Lattice Girder Bridge

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1874

Graduated in 1875

Design for a Double Warren Iron Girder Bridge.

In a girder, the load is resisted by the counteractive power of the particles of the material, so, to have a structure of sufficient strength to resist the forces caused by the load, and to realize equilibrium, we must know what the forces are to be, because the proportions throughout, have to be determined with reference to the amount of force, either of extension or compression, which the material is capable of exerting in resisting the action of the load. It is necessary also in calculations fixing the dimensions of the pieces to distinguish between the action of the permanent load, and that of a rolling load, or that action coming from a load which is applied and then removed, and which for the traffic of a railroad arises from passing trains. For, a steady load, which for a bridge would principally be the weight of the bridge, produces from the effect of gravity, one unvarying action, while the result produced by a rolling load varies, causing a sudden stress in the parts of the structure.

Formerly the arched form of bridges was almost universally employed; then when iron began to be more employed in building, it was applied to bridges at first in the cast shape.

The limitation of span for which girder bridges were safely applicable had restricted their employment, about forty feet having been commonly considered the maximum length to which single cast iron girders could safely be applied, liable to be loaded with railway trains or other heavy weights. The desire to retain this convenient form of structure, however, and to extend its use to longer spans, induced attempts to combine wrought iron with cast metal in such a manner as should impart to the compound structure the superior power to resist extension which wrought iron is well known to possess. But here must be noticed the defects of compound iron structures, as it points directly to the superiority of homogeneous fabrics, and this is the liability of strain being caused in different parts from the difference in the effect of the temperatute or cast and wroughtt metal, and the difficulty of combination to take into account this difference. It is from this consideration, mainly, that in this design the cast iron form of bars is almost wholly rejected.

In the girder form of bridge the depth of the structure is reduced to that of the section of material due to the maximum load and hence the peculiar applicability of this form of railway bridge in which it is desirable to preserve a minimum distance from the under side or soffit of the girder to the level of the top above, and moreover depending for its strength upon the sectional area of the girder at that point in its length over which the weight or load acts, requires abutments to resist vertical pressure only, while those of arched bridges have to resist the lateral thrust of the arch. In choosing this particular form of girder bridge, we are led by the consideration of extraordinary simplicity and economy which it has and the ease with which it is put together, each piece having dimensions different from the others, and therefore its own peculiar place, the holes for rivets and bolts always being punched before transportation which as the pieces are generally short is comparatively easy. As the design is particular and not general we shall employ general formulae as little as possible. From experiments on actual cases and examples given by different authorities, it is seen that the passage of heavy weights produce no sensible deflection, so from this we will not consider camber, but make the estimates with the intention of having the bridge stiff enough to do without it. The principal dimensions are given us, namely span, 192 ft. divided into 16 panels, height 18 ft., 15 ft. clear between the two girders, and the bridge to be rivetted. As there are sixteen panels each panel is twelve feet long.

As the load is distributed symmetrically in regard to the middle point of the bridge it will be sufficient to consider the action of the forces on one half of the bridge only, the dimensions of every piece being such that there

shall be enough material to resist the greatest force liable to be brought to bear on that piece, and here as before stated we must distinguish between the action of the dead and that of the live load. Another requirement is that the bridge shall be designed to bear a dead load of eight hundred (800) lbs per ft. run, and a live load of twelve hundred (1200) lbs. per foot run.
Generally in designing a structure, it is customary to consider the breaking strength, and this is for a certain material a certain number of times larger than the working strength, and for wrought iron, for a dead load this number we will take as three and for a live load as six.

For certain reasons the calculations will be made both considering the working strength and the breaking strength using the numbers, or factors of safety. The weight extending over one panel is to be supposed as acting at one point, that at which the diagonals meet and the load is supported. The weight on half a panel at each end is not taken into consideration as it is supposed to go directly into the abutments.

