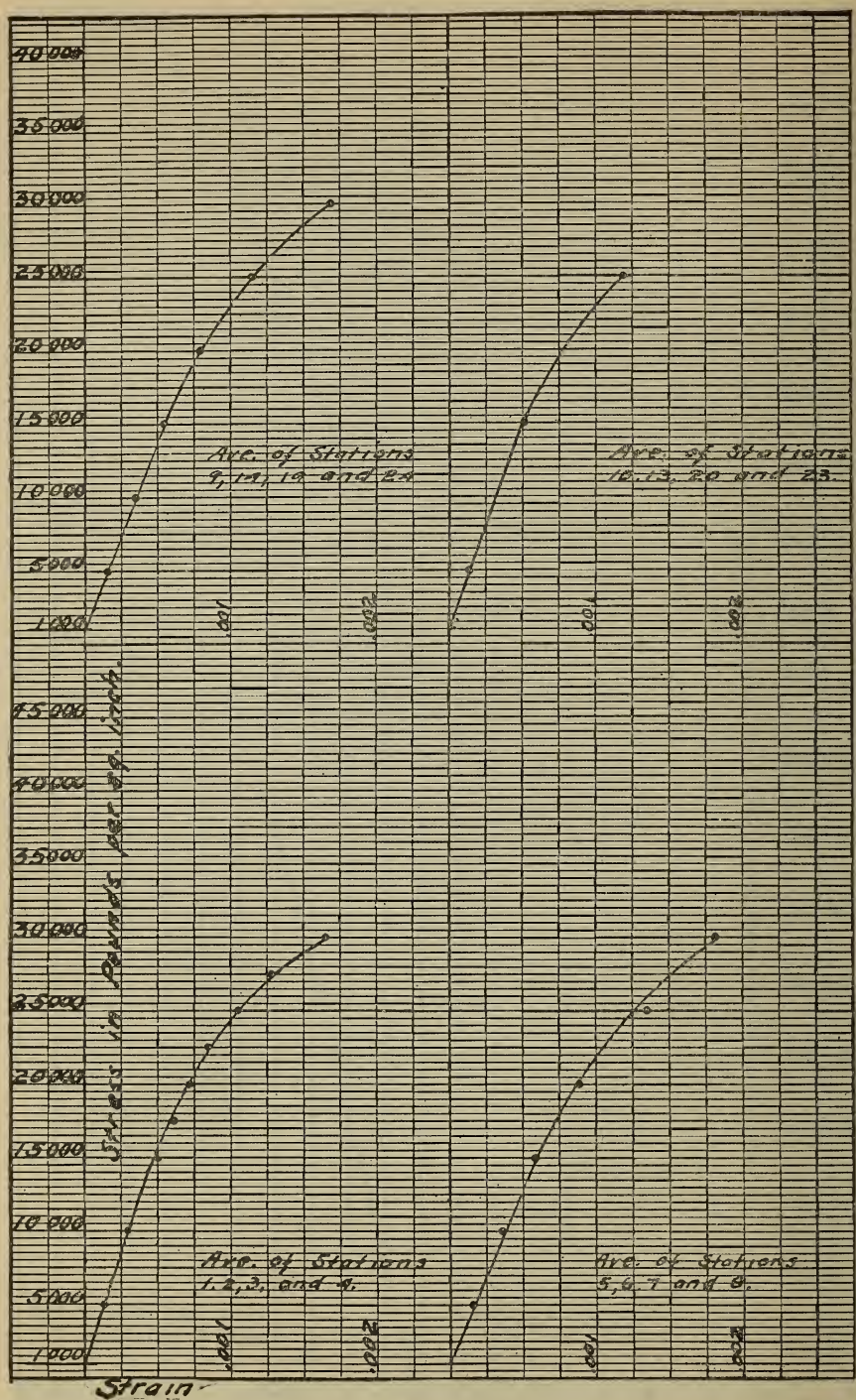


FIG. 40.—*Column LC_0 LC_1 (carbon steel)*

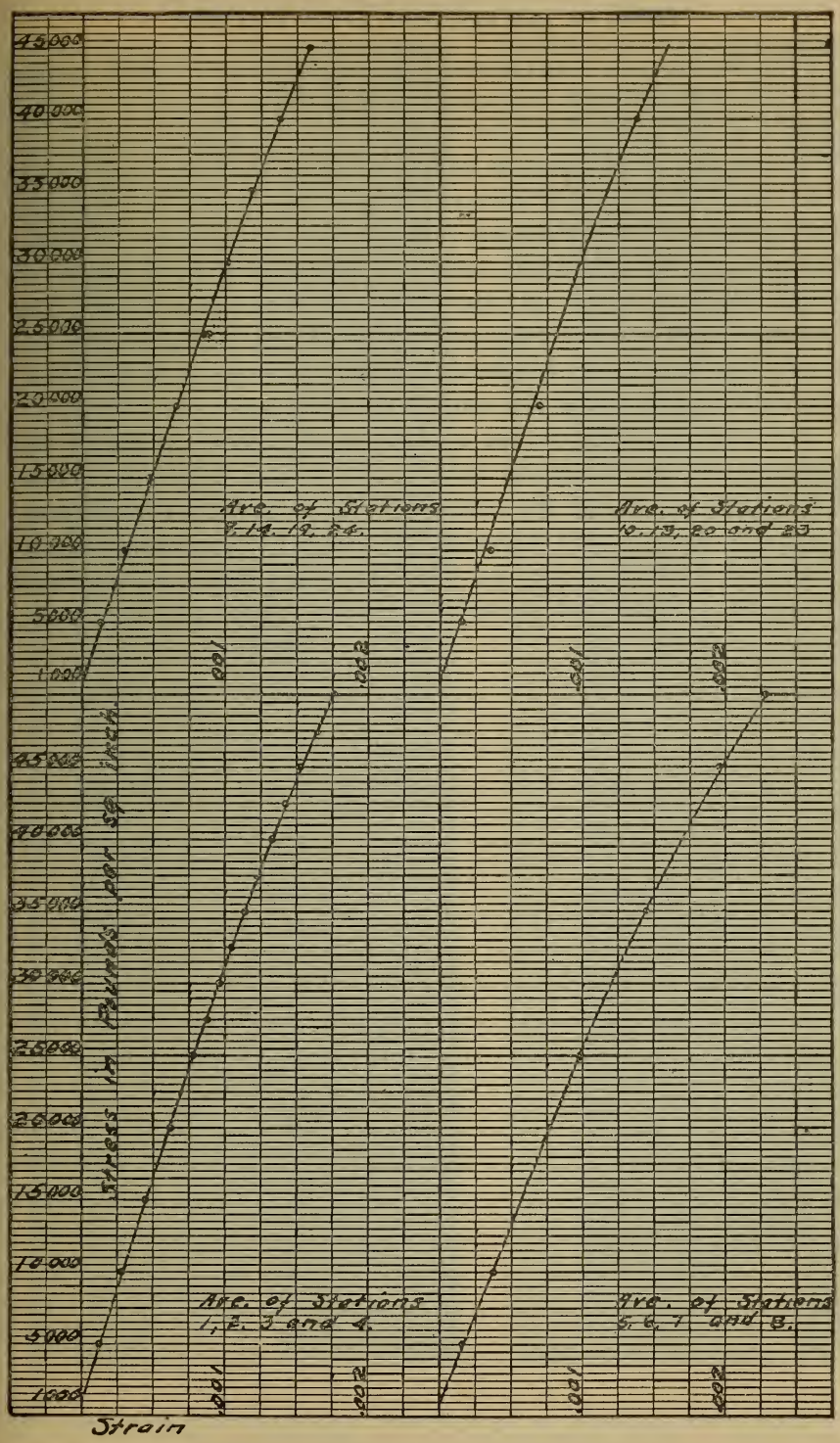


FIG. 41.—*Column LC_0 LC_1 (Mayari steel)*

VIEWS OF THE COLUMNS AFTER FAILURES.

The photographs from which Figs. 42 to 59 were made, with a few exceptions, were furnished through the courtesy of the American Bridge Co. and the Pennsylvania Steel Co.

In the case of the columns failing locally at the ends, the failures were very abrupt, and occurred practically simultaneously with the maximum loads obtained. In the case of the other columns, the amounts of distortions were as a rule somewhat in excess of those which occurred at the points of maximum loads. The pressures were applied continuously for several minutes after the maximum loads were obtained in order to accentuate the characteristic distortions of the column structures.

Attention is called to the continuous deflection curves of the columns of the Metropolis Bridge, and to the behaviors of the lattice bracing in all the columns. The characteristic slipping of the pin plates is well shown but in a slightly exaggerated degree by the white lines of Fig. 43, the "stepping" being a little more pronounced than that actually occurring under the maximum load.

