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BRIDGE SPECIFICATION MATERIAL REQUIREMENTS

February 1939

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BRIDGE SPECIFICATION MATERIAL REQUIREMENTS

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Bridge Specification

Material Requirements

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Summary of Miscellaneous Specifications for Steel

Year	Name	Wrought Iron: Tensile strength lbs. per sq. in.	Wrought Iron: Elastic limit lbs. per sq. in.	Steel: Tensile strength lbs. per sq. in. Soft	Steel: Tensile strength lbs. per sq. in. Medium	Steel: Tensile strength lbs. per sq. in. Rivet	Steel: Elastic limit lbs. per sq. in. Soft	Steel: Elastic limit lbs. per sq. in. Medium	Steel: Elastic limit lbs. per sq. in. Rivet	Full-Sized Eye-Bars: Tensile strength lbs. per sq. in.	Full-Sized Eye-Bars: Elastic limit lbs. per sq. in.	
1883	Kentucky and Indiana Bridge	"Specifications for iron and timber are as usual in first-class work"		70,000 min. (2)	80,000 min. (1)	58,000 65,000	40,000 min.	50,000 min.	Not given	65,000 min.	35,000 min.	1/
1887	Keystone Bridge Co.	Bars in tension: 52,000 - $\frac{5000 \times \text{area orig. bar}}{\text{Circ. orig. bar}}$ Shapes - 48,000 Plates < 24" - 48,000 Plates > 24" - 48" - 46,000	 26,000 26,000 26,000 25,000	60,000 68,000 (3)	76,000 84,000 (1) 68,000 76,000 (2)	55,000 63,000	35,000	45,000 40,000	33,000	Reduction per sq. in. of (500 x width of bar) lbs.	Not given	2/
1888	Phoenix Bridge Co.	Tension members: 52,000 - $\frac{7000 \times \text{area orig. bar}}{\text{Circ. orig. bar}}$ Plates < 24" - 48,000 Plates > 24" - 46,000 Shapes: 50,000 - $(\frac{7000A}{C})$	 1/2 T.S. min. 26,000 26,000 1/2 T.S. min.	62,500 (2) ±4,000	68,000 (1) ±4,000	60,000 ±4,000	1/2 T.S. min.	1/2 T.S. min.	1/2 T.S. min.	Reduction per sq. in. of (500 x width of bar) lbs.	Not given	3/
1891	Reading Term. Philadelphia	Tension members: 52,000 min. Other iron: 48,000 min. Plates > 24": 46,000 min.	26,000 min. 26,000 min. 26,000 min.		62,000 68,000			1/2 T.S. min.		No tests specified		4/
1894	F. H. Lewis	Tension members: 50,000 min. Shapes - 48,000 min. Plates < 36" - 48,000 min Plates > 36" - 46,000 min	26,000 min. 26,000 26,000 25,000 min.	54,000 62,000	60,000 70,000	Rivets to be soft steel	32,000 min.	35,000 min.		56,000 min.	Not specified	5/
1896	Theodore Cooper			54,000 62,000	60,000 68,000	50,000 58,000	1/2 T.S.	1/2 T.S.	Not specified. Elong. = 26%	56,000 min.	Not specified	6/
1896	Ass'n. Amer. Steel Manuf.			52,000 62,000	60,000 70,000	48,000 58,000	1/2 T.S.	1/2 T.S.	1/2 T.S.			7/
1898	Pencoyd Iron Works			50,000 60,000	60,000 70,000	Rivets to be soft steel	1/2 T.S.	1/2 T.S.		No tests specified		8/
1897	Pennsylvania Lines West of Pittsburgh	Full-size tests: 52,000 - $(7,000 \frac{A}{P})$ Shapes - 50,000 Plates 12" - 24" - 49,000 Plates 24" - 36" - 48,000 Plates 36" - 54" - 46,000 Plates 54" - 45,000	26,000 min. 26,000 26,000 26,000 26,000 26,000 26,000	54,000 62,000	62,000 70,000	50,000 58,000	55% T.S.	55% T.S.	55% T.S.	Med. steel: 62,000 - $(9,000 \frac{A}{P})$ Soft steel: 54,000 - $(8,000 \frac{A}{P})$	55% T.S. 55% T.S.	9/

L-5301

See Sheet L-5302 for Cincinnati So. Rwy. 1888, and for footnotes

Proc. Assoc. of Engr. Soc., Kansas City, 1888, Mr. G. Bouscaren. Cincinnati Southern Rwy. Specs. for Wrought Iron. Tensile strength 50,000 lbs. per sq. inch. Elastic Limit 25,000 lbs. per sq. inch.

1/ (1) For compression members. (2) For tension members.
Source: The Kentucky and Indiana Bridge - Mace Moulton - Trans. A.S.C.E. Vol. 17, 1887.

2/ (1) For compression members. (2) Tension members and built beams. (3) Rolled beams. Wrought iron values are minimum.
Source: U.S. B.P.R. Sheets L-896, L-897. Copy of Keystone Bridge Co. Specs. known to have been in use in 1887.

3/ (1) Compression members. (2) Tension members. Wrought iron values are minimum.
Source: U.S. B.P.R. Sheets L-883 to L-887, inc. Copy of Std. Specs. of Phoenix Bridge Co. for Rwy. and Hwy. Bridges and Viaducts, 1888.

4/ Steel used only in bottom chords of Spring Garden St. Bridge.
Source: The Philadelphia and Reading Terminal Railroad and Station in Philadelphia. Jos. M. Wilson, Trans. A.S.M.E. Vol. 34, 1895.

5/ All eye-bars to be medium steel.
Source: Theory and Practice of Modern Framed Structures, Johnson, Bryan and Turneaure, Second Edition, Revised 1894.

6/ Steel specified for all members.
Source: General Specs. for Steel Highway Bridges and Viaducts, by Theo. Cooper, 1896.

8/ Steel specified for all members.
Source: Steel in Construction, The Pencoyd Iron Works, Eleventh Edition, 1898, C. C. Schneider, Chief Engr.

7/ Adopted October 23, 1896.
Source: The Manufacturers' Standard Specs. Albert Ladd Colby. Proc. A.S.T.M. Vol. 5, 1903.

9/ Material to be wrought iron, except that O.H. steel may be used for eye-bars, pins, rollers and pedestals.
Source: Pennsylvania Lines West of Pittsburgh. Specs. for Rwy. Bridges, 1897.

$\frac{A}{P} = \frac{\text{Area}}{\text{Perimeter}}$

U. S. Department of Agriculture
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Summary of Miscellaneous Specifications for Steel

Date	Name	Tensile Strength lbs. per sq. in.			Elastic Limit lbs. per sq. in.			Full-Sized Eye-Bars		Remarks
		Soft (structural)	Medium	Rivet	Soft	Medium	Rivet	Tensile strength lbs. per sq. in.	Elastic limit lbs. per sq. in.	
1901	Pennsylvania Railroad Co.	52,000 62,000	62,000 70,000	48,000 56,000	28,000	33,000	28,000	48,000	27,000	Medium steel for pins, lateral bolts and ex. rollers. Soft steel for all other parts. 1/
1902	Boller & Hodge	60,000 (1) 70,000	63,000 (2) 73,000	52,000 63,000	50% T.S.	50% T.S.	55% T.S.			(1) Angles and plates. (2) Eye-Bars. 2/
1903	Ass'n. American Steel Manuf.	55,000 65,000	60,000 70,000	48,000 58,000	1/2 T.S.	1/2 T.S.	1/2 T.S.			Adopted Feb. 6, 1903. Also given in Phoenix Iron Co. handbook of 1906 with no changes 3/
1903	American Rwy. Eng. and Maint. of Way Ass'n.	55,000 (1) 65,000		45,000 55,000	Not specified. Yield point to be recorded		Not specified. Yield point to be recorded			(1) Called "Structural Steel". Single grade specified. Adopted March 1903. 4/
1906	Pennsylvania Railroad Co.	55,000 65,000	60,000 70,000	48,000 56,000	28,000	33,000	28,000	Not	given	Medium steel for pins, lateral bolts and rollers. Soft steel for all other parts. 5/
1906	Pennsylvania Lines West of Pittsburgh	54,000 62,000	62,000 70,000	50,000 58,000	55% T.S.	55% T.S.	55% T.S.	Medium steel 62,000 - 9,000 Area/Per. Soft steel 54,000 - 8,000 Area/Per.	55% T.S.	6/
1908	Milo S. Ketchum	55,000 65,000		45,000 55,000	Not specified. Yield point to be recorded		Not specified. Yield point to be recorded	Not specified	Not specified	Material Specs. taken from A.R.E.& M. of W. Assn. Specs. given above, with slight modifications. 7/
1909	Theodore Cooper	55,000 65,000	60,000 (1) 70,000	48,000 58,000	1/2 T.S.	1/2 T.S.	1/2 T.S.	56,000	Not specified	(1) For eye-bars, not over 20% outside 62,000-68,000. 8/
1910	New York Central Lines	56,000 (1) 64,000		46,000 54,000	33,000		28,000	54,000	29,000	(1) 68,000 max. for eye-bar material 9/
1911	Lehigh Valley R.R. Co.	56,000 (1) 66,000		46,000 56,000	33,000		28,000	Not specified	Not specified	(1) 68,000 max. for eye-bar material 10/
1913	Office of Public Roads	60,000 (1) 70,000		48,000 58,000	1/2 T.S.		1/2 T.S.	Not specified	Not specified	(1) For eye-bars, not over 20% outside 62,000-68,000 11/
1915	Chicago & Northwestern Rwy. Co.	55,000 65,000		48,000 58,000	1/2 T.S.		1/2 T.S.	54,000	29,000	12/
1934	Seaboard Air-line Railway	56,000 (1) 71,000		46,000 (2) 61,000	Desired 30,000		Desired 25,000	Not specified	Not specified	(1) 60,000 desired, 55,000-72,000 allowed on re-test. (2) 50,000 desired, 45,000-62,000 allowed on re-test. 13/
1901	American Bridge Company	52,000 62,000	60,000 70,000	48,000 58,000	1/2 T.S.	1/2 T.S.	1/2 T.S.	5,000 lb. reduction	Not specified	14/

See Sheet L-5304 for footnotes

NOTES:

The following misc. specs. are identical with the material specs. of the A.S.T.M. for the dates indicated:

Southern Rwy. Co. - Gen. Specs. for Steel Structures - 1911. A.S.T.M. 1909.

Baltimore and Ohio R. R. System - Gen. Specs. for Steel Rwy. Bridges - 1922. A.S.T.M. A7-16. (Except yield pt. - 30,000 lbs. per sq. in. for Struc., 25,000 lbs. per sq. in. for Riv.)

Northern Pac. Rwy. Co. - Spec. E-112 for Design, Material and Workmanship of Steel Rwy. Bridges - 1925. A.S.T.M. A7-24

Union Pac. System - Gen. Specs. for Steel Rwy. Bridges - 1926. A.S.T.M. A7-24.

Source:

1/ Penn. R. R. Co. Std. Specs. for Steel Bridges. Jan. 1, 1901.
2/ Discussion on desirability of single grade of structural steel, Alfred P. Boller, Proc. A.S.T.M. Vol. 2, 1902, p. 51.
3/ The Manufacturers' Std. Specs. Alfred Ladd Colby. Proc. A.S.T.M. Vol. 3, 1903.
4/ The Manufacturers' Std. Specs. Alfred Ladd Colby. Proc. A.S.T.M. Vol. 3, 1903.
5/ Penn. R. R. Co. Gen. Specs. for Structural Steel Work. June 1, 1906 (Detached sheets).
6/ Pennsylvania Lines West of Pittsburgh. Specs. for Railway Bridges, April 1906.
7/ Design of Highway Bridges. Milo S. Ketchum, First Edition, 1908.
8/ Gen. Specs. for Steel Hwy. and Elec. St. Rwy. Bridges and Viaducts. Theodore Cooper. New and Revised Edition by Bernt Berger, 1909.
9/ New York Central Lines. Specs. for Steel R. R. Bridges, 1910.
10/ Lehigh Valley R. R. Co. Specs. for Steel R. R. Bridges, 1911.
11/ U. S. Department of Agriculture Office of Public Roads, Circular No. 100. 1913 - Typical Specifications for the Fabrication and Erection of Steel Hwy. Bridges.
12/ Chicago and Northwestern Rwy. Specs. for Design and Construction of Steel Railroad Bridges. 1912. (Completed 1915)
13/ Seaboard Airline Railway General Specifications for R. R. and Hwy. Bridges, Roofs, and Steel Bldgs., 1934.
14/ American Bridge Company. General Specifications for Steel Highway Bridges, 1901.

Condensed from table opp. p. 82, Trans. Am. Sec. I.A.T.M., Vol. 1, 1898–1902
Summary of Miscellaneous Specs. for Steel

Name and Date.	Tensile strength, pounds per sq. inch.			Elastic limit, pounds per sq. inch.			Requirements full-sized eye bars.				Remarks.
	Soft.	Medium.	Rivet.	Soft.	Medium.	Rivet.	Tensile strength, lbs. per sq. in.	Elastic limit, lbs. per sq. in.	Elongation, %.	Breakage.	
Association American Steel Manufacturers, July 17, 1896	52,000 62,000	60,000 70,000	48,000 58,000	½ ult. "	½ ult. "	½ ult. "	5,000 below min. for spec.		10 body of bar.	Not more than ½ to break in head.	Eye bar matl over 1½ in. thick; reduce elong. 1% for each ½ in. increase in thickness down to 20% for med. and 22% for soft.
Atchison, Topeka & S. F., Oct., 1895	54,000 62,000	60,000 68,000		32,000	36,000		58,000	32,000	15 in 10 ft.		Soft steel web plts. over 33 in. wide may have 20% elong. and 40% reduction.
Baltimore & Ohio, 1896	52,000 60,000	60,000 68,000		28,000 or ½ ult.	32,000 or ½ ult.		58,000	½ ult.	13 in 20 ft. Down to 8 for longer lengths.		Opening and closing tests for angles.
B. & O. Southwestern, 1898	4,000 62,000	60,000 70,000		33,000	32,000		58,000	29,000	18 in 12 in. 12 balance.	Red. 40%.	
Boston, City of, 1899		60,000 64,000	48,000 58,000		½ ult.	½ ult.					
Buffalo, Rochester & Pgh., 1898	50,000 60,000	60,000 68,000		55% ult., 1 in. and less. 50% ult. over 1 in.	55% ult. 1 in. and less. 50% ult. over 1 in.		55,000. 2 in and over 60,000. 1 in and under	½ ult.	14 in 10 ft.	All bars must break in body.	
Cin., N. O. & Tex. Pacific Ry., July, 1896	52,000 62,000	62,000 70,000		32,000	35,000		58,000. 2 in. and over 60,000. 1 in. and under	50% ult. 2 in. over 55% ult., 1 in. under	14 in 10 ft.		
Theo. Cooper, 1896	54,000 62,000	60,000 68,000	50,000 58,000	½ ult.	½ ult.	½ ult.	55,000		10	Not more than ½ to break in head.	Eye bar material over 1½ in. thick reduce elongation 1% for each ½ in. increase in thickness down to 20%.
Chicago & Northwestern, 1899	50,000 60,000	60,000 70,000	48,000 58,000	½ ult.	½ ult.	½ ult.	5,000 Below specimen		10 in 15 ft.	Not more than ½ to break in head.	Eye bar material over 1½ in. thick reduce elongation 1% for each ½ in. increase in thickness down to 20% for med. and 22% for soft.
C. M. & St. P. Ry., 1898		55,000 65,000	47,000 57,000		½ ult.	½ ult.				Not more than ½ to break in head.	T. S. to average within 2,500 lbs. of 52,000 and 60,000 for rivet and balance respectively. Opening and closing tests for angles.
C. M. & St. P. Ry., June 14, 1898		60,000 68,000	48,000 56,000		½ ult.	½ ult.	5,000 Below specimen		15 in 15 ft.	Not more than ½ to break in head.	T. S. to average within 2,500 lbs of 54,000 lbs. and 64,000 lbs. for rivets and balance respectively. Opening and closing tests for angles.
Illinois Central Ry., 1899	50,000 58,000	58,000 66,000		30,000	34,000		51,000	28,000	10		
King Bridge Co	52,000 62,000	60,000 70,000		½ ult.	½ ult.		Specimen tests cut from annealed bars to determine quality			Not more than ½ to break in eye.	High steel T. S. 65,000 to 70,000 lbs., el. 18% in 8 in., reduce 35% C. B. 180% D. = 3 t.
L. S. & M. S. Ry., June, 1894	56,000 64,000			60% ult.			Order material to give same results as specimen		15		
Missouri Pacific Ry., Jan. 1, 1899	54,000 62,000	60,000 68,000		32,000	34,000		4,000 below Min. for specimen	½ ult.	13 in 20 ft.		
N. Y. C. & H. R. R. R., 1899	56,000 61,000	62,000 70,000	48,000 56,000	60% ult., ⅝ in. t and under. 55% ult. over ⅝ in.	60% ult.		58,000	33,000	10 in 20 ft.		Rollers and roller plts.—T. S. 70,000 to 78,000 lbs. el. 22% in 8 in.
Northern Pacific Ry., Dec. 1, 1898	52,000 60,000	62,000 70,000		30,000	37,000		57,000		12 in 20 ft.		
Osborn Co., R.R. bridges, 1896	52,000 62,000	60,000 70,000		31,000	35,000		55,000		15 in 10 ft.		
Osborn Co., highway bridges, 1895	52,000 62,000	60,000 70,000		31,000	35,000		56,000		15 in 10 ft.		
Pennsylvania Lines West of Pittsburgh, April, 1897	54,000 62,000	62,000 70,000	52,000 58,000	55% ult.	55% ult.	55% ult.	$62,000 - \frac{9\,000\,A}{P}$ $54\,000 - \frac{8,000\,A}{P}$	55% ult.	2 in 10' med. 14 in 10' soft	Not more than ½ to break in eye.	
Pennsylvania Railroad, Jan 1, 1897	52,000 62,000	62,000 70,000	48,000 56,000	28,000	33,000	28,000	48,000	27,000	14 in 10 ft.	Not more than ½ to break in eye.	
J. A. L. Waddell	50,000 60,000	60,000 70,000		30,000	35,000		58,000. 2 in. and over 60,000, 1 in. and under	50% ult., 2 in. over 55% ult., 1 in. under	14 in 10 ft.	Not more than ¼ to break in eye.	High Steel { El., 40,000 lbs. red. 38 to 30% T. S. 70 to 80,000 lbs., el., 22 to 18% in 8 in. Q bend. | Med. phos. .06A to .04B, Sul. .05, Sil. .05, Mn. .70. High phos. .07A, Sul. .05, Sil. .06, Mn. .80.
Canadian Pacific Ry., 1898		54,000 65,000			33,000		58 000 to 65,000	32,000	10 in 20 ft. Red. 40	Not more than ¼ to break in eye.	
Grand Trunk Ry., Nov. 17, 1897	54,000 62,000	60 000 68,000		½ ult.	½ ult.		58 000 58,000	30,000 29,000	15 in 20 ft. 10 in 10 ft.	For bars 8 sq. in. and less. For bars over 8 sq. in. to 20	Not more than ¼ of bar to break in head.
Dominion Government, 1899	50,000 58,000	60,000 68,000		30,000	32,000		56,000, 6 in. sq. & less 54,000 over 6 sq. in.	½ ult.	15 in 10 ft.	Reduction 30 to 20%.	Opening and closing tests for angles.
Mexican Central, 1898		60,000 68,000	50,000 56,000		½ ult.		58,000	28,000	10 in 10 ft.	All bars must break in body.	
C. B. & Q. Ry., 1898	52,000 62,000	60,000 70,000		½ ult.	½ ult.		55,000 to 65,000	½ ult.	10 in 20 ft. Red. 40		
Great Northern Ry., Mar. 1, 1898	52,000 62,000	60,000 70,000	48,000 56,000	55% ult.	55% ult.	55% ult.	51,000 soft 58,000 med.	27,000 soft 33,000 med.	14 in 12 ft. average 16	Not more than ½ to break in eye.	El. for med. 26% for 57,000 lbs., T. S.—reduce ⅝% for each 1,000 lbs. down to 3
Michigan Central Ry. 1899	50,000 60,000	57,000 66,000		55% ult.	55% ult.						High steel T. S. 66 to 74,000 lbs. el., 22%, red. 45%.
Southern Railway, 1897	52,000 62,000	60,000 70,000	48,000 53,000	28,000	32,000	25,000		30,000	12 in 15 ft.	Red. 40%.	
Union Pacific Ry., 1898	50,000 58,000	60,000 68 000		½ ult.	½ ult.						
Wabash R.R., Mar., 1898	52,000 60,000	60,000 70,000	48,000 60,000	½ ult.	½ ult.	½ ult.	58,000	½ ult.	10 in 10 ft.	Not more than ½ to break in eye.	Opening and closing tests for angles.
Pencoyd Iron Works, April, 1895	50,000 60,000	60,000 70 000		½ ult.	½ ult.				10 in 10 ft.	Must break in body of bar.	
N. Y. N. H. & H. R.R., 1894	56 000 64,000		50,000 54,000	35,000							Opening and closing tests for angles.
Chicago & Alton, Oct., 1897	54,000 62,000	62 000 70,000	48,000 56,000	½ ult.	½ ult.	½ ult.	58 000	30,000	12 in 10 ft.		
Philadelphia & Reading	57,000 65,000		Not over 60 000	54% ult.							
Plant System Rys., June 2, 1898	52,000 62,000	60,000 68,000		34,000	36,000		56,000	32,000	15 in 10 ft.		
Robert Moore	55,000 65,000										
Southern Indiana R.R. Co.	54,000 62,000	60,000 68,000	48,000 58,000	30,000	34,000	23,000	52,000 soft 58,000 med.			Not more than ½ to break in eye.	Punch tests.
Chesapeake & Ohio, Feb., 1896	52,000 60,000	60,000 68,000		30,000	35,000		60,000 med. 52,000 soft		12½ med. 18 soft		
Boston Elevated Ry., 1898	48,000 56,000	60,000 68,000		30,000	½ ult.						

L-5305

Summary A.S.T.M. Specifications For Steel For Bridges

For effect of cross head speed of testing machine on structural properties see Sheet L-4578

Year	Specification	Process	Tensile Strength pounds per square inch	Yield Point pounds per square inch	Elongation		Chemical Properties				
					Min. in 8 inches percent	Min. in 2 inches percent	Copper percent	Carbon percent	Sulphur percent	Phosphorus percent	Analyses
1901 Aug. 10	Proposed Standard Spec. for Str. Steel for Brdgs. and ships	Open Hearth	Soft 52,000-62,000 Med. 60,000-72,000	Soft 32,000 Med. 35,000	Soft 25 Med. 22	No Requirement	No Reference	No Requirement	Max. .06	Max. Acid .08 Basic .06	No Requirement
1905	Str. Steel for Brdgs.	do	Desired 60,000 See Note (a)	No Requirement Other Than Yield Point shall be Recorded	1,500,000 Tens.Str.	22	do	do	.05	Acid .08 Basic .04	Required to Determine Percent of C.S.P.& Mn.
1909	Str. Steel for Brdgs.	do	do See Note (a)	do	do	do	do	do	do	Acid .06 Basic .04	do
1912	Str. Steel for Brdgs.	do	Desired 60,000 See Note (b)	do See Note (c)	do	do	do	do	do	do	do
1916	A-7-16	do	55,000-65,000	0.5 Tensile Str.	do	do	do	do	do	do	do
1918	A-7-16	do	do	do	do	do	do See Note (d)	do	do	do	do
1921	A-7-21	do	do	do	do	do	do	do	do	do	do
1924	A-7-24	do	do	0.5 Tens.Str. But in No Case less than 30000	do	do	do	do	do	do	do
1927	A-7-24	do	do	do	do	do	do	do	do	do	do
1930	A-7-29	do	do	do	do	do	When Copper Steel is Spec. Not Under 0.20	do	do	do	do Also Copper When Copper Steel is Specified
1933	A-7-33	Open Hearth or Electric Furnace	do	do	do	do	do	do	do	do	do
1933	A-7-33T	do	60,000-72,000	0.5 Tens.Str. But in No Case Less Than 33,000	do	do	do	do	do	do	do
1936	A-7-36	do	do	do	do	do	do	do	do	do	do

NOTES: (a) If the ultimate strength varies more than 4,000 pounds from that desired, a retest may be made at the discretion of the inspector, on the same gauge which to be acceptable shall be within 5,000 pounds of the desired ultimate.