For all our calculations respecting shear and the bending action at any plane of section vertically through the girder, we consider for simplicity all the forces acting at the left of the plane of section, because the resistance of particles at that section must equal the forces applied there, for if they are not equal the bridge breaks. Regarding the action of the live load, suppose this load to come on to the bridge at the right till one point is loaded. Call the supporting force at the left P_1, and it is evident that the force at that plane of section which acts at all is simply P_1. Suppose two points are loaded, from the second point to the left hand point of support the only force to act is P_1, remembering that now we are considering only the live load. We see that as the points go on being covered the force P_1 must grow larger. Referring to p. 10 fig. 1 which is a skeleton outline of this form of lattice girder, *Fig 1.*

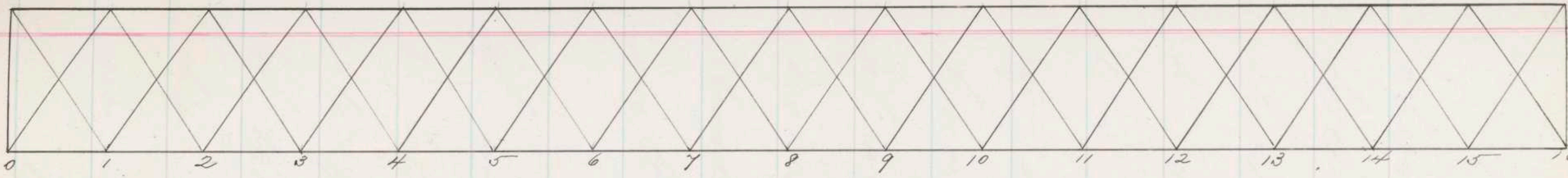


Fig. 2.

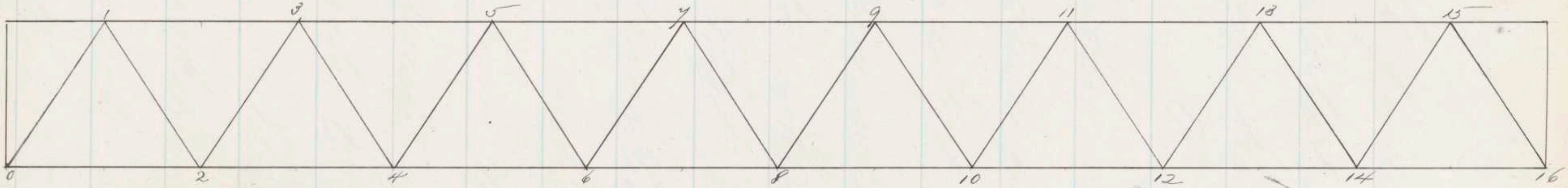
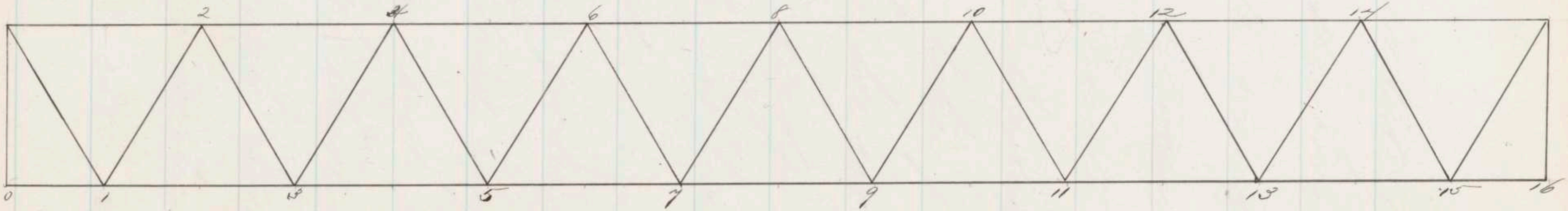


Fig 3.



Scale 20' to 1".

when all the points are loaded the force P_1 is greatest, and therefore the shear between the points 0 and 1 is the greatest. The shear between 2 and 1 is greatest when all the points up to and including 2 are loaded, hence the greatest shear takes place at any section when the longer segment into which the bridge is divided by that section is loaded with the travelling load.

The double Warren girder consists of an upper boom to withstand compression, a lower to resist extension, and two sets of diagonals to resist the shearing action of the load. These diagonals cross each other and make with the vertical, an angle which we will call i, and is found thus.