FIG. 42.—*Columns U_2U_3 and U_8U_9 (nickel steel)*

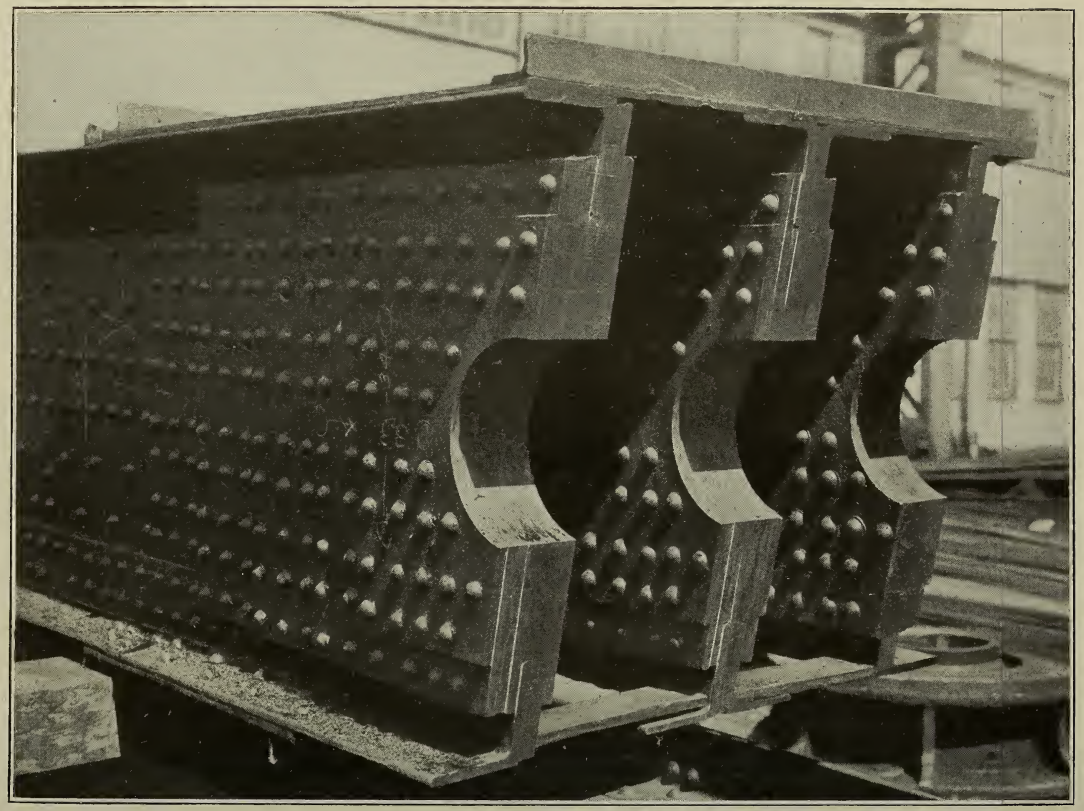


FIG. 43.—*End view of U_2U_3 after testing, showing movement of pin plates*

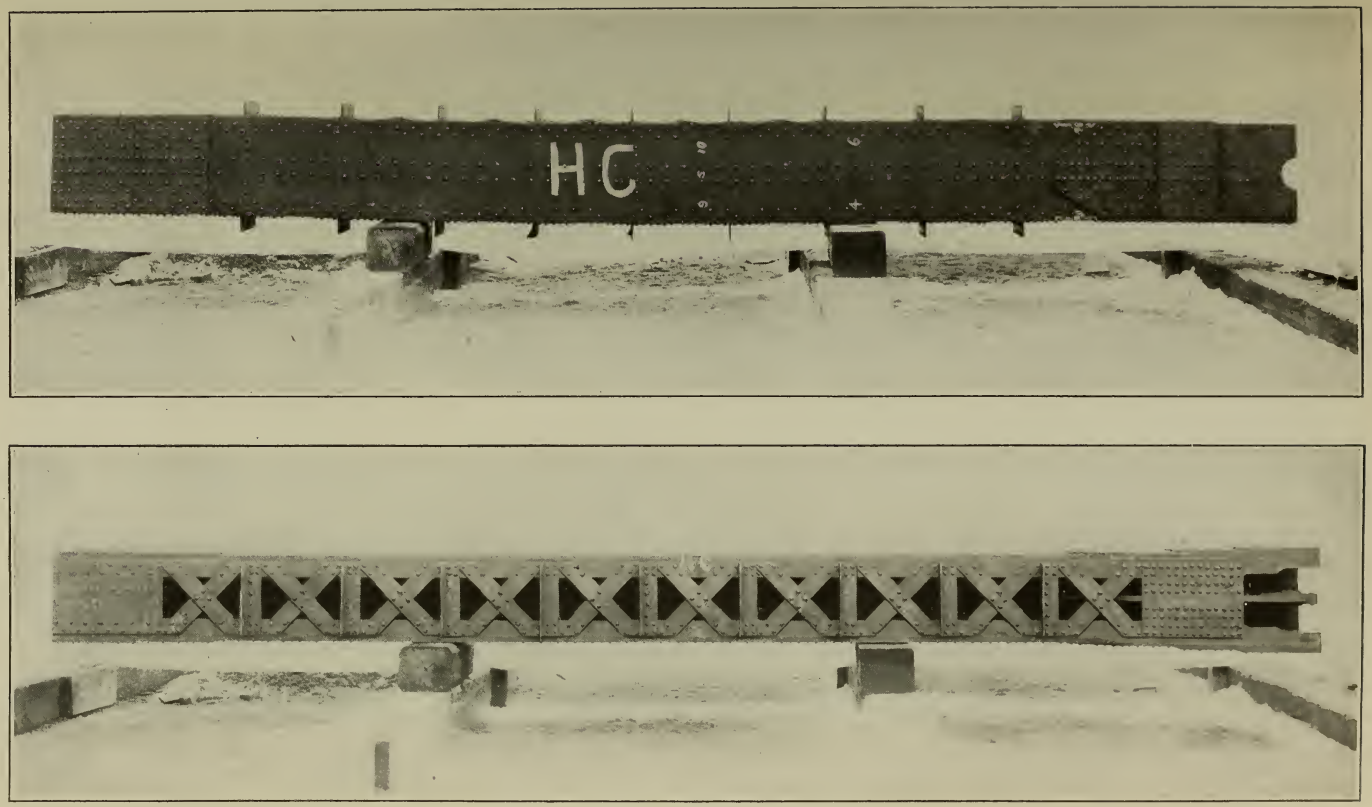


FIG. 44.—*Column HC (carbon steel)*

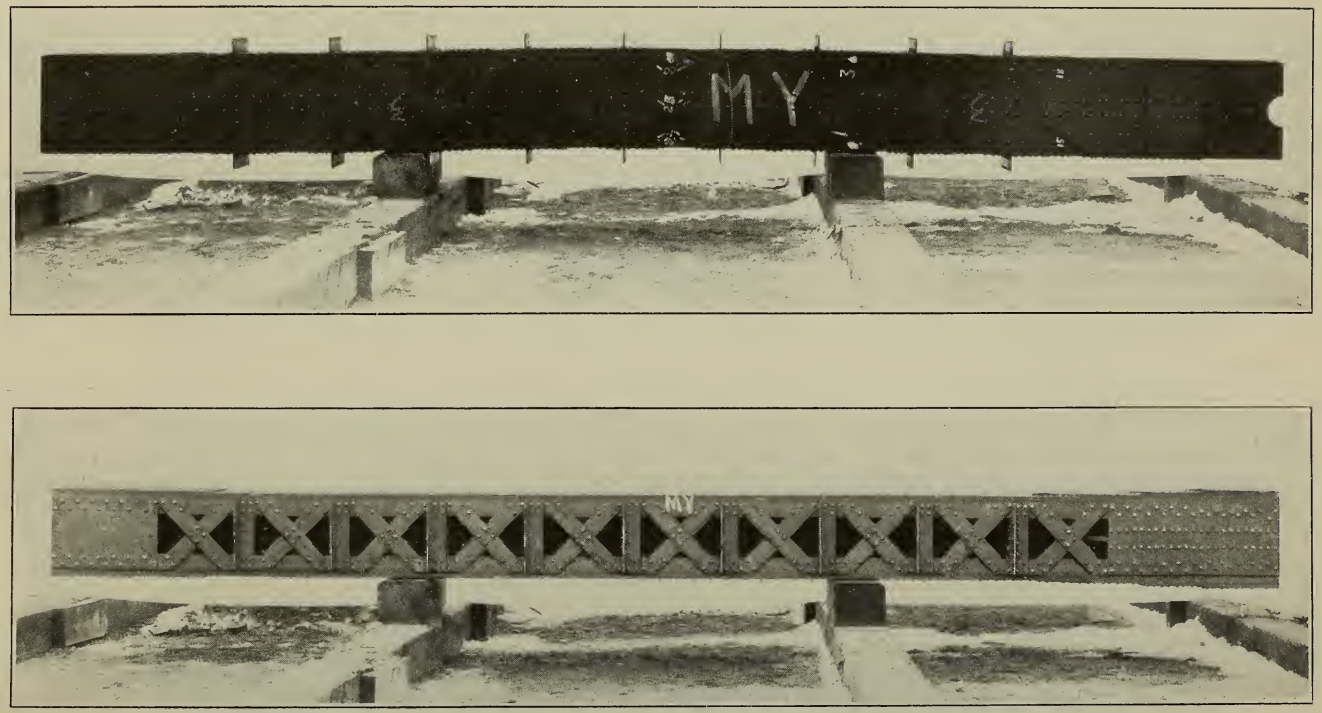


FIG. 45.—*Column MY (mayari steel)*

FIG. 46.—*Column 31HC (carbon steel)*

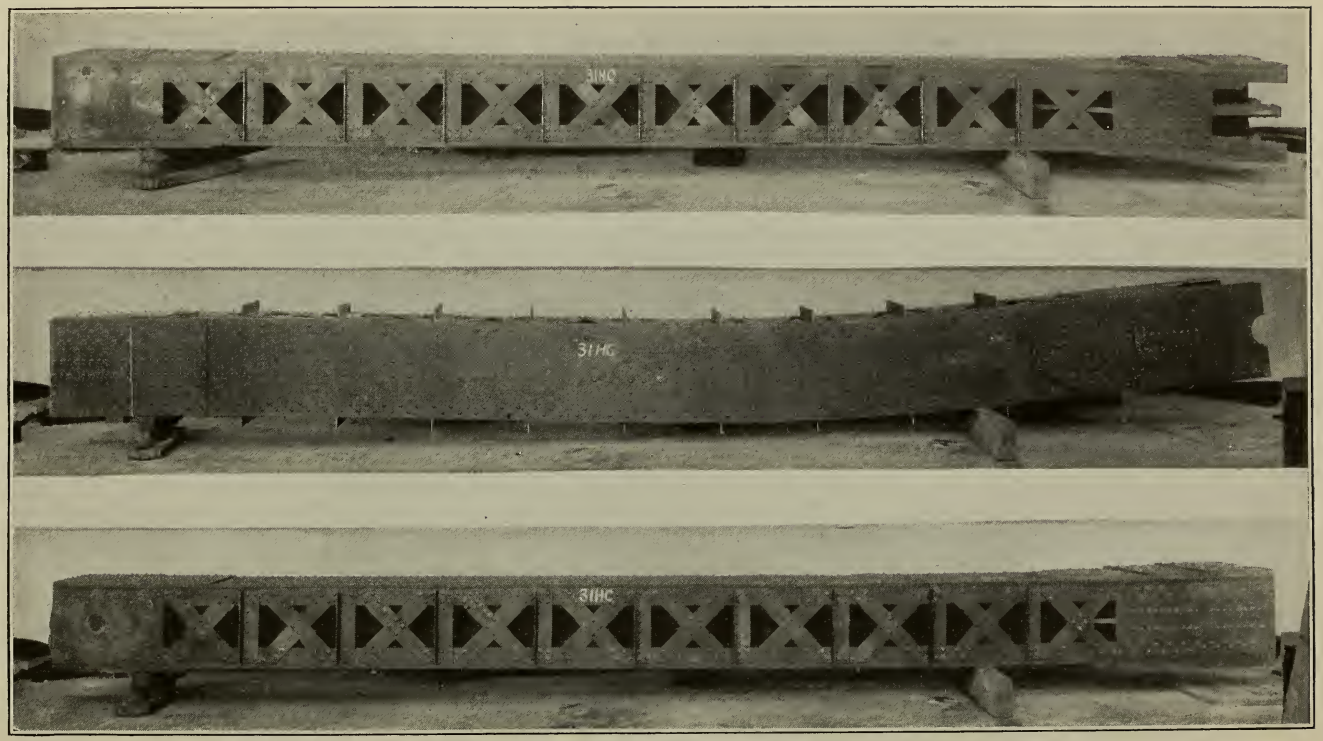


FIG. 47.—*Column 31HC (carbon steel)*

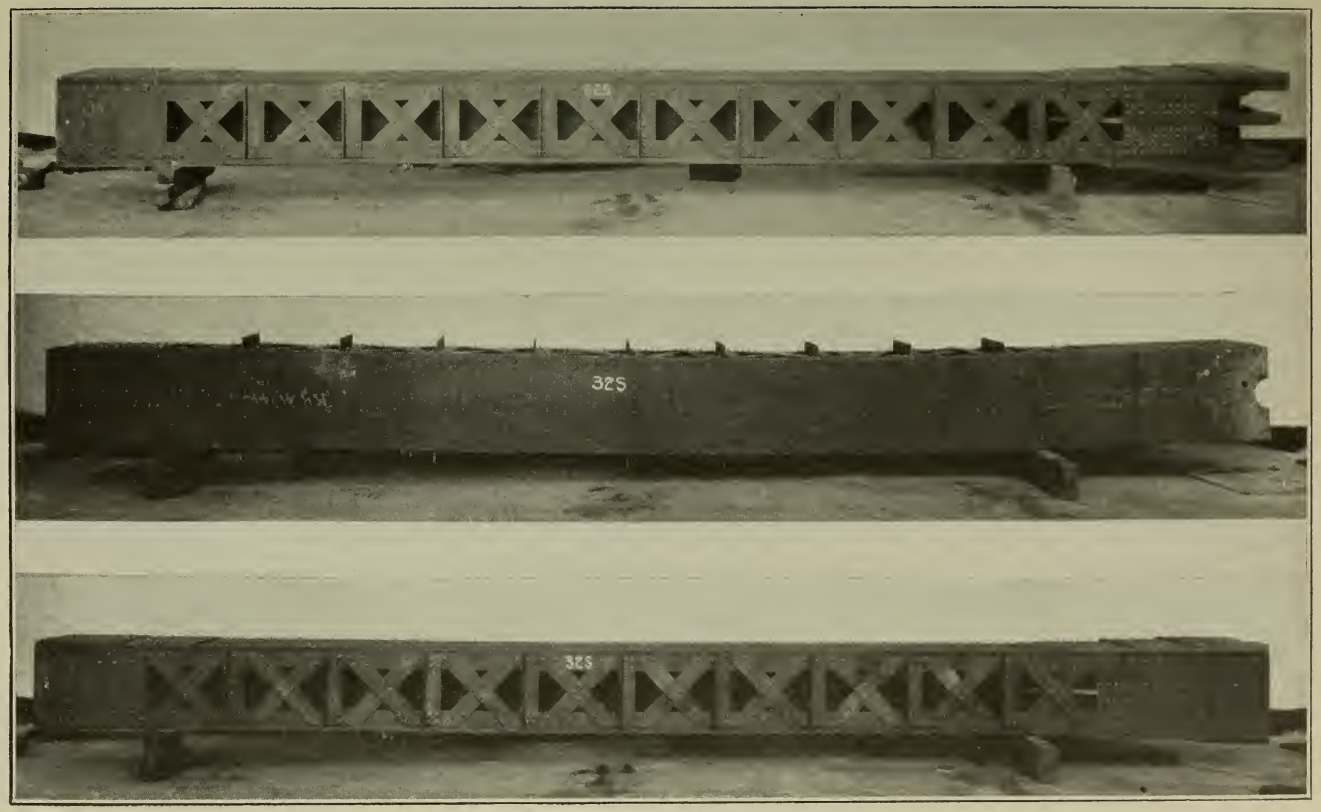


FIG. 48.—*Column 32S (silicon steel)*

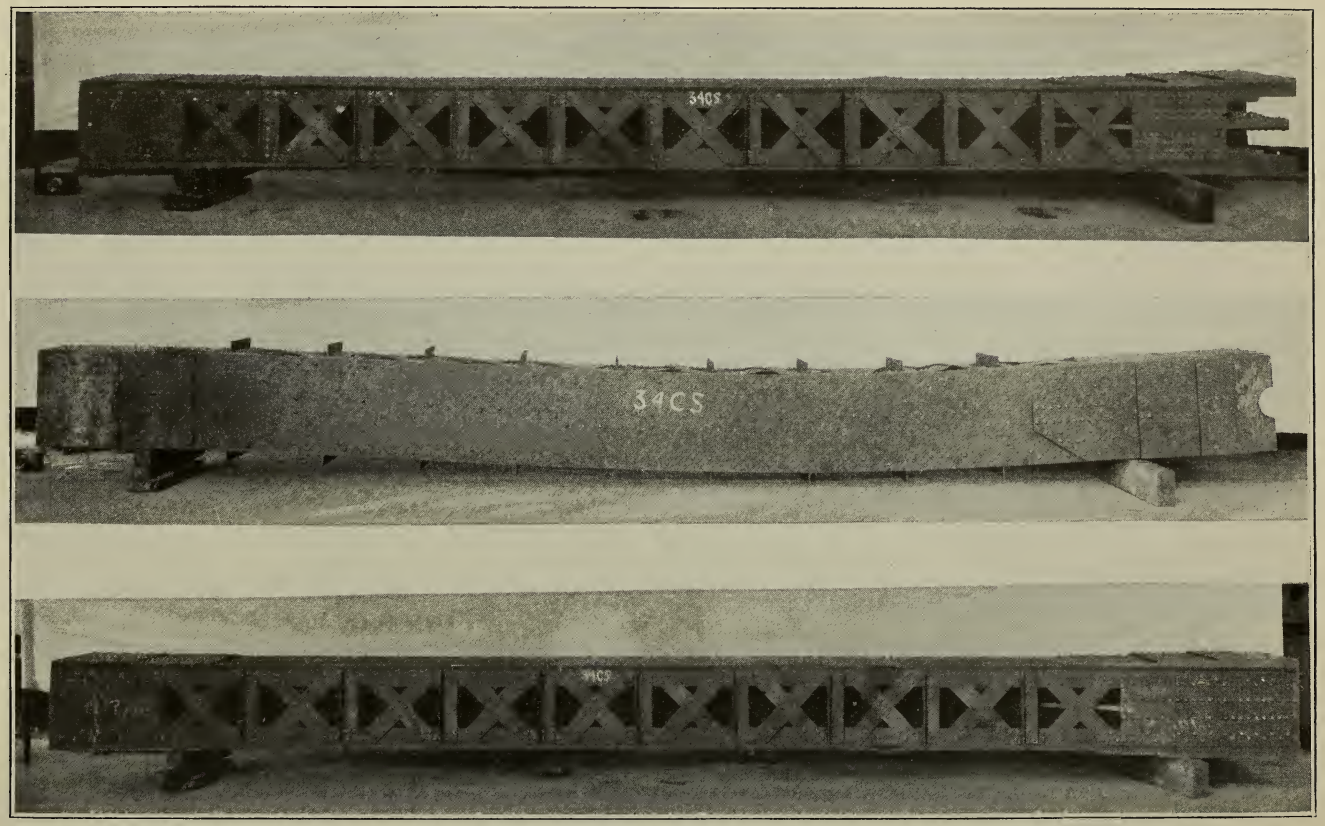


FIG. 49.—*Column 34CS (chrome steel)*

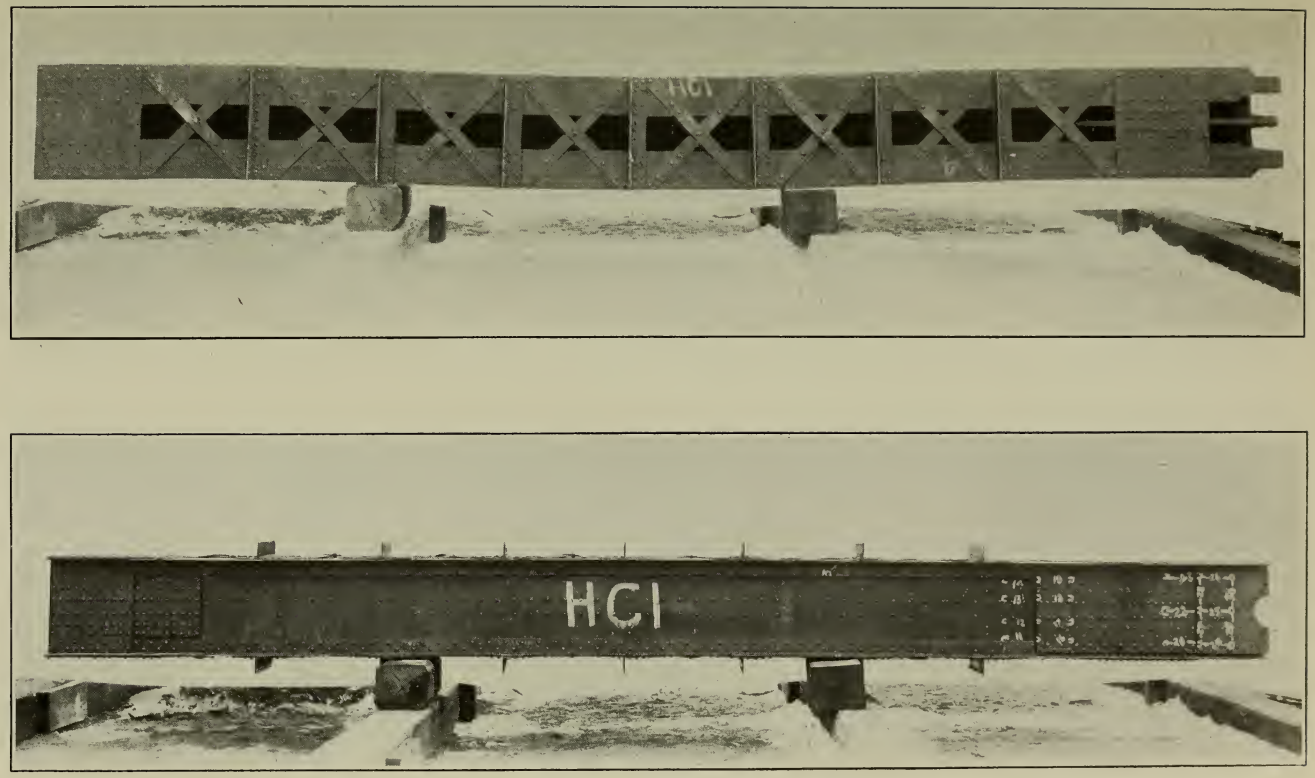


FIG. 50.—*Column HC1 (carbon steel)*

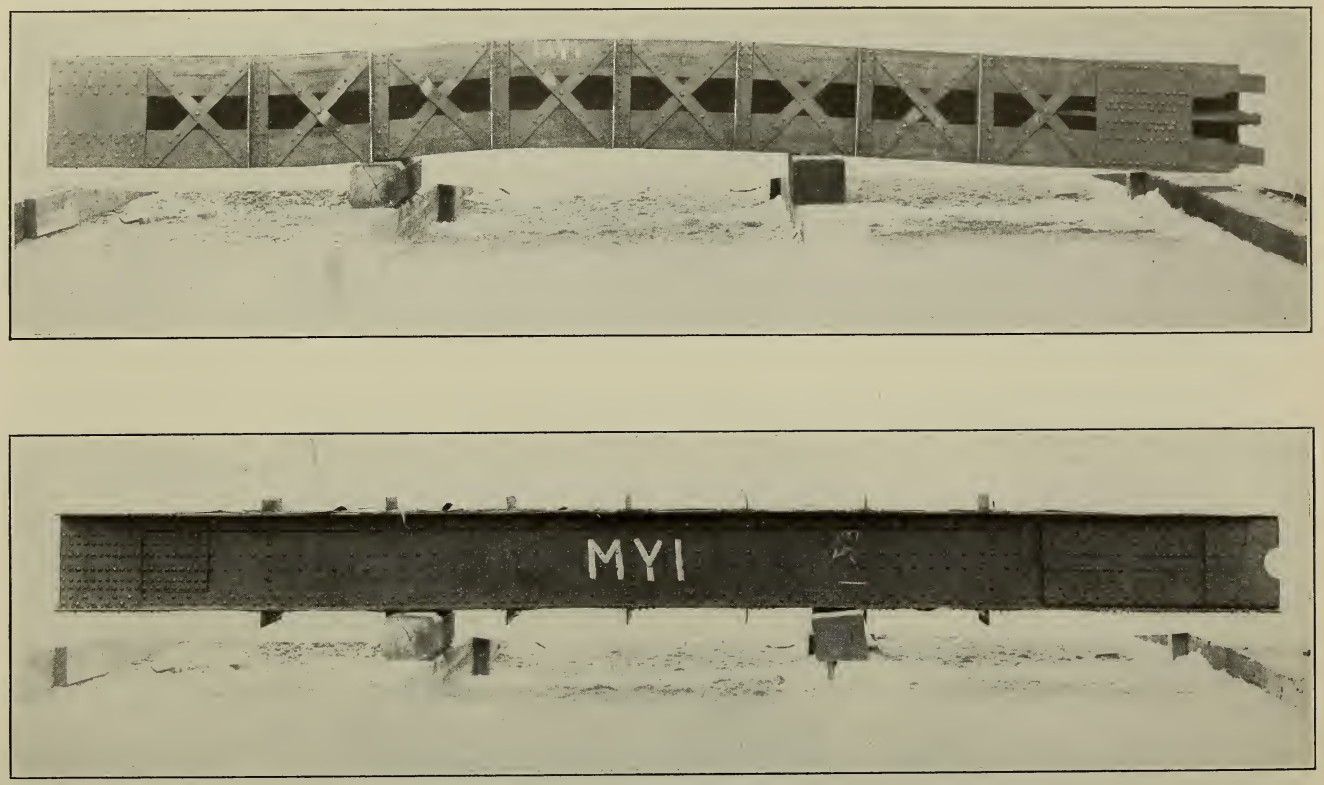


FIG. 51.—*Column MY1 (mayari steel)*

FIG. 52.—*Column 31HC1 (carbon steel)*

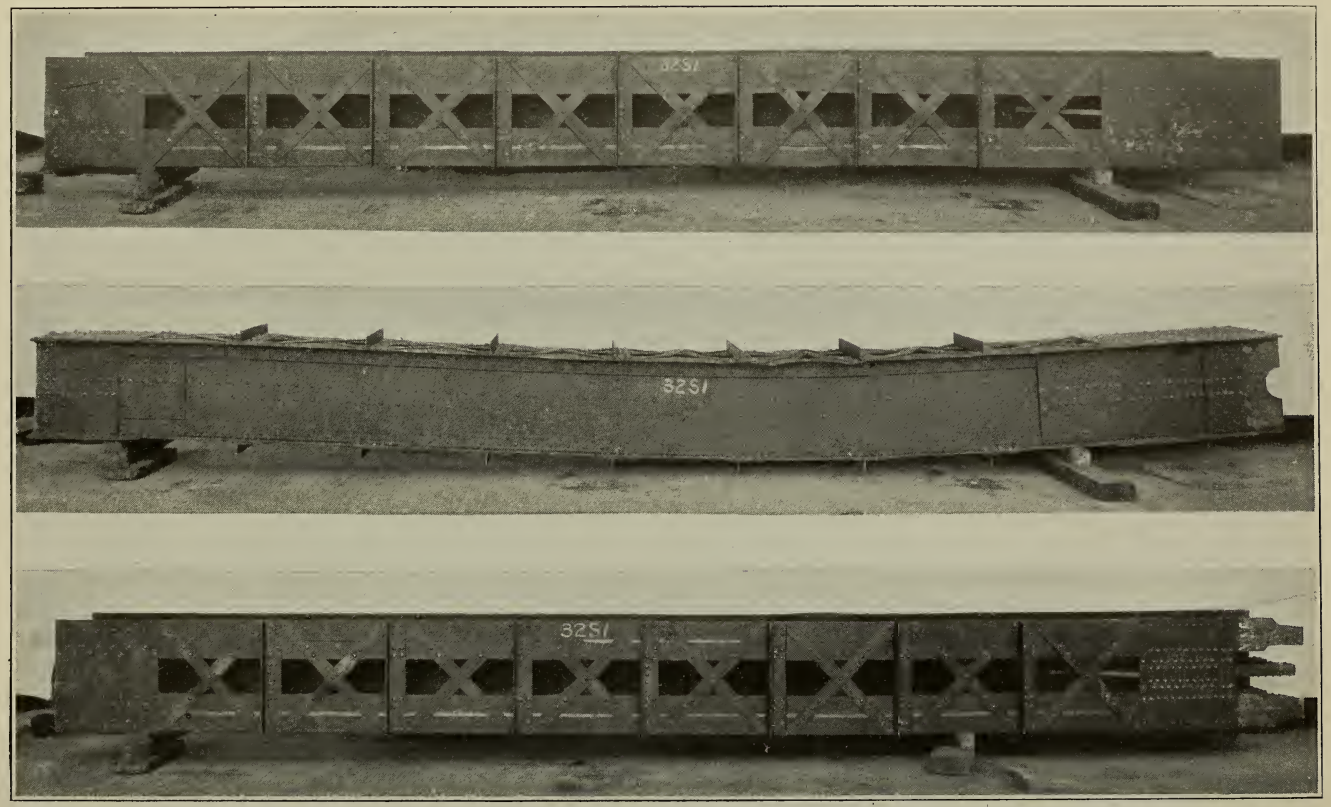


FIG. 53.—*Column 32S1 (silicon steel)*

FIG. 54.—*Column 34CS1 (chrome steel)*

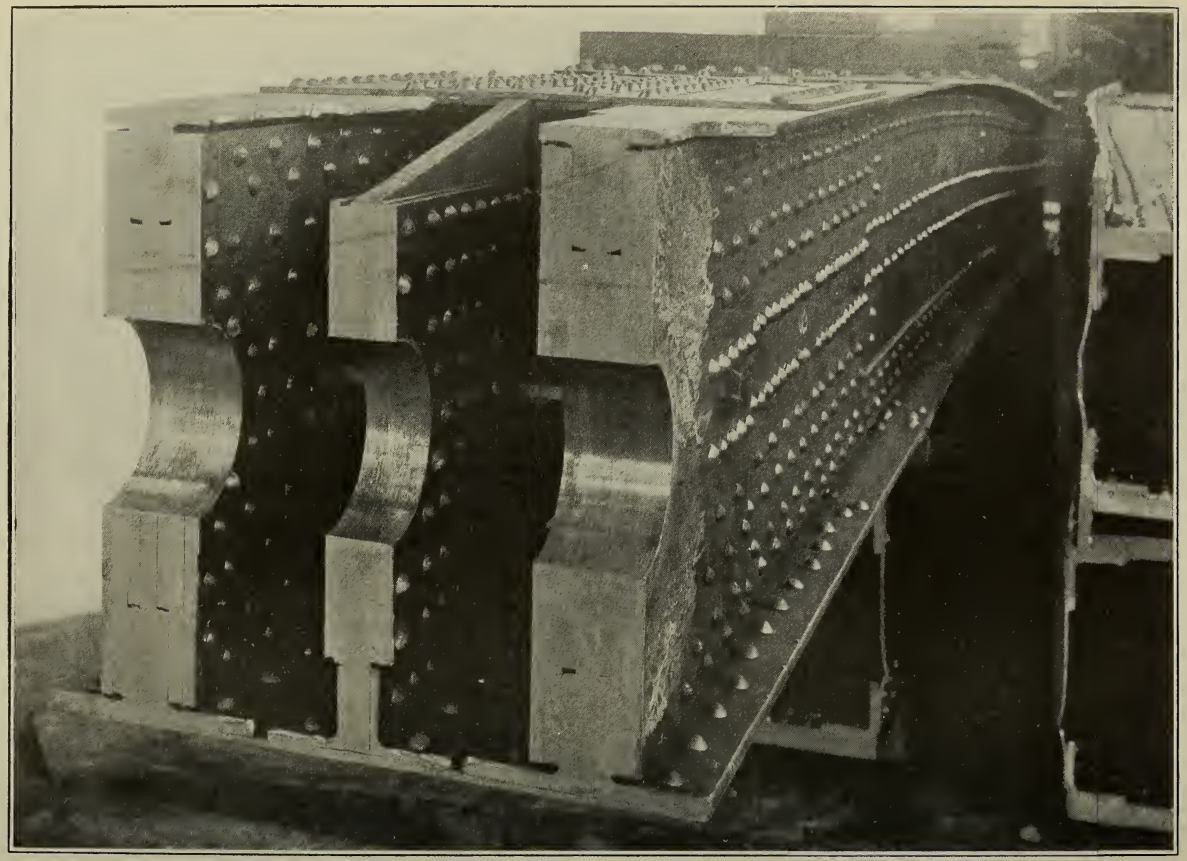
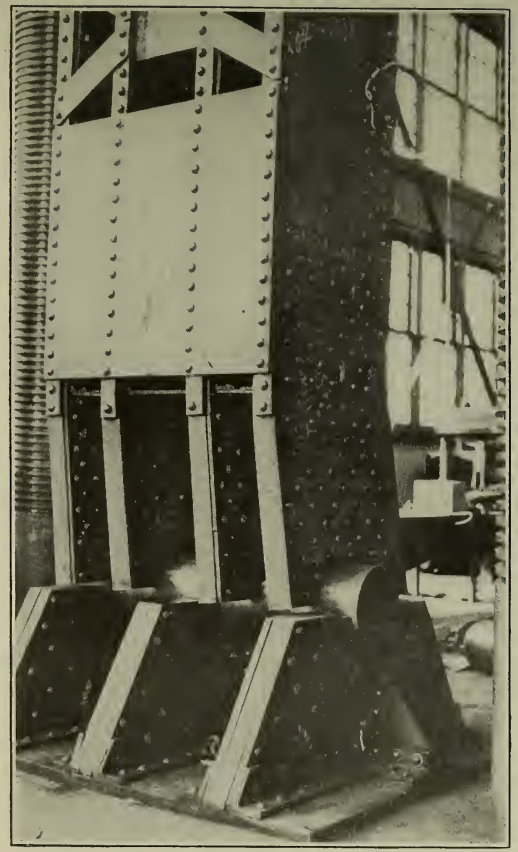