(b) Tensile strength may vary 4,000 pounds per square inch from that specified.

(c) In 1913, recommended that "min. value of one-half ultimate strength be required."

(d) "In view of the abnormal difficulty in obtaining materials in time of war, the rejection limits for sulphur in all-steels and for phosphorus in acid steels shall be raised 0.01 percent above the values given in these specifications."

Prepared by John I. Quinn
November 1938

Summary A.S.T.M. Specifications "Speed of Operation of Testing Machine."

Year	Specification	Requirement
1901	-	No requirements
1905	Report Committee "O"	"A speed of 3 inches (per minute) or thereabouts has been in practical use for commercial testing for a long time" --- We may fairly assume a 6-inch speed of testing for commercial purposes of metal of 0.45 percent carbon and less as acceptable without detriment of reliability of results."
1909		No references
1912	"Standard Methods of Testing"	"All information obtained confirms the investigations of Committee "O" (since dissolved) to the effect that within the limits of speed common in commercial testing, the effect of different speeds on results is not of observable moment, that is, within the ranges of speed varying from 1 inch to 6 inches per minute." No other restrictions.
1916	E-1-16	do
1918	E-1-18	Specified Min. Strength Structural Material (pounds per square inch): 80,000 or under — Gage Length (inches): 2, 8 — Max. Cross Head Speed inches per minute: Yield Point 0.50, 2.00 — Tensile Strength 2, 6; Over 80,000 — Gage Length: 2, 8 — Max. Cross Head Speed inches per minute: Yield Point 0.25, 0.50 — Tensile Strength 1, 2. "The pulling speed has a marked influence on the tensile properties shown by materials tested, an increase in speed increasing the values found for yield point and tensile strength."
1921	E-1-18	do — do
1924	E-8-24T	do — do
1927	E-8-27T	do — do
1930	E-8-27T	do — do
1933	E-8-33	"The pulling speed has a marked influence on the tensile properties shown by materials tested, an increase in speed increasing the values found for yield point and tensile strength. ----In determining yield strength, the cross head speed for the 2-inch gage length shall not exceed 0.125 inches per minute."
1936	E-8-36	do

Prepared by John I. Quinn
November 1918

L-4578

U. S. Department of Agriculture
Bureau of Public Roads
DIVISION OF BRIDGES
Washington, D. C.

Tests of Metals. Watertown Arsenal. Volume for 1881.

Applied Mechanics - Lanza - 1905. Pages 425 and 426.

Wrought Iron
Circular Columns, 4 Segments, Built by The Phoenix Iron Company

Tested with diameter through flanges 45° from vertical

No. of test	Kind	Diameter	Length	Sectional area	Ultimate Strgth Actual	Ultimate Strgth Per sq. inch	Manner of failure
		in.	ft. in.	sq. in.	lbs.	lbs.	
325	Flat ends	8.04	0 30.000	11.610	651,000	56,070	Web bulged near end
327	Do	8.04	0 29.940	11.902	628,500	52,800	Web bulged near middle of column
326	Do	8.00	11 10.625	12.181	466,000	38,256	Deflected downward
31	Do	8.00	31 0.000	11.430	356,000	31,150	Deflected upward and horizontally
32	Do	8.00	31 0.000	11.310	370,500	32,760	Do
33	Do	8.04	31 6.000	11.660	363,600	31,180	Do
34	Do	8.04	31 6.000	11.510	373,100	32,220	Do

American Bridge Company's Steel Columns

Column with pin ends tested, with pins vertical

No. of test	Kind	Size	Length	Sectional area	Ultimate Strgth Actual	Ultimate Strgth Per sq. inch	Manner of failure
		in.	ft. in.	sq. in.	lbs.	lbs.	
1058	Flat ends	10.27x10.30	0 43.10	15.28	719,000	47,055	Flanges buckled
30	Pin ends	10.27x10.35	30 0.44	15.28	290,000	18,980	Deflected horizontally

GB - Jan. 1939

L-5306

Tests of Metals, Watertown Arsenal, Volume for 1881

Applied Mechanics - Lanza - 1905. Pages 422 and 425

Latticed Wrought Iron Columns from Detroit Bridge and Iron Co.

Columns with pin ends tested with pins vertical; channel-bars spaced 8 inches apart

No. of test	Kind	Channel-bars	Length		Sectional area	Lattice spacing	Ultimate strgth Actual	Ultimate strgth Per sq. inch	Manner of failure
		in.	ft.	in.	sq.in.	in.	lbs.	lbs.	
1059	Flat ends	6	10	0	4.760	18	174,800	36,720	Channel bars buckle
1060	Flat ends	6	10	0	4.670	18	165,000	35,330	Do
1095	One flat end, one pin end	6	10	0	4.750	18	160,000	33,680	Do
1096	Do	6	10	0	4.580	18	154,800	33,800	Do
1107	Pin ends	6	12	0	4.600	18	159,800	34,740	Horizontal deflecti
1108	Do	6	12	0	4.570	18	156,100	34,160	Do
1	Do	6	12	6	4.560	18	163,600	35,880	Do
2	Do	6	12	6	4.740	18	153,500	32,380	Do
1231	Do	6	15	0	4.480	18	151,500	33,820	Do
1232	Do	6	15	0	4.560	18	157,500	34,540	Do
1229	Do	6	17	6	4.660	18	152,600	32,750	Do
1230	Do	6	17	6	4.740	18	147,500	31,120	Do
1117	Do	6	20	6	4.660	18	136,000	29,180	Do
1118	Do	6	20	0	4.630	18	143,500	30,990	Do
1119	Do	6	22	6	4.570	18	139,800	30,590	Do
1120	Do	6	22	6	4.660	18	144,700	31,050	Do
1121	Do	6	25	0	4.710	18	110,000	23,350	Do
1122	Do	6	25	0	4.630	18	117,500	25,360	Do
20	Do	6	27	6	4.690	18	102,500	21,850	Do
21	Do	6	27	6	4.670	18	97,200	20,810	Do
18	Do	6	30	0	4.700	18	69,300	14,740	Do
19	Do	6	30	0	4.730	18	75,200	15,900	Do
1111	Do	8	13	4	7.520	18	261,800	34,810	Defl. upward; chan-bars buckled.
1112	Do	8	13	4	7.500	18	264,300	35,240	Defl. horizon. chan-bars buckled.

Channel-bars spaced 6 inches apart

463	Pin ends	6	20	0	4.68	16	117,000	25,000	Defl. horizontally
464	Do	6	25	0	4.68	16	71,400	15,260	Do
465	Do	8	20	0	7.75	16	215,100	27,750	Do
466	Do	8	25	0	7.75	16	201,500	26,000	Do
467	Do	10	20	0	9.19	18	275,500	29,980	Defl. upward
468	Do	10	25	0	9.19	18	294,080	32,000	Defl. horizontally
469	Do	12	20	0	12.95	-	375,200	28,970	Defl. upward
470	Do	12	25	0	12.95	-	388,500	30,000	Defl. horizontally

Tests of Metals - Watertown Arsenal

1886 - Part 2

Tests of Full-Sized Eye-Bars of Wrought Iron					Tests of Full-Sized Eye-Bars of Steel				
Dimensions inches					Dimensions inches				
Length c.to c. pin holes	Width	Thickness	Elastic limit lbs.per sq.in.	Tensile strength lbs.per sq.in.	Length c.to c. pin holes	Width	Thickness	Elastic limit lbs.per sq.in	Tensile strengt lbs.per sq.in.
238.55	5.00	1.14	22,456	45,105	308.00	5.16	1.02	34,610	64,870
238.60	5.00	1.15	22,610	44,540	308.00	5.14	1.02	34,730	69,330
238.57	4.99	1.14	21,790	43,320	308.00	5.15	1.02	37,330	70,286
238.64	5.00	1.16	22,410	39,550	308.10	5.14	1.02	35,000	70,229
238.62	6.05	1.44	19,750	43,260	308.00	5.13	1.02	35,950	71,680
238.62	6.05	1.44	22,730	42,020	307.95	5.15	1.02	35,000	70,895

Tests of Steel Eye-Bars Used in the Mississippi Bridge at Memphis - 1894

Applied Mechanics - Lanza - 1905. Page 494

	Full-Size Eye-Bars					Sample Bars From Same Melt			
	Width	Thickness	Length center to center	Elastic limit per sq. inch	Maximum load per sq. inch	Original area	Elongation percent	Elastic limit per sq. inch	Maximum load per sq. inch
	ins.	ins.	ins.	lbs.	lbs.	Sq. in.		lbs.	lbs.
1	10.07	1.50	160.63	35,100	67,490	.9500	27.5	41,580	73,050
3	9.95	1.73	358.93	37,680	70,160	.9918	24.4	42,650	75,620
4	9.98	1.75	361.23	39,700	65,500	.9520	28.8	40,280	70,280
5	10.05	1.50	162.38	33,140	65,060	.9500	27.5	41,580	73,050
6	6.08	1.13	291.26	29,690	56,700	.9756	28.1	40,490	69,700
8	10.07	1.67	287.37	32,860	65,600	1.1590	20.0	43,750	75,000
9	9.92	1.95	284.28	31,110	61,060	1.0140	28.8	42,210	69,730
10	9.94	0.99	287.88	33,990	63,220	.9868	28.1	40,230	69,720
11	10.05	2.20	222.88	29,330	63,100	.9635	28.8	38,090	71,300
13	10.12	1.86	464.03	31,970	53,860	1.0201	27.0	40,200	71,860
15	7.12	1.17	314.04	30,270	51,500	1.0180	28.8	33,400	57,170
16	10.07	2.20	338.73	28,080	55,160	1.1220	24.2	38,320	70,220
18	10.03	1.81	251.58	29,670	62,140	1.0200	26.3	40,200	71,080
21	9.97	1.37	250.28	32,700	65,400	1.0670	25.0	39,360	69,360
23	7.02	1.31	385.73	28,980	52,010	1.1700	31.3	34,190	58,460
24	7.01	1.26	385.78	28,410	54,740	1.0170	28.1	41,400	67,840
25	9.99	1.62	249.98	30,500	58,870	.9338	25.0	40,910	70,360
27	9.96	2.05	341.28	33,360	73,550	.9700	25.5	40,410	69,900
28	10.13	1.30	249.48	32,520	60.710	.9504	27.0	40,400	70,490
30	9.98	1.81	284.82	28,000	58,720	.5557	29.5	40,000	66,800
31	10.15	1.83	221.98	32,290	62,270	.9746	21.3	40,530	72,240
34	10.04	0.99	361.68	29,970	58,680	1.1720	27.0	40,610	70,480
35	7.01	1.27	258.68	28,640	56,830	1.0200	28.1	40,790	68,730
42	7.98	1.20	254.63	31,930	63,870	1.0100	21.9	40,900	69,800
43	8.03	2.32	338.58	32,840	62,400	1.0620	23.1	41,710	71,000
44	7.00	1.18	258.68	27,870	53,520	1.0560	31.9	32,480	58,050
46	9.09	1.25	206.58	32,590	57,410	.9734	28.7	38,110	60,920
53	8.11	1.79	279.98	28,940	58,010	1.114	23.0	40,480	66,880
55	7.00	1.00	289.23	31,380	59,850	1.020	28.1	40,790	68,730

L-5309

Condensed from Plate VIII
Trans. A.S.C.E., Vol. LXXIII - 1911

Record Full-Sized Eye Bar Tests
Ohio River Bridge, P.& L.E.R.R., 1909

No. of test.	Heat number.	Slab number.	Size of bar, in inches.	Elastic limit.	Ultimate strength.	Original area.	Specimen Tests: Unannealed.				Specimen Tests: Annealed.				Che
							El. Lim.	Ult. Str.	El. Lim.	Ult. Str.	El. Lim.	Ult. Str.	El. Lim.	Ult. Str.	Car.
1	48 443	241 848	16 × 2 1/8	38 690	67 170	34.24	36 540	69 130	37 970	68 100	36 960	62 470	35 400	62 900	0.24
2	41 006	1 784	16 × 2 1/8	34 720	61 920	34.56	38 360	68 180	37 770	65 440	36 860	61 920	36 380	57 810	0.28
3	48 455	244 852	16 × 2 1/8	36 500	65 760	34.52	38 850	70 420	38 100	66 800	35 950	63 360	36 520	63 360	0.27
4	41 023	7 201	14 × 1 1/2	34 130	65 500	21.68	38 410	71 780	38 500	67 160	36 620	66 120	37 450	65 820	0.24
5	48 445	242 340	14 × 1 1/2	34 000	65 210	21.62	37 950	70 580	38 950	69 740	35 690	58 740	36 140	59 800	0.24
6	49 452	242 511	14 × 1 1/2	35 140	64 480	21.34	38 760	66 780	37 640	67 180	34 760	58 090	35 600	63 360	0.28
7	41 013	3 817	16 × 2	33 060	60 200	32.36	44 760	66 060	42 810	64 640	36 210	57 610	35 720	57 670	0.25
8	41 008	2 311	16 × 2 1/8	33 500	59 440	34.32	35 960	63 400	38 120	66 560	35 670	59 020	36 230	57 980	0.25
9	50 300	168 353	12 × 1 3/4	34 470	59 910	21.03	40 350	65 920	40 860	66 980	36 500	58 160	37 040	59 660	0.24
10	49 464	245 501	16 × 2	33 950	58 330	32.40	38 450	64 120	37 690	64 170	35 530	57 450	35 940	58 340	0.25
11	44 038	15 330	16 × 2	37 650	62 500	32.40	39 890	65 300	38 760	67 820	35 320	60 600	37 180	60 980	0.24
12	44 034	13 415	16 × 2 1/8	36 400	64 720	34.61	37 800	69 180	38 420	65 780	37 080	62 620	36 800	61 880	0.24
13	44 039	15 865	16 × 1 7/8	37 030	64 730	30.51	39 160	67 920	38 430	66 820	34 900	60 870	33 380	59 290	0.27
14	44 009	2 635	16 × 1 7/8	34 320	61 130	20.30	35 440	56 900	35 710	57 000	38 570	67 340	37 900	65 960	0.25
15	41 014	4 223	16 × 1 7/8	37 920	63 090	30.59	39 460	66 040	40 510	66 530	35 500	58 960	37 010	56 860	0.25
16	49 462	244 916	16 × 1 7/8	34 780	65 630	30.47	38 980	66 890	38 780	67 040	37 000	61 940	36 690	60 180	0.28
17	40 099	34 655	14 × 1 1/2	33 360	62 860	23.24	38 210	68 160	38 820	68 320	35 420	60 900	34 780	60 060	0.26
18	44 073	26 584	14 × 1 1/2	33 000	68 400	23.37	40 190	71 800	41 410	72 480	36 320	59 160	35 870	64 140	0.27
19	44 064	23 608	16 × 1 3/4	36 600	64 920	28.65	38 370	67 980	37 670	67 540	35 000	60 040	35 640	56 340	0.26
20	41 068	24 721	16 × 1 3/4	34 940	62 010	28.62	38 310	65 640	37 770	64 800	34 910	61 560	34 040	62 490	0.27
21	44 064	23 612	14 × 2	38 730	65 840	28.40	38 370	67 980	37 670	67 540	35 000	60 040	35 640	56 840	0.26
22	41 088	32 395	14 × 1 11/16	38 620	65 400	24.08	38 090	68 480	38 780	67 020	34 810	64 040	35 400	63 120	0.24
23	40 105	35 777	14 × 2	36 370	62 140	28.32	39 150	67 010	39 550	68 480	35 300	60 060	35 480	60 820	0.25
24	42 108	45 183	14 × 1 11/16	36 030	62 440	24.42	37 190	65 520	37 530	65 420	35 830	61 040	35 700	60 320	0.27
25	41 061	21 524	14 × 1 5/8	35 620	65 450	23.30	38 690	68 120	39 300	69 860	34 470	63 180	35 390	65 180	0.25
26	41 084	30 779	14 × 1 5/8	37 660	66 720	23.23	38 520	68 000	37 780	62 970	35 000	61 720	35 960	61 700	0.24
27	40 067	22 823	16 × 2 1/8	34 180	64 180	33.64	38 200	66 660	38 090	65 040	35 220	59 620	34 240	56 830	0.20
28	41 063	22 015	16 × 1 11/16	36 020	74 440	27.20	40 560	73 980	40 560	71 960	35 740	62 260	36 240	61 660	0.27
29	44 060	21 577	16 × 1 11/16	36 760	66 170	27.20	38 950	71 800	38 870	69 660	35 540	65 080	35 260	68 920	0.27
30	40 078	26 523	12 × 1 5/8	35 480	64 460	20.01	38 550	67 620	38 170	65 880	26 340	58 350	35 180	58 330	0.28
31	33 076	.....	10 × 1 5/16	35 000	63 840	13.00	41 490	67 490	43 370	67 090	39 220	62 100	40 620	63 760	0.26
32	40 106	36 291	14 × 1 7/8	34 580	62 970	26.60	39 040	65 760	38 970	67 140	35 460	56 900	35 500	59 960	0.25
33	34 079	.....	8 × 1 5/8	36 320	66 400	12.80	42 130	69 580	40 740	68 070	39 850	63 940	39 820	62 950	0.30
34	34 079	.....	8 × 1 5/8	34 140	67 180	12.80	42 130	69 580	40 740	68 070	39 850	63 940	39 820	62 950	0.30
35	50 296	166 611	12 × 1 1/2	30 500	58 510	18.03	39 160	66 300	39 460	64 700	35 000	60 780	33 660	61 240	0.27
36	45 235	161 569	12 × 1 3/4	30 800	61 610	21.10	43 900	65 440	38 790	64 060	32 530	57 570	31 290	62 240	0.25
37	45 237	162 279	12 × 1 3/4	35 390	64 560	20.91	49 470	69 200	42 650	69 000	37 190	61 740	34 140	59 020	0.26
38	45 237	162 280	12 × 1 1/2	34 620	65 060	18.06	49 470	69 200	42 650	69 000	37 190	61 740	34 140	59 020	0.26
39	45 234	161 282	14 × 1 1/2	35 300	65 870	22.77	43 270	72 420	41 160	70 500	36 020	65 720	35 340	62 510	0.27
40	45 238	162 782	14 × 1 5/8	34 640	67 960	22.80	44 220	71 780	42 700	69 330	35 390	64 120	33 790	62 120	0.24
41	46 294	169 337	12 × 1 3/4	35 980	64 910	21.26	40 520	68 420	41 020	69 200	35 800	62 310	35 860	63 550	0.27
42	42 286	.....	8 × 1 5/8	36 800	59 900	10.85–" 10.90–"	40 620	62 260	.....	.....	41 020	65 520	.....	.....	0.29
43	42 286	.....	8 × 1 5/8	L 33 170 S 32 800	62 210 61 500	10.85 10.96	40 620	62 260	.....	.....	41 020	65 520	.....	.....	0.29
44	41 011	3 121	16 × 2	29 850	64 320	32.16	38 070	72 400	35 650	69 850	34 480	61 560	36 150	57 100	0.28
45	41 011	3 099	16 × 2	39 270	67 330	32.08	38 070	72 400	35 650	69 850	34 480	61 560	36 150	57 100	0.28
46	41 086	31 744	14 × 1 11/16	29 290	61 500	23.90	38 250	67 030	38 200	65 630	35 720	57 190	34 950	60 270	0.26
47	41 086	31 730	14 × 1 11/16	37 410	64 620	23.52	38 250	67 030	38 200	65 630	35 720	57 190	34 950	60 270	0.26
48	41 063	22 009	16 × 2 1/8	34 230	65 550	34.32	40 560	73 980	40 560	71 240	35 740	68 960	36 240	61 680	0.27
49	40 066	22 400	16 × 2 1/8	31 600	61 460	34.49	37 720	65 710	38 140	65 830	33 600	61 200	34 820	59 500	0.27

L-5310

THE DELAWARE RIVER BRIDGE
Connecting
Philadelphia, Pa. and Camden, N. J.
Final Report Board of Engineers
June 1927

TESTS OF CARBON, SILICON AND NICKEL STEELS

Extensometer Tests of Rolled Steel

Under the specifications for Contract No. 14, abstracted on p. 112, occasional check tests were made on both silicon and nickel steel specimens, using extensometers. With the coöperation of the rolling mills, certain similar tests were made on structural carbon steel specimens, although not required by the specifications.

When shearing blanks for standard mill test specimens, the mill also sheared larger pieces, about 18 in. square in the case of plates. From these pieces check test specimen blanks were cut when check tests were requested by the inspector. At first the check test specimen blanks were cut out by shearing, but the bending due to this shearing was found to set up internal stresses which seriously affected the accuracy of the stress-strain curve. This is shown in tests 1 to 8 inclusive on silicon steel, Fig. 1, and tests Nos. 1 to 7 inclusive and 26 to 29 inclusive on nickel steel, Fig. 2. The remaining test coupons were all cut out of the blanks by burning, with a liberal allowance for machining, or by machining or sawing.

The requirement for the extensometer tests of silicon steel was that the elongation under a tension of 41,000 lbs. per sq. in. should not exceed 0.25%. This elongation permitted about 0.10% inelastic stretch (the elastic stretch up to 41,000 lbs. per sq. in. approximating 0.15%) and thus constituted a practical measure of the yield point. The critical elongation is indicated on Figs. 1 and 2 by the mark △. The stress-strain curves show the value for yield point thus determined as compared with the average values for material from the same heat determined by the drop of beam in the standard mill tests. Ultimate strengths of the 2-inch gauge length turned specimens used in the extensometer test are also compared with the average ultimate strengths of the standard mill tests.

On Fig. 2, nickel steel tests, are shown two curves, Nos. 26 and 27, which were of specimens turned from sheared blanks. These failed to meet the specifications. Retests Nos. 28 and 29 were prepared subsequently and gave acceptable results. As a further check on the quality of the material compressive tests were made. Blanks were cut across the entire width of the end of three plates from the heat, slit into three or four pieces, bolted and strapped together in a pack and machined into test specimens 12 inches long. These were tested by the Bureau of Standards with the very satisfactory results shown in Tests Nos. 30 to 32 inclusive.

When the carbon steel tensile test specimens, Nos. 1 to 10 inclusive of Fig. 3 were selected, small compressive test specimens were locate immediately adjacent, as shown at the bottom of the figure. The compression specimens were 2 to 2½ in. long, 1 to 1½ in. wide and of the thickness of the material as rolled. They were very carefully machined on the ends to insure parallelism and the length measured with micrometer calipers. The compressive tests were made with a 2-inch gauge length Berry strain gauge attached to the specimen. After loading to about 30,000 lbs. per sq. in. the load was released and measurement of overall length made. This was done after each additional increment of load. The micrometer measurements were taken to see whether a satisfactory determination of the yield point could be made by ordinary shop micrometer calipers on compressive test specimens. Comparison of readings by micrometer calipers after release of load, with extensometer readings before release of load, indicate that sets measured by micrometer calipers would afford a very satisfactory index of the yield point of specimens tested in compression. The large increase in load, with small increase of stress, shows that the yield point could be accurately determined in routine mill tests, by this procedure.

Conclusion. The tests indicated that the ultimate strength of the elements of the tower column could be relied upon to be between 44,000 and 46,000 lbs. per sq. in. as these strengths were attained by the specimens with single and double ¾-inch webs.

As regards Fairbairn's rule, it was shown to be approximately correct when considered to apply to the clear width of the plate between flange angles. The following table gives the ratio of width of web plates to thickness for the various types of specimens, considering the width as (a) unsupported width between toes of flange angles, which was 23 inches and (b) as the distance between lines of rivets connecting the web plates to the flanges, which was 26 inches.