$\tan i = \frac{12}{18} = \frac{2}{3}$ $\quad \log. 2. = 0.301030$

$\quad$ " $3. = 0.477121$

$\quad \log \tan i = 9.823909$

$\therefore i = 33°\,41'\,24''.31$ This subtracted from 90° gives $56°\,18'\,35''.69$ as the inclination of the diagonals, or the angle which they make with the horizon. The girder is seen to be made up of two simple ones, so in computing the stresses, it is best to pursue the method of considering the two separate simple girders, and then combine the results. Hence taking the simple trusses we have the figures 2 and 3, p.10. First we will find the supporting force at each end, or the weight that the abutment at either end must bear. Calling w the wt. per foot of the dead load and w_1 the wt. of live load per foot, and l the length we have $(w+w_1)\frac{l}{2} = (800+1200)\frac{192}{2} = 192000$

as the force resisted by either abutment.

To find the shear in the diagonals.

First we will find the shear from the dead load, both with and without the factors of safety, considering the panels in the simple trusses (figs. 2 & 3), and a weight of 12 × 800 = 9600 lbs. at each point between the points of support.

The maximum action of the live load on each panel will alone be considered, the weight of the live load on a single point being 12 × 1200 = 14400 lbs. Pages 14 and 15 show the results.

At each plane of section, we have to consider only the forces at the left hand and these are the greatest shear, which we will call P, at the ends and the load on each point of 9600 lbs up to that section. This is for a

Shearing forces for fig. 2.

From Dead Load.

$F = P_1 - \Sigma W$ $\qquad W = 12 \times 800 = 9600$

		Without factor of safety	With factor 3
From 0 to 2	$F_0 = P_1 = \frac{1}{2} \times 7 \times 9600$	$= 33,600$	100,800
" 2 to 4	$F_1 = 33600 - 9600$	$= 24,000$	72,000
" 4 to 6	$F_2 = 33600 - 19200$	$= 14,400$	43,200
" 6 to 8	$F_3 = 33600 - 28800$	$= 4,800$	14,400

Maximum Shear from Live Load.

$F = P_1$ $\qquad W' = 12 \times 1200 = 14400$

		Without factor	With factor 6.
From 0 to 2	$F'_0 = \frac{1}{2} \times 7 \times 14400$	$= 50400$	302,400
" 2 to 4	$F_1 = \frac{7}{16} \times 14400 \times 6$	$= 37,800$	226,800
" 4 to 6	$F_2 = \frac{6}{16} \times 14400 \times 5$	$= 27,000$	162,000
" 6 to 8	$F_3 = \frac{5}{16} \times 14400 \times 4$	$= 18,000$	108,000

Total Shear.

	Without factors	With factors
From 0 to 2	84,000	403,200
" 2 " 4	61,800	298,800
" 4 to 6	41,400	205,200
" 6 " 8	22,800	122,400

Shearing Forces for Fig. 3.

From Dead Load

$F' = P_1 - \Sigma W$ $\qquad W = 9600$

		Without factor 3	With factor 3
From 0 to 1	$F'_0 = P_1 = \frac{1}{2} \times 8 \times 9600 =$	38,400	115,200
" 1 to 3	$F'_1 = 38400 - 9600 =$	28,800	86,400
" 3 to 5	$F'_2 = 38400 - 19200 =$	19,200	57600
" 5 to 7	$F'_3 = 38400 - 28800 =$	9,600	28,800
" 7 to 9	$F'_4 = 38400 - 38400 =$	0	0

Maximum Shear from Live Load.

$F' = P_1$ $\qquad W' = 14400$

		Without factor 6	With factor 6.
From 0 to 1	$F'_0 = 8 \times 14400 \times \frac{1}{2} =$	57600	345,600
" 1 to 3	$F'_1 = 7 \times 14400 \times \frac{7}{16} =$	44100	264,600
" 3 to 5	$F'_2 = 6 \times 14400 \times \frac{6}{16} =$	32400	194400
" 5 to 7	$F'_3 = 5 \times 14400 \times \frac{5}{16} =$	22500	135,000
" 7 to 9	$F'_4 = 4 \times 14400 \times \frac{4}{16} =$	14400	86400

Total Shear

		Without factors 3+6	With factors
From 0 to 1	38400 + 57600 =	96,000	460,800
" 1 to 3	28800 + 44100 =	72,900	351,000
" 3 to 5	19200 + 32400 =	51,600	252,000
" 5 to 7	9600 + 22500 =	32100	163,800
" 7 to 9	14400 =	14,400	86,400

