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COMPRESSIVE STRENGTH OF COLUMN WEB PLATES
AND WIDE WEB COLUMNS

BY

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COMPRESSIVE STRENGTH OF COLUMN WEB PLATES AND WIDE WEB COLUMNS

By Robert S. Johnston

ABSTRACT

The investigation had as its major problem the verification of the design rule that wide web plates in compression should not exceed in width thirty times the thickness of the plate. It also included a study of the strength of laterally supported wide web columns and a comparison of the merits of single plate and double plate webs of equal thickness.

The tests show that while the common design rule is conservative, buckling strength is also dependent upon the mechanical properties of the material and that the buckling resistance of a plate may be computed for known material. Close agreement is shown between test results and computation by Bryan's formula.

The relative action of the single and the double plate webs as exhibited by the few tests made does not indicate that either is inferior to the other when the material is identical. The stitch-riveting used was sufficient to make the plates act as a unit and indicated no decided detrimental effects from such riveting.

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I. PURPOSE OF THE TESTS

The investigation reported in this paper was initiated by the Delaware River Bridge Joint Commission. Its purpose was three-fold: (1) To study the relation between the thickness of plate and unsupported widths of plate for compression members with particular reference to the validity of a commonly accepted structural designing rule as applied to high-strength steel; (2) to determine the ultimate strength of laterally supported wide web column sections of silicon-manganese steel of the type adopted by the commission for the towers of the Delaware River Suspension Bridge with particular reference to the factors that determine the strength of such columns; (3) to compare the action of webs of two thin plates stitch-riveted together, with a single thick plate web of equal thickness.

After consultation between the Bureau of Standards and the Delaware River Bridge Joint Commission the dimensions of the test specimens and the general test procedure were decided upon and the column sections ordered and forwarded to the bureau for test. The sections tested were full size tower section elements and represented the tower portions indicated in Figure 10.

II. TESTING MACHINES

Fourteen specimens were tested to destruction. The 6 smaller sections comprising the columns with the three-eighths, one-half, and five-eighths inch web plates and 1 with a three-quarter-inch web were tested in the 2,300,000-pound Emery horizontal testing machine, and the 7 remaining sections were tested in the 10,000,000-pound Olsen vertical machine. (See fig. 1.) The three-quarter-inch web section tested in the Emery machine was retested in the Olsen machine as a check on the calibration of the two machines. Further, it was thought that the ultimate strength of this section was beyond the capacity of the Emery machine, and, therefore, that a retest on this specimen was really necessary to determine its true strength.

The use of the two machines was made necessary by the fact that the 10,000,000-pound machine was moved from the bureau's former Pittsburgh laboratory to Washington and had not been installed in Washington at the time it seemed necessary to begin these tests.

The agreement between the two machines was close, their calibrations showing no discrepancies outside of the limits of the possible variations (approximately 1 per cent) of the calibration apparatus (calibrating bar and extensometer) so that no corrections have been applied to the indicated loads. The accuracy of the machines is also indicated by the uniformity of the initial modulus of elasticity, approximately 30,000,000 lbs./in.² of the column sections.

FIG. 1.—10,000,000-pound vertical testing machine; column G-7 under test

III. MATERIAL

The material used in the test columns was a silicon-manganese steel. This type of steel seems to have been developed initially for the United States Navy Department, where it is known as high-

tensile steel (H. T. S.) and later used for bridge construction. We believe its first use in bridges was for the Bessemer and Lake Erie Bridge, built by the American Bridge Co., and later it was used on the Metropolis Bridge. It is now quite commonly used for such structures.

All angles for the test sections were rolled at the Saucon plant and the web plates at the Maryland plant of the Bethlehem Steel Co. As far as is known no special treatment was given this material.

Chemical analysis made by the Bureau of Standards of samples from these angles and plates not only showed the degree of uniformity that might be expected but indicated that the webs and angles were of material of somewhat different chemical composition. (See Table 1). The webs were somewhat higher in carbon content and considerably lower in manganese, silicon, and phosphorus.

TABLE 1.—Mechanical and chemical properties coupon test specimens

Specimen No. ¹	P. L.	Y. P.	U. T. S.	Elongation in inches	Reduction area	Modulus	Carbon	Manganese	Silicon	Phosphorus	Sulphur
	Lbs./in. ²	Lbs./in. ²	Lbs./in. ²	Per cent	Per cent	1,000 lbs./in. ²	Per cent	Per cent	Per cent	Per cent	Per cent
G1-1	37,000	52,500	88,300	23	55	30,400	0.32	0.95	0.33	0.034	0.029
G1-5	33,000	48,500	86,000	24	55	29,000	.31	.95	.33		
G1-1 P	47,000	50,600	81,600	26	51	30,000	.36	.58	.20	.010	.030
G1A-1	40,000	49,500	86,700	24	56	30,300	.30	.95	.35	.034	.029
G1A-5	38,000	48,200	85,800	23	55	29,800	.31	.96	.35	.034	.027
G1A-3 P	32,250	48,500	83,500	24	55	29,900	.37	.57	.21	.011	.033
G2-1	45,000	50,000	88,600	22	49	29,900					
G2-5	38,000	51,500	92,500	23	48	28,800					
G2-4 P	37,000	46,900	80,000	24	43	30,200	.35	.58	.22		
G2A-1	45,000	50,000	85,700	25	57	29,400					
G2A-5	41,500	49,000	85,400	24	56	29,200	.33	.94	.32		
G2A-4 P	40,000	47,000	78,700	24	57	29,800	.34	.57	.22		
G3-1	49,000	50,000	87,400	23	53	28,500	.33	.94	.33		
G3-5	42,500	51,000	88,400	22	57	29,700					
G3-1 P	37,000	45,000	81,100	24	51	28,700	.36	.58	.22		
G3A-1	40,000	50,000	85,100	23	57	27,800					
G3A-5	30,000	43,000	77,600	25	60	27,100					
G3-3 P	32,000	44,000	83,400	25	44	29,200	.37	.60	.21	.010	.033
G4-1	48,000	50,300	86,800	24	53	29,000					
G4-5	40,000	50,400	87,100	23	54	28,300					
G4-4 P	34,000	45,600	85,800	25	44	28,300	.40	.59	.22		
G4A-1	44,000	51,500	88,800	26	51	28,800	.33	.97	.32	.039	.032
G4A-5	42,000	52,300	89,400	25	50	31,500	.31	.94	.33		
G4A-4 P	37,000	43,500	81,200	25	48	28,000	.37	.57	.20	.012	.035
G5-1	43,200	50,400	87,700	25	51	28,400					
G5-5	44,000	50,100	86,800	24	43	30,500					
G5-1 P	34,000	43,000	85,600	21	36	28,700	.42	.60	.21		
G5A-1	52,600	52,600	90,400	25	48	29,100	.31	.96	.29		
G5A-5	42,500	50,900	87,200	26	52	30,300					
G5A-3 P	(31,500)	(²)	(²)	(²)	(²)	(²)	.36	.59	.22		
G6-1	45,000	50,500	87,400	24	54	29,500	.33	.97	.36	.034	.030
G6-5	43,000	51,500	89,100	22	53	29,400	.34	.97	.34	.039	.030
G6-1 P	36,500	46,500	79,000	25	52	29,600	.34	.57	.22		
G6A-1	43,500	50,000	88,200	24	54	28,900	.34	.97	.34	.036	.030
G6A-5	43,000	48,000	89,000	25	53	28,800	.35	.97	.32	.040	.033
G6A-1 P	44,000	47,400	78,600	24	51	29,300	.34	.59	.21		
G7-1	48,500	50,500	87,700	25	53	31,300	.32	.96	.33		
G7-5	44,500	50,000	87,600	24	54	28,700	.32	.98	.32	.039	.030
G7-1 P	45,000	51,000	92,500	19	37	30,200	.43	.61	.21	.013	.041
G7A-1	45,800	50,500	87,500	23	53	25,300	.33	.94	.30	.043	.033
G7A-5	48,500	50,200	88,100	22	53	28,700	.34	.95	.31	.044	.034
G7A-1 P	39,000	46,500	86,800	20	31	29,300	.42	.58	.24	.013	.039

¹ Specimen number followed by letter "P" designates coupons from web plates; the remainder are test results from coupons from angles.

² Defective material.

IV. TESTS OF THE MATERIAL

Special attention was given to the determination of the mechanical properties of the material. The specifications governing the fabrication of the columns stipulated the method of cutting and assembling all angles and plates for the various columns. Further, coupon specimens of the material for tensile test were taken from each end

FIG. 2.—Methods of shearing and match marking flange angles of columns

of all pieces entering into the construction of the columns. Figures 2 and 3 show the manner in which the material was cut for fabrication. The test-specimen coupons were cut out and machined at the fabricating plants. The specimens from the angles were cut from one leg and from the web-plate material from the edge of the plate and in some cases from the center of the plate. All surplus material

of the ends ($G1_1$, $G1A_1$, etc.) left after the material coupon specimens were cut out was forwarded to the bureau for such supplementary tests as seemed desirable.