TABLE III. RATIO OF WIDTH TO THICKNESS OF WEB PLATES

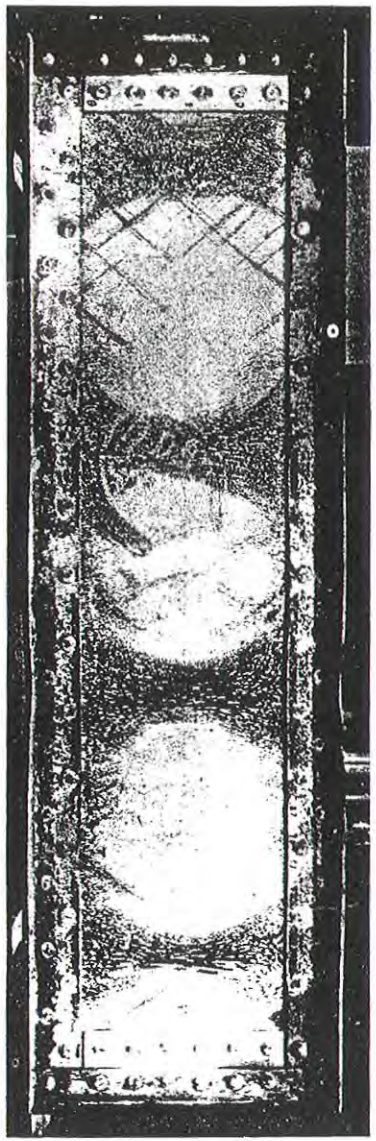
Thickness of Web Plate	Ratio Width to Thickness	
	(a) $\frac{23}{T}$	(b) $\frac{26}{T}$
⅜ inch	61.3	69.3
½ inch	46.0	52.0
⅝ inch	36.8	41.5
¾ inch	30.7	34.7
1 inch	23.0	26.0
1½ inch	15.3	17.3

In the case of the ⅜-inch and ½-inch specimens the web failed to stand up and take its proper share of the load. The ⅝-inch specimens behaved much better, though here the action of the web plates was controlled largely by the buckles which were in the plates before testing. The ¾-inch plates were much flatter initially and these plates did not buckle until angles as well as plates had reached the limit of their resistance.

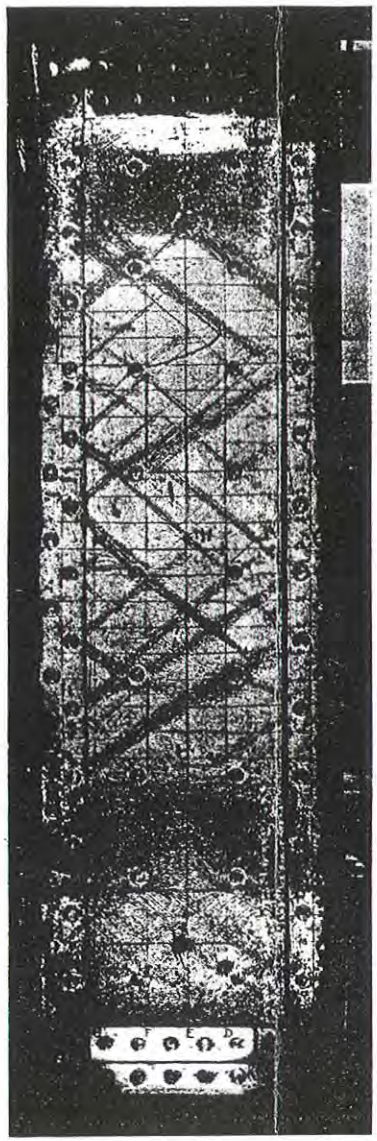
In the study of single web versus double web it was found that either should prove just as effective for carrying compressive loads provided the thickness of each plate is beyond the critical buckling thickness, the plates are equally straight, and the material in the webs is alike. Stitch-riveting of perfectly flat webs would, probably, be unnecessary. On the other hand, the usual inequalities in the alignment and in the material of the two plates of the double web, bring the stitch-riveting into play to make the plates work together. Both conditions were observed in the tests. One set of measured rivet strains was taken well away from the buckle that developed at failure, and throughout the whole test there was no indication of increased strain on the rivet shanks. In another column, the rivets on which the strains were measured were in the field of the failing buckles. 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 the buckle.

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

It is possible, as 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.

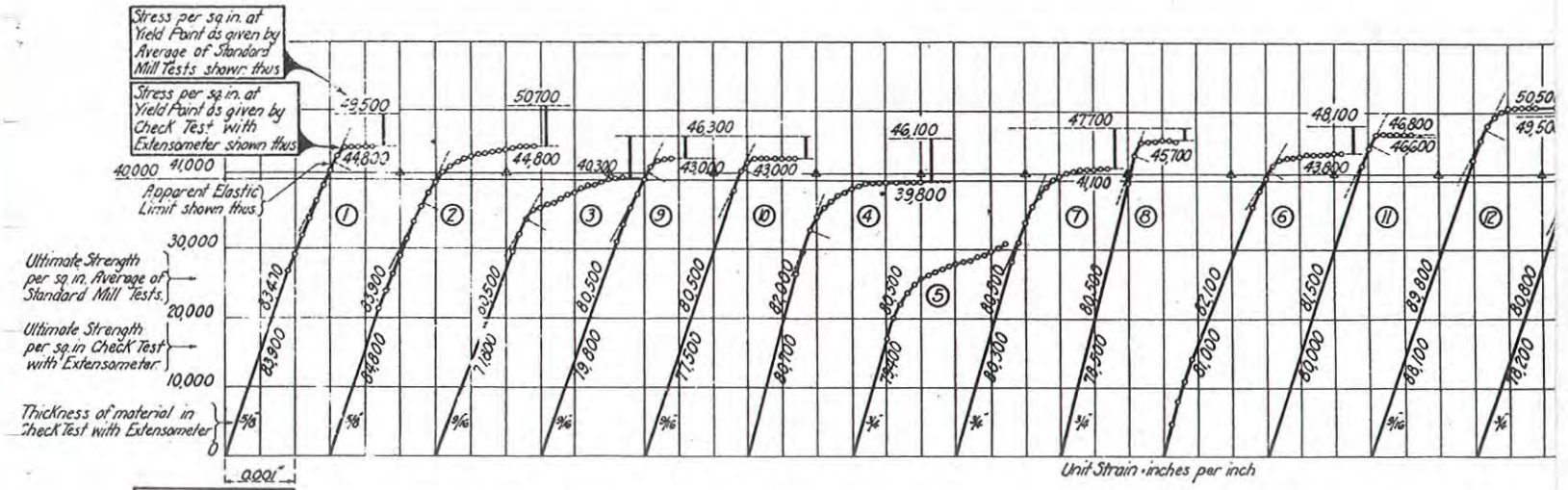


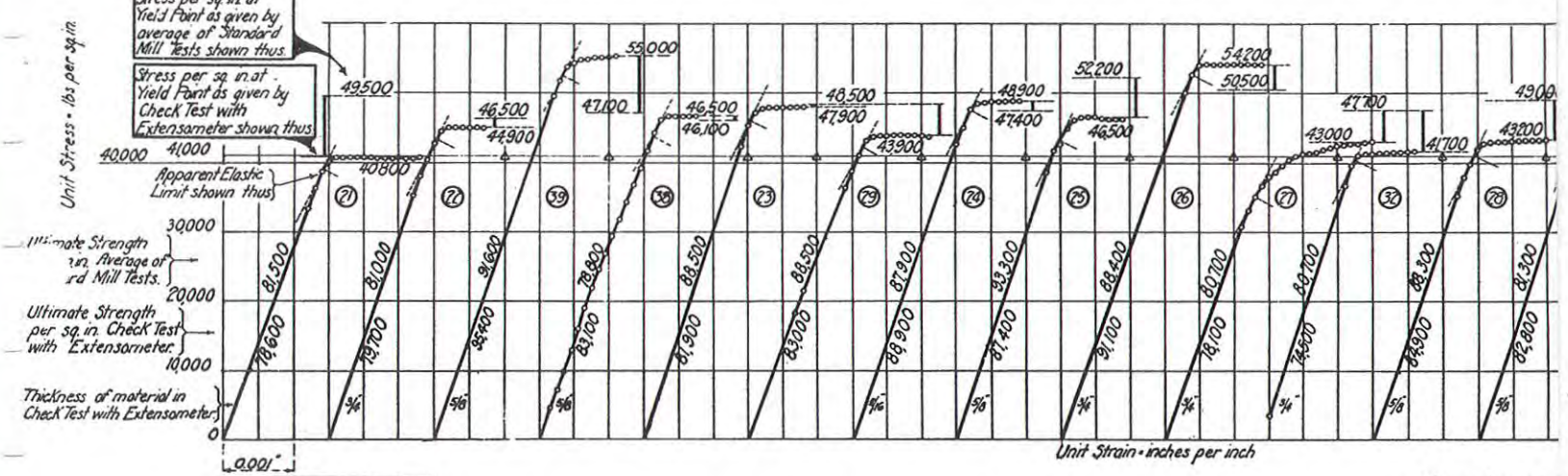
SINGLE WEB SPECIMEN, AFTER FAILURE. SHOWING HARTMAN-LUDER LINES.

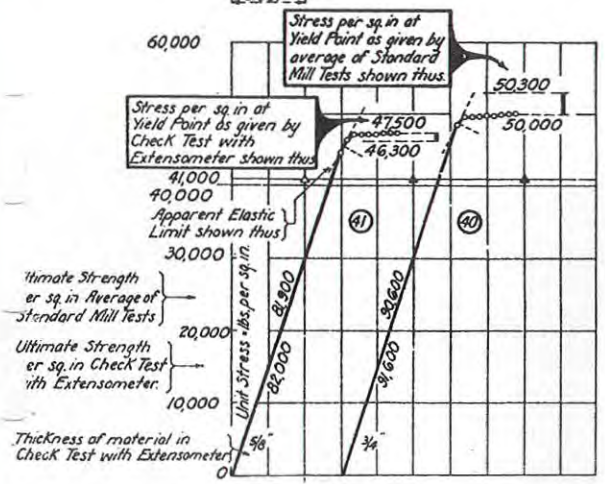


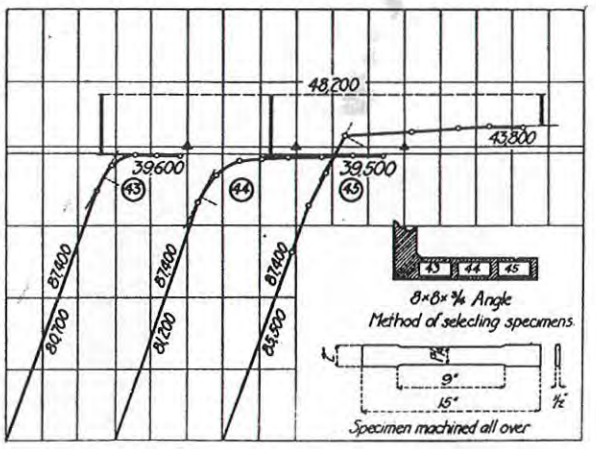
DOUBLE WEB, STITCH-RIVETED SPECIMEN, AFTER FAILURE. DARK AREAS AT UPPER AND LOWER END ARE CONCAVE SIDE OF BUCKLES.

L-4580









Material used in
" " "
" " "
" " "
Apparent Elastic
which the rate of
at its origin. (Mate
Tests No. 9 and 10
" " 7 and 8
" " 29
" " 43, 44 an
All tests excepting
Westinghouse Electri
machine with auto
Strain readings ta
No. 43, 44 and 45 i
Specimens for Tes
from material of
Specimens for Tes
No. 18 and Test No.
sheet represent ma
extensometer tests
Coupons for Tests
Resulting bending
Coupons for Test
material with libera

L-4581

NOTES

Material used in Tests No. 3, 9 and 10 from same heat.
" " " " " 5, 7 and 8 " " " .
" " " " " 23 and 29 " " " .
" " " " " 27 and 32 " " " .
" " " " " 43, 44 and 45 " " " .

Apparent Elastic Limit is point on stress diagram at which the rate of deformation is 50 per cent greater than at its origin. (Materials of Construction:- Johnson)

Tests No. 9 and 10 are retests for No. 3.
" " 7 and 8 " " " No. 5
" " 29 is " " No. 23.
" " 43, 44 and 45 at the University of Illinois.

All tests excepting Nos. 43, 44 and 45 were made at the Westinghouse Electric and Manufacturing Co. of Pittsburgh on machine with automatic stress strain recorder.

Strain readings taken on a 2" gage length except for Test No. 43, 44 and 45 in which gage length was 8".

Specimens for Tests No. 1 to 8 inclusive 21 and 24 taken from material of Contract No. 14.

Specimens for Test No. 18 represents also material of Contract No. 18 and Test No. 21 together with the remainder on this sheet represent material of Contracts No. 17 and 18 for which extensometer tests were not specified.

Coupons for Tests No. 1 to 8 inclusive sheared out of material. Resulting bending of coupon caused defective tests (No. 3, 4 and 5).

Coupons for Test No. 9 to 42 inclusive were burned out of material with liberal allowance for finishing.

SPECIFICATIONS

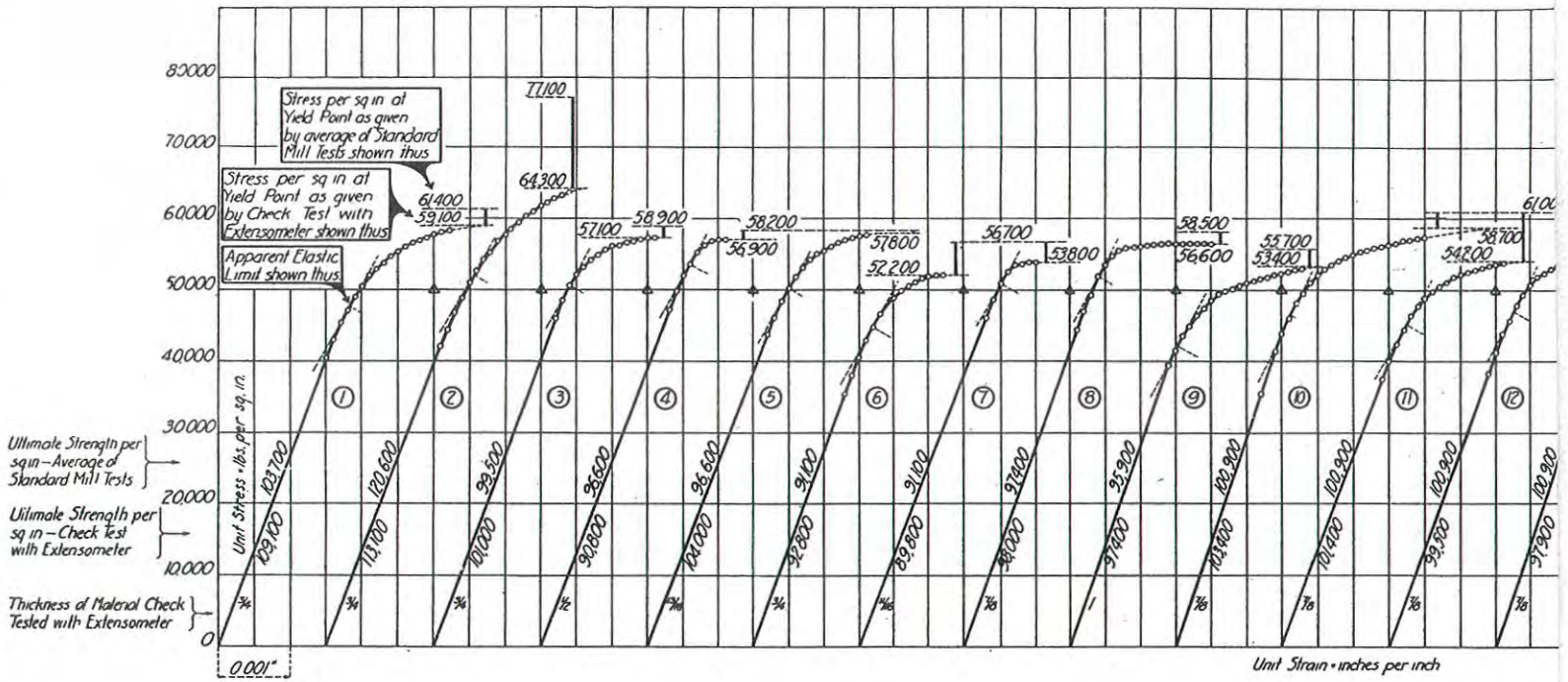
Mill Test { Yield Point, drop of beam. 45,000 lbs. per sq. inch.
Tensile Strength 80,000 to 95,000 lbs. per sq. in.

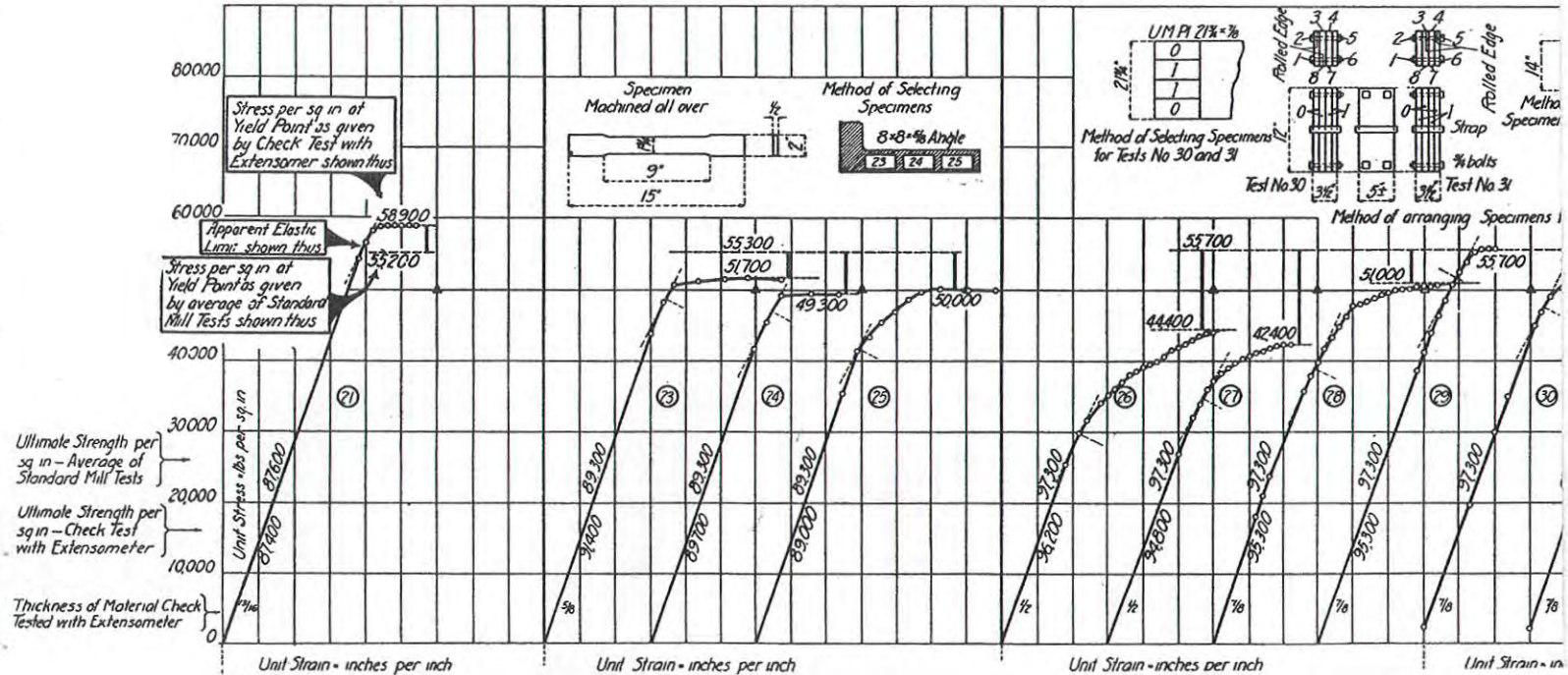
Extensometer Test (As specified in Contract No. 14) { Elongation not to exceed 0.25 of one per cent under a tension of 41,000 pounds per sq. inch. Critical point indicated thus Δ.

SILICON STEEL EXTENSOMETER CHECK TESTS

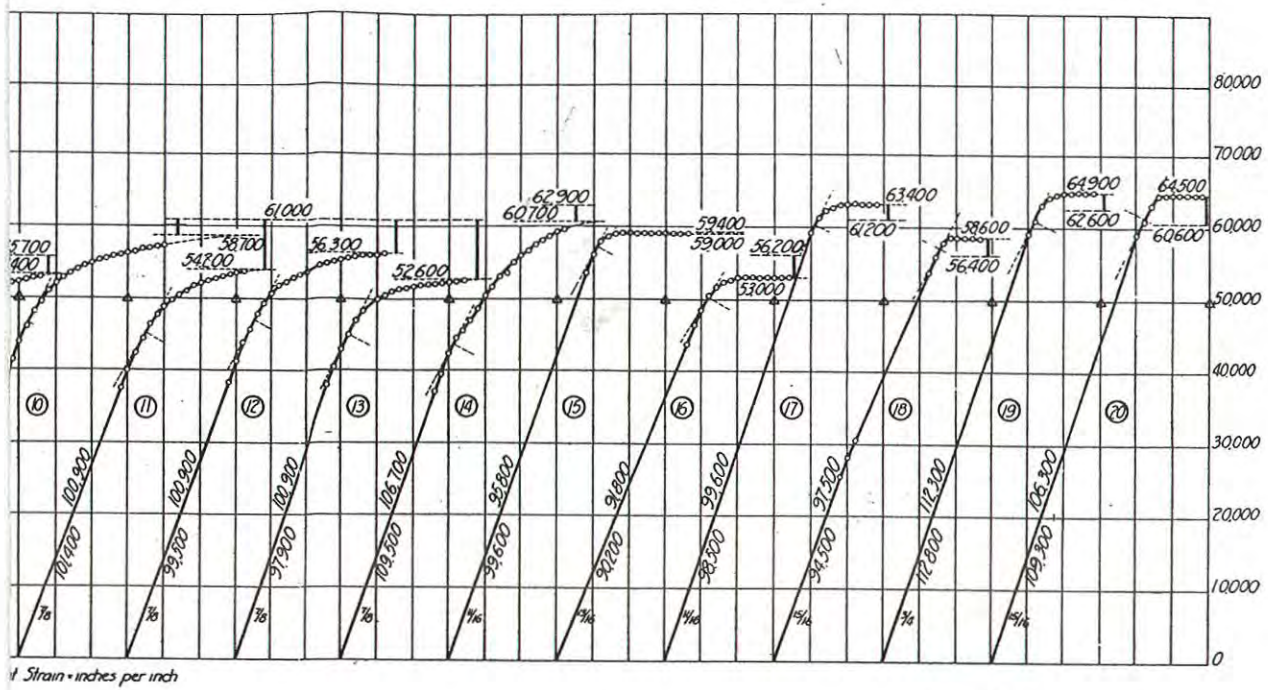
FIG. 1

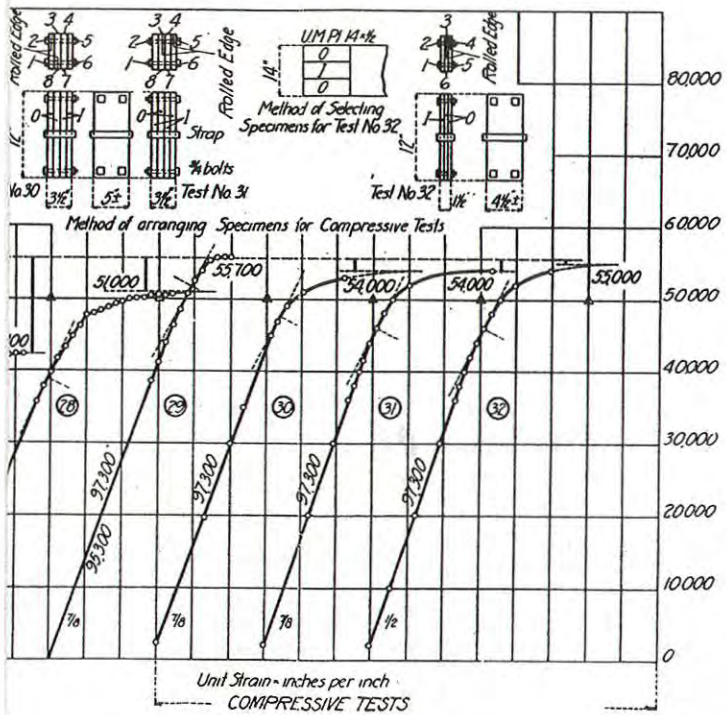
L-4581


Stress per sq in at Yield Point as given by average of Standard Mill Tests shown thus
Stress per sq in at Yield Point as given by Check Test with Extensometer shown thus
Apparent Elastic Limit shown thus
Ultimate Strength per sq in – Average of Standard Mill Tests
Ultimate Strength per sq in – Check Test with Extensometer
Thickness of Material Check Tested with Extensometer
Unit Stress = lbs per sq in
Unit Strain = inches per inch
0.001"


Stress per sq in at Yield Point as given by Check Test with Extensometer shown thus
Apparent Elastic Limit shown thus
Stress per sq in at Yield Point as given by average of Standard Mill Tests shown thus
Specimen Machined all over
Method of Selecting Specimens
8×8×⅝ Angle
Method of Selecting Specimens for Tests No 30 and 31
Rolled Edge
Strap
⅝ bolts
Test No 30
Test No 31
Method of arranging Specimens
Ultimate Strength per sq in – Average of Standard Mill Tests
Ultimate Strength per sq in – Check Test with Extensometer
Thickness of Material Check Tested with Extensometer
Unit Stress = lbs per sq in
Unit Strain = inches per inch

L-4587





SPECIFICATIONS

Mill Test	Yield Point-min drop of beam 55,000 lbs. per sq. in Tensile Strength-min 90,000 lbs per sq in
Extensometer Test	Elongation not to exceed 0.30 of one percent under a tension of 50,000 lbs per sq in Critical point indicated thus △

NOTES

Material used in Tests No 4 and 5 from same heat
" " " " 6 and 7 " " "
" " " " 10, 11, 12 and 13 " " "
" " " " 23, 24 and 25 " " "
" " " " 26 – 32 (incl) " " "

Apparent Elastic Limit is point on stress-diagram at which the rate of deformation is 50 % greater than at its origin (Materials of Construction.-Johnson

Tests No 30, 31 and 32 are average results of readings taken at instrument stations, numbered on Top View.