Diagonals

Panel. Fig. 1	Member.	Shear × Sec. of i. Without Factors	Shear × Sec. of i. With Factors	Stress. Without Factors	Stress. With Factors.	Rivets
1	Extension	96000 × 1.202	460,800 × 1.202	115,392. p.	553,882. p	10
	Compression	84000 × 1.202	403,200 × "	100,968. t.	484,646. t.	8
2	Ex.	" × "	" × "	" p.	" p.	8
	Com.	72,900 × "	351,000 × "	87,626 t	421,902. t.	8
3	E.	" × "	" × "	" p.	" p.	7
	C.	61,800 × "	298,800 × "	74,284. t.	359,158 t.	6
4	E.	" × "	" × "	" p.	" p.	6
	C.	51,600 × "	252,000 × "	62,023. t.	302,904. t.	5
5	E	" × "	" × "	" p.	" p.	"
	C.	41,400 × "	205,200 × "	49,763. t.	246,650. t.	4
6	E.	" × "	" × "	" p.	" p.	4
	C.	32,100 × "	163,800 × "	38,584. t.	196,888. t.	3
7	E.	" × "	" × "	" p.	" p.	3
	C.	22,800 × "	122,400 × "	27,406. t.	147,125. t.	2
8	E	" × "	" × "	" p.	" p.	2
	C.	14,400 × "	86,400 × "	17,309. t.	103,853. t.	2

dead load, and for a live load P_1 is alone to be considered, P_1 being in this case the greatest shear at the end when the longer segment is loaded. P_1 is an upward force and the other are downward, hence we have for the shear at any section $F = P_1 - \Sigma W$ for a dead load and $F = P_1$ for a live load.

$W = 12 \times 800 = 9600$ lbs. Pages 14 and 15 give the shears. To find the amount of stress either of extension or compression in the diagonals, multiply the shear by the sec of i as shown in the table p. 16. Looking at a single panel and considering that alone, and the action of the load on it, it is evident that in the diagonal sloping up towards the centre there is compression and in the diagonal sloping up, from the centre there is compression. Or this might be seen by taking the shearing force in any panel and as it is upwards, I_1 being always the greater, resolving it into components gives the same as before.

When the longer segment is loaded with the travelling load the diagonals act alternately as struts and ties.

When the shorter segment is loaded the shearing force acts downward, opposite to that from the dead load, and in order to have any diagonal act as strut and tie both, the shear from the travelling load on the shorter segment must be greater than the shear from the dead load.

In fig. 3 on 7-8 & 8-9 the only load acting is the live load, as the shear from the dead is nothing.

Hence 7,8 acts only as a tie.
Calculating P, fig. 2 for the several cases when 1, 2, & 3 points are covered we find that there is for 3 points, on 6-7 - 7 8 a downward shear of 10800 - 4800 = 6000 without factors and with 64800 - 14400 = 50400 and these diagonals are the only ones which act alternately as struts and ties. For the bending moments we have when the load covers the whole bridge the maximum effect. Pages 20 and 21 give the results, and the method of finding the moments. Dividing each moment by 18 the height of the girder, we have the horizontal stresses as given in page 22; - and p. 23 gives the total horizontal stress for any panel.

Bending Moments.

Fig. 2.

$$M = P_1 \times x - \Sigma\, w \Delta x = P_1 \times x - w\, \Sigma \Delta x$$

Without factor

At 1 lower boom 84000×12 = 1,008,000
" 2 upper " 84000×24 = 2,016,000
" 3 lower " 84000×36 − 24000×12 = 2,736,000
" 4 upper " 84000×48 − 24000×24 = 3,456,000
" 5 lower " 84000×60 − 24000×48 = 3,888,000
" 6 upper " 84000×72 − 24000×72 = 4,320,000
" 7 lower " 84000×84 − 24000×108 = 4,464,000
" 8 upper " 84000×96 − 24000×144 = 4,608,000

With factor

" 1 lower " 403200×12 = 4,838,400
" 2 upper " 403200×24 = 9,676,800
" 3 lower " 403200×36 − 115,200×12 = 13,132,800
" 4 upper " " ×48 − 115200×24 = 16,588,800
" 5 lower " " ×60 − " 48 = 18,662,400
" 6 upper " " ×72 − " 72 = 20,736,000
" 7 lower " " ×84 − " 108 = 21,427,200
" 8 upper " " ×96 − " 144 = 22,118,400

Bending Moments.