The standard A. S. T. M. specimen for rolled material of $1\frac{1}{2}$ inch width at the reduced section and of the thickness as rolled was used. In all tensile tests a Ewing extensometer on an 8-inch gauge length

FIG. 3.—Methods of shearing and match marking web plates of columns

was used to determine the stress-strain relations for the material. Extensometer readings were carried well beyond the yield-point stress. The stress at the proportional limit and the yield point (see fig. 4) were determined from the stress graphs. The commonly employed "drop-of-beam" method of yield-point determination was not used. Typical stress graphs are given in Figures 4, 5, and 6.

V. COLUMN SPECIMENS

The column test specimens were all 10 feet long, of the plate and four-angle section, the back to back dimension of the angles being 35 inches. (See figs. 7 and 8.) The flange angles in all sections were also identical, being 6 by 4 by $\frac{3}{4}$ inch angles with the 4-inch legs outstanding. (See figs. 7 and 8.) The differences in the sections were

FIG. 4.—Stress-strain curve of angle tensile test, coupon G3A-1

in the thickness of the web plates, which varied from three-eighths inch to $1\frac{1}{2}$ inches as shown by the dimensions in Figures 7 and 8. The specimens were furnished in duplicate. The webs of the heavier sections (fig. 8) were composed of two plates of equal thickness stitch-riveted together. The geometrical properties of the sections are given in Table 2. (See also fig. 9.)

FIG. 5.—Stress-strain curve of angle tensile test, coupon G7-1

FIG. 6.—Stress-strain curve of angle tensile test, coupon G1-1

TABLE 2.—Properties of column sections

Girder No.	Material	Total area	I (—)	r (—)	$\frac{L}{r}$ (—)	I (—)	r (—)	$\frac{L}{r}$ (—)	$\frac{d}{t}$ for unsupported width of plate			Area of flange	Area of web
									26 inches	30 inches	35 inches		
G1 and G1A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 1 plate 34 $\frac{1}{2}$ by $\frac{3}{4}$	{ 40.70	{ 7,982	{ 14.00	{ 8.571	{ 79.55	{ 1.39	{ 86.331	{ 69.4	{ 80.0	{ 94.4	{ 68.2	{ 31.8
G2 and G2A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 1 plate 34 $\frac{1}{2}$ by $\frac{1}{2}$	{ 45.01	{ 8,410	{ 13.67	{ 8.778	{ 84.26	{ 1.37	{ 87.591	{ 52.0	{ 60.0	{ 70.0	{ 61.7	{ 38.3
G3 and G3A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 1 plate 34 $\frac{1}{2}$ by $\frac{5}{8}$	{ 49.32	{ 8,838	{ 13.38	{ 8.969	{ 89.33	{ 1.35	{ 88.889	{ 41.6	{ 48.0	{ 56.0	{ 56.3	{ 43.7
G4 and G4A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 1 plate 34 $\frac{1}{2}$ by $\frac{3}{4}$	{ 53.63	{ 9,265	{ 13.15	{ 9.125	{ 94.78	{ 1.33	{ 90.226	{ 34.7	{ 40.0	{ 46.7	{ 51.8	{ 48.2
G5 and G5A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 1 plate 34 $\frac{1}{2}$ by 1.....	{ 62.26	{ 10,121	{ 12.77	{ 9.397	{ 106.96	{ 1.31	{ 91.603	{ 26.0	{ 30.0	{ 35.0	{ 44.6	{ 55.4
G6 and G6A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 2 plates 34 $\frac{1}{2}$ by $\frac{1}{2}$	{ 62.26	{ 10,121	{ 12.77	{ 9.397	{ 106.96	{ 1.31	{ 91.603	{ 26.0	{ 30.0	{ 35.0	{ 44.6	{ 55.4
G7 and G7A.....	{ 4 Ls 6 by 4 by $\frac{3}{4}$ 2 plates 34 $\frac{1}{2}$ by $\frac{3}{4}$	{ 79.51	{ 11,832	{ 12.21	{ 9.828	{ 137.47	{ 1.31	{ 91.255	{ 17.3	{ 20.0	{ 23.3	{ 35.0	{ 65.0

NOTE.—The $\frac{L}{r}$ ratios given do not include the stiffener channels.
 I =moment of inertia; r =radius of gyration; L =length of column (10 feet); t =thickness of web; d =“unsupported” width of plate.

The columns were fabricated at the Steelton plant of the Bethlehem Steel Co. The shop drawings called for 1-inch carbon-steel rivets with the holes either punched thirteen-sixteenths inch and reamed to $1\frac{1}{16}$ inches while assembled or drilled from the solid. In fabrication the plates three-quarters inch thick and under were subpunched and reamed, while the thicker plates were drilled from the solid.

An inspection of the columns before testing indicated a good quality of fabrication with no overdriven or eccentric head rivets, and initial measurements for straightness exhibited only such curvature or buckling as might be expected in well-fabricated steel. Imprints of the angle ends and planimeter computation of areas gave values in close agreement with the standard area for a 6 by 4 by $\frac{3}{4}$ inch angle. No correction to the theoretical areas for the sections has, therefore, been necessary in the analysis following.

FIG. 9.—Location and assembly of deformation instruments

VI. ASSEMBLY OF TEST SPECIMENS

Previous to the assembly of the test specimens in the testing laboratory ink-pad impressions of the flange angle areas and measurements of web size and column straightness were made.

For use in testing the column test sections there was supplied a pair of stiffening channels of the detail shown in Figure 9. These channels were bolted to the column section flanges by S clamps as indicated, and were intended to afford lateral support against buckling and failure in simulation of the stiffening effect of the transverse members actually employed in the bridge towers. (See fig. 10.) It was not desired to have these stiffening channels carry direct stress, however, and for that reason they were made 9 inches shorter

than the column section, and fastened to it with clips, rather than rivets.

The method adopted for the assembly of the combined column and stiffening channels was governed by the desire to combine maximum lateral support of the column with freedom of the stiffening channels from direct loading. The S clamps which were opposite and spaced 12 inches apart on centers were greased to reduce their frictional longitudinal grip. In a number of cases it was necessary to pack these clamps with greased shims in order to connect the stiffening channel to the column section as rigidly as possible, owing to deformed clamps (as a result of previous tests), or to inequalities in alignment of the flanges, or both.

In order to use the stiffening channels on the column sections of different web thicknesses the holes for bolting on the S clamps were

FIG. 10.—Cross section of bridge tower, illustrating lateral stiffening of columns

slotted. To restrain the clamps from sliding sideways under lateral deflection of the test pieces, these slotted holes were filled with molten lead after the S clamps had been properly adjusted by shimming, brought in full contact with the flange angle edges and bolted securely. For bolting, all final tightening was done with a 2-foot ratchet wrench except in the case of column G3A, which was tested first. The clamping bolts were standard three-quarters inch.

The method of assembly proved entirely satisfactory for the purposes intended. The special compressometers placed on the stiffening channels showed no indication of direct stress during the early stages of loading the specimen, and where crushing of the specimen was continued to failure, there was little evidence of any lateral slippage of the clips or bolts, the clip bolts always failing by shearing or tension where the clip was located at a flange buckle.

VII. TEST MEASUREMENTS

1. COLUMN ADJUSTMENT

The column sections were centered in the testing machine as carefully as possible. There is, however, no assurance in the average machine that after centering the application of load will be uniform over the section, even supposing the column ends perfectly plane and parallel. There is some lost motion and give in the straining heads of a testing machine under the first application of load. To eliminate this and insure as near as possible a uniform application of load at the early stages of the test, the following procedure was adopted:

The four 60-inch compressometers located on the column section itself were adjusted, after centering the specimen in the machine, and an initial reading taken on each. The specimen was then loaded to an amount equal to 8,000 to 10,000 lbs./in.² on the cross section of the column and the compressometer readings taken again. The difference between the initial and second reading for each gauge gave the compression on its particular gauge length of the member to be tested and made possible a comparison of the shortening on each gauge length; that is, the uniformity of compression over the column section. Where these were not in agreement, the load was released, the movable head (spherical joint) of the testing machine was adjusted and the trial loading process repeated until the strain-gauge readings indicated extreme stress differences of not more than 500 lbs./in.².

This method of adjusting a member for uniform distribution of loading is expeditious and seems to be as accurate as it is possible to be with present facilities, as it eliminates in large part the machine irregularities, does not damage the specimen, and possesses the further advantage that it insures that the deformation measuring apparatus is in proper working order before actual testing is commenced. After adjustment by this method such supplementary apparatus as is desired may be placed with assurance that its adjustment will not be disturbed.

2. MEASUREMENT OF WEB DEFORMATION

As stated above, the tests were made principally to study the comparative action and strength of the webs of the column section, a study of the compressive strength of web plates of high-strength steel. For this reason considerable attention was given to determining the deformation of the web under load, both transversely and longitudinally.

The transverse deformations were obtained (see figs. 9 and 14) from readings of a micrometer dial gauge attached to a special sliding carriage mounted on a runner bar, the latter being adjustable trans-

versely on end guide rods. (See fig. 14.) The micrometer dial could thus be made to traverse all points on the web face so that the deformations of the web could be determined at any point.

This contour apparatus was attached to only one side of single web specimens. Where two plates were used for the webs, duplicate apparatus was used on each side to make possible a study of the relative deformation of the two webs.