Test No 28 and 29 are retests for No 26 and 27 as are compressive tests No 30, 31 and 32, the latter three of which were made at the U. S. Bureau of Standards

Tests No 23, 24 and 25 at the University of Illinois

All tests excepting Nos 23, 24, 25, 30, 31 and 32 made at the Westinghouse Electric and Manufacturing Co. of Pittsburgh on machine with automatic stress strain recorder

Coupons for Tests No 1 to 7 incl and 26 to 29 incl sheared out of material. Resulting bending of coupon caused defective tests (26 27)

Coupons for Tests No 8 to 21 inclusive burned out of material with liberal allowance for finishing

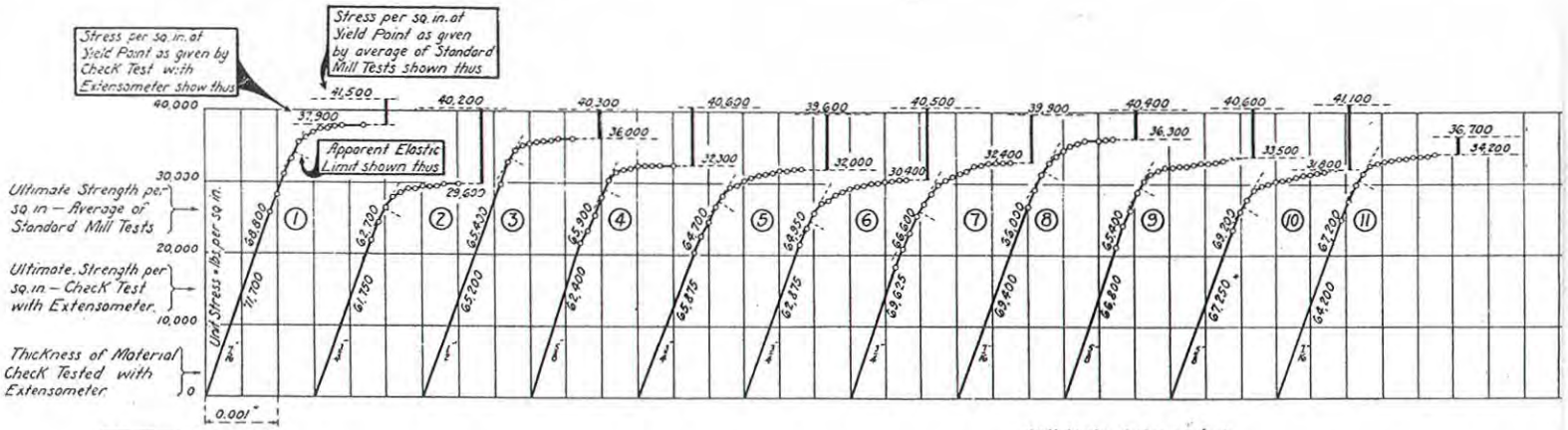
Strain readings taken on 2" gage length except for Tests No. 23 24 25, 30, 31 and 32 where gage length of 8" was used

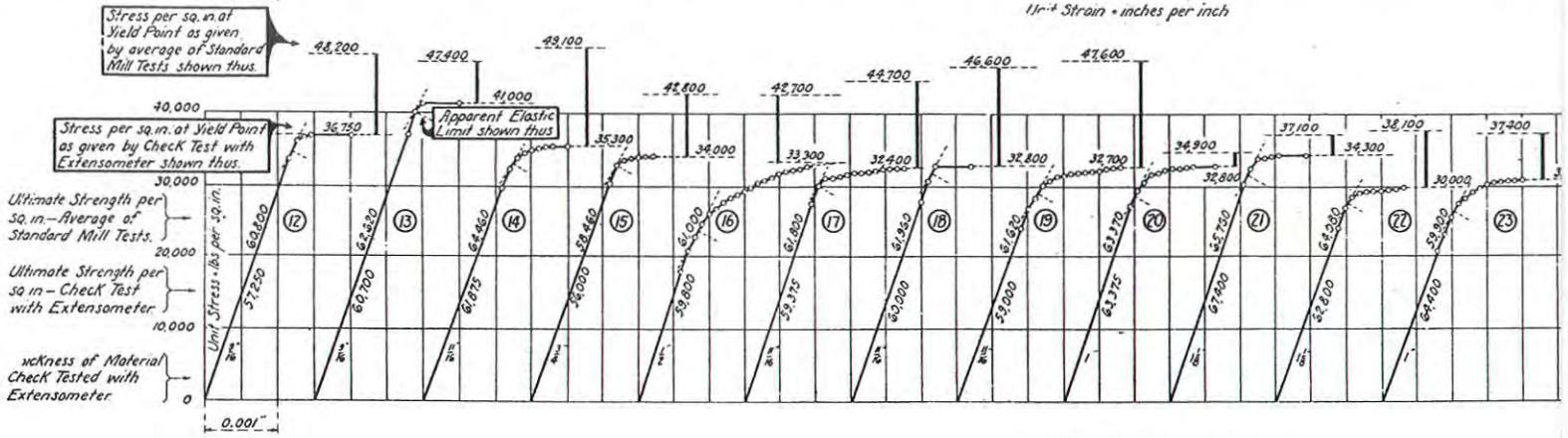
Specimens for all tests represents material on Contract No. 14

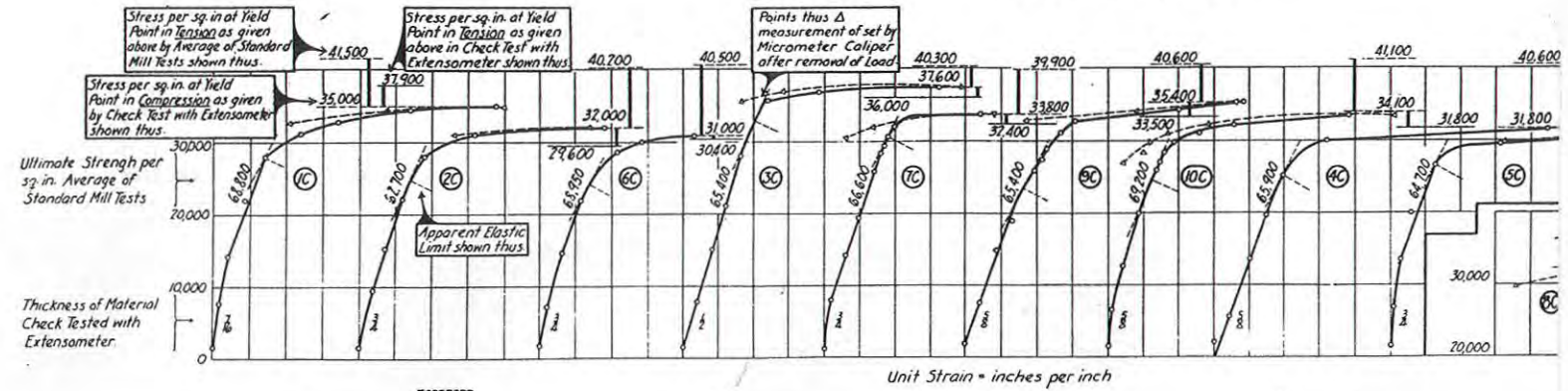
NICKEL STEEL EXTENSOMETER CHECK TESTS

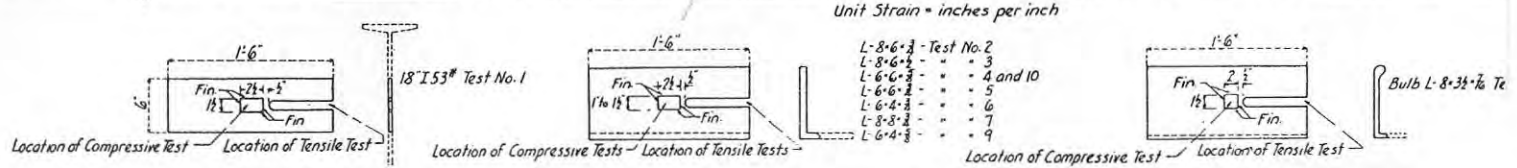
FIG. 2

T-4582

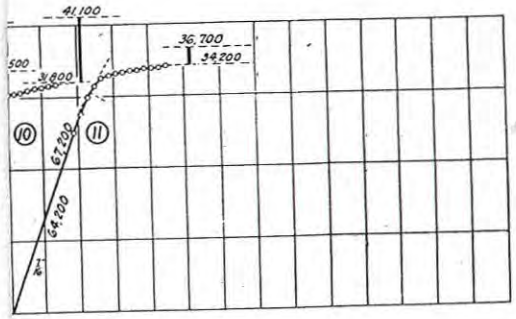

Stress per sq. in. at Yield Point as given by Check Test with Extensometer show thus
Stress per sq. in. at Yield Point as given by average of Standard Mill Tests shown thus
Apparent Elastic Limit shown thus
Ultimate Strength per sq. in. – Average of Standard Mill Tests
Ultimate Strength per sq. in. – Check Test with Extensometer
Thickness of Material Check Tested with Extensometer
Unit Stress = lbs. per sq. in.
0.001"
Unit Strain = inches per inch


Stress per sq. in. at Yield Point as given by average of Standard Mill Tests shown thus.
Stress per sq. in. at Yield Point as given by Check Test with Extensometer shown thus.
Apparent Elastic Limit shown thus
Ultimate Strength per sq. in. – Average of Standard Mill Tests.
Ultimate Strength per sq. in – Check Test with Extensometer
Thickness of Material Check Tested with Extensometer
Unit Stress = lbs. per sq. in.
0.001"
Unit Strain = inches per inch

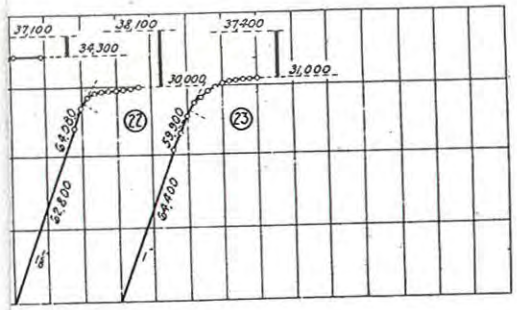

Stress per sq. in at Yield Point in Tension as given above by Average of Standard Mill Tests shown thus.
Stress per sq. in. at Yield Point in Tension as given above in Check Test with Extensometer shown thus
Points thus Δ measurement of set by Micrometer Caliper after removal of Load
Stress per sq. in. at Yield Point in Compression as given by Check Test with Extensometer shown thus.
Ultimate Strength per sq. in. Average of Standard Mill Tests
Apparent Elastic Limit shown thus
Thickness of Material Check Tested with Extensometer
Unit Strain = inches per inch


1'-6"
Fin.
Fin.
Location of Compressive Test
Location of Tensile Test
18" I 53# Test No. 1
Location of Compressive Tests
Location of Tensile Tests
L-8·6·3/8 - Test No. 2
L-8·6·1/2 - " " 3
L-6·6·3/8 - " " 4 and 10
L-6·6·1/2 - " " 5
L-6·4·3/8 - " " 6
L-8·8·3/8 - " " 7
L-6·4·3/8 - " " 9
Location of Compressive Test
Location of Tensile Test
Bulb L-8·3½·7/16 Te

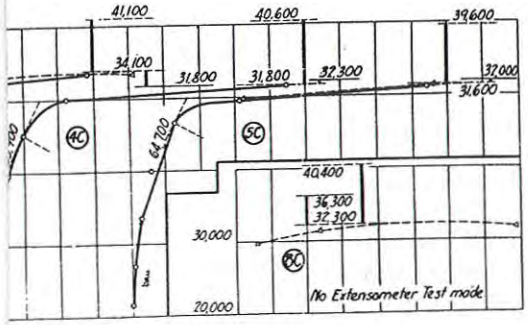
L-4583



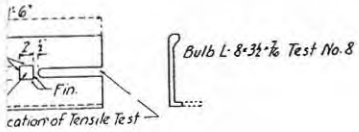
MEDIUM CARBON STEEL
TENSILE TESTS



MILD CARBON STEEL
TENSILE TESTS



MEDIUM CARBON STEEL
COMPRESSIVE TESTS



SPECIFICATIONS

MEDIUM CARBON STEEL

Mill Tests { Yield Point, drop of beam — 37,000 lbs. per sq. in.
Tensile Strength — 62,000 to 70,000 lbs. per sq. in.

MILD CARBON STEEL

Mill Tests { Yield Point, drop of beam — Not specified.
Tensile Strength — 55,000 to 65,000 lbs. per sq. in.

NOTES

Apparent Elastic Limit is point on stress diagram at which the rate of deformation is 50 per cent greater than at its origin. (Materials of Construction:- Johnson)

All tensile Tests made at Westinghouse Electric and Manufacturing Co. of Pittsburgh on a machine with automatic stress strain recorder.

Compressive tests made at Lafayette College.

Compressive tests (suffix C) cut from material adjacent to tensile tests bearing same number.

Specimens for Tests No. 1 and 8 represent material of Contract No. 14.

Specimens for Tests No. 2 to 7 inclusive and 9 to 11 inclusive represent material of Contracts No. 17 and 18 jointly.

Specimens for Tests No. 12 to 23 inclusive represent material of Contract No. 23.

Coupons for Mild Carbon Steel Tests were burned out of material with liberal allowance for finishing.

Coupons for both sets of Medium Carbon Steel Tests machined out of material.

CARBON STEEL
EXTENSOMETER CHECK TESTS

FIG. 3

L-4583

REPORT ON COLUMNS

BUREAU OF PUBLIC ROADS

Washington, D.C.

August 1, 1924.

Submitted by
John I. Quinn.

L-3845

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Collapse Steel Frame of Theater		L-3877
Columns	Basquin	L-3877
Ultimate Strength of Carbon Steel Models of Quebec Bridge Members		L-3877
Column Tests	A.R.E.A.	L-3878

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Appendix (Continued)

Report on Columns - - - - - - - - - - - - - A.R.E.A. L-3879

Final Report on Steel Columns and Struts - A.S.C.E. L-3883

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Compression Tests of Structural Steel Angles - - - - Bureau of Standards L-3888

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Tests - - - - - - - - - - - - - - - - - - - Tetmajer L-3890

Report

(1) In some of the Standard Designs for Pony Trusses submitted to the Bureau for approval a section has been used for main compression web members of the general form shown in Fig. "a".

Fig. a

The question has been raised as to the efficiency of a section of this kind; whether members so tied together with batten plates only, will act as a unit, and whether a section could not be substituted that would be as economical as this section, even with the most favorable assumptions, or at the most at only slightly greater cost.

(2) It was felt that the best means of settling these questions was through the experimental work done on columns by various investigators. To that end a search was made of all available experimental data on columns. The results of that research are abstracted in the appendix attached to this report. Much of the data so abstracted has no bearing on the questions at issue. The information, however, was noted so that in the future it would furnish a reference for the solution of other column problems that might arise. The comments accompanying the abstracts

are as closely as possible the language of the authors.

(3) These experiments include all of the more important work of American investigators along this line. They include also one series of late German experiments - the only foreign series available. Highly valuable work has been done by Tetmajer, more especially on columns or struts of light section. His results, however, are only incompletely available in two graphs taken from an article by Marston in the Trans. A.S.C.E., Vol XXXIX, p. 111.

(4) The search revealed, however, that no work had been done on a strut of this section. An incomplete answer to the questions in the first paragraph of this report can therefore only be obtained by studying results which may reasonably be construed as having a bearing on the question at issue.

(5) This study had the following aims in view:

(a) The efficiency of batten plates in general as shown in tests of columns of a section other than that primarily treated in this report.

(b) The unity of action of columns of more compact section as determined by experiment under load so that we may roughly judge the unity of a section of this kind.

(c) A study of column sections to determine the most efficient section that could be adopted to take care of the stresses which members of this kind would be called upon to carry.

(d) The relative economy of the section shown in Fig. "a" with proposed alternates.

(6) Under (a) of the preceding paragraph the best information is found in the "Report on Columns" in the Proceedings of the A.R.E.A., Vol. 19, p. 789. This committee tested twenty-four columns; fifteen

of which were composed of two channels, 8" at 16 1/4 lbs. and nine of two channels, 8" at 21 1/4 lbs. Half of the columns were tied together with lacing and half with batten plates. The detailed reports of the tests are shown on page L-3879 of the Appendix.

(7) The table below has been computed from the above-mentioned tests and it shows quite conclusively for the tests in question that the battens did not develop at all the efficiency of the column.

Table I

Area	l/r	Ave. Ult. Strength Laced	Ave. Ult. Strength Batten	Min. Ult. Strength Laced	Min. Ult. Strength Batten	Max. Ult. Strength Laced	Max. Ult. Strength Batten	% of Batten to Laced
9.56	85	33800	26150	33623	23987	34165	27823	77.4
12.50	85	31800	24000	31000	23039	32353	24893	75.5
9.56	120	33070	15090	32000	15000	34214	15200	45.6

(8) It should be noted also in the photographs - see pages L-3881 and L-3882 of the Appendix - of the failures accompanying the above-mentioned report, that the battened columns failed in a plane parallel to the batten plates and the laced columns in a plane perpendicular to their lacing. In the first case this failure was perpendicular to the axis of the greatest radius of the member considered as a whole and in the second case perpendicular to the axis of the least radius as it should be. This taken in conjunction with the rest of the tests would show that the batten plates did not effectively tie the component members together.

(9) The report of the Committee further states on page 793, that "columns in which batten plates are substituted for lacing bars will not develop the full strength of the section and should not be used."

(10) While it may be pointed out that the distance between batten plates in the columns tested by the above-mentioned Committee were greater than generally used in the angle struts under consideration still it is probable that the shear would be more efficiently taken care of with the symmetrical bracing of two plates as used in the channel columns than with the unsymmetrical bracing of one plate in the angle struts even with the greater spacing in the first instance.

(11) Turning next to the question of unity of action of the section treated in this report, we may quote the following from Salmon's work on "Columns," page 184:

> "A very large proportion of the columns used in practice are built up of separate sections riveted together. It is usually taken for granted that such columns behave as solid specimens provided the web and the riveting be of sufficient strength, and formulae for solid columns are commonly applied to them.
>
> "Now if one point be clearer than any other with regard to built-up columns it is that they do not act as solid or homogeneous columns. Unless improperly designed they always fail locally by the flange buckling between the panel points and it is their strength in this connection which determines their strength as a whole. Hence, it follows that all rules and deductions based on assumptions of solidity or homogeneity are beside the question.
>
> "Two noteworthy experimental analyses of the strains in built-up columns have been published in America, one by Talbot and Moore (1909), the second by Howard (1911). In drawing deductions from the first it should be borne in mind that the specimens were abnormal. The majority were old compression members cut out of a bridge and subjected without doubt to severe treatment during the process. The new specimen was intentionally built of thin material. Nevertheless the results, though for this reason probably exaggerated, are sufficiently remarkable.
>
> "It was found that the stress varied considerably over the length of the specimens, making it doubtful whether the

component parts of a built-up column act together to form an integral compression member. There were indications of stresses in the extreme fibres from 40 to 50 percent in excess of the average stress and in some cases even higher. In a compression member of an actual bridge the maximum stress was 73 percent in excess of the mean. The maximum stress would occur at one cross section in the extreme fibres on one side of the channel, at a nearby section on the other side. These irregularities appear to have been due to local flexure for the authors record that the channels forming the flanges showed evidence of considerable local flexure, due apparently to initial want of straightness. They conclude in fact that want of straightness of the center line of the column and eccentricity of loading may have much less effect than local want of straightness. In an actual bridge secondary bending and twisting due to a cross girder may even have more effect than any of the above imperfections. In a U-shaped girder flange the addition of the top or bottom plate tends to reduce the local irregularities and the stresses were found to be more uniform.

"In the second experimental study of the stresses in a built-up column by Howard and Buchanan (1911) the specimens were more normal. Careful measurements of the strain in various parts of the column were made. The ultimate strength of the specimens was 30,490 lb. sq. in. For a range of load from 220 to 8,817 lb. sq. in. the longitudinal contraction measured in a gauged length of 150 in. along the middle of the flange plate was 0.0408 in. The corresponding readings at the edges were 0.0394 and 0.0410 in., the mean of which is 0.0006 less than the observed contraction of the centre gauged length. The readings at the opposite corners were 0.0467 and 0.0485 in. respectively, the mean of all four corners being 0.0439. The contractions in local gauged lengths of 10 in. were measured at different parts of the column. On the flanges these varied from .0026 to .0033 in. In stepping from the pin plates to the flanges, the contractions varied from 0.0033 to 0.0043 in. indicating a movement of the pin plates along the columns. On the pin plates themselves the maximum contraction was observed directly in front of the pin. Abreast of this place, toward the edges of the plate, the stress became tensile.

"It is, in fact, as all experiments show, the local strength to resist buckling of the individual parts which determines the strength of built-up columns, and if the

web be sufficiently strong to resist the shearing force, the built-up column always fails by the flange crippling between the panel points."

(12) The tests commented on above are abstracted in the appendix. See pages L-3866 and L-3871.

(13) In Technologic Paper No. 101 of the Bureau of Standards are given the results of tests on 18 large columns. The results of the tests on member US_2MS_1 as to the distribution of stress over the cross section are instructive. The steel in this column was Mayari Steel with a mean yield point of 62,430 pounds per square inch. The member was composed of 1 cover plate 27 x 7/16, 2 web plates 18 x 1/2 and 6 angles 3 x 3 x 3/8, laced with 3 x 1 3/4 x 1/4 angles; the length was 15'-10" and the slenderness ratio 19.4. From Table 17, page 5, of this paper we note the cumulative elongation at the center for a unit load of 20,000 pounds per sq. in. ranges from -.0040 to -.0059 indicating a considerable variation in stress across the section of the column.