FIG. 55.—*Column 34CS1 (chrome steel)*



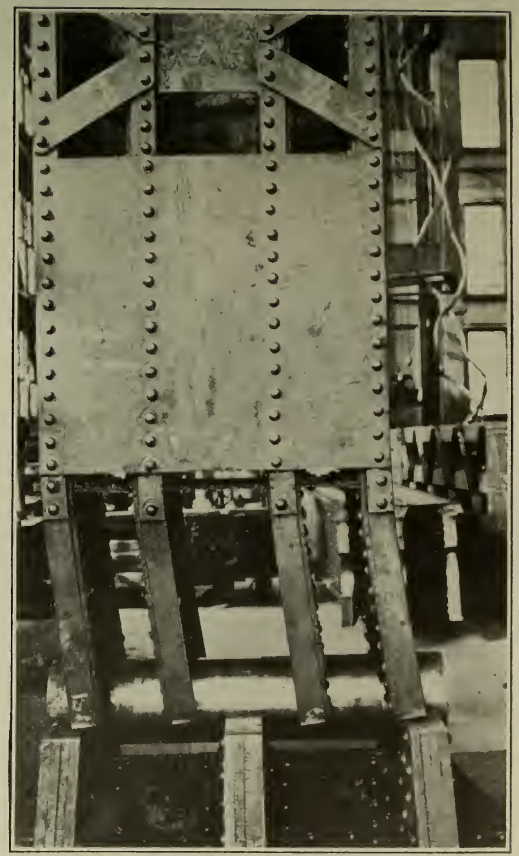
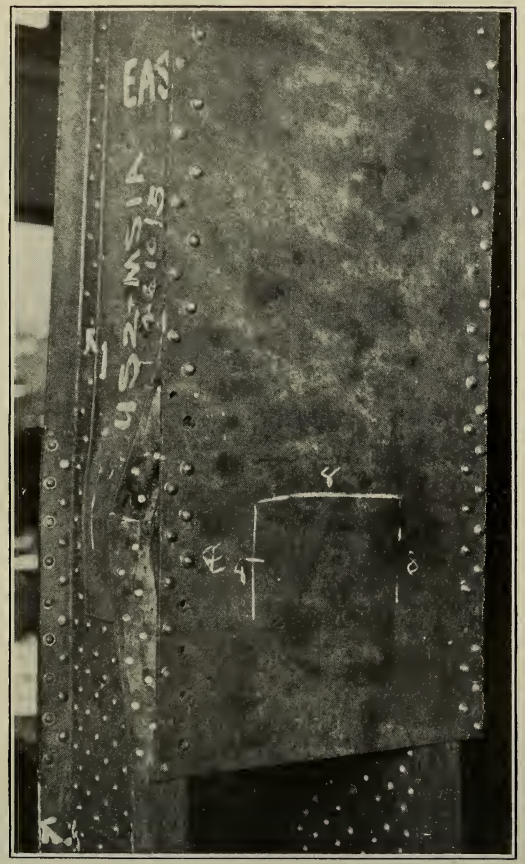


FIG. 56.—*Column* US_2MS_1 (*carbon steel*)



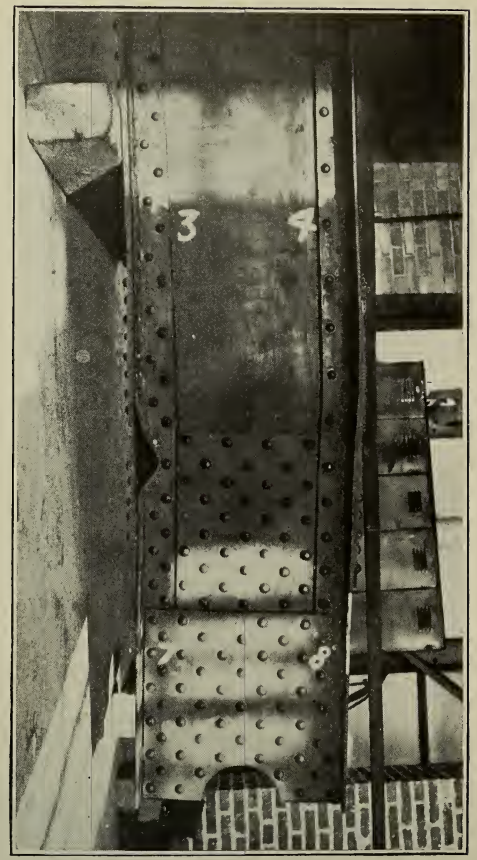


FIG. 57.—*Column US_2MS_1 (mayari steel)*

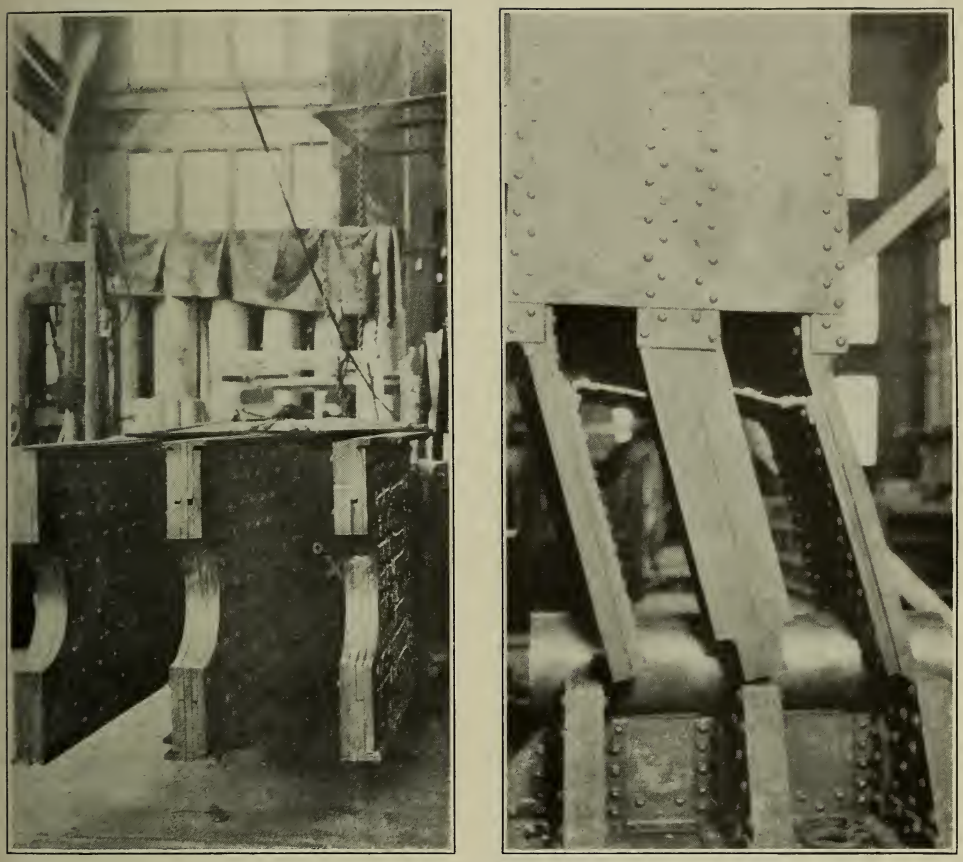
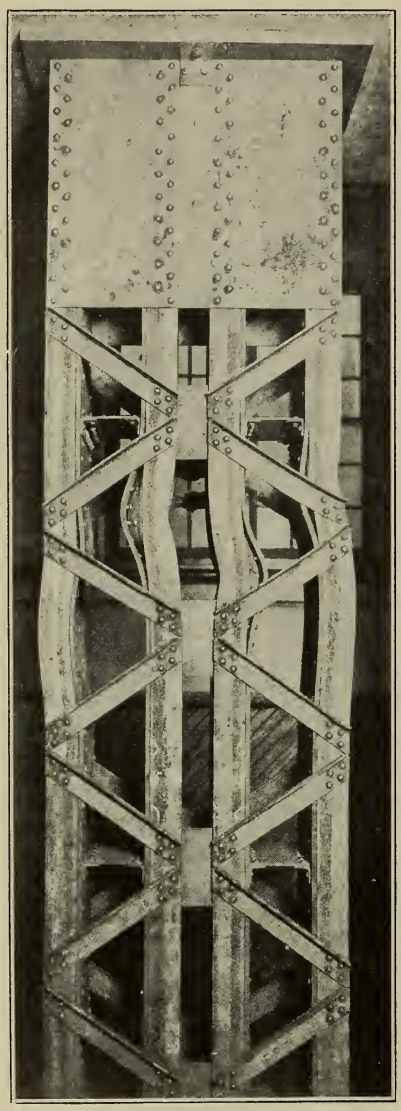
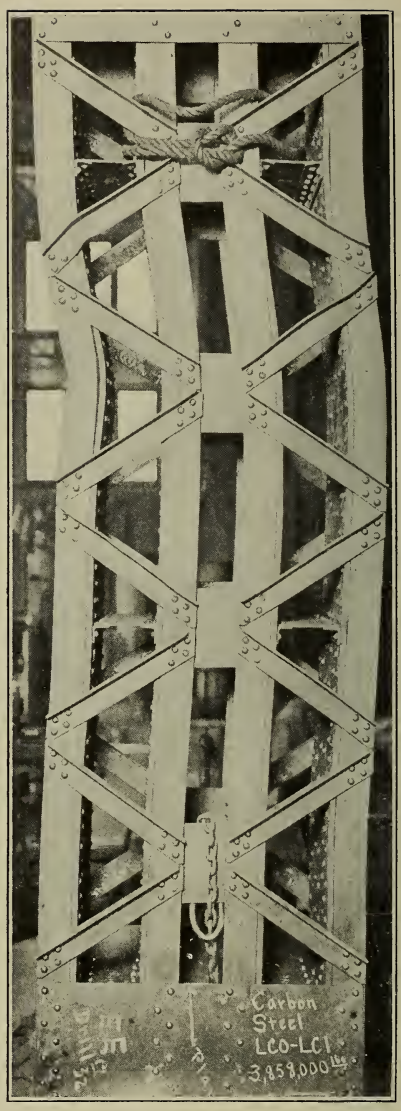


FIG. 58.—*Column U_2M_1 (mayari steel)*



Mayari



Carbon

FIG. 59.—*Columns LC_0LC_1 (Mayari and carbon steels)*

IV. GENERAL DISCUSSION OF RESULTS OF TESTS

1. BEHAVIOR OF THE COLUMNS AS UNITS

It is the purpose of this discussion to analyze first the general laws of behavior of the columns considered as units. The behavior of the details will then be discussed. The test members will usually be treated comparatively and collectively in groups. The tables, stress-strain curves, and views of the columns after tests, already given in Part III, form the basis of the discussion. The main lines of research only are followed, and no attempt is made to give an exhaustive discussion of the results of the tests.

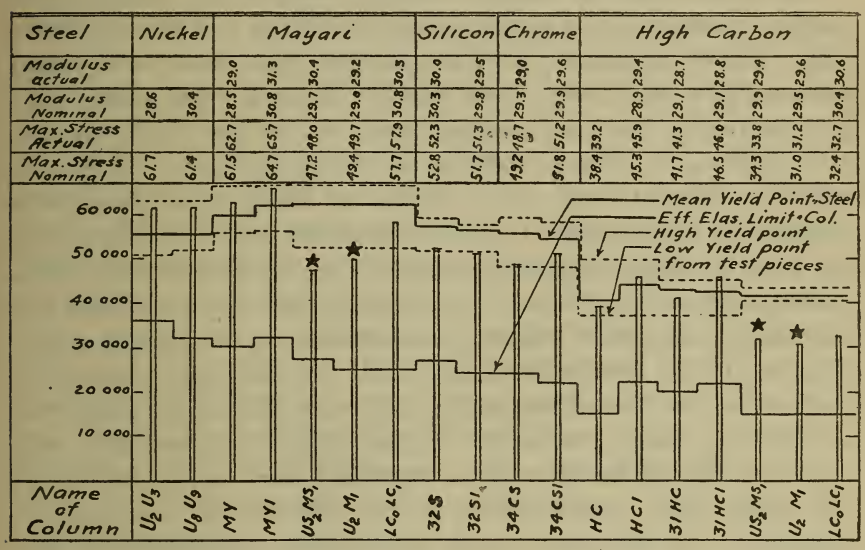


FIG. 60.—*Correspondence between maximum loads and yield points of steel*

(*a*) *Relations of Loads at Failure to Yield Points of Steels.*—The analysis of the behavior of the columns as units is conducted upon the assumption that the applied loads on the columns are uniformly distributed over their respective cross sections. The stress determined in this manner may be considered a first approximation to the real stress distribution. An analysis of the laws of flexure will be given later.

A graphical representation showing the relation of the observed maximum loads to the yield points of the component steels is given in Fig. 60. The data have been plotted as ordinates in descending order with respect to the strengths of the various steels used in the construction of the columns. The observed load on a column at failure is divided by the nominal cross section of the shaft. (See Table 1.) This is recorded in Fig. 60 under the

heading "Maximum stress, nominal." The load at failure is again divided by the actual cross sections calculated from the scale weights of the steel. This is recorded under the heading "Maximum stress, actual," and is also shown to scale by the heights of the respective bands shown in the figure. The heights of these bands in the case of columns U_2 U_3 and U_8 U_9 of nickel steel may be in error ± 1 per cent, as it was impossible to obtain the precise cross-sectional areas, the nominal areas being used instead. The relation of these loads to the yield points of the various component steels is to be considered.

The upper limit of loading which a compression member can sustain, whether it is a large column or a small test piece, is commonly held by engineers to be the sensible yield stress or so-called commercial elastic limit of the steel. Howard [12] has stated that it is impossible that the strength shall exceed this limit, because it is the point at which cold flow of the steel occurs. If the observed maximum load on a column of small slenderness ratio exceeds the yield stress, it may be inferred that there are some abnormal or indeterminate elements to be considered in the tests. Among such as may occur, for example, may be mentioned the restraining influence of the frictional stresses on the platens, a possible raising of the proportional limit by the cold working of fabrication, the lateral restraining influence due to the stress put on rivets in cooling, variations in properties of shapes, etc., with others, which will be discussed rather fully later, and such factors will cause some modifications in the theoretical values to be expected. On the other hand, it is equally important to consider, as has been stated by Waddell,[13] that when a member is efficiently designed and the $\frac{l}{r}$ reduction factor is duly taken into account the load should not fall below the true elastic limit of the steel.

The mean yield points as given in Table 2 are averages of 8 or 10 determinations made on tensile test pieces for the component shapes of each column. The high and low values for the different test pieces have also been recorded. It was impracticable to obtain the actual component yield points in respect to the positions and orientations of the different shapes in each particular column, and a special investigation of the influence of the variation of the yield point over a column cross section has long ap-

[12] Cf. Trans. Am. Soc. C. E., 73, p. 492.

[13] Engineering News, Jan. 16, 1908, p. 62.

peared desirable. It is reasonable to suppose, however, that incipient failures first occur in the steel shapes having the lowest yield points; a local weakening of the columns takes place at this stress and progressive redistributions of load occur from time to time over the cross section. This inference is warranted by the slight variations usually perceptible in the rates of taking load, there being alternate increments and decrements in loads apparent on the scale beam, these occurring in small cycles of action as power is applied from the testing machine.

The curve showing the mean yield points of the columns as found from the different test pieces has been platted on Fig. 60 in order to show graphically its relation to the curve of corrected maximum loads at failure as found from the testing machine which is defined by the locus of the heights of the vertical strips. There is a fair agreement, it will be seen, with the stress at failure except in the case of certain columns (starred in Fig. 60) which failed locally at the ends and two others, the columns LC_0 LC_1 of Mayari and carbon steels, respectively, which strictly did not fail as units. There is a deviation in some cases of several thousand pounds per square inch, as, for instance, in the case of the nickel steel members U_2U_3 and U_8U_9. It will be found, on the other hand, that if the high and low values of the yield points from Table 2 are also platted on the diagram, the extreme curves define a zone which includes the locus defining the failures of the columns at maximum loads for all columns failing as units, excepting 32S1. (silicon), which was just outside the lower edge of the belt and 31HC1, which was just over the upper edge. The points above mentioned tend to confirm the remarks of the late C. C. Schneider, made some three decades ago, relative to the importance of considering the elastic limit instead of the ultimate strength in design, the well-known experiments by Considere [14] confirming this and the later emphasis which has been placed upon the matter by Howard [15] in some large column tests which he recently made.

The relation of failure to yield point may be stated, perhaps, a little more succinctly. The average value of the maximum loads of the 12 columns failing as units will be found to differ but one-half per cent from the grand average of the tensile yield points of the steels as found from the separate groups of test pieces. The difference in absolute value is about 200 pounds per square inch. The range of the separate deviations taken in absolute values per

[14] Johnson's Materials of Construction, 4th ed., p. 361. [15] Trans. Am. Soc. C. E., 73, p. 492.

cent is from 3.7 to 12.5. It is believed to be a reasonable induction from the above considerations that the failure of a large column of small slenderness ratio will occur at or near the yield point of the steel, as is commonly the case with a small test piece in tension or compression. The range in variation between high and low limits evidently depends upon that found from the independent tests of the component steel shapes. If this range can be constricted in practice, the predicted strength of a column may doubtless be made to agree closer with the results of experiment than heretofore.

The ranges between the high and low yield points of this investigation are not as small as can be obtained when special selections are made of the heats by the bridge companies. However, they may be considered reasonable variations from the mean and are fairly representative of the conditions which exist in engineering practice. It is advisable, however, in estimating the maximum load of a column of small slenderness ratio at or near the yield point of the material that the range between the "high" and "low" values should be considered. While the values used may doubtless be weighted proportionately to the strengths of the various steels to give a slightly closer prediction of average range of any particular maximum load from the mean yield point than has just been given, particular refinements of calculation appear unnecessary because of the effect of other variables which influence the apparent maximum loads at failure, and which will be discussed later.

(*b*) *Proportional Limits of Columns.*—It has been seen from the separate stress strain curves plotted for each column, Figs. 22 to 41, that the constant proportionality of stress to strain ceases at a point which is some 50 per cent of the maximum stress at failure which has already been shown to be near the yield point of the material. C. P. Buchanan [16] in his analysis of the tests for the Pennsylvania Lines called this critical point of the curve the "elastic limit" and reserved the name "yield point" for that point somewhat higher on the curve where there again occurs a scarcely perceptible change in the rate of slope of the curve above his "elastic limit" as above defined. The difference in stress is usually about 2000 to 3000 pounds per square inch in the interval. It is considered important by bridge engineers that the ensemble of working stresses, dead, live, snow, impact

[16] Engineering News, 58, p. 685; Dec. 26, 1907.

stresses, etc., when appropriately combined shall never exceed the lower critical point which in this paper is called the proportional limit. In the event of such contigencies occurring, there would undoubtedly be a perceptible change in the initial cambering of the lower chord of the bridge and a redistribution of stresses in the truss differing from that originally calculated by the designers.

The yield point above the proportional limit, especially for the harder steels, was too poorly defined in these experiments to justify any attempts to determine it. There is no discountinuity present in the slope of the curve such as occurs in the case of the tension test piece. Howard in his published statements seems to confirm this whereas Prichard has stated that he has found such a yield point in very high carbon steels. The curve which is the locus of the ordinates representing the proportional limit from each column test is shown also in Fig. 60.[17] The average value of these proportional limits expressed with reference to the observed maximum stresses is 48.4 per cent, and with reference to the tensile yield points of the steels is 48.6 per cent.

It should be stated that the difficulties of obtaining the value of the proportional limit for a column are comparable to those of finding the proportional limit in a tensile test. The range from the mean can be expected to be at times relatively large in columns of built-up material and rather delicate recording instruments would be needed for rigorous determinations. A study of the causes which determine the height of the proportional limit on the curves will be further and more appropriately analyzed in connection with the subject of initial strain. It will be shown later that the deviation from the right line begins at the initial load. The point taken simply defines that stress where the change in rate of stress to strain becomes sensibly apparent.

(*c*) *General Remarks on the Maximum Loads.*—Considering the relations of the maximum loads to each other the data show (see Fig. 60) that the nickel and Mayari steel columns fail at from 2000 to 6000 pounds per square inch above their respective yield points, the silicon and chrome steel members about the same stress below, while the carbon steel specimens fail very close to the particular yield points of their respective steels. Considering the yield point as a datum of unity the nickel steel members have the highest ratio of strength to yield stress. The two columns

[17] These values of the proportional limits, as shown by the curve of Fig. 20, are conservative.

of Mayari steel have the greatest loads in absolute value. The yield point of the Mayari steel is slightly above that of the nickel steel, and in this respect the presence of the chromium and 1.6 per cent nickel in the Mayari steel makes a favorable showing in relation to the nickel steel.

In the columns of the Memphis Bridge, (see Fig. 60, columns "starred") already mentioned as failing at the ends, the space for eyebar entrance necessitated rather long unsupported forked ends. Although calculation would appear to indicate a reasonable "short column" resisting section at the pin plates, the matter was evidently one which could only be ascertained by the designers experimentally. The two members LC_0LC_1 failed in the direction of the major principal axis of the cross section, the radius of gyration being relatively larger than in the direction at right angles. (See Table 1.) It will be noted that the effective strength of the ribs acting as struts between lattice joints is less as determined experimentally than the resistance of the column as a unit would be expected to be from calculation, and this undoubtedly was the reason for the earlier failure.

A further differentiation may be made with regard to the columns of the Metropolis Bridge, Table 1. There is a preponderance of strength when maximum load is considered with reference to yield points for the columns with the flanges turned outward from column axes, over those where the flanges are turned inward. In the former case 11/16-inch web plates and rather slender lattice are used while 1-inch plates and a heavier lattice are used in the latter, the cross sectional areas and lengths being essentially the same. There are three times as many failures occurring above the mean yield point for the first set of columns as there are in the second. Some engineers are of the opinion that a column of small slenderness ratio may be designed very compactly without particular reference to a cross sectional form factor, such, for example, as the radius of gyration of the column section. The tests of the 10 columns for the Metropolis Bridge indicate in this respect that such a factor will enter to some extent, at least so far as this can be differentiated from the effect of yield point variations. In other words, there is apparently an effective distance for placing the material from the column axis when thickness of steel, length and width of columns are considered as major variables. This will be discussed further in connection with the analysis of lattice action:

(*d*) *Maximum Stress as a Function of Slenderness Ratio.*—It may be stated that for the range of slenderness ratios (l/r) existing for the columns of this series several of the best specifications have already taken what amounts to a uniform constant working stress for design. For example, the American Bridge Co. specifications of 1912 takes a safe stress of 13000 pounds per square inch up to values of $\frac{l}{r}=60$ without any correction for the influence of flexure. While the writers are of the opinion that this is very satisfactory practice in design, there is some lack of uniformity in the stress distributions of the columns denoting the presence of flexure. This will be briefly considered for the present in connection with the formula used in the design of the columns of the Metropolis Bridge.