All contour readings were referred to a plane surface (except for slight and unimportant deformation of the measuring apparatus), which was established as follows:

The upper stud bolts holding one end of the guide rods were screwed in place and fastened with the lock nuts. The guide rods were then assembled on these studs so that each was the same distance away from the upper corner face of the web plate, and fixed in position by lock nuts. Then the guide rods were plumbed vertically and locked in position on the bottom stud bolts. To facilitate this latter adjustment the lower ends of the guide rods were slotted to permit lateral adjustment. These guide rods then became two controlling lines in the plane which the runner traversed. The general arrangement of this apparatus is shown in Figure 9.¹

For convenience in studying the web changes the dial readings were taken at the intersection of transverse and longitudinal lines marked on the web-plate surface as shown in the various photographs. The transverse lines were spaced 3 inches apart, and, in general, there were five longitudinal lines between the inside edges of the flange angles, one set just within the flange angles, one on the longitudinal center line, and a set midway between these two. Additional readings were taken on the same transverse stations, but on the flange angles, at least one line being taken on each angle.

3. STRAIN-GAUGE READINGS AND DEFLECTIONS

In the study of column action as a whole four compressometers (*S. T. G.*, *N. T. G.*, *S. B. G.*, and *N. B. G.*, fig. 9) of 60-inch length (one-half the length of the specimen) and located at mid length of each flange angle were used. These consisted of a micrometer dial, reading to one one-thousandth inch, rigidly attached to a stud inserted at one end of the gauge length and actuated by a bar which was also rigidly attached to another stud at the other end of the gauge length and connected to the micrometer dial. The actuating rod had a sliding sleeve with lock-nut stud to facilitate dial adjustment.

Four similar compressometers (*S. T. S.*, *N. T. S.*, *S. B. S.*, and *N. B. S.*, fig. 9) of 80-inch gauge length were also attached to the flanges of the stiffening channels. These were used to indicate the character and amount of stress carried by the stiffening channels.

¹ See also Eng. News-Record, 89, No. 23, p. 986; 1923, for description of apparatus.

For determining the lateral displacement of bending of the test section, six Wissler dials were attached at mid length of the specimen in the manner shown in Figure 9.

In an endeavor to determine the amount and distribution of stress in the web plates, a set of 12 telemeter gauges² were used on the sections having single webs. These instruments were attached by small saddles to the face of the web opposite that on which contour readings were taken. Nine of them were in three groups, located, respectively, at the section at mid length, at the quarter point and as near the end filler plate at one end of the section as possible. In each of these three groups one instrument was on each flange angle and one on the longitudinal center line. The remaining three telemetric instruments were used on the longitudinal center line, two diagonally across the line, and the third perpendicular to the instrument placed longitudinally on the center line near the filler plates. In a few instances in the earlier tests all the instruments were placed on the webs to study the development of stress across the web, but it was thought they would furnish more valuable data if part were located on the flange angles.

When the telemeters were used, check readings at similar locations on the mid sections, but on the opposite side (that on which the contour readings were taken) were taken with a West strain gauge.³

On the test sections with double webs, the duplicate contour apparatus precluded the use of the telemeters. For these specimens the strain-gauge readings were confined on account of the working facilities to West strain-gauge readings at the standard locations at mid length but on both sides of the test section.

VIII. METHOD OF TESTING

In general, the following procedure was used in testing a specimen. The sections were loaded by increments, the initial increments each being equivalent to an additional stress of 5,000 lbs./in.² until stresses above 20,000 lbs./in.² were reached, after which the stress increments were reduced to 2,000 lbs./in.². At indicated stresses of about 40,000 lbs./in.² the increments in most cases were reduced to 1,000 lbs./in.² and this rate of loading carried on until the maximum load was reached.

During the process of loading, the average total compression (60-inch compressometers) was plotted against the indicated average load and when the plotted stress-strain relation indicated that the proportional limit (*P. L.*) had been exceeded (generally at indicated stresses of 32,000 to 38,000 lbs./in.²) the total load was reduced to the

² O. S. Peters and R. S. Johnston, New developments in electric telemeters, *Proc. Amer. Soc. Test. Mat.*, 23, II, p. 592; 1923.

³ C. E. Fowler, Revision of Niagara railway arch bridge, *Trans. Amer. Soc. Civil Engrs.*, 83, p. 1945; 1919-20.

initial reading at the beginning of the tests, and then reapplied by the same increments used in the initial loading, the load being carried to failure in the second loading.

For each load increment the various instrumental observations were made with the exception of the contour readings which showed only slight changes until the indicated *P. L.* (as shown by the average compressometer curve) was reached. For that reason, after a thorough preliminary exploration of the web face under the initial load, the contour readings were confined to the longitudinal center line, and often to the immediate vicinity of initial buckles. Full contour readings were taken (1) at the stress just beyond the *P. L.* before the load was released, (2) after the release of the load, and (3) at the maximum load. The number and location of the contour readings taken were determined by the study of an approximate web buckle curve plotted during the test.

FIG. 11.—Extensometer for rivet deformation (double plate webs)

IX. RIVET DEFORMATION

In the study of the action of test sections of double web plate construction two special forms of extensometer were devised and applied to some of the stitch rivets to determine the amount of deformation developed in these rivets as the section was loaded, and through this means to provide a possible measure of the relative action of the two plates.

The form of extensometer used and the method of attachment is shown in Figure 11.

X. DEVELOPMENT OF STRESS LINES

The value of a visual method of observing stress distribution in large test pieces has been recognized for some time. Frequently the development and distribution of the Hartman-Luder lines as indi-

cated by the cracking of the black oxide scale on rolled material has rendered valuable aid in determining stress distribution. It has, however, been generally necessary to make line drawings of the distribution of the Hartman-Luder lines, as they do not show enough contrast even when the specimen has been oiled, and especially until well developed, to allow of good photographic reproduction. Further, the complexity of these lines render exact duplication by drawing almost impossible. For these reasons their value has not been

FIG. 12.—Columns G3 after test, showing strain lines

fully attained, as reliance has more often been placed primarily on memory records or log-sheet notes.

In this investigation it became evident that the development of the Hartman-Luder lines would furnish interesting and valuable additional data on the variation in web-plate failure if they could be intensified so that their development could be readily followed and permanent records of such development easily secured. A simple, accurate, and sensitive method was finally developed by the writer

which made photographic reproduction easy,⁴ with the result shown in Figures 12 to 17. To secure the necessary contrast the test member is first thoroughly cleaned of all grease and then painted with a wash of white Portland cement in water. It is essential that a very thin wash be used; in fact, as first painted on the specimen, it should look more like dirty water than a wash coating. As the water evaporates a thin white coat of Portland cement is left deposited on the metal surface. For best results a relatively wide soft bristle brush should be used, and care should be taken to smooth out brush marks or running of the wash coat. If specially strong contrast is desired, a second coating may be necessary after the first has dried, but care must be taken to avoid a thick film of cement.⁵

With a proper coating, the Hartman-Luder lines will develop by the flaking of the cement coat with the oxide scale; the lines will progress across the member readily and in their initial stages appear like pencil lines drawn on the surface, thus affording clear lines of good contrast for photographic purposes.

This method appears to be far more sensitive in detecting stress lines than the uncoated plate, and lines that develop but are otherwise invisible except perhaps under very favorable lighting show out plainly. A further indication of the sensitiveness of the method was shown by the fact that where the match marks were stamped into the cold plate (leaving residual stress about the marks) scaling that was normally invisible to the eye would develop as small scattered flaking encircling the marks, and as loading was increased would complete itself as a ring with more or less radial flaking about the match mark while adjacent metal would show no such flaking.

While our present knowledge indicates that these stress lines do not appear until the yield-point stress of the material has been reached, they at least show plainly where such intensity of stress is first reached, the general distribution of coincidental yield-point stresses, and their future development. The simplicity of the method and the fact that it can be applied to structural shapes, etc., without extended special preparation suggests a possible wide field of application in future investigations.

XI. ANALYSIS OF TEST DATA

Before attempting a detailed discussion of test results it will be advisable to define a few terms used in the following analysis.

Figure 4 shows a stress-strain curve (tension) for a coupon specimen of the investigation and has indicated upon it certain critical

⁴ G. K. Burgess, *The study of steels for engineering structures*, Papers and discussions of the A. S. C. E., p. 524; Jan. 19, 1923.

⁵ It is, of course, essential that the black oxide mill scale on the structural members should be protected from rusting before the cement wash is applied.

points. The stress corresponding to the letters *P. L.* is called the "proportional limit" and is defined as that stress at which the ratio of stress to strain ceases to be a constant. Above the *P. L.* is a horizontal portion of the curve beginning at *Y. P.*, the stress at which point is commonly designated as the "yield point" and which is defined as the stress at which deformation increases without increase of load. The term "elastic limit" has not been used, primarily because of the confusion of ideas caused by the commercial world's designation of the yield point as the "commercial elastic limit." The constant ratio of stress to strain for stresses below the *P. L.* is known as Young's modulus (*E*) or the modulus of elasticity of the material. Young's modulus is then the tangent of the angle α , and the slope of the initial portion of the stress-strain curve, therefore, affords a means of visual comparison of moduli. By definition $E = \frac{s}{\epsilon}$ and is a measure of the stiffness of the material. (See also figs. 5 and 6.)