(14) In view of the experiments noted above it would be a reasonable conclusion that the stress variation in the section, shown in Fig. "a" would be extreme. More especially would this be true when we consider that the columns experimented upon were far more compact than the one treated in this report. In fact it would only be a fair assumption to consider that this latter member would act as two separate pieces rather than as a unit, with the further likely probability that either one of the pieces would carry more than fifty percent of the total load.

(15) The problem now then is to find a column of compact section in which we may expect reasonable unity of action, and at the same time would be economical for the small loads and short lengths of web members of Pony Trusses.

(16) To this end we may turn to Plate XXXVI, see page L-3884 of the Appendix - in which are summarized the results of the Committee on Columns and Struts of the A.S.C.E., Trans. Vol. LXXXIII, p. 1583. It will be at once noted from this graph that the columns of the light Bethlehem H section developed the greatest strength of the columns tested. Column No. 1 of that series of tests composed of 4 angles and a plate also showed up very well.

(17) In the table below are compared the relative weights of the section shown in Fig. "a" taken from actual standards submitted for approval with an alternate Bethlehem H Section and a built H section.

Table 2

Truss	Section as submitted				Alt. Beth. Sec.		Alt. Built "H" Sec.	
Span	Lt. Mem.	Stress	Section	Wt.	Section	Wt.	Section	Wt.
50'	11.20	54.00	2Ls-5x3x5/16	179	8" B.G. at 33#	322	4Ls-3x3x5/16 1 Pl. 8x5/16	321
70'	12.55	93.00	2Ls-6x4x7/16	331	8 15/16" B.G. at 36#	381	4Ls-3½x3x5/16 1 Pl. 9x5/16	381
80'	14.14	103.00	2Ls-6x4x1/2	433	Do	450	4Ls-4x3x5/16 1 Pl. 9x5/16	479
90'	15.74	117.00	2Ls-6x4x5/8	598	10" B.G. at 45#	474	4Ls-5x3x5/16 1 Pl. 10x5/16	608

(18) This analysis shows that in the spans 80 feet and below the adoption of the Bethlehem Section would mean only a slight increase in weight and for the 90-foot span, a positive decrease. Considering the undoubtedly greater efficiency of the solid rolled section the very slight possible increase in cost hardly deserves any consideration.

(19) It is, therefore, in conclusion, the opinion of this report that the two-angle strut as used in the standard designs submitted are not efficient and that they are an element of weakness in a structure; that their use be discouraged in the future and that Bethlehem sections be adopted in their place; or if impossible to secure this section at a reasonable cost the use of the alternate built H section is advised.

APPENDIX

The Strength of Wrought Iron Columns
by G. Bouscaren
Trans. A.S.C.E., Vol. IX

These experiments have enabled me to accomplish the object for which they were instituted, viz., the proportioning of compression members built by different contractors with the same margin of safety and have led me to the following conclusions.

3d. It is essential in built columns to thoroughly fasten together the several parts or segments that they may act in a solidary manner as one continuous metallic body and to guard against lateral flexure or buckling of each part and also against longitudinal relative motion.

4th. A due regard to the economy of material demands that the thickness of metal and in built columns the spacing of the rivets shall be so proportioned that the column shall fail by flexure as a whole before any local buckling or flexure shall take place.

It was established that for that ratio of length to diameter the thickness of metal should not be less than 1/30 of the distance between supports transversely and that the distance between rivets longitudinally should be such that the length of channel spanning it considered as a column shall give the same resistance per square inch of area as the column itself.

Tests shown are largely of Phoenix and Keystone sections as well as box and I sections of wrought iron.

Tests of Metals
Watertown Arsenal, 1881

Tests of W. I. columns of lengths up to 31'-6" of box sections, I sections, and Phoenix sections. Areas from 2.33 to 17.65 sq. in.

Tests of Metals
Watertown Arsenal, 1883

Tests of Built-Up W. I. columns of various shapes. Areas from 7.621 sq. in. to 20.954 sq. in. Lengths up to 320 inches.

Tests of Metals
Watertown Arsenal, 1884

Tests of Built-Up W. I. columns of various shapes. Areas from 9.14 sq. in. to 21.49 sq. in. Lengths 10 feet to 25 feet.

Tests of Metals
Watertown Arsenal, 1885

Tests on W. I. latticed columns.

Tests of Metals
Watertown Arsenal, 1909

Tests on steel columns of various sections.

Tests of Metals
Watertown Arsenal, 1910

Tests on columns of carbon and nickel steel of I section.

Tests of Metals
Watertown Arsenal, 1912

Continuation of work of 1910.

Tests of Metals
Watertown Arsenal, 1914

Tests on steel columns of I sections.

"The Strength and Elasticity of Structural Steel"
by James Christie
Trans. A.S.C.E., Vol. 13, p. 254

No.	Shape	Section	l	l/r	A	Ultimate Resistance per sq. in.	
1	Angle	3 x 3 x 5/16	182"	303	1.9	7895	
2	Do	Do	169.4	282	1.9	9084	
3	Do	Do	132.1	220	1.8	14230	
4	Do	Do	84.4	141	1.84	25434	Note
5	Do	3 1/2 x 3 1/2 x 11/32	168.5	241	2.18	9525	Mild Steel
6	Do	Do	133.5	191	2.24	21101	Flat Ended
7	Do	Do	154.5	221	2.25	14321	Struts
8	Do	3 1/2 x 3 1/2 x 5/16	182.6	261	2.25	12364	
9	Do	2 13/16 x 2 13/16 x 5/16	182	324	1.86	5800	
10	Do	2 1/2 x 2 1/2 x 1/4	84.2	169	1.24	25560	
11	Do	Do	84.3	169	1.22	23176	
12	Do	Do	42.3	85	1.24	34443	
13	Do	Do	41.4	83	1.24	34390	
14	Do	Do	24.1	20	1.22	No fail with total load of 50000	
15	Do	2 x 2 x 1/4	24.3	61	.98	40803	
16	Do	Do	24.1	60	.97	44098	
60	Do	3 1/2 x 3 1/2 x 5/16	145	207	2.06	12238	
61	Do	Do	139.2	199	2.06	18835	
62	Do	2 1/4 x 2 1/4 x 11/32	89.2	198	1.45	20400	

"Experiments on the Strength of Wrought Iron Struts"
by James Christie
Trans. A.S.C.E., Vol. 13, p. 85

As tests were not on steel sections article is not summarized.

Experiments on Z-Iron Columns
by C. L. Stroebel
Trans. A.S.C.E., Vol. XVIII, p. 103

In the test columns 2 1/2" x 3" x 2 1/2" Z irons were used 5/16" thick. Six columns were tested of lengths 10'-11 1/4", 15', and 19'-3/4", two columns for each length.

Tests on specimens show reduction in area ranging from 15 percent to 24.80 percent
Elastic limit from 31230 to 36010 lb. per sq. in.
Max. load from 46240 to 54820 lb. per sq. in.
Extension from 11 percent to 21.50 percent

Section 4 Z bars. Laced: R = 2.05

No. Col.	Lt. Col.	l/r	Wt.	A	Ult. Str. per sq. in.
1	19'-03/4"	112	699	9 x 241	33800
2	15'-0"	88	570	9 x 48	34600
3	10'-11 1/4"	64	421	9.435	36800
4	10'-11 1/4"	64	441	9.984	34600
5	15'-0"	88	560	9.28	36600
6	19'-03/4"	112	754	10 x 104	33700
7	22'-0"	129	805	9 x 286	30700
8	22'-0"	129	805	9 x 286	29500
9	22'-0"	129	805	9 x 286	30700
10	25'-0"	146	900	9.156	28100
11	25'-0"	146	925	9 x 456	28000
12	25'-0"	146	930	9 x 516	28400
13	28'-0"	164	1025	9 x 375	27700
14	28'-0"	164	1050	9 x 643	28000
15	28'-0"	164	1025	9 x 375	27600

Some Experiments on the Strength of Bessemer Steel Bridge Compression Members
by James G. Dagron
Trans. A.S.C.E., Vol. XX - 1889, p. 254

The columns tested were eight of the following lengths between pin centers, viz., 16 feet, 20 feet, 24 feet and 25 feet 7 1/2 inches, two columns of each length being experimented upon.

The six shortest columns were made up as follows:

4 angles 2 1/4 x 2 1/4 x 1/4 inches . . .	=	4.24 sq. in.
2 plates 8 x 1/4 inches	=	4.00 sq. in.
		8.24 sq. in.

The two longest columns were composed of the following sections:

4 angles 2 3/4 x 2 3/4 x 5/16 inches . .	=	6.48 sq. in.
2 plates 9 x 3/8 inches	=	6.75 sq. in.
		13.23 sq. in.

The rivets were of soft steel 5/8" in diameter and power driven.

	Blow 8845		Blow 8847	
Elastic Limit in pounds per sq. in.	51190	53890	53140	53150
Ultimate Strength	84440	84750	83680	83760
Elongation in 8 inches, percent	18.75	18.75	20.75	20.2
Reduction in area, percent	33.23	30.55	37.78	39.2

No. of Col.	Area	Lt.	l/r	Ultimate Str.	Remarks
1	8.24	16'-0"	42.05	41020	Failed by bend. down at rivet in latticing 1'-10½" from ctr. buck.fl.angles & web pl.
2	8.24	16'-0"	42.05	41650	Failed by bend. up at riv. in lat. at ctr. buck. fl. ang. & web. pl.
3	8.24	20'-0"	52.564	39440	Fail by bend. up bet. latt. 3' from ctr. buck. fl. ang. & web pl.
4	8.24	20'-0"	52.564	41650	Fail by bend. up bet. latt. 4" from ctr. buck. fl. angles & web pl.
5	8.24	24'-0"	63.075	40230	Fail by bend. upward bet. latt. 9½" from ctr. buck. fl. ang. & web pl.
6	8.24	24'-0"	63.075	40070	Fail by bend. up bet. latt. 1'-5 3/4" from ctr. buck. fl. ang. & web pl.
7	13.23	25'-7½"	58.795	35570	Fail by bend. up at riv. in lat. 3' from ctr. buck. fl. ang. & web pl.
8	13.23	25'-7½"	58.795	38810	Fail by bend. up at riv. in lat. 1' from ctr. buck. fl. ang. & web pl.

Failure Tests of Full Size Compression Members
by C. P. Buchanan
Eng. News, 1907, p. 685

Describes tests to failure of W.I. and Steel Columns
from 1888 to 1907.

Only a few columns listed here.

Col. IX. Material, Bessemer Steel.

Section: 4 Ls - 6 x 3 1/2 x 3/8" laced
Single 2 x 3/8" bars

Shape I, length 25'0"

A = 14.22 sq. in. r = 3.1 l/r = 97

Failed near center, the outstanding legs of angles bucking.
Elastic limit 12600 lb. per sq. in.
Max. strength 27900 lb. per sq. in.

Col. XVI. Material, Open Hearth Steel.

Section: 2 Ls - 3 x 3 x 3/8"
2 Ls - 4 x 3 x 3/8"
1 cover pl. 22 x 3/8"
2 web pls. 16 x 3/8"
Lacing 5 x 7/16" single

A = 29.36 sq. in. r = 6.44 l = 24'-9 1/2" l/r = 46

Failed by crippling of lower flanges near center of chord.
Elastic limit 25500 lb. per sq. in.
Max. strength 30640 lb. per sq. in.

Col. XVII. Material, Open Hearth Steel.

Section: Box 2 webs 10 x 3/8"
4 Ls - 3 x 3 x 3/8"
Latticing, single-size not stated.

A = 15.8 sq. in. r = 3.72 l = 14'-0" l/r = 45

Failed by crippling of flanges near center.
Elastic limit 19000 lb. per sq. in.
Max. strength 31680 lb. per sq. in.

Col. XVIII. Material, Open Hearth Steel.

Section: Box 1 cover pl. = 22 x 3/8"
2 L^s - top 3 x 3 x 3/8"
2 L^s - bot. 3 x 3 x 1/2"
2 web plates 18 x 3/8"
Latticing, single, 2 1/4 x 7/16"
Length 20'-9"

A = 32.11 sq. in. r = 7.29 l/r = 34

Failed by crippling lower flanges near center.
Elastic limit 22600 lb. per sq. in.
Max. strength 31890 lb. per sq. in.

Col. XIX. Material, Open Hearth Steel.

Col. essentially same as Col. XVIII.
Failed by crippling of top flange near pin plates at one end.
Elastic limit 18700 lb. per sq. in.
Max. strength 33950 lb. per sq. in.

On the following pages are shown photographs of typical failures of the box sections tested. It will be noted that the region of failure was near the end of the column where the lacing had been stopped to allow for the entering web members.

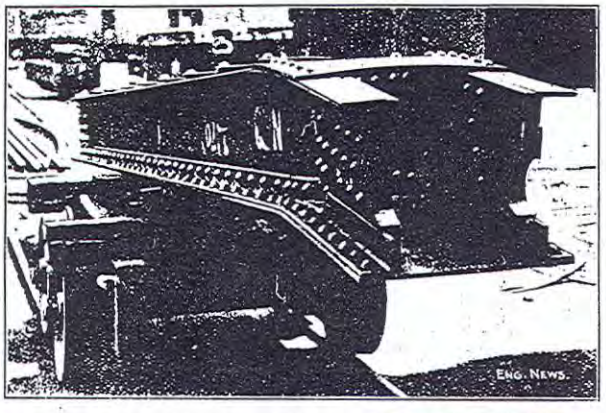


FIG. 7B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 1, T., W. V. & O. R. R.
Maximum load, 20,210 lbs. per sq. in.

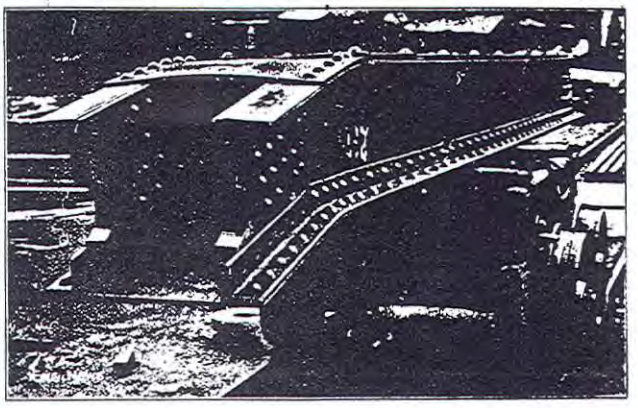


FIG. 8B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD BRIDGE 3, T., W. V. & O. R. R.
Maximum load, 20,900 lbs. per sq. in.

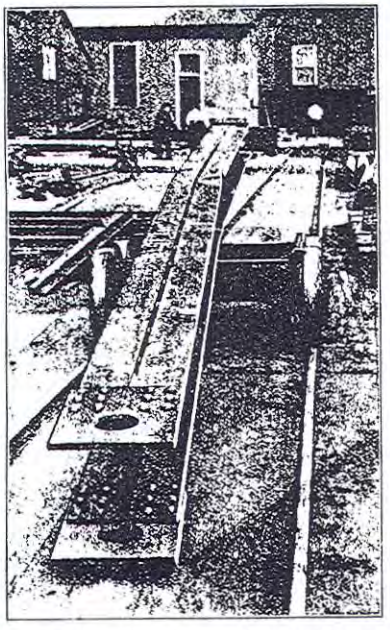


Fig. 9b. View After Failure; Steel Post, Bridge 17, T., W. V. & O. R. R.
Maximum load, 27,790 lbs. per sq. in.

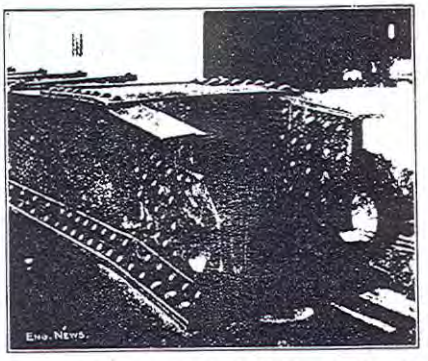


Fig. 10b. Tail-Block End After Failure; Wrought-Iron Top Chord, Bridge 18, T., W. V. & O. R. R.
Maximum load, 26,200 lbs. per sq. in.

Typical Failures of Columns in Buchanan's Tests

L-3863

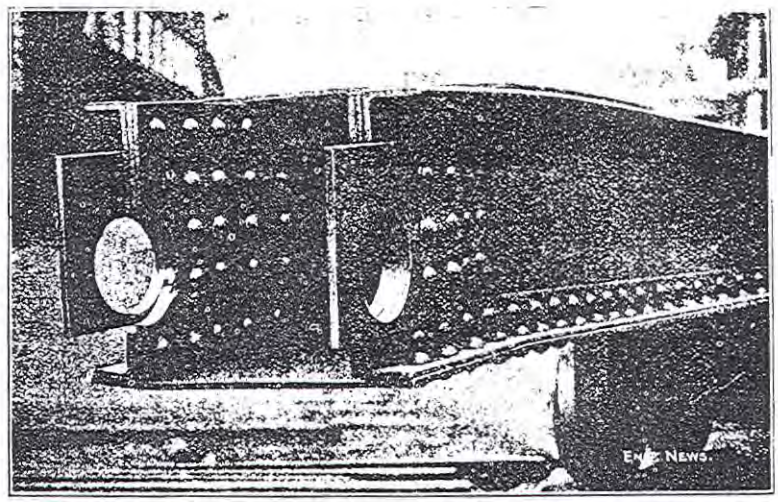


FIG. 5B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 75, CINCINNATI DIVISION, P., C., C. & ST. L. RY.
Maximum load, 23,200 lbs. per sq. in.

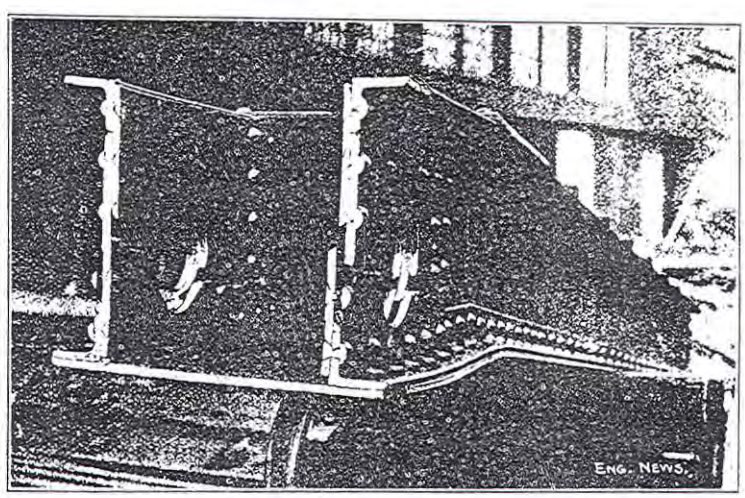


FIG. 6B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 25, C., R. & C. R. R.
Maximum load, 28,220 lbs. per sq. in.

Typical Failures of Columns in Buchanan's Tests

L-3864

Report of Royal Commission on Quebec Bridge
Eng. Rec., Vol. 57, p. 529

After the collapse of the bridge, the Phoenix Bridge Co., at its own cost and on its own initiative, built and tested "one" chord. This model chord had the same relative dimensions as the No. 9 chords of the Quebec Bridge. The test was made on November 21 and 22, 1907 under the direction of Professor W. H. Burr.

Nickel Steel for Bridges
by J. A. L. Waddell,
Trans. A.S.C.E., Vol. LXIII, p. 101

P. 246 - Twelve struts were designed for this test, three of nickel steel 10 ft. long, three of the same, 30 ft. long from center to center of pin holes and the same number of each length of medium carbon steel. Each was built of four angles 3 x 3 x 3/8 in. and two plates 12 x 3/8 in. laced on two sides with bars 2 1/2 in. and 3/8 in. and had an area in cross section of 17.44 sq. in. The ends were heavily reinforced to insure failure in the body of the strut.

Physical Properties

Heat No.	El.Lim.	Ult.Str	Elong. in 8 in.	Red. of Area	Heat No.	El.Lim.	Ult.Str.	Elong. in 8 in.	Red. in Area
Nickel Steel					Carbon Steel				
17673	67500	112600	16.5	50	41520	44200	66200	29.25	56.2
17749	-	-	-	-	45254	37100	61300	31.75	56.2
2116	58900	92800	19.5	28.4	60161	40700	60800	30.25	60.8
17065	68500	108600	18.75	41.3					
2192	56300	77600	25.50	51.3					

The short struts failed by the buckling of the angles and side plates, in four cases at the middle panel, and in two cases at the panel next to the middle.

The long struts failed by bending. The distortion of the pin hole was not appreciable.

Table 51. - Compression Tests

Col. No.	Steel	Lt.	Ult. Str. lb. per sq. in.	l/r
1	Nickel	10'	68500	26.5
2	Nickel	10'	68500	26.5
3	Nickel	10'	69200	26.5
1	Carbon	10'	38000	26.5
2	Carbon	10'	39800	26.5
3	Carbon	10'	38900	26.5
1	Nickel	30'	44400	79.5
2	Nickel	30'	47200	79.5
3	Nickel	30'	42500	79.5
1	Carbon	30'	30500	79.5
2	Carbon	30'	33300	79.5
3	Carbon	30'	30500	79.5

r = 4.53

"Tests of Built Up Steel and Wrought Iron Compression Pieces."
By A. N. Talbot and H. F. Moore.
Trans. A.S.C.E., Vol. LXV, p. 202.

The principal tests were made on the following compression pieces: (a) a steel column (called Column No. 1); (b) four wrought iron bridge posts which had seen long service in a bridge truss; and (c) three posts and a top chord in a railroad bridge under service.

These tests were taken up with a view of determining experimentally: (1) something of the variation of stress both longitudinal and lateral throughout the channel members of the compression piece; (2) something of the amount and distribution of stress in the lattice bars of columns and the action of similar bars under separate test; and (3) the general relation between the component parts of the column as a whole.

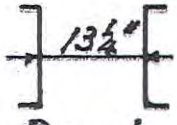
Data of Columns

Col. Designation	Col. No. 1	W. I. Posts		White Heath Bridge	
		Nos. 2,3,4,5	Retests 2a64a	Posts	Upper chd.
Area of col. in inch	18.76	17.64	17.64	12.02	48.67
Length cent. to cent.	21'-0"	15'-10"	14'-7"	25'	19'-10"
End conditions	Pin	Pin	Pin	Low. end pin up end riv.	Riveted
1/r axis par. lacing	37.8	43.5	40.1	66.1	40.7
1/r axis per. lacing	37.2	41.2	38.0	41.0	29.6
1/r of each fl. axis per. to lac. for full lt. col.	593	400	367	416	
1/r of each fl.mem.axis as before for dist. between adj.lac.rivets	37.7	33.7	33.7	22.2	
Latt. bars dimension of sect. in inches	1x1/4" & 1-1/4x7/16	2-1/2 x 3/8"	2½ x 3/8	2-1/4x3/8	2-3/4x3/8 on bot.
Kind of lacing	Single	Double	Double	Dou.riv. at cross	One cover pl.
Angle of latt.bar with axis of col.	63°30'	45°	45°	45°	45° on bot.