It is stated in the specifications of the Chicago, Burlington & Quincy Railroad that compression stresses are to be reduced by the Rankine formula,[18] $f=p\left[1\pm a\frac{l^2}{r^2}\right]$. This is an empirical formula with a certain rational basis. The maximum stress f consists of two components. The component p arises from the uniform distribution of the maximum load over the column cross section at any point along the axis, as has been discussed. The component $\pm pa\frac{l^2}{r^2}$ is due to a stress couple occasioned by the load acting eccentrically to the column axis, as a result of initial curvatures, imperfect bearings, and other causes which will be discussed more fully later. The constant a is determined experimentally from statistical analyses of tests of columns having different slenderness ratios.

The maximum loads at failure are platted in Fig. 61 after the manner of Buchanan [19] as functions of the slenderness ratio for the 12 members which failed strictly as columns. The different values of the maximum loads correspond to the grades of steel used, the respective zones being shown at the right of figure. Arranging the results with reference to the grades of steel there is no influence discernible here, which can be clearly attributed to that of the reduction factor $p\,a\frac{l^2}{r^2}$. While there is a possible reduction as the $\frac{l}{r}$ increases in some cases, there is a rise in others, so that on the whole a constant working stress for columns similar to these may be considered satisfactory.

[18] Burr's Elasticity and Resistance of Materials, 6th ed., p. 467. [19] Engineering News, Dec. 26, 1907, p. 695.

(*e*) *Deflections Analyzed—Eccentricities.*—It is shown in the mechanics of columns as given by Euler [20] that there is a critical length at which flexure will begin. Below this point the member will be subject to pure compression stress when the load is concentric with the column axis. Flexure, for example, occurs in square-ended members at lengths $l \geqq 2\pi\sqrt{\frac{EI}{P}}$ where P is the load and EI is the so-called "flexural rigidity" of the piece, E being the Young's modulus and I the moment of inertia of the cross section. In the columns of the present investigation,[21] where an intentional

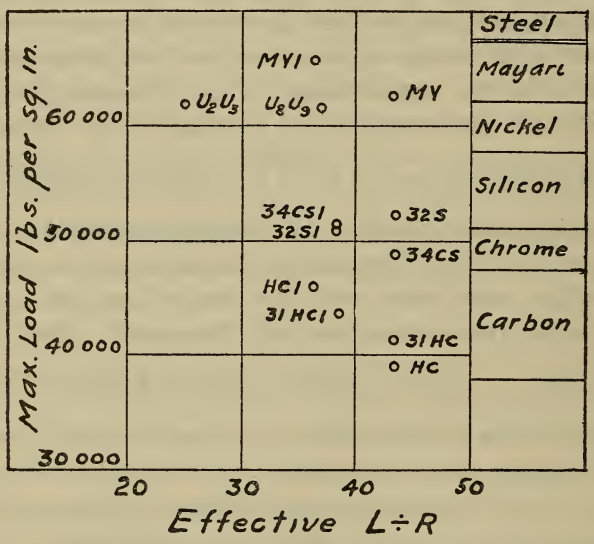


FIG. 61.—*Maximum load as function L÷R*

eccentricity does not exist, such deflections as occur must be due to essential eccentricities in one form or another. These arise as a result of unsymmetric variations in the elastic properties of the material about the centroidal axis, or through displacements of the resultant force from the axis for any reason. The types of eccentricity will be discussed later in the analysis of flexure.

Briefly reviewing the observations for deflection it will be stated that the first intimations of column failure, in the case of the members failing as units, usually occurred in some proximity to the maximum loads, and not exceeding 5000 to 8000 pounds per

[20] See Merriman's Mechanics of Materials for Euler's theory of long columns, 10th ed., p. 192.

[21] The columns US_2MS_1 (carbon and Mayari), and U_2M_1 had cover plates (see Table 1) which were partially "balanced" by flats. These columns, however, had local failures and are exempted in the above remark.

square inch below this load. Scaling was usually the first intimation of proximity to the maximum load. In no case were visible deflections noted below 6000 pounds per square inch from the maximum load. Usually the deflection was not sensible enough to measure, and in any case did not occur with that gradual variation from initial loads which is postulated in the Rankine type of formula. When the deflection was first noticed it increased noticeably in a short range of loadings, as is noted, for example, in column 32S (silicon) (see Table 9); i. e., at 46 000 pounds per square inch there was scaling, but no deflection; at 48 700 there was 3/16 inch deflection observed, with a maximum load at 52 800. For 31HC1 (carbon) (see Table 13) the corresponding values are 34 000 scaling, deflection 1/8 inch at 44 000, and failure at 46 550. For column 34CS1 (chrome) (see Table 15) on the first run to 25 000 no deflection was observable, but at 32 000 of the third run it was found to be but 0.005 inch, and similarly for column LC_0LC_1 (carbon). (See Table 20.) The larger deflections recorded for various columns, while interesting, will not be dwelt upon here as having no bearing on the present analysis of elastic behavior. In view of the "angular" type of failures for the columns U_2U_3 and U_8U_9, the deflections measured (see Table 5), while deserving of study in this connection, are mainly of interest in indicating the character of local phenomena.[22]

(*f*) *Remarks on Deflection Curves.*—An interesting feature is brought out in these tests in connection with the deflection deportment at or near failure. Referring to the 10 columns for the Metropolis Bridge (see Table 1), it can be easily verified that when flexure occurred about the pin there was a point of inflexion (or contraflexure) approximately one-third the length down from the upper square bearing, as is given in the conventional curves of engineering mechanics in connection with the analysis of columns. The deflection curve between this point and the pin takes the flat parabolic form. This may be noted as an example in the view of column 31HC, Fig. 46, while in the machine just after test. On the other hand, when failure occurs in the principal plane longitudinal to the pin, it may be seen that the conventional curve for two square ends was closely realized,

[22] Talbot and Moore remark in their analysis of built-up columns: "The amount of deflection from the original axis for loads up to a point somewhat below incipient failure was found to be slight (generally between 0.04 and 0.1 inch), much smaller than necessary to account for the stresses observed in the columns." Bull. No. 44, Ill. Exp. Sta., p. 51, 1910.

as may be verified by laying a ruler on the column HC1 of Fig. 50. The character of these deflection curves, considered with the fact that the columns of the Metropolis Bridge failed throughout in smooth curves, indicates that the integrity of action of fabricated steel can be realized in practical column construction to a larger extent than has heretofore been considered possible.

(*g*) *Column Moduli.*—In determining the column mean modulus as given in the upper rows of Fig. 60, the mean strain was found by averaging the four rod readings and dividing by the gauge length of 80 inches. The mean stress is the load divided by the cross-sectional area of the shaft. The ratio of the latter to the former for corresponding values gives a single determination of the modulus, if the values are taken well within the proportional limit. The separate determinations were adjusted by least square methods and the usual formula as follows: There were on an average five separate determinations found from the different stress and strain data within the proportional limit, these being as low as three and sometimes as high as seven, depending on the number of readings. The exact cross-sectional areas were used (see Table 1) for all columns, except U_2U_3 and U_8U_9 of nickel steel, where they were not available, so that the moduli of these columns are omitted in the upper row of Fig. 60. The values for column HC were too few and somewhat discordant, and the modulus for this member was also omitted in the results. The probable error was always taken as two-thirds of the mean error in the following determinations. The values obtained on this basis were:

	Pounds per square inch
Mean modulus of 15 columns	29 600 000
Mean error of modulus	±180 000
Probable error of modulus	±120 000
Mean error of a single observation	±680 000
Probable error of a single observation	±450 000

When the weighted mean was taken, each separate value of modulus being multiplied by the number of observations used in determining it and the total sum being divided by 76, the sum of all the observations, the modulus is again practically as before, 29 500 000 pounds per square inch. The uncorrected average of U_2U_3 and U_8U_9, which were not included in the results of the 15 above, also gives 29 500 000. This value is a fair mean of that for ordinary steel and checks well with other observations on columns

of a size at all commensurate with these.[23] This agreement is believed to show that the testing machine has not an appreciable error in giving loads as recorded, also that there is a good action of the members as a whole with respect to their axial compressions in the testing machines.

(*h*) *Ratio of Lateral Expansion to Longitudinal Compression.*—There are four elastic moduli for isotropic materials, two of which may be considered as independent. These two may be taken in the mechanics of the column as the Young's modulus already found and Poisson's ratio, the latter being the ratio of the lateral expansion of the material to its longitudinal compression. This ratio is of value in estimating the mean lateral strain of the column when under particular loads in lieu of actual measurements. In columns of plain steel there is no stress accompanying this extension, since the column surface is free from stress. Slight stresses doubtless exist, however, in columns of built-up steel. On account of the perturbations experienced from longitudinal diaphragms, column flanges, etc., absolute values of Poisson's ratio can not be found, but the mean more or less qualitative results are sufficient for practical purposes.

Probably the best value of this ratio for the series is obtained by taking the ($\pm$) elongation at stations 53 and 54 of the column U_8U_9 (nickel) (see Table 4), the elongation for 54 being divided by the compression under 53 for the various loads within the proportional limit and averaged. In this way the mean value for loads under 30 000 pounds per square inch is found to be 0.291.[24] In the case of eight other sets of readings taken on both alloy and carbon members the mean was found to be, after suitable averaging, 0.327. The value of 0.30 appears to be a fair value for calculation on the built-up co.umn, this being in close agreement with the values often given for ordinary steel, where riveting is not present.

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[23] For example, the modulus of column No. 1 of Steubenville Bridge tests, by J. E. Howard, the cross section being 90.7 square inches, the strain average at four corners being taken, was 29 375 000. On columns 2, 3, 5, 29 500 000 was selected by Howard as most representative for the curves taken from the data. (Trans. Am. Soc. C. E., Vol. LXXIII, pp. 435, 443–444.) Talbot and Moore, in their investigation of columns of smaller cross sections and less stiffness than these, remark: "The modulus of elasticity of 28 000 000 * * * checked closely with the total shortening of the columns." (U. of Ill. Exp. Sta., Bull. No. 44, p. 21.) For range of moduli for steel, see Burr's Elas. and Strength of Materials, 6th ed., pp. 274–275. Results range between 28.5 and 29.9 x 10^6 in compression. Also see curves of Buchanan in Eng. News, Dec. 26, 1907, p. 685 (measurements were not made as above), and Johnson's Materials of Construction, 4th ed., p. 510.

[24] In the tests on the Steubenville Bridge columns, by J. E. Howard, Trans. Am. Soc. C. E., 78, p. 429, high carbon steel, this value is given as 1:3.55 (0.282), and this result checked closely with the average of the measured value for the full width of columns. The value of 0.267 from Wertheim is often quoted for plain steel.

2. GENERAL LAWS OF FLEXURE

(*a*) *Primary and Secondary Stresses Discussed.*—It has been stated that flexure in the concentrically loaded column is theoretically impossible within the limits of length as given by Euler. Unsymmetrical stress distributions at the extreme filaments of the cross section of a column indicate the presence of a stress couple which varies along the axis. The line of action of the resultant force on the column deviates from its centroidal axis by a variable amount e, the arm of the couple and the eccentricity of the column. The couple Pe and the eccentricity e were neglected in the earlier discussion of maximum loads and yield points. The stress f at the extreme filaments under the assumed conditions of concentric loading was taken equal to p, the applied stress on the test member. It represented a first approximation to the actual stress distribution in the column. In the theory of structures p is called the primary stress.[25] It is more commonly known as the mean or direct stress.

A second approximation to the actual stress system is obtained when the stress resulting from the presence of the couple is added to the mean stress p. The stress difference $f-p$, denoting flexure, is known as the secondary stress.[25] It will be called the bending stress in the following discussion. The bending stress at a point in the column cross section varies directly with its distance from the centroidal axis, according to the law of the beam. It has its greatest and least values at the extreme filaments of any cross section at a point on the axis of the column where the stress couple is a maximum. The analytical expression for the bending stress is $f-p=\frac{Pec}{I}=\frac{pec}{r^2}$, where c represents the distance from the column axis to the extreme filament of the cross section, I the moment of inertia of the section, and r its radius of gyration. The average stress at the extreme filaments is found in terms of the observed compressions given in the tables by the relation $f=E\frac{g}{l}$, where g is the compression, l is the gauge length of the extensometer, and E is the modulus of elasticity. Similarly $p=E\frac{g_0}{l}$, where g_0 is the calculated compression on the axis determined by taking the average of the four compressions g at the extreme filaments.

[25] Johnson, Bryan, and Turneaure, Modern Framed Structures, 9th ed., 1916, Pt. II, p. 500.

Since the law of variation of $g-g_0$ and g_0 is known from the tests and c and r are constants for a particular column, it is important as the basis of further discussion to consider how the eccentricity varies as the load increases. The principle of superposition of small strains in mechanics permits component stresses, strains, couples, and eccentricities to be considered separately in this discussion. The component functions may be added algebraically in obtaining the resultant. Certain restrictions which are to be considered will be given later.

(*b*) *General Types of Eccentricity.*—There are two general types of eccentricity to be considered in the theory of columns. The first type includes all eccentricities which are inherent in the columns in its unstrained state. They may be present either as the result of initial curvature introduced in the column during fabrication or as the result of variations in the elastic properties of the different steel shapes. An actual eccentricity of this type also mav be introduced in conformity with the column design. While the inherent or initial eccentricity may vary along the column axis, it remains constant by definition at a particular point as the load on the column increases. Therefore, the portion of the bending stress $f-p$ due to this eccentricity is proportional to p alone.

The second type of eccentricity to be considered is that which is introduced in the straining of the column. The line of action of the resultant force is in general displaced from its original position at the initial load as different increments of load are applied. The effect is to modify the curve of initial eccentricity for the column. The increment ($\pm$) to the curve of initial eccentricity may occur in a number of ways: (1) When the terminal initial eccentricities are maintained constant, as in the case of a column with restrained ends, the centroidal axis is displaced laterally a variable amount $y=f(x)$. This is known to designers as the secondary flexure. (2) There may be rotations of the column at the ends due to imperfect end bearings, relative slipping of pin plates, or the overcoming of the pin friction. (3) The integrity of action of the different shapes is in general not conserved. The stress and strain functions or their derivatives are subject to finite discontinuities at certain periods of the loading as a result of the laminated character of the built-up steel, the slipping of pin plates, the irregular action of lattice bars, and transverse diaphragms, the effects of initial strain of fabrication.

It is the purpose to determine from the test data as far as is practicable the relative weight of influence of the different factors

which modify the curve of initial eccentricity. The theoretical law of variation of the maximum stresses in an ideal column of known initial eccentricity will first be treated as the norm or

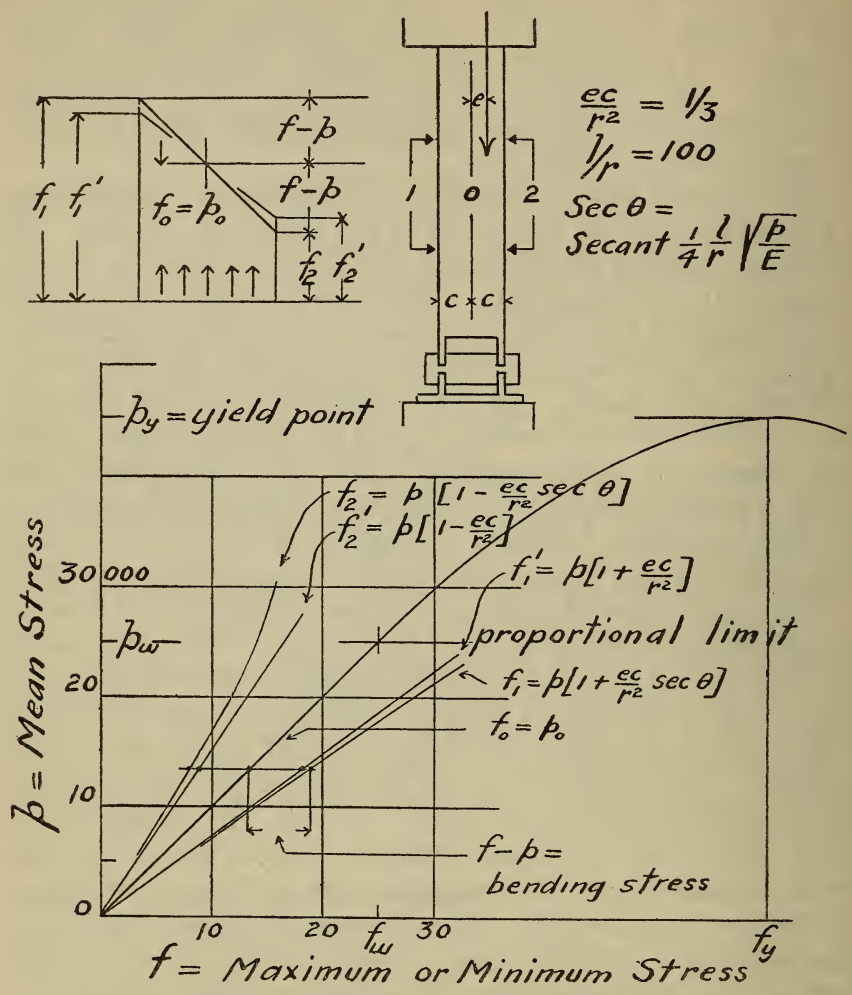


FIG. 62.—*Normal stress distribution in the ideal column with square ends by the laws* $f=p\left[1\pm\frac{ec}{r^2}\sec\frac{1}{4}\frac{l}{r}\sqrt{\frac{p}{E}}\right]$, $f=p\left[1\pm\frac{ec}{r^2}\right]$ *and* $f=p$.

standard of comparison. The departures of the test columns from this ideal case will be discussed later in the text.

(*c*) *Law of Stress Distribution for the Ideal Case.*—The column shown diagramatically in Fig. 62 is supposed to be perfectly elastic and continuous in structure within the interval of working

stress $o<f<f_w$, $o<p<p_w$. It is assumed to have a uniform square bearing at the upper end and a pin bearing at the lower end. Since no deflection was observed in the interval of working stress, the column will be considered for the purpose at hand to be perfectly restrained at the ends in this interval. The average slenderness ratio of the test columns is 30, the maximum value being 44. The value of $\frac{l}{r}=100$ has been used in drawing the following curves. The mean value of the initial eccentric ratio $\frac{ec}{r^2}$ is taken as 1/3, this being the approximate average initial ratio for the test columns. The applied stress p is given in increments of 10 000 pounds per square inch. The relation between the stress f at the outer filament at mid length and the applied stress p is given by the expression[26] $f=p\left[1\pm\frac{ec}{r^2}\ sec\ \frac{1}{4}\frac{l}{r}\sqrt{\frac{p}{E}}\right]$, this formula being correct within the limitations of the ordinary law for the beam. The term secant $\frac{1}{4}\frac{l}{r}\sqrt{\frac{p}{E}}$ is found from a table of radians. The formula $f=p[1\pm\frac{ec}{r^2}]$ may be used for small values of $\frac{l}{r}$ and has been platted in Fig. 62. When there is no eccentricity $f=p$, the law for the concentrically loaded column holds.

The method of platting to be described is simply an extension of the ordinary method of platting stress-strain curves. The applied stress p is taken as the ordinate, the calculated stress f is taken as the abscissa. The measured deformation g at the outer filament in the test member was previously shown to be proportional to f. The average g_o of the measured values g is the mean compression at the column axis. The curve $f_o=p_o$ is the ordinary stress-strain curve whose slope represents the modulus of the column when f_o is taken in appropriate units. This is drawn in the figure with a slope of 45°.

The bending stress $f-p$ in the case of this ideal column hypothecated for the purpose of the discussion is seen on the diagram of Fig. 62 to increase in almost direct proportion with the applied stress p within the proportional limit of the column which has also been taken to be the upper limit of the working stress f_w, p_w. For the purposes of a practical discussion the p, f curves may be assumed to diverge radially from the origin, minor curvatures

[26] Johnson, Bryan, and Turneaure, Modern Framed Structures, 9th ed., Pt. III, 1916, p. 38.

being neglected, under the premises defined. The working limit f_w, p_w in engineering practice is commonly given by a column formula which shows implicitly how f_w, p_w varies with the $\frac{l}{r}$ or other form factors as has been illustrated in the case of the Rankine formula. For a particular column, however, where the latter are held constant, the linear law $f \propto p$; $o < f < f_w$, $o < p < p_w$, may be assumed to hold in all cases, within the domains $o < \frac{l}{r} < 80$ or even 100, and $o < \frac{ec}{r^2} < \frac{1}{3}$. The deviation from linearity is somewhat greater in the case of pivoted columns but is never large in the columns of engineering practice.

(*d*) *Method of Calculation of Maximum Stresses.*—The law of the distribution of maximum stresses in the columns to a second order of approximation will now be discussed in the light of the preceding remarks. A portion of the observed compression data of column 32S (see Table 9) is first reproduced in the following Table 22 for the purpose of illustrating the method of calculation of the maximum stress from the data. The actual laws of variation of the stresses f and $f-p$ will then be shown graphically for a number of the test columns in order to direct attention to the typical stress phenomena which are found for the whole series of columns.

TABLE 22.—Illustrating the Calculation of Stresses f and $f-p$ at Station 1 of Column 32S

(a) Applied load, pounds per square inch	(b) Axial compression, mean of stations 1–4 g_o	(c) Axial stress $=(b)\times\frac{E}{80}$ p	(d) Compression at station 1 g	(e) Stress at station 1 $=(d)\times\frac{E}{80}$ f	(f) Bending stress at station 1 $=(e)-(c)$ $f-p$	(g) $(f-p)\propto p$ as in normal case of Fig. 62
1000	0	0	0	0	0	0
5000	0.0105	3890	0.0065	2400	1490	1500
17 000	.0422	15 600	.0380	14 100	1500	6000
21 000	.0526	19 400	.0480	17 800	1600	7500
25 000	.0644	23 800	.0592	21 800	2000	9000

The increments of applied loads under the heading (*a*) are given in multiples of 4000 pounds per square inch, beginning with 1000 pounds per square inch as the assumed point of zero strain.