XII. WEB PLATE COMPRESSION STRENGTH

WEB BUCKLING

The principal purpose of this investigation was to determine the comparative behavior of columns of high-strength steel in which the web thickness was made the variable; that is, web plate compression strength. There has been in vogue for a number of years, as the result of two series of tests, a belief that if the plate thickness was not less than one-thirtieth of its unsupported width the plate would be safe against failure by secondary buckling.

The original statement of the rule seems to be credited to Fairbairn.⁶ Apparently, it was proposed as a result of the analyses of Hodgkinson's tests. This question was first brought to attention through the crinkling failure of the compression plate in the first tubular girders designed and tested for Robert Stephenson in connection with the Britannia and Conway Bridges.⁷

Stephenson secured the services of Fairbairn and Hodgkinson in connection with the experimental work. Hodgkinson early realized that the crinkling strength of the compression plates of the tubular girders was of equally great importance in columns and secured Stephenson's consent to an extended series of tests on the compressive strength of plates. As a result of this investigation he concluded that the resistance to buckling varied as the cube of the thickness. Later in his celebrated column tests he found that it was necessary to have material twice or even three times as thick as his previous

⁶ G. Bouscaren, The strength of wrought iron columns, T. A. S. C. E. **9**, p. 447; December, 1880.

⁷ (a) Edwin Clark, The Britannia and Conway tubular bridges, London, 1850; (b) Report of Commissioners appointed to inquire into the application of iron railway structures, Parliamentary Paper **29**, London, 1849; (c) Fairbairn, William, Experimental inquiry into the strength of wrought-iron plates and their riveted joints, Philo. Trans. **140**, pp. 677-725; 1850.

tests indicated in order to insure freedom from buckling failure, but no explanation of this result was given. It has not been established, however, that Hodgkinson himself proposed the commonly used rule.

In 1880 Bouscaren reported on some tests of columns made under his direction for the Cincinnati Southern Railway⁸ and called attention to the fact that several latticed channel columns of the group failed by secondary buckling of the web and that in these columns the thickness of the web was about one-thirtieth of its unsupported width, "which confirms Fairbairn's rule."

As a result of these two series of tests the rule seems to have become quite firmly established in structural-engineering practice and applied wherever relatively wide plates are used, although in recent years the increasing desire for economies of design has led to a tendency toward the acceptance of plates as thin as one-fortieth of their width.⁹ As far as is known, no detrimental results have developed as a result of the modification of the original rule.

While the common-design rule has produced entirely satisfactory results it is nevertheless open to question on the basis of the results of previous tests, especially when it is to be applied to present-day use of high-strength steels. Further, engineering practice to date has been quite largely based on test results and experience, uncorrelated with theoretical analysis.

The question has been discussed mathematically in reference to ship plating in a very interesting paper by Bryan.¹⁰ The relation between plate thickness and width and the mechanical properties of the material, for buckling failures, is given by the equation (combining two of Bryan's equations)

$$t^2 = \frac{3b^2S(1-m)}{\pi^2 E}$$

in which

t = thickness of plate,

b = width of plate parallel to direction of buckling,

S = stress at which buckling will occur,

m = Poisson's ratio,

E = Young's modulus.

The foregoing equation is based upon the consideration of a plate having infinite length and presupposes that the plate is supported at its edges on parallel ribs in such manner that there is no shear displacement, at the supports, in a direction perpendicular to the plane of the plate. Further the plate is acted upon by forces applied normal to its ends and in the plane of the plate.

⁸ See footnote 6, p. 752.

⁹ See Manual, A. R. E. A., p. 751, sec. 65.

¹⁰ G. H. Bryan, On the stability of a plane under thrust in its own plane, Proc. Math. Soc., **22**, p. 54; 1890.

Assuming a constant plate width (b) and a constant value for the stiffness of the plate (E) it is evident that the resistance to buckling of a plate of definite thickness depends directly on the development of a certain plate-stress intensity. A thin plate would, therefore, fail at a lower plate-stress intensity than would a thick plate. Buckling failures are, however, secondary failures, and secondary failures in structural members are undesirable.

It is obvious then that if the plate thickness can be increased to such an extent as to at least make it equally as liable to fail by a direct failure as by a buckling failure the plate material will be used more effectively.

Such a plate thickness may be defined as the critical plate thickness. Critical plate thickness by definition thus becomes a thickness at which primary failure only may be expected. A consideration of the stress-strain characteristics of steel suggests that primary failure will occur only after the *P. L.* of the material has been passed. After passing the *P. L.* the material ceases to be absolutely elastic and develops a plastic tendency. As seen from the respective stress-strain curves this tendency varies in intensity with the different grades of steel.

A plate stressed beyond the *P. L.* will take "sets" and further is subject to the continued strain development commonly spoken of as "creep." Permanent deformation of the material, "set," must be looked upon as one criterion for the determination of the useful limit of a structural element. Further, the development of set through exceeding the *P. L.* of the material means that at least part of the material, which is that which is most stressed, has had its modulus lowered, as can be seen from Figure 4, so that the resulting average effective modulus of the plate has been lowered. A material with lower modulus would require, according to the Bryan theory, a thicker plate to sustain the equivalent load. Modulus changes above the *P. L.* are far more rapid than the rate at which increased stress may be developed. With this change in modulus the eccentricity of load, due to initially warped and buckled plates (and no plate is a true plane surface), will be increased and the plate stresses at the most stressed sections thus further augmented. A stress in excess of the *P. L.*, therefore, produces a condition of questionable stability on further loading, which for a plate of critical thickness must necessarily accelerate the final failure.

With these considerations in mind, it is believed that the maximum stress based on the mechanical properties of the plate material, that may be considered for use in the Bryan formula for the determination of critical plate thickness—that is, the plate thickness at which the failure may either be a secondary buckling failure or a primary failure—is the *P. L.* of the material. In fact, from the theoretical

standpoint, the $P. L.$ is the maximum stress that may be used legitimately with the Bryan formula, as the theory is based on considerations of elastic failure and beyond the $P. L.$ the material ceases to be wholly elastic.

A plate thickness determined by the Bryan formula using the $P. L.$ of the material as S gives the critical plate thickness; that is, a plate thickness that may fail either by buckling or by a primary failure.

A comparison of test results with the theoretical critical plate thickness obtained from the Bryan formula is of interest. For these computations the $P. L.$ and E values obtained from the coupon test specimens were used and the critical plate thicknesses based on these are given in Table 3. The computations by Bryan's formula are for assumed plate widths of 26, 30, and 35 inches. The first is the distance between the two inner rows of rivets connecting flange angles to web plate and was chosen to at least comply with Bryan's assumption that there shall be no lateral shear displacement at the supported plate edge. It also agrees with one of the common methods of design for determining unsupported plate width.¹¹

TABLE 3.—Critical web thickness—Failure by buckling for web plate widths shown

[Based on Bryan's Formula]

Column section number	Properties of material		Critical web thickness in inches for unsupported width (b)		
	Modulus 1,000,000 pounds per square inch (E)	$P. L.$ 1,000 pounds per square inch (S)	26 inches	30 inches	35 inches
G1.....	30.0	47.0	0.491	0.567	0.661
G1A.....	29.9	32.5	.409	.472	.551
G2-4.....	30.2	37.0	.438	.506	.590
G2A-4.....	29.8	40.0	.454	.524	.611
G3-1.....	28.7	37.0	.445	.514	.599
G3A-3.....	29.2	32.0	.410	.474	.553
G4-4.....	28.5	34.0	.430	.496	.579
G4A-4.....	28.0	37.0	.451	.520	.607
G5-1.....	28.7	34.0	.427	.493	.575
G5A-3 ^a					
G6-1.....	29.6	36.5	.436	.503	.587
G6A-1.....	29.3	44.0	.480	.555	.647
G7-1.....	30.2	45.0	.479	.553	.645
G7A-1.....	29.3	39.0	.453	.523	.610
Average.....			$\left\{ \begin{array}{l} .447 \\ \frac{7}{16} \text{ in.} + \end{array} \right.$	$\left\{ \begin{array}{l} .522 \\ \frac{1}{2} \text{ in.} + \end{array} \right.$	$\left\{ \begin{array}{l} .600 \\ \frac{5}{8} \text{ in.} \end{array} \right.$

^a Defective.

¹¹ See Manual, A. R. E. A., p. 751, sec. 65

The test results show, however, that there are distortions in the flange angle legs beyond the first rivet row and actual measurements of the distance between buckles in the test specimens give values of 27 to 35 inches, with 30 inches representing about the mean length. The actual length of buckle, as will appear in a further discussion of Bryan's theory (see p. 764), should be equal to the unsupported width of plate and for that reason the 30-inch width has been included. As some of the thicker web sections show more pronounced flange-angle distortions and longer buckle lengths, the values for the full width of the plate 35 inches (back to back of flange angles) are also added for comparison. For the material used in the webs the average critical plate thickness is 0.44 inch for the 26-inch width, 0.52 inch for the 30-inch width, and 0.60 inch for the 35-inch width.