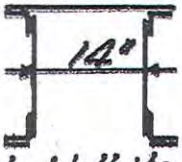
Col. No. 1

Section
2 pls. 20" x 3/8"
4 Ls-2"x2"x1/4"

W.I. Post
Cols. 2-3-4-5
2-10" Cs



Post
White Heath
Bridge
2-12" Cs



Upper Chd. White H. Brdg.
2 pls. 14" x 11/16"
4-Ls-3½" x 3½" x 7/16"
1 Cover Pl. 21" x 9/16"
2 fl. 4" x 3/4"

L-3866

P. 240 - "Does the built-up Column act as a Unit?" Engineers have often expressed doubt as to whether the parts of a built-up column act as a unit, although column formulas assume this unity of action. The tests threw some light on the question of the integrity of cross section under load. The individual channel, of course, acts as a unit to resist bending action, though there are indications of twisting. The integrity of the whole section with reference to a plane parallel to the lacing seems probable, except as twisting action results. With reference to a plane through the axis perpendicular to the plane of the lacing, this unity of action is not so certain. The tests on the distribution of compressive stress and likewise the cross bending tests indicate that these built-up columns did not in all cases act as a unit but rather as two columns not fully restrained by the lacing. The stresses in two channels at points in the same cross section do not give the regularity of variation which would exist if the column bent as a unit. The elastic curve assumed by Col. No. 1 under cross bending load, shown in Fig. 2, Plate XII, differs from the computed elastic curve, though that for the wrought iron columns give little difference. In the case of the posts of the White Heath Bridge, however, there is much closer agreement and a seemingly closer approach to unity of action.

"Excess of Max. Fibre Stress over Average Stress in Channel Members." The diagrams and data show that the compressive stress is unevenly distributed over the cross section of the columns listed, and also that there is great variation in the distribution at the various sections along the length of the column. It will be noted that in a number of sections the excess of stress was from 40 to 50 percent. In one test of Column No. 1 an excess of 67 percent was found and in the White Heath Bridge an excess of 73 percent. Possibly these values were unusual or the observations were erratic, but the indications of a fibre stress of from 40 percent to 50 percent in excess of the average stresses were not uncommon.

It may be seen that among the causes to which the high fibre stress may be attributed are: (a) non-straightness of the column as a whole, (b) non-straightness of the component channels, or eccentricity in the delivery of stress to them by the lacing, and (c) unknown eccentricity in the application of the load.

"Effect of Cover plates and End Connection." In the tests of the White Heath Bridge, the effect of the cover plates seems striking. The upper chord, U_3U_4 composed of two built-up channels with one cover plate gave an excess fibre stress of 20 percent at the worst section, while the posts composed of two channels laced on both sides gave a maximum of 73 percent. The high value in the posts may be due to other causes, but it seems reasonable to expect that the cover plate will act to reduce the irregularities in fabrication.

The connections of the ends of the posts evidently exerted a very noticeable effect on the stress distribution. In one of the posts tested, the stress was greatest at one corner of the post at the top and at the diagonally opposite corner at the bottom. It will be remembered that the posts were riveted to the top chord and were connected with the lower chords by pins. The floor beams were riveted to the side of the posts and this connection affects the stress distribution. Readings of deformation taken on the floor beams and posts show that the loaded beam was partly restrained at the ends by the post and that there was an appreciable bending in the post.

"Stresses in Column Lacing." It is evident from the tests that the relative stress in the two channel members varies considerably from end to end and that the stress in the lattice bars also varies.

The amount of transverse shear necessary to produce the maximum observed lattice-bar stress is of interest though, of course, it cannot be taken to be conclusive. The measurements were generally made at working loads. So far as measurements were made on columns tested to failure, the distribution of stress remained much the same up to incipient failure. The values indicate maximum average stresses in the bars such as would be caused by a transverse load ranging from 2 to 6 percent of the central compression load or of a transverse shear one-half as great.

Table No. . - Stresses in Column No. 10
Test No. 3

	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
Panel	Outer Fibre	Cent. Grav.	Inner Fibre	Outer Fibre	Cent. Grav.	Inner Fibre	Outer Fibre	Cent. Grav.	Inner Fibre	Outer Fibre	Cent. Grav.	Inner Fibre
1				13800	10400	9500	7500	8800	8900			
7	10400	10700	10700	11900	11000	10800	9200	10000	10000	8400	8800	8800
13	12600	9800	9500	8300	8400	8200	7200	9800	10200	11000	10800	10900
Column No. 3 , Test No. 7												
1	9300	9000	9000	8400	9500	10000	10000	9000	10100	8200	8000	7900
5	10200	10400	10500	8800	9600	9900	10100	9800	9700	10400	10200	10200
8	9800	9600	9500	10200	10900	11000	7600	9600	10300	7800	9900	10600
Stresses in Posts of White Heath Bridge												
Post U_3L_3 South												
1	5000	5300	5400	2600	2700	2700	4900	5000	5000	2300	2500	2600
6	3600	3600	3600	3400	3800	3900	3400	3400	3400	4100	3400	3200
13	1300	1500	1500	2800	2700	2600	2600	2900	3100	5200	4900	4800
Post U_3L_3 North												
1	4900	4900	4900	2600	2500	2500	4800	4900	4900	2100	2700	2800
6½	3300	3300	3300	3600	3500	3500	4500	4700	4800	2000	2400	2600

Table No. 9. - Stresses in Upper Chord at White Heath Bridge

Lower Side (Laced)					Upper Side (Cover Plate)				
North Channel			South Channel						
Panel	Outer Fibre	Inner Fibre	Outer Fibre	Inner Fibre	Dist. from end in in.	North Edge	Over North Web.Pl.	Over South Web.Pl.	South Edge
1	4300	3900	5000	5500	23	5500	6000	5200	5700
5	5700	5400	3900	4800	129	4800	5900	5900	5900
10	5900	5100	4400	4900					

Note by J.I.Q. - The readings above given are taken at random and are at both ends and center of column. It must be borne in mind that the observations are simultaneous only over one section at a time. The observations given at the center are not simultaneous with those at the ends nor those at the ends with each other. "The load generally used in the laboratory tests was 10000 lb. per sq. in. of section of the column in excess of the initial load. (Note - initial load was 'light')."

Table 7. - Results of Tests to Failure

Column No.	2	3	4	5	2a	4a
Test No.	6	7	8	9	13	10
Method of loading	Central	Central	Central	Central	Oblique Arm 2"	Central
Load at fail. lb.	466000	480000	452000	492000	475000	475000
Ave. stress at fail. lb. per sq. in.	26400	27200	25700	27900	26900	26900
Ave. stress at fail. in tests of short pieces of fl. members in lb. per sq. in.	36800	36800	36800	36800	36800	36800
Percent of excess of ultimate strength of short pieces over col. strength	39.5	35.5	43.5	32.0	37.0	37.0
Load at first sign of yield in lb.	317000	264000	256000	264000		
Ave. stress at first sign of yield in lb. per sq. in.	18000	15000	14500	15000		
Method of failure and remarks	End failed	End failed	End failed	Bowed at mid. in plane perpen. to lac.	Buckled near bot. in plane parallel to lac.	Bowed at mid. in plane perpen. to lac.

Table No. 9. - Stresses in Upper Chord at White Heath Bridge

Lower Side (Laced)					Upper Side (Cover Plate)				
North Channel			South Channel						
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1	4300	3900	5000	5500	23	5500	6000	5200	5700
5	5700	5400	3900	4800	129	4800	5900	5900	5900
10	5900	5100	4400	4900					

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Method of failure and remarks	End failed	End failed	End failed	Bowed at mid. in plane perpen. to lac.	Buckled near bot. in plane parallel to lac.	Bowed at mid. in plane perpen. to lac.

I. Variation of Stress Throughout the Length and Cross Section of the Channels and of the Columns as a Whole. - Channel members show evidence of considerable local flexural action, such as may be produced by lack of straightness or by any method of applying the load eccentrically. This is especially true in the flimsier column.

The condition of flexure varies markedly throughout the length of the member, in some cases the maximum compression in one cross section being at the extreme fibre on one side of the channel and in a nearby section the other side of the channel showing the excess of stress.

There were indications of sudden changes in the relative amount of stress carried by the two channels at nearby sections indicating general flexure of the column.

The measurements made indicate in a number of cases stresses in the extreme fibre from 40 to 50 percent in excess of the average stress and in some cases even higher.

The amount of eccentricity necessary to account for the increase of stress found in individual channels, based on lack of straightness and the ordinary theory of flexure is relatively small.

II. Stress in Lattice Bars. - The amount of deformation observed in lattice bars is relatively small.

The measurements indicate a stress in the lattice bars which would be produced by a transverse shear equal in amount to 1 to 3 percent of the applied compression load or that produced by a concentrated transverse load at the middle of the column equal to 2 to 6 percent of the compression load.

III. Test of Lattice Bars. - Tests of lattice bars for load carrying capacity show the usual form of bar is a very inefficient compression member when loaded eccentrically through a riveted connection.

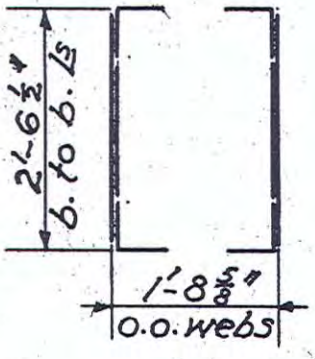
IV. Relation between Component Parts and the Column as a Whole. - It is a question whether the component members of a built-up column act together to form an integral compression piece especially in resisting oblique distortion.

The distribution of stress under working loads, and even up to incipient failure may be quite different from that which exists when the column becomes crippled. This is due to the yielding of the more strained parts after the yield point is reached at any fibre and the consequent redistribution of stress.

No relation has been found between the stresses actually observed and the stresses computed by column formulas.

Some Tests of Large Steel Columns.
By James E. Howard.
Trans. A.S.C.E., 1911, Vol. LXXIII, p. 429.

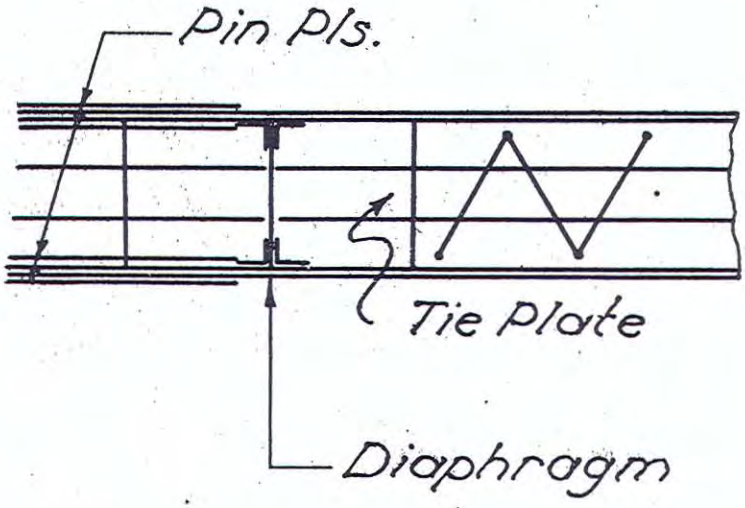
This paper describes the tests of five columns representing members of Bridge No. 42 of the Pittsburgh Division of the Pittsburgh, Cleveland, Chicago and St. Louis Railway over the Ohio River, at Steubenville, Ohio.



Section:- 4-Ls-6"×6"×⅝"
2 webs-30"×11/16"
2 webs-17½"×⅝"
Latt. Sing. 2½"×⅜"
Rivets ⅞" dia.
All holes subpunch 13/16" and reamed

Vert. Diaphragms - 4-Ls-3½"×3½"×7/16" } about
1 Web Pl. 18"×7/16" } 5'-6" ctrs.

Tie Pl. at ends 19"×⅜"×2'-10½"
Pin Pl. 30"×½"×2'-3¼" Outside
Pin Pl. 28"×⅝"×2'-3¼" Inside



L-3871

Table 1. - General Results of Tests

No. of Col.	Length Feet	Length Inc.	l/r	Sect. Area in square inches	Max. Loads App. Total in pounds	Max. Loads App. Lb. per sq. in.	
1	20	0	26.2	90.73	2,600,962	28667	Ult. Fail. not reach.
2	36	5	47.2	90.33	2,600,962	28794	Ult. Fail. not reach.
3	36	5	47.1	90.78	2,675,183	29469	Fail. locally at pin plates
4	36	5	47.2	90.32	2,726,815	30191	Fail. locally at pin plates
5	36	5	47.1	89.96	2,742,950	30490	Fail. by web pl. buck. in at dist. of 12'-8" from end

Table 3. - Results of Tests of Rivet Steel

Melt.	Pounds per square inch	
	Elast. Lim.	Tensile Strength
8174	35120	52580
12218	38430	53510
16283	33730	51370
18184	36320	50180

Table 4. - Maximum and Minimum Values of Elastic Limit and Yield Point in the Same Melt.

Melt.	Elastic Limit in lb. per sq. in.			Yield Points in lb. per sq. in.		
	Max.	Min.	Diff.	Max.	Min.	Diff.
Plate Metal						
6542	32250	28370	3880	38650	30940	7710
16266	36200	29600	6600	39200	33460	5740
21943	31300	30300	1000	35070	34440	630
Metal of the Angles						
24879	38100	30000	8100	38780	35950	2830
23983	37300	30200	7100	38810	32860	5950

If it is accepted that the elastic limit has a controlling influence on the ultimate strength of a column (as it is believed to have), then variations of from 25 to 27 percent in this value as witnessed in the tests of the metal of these plates and angles, would overshadow those considerations which find expression in empirical formulas for strength and take no account of such features.

The ultimate resistance of Column No. 5 rose about 1000 lb. per sq. in. above the elastic limit of the weakest plate but did not reach that having the next higher value. The reinforcing effect of the higher elastic limit metal of the angles and that of some of the plates was not adequate to raise materially the strength of the column. While a compact type of column might receive substantial benefit from its more rigid members, it would appear from the present example that such would not be the result ordinarily. It cannot be asserted by reason of lack of identity that the weakest plate in this column actually was the one which failed first but it seems reasonable to assume that it was.

The compression of the web plate of Column No. 1 on a gauged length of 150 in. established along the middle of its width was 0.0408 in. At the edges of this plate the strains were 0.0394 and .0410 in. respectively, the mean of which is .0006 in. less than the observed strain on the middle gauged length.

Corresponding measurements on the edges of the web plate on the opposite side of the column gave readings of 0.0467 and 0.0485 in. respectively.

The measurements which next followed were on gauged lengths of 10 in. each referring to local strains in the columns, over the same range of stresses as before.

The range of stress applied called for a compressive strain of .0029. On the web the observed strains were found to range from .0026 to .0033. The influence of the diaphragms and connecting angles, if any, could not be traced in the results.

The compressions of the pin plates ranged from a plus strain of .0040 to a minus strain of .0006 in. Directly in front of the pin the compression naturally reached a maximum while abreast this place at the edges of the plate, the minus readings indicated that the metal was in a state of tension.

Observations were made on four of the lattice bars but in none of them were any strains detected, the range of stresses applied to the column being as first mentioned.

Further tests were made on the compression of the metal in front of one of the pins. Rapid yielding occurred as the load was increased from 28000 to 33000 pounds per sq. in. referring to the stress on the bearing surface of the pin. The stress on the sectional area of the column was then only from 15000 lb. to 18000 lb. per sq. in.

On Figs. 3 and 4, Plate XXXIX, are recorded the permanent sets found on a gauged length of 10 in. each which resulted from applying a load of 28794 lb. per sq. in. to the column.

The mean set per 10 in. of length of web of this column was .0030. The individual sets are seen in range from .0006 to .0180 in.

It is known that in steel and iron shapes the parts first to cool after hot workings are left finally in a state of initial compression.

The mean unit stress could be well within the elastic limit of the member and yet local zones of overstrained metal might exist.

Initial strains induced by the rapid cooling of the shapes from the temperature of the rolling may be augmented by initial strains introduced by mechanical operations on the cold metal.

Straightening the shapes when cold introduces initial strains.

Punching and shearing affect the metal over a wide area.

Strains may be introduced by the use of the drift pin.

The contraction of the rivets in cooling after driving in the direction of their length is also a cause for the introduction of internal strains.

Column Failure Wrecked Gas Holder.
Eng. News, Vol. 66, p. 9.

On afternoon Dec. 7, 1909, a gas holder collapsed. The holder was new and had been in service for only a week or two.

One of the columns supporting the holder floor gave way so that the floor collapsed and that the failure of the column resulted from faulty design.

The struts were composed of two channels connected only by tie plates at intervals. At least one of them showed torn tie plates and sheared rivets which indications presumably led to attributing the primary failure to a particular strut.

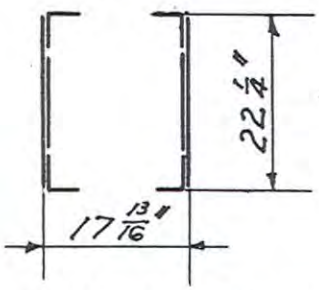
The ruptured member in question was 11.1 ft. long c. to c. of end connections and was designed to carry a maximum compression of 118,000. The city's expert estimated that at the time of collapse it may have borne a load of 133,000 lbs. taking account of all unfavorable circumstances.

This strut consisted of two 6.3 in. channels (German standard channel No. 16) back to back 1 in. apart. The combined sectional area of both channels was 7.4 sq. in. They were not latticed but merely tie-plated at intervals of 40 in. by plates 6 x 5/16 in. by 5-1/2 in. long; these plates were attached by four 13/16 in. rivets. If the column section be assumed to be integral, its least radius of gyration is 1.44 in.; the least radius of a single channel is 0.74 in.

It may be noted that the mean load per sq. in. of section at the time of failure was just 18,000 lb. per sq. in. For the length between tie plates (38 in. clear) l/r for one channel was 51. For the whole length of strut, the value of l/r was 93.

Bureau of Standards Report on Test of Steel Column
For the N.Y.C. & H.R. R.R.

Tested with flat ends not counterweighted; nom. sectional area 65.58, r = 7.06; l/r = 52.00
Length over all 30'-2"; weight in lbs. 8285
Sectional area actual in sq. in. 64.92



Section
2 pls. 22" x 11/16"
4 ∠s - 4 x 4 x 16.1 lb. = (4 x 4 x 5/8)
2 pls. 13-1/2 x 5/8
lacing 5" x 1/2" Sing.
Diaph. abt. 8'-0" ctrs.

Failed at 1,996,900 lb. = 30759 lb. per sq. in.

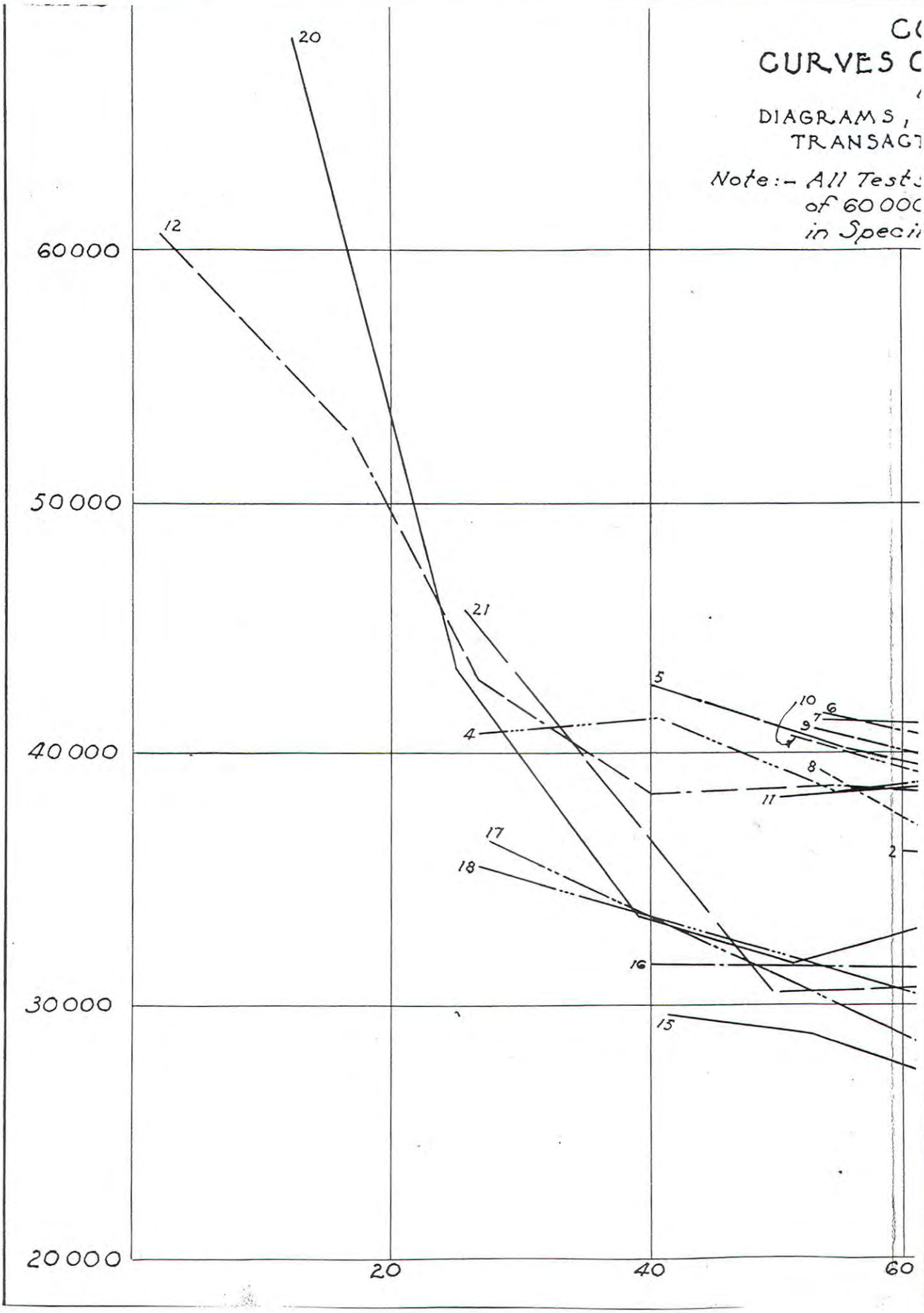
Progress Report on Steel Columns and Struts

Trans. A.S.C.E., Vol. LXVI, p. 401

Committee has examined records of numerous compression tests and has tabulated results in a number of graphs. On page L-3876 is shown a diagram summarizing all of the important American tests on Columns to the date of the report.