The heading (*b*) gives the mean compression g_o on the axis at a particular load and is found by averaging the compressions given for stations 1 to 4 in Table 9. The corresponding axial strain is determined by dividing the mean compression under (*b*) by the gauge length of the extensometer. The axial stress under heading

(*c*) is determined by multiplying the mean strain of (*b*) by the mean modulus E, here taken as 29 600 000 pounds per square inch as was previously calculated. Similarly, the compression observed at station 1 for the outer filament is recorded under (*d*). When the quantity (*d*) is multiplied by 29 600 000 and divided by 80 the fiber stress f is determined as is given under (*e*). The difference between the values for (*e*) and (*c*) gives $f-p$, the bending stress as recorded under (*f*). The last heading (*g*) shows the ideal variation of the stress f at station 1, in accordance with the law of proportionality of stress holding for a prism or column of small slenderness ratio and constant eccentricity as has been given but which is not assumed to hold for the test columns.

It may be seen by reference to the above table that if the initial load is added to items (*c*) the results correspond closely with 1000, 5000, 17 000, etc., the loads applied on the testing machine as given under (*a*). The slight discrepancies between the two sets of values are due to the fact that small variations occur in the modulus when it is calculated from particular loads and compressions, intermediate values contributing to the mean being here omitted. The bending stress under (*f*), viz, $f-p$ for 25 000 pounds per square inch, should be approximately six times the amount found for 5000 pounds per square inch if the law of proportionality holds in column 32S. Further data will be submitted before discussing these discrepancies.

(*e*) *Variations of Stress Shown Graphically.*—A complete set of stress-strain curves is shown to an enlarged scale on Figs. 63 and 64 for columns MY, HC, 32S, 34CS, LC_0LC_1, US_2MS_1, and U_2M_1.

These columns are selected for the purpose of illustrating the characteristic types of curves which occur in the larger series. The observed mean stress p found from the testing machine is platted as the ordinate of each curve as was done in Fig. 62. The observed compressions g in 80 inches for the extreme filaments at the mid lengths of column are platted as the abscissas of the curves. (See data of Tables 6, 7, 9, 10, 16, 18, and 20.) These abscissas are proportional to the mean stresses f at the respective stations when the modulus of the column is considered a constant. The average curve is also platted as has been done elsewhere for all the columns, its abscissa being g_0. The distance $g-g_0$ along a particular abscissa between the average curve and the curve corresponding to a given station is proportional to the bending stress $f-p$ at the station. It will be seen that $g-g_0$ as a rule is constant

above a critical value of mean stress, $p_c = 5000$ pounds per square inch, as was already found analytically in the case of column 32S. In other words, the curves are generally equidistant above the ordinate p_c. This general parallelism of the curves is characteristic of the whole series of columns. There is, however, a diver-

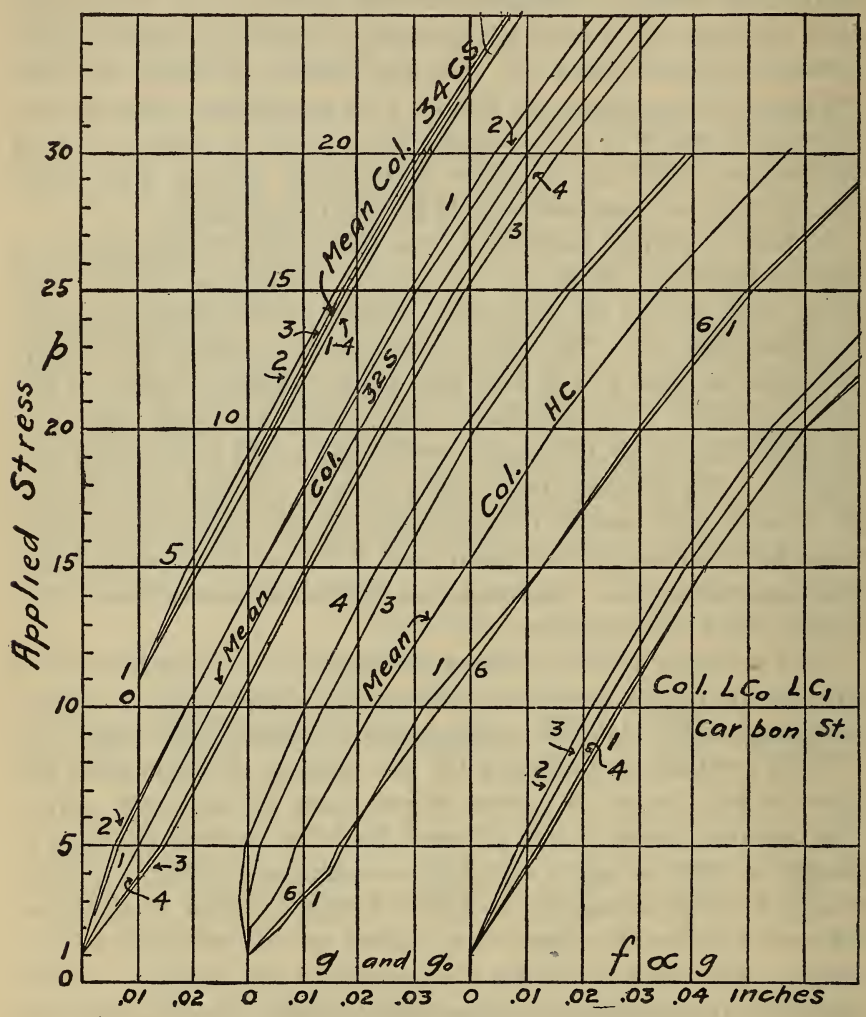


FIG. 63.—*Typical stress deformation curves*

gence instead of a parallelism of curves in the case of column US_2–MS_1 (carbon). Enough observations were taken in the case of columns HC and MY to show definitely that there is practically a radial divergence of the curves below the critical value p_c; in other words, neglecting minor departures and curvatures, the stress f is

a linear function of p for low loads. The law for the larger number of columns of the series may therefore be stated analytically to a reasonable degree of approximation:

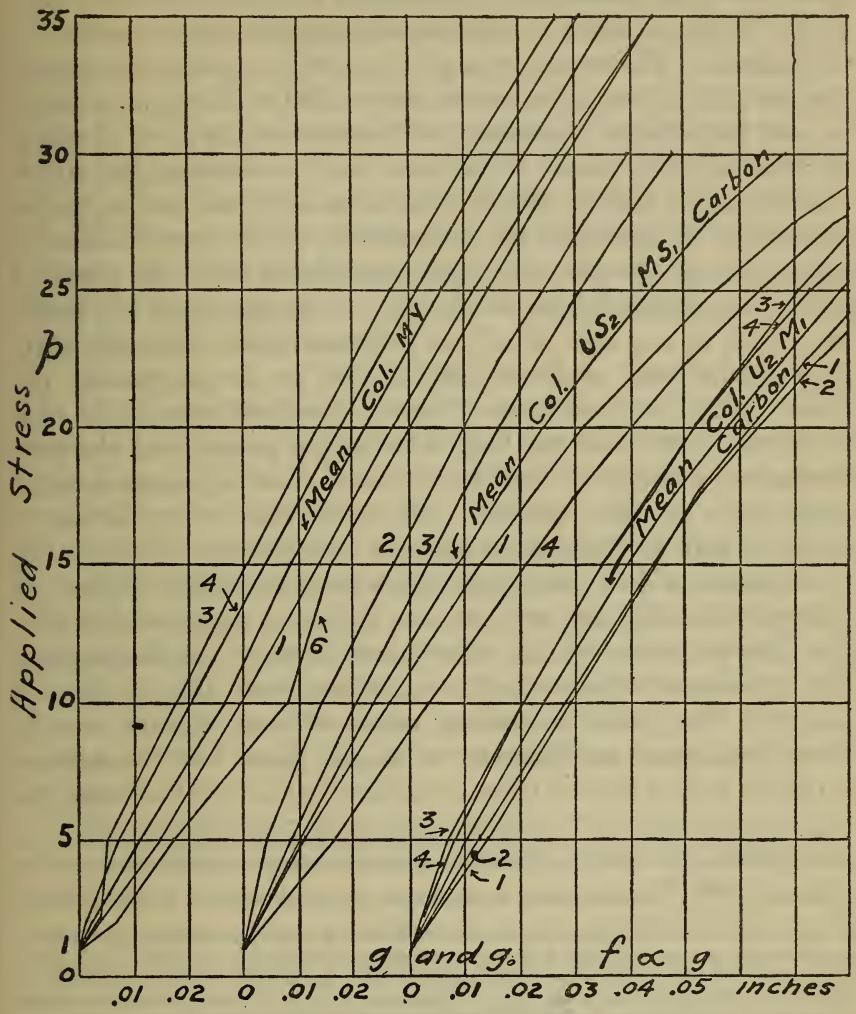


FIG. 64.—*Typical stress deformation curves*

$$f-p=(g-g_0)\frac{E}{80}=\pm p\frac{ec}{r^2} \text{ in the interval } o<f<f_c,\ o<p<p_c$$

$$f-p=\text{a constant in the interval } f_c<f<f_w,\ p_c<p<p_w.$$

In the above expressions f_c, p_c denote the stresses at the points of discontinuity in the slopes of the curves, and f_w, p_w denote the limits of safe working stresses. The parallelism of curves was

found in most cases to hold with little variation up to the stresses f_y, p_y at the mean yield point of the steel.

The nearly uniform divergence of the p, g curves below the stress $p_c = 5000$ pounds per square inch denotes that the eccentricity of the columns is maintained approximately constant in the interval. The mean stress p_c at which the rate of divergence changes and the curves become either parallel or diverge at a different rate indicates, in the opinion of the authors, the point at which the built-up steel ceases to act as a unit in resisting the stress. The slipping of the pin plates of the columns is believed to be the main factor in producing the discontinuity in the curves although slipping about the pin may have contributed when the plane of flexure was perpendicular to the pin. The slipping of pin plates was noted in the case of all the columns upon which pin-plate observations were made at loads as low as 10 000 pounds per square inch of shaft section. There is strong evidence in the case of column 31HC that the large interior pin plates next the continuous webs slipped as a whole at a load as low as 5000 pounds per square inch of shaft section. The observations were extended nearly as high as the quarter point of the columns in some cases. If wide areas of close riveting are inadequate to prevent slipping at relatively low working stresses, it is logical to presume that slipping may have occurred at other points between the shapes composing the shaft where the riveting is less dense than at the pin plates.[27] The relative movement observed is never large but the stress phenomena and behavior of the pin plates lead the authors to believe that it may at times be sufficient in amount to cause the

[27] The total shearing force along the plane of contact between two shapes at a particular point in the built-up column is given in the mechanics of materials (Merriman's Mechanics of Materials, 10 ed., p. 270) by the relation $F = \frac{\Delta M}{I} \int_y^{y_1} zydy$ where ΔM is the increment of the bending moment in the distance Δx along the column, I is the moment of inertia, z is the width of the shape or lamina, y is the distance to the surface in question and $y_1 = c$ is the distance to the extreme filament of the column cross section. If F has an upper limit, F_1, the frictional resistance of the contiguous shapes against relative sliding, the column will act as a unit so long as the shear at any point does not exceed the limit F_1. When this limit is exceeded in a particular region, sliding will occur along some contact plane at which the shear is F_1. The divided portions no longer act in unison. The sliding action becomes progressive as other contact surfaces experience the shear F_1. The slopes $\frac{dp}{df}$ of the stress strain curves become discontinuous. The general effects on the curves is illustrated conceptually in Fig. 65(f). The stress strain curves of columns MY and HC show evidences of this behavior in the interval of loading $o < p < p_c = 5000$ *pounds* per square inch. (See Figs. 63 and 64.) The action is probably here confined to the pin plates. It is the opinion of the authors that generally wherever the stress strain curves show abrupt changes in curvatures above the mean stress $p_c = 5000$ pounds per square inch this slight relative sliding may be taking place at the point in question. The behavior of the stress strain curves of column HC, Fig. 63, may be cited as an instance. The pronounced changed in curvature in column MY at 10 000 pounds per square inch probably denotes an error in reading. The "murmurs" which may be heard in the straining of the built-up columns, and which do not occur in tests of rolled sections, when they are not directly traceable to the behavior of lattice bars and transverse diaphragms, may be attributed to such actions as have been described.

stress and other functions to experience finite discontinuities along the surfaces of contact of the different shapes. Such surfaces of contact are treated in the general theory of elasticity as surfaces of limited stability.[28] It may be noticed that the average curves as given to an enlarged scale in Figs. 63 and 64 have somewhat larger curvatures than are commonly found for rolled sections tested as columns, the stress-strain curves of the latter being nearly straight lines within the intervals of working stress. This is a further indication in the opinion of the writers that there is not perfect integrity of action of the shapes. The extent to which the continuity of structure of built-up steel is conserved during straining is discussed further in connection with the subjects of initial strain and the behavior of lattice, diaphragms, pin plates, and other details.

(*f*) *Components of Eccentricity Discussed.*—In placing a column in the testing machine it was balanced under its own weight on the pin and bolster after these had been accurately centered on the lower platform. It may be assumed for all practical purposes that there was no play or lost motion at the pin, and that the axes of the member and the testing machine intersect at the pin. A nearly uniform contact at the upper head of the testing machine was secured by oscillating the lower or adjustable head until the planes of the upper face of column and the testing machine head were brought into parallelism. This was determined by suitable measurements at the outer corners of the upper face of the column. The initial load of 1000 pounds per square inch was then applied.

The components of the eccentricity e in a column in its initial state of strain before slipping of plates occurs are as follows (see Fig. 65): (1) There is a component e_1 from initial curvature induced during fabrication. (2) There is a component e_2 due to slight variations in Young's modulus of the different shapes. This is theoretically determined by the formula $e_2 = \sum \frac{p_i a_i d_i}{p_i a_i} = \sum \frac{E_i a_i d_i}{E_i a_i}$ where $p_i = p_1, p_2 \ldots$ represents the stresses in the respective shapes, $a_i = a_1, a_2 \ldots$ are their areas, $d_i = d_1, d_2 \ldots$ are their distances from the column axis and $E_i = E_1, E_2 \ldots$ are the corresponding moduli. An actual determination of this component requires that each modulus E_i be definitely known together with its orientation

[28] Talbot and Moore have called attention in their tests of riveted joints to the fact that the paint used in the assembly has some influence in determining the yield stress at which slipping occurs. (Ill. Eng. Exp. Station, Bull. 49, p. 47.)

with respect to the column axis. (3) There is a component e due to the fact that perfect parallelism of the planes of upper

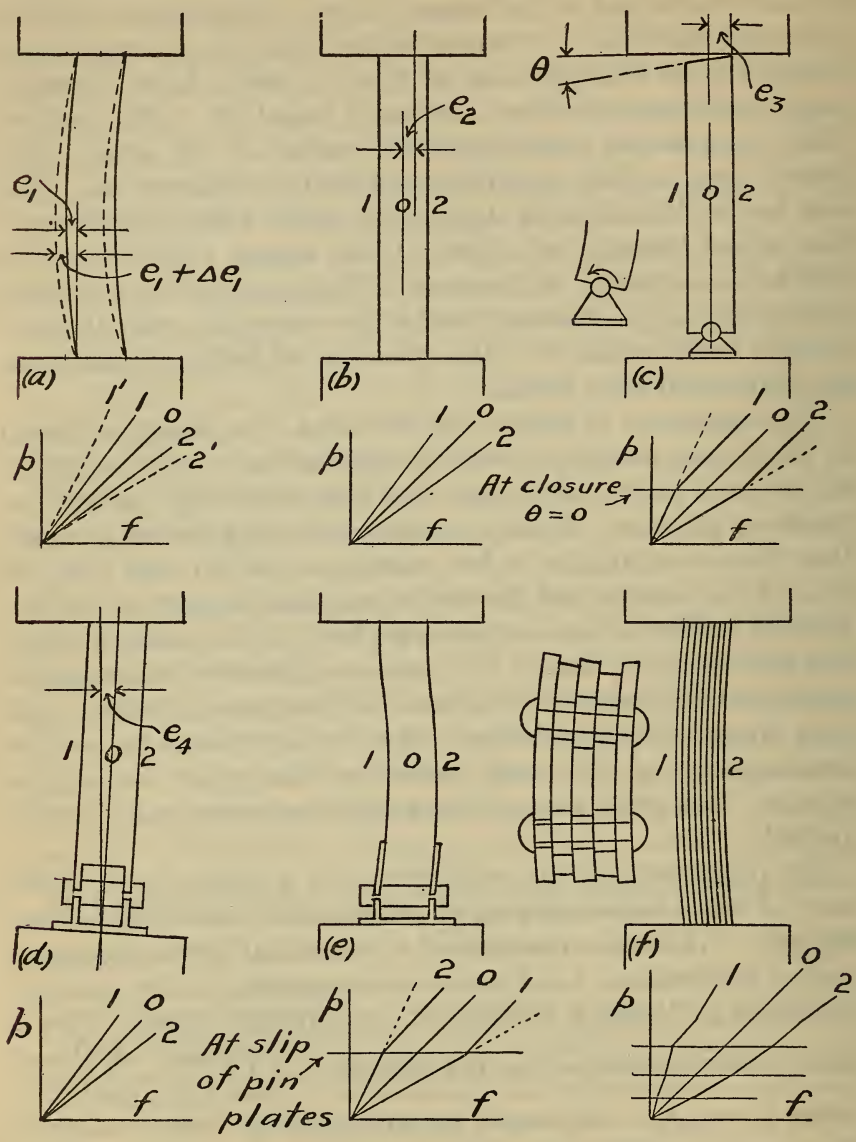


FIG. 65.—*Showing types of eccentricity present in the columns tested and the character of the corresponding stress strain (p, f) curves as a result of (a) initial curvature, (b) variations in component moduli, (c) imperfect bearing at upper head, rotation on pin, (d) angular deviation of axis of column from axis of testing machine, (e) slips from pin plates, (f) laminated structure*

face of column and head of the testing machine is unattainable. The planes will differ in general by a small angular amount θ.

There will be under these conditions either one, two, or three point contacts on the upper head, the three or higher point contact denoting parallelism of planes, in which case $\theta = o$, and e_3 vanishes. (4) There is a component e_4 due to the fact that even when parallelism is attained the axis of the column is not in general truly normal to the planes of upper and lower heads of the testing machine. It will intersect the planes of the lower and upper heads at a small angle with the axis of the testing machine.

The component e_1 from initial curvature may be neglected in comparison with the others. The component e_2 on account of the relative constancy of the modulus is doubtless initially small, but may increase if there is not perfect integrity of action of the different shapes in resisting the applied load. It is impossible to obtain e_2 from the data available to the experimenters. The components e_3 and e_4 can not be quantitatively differentiated with any degree of accuracy. It is only practicable to consider the total eccentricity e, and this may be determined for any particular column by the formula $e = \frac{f-p}{p}\frac{r^2}{c} = \frac{g-g_0}{g^0}\frac{r^2}{c}$ where g and g_0 represent the compressions at the outer filaments and centroid of the column cross section, respectively. The mean total eccentricities at the, mid lengths for the columns given in Figs. 63 and 64 are calculated by means of the above equation at various loads, and are shown in Table 23 below.

TABLE 23.—Mean Eccentricities in Inches at Mid Lengths, Calculated by the Formula $e = \frac{g-g_0}{g_0}\frac{r^2}{c}$ for Seven Columns at Different Loads p in Pounds per Square Inch

Loads in pounds per square inch	Column						
	34CS	32S	HC	LC_0-LC (carbon)	MY	US_2-MS_1 (carbon)	U_2M_1
5000	0.35	2.94	3.60	0.71	1.79	1.66	1.71
10 000	.18	1.38	1.90	.36	1.13	1.25	.94
20 000	.10	.60	1.10	.17	.50	1.00	.56
30 000	.09	.40	.74	.09	.35	1.24	.63
Values of $\frac{r^2}{c}$	4.03	7.34	4.03	4.28	4.02	4.82	5.67

According to the above equation, e is inversely proportional to p in the case of the columns where $f-p$ is constant. It is nearly constant above $p = 10\ 000$ pounds per square inch in the case of column US_2MS_1 (carbon), where there is some divergence of the p, g curves. In the case of column HC the eccentricities at loads

of 2000, 3000, 4000, and 5000 pounds per square inch are 4.93, 4.54, 3.86, and 3.60, respectively. If there were a true radial divergence of the curves in the interval $o < p < p_c = 5000$ pounds per square inch, the theoretical considerations previously discussed show that the eccentricity would be maintained constant in the interval. The gradual decrease shows that there is some slipping occurring in the interval below 5000 pounds per square inch. The authors are of the opinion that the initial eccentricity is principally introduced at the upper head, and that the inherent eccentricities in the columns are small in all cases.

A suggestion to the authors by H. S. Prichard is interesting in showing qualitatively how the discontinuity in slopes of the p, g curves may occur in an ideal case, although the calculated stresses f and $f-p$ are several times larger than the values determined from the observed data. Let a column be considered first to act as a rolled section without discontinuities in the functions. Assume the angle θ (Fig. 65(*c*)) between the planes of upper face and head is small, but of finite amount. Let the component of eccentricity as a result of point or line contact be e_3 as previously given. The analytical relation for the bending stress under a load P_1, which does not exceed the critical value $P_c = Ap_c$, is:

$$(1) \qquad f_1 - p_1 = \pm\frac{P_1 e_3 c}{I} = \pm\frac{p_1 e_3^2}{r^2}: \; o < f_1 < f_c, \; o < p_1 < p_c.$$

The column is subject to slight curvature under strain, so that θ eventually becomes zero, and the planes at the upper head have parallelism when the critical stress p_c is reached. The stress relation becomes (2) $f_c - p_c = \pm\frac{p_c e_3^2}{r^2}$ at this point. The eccentricity e_3 vanishes. Any further increment of stress (3) $f_2 = p_2$, is uniformly distributed over the column cross section, p_c still acting to maintain closure of the joint at the upper head. Adding (2) and (3) there results (4) $f_c + f_2$ = resultant maximum stress $f = p_c + p_2 \pm \frac{p_c e_3^2}{r^2}$ $= p \pm \frac{p_c e_3^2}{r^2}$. In other words, f is equal to the mean stress plus or minus a constant bending stress $\frac{\pm p_c e_3^2}{r^2}$ as is shown generally in Figs. 63 and 64 for the p, g curves above the mean stress p_c.