On the basis of a 30-inch unsupported width of plate a web plate of about $\frac{1}{2}$ -inch thickness would be of critical thickness, and might be expected to fail by buckling from the effects of initial bends tending to produce local eccentricities of loading or, on the other hand, if quite straight might resist a secondary buckling failure and produce a primary failure. Due to the fact that no web plate will be without initial bends it may be expected that a plate slightly thicker than $\frac{1}{2}$ inch would be required to assure against a buckling failure.

The actual agreement of test results with this theoretical value given in Table 3 can be judged by comparison of Table 3 with Figure 18. This figure gives the development of the largest buckle in the web (with applied stress) as determined by contour readings on the longitudinal center line of the web. There is no doubt about the difference in action between the thin webs and thick webs; further, division occurs at webs of $\frac{5}{8}$ -inch thickness, one of which buckled readily and one of which stood up longer. While the latter still gave a buckling failure, comparison with the specimens of $\frac{3}{4}$ -inch web thickness indicates that a web of $\frac{5}{8}$ -inch thickness must be about the critical thickness. Reference to Figure 19, which gives the initial condition of the webs before tests, shows that the failure of the $\frac{5}{8}$ -inch web of specimen G3A was probably accelerated by local eccentricities due to a badly bent plate.

The characteristic difference in failure of the thick and thin webs; that is, those above and below the $\frac{5}{8}$ -inch thickness is also illustrated by the difference in character of final failure, as shown on the contour curves, Figures 20 to 26, and the difference in character of the Hartman-Luder lines, shown in Figures 14 to 17.

The computations for the data presented in Table 3 were made with the belief that when the plate thickness is such that the *P. L.* of the material is reached before a buckling failure occurs the critical

web plate thickness is obtained. That this belief is correct is, perhaps, better shown by using the Bryan formula in the form

$$S = \frac{4.39}{b^2} Et^2$$

in which the symbols have the same significance as given on page 753 with the exception that for the above formula a numerical value of m , Poisson's ratio, of 0.25 has been assumed. This value for m is one that is frequently given and is also about the mean value determined from the telemeter readings on the test specimens under consideration.

The results of the computation for stress at which buckling will occur, as determined by the above formula, are given in Table 4, and, for comparison, the stress at the *P. L.* of the stress-buckle curves, taken from Figure 18, and the *P. L.* of the coupon test specimens of the web plate material are also given.

In Table 4 the values of computed buckling stress below the heavy (zigzag) line are greater than the yield point of the material and are, therefore, obviously to be discarded. They simply show the stress that would cause a buckling failure in the plate of indicated thickness if the material were of a quality that could sustain such stress without developing a primary failure.

It should be noted, first, that for all web thicknesses above the $\frac{5}{8}$ inch the *P. L.* of the stress buckle curves show very close agreement with the *P. L.* of the coupon test of the material. The $\frac{5}{8}$ -inch thickness it has been pointed out in reference to Table 3 and Figure 18 seems to be about the critical thickness. The agreement between the two *P. L.*'s (that is, stress-buckle and material) for the thicker webs confirms the belief that above the critical web thickness primary failures will occur with the development of the *P. L.* of the material. The *P. L.* of the material for each specimen has two values given, the one in parentheses in each case being the *P. L.* of the coupon specimen from the other end of the web plate. The method used in assembling web plates is shown in Figures 7 and 8. These two values show the variation in the material as exhibited by coupon test and by permitting a better judgment of the probable quality of the plate material assist in arriving at the conclusion relative to the relationship between the stress-buckle *P. L.* and the *P. L.* of the material for plates beyond the critical thickness. This comparison suggests also that the early failure of specimen G3A ($\frac{5}{8}$ -inch web) may have been partly due to a weaker material in this plate.

TABLE 4.—Fiber stress at which buckling will occur

[Computed by Bryan formula $S = \frac{4.39 Et^2}{b^2}$ for columns and material of this investigation]

Column No.	Web thickness	Stress at <i>P. L.</i> of stress-buckle curve ¹	<i>P. L.</i> of coupon test specimens	Stress by Bryan's formula at which buckling would occur, based on coupon tests of materials for unsupported width ²		
				26-inch	30-inch	35-inch
	<i>Inches</i>	<i>Lbs./in.²</i>	<i>Lbs./in.²</i>			
G1-----	$\frac{3}{8}$	20,000	47,000	27,400	20,600	14,700
G1A-3-----	$\frac{3}{8}$	18,000	32,500	27,300	20,500	14,600
G2-4-----	$\frac{1}{2}$	21,000	{ (36,500) 37,000 }	49,000	36,800	26,300
G2A-4-----	$\frac{1}{2}$	25,500	{ (44,000) 40,000 }	48,400	36,300	26,000
G3-1-----	$\frac{5}{8}$	35,000	37,000	72,000	54,700	39,000
G3A-3-----	$\frac{5}{8}$	27,000	32,000	74,000	55,600	39,700
G4-4-----	$\frac{3}{4}$	40,000	{ (45,000) 34,000 }	103,000	77,600	55,500
G4A-4-----	$\frac{3}{4}$	38,000	{ (39,000) 37,000 }			
G5-1-----	1	36,000	34,000			
G5A-1-----	1	34,000	31,500			
G6-1-----	$2\frac{1}{2}$	36,000	{ (37,000) 36,500 }			
G6A-1-----	$2\frac{1}{2}$	38,000	{ (40,000) 44,000 }			
G7-1-----	$2\frac{3}{4}$	37,000	{ (34,000) 45,000 }			
G7A-1-----	$2\frac{3}{4}$	41,000	{ (37,000) 39,000 }			

¹ See Fig. 18.² Values below heavy line are above the yield-point stress of the material under consideration.

Further confirmation of the fact that the critical thickness will be obtained when it is such as to develop the *P. L.* of the material is furnished by the following. Specimens G1 and G1A-3 ($\frac{3}{8}$ -inch webs) according to Bryan's theory should have buckled at 20,600 and 20,500 lbs./in.² (on basis of 30-inch unsupported plate width) and the *P. L.* of their stress-buckle curves occurs at 20,000 and 18,000 lbs./in.², respectively. These values are below the *P. L.* of the material—the ultimate buckling is due to initial buckles. Likewise, specimens G2- and G2A- ($\frac{1}{2}$ -inch webs) should have buckled at 36,800 and 36,300 lbs./in.², respectively, and actually gave buckling at 21,000 and 25,500 lbs./in.². A comparison of the type of buckling occurring in G1 and G1-A with that in G2 and G2-A shows that the former buckled quite regularly and uniformly throughout the plate length, while in the latter the buckles were quite definitely localized. This suggests that G2 and G2-A, due to plate eccentricities, probably buckled at a stress somewhat below that at which they should have, a fact that is borne out by a comparison of the buckling stress for specimens G6 and G6-A. The latter were made up from the same plates as G2 and G2-A and buckled at a stress equivalent to the theoretical value obtained by the Bryan formula for the one-half inch plate and at a value closely approximating the *P. L.* of the

material. In other words, the buckling of G2 and G2-A probably should have occurred at about the stress given by the Bryan formula, which is also the *P. L.* of the material, but due to local eccentricities in the plates they buckled below this stress. When the local eccentricities of the plates are neutralized by being grouped as a double web specimen as in G6 and G6-A the material is enabled to develop its true *P. L.*

Likewise, when allowance is made for the fact to be discussed later that the thicker plates seem to require the acceptance of full plate width if Bryan's assumption of no lateral displacement at the plate edge is to occur, specimens G3 and G3-A show good agreement between resistance to buckling and the *P. L.* of the material.

The whole analysis shows first, that reliance can be placed on the validity of the Bryan formula for determining the buckling strength of a plate; secondly, that if the *P. L.* of the material is used in the Bryan formula the plate thickness so obtained will be the critical plate thickness, the plate thickness at which buckling, a secondary failure, might take place, or if the plate is quite free from local eccentricities, at which a primary failure would occur; thirdly, that the plastic flow of material leading to primary failure begins with the *P. L.* of the material in plates thicker than that spoken of as the critical thickness.

A noticeable phenomenon exhibited especially by the webs five-eighth inch and below in thickness is that the final buckles in the thin plates seem to develop as a continuation of the initial bends exhibited by the plates. (See figs. 20 to 26.) The initial corrugations of the web plates might be due to some irregularities of the rolls in rolling. If this were so, the same duplication of corrugations should be found to some extent, at least, on the thick and thin plates, although the magnitude of the effect would be more noticeable on the thinner plates. The evidence on this point does not seem to warrant its acceptance as the cause of the initial corrugation. Further, the agreement of the test results with the mathematical analysis shows a consistency that would not warrant the acceptance of the above cause without much stronger evidence. A suggested cause for the noted condition may lie in the mill phenomena known as "creeping," due to punching, riveting, etc. It appears possible that in forcing the web and angles together the web plates may have been put under an initial compression sufficient to develop incipient buckling, which would naturally predetermine the location of the buckle when developed by reloading. Such incipient buckling would, of course, be developed to a greater extent in the thin plates or in a low strength plate like that of the web of the column G3A—a five-eighth inch web. Possibly these initial buckles may be due to the differential cooling in the plate material after rolling—the earlier cooling and contraction

at the edges of the plates being responsible for the production of the buckles.¹²

Another feature in the behavior of the web which deserves attention is the manner in which strains developed and the location of

FIG. 13.—Column *G*₄ after test, showing strain lines

the first buckle. In all the thin sections the first buckle developed about half a corrugation length from the column end. It is suggested that this is due to the support of the web plate at the ends—a sup-

¹² This explanation has been offered by C. E. Chase, principal assistant engineer, of the Delaware River Bridge Joint Commission.

port brought about by the additional pieces (filler plates and bearing angles) riveted to the webs at the ends. The effect of such extra support would be to prevent natural deformation of the web and compel the web to fail at the ends by the direct compression stresses

FIG. 14.—Column G5 after test, showing strain lines

or the simultaneous shear stresses along diagonal lines approximating a 45° angle, the plane of maximum shearing stress. Proof of this seems to be given by the development of the first Hartman-Luder lines at the ends, as shown by Figures 14 to 17. While the

tendency to produce the buckle seems to persist, the shear lines become predominant and mask the buckle as the increasing stresses are developed. The comparison of the various figures showing the development of the Hartman-Luder lines shows the ascendancy of

FIG. 15.—Column G6A after test, showing strain lines

the shear failure at the ends of the thin webs and in the thicker webs, while buckle failure develops normally throughout the unsupported center section of the thinner plates.