Explanation of Diagram Shown on Page L-3876

To reduce the results of the various series of tests compiled by the Committee to a uniform basis, the values have been adjusted to a grade of steel of 60000 lb. ultimate tensile strength in specimen tests, by assuming that the ultimate compressive strength will vary as the ultimate tensile strength. This method of adjustment probably does not accurately represent the law of variation of the different grades of iron and steel, still, as the adjusted value is a fair medium between the older iron and the newer steel it affords some basis of comparison.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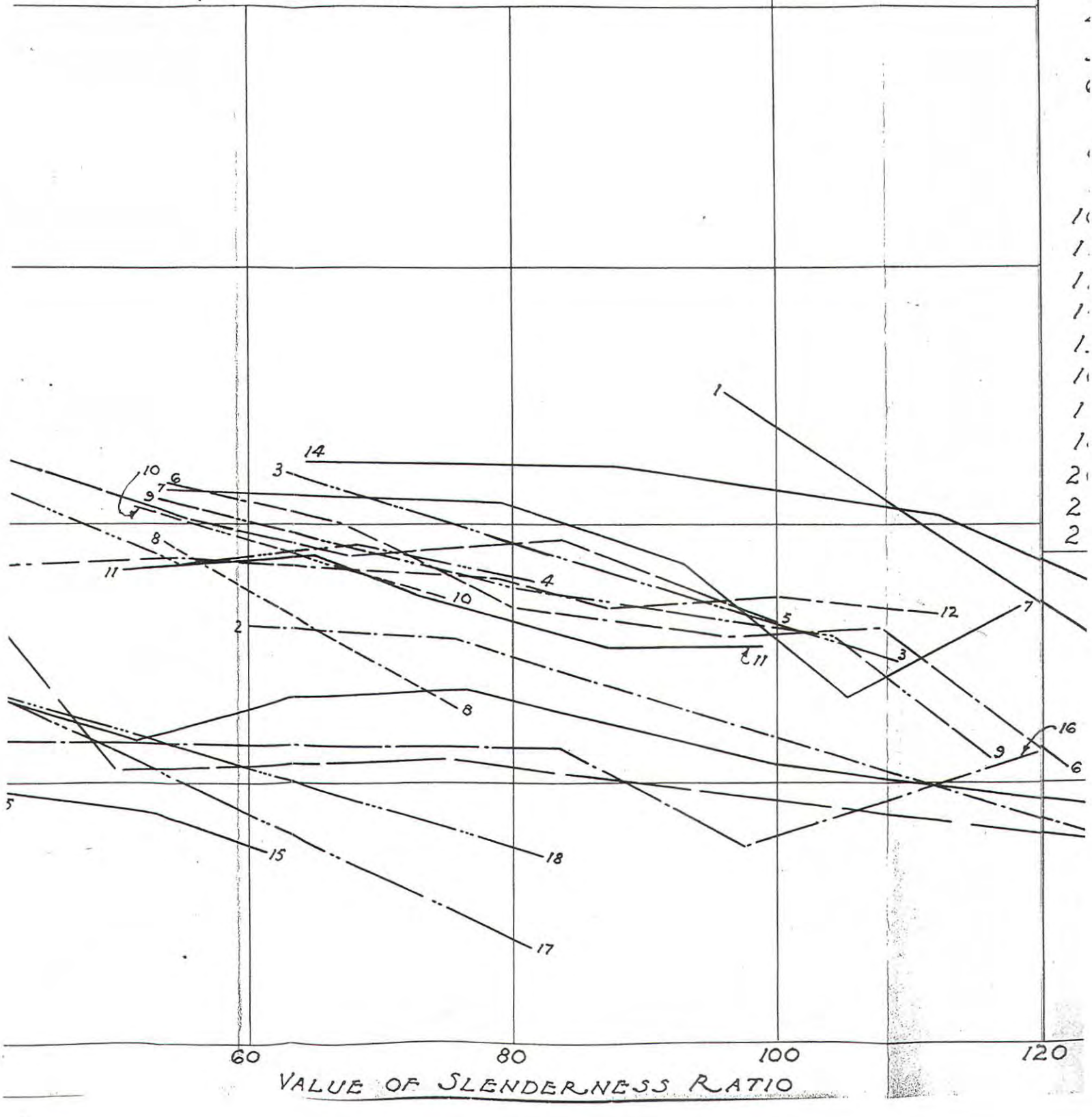
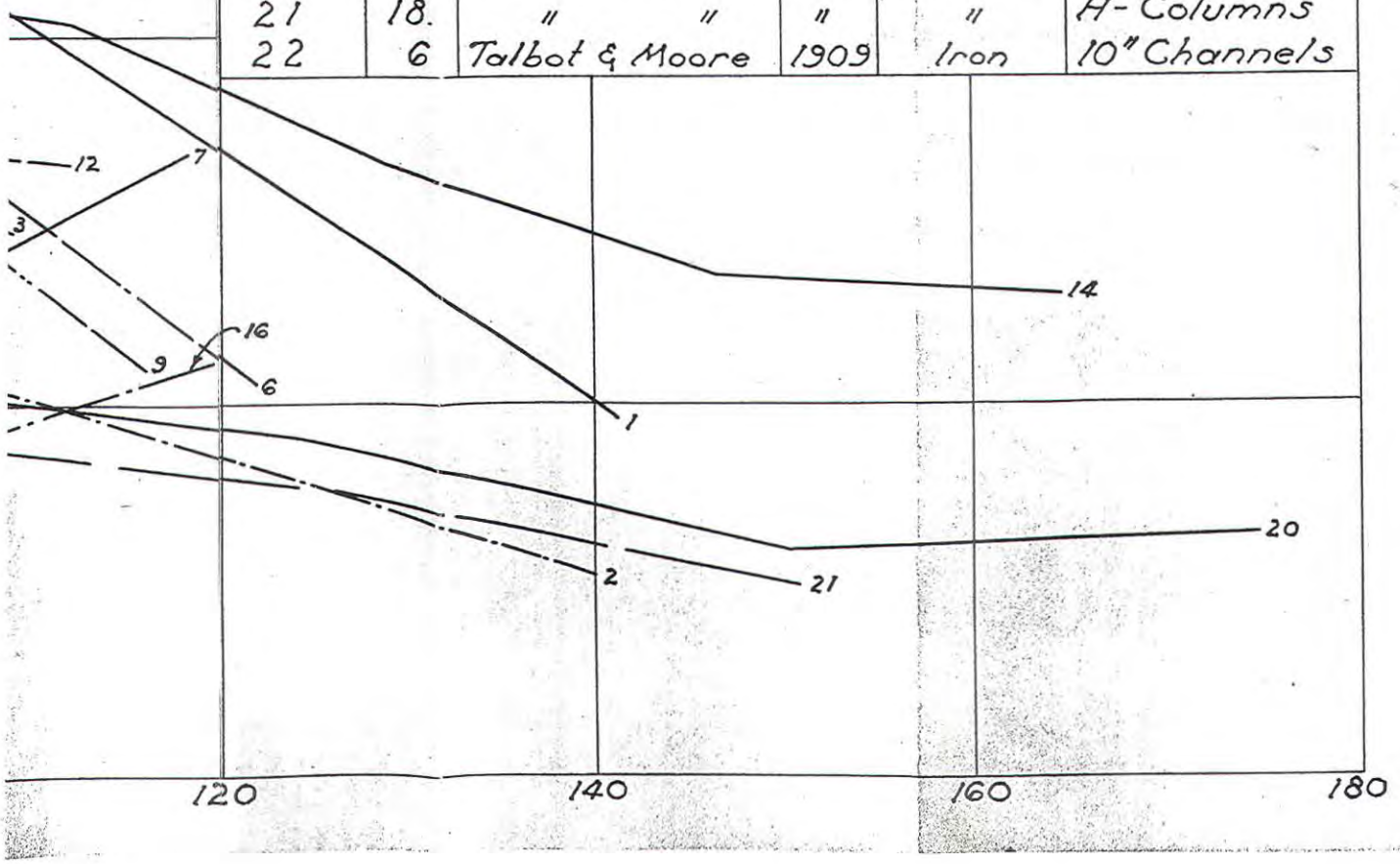
COLUMN TESTS
CURVES OF ULTIMATE FAILURE
AS INDICATED ON
DIAGRAMS, FIGURES 1 TO 22 INCLUSIVE,
TRANSACTION A.S.C.E. MARCH, 1910

Note:- All Tests Adjusted to a Grade of Steel of 60 000 Lbs. Ultimate Tensile Strength in Specimen Tests.



INDEX

Diagram Figure	No. of Tests	Name	Date	Material	Section
1	6	Phoenix Iron Co.	1873	Iron	Phoenix Cols.
2	9	G. Bouscaren	1880	"	Channel & Box
3	5	"	"	"	Phoenix Cols.
4	12	Watertown Arsenal	1881	"	12" Channels
5	12	" "	"	"	10"
6	14	" "	"	"	8"
7	17	" "	"	"	6"
8	12	" "	1883	"	10"
9	17	" "	"	"	8" [& Box
10	20	" "	1884	"	7" to 10" [& Box
11	12	" "	"	"	6" Box
12	22	Clark Reeves & Co.	1882	"	Phoenix Cols.
14	15	C.L. Strobel	1888	"	Z-Bar Cols.
15	8	J.G. Dagron	1889	Steel	Open Box
16	12	C.P. Buchanan	1897	Iron & Steel	Various
17	6	J.A.L. Waddell	1908	Nickel Steel	Open Box
18	6	"	"	Carbon "	" "
20	37	Watertown Arsenal	"	Steel	Welded Pipe
21	18.	" "	"	"	H-Columns
22	6	Talbot & Moore	1909	Iron	10" Channels



TAKEN FROM PROGRESS REPORT OF SPECIAL COMMITTEE ON STEEL COLUMNS & STRUTS
TRANSACTION A.S.C.E. MARCH 1910

Report on Collapse of Steel Frame of Theater
Eng. News, Vol. 71, p. 34

On January 22, 1913, theater in New York City under construction was wrecked by collapse of its roof.

The various opinions on the collapse differ radically and due to this obscurity the article is not summarized further here.

Columns
O. H. Basquin,
Western Society of Engineers - June, 1913,
Vol. XVIII, p. 457

Discusses tests of Wrought Iron Columns made at

Watertown Arsenal, 1881-1882-1883.

Ultimate Strength of Carbon Steel
Models of Quebec Bridge Members
Eng. Rec., Vol. 70, p. 110

The models of the new bridge members have been made and tested and may be considered as the latest development in compression members to resist excessively heavy stress in long span truss.

Inasmuch as sections are of a design not ordinarily under consideration in short span bridges the results are not tabulated here.

Also further tests on columns for Quebec Bridge noted on p. 541, Vol. 70, Eng. Record.

Column Tests
Proc. A.R.E.A., Vol. 16, 1915, p. 636

Cols. 1A, 1B, 1C. 2 Channels 8" at 16 1/4 lb., flanges turned in, single lattice, 2 1/4" x 3/8". Rivets 3/4" dia.
Cols. 1D, 1E, 1F. 2 Channels 8" at 21 1/4 lb., flanges turned in, single lattice, 2 1/4" x 3/8".
In both cols. bases both ends of 1 base pl. 16" x 5/8" x 1'-6"; 2 pls. 14" x 3/8" x 1'-6"; 2 angles 6" x 3 1/2" x 3/8" x 1'-6".
The specimen tension tests of the material of the column gave the following results.

Side plates, 22" x 11/16"
Yield point, 37800 lb. per sq. in.
Ultimate strength, 58760 lb. per sq. in.
Elongation in 8 in., 29 percent.
Reduction in area, 59.3 percent.
Side plates, 13 1/2" x 5/8".
Yield point, 37230 lb. per sq. in.
Ultimate strength, 58500 lb. per sq. in.
Elongation in 8 in., 27.5 percent.
Reduction in area, 58.8 percent.
Angles 4" x 4" x 5/8".
Yield point, 39270 lb. per sq. in.
Ultimate strength, 60850 lb. per sq. in.
Elongation in 8 in., 31 percent.
Reduction in area, 53.4 percent.

	No. Test Piece	l/r	Nom. Sect. sq. in.	Act. Sect. sq. in.	Fail lb. per sq. in.	Method of Failure
1	12	50	9.56	9.55	37930	
2	57	50	9.56	9.12	39254	Fail by deflect. south and up.
3	74	50	9.56	9.29	38200	Fail by deflect. south and down.
4	7	50	12.50	12.55	38500	Fail by triple flexure.
5	72	50	12.50	12.45	37765	Fail by buckling down.
6	73	50	12.50	12.48	35380	Fail by buckling down and N.
7	17	85	9.56	9.636	32860	Fail by triple flexure.
8	58	85	9.56	9.12	35000	Fail by triple flexure buck. in ctr.
9	61	85	9.56	9.38	34093	Fail by deflect. N. and up.
10	20	85	12.50	11.65	34060	Fail by triple flexure buck. in ctr.
11	60	85	12.50	11.83	34937	
12	62	85	12.50	12.45	34000	
13	29	120	9.56	9.42	33020	Fail by triple flexure buck. in ctr.
14	55	120	9.56	9.54	32000	
15	66	120	9.56	9.45	34000	Fail by deflect. S. and up.
16	25	120	12.50	12.28	32565	Fail by triple flexure buck. in ctr.
17	67	120	12.50	12.34	29000	
18	68	120	12.50	12.39	30863	

Summary of Reports on Columns
Proc. A.R.E.A., Vol. 19 - 1918, p. 788
Art. "Column Tests"

Report covers tests of twenty-four columns made during 1916 by the Bureau of Standards.

These columns are variations of Column No. 1, Series No. 1; ([ ] two channels with flanges turned in) the same general dimensions, area of cross sections, and slenderness ratios are used, the variations consisting of (1) changes in size of rivets and lacing bars; (2) the leaving off of wing and base plates; (3) the substitution of batten plates for lacing with varying distances between the batten plates.

Conclusions - p. 793.

(1) Columns in which batten plates are substituted for lacing bars will not develop the full strength of the section and should not be used.

(2) The specimen tensile tests on which material is ordered and accepted affords no proper criterion for the strength of a column.

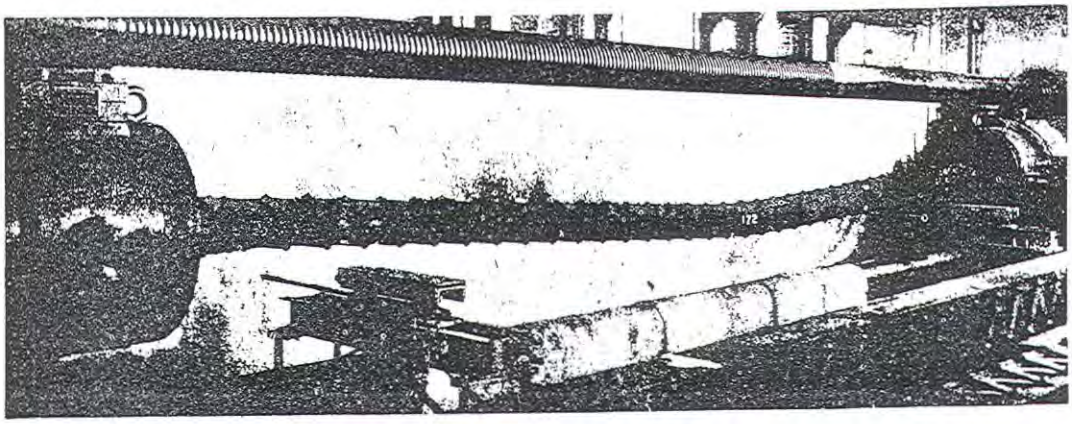
(3) A column designed so that it fails as a whole and not by reason of local weakness will have an ultimate strength of which the compressive yield point of the material of which it is made up is an index, since the higher this yield point is, the stronger will be the column.

P. 789 - It may be stated that the short specimen tests show that, in compression, the thin material has a higher elastic strength than the thick material, although the original specimen tests on which the material was accepted gave practically identical results as regards the elastic limit in tension for the thick and the thin material.

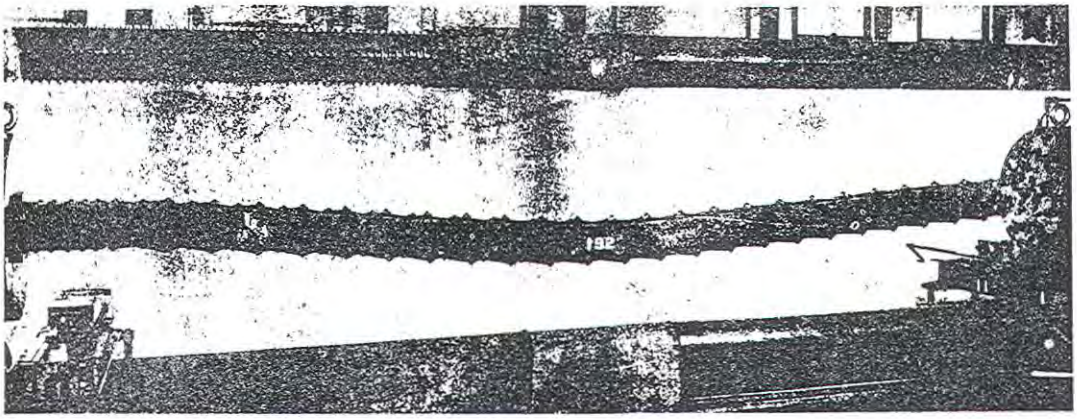
Table No. 1. - Column Tests
American Railway Engineering Association
Department of Commerce, Bureau of Standards, Washington, 1916

Type No.	Test No.	Section	l/r	Bracing	Ult. Str.	Remarks
1G	172)	[ ] 2[s 8"		2 x 1/4"	34214	
				sing. lac.		
1G	173)	at 16 1/4#	120	5/8" riv.	32000	
		A = 9.56				
1G	174)				33000#	
1H	191			(2 1/4 x 3/8"	33860	
1H	192	[ ] 2[8"		(3/4" riv.	33870	Failed after 10 min. under load
1H	197	at 16 1/4#	85	(Sing.	33850	
		A = 9.56		(lattice		
1N	199			(2" x 1/4"	33623	Sharp snapping at 29000# per sq. in.
1N	200			(5/8" riv.	33475	
1N	204			(Sing.	34165	
				(lattice		
10	201	[ ] 2[s 8"		2 x 1/4	32100	
10	202	at 21 1/4#	85	5/8" riv.	31000	
		A=12.50		(Sing.		
10	203			(lattice	32353	
1K	184	[ ] 2[s 8")		(& l/r for	27000	Failed under load in about 3 min.
		)		(indiv. [		
1K	193	at 16 1/4#)	85	(65.57	27823	Failed by deflecting south
		A = 9.56)		(		
1K	185	)		(Batten	26000	Failed after 10 min. under load
1I	186	)		(	27000	
1I	190	)		(83.93	25142	Col. was failing steadily after
		)		(		application of 25000# per sq. ft.
		)		(		and failed completely after 10 min.
		)		(		upon inc. load to ult. load.
1I	198	)		(	23987	Failed suddenly by deflect. north
1L	194	[ ] 2[s 8"	)		24199	Fail. by def. north
		at 21 1/4#	85)	(Batten		
1L	195	A = 12.50	)	(80.73	23039	Fail. by def. north
1L	196		)		24893	Fail. sud. by def. in ctr.
1M	187	[ ] 2[s 8")		Batten	15000	Broke down under load in abt.2 min.
1M	188	at 21 1/4#)	120	121.1	15200	Failed by deflecting bet.batten pls.
1M	189	A = 12.50)			15065	

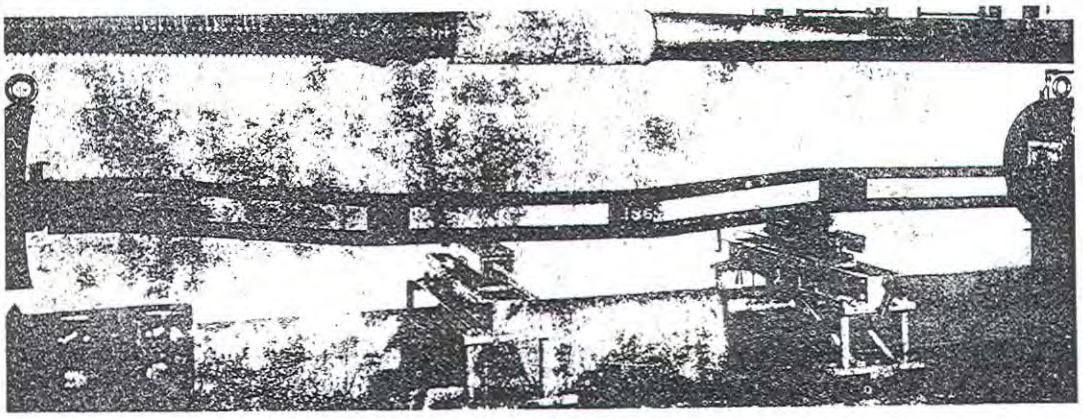
The specimen tensile tests in all cases gave results reasonably close to the prescribed limits of 37000 to 39000 lb. per sq. in., the short compressive tests showed yield points varying from below 30000 lb. per sq. in. to above 41000 lb. per sq. in.



COLUMN 1-G.

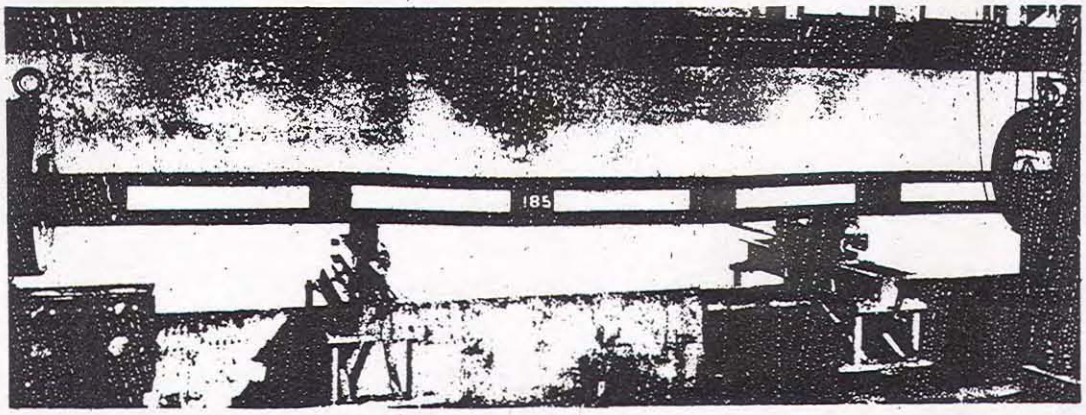


COLUMN 1-H.

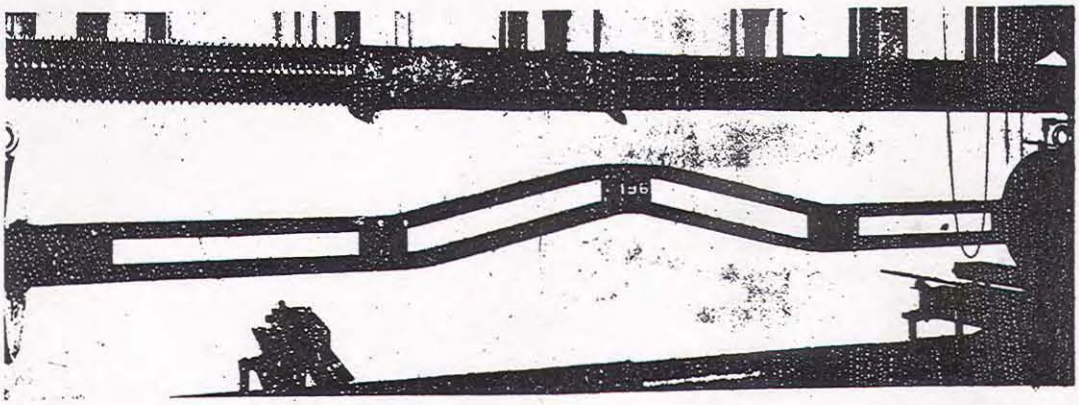


COLUMN 1-I.

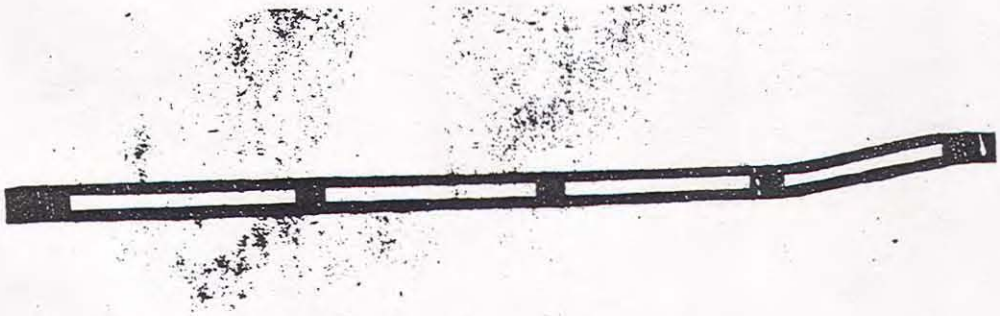
Typical Failures of Columns in A.R.E.A. Tests



COLUMN 1-K



COLUMN 1-L



COLUMN 1-M.

Typical Failures of Columns in A.R.E.A. Tests

Final Report of the Special Committee on Steel Columns and Struts

The Special Committee authorized "to consider and report upon the design, ultimate strength, and safe working values of steel columns and struts."

Material, Basic open hearth steel.
Physical Properties
Ultimate tensile strength - Desired 60000 lb. per sq. in. (may vary 1500 either way).
Elastic limit (yield point) - Desired 38000 lb. per sq. in. (may vary 1000 lb. either way).
Elongation in 8 in. Desired 28 percent (may vary 2 percent either way).
Reduction in area. Desired 56 percent (may vary 4 percent either way).
Rivet spacing, 4 diameters throughout column.
In general shop work better than the average commercial grade.

Test columns developed variations in their sectional areas amounting in extreme cases to as much as 4 percent from their nominal area.