It is quite probable that a somewhat similar behavior, but in inverse order, is occurring in the actual test columns at the pin end

from an unequal and limited slippage of the pin plates. The column is subject to some slight initial eccentricity, as has been shown. The end is practically restrained to perpendicularity with the lower head up to the point of main slipping at p_c, when the end tangent to the strained axis deviates an amount θ from the original unstrained position as is indicated in Fig. 65 (e). The plates and perhaps other shapes slip relatively; f and p receive uniform increments of stress and the rate of slipping is approximately proportional to the mean stress, neglecting minor variations. The slipping appears to operate as an easement to the stress. For if the eccentricity remained constant throughout the test, the formula $f = p \pm \frac{pec}{r^2}$ shows that f is proportional to p and there would be a divergence instead of a parallelism of the p, g curves, and correspondingly higher stresses f and $f - p$ somewhat after the manner as is indicated for column US_2MS_1 (carbon), Fig. 64.

The divergence of the p, g curves in the case of column US_2MS_1 (carbon), in the opinion of the authors is explained by the fact that the column consists of two component struts joined by the battens and intermediate ties. (See Fig. 56.) One of these struts appears to have received a greater proportion of the load than the other in bringing the upper head to bear on the column. The load on the column is delivered eccentrically as a result, and the eccentricity remains fairly constant after a load of 10 000 pounds per square inch is reached.

In this discussion the point or line contact was taken for the purpose of analysis at the outer periphery of the upper face of the column. The amount of actual bending stress indicates that the initial eccentricity is not as large as will be found upon the above assumption. It is probably more reasonable to assume in the light of actual stress distributions that generally the upper surface has an imperceptible curvature or waviness and that the load is applied on a high spot with less eccentricity than the amount c, as appears to have been the case with all the columns. (See Table 23.) The analysis is similar to the preceding. The important distinction the authors wish to make is that the initial eccentricity at the pin is nearly zero from the manner of setting the columns in the testing machine. The unequal rates of slipping of the pin plates are caused by the stress couple, and are also probably influenced by different degrees of resistance of the riveted surfaces. The force on the columns, in the opinion of the

authors, is applied eccentrically not as a result of inherent or primary eccentricities in the columns, at least to any extent, but rather as a result of the initial eccentricity at the upper surface of contact, this depending on the degree to which a perfect bearing was obtained.

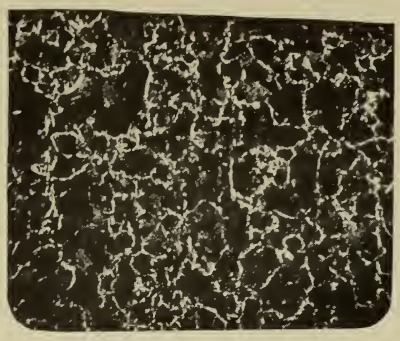
3. INITIAL STRAIN IN BUILT-UP COLUMNS

When stress-strain curves are platted from the data found on two ordinary test specimens of plain steel of appropriate lengths, one being tested in tension and the other in compression, it has been found that the slopes of the curves are in practical agreement at the same stresses up to the yield point of the material. After this point is reached the curve in tension takes the usual form up to the strength of the material. The maximum ordinate of the compression curve is, however, in any case scarcely greater than the yield point of the steel.[29] In the case of the built-up columns here considered it has been shown on the contrary by the tests that the proportional limit is only about half the yield stress of its component steel. This is in substantial agreement with the type of stress-strain curves usually found for large columns. Evidence will be given to support the theory that the low proportional limit is principally due to two causes. One is the presence of initial strains in the columns occurring from fabrication and chiefly from riveting; the other appears to be incidental to the physical constitution of the fabricated steel itself, as a result of bending strains induced by initial curvatures in the component shapes from the clamping action of riveters.

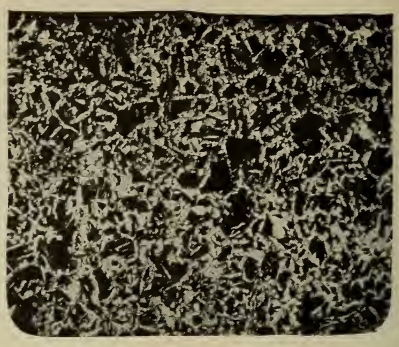
(*a*) *Initial Strains from Punching and Riveting.*—The important initial strains, apart from those introduced by straightening of shapes, are commonly considered to be caused by the process of punching and riveting and clamping actions of rivets. The experiments of Barba[30] on steel plates (French) had previously appeared to establish the fact that the effect of punching in deforming the material is confined to a region within a millimeter of the sheared edge, which in the ordinary process of subpunching 1/16 inch less and reaming to 1/16 inch more than the nominal diameter of rivet would undoubtedly remove. It has recently been remarked

[29] See, for example, numerous such curves platted by Johnson from reports of Watertown Arsenals; "Materials of Construction," 4th ed. (1901), p. 490 et seq.; also parallel values of elastic limits for tension and compression, Burr's Elas. and Resist. of Materials, 6th ed., p. 274; and Rickett's paper, Trans. Am. Soc. of Civ. Engrs., 1887.

[30] Burr gives results of Barba in "Elasticity and Resistance of Materials," 6th ed., p. 324.

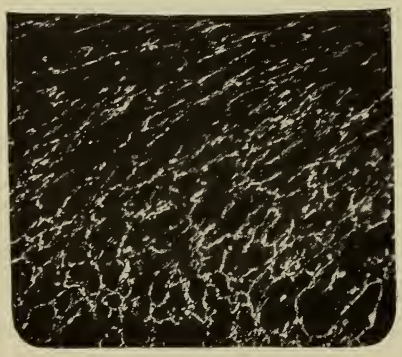


Mayari steel



Carbon steel

FIG. 66.—*Drilled sections of steels magnified 67 diameters. The upper edges are the parts directly adjacent to the drill. The microstructure shows no visible distortion present*

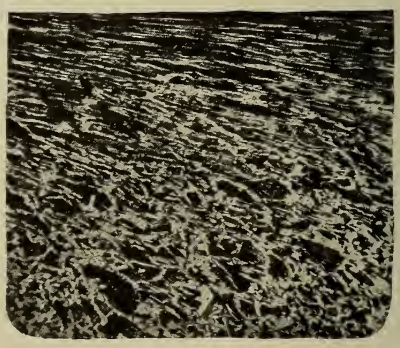


Mayari steel

Carbon steel

FIG. 67.—*Punched sections of steels magnified 67 diameters. The upper edges are the portions most severely "worked" from the shearing action of the punch*

Mayari steel



Carbon steel

FIG. 68.—*Riveted sections of steels magnified 67 diameters. The upper edges are adjacent to the rivets and show the combined effects of "working" from punch and rivet*

by J. E. Howard[31] that reaming commonly does not remove all the material that is disturbed by the operation of punching. It seemed advisable under the conditions of modern manufacture and workmanship to briefly examine this point again. Two plates 5 by ⅜ inch of high carbon and Mayari steel, respectively, were tested. A double row of punched, riveted, and drilled holes of ½ inch diameter were placed in series in the plates. There were two holes of each set spaced 2 inches apart, the rows being 2½ inches apart. The material was planed in a shaper until the midsection of one row of rivets and holes was exposed. Etchings were made by the metallurgical division of the Bureau of Standards on polished sections taken through the rivets and holes and magnified to 67 diameters. (See Figs. 66, 67, and 68.) The thicknesses of distorted regions as measured are given below. They may be assumed to apply practically to the other steels without material error.

Sample	Mechanical treatment	Radial thickness of distorted area
		Inch
Carbon	Riveted	0.031
	Punched	.024
	Drilled	Not enough to measure.
Mayari	Riveted	027
	Punched	028
	Drilled	Not enough to measure.

The evidence submitted warrants the inference that the usual practice of reaming ⅛ inch is quite ample to remove all sensibly distorted material,[32] and it is difficult to conceive of the existence of internal stresses in absence of a restraining force on the periphery of the holes.[33] The field of action is too close to the machined surface. There is no outer ring of stressed material to maintain the stresses in the absence of the rivet pressure.[34] Further evi-

[31] Trans. Am. Soc. of Civ. Engrs., 73, p. 445; Trans. A. S. C. E., 63, p. 101. Waddell.

[32] Burr's Elas. and Resist. of Materials, 6th ed., pp. 317–318.

[33] That is of the order of magnitude here considered and involving values which would have an effect on the elastic limit. There are undoubtedly elastic sets which tend to vanish in time.

[34] A rigorous distinction should be made as to what constitutes initial strain in steel, as very imperfect notions seem to be entertained commonly in this respect. A beam from the rolls after cooling will sometimes fracture its entire length of web with a loud report when a tool first enters same. Similarly, an ingot partly sawed through longitudinally will burst apart violently at the remaining portion and with a measurable deflection visible in the pieces. This strain is then somewhat comparable to that stored up in a Rupert drop and is obviously a proper initial strain in the pieces. On the other hand, the mere cold working of pieces, as in straightening shapes by straining beyond the yield point in "fiddles," rolls, and "gag presses" may or may not introduce initial strains proper, except as strains are entrained in the parts, through the manipulation, which are less than the yield point of the steel. What the designer obviously desires is that there shall be no resultant strain occur from the ensemble of stresses in the column and in

dence will now be adduced to sustain the opinion of Howard that the chief internal strain occurs from the rivet itself. The driving introduces a radial compression, theoretical considerations will show, which falls off inversely with the distance from the face of the rivet, together with conjugate circumferential compressions in the ring elements around the rivets. The clamping power of the riveters, which is large, in compressing the plates, the contractile force of the rivet in cooling, and some annealing action of cooling rivets undoubtedly complicate the situation. The rivets in a number of cases during the experiments at a point of loading well advanced parted with their heads with sharp reports, showing the tendency of the plates to expand outwardly with increase of crimping, the contractile force of rivets resisting this action. Considering the essentially radial character of the stress, whatever the resultant action of the composite effects may be, there are effective components according to Howard which in so far as a free stress exists and the material has not been permanently strained, unite with the direct stress from loading. The result is that the rate of increase of strain becomes variable at a considerably lower point than would be found in the independent results on a test piece. The action would be much the same in many respects as if at the beginning of the column test an initial load had been placed upon the test member unbeknown to the observer and the weighing beam had been then rebalanced at zero. In the course of the experiments the observer would get an unexpected lowering of results under the normal values due to the initial load. In the type of initial stresses under consideration, however, while the proportional limit is lowered, the columns still fail at the yield points as was shown. This is believed to be explained by the fact that the flow is not uniformly distributed through the column, but occurs in regions locally around each rivet. New regional equilibriums very probably occur as a result of compressive density changes, so that while the rate of

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proximity to the yield point which might cause the material to flow under working conditions. Analytically, the criterion to be applied in the case of a member free from surface stresses is that the differential equations for stress on an internal volume element, viz, $\frac{\partial X_x}{\partial x}+\frac{\partial X_y}{\partial y}+\frac{\partial X_z}{\partial z}=o$ and two others shall have $X_x=X_y=X_z=o$, etc, for a solution. (See Burr's Elas. and Resist. of Materials, pp. 877–880.) In general, however, it will be found that this is otherwise on account of the presence of initial stresses, which do not affect the static equilibrium of the body under external forces. The method of selecting arbitrary functions which may be so adjusted that the solutions obtained for X_x * * * shall be compatible with external boundary surfaces free from stress, has been suggested and illustrated by Love, Art. Elasticity Encyc. Brit., 11th ed., **9**, p. 160. For practical experimental methods suggested for determining such strains, see Marburg, Pro. Am. Soc. Testing Materials, **9**, p. 386.

strain is increasing above the proportional limit the column load still ultimately reaches the mean yield point of the component steel. There is a tendency shown, moreover, in test pieces to hold the load at constant stress with increasing strain after the yield point is reached. The tendency for the load to be kept up to a maximum also appears the more probable in riveted steel in that the contractile power in each rivet acts to cause constraints like a tie on the otherwise free surface of the column. This undoubtedly has some restraining action to maintain load at the yield point of the steel, even if the pressure from the rivet heads is not continuously distributed over the column face.

(*b*) *Tests to Show the Effect of Riveting on the Proportional Limit.*—Four tests were made on 10-inch I beams (40 pounds per foot) 4 feet 6 inches long. The specimens had small slenderness ratios so that they might fail near the yield points in compression without any appreciable reduction factor through column flexures, and no plates were riveted to the beams. For the two plain beams the maximum loads were 26 750 and 28 700 pounds per square inch, respectively, with a mean of 27 730, scaling occurring at 25 000. The yield point from separate tensile tests was found to be 31 410. Six sets of extensometer readings were taken at the mid lengths on the points of flanges and center line of web. No deviation occurred from the initial tangent of the stress-strain curves, until 21 000 to 22 000 pounds per square inch was reached. In the two other beams, two rows of ¾-inch rivets in punched and reamed holes were driven in the flanges and three rows in the web, and similar readings taken as before. The maximum stresses were found to be 29 450 and 30 000 pounds per square inch, respectively, with a mean of 29 730 pounds per square inch, the tensile yield point as independently determined being 32 800 pounds per square inch. The deviations from the right line of stress-strain curves in each case was 6000 to 7000 pounds per square inch lower than the corresponding values for the first case. The lowest values occurred for those readings which were directly over the line of rivets on the center line of the web of the 10-inch beam. Since no plates were riveted on these beams to cause a possible slipping, any crimping action other than that of the possible wrinkling of the beams themselves was eliminated.

In view of the above tests it is quite possible that the proportional limit of steel is lowered a certain amount, perhaps some 20 to 25 per cent, by the process of punching and riveting, and

the induced clamping strains in shapes. The matter is one for further investigations before drawing, however, definite conclusions. Some writers have considered that initial strains from riveting are in themselves quite sufficient to account for the variable sloping of the upper part of the stress-strain curves. It is believed, however, that there is another important type of strain to be considered, the second type previously mentioned, which will now be discussed as one of the probable causes of the flattening of the column curves after the proportional limit is passed.

(*c*) *Elastic and Inelastic Strains Due to Texture of Fabricated Steel.*—It has been shown in experiments by Basquin,[35] that "dimples" covering the region around rivets have been caused by the driving. The over-all thickness of two plates that had been riveted together was measured at numerous stations. The dimples were made evident by drawing on a map of the plates contour lines that showed the variation in thickness. These dimples may, in fact, often be seen in small columns of thin shapes. The force exerted by the riveting machine in driving the rivets above referred to was estimated as 40 tons. It was shown previously by Bach and Baumann [36] that this high power of clamping of riveters is not only needless, but it was stated to be actually injurious, in that the plates in the vicinity of the rivets are subject to stresses beyond the elastic limit. Various interesting cuts were given by them to illustrate the different degrees of disturbance experienced. The excess thickness in the case of Basquin's experiment varied between 0.005 and 0.010 inch as a rule.

It will be evident from the above considerations that each elementary portion of the steel between the rivets is subject to considerable initial curvature as a result of the clamping action of the riveter and the contractile force in the rivets. Each element carries its portion of the applied load on the column not alone as a straight bar, but as one which has considerable initial bending stress. This bending stress increases with the load. Since the applied stress on the element is superposed on the bending stress from curvature, the point of cold flow of the material must occur at a lower applied stress than occurs in the case of a similar ele-

[35] Jour. West. Soc. of Engrs., 18, p. 537; 1913.

[36] C. Bach and R. Baumann, "Zeit. des Vereines Deutscher Ingenisure," 56, p. 1890; 1912. See abstract, also, Jour. Am. Soc. M. E., February, 1913. They suggest lower clamping pressures in riveting.

ment which is free from rivets and initial curvature. Taking, for example, a strip of metal of dimensions 1 by ½ inch with a length between rivets of 4 inches and a mean eccentricity from curvature of 0.004 inch, the approximate formula $f = p\left(1 + \frac{ec}{r_2}\right)$ shows that the maximum stress of f in the strip is 31 500 pounds per square inch when the mean stress p is 30 000 pounds per square inch. The fact that the column as a whole develops the mean yield stress for the component shapes, even after the point of cold flow is reached may again be attributed to the ability of the material to hold its load at the yield stress, for some distance after cold flow has occurred.

There are three lengths of the elementary strip between rivets to be considered: (1) The length of l_1, before loads are applied; (2) the length of l_2 under direct compression alone; (3) the length l_3 of the chord after the strip is subject to both compression and curvature. The distortion $l_2 - l_3$ will contribute a small amount toward the reduction of the proportional limit. Calculations by the authors show that this distortion is quite insignificant in the case of the close-riveted columns until the seams begin to open appreciably at or near the maximum load on the member. This distortion in the opinion of the authors may be somewhat larger in open-work lattice columns, such as those of the Memphis Bridge.

An important point to be considered in the above connection is the fact that the average stress strain curves drawn from the compression data obtained with 8-inch gauges have in general somewhat flatter slopes than those obtained from the 80-inch gauges. The slopes are practically identical in the case of columns HC1, MY1, and U_2M_1 (Mayari). (See Figs. 30, 31, and 39.) There is a small angular difference in the case of columns HC and 31HC (see Figs. 24 and 26), and a more pronounced difference in the case of columns U_2M_1' (carbon) and LC_0LC_1 (carbon). (See Figs. 38 and 40.) A corresponding difference is also found in the case of calculations of the modulus. The mean modulus for the above columns when determined from the 80-inch readings is 29 700 000 pounds per square inch, this differing but 100 000 pounds per square inch from the mean modulus 29 600 000 pounds per square inch previously given for the entire series of columns. The mean modulus deduced from the corresponding 8-inch readings is found to be 27 600 000 pounds per square inch from the recorded compressions, and when this is divided by the constant

0.956 of the 8-inch Berry gauge, the mean corrected modulus is found to be 28 900 000 pounds per square inch, a difference in the moduli of 800 000 pounds per square inch. This difference is not large if it is considered that the open-work columns of the Memphis Bridge behave as trusses rather than rolled sections.

It is desirable that the built-up column should develop a strength in compression as close as practicable to that of a rolled section of the same dimensions. It is seen from the present tests that the working stress can be taken only about one-half that of the corresponding rolled section, if the proportional limit is considered to be the upper limit of working stress as is done by designers. The problem of determining why the proportional limits differ so widely in the two cases is one which can only be definitely determined by comparative tests of plain and riveted steels made under a variety of conditions.

4. EFFECT OF REPETITION OF LOADS

When the applied stress exceeds the proportional limit of the column there is a possibility of readjustment of stresses in the member such that a designer can not easily foresee. The effect of repeated application of load in the case of columns 34CS1, 32S1, and 31HC (see Tables 15, 14, 8) indicates that such strains as have been enumerated are, so to speak, squeezed out to a certain extent. The proportional limit is raised appreciably in column 34CS1. But equilibrium against such "lost motion" would appear to take place in time as the column receives successive repetitions of stress. The behavior of columns under repeated loads, indicated by the curves of Figs. 15, 14, and 8, is in accord with certain well-known phenomena among practical men, such, for example, that the curvature in the stress strain curve for cast iron can be considerably reduced by what is familiarly spoken of as "loading the material" before inspection. Moore [37] in deflection tests on I beams has shown that there is a gradual improvement in the elastic qualities of structural steel after various repetitions of the loading, the third cycle showing almost perfect elastic action, the effect of rest on the steel being eliminated by the smallness of the intervals taken. With the built-up steel, in view of its laminated character, a number of cycles of loading would be required to produce effects at all comparable with those mentioned for plain materials. Since rivets, lattice, pin plates, etc., ultimately take their final

[37] Univ. of Ill. Bull. 48, "Strength of I Beams in Flexure," p. 12.

positions and snuggest bearings, there is a fair amount of evidence of a probable ultimate practical conservation of the stress strain cycle in which the hysteresis phenomena described will nearly disappear. The inelastic action is perhaps most pronounced in the initial loadings from zero to 1000 pounds per square inch, for in the case when a stress strain curve is taken from zero loading a pronounced jog will often appear [38] in the curve at about 1000 pounds per square inch, after which the slope will be uniform. It will be seen from Fig. 34 that the slopes of the several curves are not appreciably changed within working loads, this indicating that the rate of stress to strain is not materially affected.

5. GENERAL ACTION OF LATTICE

Several kinds of lattice bracing have been presented for comparative examination in the various columns under consideration. The lattice used may be subdivided into two general classes. One consists of either a double or single set of diagonal bars in series superposed on the several column ribs which are either two or three in number in the columns of the investigation. The other employs, in addition to the double diagonals, transverse bars placed between successive panels of diagonals. The columns of the Municipal and Memphis Bridges have the first type, while the second is peculiar to the members for the Metropolis Bridge.

In determining the laws of action of these general types of lattice, it will first be broadly stated that the ribs of a column, considered apart from the lattice bracing, are very flexible pieces, as may be seen when they are lifted by a crane during fabrication. This point will be used in the interpretation of results. The ribs are moreover the main carriers of the load, offering the most direct paths for the stress and having the greatest rigidities (defined as the reciprocal of the strain). Very little strain energy of the column is absorbed by the lattice in columns of the small slenderness ratios under consideration here, this being greatest at the ends of columns close to the large batten plates on account of the large restraints imposed at these points.