Bryan's analysis is of further direct interest, in that it points out some very important considerations relative to column or tubular girder design. To begin with, the number of corrugations in a plate will depend upon the length of the plate relative to its width—"the

FIG. 16.—Column G7 after test, showing strain lines

number must always be such that the ratio of the longer to the shorter side of the rectangles (of a corrugation) differ as little as possible from unity; that is, their shape must be as nearly square as possible."

The nodular lengths of the buckle of the web plates of these columns, as may be seen in Figures 12 and 13, comply very nicely with this requirement, as is indicated by the circular conformation of the strain lines between buckles. This points to the fact that the

FIG. 17.—Column G7A after test, showing strain lines

unsupported plate width must be somewhat more than the clear distance between flange angle toes; in fact, taken at least equal to the distance between the inner rivet gauge lines on the angles—about 26 inches in this case. It would seem that the breadth should

Fig. 18.—Buckle deformation of critical section of webs
(Contour intervals indicated in parentheses. See also figs. 20 to 26)

be more than the clear distance if Bryan's condition of no lateral displacement at the edge is to be fulfilled. Such a requirement would necessitate stiffening support from the flange angles and this would not be assured, and the contour readings on the angles indicate this (figs. 20 to 26) until at least one line of rivets was passed. The unriveted extension of the leg of the angle beyond the first or in-

Fig. 19.—Initial web plate contours
(All deformations plotted with reference to a base on the north side. Location of final failure indicated by "X"—in thin webs "X" is put on deepest buckle)

ner rivet line would afford quite a perceptible freedom for lateral displacement.

In common design practice as permitted, for example, by the general specifications for steel railway bridges of the A. R. E. A.¹³ the unsupported plate width is taken as "one-thirtieth of the dis-

¹³ See Manuel A. R. E. A., p. 751, sec. 65.

tance between lines of rivets connecting it to flange angles" for webs of compression members, and for cover plates as "not less than one-fortieth of the distance between the nearest rivet lines." On the basis of these test specimens, the distance between rivet lines is 26 inches. Under these rules a $\frac{7}{8}$ -inch plate would be required for web plates and a $\frac{5}{8}$ -inch plate for cover plates. Obviously, the $\frac{7}{8}$ -inch plate is conservative as judged by these tests. The $\frac{5}{8}$ -inch plate, perhaps is at the critical thickness and on account of local

FIG. 20.—Web deformation; contour measurements, column G1A

eccentricities might better be made a $\frac{3}{4}$ -inch plate if primary failure is to be assured. In other words, a plate thickness of one-thirty-fifth of the distance between rivet lines would be permissible.

There are indications, however, that the unsupported width of plate should, perhaps, at certain times be taken as the full width of the plate to its transverse connections (especially as Bryan's formula presupposes a perfectly plane plate, a condition that can not exist practically). Table 4 shows that for the 35-inch width (full width

FIG. 21.—Web deformation, contour measurements, column G2

FIG. 22.—Web deformation, contour measurements, column G3

FIG. 23.—Web deformation, contour measurements, column G4

FIG. 24.—Web deformation, contour measurements, column G5A

of plate) the critical thickness averages about five-eighths inch, the thickness at which there is a distinct change in the character of stress-buckle curves in Figure 18. Further the photograph of "column G4 after test" (fig. 13) which is one with a $\frac{3}{4}$ -inch plate indicates that while the tendency to fail by buckling is practically eliminated, it nevertheless persists enough to show its effects. In this particular column the length of the incipient buckle was almost exactly the full width of the section (35 inches), and this suggests

FIG. 25.—Web deformation, contour measurements, column G6A

that for thick web plates, that is, thick in comparison with the members connecting them to stiffening members their full width should preferably be used as the unsupported width. This last seems reasonable when it is considered that the restraining action of the flange angles would be far more effective on thin webs than on thick ones, and that for thick plates the fulfillment of Bryan's condition of no lateral displacement would require taking the full width of the plate to its lateral connections.

If then, the results of these tests indicate that for the thicker plates the full width of plate should, perhaps, be used, they show that a plate whose thickness is not less than one-forty-sixth of its width ($\frac{3/4}{35} = \frac{1}{46}$) would be safe against buckling if proper care is taken in the selection of plate material. The present commonly used rule (Fairbairn's) that the plate is safe against buckling if its thickness is not

FIG. 26.—Web deformation, contour measurements, column G7A

less than one-thirtieth of its width with the unsupported width properly chosen, is therefore conservative.

From theoretical considerations alone, assuming a perfectly plane plate the ideal condition, a mild steel, with a *P. L.* of 30,000 lbs./in.², a modulus of 30,000,000 lbs./in.², and a Poisson's ratio of 0.25, would give a critical buckling thickness of one sixty-sixth of the unsupported width, while a medium steel, with a *P. L.* of

38,000 lbs./in.² and a modulus of 30,000,000 lbs./in.² and a similar Poisson's ratio, would give a critical thickness of one fifty-ninth of the unsupported width. (See fig. 27.) Obviously, as the results of these tests show, it is not safe to assume the ideal plate and it is therefore suggested that the thickness of a plate which is to be considered safe against a buckling failure should not be less than one forty-fifth of the unsupported width, if the width is taken as the full width of the plate, with care exercised in the selection of the plate material.

In many large-sized columns and tubular girders, cross-stiffening members, diaphragms, are added to increase the stiffness of the built-up member and make it act as a unit. The value of diaphragms

FIG. 27.—Critical buckling thickness for an ideal plate, based on Bryan's formula

has been under dispute, principally as to their placement and alignment; that is, how far apart they should be, if placed crosswise, or whether they should be placed lengthwise of the section.

Bryan's discussion, while dealing with ship plating, applies equally as well to the use of diaphragms. He says:

In order to obtain the greatest strength the ribs must be placed parallel with the greatest thrust, and that to obtain the same strength they must be placed twice as far apart as if they were perpendicular to the thrust.

The strength of the plate will not then be increased or altered in any way by the addition also of a second set of ribs perpendicular to the first and at distances apart equal to the natural length of the corrugations.

Thus, if there is no lateral thrust, a plate supported on parallel longitudinal ribs will not be strengthened by the addition of transverse ribs which divide it into squares.

If the transverse ribs are at any other distance apart, however, the system will be strengthened. In this particular case the most effective distance between the transverse ribs will be $b\sqrt{2}$. The same increase in the strength could be obtained much more economically by diminishing the distance between the (*longitudinal*) ribs from b to $\frac{2}{3}\sqrt{2}b$; that is, to $\sqrt[3]{\frac{2}{9}}b = 0.9428b$.

When, however, the structure is subjected to a thrust of about equal intensity in all directions, the advantage of a network of ribs will be much greater; moreover, the framework itself must be strengthened—a consideration not brought out in our present theoretical treatment, in which the framework is supposedly perfectly rigid.

Considering the diaphragms as stiffeners for web plates (and not as ties to stiffen the framework as a whole) Bryan's analysis shows that the longitudinal diaphragm is more effective and that transverse diaphragms if placed at distances apart equal to the effective width of the plate or any multiple of that value are useless; that if transverse diaphragms are used they are most effective when placed at a distance apart one and four-tenths times the effective width of the plate; and that a more economical design would be secured if the transverse diaphragms were omitted and the web plate decreased 6 per cent in width for the same thickness and material. A recent review of some tests of 12 large columns¹⁴ in which the value of diaphragms is considered will give an interesting comparison of the correctness of the above statements. The diaphragms in these columns appear to have been spaced at approximately multiple lengths of the natural corrugation length and showed no advantage in test strength by their addition.

XIII. SINGLE PLATE VERSUS DOUBLE PLATE WEBS

In the investigation there were two sets of double web columns whose webs were stitch-riveted as shown in Figure 8.

In one set (G6 and G6A) the web was made up of two $\frac{1}{2}$ -inch plates, giving a total thickness of 1 inch. The action of these two columns can be directly compared, therefore, with the columns having a single web of 1-inch thickness (G5 and G5A). The remaining set with webs of two $\frac{3}{4}$ -inch plates (G7 and G7A) can only be compared indirectly.