Fig. 13.

$M = P_1 \times x - w\Sigma_1 x$ Shear at end $= P_1 = 96000$ without factor.

At 1 upper boom	96000×12	= 1,152,000
„ 2 lower „	96 000×24−12×24000	= 2,016 000
„ 3 upper „	„ ×36−24× „	= 2,880,000
„ 4 lower „	„ ×48−48× „	= 3,456,000
„ 5 upper „	„ ×60−72× „	= 4,032,000
„ 6 lower „	„ ×72−108× „	= 4,320,000
„ 7 upper „	„ ×84−144× „	= 4,608,000
„ 8 lower „	„ ×96−192× „	= 4,608,000

Without Factor

At 1 upper „	460800×12	= 5,529,600
„ 2 lower „	„ ×24−12×115200	= 9,676,800
„ 3 upper „	„ ×36−24× „	= 13,824,000
„ 4 lower „	„ ×48−48× „	= 16,588,800
„ 5 upper „	„ ×60−72× „	= 19,353,600
„ 6 lower „	„ ×72−108× „	= 20,736,000
„ 7 upper „	„ ×84−144× „	= 22,118,400
„ 8 lower „	„ ×96−192× „	= 22,118,400

With Factor

Upper Boom

At	Bending Moments		$\frac{M}{18}$ Horizontal Stresses		Between
Points	Without Factors	With Factors	Without Factors	With Factors	Points
1	96000×12 = 1,152,000	460800×12 = 5,529,600	64,000	307,200	0 & 2
2	84000×24 = 2,016,000	403,200×24 = 9,676,800	112,000	537,600	1 & 3
3	96000×36−24×24000 = 2,880,000	460800×36−24×115200 = 13,824,000	160,000	768,000	2 & 4
4	84000×48−24×24000 = 3,456,000	403200×48−24×115200 = 16,588,800	192,000	921,600	3 & 5
5	96000×60−72×24000 = 4,032,000	460800×60−72×115200 = 19,353,600	224,000	1,075,200	4 & 6
6	84000×72−72×24000 = 4,320,000	403,200×72−72×115,200 = 20,736,000	240,000	1,152,000	5 & 7
7	96000×84−144×24000 = 4,608,000	460800×84−144×115,200 = 22,118,400	256,000	1,228,800	6 & 8
8	84000×96−144×24000 = 4,608,000	403,200×96−144×115,200 = 22,118,400	256,000	1,228,800	7 & 9

Lower Boom

At Points	Without Factors	With Factors	Without Factors	With Factors	Between Points
1	84000×12 = 1,008,000	403200×12 = 4,838,400	56,000	268,800	0 & 2
2	96000×24−12×24000 = 2,016,000	460800×24−12×115200 = 9,676,800	112,000	537,600	1 & 3
3	84000×36−12×24000 = 2,736,000	403200×36−12×115200 = 13,132,800	152,000	729,600	2 & 4
4	96000×48−48×24000 = 3,456,000	460800×48−48×115200 = 16,588,800	192,000	921,600	3 & 5
5	84000×60−48×24000 = 3,888,000	403200×60−48×115200 = 18,662,400	216,000	1,036,800	4 & 6
6	96000×72−108×24000 = 4,320,000	460800×72−108×115200 = 20,736,000	240,000	1,152,000	5 & 7
7	84000×84−108×24000 = 4,464,000	403200×84−108×115200 = 21,427,200	248,000	1,190,400	6 & 8
8	96.000×96−192×24000 = 4,608,000	460800×96−192×115200 = 22,118,400	256,000	1,228,800	7 & 9

Total Horizontal Stresses in Upper & Lower Booms by Combining Horizontal Stresses