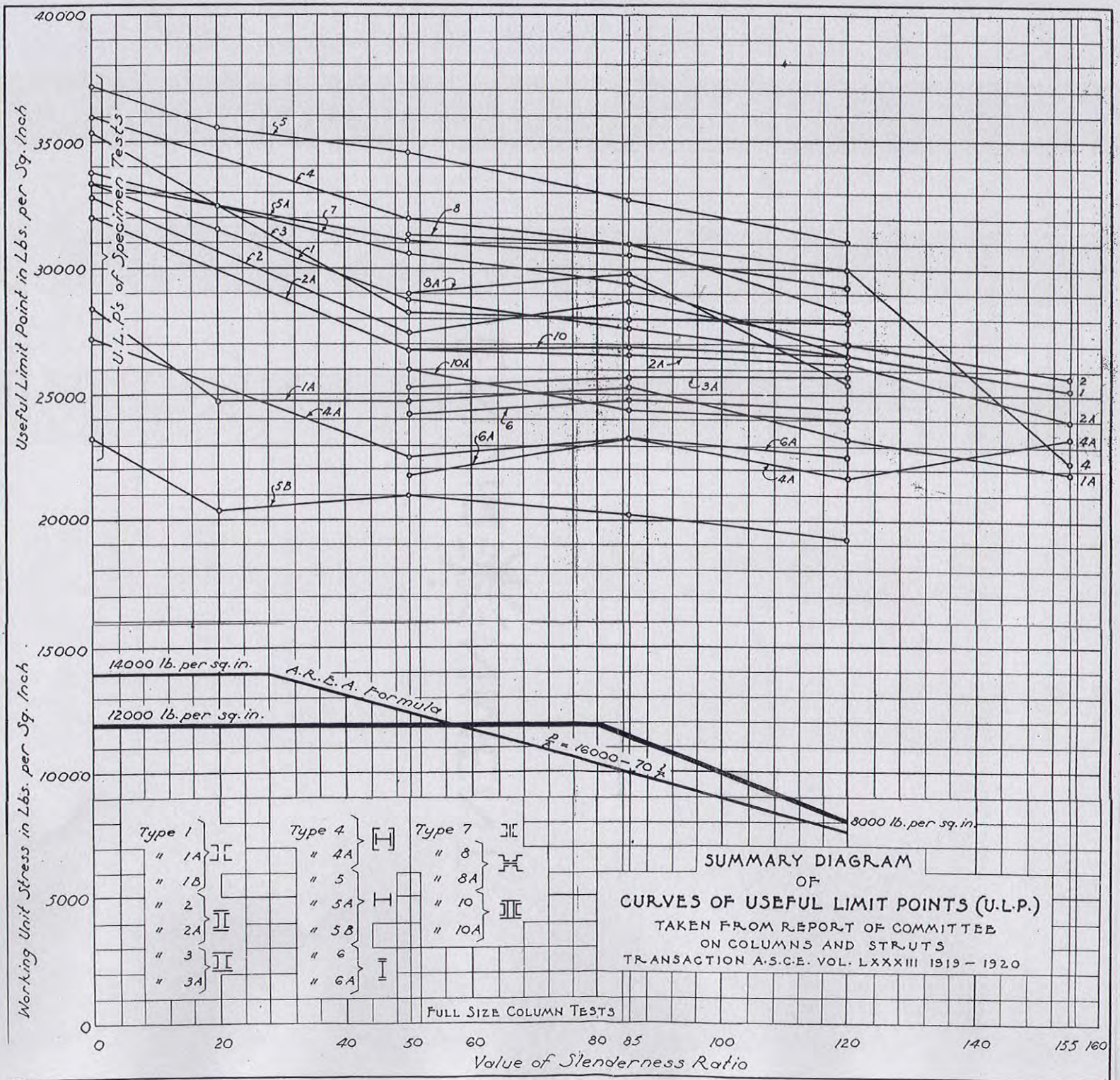
The program was arranged "with the intention as far as practicable of eliminating all variables except that of form" and was subdivided so as to consider the effect of light and heavy sections. The Committee was influenced somewhat in the certain forms of columns by an opinion that columns failed largely through the crippling of the thin outstanding legs.

The results as shown hardly prove or disprove the original question as to the influence of the thin outstanding legs. It is possible that for very thin metal the ratio of the thickness of the outstanding leg to its width might have an effect on the strength of the column but for the thicknesses of the material which have been used in the program of tests outlined by the Committee it is evident that the ratio of thickness to width is not a controlling factor.

The physical characteristics and strength of structural steel are affected by the amount of working which it receives in passing through the rolls and for the same size of ingot the heavy material does not receive as much working as the light; consequently, for the same chemical composition, the heavier material is weaker. In ordering the steel the Committee provided for this difference by allowing the mill to change the carbon content so as to endeavor to produce the same strength for the light material as for the heavy.

The Committee has defined the critical point as the point which is determined graphically by drawing a line tangent to the envelope of the stress strain curve having a slope of one-half that of the last run-up line for its straight or nearly straight portion and calls this the Useful Limit Point.

Plate XXXVI is a summary of the curves of all the U.L.P. values. A surprising feature of these curves is the variation in U.L.P. of 15000 lb. per sq. in. between columns of Types 5 and 5B. The average U.L.P. for $l/r = 50$, for columns of Type 5, light section is 34700 lb. per sq. in., and for Type B extra heavy section it is 19700 lb. per sq. in. The lower value is but 57 percent the higher and there is a variation of 28 percent from the mean. These extreme variations may be abnormal but it must be remembered that the tests were made on materials specially chosen, and supposedly the best that the mills could produce.


SUMMARY DIAGRAM
OF
CURVES OF USEFUL LIMIT POINTS (U.L.P.)
TAKEN FROM REPORT OF COMMITTEE
ON COLUMNS AND STRUTS
TRANSACTION A.S.C.E. VOL. LXXXIII 1919 – 1920
Useful Limit Point in Lbs. per Sq. Inch
U.L.P's of Specimen Tests
Working Unit Stress in Lbs. per Sq. Inch
Value of Slenderness Ratio
14000 lb. per sq. in.
12000 lb. per sq. in.
A.R.E.A. Formula
P/A = 16000 – 70 l/r
8000 lb. per sq. in.
Type 1
" 1A
" 1B
" 2
" 2A
" 3
" 3A
Type 4
" 4A
" 5
" 5A
" 5B
" 6
" 6A
Type 7
" 8
" 8A
" 10
" 10A
FULL SIZE COLUMN TESTS

Tests of 18 Large Bridge Columns of Various Shapes and Various Steels

The primary purpose of the tests was to determine the strength of such members under axial loads in relation to their general design and the grades of steel used in construction. The secondary purpose was to study the distribution of stress and strain throughout the shafts of the columns and their different details.

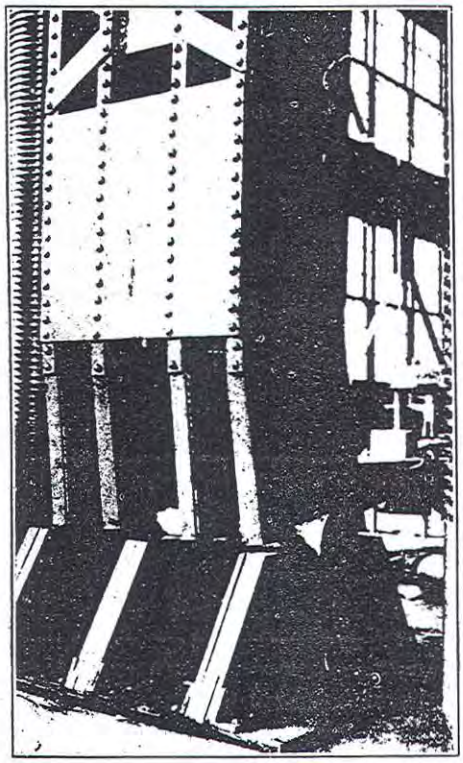
Inasmuch as the columns tested were either of different steels or of different sections than ordinarily under consideration in highway bridges the results are not tabulated here.

On page L-3886 are shown photographs of "four test columns of the Memphis Bridge (which) failed locally at the ends, which ... was largely incident to difficulties experienced by the designers in providing for sufficient space for the eyebars at the pin connections." Along this line also note the photographs of the failures in Buchanan's tests shown on pages L-3863 and L-3864.

Explanation of Graph Shown on Page L-3887

A graphical representation showing the relation of the observed maximum loads to the yield points of the component steels is shown on page L-3887. 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. This is recorded in the graph 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 columns starred are the columns shown in the photographs and it will be noted that in all cases but one for the same class of steel that they failed at lower unit loads than the columns tied with lacing or plates throughout.



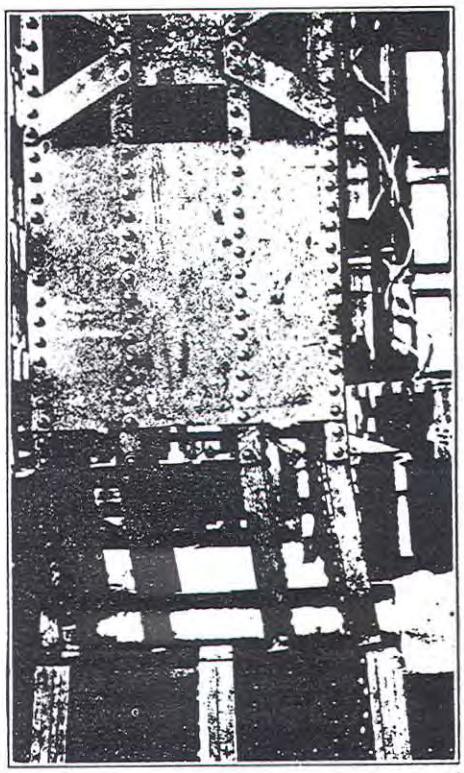
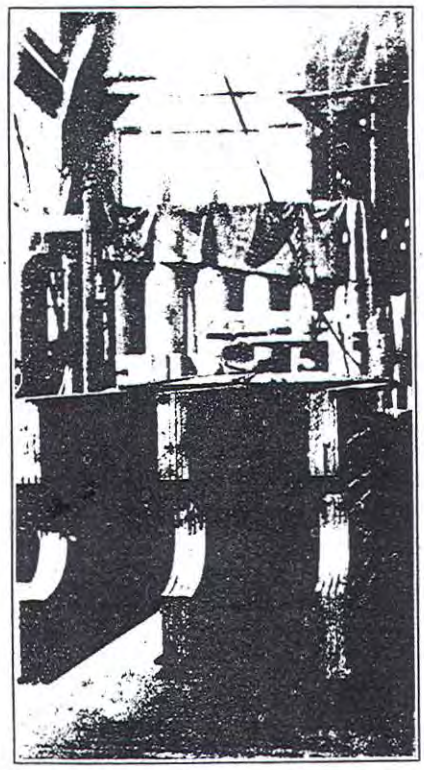


FIG. 56.—*Column U.S.M.S.$_1$ (carbon steel)*



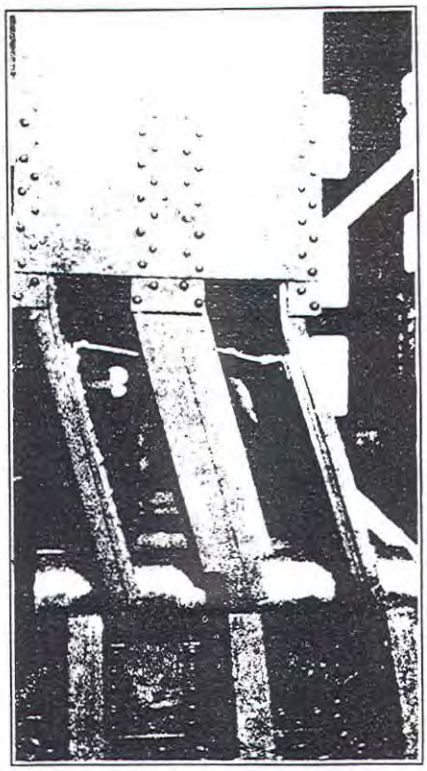


FIG. 58.—*Column U.M.$_1$ (mayari steel)*

Failures of Bureau of Standards Tests
(see comments L-3885)

STEEL	NICKEL		MAYARI					SILICON		CHROME		HIGH CARBON						
MODULUS ACTUAL			29.0	31.3	30.4	29.2	30.3	30.0	29.5	29.0	29.6		29.4	28.7	28.8	29.4	29.6	30.6
MODULUS NOMINAL	28.6	30.4	28.5	30.8	29.7	29.0	30.8	30.3	29.8	29.3	29.9		28.9	29.1	29.1	29.9	29.5	30.4
MAX. STRESS ACTUAL			62.7	65.7	48.0	49.7	57.9	52.3	51.3	48.7	51.2	39.2	45.9	41.3	46.0	33.8	31.2	32.7
MAX. STRESS NOMINAL	61.7	61.4	61.5	64.7	47.2	49.4	57.7	52.8	51.7	49.2	51.8	38.4	45.3	41.7	46.5	34.3	31.0	32.4

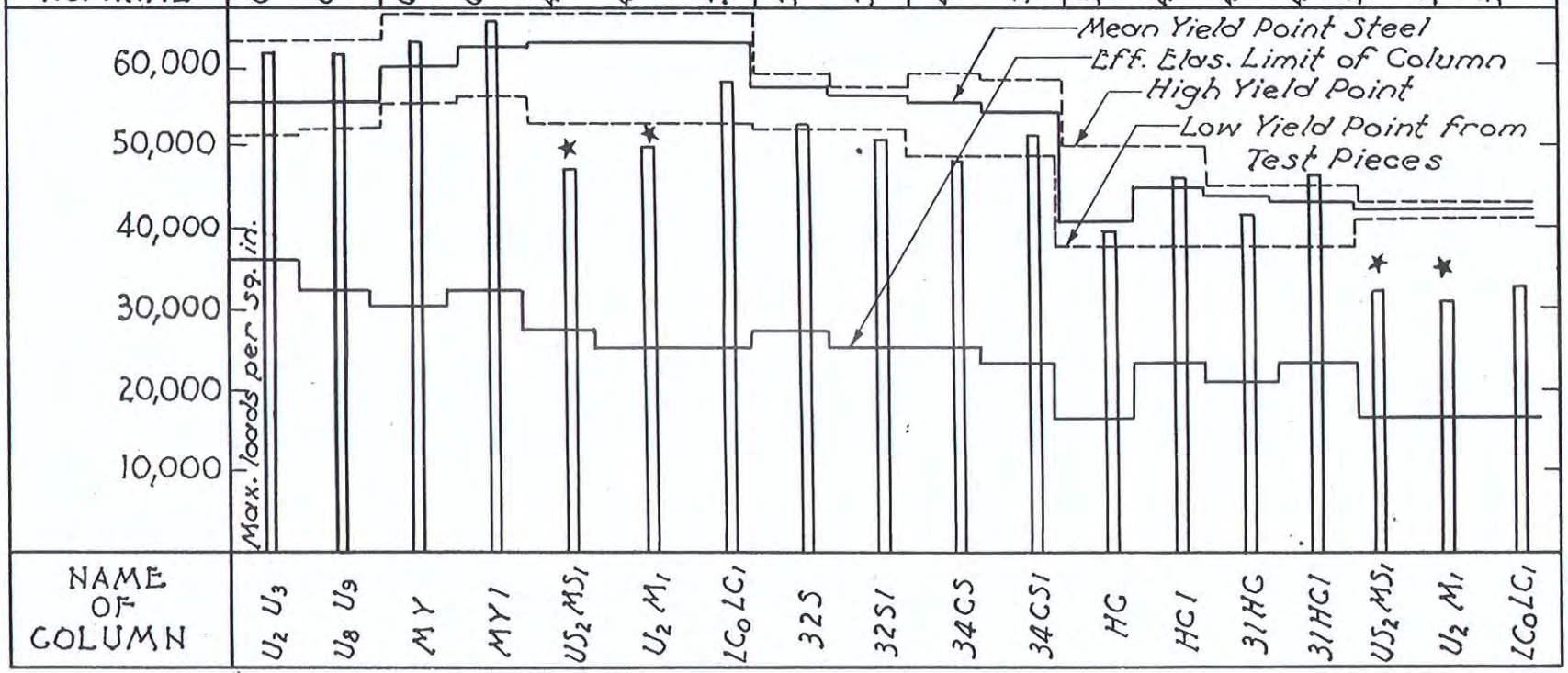


CHART SHOWING CORRESPONDENCE BETWEEN MAXIMUM LOADS AND YIELD POINTS OF STEEL

Results of Some Compression Tests of Structural Steel Angles
Technologic Paper 218
Bureau of Standards
August 3, 1922

This article presents the results of compression tests of 170 structural angles made at the Pittsburgh branch, Bureau of Standards. The object of the tests was to determine the ultimate compressive strength of angles fastened at the ends in such ways as would closely correspond to their connections in the construction of transmission towers. There was also tested a series of angles with square ends. An end fixation factor was found to represent satisfactorily the effect of different types of end connections. Using this fixation factor, the average values for large slenderness ratios were well represented by Euler's formula. The results obtained from shorter columns agreed with the experimental and theoretical results of Karman. The effect of eccentric loading was most marked at the slenderness ratios indicated by Karman's theory.

Tests of Twelve Large Columns
Eng. News-Record, Vol. 88, p. 877

Results of failure tests on twelve large riveted steel columns carried out at the Lichterfelde laboratory near Berlin are reported by F. Voss in "Der Baumgemeier" of January 15.

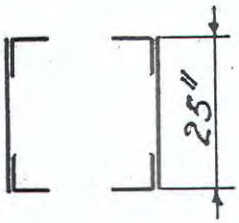
Five of the columns were tested in duplicate.

In the case of four of these the agreement of the two separate tests is very close. Column VII shows a large difference but here the specimens were not precise duplicates. The batten plates in the first were 9 1/2 in. wide and had two rivets in each angle while those of the second column were 14 in. wide and had three rivets; shearing of the batten plate rivets of the first column limited its capacity. Latticing was proportioned for a shear of 1 percent of the column load.

All material accepted on mill test 53000 to 63000 lb. per sq. in. ultimate strength, 20 percent elongation. It appeared from the column tests that the ultimate loads depended on the yield point strength of the material.

It was deduced from the tests that strong latticing gives the same column capacity as the use of a central web.

Col. No.	Area	Break load 1000#	Fail. Lb. per sq. in.	Ave.yield Pt. Mat.	l/r
I-1	63	1865	29600	31200	51
I-2	63	1860	29500	31200	51
II-1	70	2300	32850	34000	58
II-2	70	2430	34700	34000	58
III-1	70	2325	33200	34000	58
III-2	70	2260	32300	34000	58
IV	60	1875	31300	33450	52
V	60	1867	31100	33450	52
VI-1	64	2330	36400	37600	51
VI-2	64	2395	37400	37600	51
VII-1	64	2195	34300	40500	51
VII-2	64	2445	38250	40500	51



Cols. I-1 & I-2
Cols. IV & V
4-Ls-6"x6"x 5/8"
2-Pls.-25"x 5/8"

Col. II-1 & II-2
Col. III-1 & III-2
4-Ls-6"x6"x 5/8"
2-Pls.-25"x 1/2"
4-Ls-3 5/8"x3 5/8"x 3/8"
1-Pl.-24"x 5/16"

Cols. VI-1
" VI-2
" VII-1
" VII-2
4-Ls-6"x6"x 11/16"
2-Pls.-25"x 5/8"

Lacing Cols.-I-II-IV-V-VI
Sing. L.-3"x2"x 3/8" with vert. struts
Cols. III-VII-Batten Pls. abt. 4'-4" ctrs.
-IV-V
Col. III Dia.-7'-7" ctrs.
Rest Cols. Diaphragms at ends

L-3889

Column Tests for Delaware River Bridge
Eng. News-Record, Vol. 89, p. 987

Strength Tests of Fourteen Wide Web Columns
at Bureau of Standards

Material, Silicon Steel, 80,000 lb. per sq. in. ultimate strength and 41,000 lb. per sq. in. yield point.

Tetmajer's Tests

On page L-3891 are shown the graphs of Tetmajer's tests taken from an article by Marston in the Trans. A.S.C.E., Vol. XXXIX, p. 111.

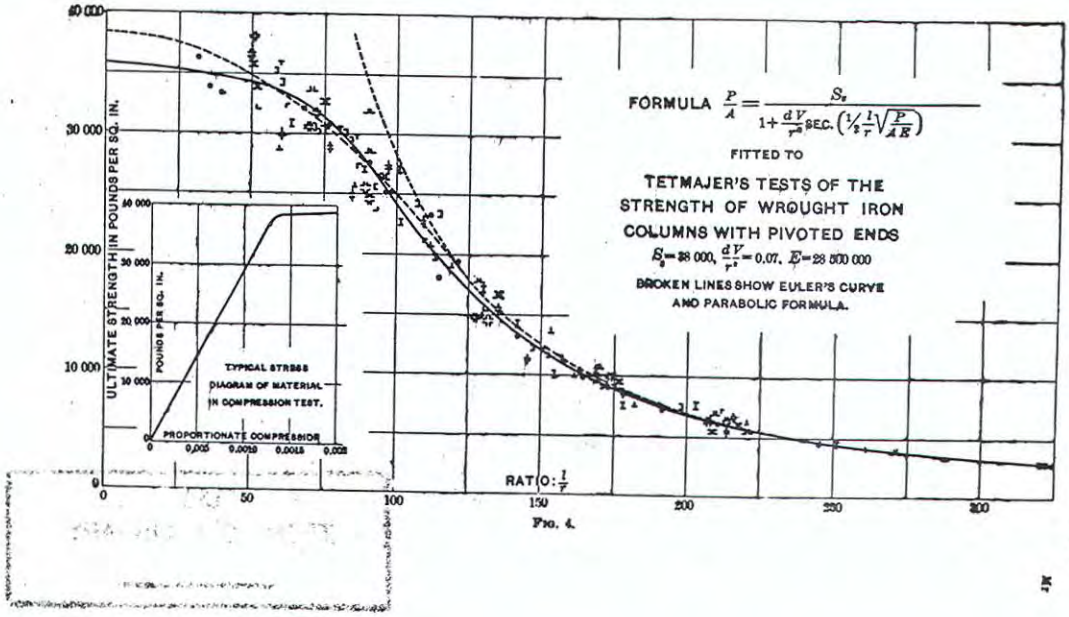


FIG. 4.

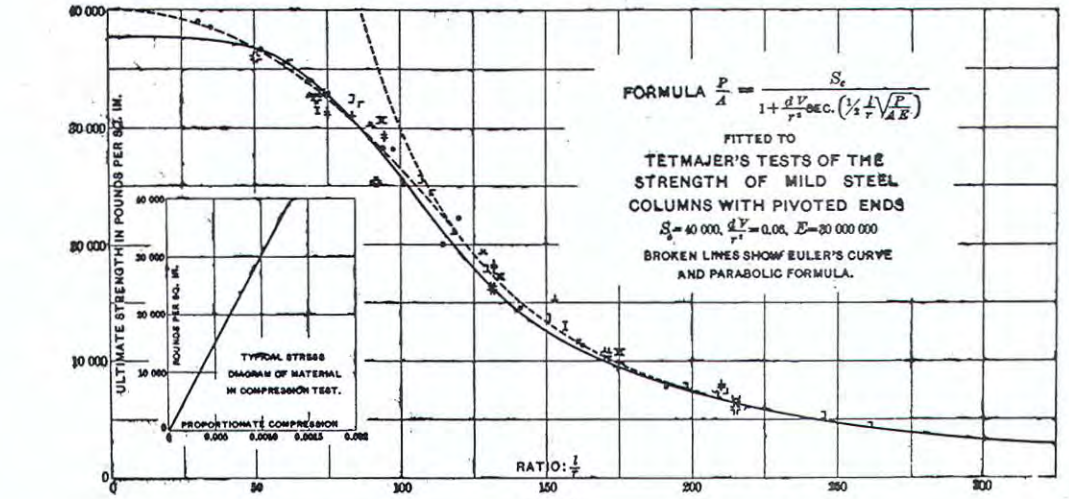


FIG. 5.

Tetmajer's Tests
(see L-3890)

L-3891