A study of the stress-buckle curves (fig. 18) shows that on the whole the thicker webs developed less buckling, up to the stress at failure, and that the columns with double webs were better in this respect than those with solid webs. It is very probable the increased stiffness of the double webs may be partly due to the compensating or neutralizing action of the initial bends in the two plates (see G6 and G6A, fig. 19), the combined action being more nearly similar to an unbent plate, but the increased stiffness of the 1-inch double

¹⁴ Eng. News-Record, 88, No. 21, May 25, p. 877; 1922. Fr. Voss, Prüfung von Druckstaben für Brücken des Kaiser-Wilhelms-Kanals. Der Bauingenieur, 3, No. 1, pp. 8-11; Jan. 15, 1922.

plate web over the 1-inch solid web is also due in part to the fact that the mechanical properties of the thick solid plate are inferior to those of the thinner plate (see G5-1 and G6-1, Table 5), so that in this case the solid web was inferior both as to initial curvature and material.

TABLE 5.—*Relation of mechanical properties of web plates to plate thickness*

Specimen No.	Plate thickness	Modulus <i>E</i> , 1,000,000 pounds	<i>P. L.</i> 1,000 pounds	<i>Y. P.</i> 1,000 pounds	Ultimate tensile strength, 1,000 pounds	Elonga- tion in 8 inches	Reduc- tion of area
	<i>Inch</i>	<i>Square inch</i>	<i>Square inch</i>	<i>Square inch</i>	<i>Square inch</i>	<i>Per cent</i>	<i>Per cent</i>
G5-1.....	1	28.7	34.0	43.0	85.6	21	37
G5A-3 ¹							
Average.....		28.7					
G7-1.....	$\frac{3}{4}$	30.2	45.0	51.0	92.5	19	37
G4-4.....	$\frac{3}{4}$	28.3	34.0	45.6	85.8	25	44
G7A-1.....	$\frac{3}{4}$	29.3	39.0	46.5	86.8	20	31
G4A-4.....	$\frac{3}{4}$	28.0	37.0	43.5	81.2	25	48
Average.....		28.9					
G3-1.....	$\frac{5}{8}$	28.7	37.2	45.0	81.1	24	51
G3A-3.....	$\frac{5}{8}$	29.2	32.0	44.0	83.4	25	44
Average.....		28.9					
G6-1.....	$\frac{1}{2}$	29.6	36.5	46.5	79.0	25	52
G4-4.....	$\frac{1}{2}$	30.2	37.0	46.9	80.0	24	43
G6A-1.....	$\frac{1}{2}$	29.3	44.0	47.4	78.6	24	51
G4A-4.....	$\frac{1}{2}$	29.8	40.0	47.0	78.7	24	57
Average.....		29.7					
G1-1.....	$\frac{3}{8}$	30.0	47.0	50.6	81.6	26	51
G1A-3.....	$\frac{3}{8}$	29.9	32.5	48.5	83.5	24	55
Average.....		30.0					

¹ Defective material.

Aside from these differences of material and effective straightness there is no indication that there is any advantage in either type of web.

A web made up of two similar plates should, therefore, be just twice as strong as a single web if both are free from initial bends. If the thickness of each plate is greater than the critical buckling thickness, stitch-riveting on such members might not be necessary, since the stitch rivets would be unstressed until failure occurred. On the other hand, if buckling occurs, these rivets near the bend would be brought into tension and shear in forcing the two plates to act as a unit. Both conditions were observed in the tests. One set of measured rivet strains was taken well away from the buckle that developed at failure (fig. 15), and throughout the whole test there was no indication of increased strain on the rivet shank. In another column the rivets on which the strains were measured were in the field of the failing buckle, and in this case, while there was only a

slight strain developed in the early stages of the test it grew steadily and at an increasing rate, showing that the rivet was being stressed through the interaction of the two webs in the development of a buckle.

Practically, however, all web plates will show irregularities in alignment which will tend to cause weakness in the plate through the development of bending stresses. With such conditions, stitch riveting a double plate web is decidedly advisable¹⁵ for in causing the two plates to act as a unit and thus making them mutually supporting the stitch riveting will tend to neutralize the independent action of the individual plates and stiffen their action as a whole. It is not known at present how far apart such riveting may be to insure mutual support of the plates; as far as these tests indicate, the riveting used was sufficient. A study of the relative action of the double webs under load shows no tendency for these to separate or act independently.

One phenomenon noted in these tests that suggests the need of further consideration relative to stitch riveting is shown very strikingly on the photograph of G7A north side after test (fig. 17) and G7 south side after test (fig. 16). Where stitch rivets lie on a diagonal line of proper slope with the axis of the piece the stress lines pass continuously through all of them, indicating that, perhaps, a plane of somewhat less strength is developed by the multiplicity of rivet holes. Where rivet holes are not so aligned the stress lines seem, in general, to "run out" after progressing a short distance from the holes. This phenomenon suggests that the effect of rivet holes may be minimized by proper alignment and staggering.

XIV. STRAIN GAUGE AND DEFLECTION READINGS

In the previous analysis very little weight has been given to the strain gauge or deflection observations.

Examination of the data furnished by the strain gauges, which were used to indicate strains on the webs of the columns, showed that for columns whose web thickness was three-fourths inch or 1 inch a fairly close agreement existed between the stresses computed from the indicated strains and those computed from the applied load. For columns whose web thicknesses were less than three-fourths inch, the two were not in agreement.

¹⁵ It perhaps is desirable for the present at least to qualify this statement by saying that stitch riveting is advisable provided it does not in itself cause detrimental conditions by the development of overstress around the holes. The effect of such riveting needs investigation. The early development of the Hartman-Luder lines about stitch rivets and a possible slight tendency (see G5, G5A, G6, and G6A, fig. 18) of the double web plates to fail at a lower stress suggests the importance of such an investigation. G7 and G7A, figs. 16 and 17, of the Hartman-Luder lines of the columns with double webs show the decided influence of holes in causing early overstress in the vicinity of the holes, and holes as small as 0.2 inch in diameter show decided effects.

In examining the irregularities of the results of the gauges a study was made of the relative position of the strain gauges and the buckles in the web. This examination led to an explanation which met all conditions and which showed the inadvisability of giving weight to the strain-gauge readings. It was found that the strain-gauge readings were seriously affected by the web buckles and by the initial irregularities of the web. The greater the radius of curvature of such buckles the less was the discrepancy between computed and applied stresses.

In the case of nearly every gauge the indicated compressive deformation was in excess of what might be expected for the given applied load.

Remembering that the gauges are capable of measuring only the chord distance of the buckle, consideration of the conditions developed make it quite evident that even if the strain gauge is located on the convex side of the buckle the instrument will indicate chord shortening and tensile deformation. The tendency of the webs to bend abruptly upon themselves was especially true of the thinner sections. In these cases a portion of the web would be stressed very highly over a short length at the bend and the extreme fiber upon which the gauge is located would be elongated due to the tensile stresses produced by the bending, but this deformation would be seriously affected in the opposite direction by the chord shortening.

So far it has been assumed that the strain gauge was located directly over the buckle. In this position the effect of the chord shortening would be a maximum. In the condition where only a portion of the buckle was included in the gauge length, the error due to the chord shortening would be less.

For some of the gauges, however, the indicated strains were less than might be expected from the load. This condition is explained by the fact that these gauges were located on the convex side of buckles and that the increase of gauge length due to tensile stresses produced by bending partially neutralized the deformations produced by loading and chord shortening. The gauges thus affected were found only on girders whose web thickness was three-eighths inch.

Theoretically the strain-gauge readings could be corrected from a knowledge of the curvature of the web, but the contour measurements were not sufficiently detailed to make a determination of the radius of curvature with the required accuracy, so that it has not been thought advisable to incorporate the strain-gauge data in the report.

These inaccuracies in strain-gauge data prevented a determination of the variation in stress along the length of the web, the indicating instruments having been influenced to such an extent by buckling of the web that no definite conclusions could be drawn.

It is obvious that where local bending is appreciable caution should always be used in estimating stresses from strain-gauge readings. If it is desired to estimate the effect of bending, very careful measurements of the radius of curvature would be necessary.

The gauges which were placed perpendicularly to each other in order to determine the Poisson ratio were influenced in the case of the thin webbed girders by buckles, but in the case of girders whose webs were 1 inch thick the values of this ratio were for G5A, 0.278, and for G5, 0.215.

Likewise, lateral deflection readings have been omitted. Reference to Figure 9 will show that all lateral deflection readings were taken on the stiffener channels. The indicated deflections are quite variable, and apparently follow no definite order. The deflections were gradual at first, later showing pronounced changes in value but giving no definite indication except that possibly there was a tendency for the flange angles to change the included angle between the flange-angle legs. This latter indication is erratic, and probably is due to the effect of web buckling. The data available warrant no conclusions based on lateral deflection readings.

XV. EFFECT OF REPEATED LOADING

As previously described (p. 748), the load applied on the column was reduced to the initial load after the initial *P. L.* determined by the 60-inch compressometers had been passed. It has been found impossible to arrive at any decision relative to the effect of such reloading on the apparent modulus of elasticity of the columns. The mechanical lost motion of the indicating devices on the release of load is sufficient to throw doubt upon the correctness of any conclusions arrived at. The only statement, therefore, that can be made relative to the effect of reloading is that in conformity with previous test results it seems to "iron out" the initial irregularities and raise the proportional limit on the second loading to the stress from which the load was previously released.