From	Without factors	With Factors	Thrust in Upper Boom Without	With
0 to 1	64000	307200	64000	307,200
1 " 2	64000+112000	307,200+537600	176000	844,800
2 " 3	112000+160000	537,600+768,000	272000	1,305,600
3 " 4	160000+192000	768,000+921,600	352000	1,689,600
4 " 5	192000+224000	921,600+1,075,200	416000	1,996,800
5 " 6	224000+240000	1,152,000+1,075,200	464000	2,227,200
6 " 7	240000+256000	1,152,000+1,228,800	496000	2,380,800
7 " 8	256000+256000	1,228,800+1,228,800	512000	2,457,600
	Pull in Lower Boom			
0 to 1	56000	268,800	56,000	268,800
1 " 2	56000+112000	268,800+537,600	168000	806400
2 " 3	112000+152000	537,600+729,600	264,000	1,267,200
3 " 4	152,000+192000	729,600+921,600	344,000	1,651,200
4 " 5	192,000+216,000	921,600+1,036,800	408,000	1,958,400
5 " 6	216,000+240,000	1,036,800+1,152,000	456,000	2,188,800
6 " 7	240,000+248,000	1,152,000+1,190,400	488,000	2,342,400
7 " 8	248000+256000	1,190,400+1,228,800	504,000	2,419,200

Total.

Page 16 the stress in the diagonals is given, "p" representing pull, and "t" thrust. To find the sectional area to stand the stresses, we divide the amount of pull (this being for pull only) by the intensity of the stress, or the amount which one square inch of the metal will withstand. Page 25 gives the calculated sectional area of the ties and the area given, using 10,000 lbs. as the working tensile strength of wrought iron. Calling the bars 1 inch thick the widths are given in the column marked "area given". For the struts we use Mr. Gordon's formula for the ultimate strength of wrought iron, deduced from Mr. Hodgkinson's experiments, which is $\frac{P}{S} = 36{,}000 \div 1 + \frac{al^2}{h^2}$. Making

Mistake. The areas of upper chord should be reversed, the 1st coming last.

Sec. Area = $S = \frac{P}{10000}$

Panel	$S = \frac{P}{f}$ Stress	Calculated Area of Ties	Sectional Area Given	Horizontal Stress in Lower Boom without factor	Cal. Area	Sectional Area Given
1	115,392	11.5	11	56,000	5.6	4 7.5 12.5 24
2	100968	10.09	11	168,000	16.8	11.5 25 36.5
3	87626	8.76	8.5	264,000	26.4	11.5 25 36.5
4	74,284	7.42	8.5	344,000	34.4	11.5 25 36.5
5	62,023	6.20	8.5	408,000	40.8	36.5 15 51.5
6	49,763	4.97	6	456 000	45.6	51.5
7	38,584	3.85	6	488,000	48.8	51.5
8	27,406	2.74	3	504,000	50.4	51.5
		Cal. Area of Struts	Sectional Area Given	$S = \frac{P}{36000}$ Upper Boom with factor		
1	484,646	13.8	14	307,200	8.5	$4 \times 18 \times \frac{3}{4} = 54$ $8 \times 3 \times \frac{3}{4} = 18$ 72
2	421,902	12.1	12	844,800	23.5	$4 \times 16 \times \frac{3}{4} = 48$ 18 66
3	359,158	10.3	12	1,305,600	36.3	$15 \times 3 = 45$ * 18 63
4	302,904	8.7	12	1,689,600	46.9	$13 \times 3 = 39$ 18 57
5	246,650	7	8	1,996,800	55.4	$10 \times 3 = 30$ 18 48
6	196,888	5.7	6	2,227,200	61.8	$8 \times 3 = 24$ 18 42.
7	147,123	4.2	6	2,380,800	66.1	42
8	Acts only as tie 103,853	3	3	2,457,600	68.2	42

$a = \frac{1}{3000}$ and using the radius of gyration, we have $\frac{P}{S} = 36000 \div 1 + \frac{l^2}{36000\,r^2}$ As the cross section is rectangular $r^2 = \frac{h^2}{12}$ Call $h = 2$ then $r^2 = \frac{1}{3}$

$l^2 = 12^2 + 18^2 = 468$ $\therefore \frac{P}{S} = \frac{36000}{1 + \frac{468}{12000}} =$

$\frac{36000}{1.039} = 34648.7$

$\therefore S = \frac{P}{34648.7}$ p. 25 giving sectional areas.

For convenience we make two struts, — 7 by 1 for the first, two 6 by 1 for the second, third, + fourth, till the last which is made single 3 by 1 and acts only as a tie. For lower boom the sectional area is found as for diagonal ties.

For the upper boom Mr. Gordon's formula is used, the section of the boom in finding the radius square of gyration being considered a thin square cell. This form of section is chosen because it is the stiffest.