A differentiation must be made as to the manner of connection of the bars to the ribs. The lattice of either type is mechanically intermediate to two abstract cases which are presented in analysis: One is that case (*a*) where the joint connection may be considered frictionless without any restraint to rotation of bars about the

[38] See Buchanan's curves, Engineering News, p. 685, Dec. 26, 1907.

joints. This is best realized practically by a single loose rivet or bolt connection of bars to ribs. The other is the case where (*b*) the angles between the ends of lattice and the ribs are completely conserved during strain, by rigid connections of the bars to the ribs, such as is approximated practically by a two or three rivet connection. The bars in the latter case when the column is loaded will be bent by variable stress couples acting throughout their length, which are brought into play, in addition to some direct strains in the bars, by reason of the fact that the triangular and quadrilateral elements or cells of the linkage formed by the lattice network are warped slightly from their initial shapes by the straining. The lattice bars can only accommodate themselves to the new configurations by being bent into flat curves, which in general have a point of inflection. In any actual lattice system which is intermediate to the limiting cases (*a*) and (*b*) the direct strains in the bars are found first upon the assumption (*a*) because a slight change in the configuration of the network formed by the bars and ribs does not appreciably affect the values of the direct strains in these members. The flexural stresses are then analyzed by the method of secondary stresses upon the assumption (*b*) of rigid connections at the apices. The resultant action is gotten by the superposition of the two strain systems, according to the usual practice in structual design.

(*a*) *Behavior of Ordinary Lattice Analyzed.*—Considering columns U_2U_3 and U_8U_9 the mechanical action under type (*a*) is that which will occur in a pantograph linkage formed by the bars, when it is superposed upon the three-column ribs. As the ribs contract under load from the testing machine the pantograph of bars merely draws together in the direction of the column axis so as to cause a slight bowing out of the outer ribs, the inner rib remaining quite neutral as to flexure and taking only direct stress. The theoretical effect upon the individual bars in the body of the columns and apart from the regions of the end battens, is to cause them to largely experience, neglecting minor considerations, that motion of translation and rotation which is defined mechanically for the rigid body, viz, one theoretically without strain. The configuration of the bars before and after strain is shown in the case of column U_8U_9 in Fig. 69 (*a*). The effect of the several transverse diaphragms causes constrictions to this action to some extent, as is indicated. The almost purely kinematic action that is occurring may be most easily illustrated in bracing

of this type by contructing a simple model to depict the behavior. Three rubber bands may be placed under equal initial tensions to simulate the unstressed ribs, and lattice of deal sticks may then

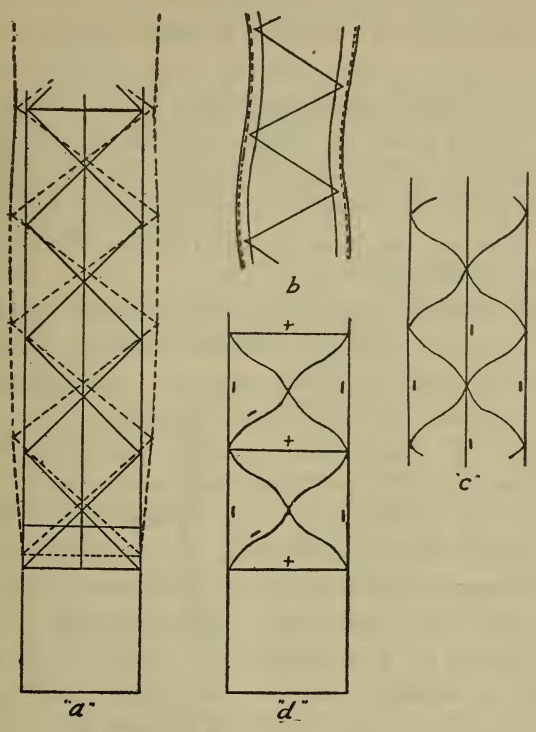

FIG. 69.—*Behavior of lattice*

be fastened upon them by thumb tacks or other means in the manner of the braces of $U_2U_3U_8U_9$. A somewhat elastic cross-piece may be placed at certain intervals for the diaphram and wider stiff pieces at the ends for the battens.[39] As the system is allowed

[39] While the action here depicted appears to be well known to practical engineers engaged in the work of column fabrication, and has been already variously termed "the accordeon effect," "hat rack action," etc., it seems to be ignored by column authorities who seek to explain the elastic behavior of column lattice by methods which assume a flexural action in the column units as a whole. The above considerations however, are believed to indicate the essential and controlling factors for the "stocky" type of columns here considered, and, in fact, should be taken into account to a large extent in columns of slenderness ratio up to 120 l/r. In the opinion of the writers this behavior is shown very clearly and in an exaggerated manner in column T_5B (long) tested by the Phoenix machine for the Quebec Bridge Commission (Engineering Record, Nov. 19, 1910, p. 568). The "bowing out" of the "ribs" as the lattice X's draw together occurs just as is found in the small model, and as is illustrated for U_8U_9 in Fig. 69 (*a*). There is a very complete constriction against this "bowing" of the ribs at the transverse diaphragm, which is clearly depicted in the sketch of column T_5B. While there is undoubtedly a small flexural action present in general, in addition to the above action, its influence is thought to be insignificant in well-designed columns under 60 l/r. Inasmuch as no theoretical flexure occurs under the critical length as determined by Euler, such actual flexure as is present must be attributed to the tendency of the bars to form unsymmetrical configurations with respect to column axes, on account of actual inequalities existing in the elastic properties of the ribs and lattice bars varying degrees of snugness in rivet connection to bars of ribs, etc., and some eccentricity of loading.

to contract to simulate the direct strain of the load from the testing machine, the translation and rotations of the bars will be seen to occur as described; there will be the constrictions at the "diaphrams" and the discontinuities at the ends due to the constraints of the battens.[40] A more perfect model for the magnification of any type of strain desired, and for any network, may be gotten by the use of telescopic types of flat strips, where any configuration, apart from the original unstrained status is obtained by taking arbitrary displacements in the elements, either plus or minus, to suit the class of strain, and reclamping for the final configuration. This can be done for the bars as well as the ribs, to best suit the conditions.[41]

A fair experimental verification of the above actions in the columns U_2U_3 and U_8U_9 will be obtained through an examination of ± elongations given for these columns. (See Tables 3 and 4.) The strains for the lattice of U_8U_9 under station 33 and 34 are found by dividing the (±) elongations by eight, the gauge length. For the entire range of loading to failure they never reach 0.0001. The stresses found by multiplying the strains by the modulus = 30 000 000 pounds per square inch vary throughout the range of loading between 0 and 1200 pounds per square inch. Taking the bars of U_2U_3 and U_8U_9 nearest the diaphragms they range somewhat higher from 0 to 2500 pounds per square inch near failure, and generally for working loads in the bridge never greater than 1200–1500 pounds per square inch, as is expected. On the other hand, the effects in bars of panel next the battens are much more pronounced, being for working loads from 0 to 9–12 000 pounds per square inch for U_2U_3 and with a tendency to overstraining at higher loads. In view of the fact that the stresses practically are not always negative—i. e., compressions—the strain system evidently is not perfectly symmetrical about the axis, as in the theoretical case. This is due to more or less actual assymmetry with respect to elastic properties of the bars, readjustments and play of holes in lattice, and perhaps some slight flexural action in the columns which can not be strictly taken into account practically. With longer portions of ribs between the cross diaphragms, or with these diaphragms omitted, the pantograph action would more perfectly

[40] In the Steubenville Bridge tests (Pennsylvania Lines) it was conclusively shown that where single-riveted lattice is used there is no strain in the bars at a distance from diaphragms, etc. In the present investigation attention was given to the strains in bars close to the transverse diaphragms and end battens where certain strains are maxima. (See Fig. 2, Pl. XXXVIII, Trans. Am. Soc. C. E., Vol. LXXIII, 1911, p. 443.)

[41] Mich. Technic, February, 1909, p. 34.

approach that described and the stresses of the bars be correspon-d ingly less in the body of the columns. Reference may be made to the tests by Howard on the Steubenville Bridge columns in this connection for a closer verification. Talbot and Moore state in their paper "the amount of deformation observed in lattice bars is relatively small."

It may be remarked that in certain inward bucklings of bars, the phenomena are those of failure and not elastic action. They only appeared toward the end of the tests and were first discernible as shadow effects on the lattice, denoting the immanence of failure. (See photos after failure.)

(*b*) *Action of Lattice in Deforming the Column Cross Section.*—It will be seen also in studying the action of the lattice bars in members of the type of $U_2U_3U_8U_9$ either with three or two ribs, that as the separate X's of the individual panels of lattice flatten out they exert collectively well distributed pressure outward on the rib flanges, causing bending in the webs of ribs about the longitudinal diaphragm as a fulcrum. This is not only evidenced by the more frequent tendency to compression stresses in the lattice but by the extensions on the cross diaphragms as given by the elongations under stations 41–42 of U_8U_9 and 17–18 of U_2U_3. (Tables 3 and 4.)[42] It is difficult to attribute these elongations to other causes. They are rather large, considering the otherwise relative rigidity of the columns. Taking the restraint of the continuous longitudinal diaphragm into account with the outward pressure of the lattice taken as a whole, the cross section of the members of this type then tend to be bent into the form shown in Fig. 70. This bending will be least at the cross diaphragms, of course, and greatest between them where the flattening out of the X's is least restrained.

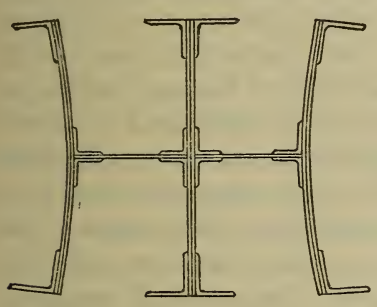

Fig. 70

In view of the tendency of large strains in the battens due to the "spreading" effect of the lattice, the angle at the edge of the battens seems a wise provision of the designer in preventing overstraining. This point will be taken up later in other columns.

[42] Column U_2U_3, on account of the closer spacing of diaphragms, is subject to more local perturbations than U_8U_9, which have a modifying effect on the action of lattice bars, which has been described. But in general the law of action is as described.

The ribs just at the batten experience rather severe bending in most of these types of lattice.

(*c*) *Flexural Stresses in Ordinary Lattice.*—In the analysis of the purely kinematic action in the columns of nickel steel, U_2U_3 and U_8U_9, the configuration of the strained system of lattice and ribs is found by connecting the displaced positions of the joints by straight lines. The flexural actions now to be described exert no appreciable influence in modifying this configuration for direct strains, except that the neutral axes of the bent bars will in general deviate from the straight lines connecting joints, the deviations being zero at the ends of lattice bars and at the point of inflection of the neutral axis. This is owing to the fact that in the case where there is a flexural system due to the moments of restraints caused by the riveted joint connections at the apices the changes in the lengths of the ribs and lattice are still those of the direct strains in the pieces. The length increments ($\pm$) due to the slight curvatures experienced in the flexure of the bars or ribs per se are negligible, since for the flat curves considered in elastic analyses the arc does not sensibly differ from the chord. In other words, the decrement due to the curving is relatively small compared with the compression or extension from direct stress. The flexural stresses will be most apparent where there are wide bars well riveted at the ends or intersections. In general, if extensometer readings are taken at the outer edges of the latter, the differences between the measurements show that flexure is taking place. By reference to the elongation data on the wide bars of column MY (Mayari) (see Table 7) and column US_2MS_1 (carbon) (see Table 16), it will be seen that these flexural effects are often pronounced even for the common range of working loads. On account of the relative width of the bars, however, no refinement of analysis is possible, as the beam principle, of which Manderla's method is a generalization, is only applicable strictly to more slender lattice. The average of the strains determined give the direct strains on the axis. The difference between this axial strain and that at the outer fibers of lattice bar is the mean bending strain in the piece. In view of these remarks it will be seen that as the column ribs contract under load, these wide bars must take considerable flexure as shown by the data to accommodate themselves to the new configuration imposed by straining. As the rivets are sufficiently far apart at the joints to cause strong moments of resistance at the ends of lattice, a series of these moments act on the column

ribs throughout their length. The question here raised is, of course, largely one of efficiencies in proportioning the material and not particularly one of the strength of the column as a whole.

The general type of flexural strain occurring in U_2U_3 and U_8U_9 is shown in Fig. 69 at (*c*). Taking into account the method of secondary stresses, this follows directly from the following considerations, but it was also confirmed by trial with a model of the kind already described. The direct strains in the ribs cause the angles at the joints formed by the axes of the bars which face longitudinally to the column to take small angular strain decrements, whereas the angle between the bars at the joints is maintained constant by the rigidity of the connection. The bars then are bent into the flat reversed curves indicated. By reason of the essential symmetries of the strain system in the mid panel between diaphragms, the point of inflection occurs approximately at the mid point of the bars. The constriction of the transverse diaphragm simply modifies this flexure in degree, and causes a displacement of the inflection point along the axis, the action evidently being most severe at the end battens where considerable flexural stresses must occur. There is every indication that the bending in the rib near the batten is very abrupt.

The outer ribs of these columns are in general flexed in the wavy type of curve usually shown for the continuous beam. This follows from the fact that the end stress couples of the pairs of bars proceeding from the joints along the ribs can not be equal, except in the ideal case of a column having long ribs without constraining diaphragms where a perfect symmetry of the different strain configurations about the column axis may be assumed to exist. Readjustments in the lattice and differences in the elastic properties of the material will also have an influence in causing the ribs to be bent into sinuous curves.

(*d*) *Action of Lattice in Modifying the Continuity of Strain Distribution.*—The essential point to be considered and brought out in a study of the latticing of the columns of this investigation is not the extent of the strain so much, as this is small, as its character and influence with respect to the strain deportment of the column as a whole. With perhaps one or two exceptions, to be discussed, there would seem to be no special action in the lattice here present, either with respect to its strength or the particular type used, to influence very appreciably the strength of the column considered as units. It is thought, however, that it can be shown there are

actions introduced by certain types of lattice which will make it quite doubtful that the principle of mechanical continuity which the designer assumes and desires shall always hold throughout the range of loading. It is this point, it is believed, that should chiefly be considered in a study of the deportment of lattice systems. Those who have aimed to formulate a mechanics of the columns have of necessity based it upon elastic principles, which imply that the system of strain shall be continuous and derivatives of the first and second orders shall exist during the range of working loads. In other words, in designing the stress couple along the column $\left(\frac{d^2y}{dx^2} EI\right)$ is taken proportional to the curvature of the strained centroidal axis as in a beam $\left[M \propto \left(\frac{1}{r} = \text{curvature}\right)\right]$. A superficial examination of the results of many investigations is sufficient to show that the crux of the situation is largely to be found in the effects of the lattice. In the present investigation the variability of the action could often be noticed in the rate of taking load, the sounds and creakings accompanying readjustments, etc. It is not that difficulties of this description can be overcome entirely, but perhaps in the light of investigations they may be more perfectly understood and the conditions ameliorated by taking due account of them in design, as will be shown later.

(*e*) *Assymetrical Action of Single Lattice.*—In single latticing there is obviously assymetry of configuration of the bars and the strain system about the axis of columns. The elastic lines of the outer ribs are unsymmetrical in general, as shown in Fig. 69 at (*b*). This is equivalent to an essential eccentricity of the loading. In the struts of column US_2MS_1 (carbon), the designer has counteracted this assymetry by an ingenious arrangement of balancing one component strut of the column against the other. The unsymmetrical action of single lattice was further shown apart from theoretical considerations by a model. The main distortion occurred at the ends of ribs next to battens. Although this "balancing" of the lattice was arranged in columns LC_0LC_1 of both steels, it is suggested that the unsymmetrical character of the lattice has very probably contributed to the failure as presented, in spite of the effective breadth of the member in the plane of distortion and the care exercised in the design.

(*f*) *General Character of Failures in Columns with Ordinary Lattice.*—It will be noticed that all of the columns using the ordi-

nary type of lattice (omitting those which bent at the pin plates) failed with what may be called angular fractures. These were of the type somewhat comparable to those presented in a short prism and ordinarily spoken of as shearing failures. Or they failed locally apparently from the effects of the lattice, in acting unsymmetrically with respect to the column axis. The theoretical considerations based on Manderla's method show that the stress couples at the joints will be maxima or minima (or more properly "extrema"). In view of the reduction of area in the ribs throughout rivet holes, the action of internal strains, and the possible occurrence of fortuitous combinations of untoward stress couples, the rib at point of connection with lattice is usually the weak point in most columns of small slenderness ratios. The failure in U_8U_9 at the joint (see Fig. 42) may be noted. That this is not as well borne out in this investigation as in some others is attributable to the uniform excellence of the designs and fabrication of the members throughout.

(*g*) *Behavior of Lattice with Transverse Bars.*—In the ordinary lattice already discussed it was seen that in considering the system of lattice as a linkage superposed upon the ribs, relatively speaking, considerable freedom of motion is permitted as the ribs contracted under stress. In the case of the Metropolis Bridge columns the linkage action is almost completely restrained by the transverse bars. Whatever motion occurs must either be due to the elastic action of the members themselves or to some inelastic play at the rivet connections as the load progresses.

The main function of the transverse bars, the tests show, is to largely prevent the tendency of the outward bowing of the ribs as the column compresses and the X panels tend to draw together. The elongations for the transverse bars as given for columns MY, MY1, 32S, and 34CS1 (see Tables 7, 9, 12, and 15), taken in general, show strains averaging from 0.00005 to 0.00010 for column working loads of 10 000 to 20 000 pounds per square inch. With a modulus of 30 000 000 this implies stresses of some 1500 to 3000 pounds per square inch. In examining the action of the lattice bars, it should be noticed that where the measurements include spaces having riveted connections to the ribs that the strains are indeterminate analytically, but nevertheless show experimentally the real action that is taking place. Some of the results must accordingly appear somewhat erratic on account of the perturbations indicated in the bars by the rib portions in contact with them.

In a study of the stresses of the diagonal bars it is logical upon first examination to expect these also to resist outward bowing and be in tension, as are the transverse bars. The controlling influence, however, is again the main axial compression in the ribs, and accordingly (from Kirchhoff's law for subdivided currents which also applies to stress) these bars are in compression, taking their portion of the axial load from the testing machine with the ribs, since they can not move in the manner of the previous type of diagonal bars. An examination of the data of column 32S shows that the strains are of the same relative magnitude as in the transverse pieces. The components of the diagonal strains do not balance necessarily those of the transverse bars, for a portion of the stress is taken by the diaphragms and end battens. The system is statically indeterminate.

There is evidence of large local strains at the batten edges, and perhaps the same reinforcing angle here as was used in the case of U_2U_3 and U_8U_9 of the other type would be efficient. The transverse diaphragms are not as essential in the present case as in the other columns, as the transverse bars undoubtedly prevent any considerable outward bending of the rib flanges about the continuous longitudinal diaphragm as a fulcrum. The chief function of the transverse diaphragm is to reduce twisting and warping of the shaft. The abrupt bending of ribs at the battens is largely eliminated by the transverse braces.

It has been suggested to the writers that the lattice of these columns of the Metropolis Bridge designs should be larger—more comparable in size to the transverse bars. It is the opinion, however, that in view of the fact that the loads of long span bridge members are intended to be axial, scarcely any more metal is needed in these diagonals than to take up such transverse loads as occur from gravity, and some actual column flexure which may occur as a result of the initial eccentricity. This could be determined on the basis of an arbitrary shear taken as a small per cent of the axial load.[43] It is considered more logical to have the lattice flexible enough to bend and adjust themselves after the range of working loads has been well passed. Otherwise the diagonals will store up considerable strain energy which more properly should be absorbed by the ribs, the main carriers of the stress from loadings. In view of the preponderance of the strength of the columns of this series having the rib flanges turned out,

[43] This has already been suggested by a number of engineers.

with lighter lattice, over those where they are turned in and heavier lattice are used, there is nothing to indicate that the more slender diagonal is inefficient in any manner whatsoever. It was apparently the intent of the designer to obtain data covering this point.

In this type of lattice the bending stress is of the same character as before, the transverse bars in the main remaining neutral as to flexure, except as slight assymetries in the strain configurations exist and moments at joints occur to modify this. (See (*d*), Fig. 69.) With the wider bars, however, as has already been shown the flexure will be rather large, which is believed to be another point in favor of the more slender diagonals.

(*h*) *Comparison of the Two Types of Lattice.*—As far as the relative efficiencies of the two types of lattice are concerned, it is perhaps too early to pass upon the comparative merits of the systems presented, as this will fall more properly to the province of designing engineers. Moreover, a larger number of tests would appear desirable before very complete judgments can be formed as to the mechanical principles which control. The fact has already been mentioned, however, earlier in this paper that the 10 columns of this series failed in smooth curves with inflections corresponding to those commonly given in the mechanics of materials for a pin and square or two square ends according as the deflection was perpendicular to or in the plane of the pin. If this uniformity of action is properly traceable to the influence of the transverse bar type of lattice, and it is believed it is, it would be an excellent feature to introduce these additional bars between panels into column design with a view to conserving the cross sectional form of the column and insuring a more uniform action of the lattice throughout the entire range of loading than it has been possible heretofore to obtain, judging from previous tests. If the deportment of the column as a unit can be controlled in a manner which is in closer conformity with those theoretical postulates which have been laid down by experts in the past, but which have been so often found to be violated as shown by the practical behavior during tests, it is desirable that any mechanical features in construction which will contribute to this end are worthy of further study and experimental investigation. The writers will recommend that the transverse bar be studied and experimented upon further on the basis of the good results obtained in the present tests.

6. BEHAVIOR OF PIN PLATES

(*a*) *Slipping of Pin Plates Discussed.*—A study of the compressions corresponding to stations 15 to 16, 17 to 18, etc., on columns 31HC, 31HC1, 34CS1, 32S, 32S1, Figs. 26, 32, 34, 27, and 33, will show that there is in general a difference in the values for the parallel gauge lengths, these being taken so as to show any relative displacement of one plate upon the other, for example, as at stations 15 to 16 of column 32S1, and similarly for the others. In the stress-strain curves corresponding to these stations the curves whose abscissas are the average compressions for 15+19 and 16+20, etc., are platted side by side. The resultant curve adjoining, which was gotten by algebraic addition of the separate abcissas, shows the detrusion of the plates. The shape of the latter curves indicates the relative movement is increasing quite uniformly with the loadings from the beginning of tests. This slip in some cases could be actually discerned by the "stepping" of the originally plane surface at end of pin bearing.