XVI. ULTIMATE STRENGTH OF THE Laterally SUPPORTED COLUMNS

The average of the readings of the four 60-inch compressometers used on each column are plotted in Figure 28.

The loading was continued until either the load reached a maximum or until the bolts holding the clips of the stiffeners failed. The failing stresses are tabulated in Table 6, together with the weighted average *P. L.* and *Y. P.* of the material for comparison. The weighted average *P. L.* and *Y. P.* was computed on the basis of the areas of the individual pieces composing the column.

Fig. 28

FIG. 28.—Average compression in 60 inches, for each column

In interpreting these failing stresses, the type of support afforded by the stiffeners must be considered. These stiffening channels were designed to simulate the lateral support the column would receive as a member of the bridge tower. (See fig. 10.) The method of application is shown in Figure 9 and described on page 744. They offered a support against the lateral deflection of the column as a whole, without carrying any appreciable portion of the axial load. Strain-gauge readings taken on the stiffeners showed no noticeable deformation until the *P. L.* of the column material was reached. From then on the buckling and deformation of the column brought transverse loads of considerable magnitude on the stiffener channels. The erratic reading of the gauges on the stiffeners, indicating sudden increase or decrease of stress, showed that the conditions under which the stiffeners were acting were irregular, as might be expected with the column buckling under changing eccentricities and restraint. In none of the tests was there any permanent deformation of the stiffeners themselves and only little evidence of lateral slippage of the clips or bolts. The failure of the lateral support where it occurred was due to shearing of the bolts holding the clips of the stiffeners. The stiffeners restrained the final failing action as well and as long as their fastenings were strong enough to do so.

TABLE 6.—*Ultimate strength of laterally supported columns*

Column No.	Failing stress	Material weighted average	
		<i>P. L.</i>	<i>Y. P.</i>
	<i>Lbs./in. ²</i>	<i>Lbs./in. ²</i>	<i>Lbs./in. ²</i>
G-1.....	46,000	38,700	50,400
G-1A.....	44,000	36,800	45,200
G-2.....	44,700	39,800	49,300
G-2A.....	44,200	42,000	48,500
G-3.....	44,500	41,900	48,100
G-3A.....	41,000	33,700	48,400
G-4.....	46,000	39,200	48,500
G-4A.....	44,000	40,100	47,800
G-5.....	46,000	38,300	46,200
G-5A.....	47,800	41,900	46,900
G-6.....	42,200	39,800	48,500
G-6A.....	43,800	43,700	48,100
G-7.....	45,900	45,500	50,700
G-7A.....	46,400	41,800	47,800

The construction and application of these stiffeners was such that a fairly uniform strength might be expected of them in action, and all columns then would show the same ultimate stress irrespective of any variations in material. Figure 28 shows the average stress-strain relation from the column compressometers, plotted without corrections of any sort up to what was considered complete failure. The uniformity of the failing stress for all columns irrespective of web thickness or total area, and the fact that it bears no direct relation

to either the *P. L.* or *Y. P.* stress of the material, the further fact that all columns failed about the axis parallel to the webs, and finally the fact that for some of the columns the loading was carried on till the bolts holding the clips of the stiffeners to the deformed column were sheared under decreasing loads with no sign of failure in the stiffeners themselves, indicates that the restraining action of the stiffeners came into play at the critical load on the unsupported web and by keeping the column in line and thus restraining the tendency to increased deflection enabled the *combination* to take additional load until the strength of the supporting element was exceeded.

Column G3A furnishes an interesting example of the effect. This column was the first one tested. There was considerable doubt as to just how tight the stiffeners should be bolted to the column section—it was not desired to have them show a development of stress—and consequently the clip bolts were not drawn to full bearing or shimmed for full contact as was done with all other specimens. Light showed between the stiffeners and specimen flanges in a number of places. The fastening of the stiffeners was, therefore, weaker and showed this distinctly in the ultimate load carried by the combination. This last indicated small likelihood of the stiffeners developing stress and all later tests were made with stiffeners shimmed for tight bearings and the bolts set with a two-foot wrench.¹⁶

The stress at failure as indicated by Figure 28 can, therefore, be considered as the strength of the combination of pieces (column plus stiffeners). How much of such increase in strength may be advantageously used in the tower construction will depend entirely in the judgment of the designer relative to the similarity of action between the unstressed stiffeners as used in test and the stressed supporting members of the tower design. In this connection, the effect of distortion of the supporting member under load upon the section it is supporting that is, the mutual interaction of one piece on another—must be given consideration.

It seems probable, that, with the tower section as built with horizontal stiffening diaphragms or girders, a strength at least equivalent to that obtained by the stiffened test specimen can be counted upon, especially as any local weakness of one section would cause a redistribution of total load, and thus relieve a possibly overstrained member.

XVII. ACKNOWLEDGMENT

The writer wishes to acknowledge here his indebtedness to Leon Moisseiff, C. E. Chase, and A. B. Dana, of the Delaware River Bridge Joint Commission, for their helpful criticisms and suggestions in the development of this investigation.

¹⁶ This explains the apparent discrepancy noted in Engineer News Record, 89, No. 23, p. 986; 1923.

XVIII. CONCLUSIONS

1. WEB PLATE COMPRESSION STRENGTH

1. A web plate in compression may fail through the development of a secondary failure by buckling, or by a primary failure.

2. The type of failure to be expected depends on the dimensions, support, and condition of the plate and the mechanical properties of the material.

3. The results of this investigation show that computed values from the theoretical formula developed by Bryan are in close agreement with best results, provided proper values are chosen for the Bryan formula.

4. The unsupported width must be so chosen as to insure freedom from lateral displacement at the plate boundaries and should, at least, be equal to the distance between connecting rivet lines, and preferably, in the case of thick plates to the full width of the plate.

5. The critical plate thickness—that is, the thickness at which either a secondary failure by buckling or a primary failure may take place—may be determined by substituting the value of the $P. L.$ of the material for the stress at which buckling will occur in the Bryan formula.

6. For thin material in which the buckling stress by Bryan's formula lies below the $P. L.$ of the material failure by buckling will occur.

7. For thick material in which the buckling stress by Bryan's formula lies above the $P. L.$ of the material failure by buckling will not occur, a primary failure will take place.

8. On the basis of Bryan's formula it appears possible to make safe and more economical designs by computing the web plate thickness for a selected high-grade material, provided due care is taken to insure the plate material being free from excessive local curvatures. To provide for the latter, it may be necessary to increase the computed thickness arbitrarily so that for general design work an approximate rule like Fairbairn's may prove the more useful.

9. The Fairbairn rule limiting web thickness to one-thirtieth of the unsupported width is conservative for structural steels, provided the plate width is properly chosen. With care exercised in the selection of the plate material, the unsupported plate width may probably be extended to forty-five times the plate thickness, if the width is taken as the full width of the plate, or to thirty-five times the plate thickness if the width is taken as the distance between rivet lines, without danger of developing a secondary or buckling failure. For plate material five-eighths inch in thickness or over the unsupported plate width probably should be taken as the full plate width, especially if relatively thin flange angles are used.

To these conclusions may be added the following statements based on Bryan's theory relative to methods of stiffening wide web plates against buckling:

10. The strongest and most economical stiffening may be secured (a) by placing the stiffening element parallel to the direction of thrust, or (b) by decreasing web plate width about 6 per cent rather than by incorporating transverse stiffening frames.

11. If transverse frames are used they (a) should not be placed at distances equal to the width of the plate or multiples thereof, (b) but should preferably be spaced one and four-tenths times the width of the plate, proper allowance being made for increased end stiffness of a plate due to filler plates, connections, etc.

2. SINGLE WEB VERSUS DOUBLE WEB

1. Provided the thickness of each plate is beyond the critical buckling thickness, the plates are equally straight and the material in the webs is alike, a double web column section should prove as efficient as a single web of equal thickness. Stitch riveting of such webs would, therefore, be unnecessary.

2. If there are inequalities in the alignment or in the material of the two plates of the double plate web, secondary effects will result which will make stitch riveting necessary in order to assure cooperative action of the two plates.

3. The stitch riveting used in these columns was sufficient to develop the strength of the web material, with no noticeable detrimental effects.

4. It is possible, as is indicated by the early development of stress lines about the rivets joining the webs, that riveting under heavy pressures and improper stagger of holes may be detrimental to the strength of double web sections.

3. INTERPRETATION OF STRAIN-GAUGE READINGS

1. Where local bending is appreciable, caution must be used in estimating stresses from strain-gauge readings.

2. For reliable results, accurate measurements of the radius of curvature should accompany the strain-gauge readings.

4. STRENGTH OF Laterally SUPPORTED Wide-Web COLUMNS

1. The strength of laterally supported wide-web columns depends upon the strength of their connections to and the relative stiffness of the supporting member.

2. The support furnished the columns of this investigation was sufficient to develop the strength expected as laterally supported columns before the strength of the connections to the stiffening element was exceeded.

WASHINGTON, December 21, 1925.

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