The formula becomes calling $l = 12$ & $r^2 = \frac{h^2}{6}$. $\frac{P}{S} = \frac{36000}{1 + \frac{144}{36000 \times \frac{h^2}{6}}}$ Calling $h = 8$ we have $\frac{P}{S} = \frac{36000}{1 + \frac{144}{6000 \times 64}} = \frac{36000}{\frac{8003}{8000}} = 36000$ nearly.

$\therefore S = \frac{P}{36000}$

p. 25 gives the results for the different panels. Making the middle chord piece at the centre 18" on a side we have the minimum area, as $4 \times 18 \times \frac{3}{4}'' = 54 + 8 \times 3/4 \times 3 = 18 + 54 = 72$ where 68 is required. Call the next from the centre 16" on a side, the next 15" the next 13" then 10" then 8" to the end. The end chord piece is 18 ft. long. The lower chord is made up as in the figure, on the plate of details, the thickness of iron in flange at the centre = 2", flange 20" wide. Fig. 3 shows the method of obtaining the required sectional area at any point, in the distribution of the plates making the flange.

In the upper chord the size, or the width of a side diminishes from the centre of one panel to the centre of the next. The iron is 3/4" thick, and the lower part, at the points where the diagonals are fixed has a depth of 10".

For the end post we take a square cell cut at the top to fit the upper boom, and fastened to the lower boom by angle irons. The compressive force at the end is the sum of the forces at the end of each simple truss figs 2 & 3 and is 403200 + 460800 = 864000.

$$\frac{P}{S} = \frac{36000}{1 + \frac{l^2}{36000 r^2}} \qquad r^2 = \frac{h^2}{6} \quad h = 11 \quad h^2 = 121$$

$$\therefore r^2 = \frac{121}{6} \quad l = 18 \quad l^2 = 324$$

$$\therefore \frac{P}{S} = \frac{36000}{1 + \frac{324}{36000 \times \frac{121}{6}}} = \frac{36000 \times 121}{121.054} = 36000 \text{ nearly}$$

$$\therefore S = \frac{P}{36000} = \frac{864000}{36000} = 24$$

The inside dimensions are 11", and thickness of iron is 3/4", making total area of metal given as 51 sq. in.

The I beams are to be suspended from the bottom flange, and rest on iron castings, as shown in the details. To calculate the size, we assume a beam of 15" ht. 15' length weighing 200 lbs. to the yd. Now we consider the weight of stringers ties and rails with the load from a heavy locomotive to act through two points, each five feet from ~~either~~ the ends. Two stringers are really to acts 6 inches from the end but it will make no appreciable difference to take them acting as above.

From many examples we see that a locomotive has ~~its~~ the greater part of its weight resting on the driving wheels, and twenty tons over twelve feet is taken for our live load in calculations for the beam. The action of the live load is intermediate, between that of an absolutely sudden load and a perfectly gradual load, but in practice the additional strain from the swift motion of the train is provided for in the factor of safety, six.

	No.	Dimensions	Cu. ft.
Stringers	4	12'×8"×14"	37
Ties	9	13'×8"×8	52
			89
			55 wt. 1 cu. ft. of wood.
Rails	2	8 yds.	~~45~~95

65 lbs. per. yd.
520

One I beam weighs 1000 lbs.
3
3000

520
5415 dead wt.
Call 5400
2700 through pt. 5 ft. from end.
3 factor
8100

$M_0 = \frac{1}{8} Wl = \frac{3000}{8} \times 12 \times 15$
$= 67500$

Live load
2|40000
20000
6
120000
8100
128100 wt. on 1 pt.
60 length of arm.
2|7686000
3843000 as there are two I beams.

3843000
15|3910500
4000|260700
6.5 area of 1 flange.

6.5
6.5
7.2
20.2 area of I beam.

Because there are two equal couples at the ends of the beam, making the bending moment the same between the points referred to, we add this moment to the greatest moment from the weight alone and divide by 15 the height to obtain the horizontal stress. To find the sectional area of bolts to sustain I beams. Wt. at each end is 23700

2000 I beams.
40000 Locomotive
5400 Ties etc.

2)47400
23700

10000)23700
2.37

Take 2 bolts 1 1/2" diam. or 3/4" radius.

Area of cross section is therefore 1.77 sq. in.

2 × 1.77 = 3.54 where 2.37 is required.