The movements above described are partially elastic. An examination of the pin-plate stations of columns 34CS1, which was especially studied under repetitions of loading, show that when a load of 10 000 pounds per square inch was applied, and a repetition was again made at zero, that there was a set averaging from about one-half to two-thirds of the relative movement of the plates. It is difficult to assign the cause of this absolutely. Doubtless it is made up of a combination of effects from some infinitesimal play in holes, initial strain in and around rivets, etc

After several repetitions of the load it will be noted in many. cases that there is finally realized a tendency to reduction of the areas of the stress-strain cycle as if the sets, so to speak, were "ironed out," by the loading as in the previous discussions. For example, under station 15 of column 34CS1 (chrome) at the three successive loadings of 10 000 pounds per square inch, the compressions in 16 inches are 0.0042, 0.0042, and 0.0043 inch, the values at 1000 being 0.0011–0.0010 inch, as if a final equilibrium were taking place. This is fulfilled in some of the others. In several cases a marked discontinuity occurs, as, for example, under station 21 the three compressions at 10 000 are 0.0076, 0.0091, and 0.0124 inch, respectively, a progressive action, the compressions at 1000 being 0, 0.0021, and 0.0051 inch.

In the mechanics of deformable media overstraining at a point or small region generally merely causes a local change in the density

of the material in the region under consideration, so long as the general strain configuration is not changed and fissures or cracks do not develop. If the continuous web of a column at the pin is overstrained so as to cause a slight flow of the material at the pin, probably the most direct influence would be exerted in decreasing to some slight extent the configuration of the truss, without affecting the column strength, except as the truss was deformed itself and a redistribution of stresses occurs in the members. The column U_2U_3 in which the movement of the plates was most prominent is, in fact, noteworthy also in failing at the highest load with reference to the yield point of its steel. (See Fig. 43.) The effect of a movement of plates on the truss pin, however, would be relatively more important and probably would cause a readjustment of forces and a change in bending moment of the pin. It is believed therefore in this respect the tests, taken into account with other investigations, show that a further study of the causes which affect detrusion of plates is advisable. The length of plate is manifestly not the only variable to be considered, and the initial strains on rivets undoubtedly play a part.

An examination of the views of columns taken after failure indicate that the designers in all cases have effected a gradual absorption of the strain from the pins into the bodies of the columns. There is no evidence of that buckle in the web at the end of pin plates (with the exception of US_2MS_1) which is so prominent in many previous tests, and which is caused by too abrupt a transition in the flow of the strain and stress into the webs. The fact that such a large percentage of the members failed as units, in comparison with previous investigations, offers abundant evidence that efficient designing of details can be easily assured in engineering practice.

In the case of the four columns failing at the ends, the problem of designer was a rather difficult one, owing to the long jaws required for egress of eye bars. In spite of the large battens and transverse diaphragms used a local failure occurred as is shown in cuts. An idea of the transverse component causing buckling in US_2MS_1 (Mayari) is gained from the fact that 14 rivets are simultaneously sheared (see Fig. 57) at the local failure.

(*b*) *Distribution of Stress and Strain near Pin.*—In measuring strains or displacements around the pin 8-inch gauges were used to give sufficient magnification to the deformations, which are not in any event large. Two-inch gauges would have been

better for defining the field of stress, but on account of the thickness of the plates, the proper clearing of rivets, etc., the 8-inch gauges were used and will give a better practical and experimental determination, from the engineer's standpoint, of the behavior that occurs in this vicinity. On account of the more rapid rate of variation in the stress and strain close to the pin the mean functions taken in 8 inches will, of course, not afford much idea as to the exact values of the stress constituents at this point. Some approximate idea of these may be gained by an examination of related problems which may be referred to in this connection.[44] The problem of distribution is complex viewed theoretically, but keeping certain essential points in mind, it will not be very difficult to analyze the distribution with respect to the practical engineering points at issue and also theoretically, for the controlling factors.

The stress field in the neighborhood of pin may be very appropriately called a fan distribution from certain analogies presented in the surface loading of beams as given by Sir G. G. Stokes.[45] While designers recognize the general character of stresses to be met here, few, if any, critical studies have been made in defining the intensities, and the common assumptions in some cases are not actually realized experimentally. The matter has been left largely to the judgment of the designer.

Taking up first the distribution of stress upon the pin at the base of the fan (see Fig. 71), specifications call for the stress on the bearing plates and webs of columns per unit of thickness, to be determined by dividing the working load on the plates by the diameter of pin. This is tantamount, of course, to the assumption of a uniform normal pressure over the periphery of pin. Obviously, such a law can not even approximately hold. A normal pressure on the pin surface varying as the ordinates of a parabola from zero to a maximum just over the pin would be more in accord with mechanical methods, for a first approximation to the stress constituents on the surface boundary at pin. In the case of column U_2M_1 (Mayari) a study of the compressions at stations 35, 36, 37, and 38, Table 19, show that the maximum intensity of

[44] See Chapters VIII and IX, Love's Theory of Elasticity, 2d ed. Also Comptes Rendus, Tome 114, 1892, p. 1467, "Sur la répartition des pressions dans un solide rectangulaire chargé transversalement," by Flamant.

[45] Discussed by Sir G. G. Stokes in paper by Wilson, Proceedings Physical Society of London, Vol. XI, 1891, p. 194, The Influence of Surface Loading on the Flexure of Beams. Also by Love, Theory of Elasticity, 2d ed., p. 351.

stress is on the axis just over the pin [46] and that there is a rather pronounced decrease at 35 to 38 from 36 to 37 and more so as one goes from the axis laterally. In the case of column HC (car-

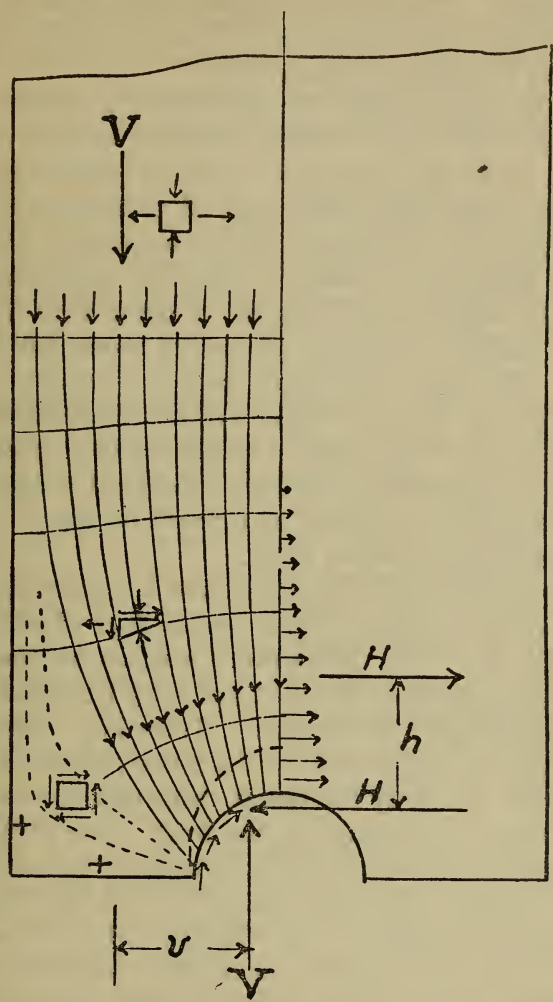

FIG. 71.—*Field of strain at pin (conceptual)*

bon), see Table 6, the distribution over the pin is more uniform, but the influence of the longitudinal diaphragm is thought to be present here to modify the action described. Now as rivets are designed for bearing and shear with respect to their uniform dis-

[46] This is a common physical phenomenon; for example, when a positive electrode is placed conveniently near an equally strong negative electrode, these constituting a "source" and "sink," the main flow is in the straight line between them, but a portion will spread out over the intervening medium in the above manner. In the present case, carrying out the analogy, the column may be considered "grounded" at the upper end, the strain energy being transmitted from the pin through the member.

tribution on plates, it will be rather obvious, in common practice, that there is a tendency for rivets to be overstressed just over the pin. This, it is believed, is a contributing influence in causing the slip in plates. The play usually allowed in pinholes augments the tendency to concentration of the load on plates at the pins to the portion in region of column axis.

Analytically, there should be considered a friction stress on surface of pin, due to the wedging down of column on pin, this being in the main symmetrically distributed as in the preceding case and, upon the usual assumption of a constant coefficient of friction, varying as in the case of the law for normal stress. When the flexural moment in column causes an appreciable turning about the pin, which is not believed to occur until an advanced stage of the loading is reached, the law of course will change and the friction will resist the flexural action of column. The frictional stresses are then no longer symmetrically distributed, or approximately so. The imperfect symmetry of distribution sometimes existing at the earlier loadings is believed to be due to slips and readjustment in the parts as a result of column flexure, as has been mentioned for other cases.

In related problems of this kind there is a practical boundary to the compression area as defined by the lines of principal stress composing the fan. The differential equations for these lines are ordinarily independent of the load and may be found as soon as the stress constituents are known.[47] There is a neutral area beyond the region of compression at the lower corners of columns in which the strain is mainly rotational, distorting the material without a volumetric compression of the steel. The evidence of this region is shown in the tests by the smallness of the measured direct strains and the fact that they change from compressile to tensile. Reference may be made to the character of the elongations at stations 42, 43, 44, and 45, etc., of column HC (carbon), Table 6. If compressions occur at the outer stations, they are low, being reduced by the general tensile tendency of the stress on the outer face of corners. In these neutral regions at the lower corners the rivets are ineffective and offer little help until a movement of plate is caused by the heavy compression over pin when a redistribution of stress occurs, and the rivets at these "dead ends"

[47] The question of the existence of stress functions which shall be congruous with any assumed law of stress distribution at the pin or over the cross sections of the body of the column is not one that need enter here. Even in analysis these can usually be expressed only as "average stresses" in this type of problems. Suffice it to say, however, in any practical case the laws of variation can be found by suitable convergent processes, i. e., successive approximations, which will serve all the needs of the designer.

then help in distributing the stress. In the analysis of stresses in pin plates of the Steubenville Bridge test columns it was stated that the lines of boundary of the compression area over pin should be taken at 45°. An elliptical boundary is, however, more in accord with theory and other experimental determinations. The rotational character of the strain in the "dead end" portions is accounted for by the region being caught up between the confluence of strains of opposite sign which bound the triangular-shaped vortex of the region.

(*c*) *Splitting Action of Pin on Web and Plates.*—Although few observations are recorded here to show the type of stress to be described, it is important also to mention that there is a wedge or splitting action of pin on the column web plates that is never considered in specifications, and it will be well to investigate it in connection with this stress distribution. Taking into account the static equilibrium of the half-section at the pin shown in Fig. 71, the line of upward pressure on the pin surface does not coincide with that of the resultant of the fairly uniform direct stress on the half cross section of the columns. There is then a stress couple Vv, which must be resisted by a couple Hh, composed of the horizontal stress area over the pin acting with an effective arm whose length is equal to the distance along column axis between the line of action of the outward thrust of the pin and that of the resultant of the stress area covering the region of splitting action [48] above the pin. As the only other horizontal stress to consider in the equilibrium of the half section is the outward component of thrust of the pin, the stress area in question must be equal to this thrust. This would be determined as the sum of the effective components of normal and frictional stresses on pin already mentioned and would be difficult to determine by direct experiments. As a first approximation this stress may be assumed to vary as a linear function of the distance from pin along axis, this being the simplest function which vanishes at that point where the direct stress is finally uniformly distributed over column cross section, for after this point is reached the major principal stresses being coaxial with column and the column surface free of stress, no horizontal stress on the axis is needed to maintain equilibrium.

The general character of all the limiting stresses, so far as they are needed in this discussion, have been indicated in Fig. 71. In

[48] Columns have been noted to split at the pin plates in tests. (See, for example, Fig. 6B, p. 687, of Buchanan's tests, Eng. News, Dec. 26, 1907.)

column U_2M_1 (Mayari), Table 19, the readings at stations 46 and 48 will show the amount of this horizontal "splitting" stress for working loads is not large, the value for 46 close to pin undoubtedly being influenced by the frictional components perpendicular and transverse to the pin at its surface. Taking, for example, as an approximate determination, the elongation at a load of 26 960 pounds per square inch is 0.0026 inch under station 48. Subtracting from this the natural expansion under the compression load or (Poisson's ratio = 3/10), 3/10 of the mean compression as found from averaging corresponding readings for stations 27, 28, 29, 30, 35, 36, 37, and 38 (= 3/10 by 0.0051), there is found 0.0026 − 0.0015 = 0.0011. Which corresponds to a tension of 4200 pounds per square inch on basis of $E = 30\ 000\ 000$, as an average value for the intensity of the stress resisting splitting action at a point on the column axis and near the pin.

The distance from the pin at which the stress becomes fairly uniform over the cross section, as is evidenced by the compressions converging to a common value, is seen to be approximately at the end of the second pin plate in column HC (carbon), Table 6, and near the end of first plate in U_2M_1 (Mayari), Table 19. They are, in other words, long enough in this respect.

That there is evidence of internal strain, probably of the character already discussed in connection with those due to rivets, to be considered in connection with a study of the strains and stresses in pin plates, also possibly some small flexural action in the column, is evidenced by the unsymmetrical arrangement to some extent of the distribution, as may be noticed in the elongation reading under stations symmetrically taken with respect to the axis. This is believed to be due to imperfect unisons of action of the parts rather than any bending proper in column. Of course there is some bending moment at the ends of the column, but what its real action is and its amount is more or less indeterminate. From the fact already mentioned that the deflections only are apparent near failure, the writers are led again to believe that the short column action is maintained within limits through the greater portion of the load.

V. SUMMARY AND CONCLUSIONS

1. REMARKS

The 18 test members were of modern design and workmanship and were straight without defects from handling. It is believed the columns are representative of the most recent engi-

neering bridge practice in the construction of members for long-span bridges and that the tests will throw considerable light upon the behavior during tests that may be expected of other columns of the same general type of construction. The members were preliminary studies of the designers. The purpose of the tests was to determine the strengths of the columns under intended axial loads and to obtain data for studying the laws of distribution of stresses in the columns with a view to estimating the probable strength of the full size chord members in the bridges.

2. RECAPITULATION

1. Twelve of these 18 columns failed as units, viz, the two test members of 3¼ per cent nickel steel for the Municipal Bridge and the 10 specimens of Mayari, chrome, silicon, and high-carbon steels for the Metropolis Bridge. When the maximum loads as determined were platted as functions of the grades of steel used in their fabrication, the mean curve taken through the centroids of the groups of observations showed a fairly close agreement with the mean curve determined by averaging and platting the respective mean yield points as found for the corresponding test pieces. The average deviation of the maximum loads for the 12 columns from the mean yield point of all the test pieces was ½ per cent. The minimum and maximum deviations in absolute values were 3.7 and 12.5 per cent. The curve for the failures fell within the zone bounded by the curves of high and low yield points with two exceptions, one point being just above and another just below this zone.

2. Four test columns of the Memphis Bridge failed locally at the ends, which the experiments showed was largely incident to difficulties experienced by the designers in providing for sufficient space for the eyebars at the pin connection. The two remaining test members failed in the plane of greatest theoretical strength, apparently due to the lack of full unity of action of the component ribs of which the columns were composed, the effective strengths of the ribs, as determined experimentally, being less than that expected for the columns as a whole, although one of these members of Mayari steel was well within the zone of failures as defined by the extreme yield points.

3. Taking into account the failure of the nickel and Mayari steel columns at stresses of 3000 to 5000 pounds per square inch above the mean yield point, and the chrome and silicon steel

members about the same amounts below it is believed that there is some indetermination to be expected in obtaining the real strengths of these members in contradistinction to the effective strengths as recorded. In view of the care exercised in centering the columns and insuring parallelism of the bearing plates to the platens of the testing machine, it is considered that a certain small portion of this indetermination is largely inherent in present testing-machine practice, because of the impracticability of determining precisely the true center of pressure of the resultant force for pin and square bearings. The remaining and larger portion is believed to be incident to the fact that there is not a perfect unity of action of the component steel shapes in distributing the stress from load, this, however, being most in evidence after the region of yield points are reached and redistributions of stresses are occurring.

4. The specimens are too few, considering the variations in the grades of steel used and differences of design, to make precise inductions (apart from the larger field of experimentation already conducted on large columns) which are favorable to existing types of column formulas. As the columns have a range of slenderness ratio of 15 to 43, the influence of a column reduction factor would be less than some of the deviations from the yield point as found experimentally. The behavior with respect to the deflections and variation of stress in the members, however, leads to the opinion that a reduction factor is unnecessary for calculating strengths within the range of slenderness ratios existing for the columns.

5. In the 10 members for the Metropolis Bridge, the lengths were constant and the cross sectional areas equal (very nearly) throughout. The strengths of those columns having the flange angles turned outward and thinner webs and lattice, when referred to the mean yield points of the respective steels as datum, was greater in the ratio of 3:1 than in the case of the more compactly built members having thicker webs and wider lattice, the flange angles being turned inward. As many engineers are of the opinion that stocky columns $\left(\frac{l}{r} \text{ small}\right)$ can be designed without special reference to a cross sectional form factor, such as the radius of gyration, the tests appear to show that there is for such columns as for more slender members an effective distance from the column axis for placing the steel when thickness of metal and length and width of members are considered.

6. The 10 columns of the Metropolis Bridge of Mayari, chrome, silicon, and carbon steel all failed in smooth curves, as against irregular failures of various types in the others. This is attributed to certain features of "compact" design and the type of lattice bracing used.

7. It is believed that column loads in practice should not exceed the proportional limit of the fabricated columns. The mean value of this proportional limit for the columns tested was 48.6 per cent of the maximum stress and 48.4 per cent of the tensile yield stress of the respective component steels.

8. The modulus of elasticity was taken as the ratio of the longitudinal stress to strain for values below the proportional limit of the columns. The value found from the mean of determinations on 15 columns was 29 600 000 ± 120 000. The closeness of this result to the mean value found for plain steel and to the results as found from tests of other large columns of a size commensurate with those of the investigation indicate that the methods used afford a reasonable calibration of the testing machine.

9. The ratio of lateral expansion to longitudinal compression was approximately that of plain steel. The range of variation observed was from 0.29 to 0.327 with an average of 0.30.

10. There was considerable evidence throughout the tests of some lack of elastic action in the "built-up" steel. Apart from the variations in the properties of the different shapes this is interpreted as due to (1) the unequal degrees of wrinkling in shapes while under stress, (2) a partial breaking down ultimately of the friction between surfaces of shapes as the load increases; and (3) the effects of initial strain induced by the driving of rivets.

11. The lowering of the "proportional limit" of a column below the elastic limit found in the compression of a test piece appears to be due in part to (1) the presence of initial strain induced by the radial pressure from the driving of the rivets, which unites with the direct stress from loading at a certain critical point of stress. The presence of these stresses are indicated not only by the theoretical considerations but also by supplementary tests on riveted and plain solid piece struts under compression loading; (2) a further cause is believed to be due to initial curvatures introduced in the elementary portions of the shapes between rivets as a result of the clamping action of riveters. The curvature in the elements which becomes sensibly apparent at a late stage of loading is believed to be one of the causes for

breaking down of the frictional resistances between plates and reducing the longitudinal shearing resistance. It is the opinion that fabricated steel should be studied in this connection with a view to raising the proportional limit of the column.

12. The action of the ordinary type of lattice bars in the body of the column is to draw together in the direction of column axis like a pantograph as the ribs shorten under load, the strains in lattice being small, such as occur being due to the slight resistance induced against bowing out of the flexible rib portions, as the lattice panels flatten out with the compression of the columns. The large strains are at those panels, close to end battens, where the restraint against the above uniform action causes more or less discontinuity of action, and produces bending in the ribs and a tendency to straining of inner edge of battens. The tendency of strong riveting at ends of lattice is to cause bending of lattice as the column ribs shorten, the stresses being largest in wide lattice bars. Column flexure proper apparently does not have much influence in members of small slenderness ratios, the more or less unsymmetrical distribution being due probably to readjustments in the bars themselves. The tendency of single lattice, on account of assymetry of the strain configurations induced, causes different curves of bending in the outer ribs, which in effect is equivalent to an eccentric application of the load on the column rib. The general tendency for these columns with the ordinary type of single and double lattice is to fail with irregular and local fractures. Symmetrical groupings of bars or component struts as far as possible operate advantageously.

13. The specific influence of transverse lattice bars between the panels is to prevent pantograph action of lattice bars, outward bowing of the ribs, and their abrupt bending at the batten plates and transverse diaphragms. It is the opinion that after the lattice has been designed for an arbitrary transverse shear taken as a safe per cent of the axial load to provide efficient trussing of the members for resistance against gravity and other transverse forces little advantage is gained by adding material to the intermediate diagonal bars. It should rather be placed in the transverse bars and ribs. The load on the column should be carried by the shortest path through the ribs alone, and not through ribs and lattice, as far as is practicable.

All the columns with transverse bars failed in smooth curves. This is regarded as favorable for maintaining the mechanical

principle of continuity, that the bending moment shall vary with the curvature of column, upon which correct principles of structural design are necessarily founded.

14. The tests showed that in nearly all cases the pin plates were long enough to give an easy transition of stress from the pin into the body of the web plates without buckling of the web plate at the end of the last pin plate. The pin plates appeared from the strain measurements to be of sufficient length to insure a quite uniform distribution of stress over the cross section before the web proper was reached. All the pin plates showed some relative slipping, which appeared to be due primarily to overstressing of rivets just over the pin on account of the intensity of stress at this point. This behavior, however, does not appear to reduce the column strength as a whole. There are, in fact, some grounds for believing that slipping of the pin plates or other portions relatively may at times have operated advantageously as an easement to adverse stress conditions. Many points of this nature can only be determined by independent tests on plain and laminated riveted steel. Tests of this character are recommended.

WASHINGTON, May 4, 1917.