The bolts are strong enough to bear the greatest weight that will ever come on them

The bolts to hold the I beams are held by the flange of the lower boom, and the mode of arranging the bolts is shown on the plate of details, fig. 4.

There is no danger of the bolt head being pulled through the flange.

For, the diameter of head of bolt is 3 in. ∴ the outer circumference of bolt head $= 2\pi r = 2 \times 3.14159 \times \frac{3}{2} = 9.42477$ Multiplying this by 1/2" = the distance from the bolt to outside of head, we have an area of 4.712 sq in. giving ample resistance to any force that may be applied to the bolt. The flange of the lower boom, holding the I beams by means of the bolts, and thus supporting the load, is fastened to the web by "angle irons" 4" × 4" × 1/2", as shown in fig. 3 + fig. 1, plate of details.

Now, to see if these are rivets enough, if placed 3" from centre to centre, and the rivets being 1⅛" in diameter, to hold the flange with its dependent load. It either gives way by tearing off the rivets in lower part of the angle irons or by shearing off those in upper part. Weight at one end of I beams is about 25,000 lbs. A rivet of 1⅛ diam. has an area of about 1 sq. in. Now 1 sq. in. will bear 10,000 lbs., so it would take less than three. Shear of one rivet 1 sq. in in cross section is 8000 lbs. therefore it would take only about 4 to hold, reckoning shear in upper part of angle iron. It may likewise give way by the rivets in upper part of angle iron, bearing through the ½ in.

iron composing the angle iron.
Bearing area $= 1\frac{1}{8} \times \frac{1}{2} = \frac{9}{16}$ 8000 lbs for bearing of 1 sq. in. gives about 4500 for bearing of one rivet. We thus see that it will take at least six, to hold. Having rivets at every three inches gives more than enough, for the weight at one point is distributed by the angle irons through the length of one panel or twelve feet.

To support the I beams, the bolts are placed between the two at each point, and support the castings shown in fig. 4, which gives the method of hanging the I beams.

Rivetting.

One sq. in. bears or shears 8000 lbs.
First calculate the number of rivets for the diagonals.
Diameter of rivets = 1"/2.
A rivet of this diameter bears 12,000 if the thickness of the diagonals is 1 in.

$$\text{Shear} = \pi r^2 \times 8000 = 3.14159 \times \tfrac{9}{16} \times 8000$$

= 14000 Take bearing.

In this calculation we use the stress in diagonals without using the factors of safety as it gives a larger number of rivets than stress with factors.

Page 16 gives the number of rivets in each diagonal, the disposition of the rivets being shown in the elevation of the half bridge on the sheet of details.

Fig. 1 shows the arrangement, on the first strut, a small piece of iron being inserted between the two members of the strut, in order to accommodate the required number of rivets.

For the upper boom, we use a less number of rivets than the calculation would give, because the parts of the boom abut one against the other and it is only necessary to keep them in place, which is done by the arrangement shown in fig. 7, using angle irons of 1/2 in thickness and rivets of 1½ diam. For the lower boom we use covering plates of 5/8" iron.

Bearing of 1½" rivet $= 3/2 \times \frac{5}{8} \times 8000 = 7500$.

Dividing the pulls in each panel as shown on p. 23 we have as the required number of rivets respectively 7.4, 22, 35, 46, 53, 60, 65, 68. Now as we do not have to depend entirely on the covering plates, because there are angle irons extending the length of the girder, of different lengths from the plates as cut, we are safe in calling the number of rivets at each point, 12, 18, 30, 42, 48, 54, 60, & 66 respectively.

The plates composing the flange of the lower boom are cut in the following manner, from the end to the centre.

The first is composed of iron 5/8" thickness, lengths 18'7", 24'; 24'; 24'; 12' which brings it to 6' beyond the centre point.

The second plate is 5/8" iron, and commencing at a point, 2' distant from the first bolt, is cut in lengths of 20'; 24'; 24'; 12'.5 and 11' carrying it 5'.5 beyond the centre. The third and lower plate is 3/4" in. which makes the total thickness of flange 2". The plate is cut in lengths of 20'; 24' and 12' commencing 2' nearer the supports, than the 2nd

point of suspension of I beams.
The end posts are rivetted to the bottom flange as shown in fig 1. and is cut at the top so as to admit of the upper boom fitting into it, the boom having as the side of the square a measurement of 8 in. The end of the boom projects three inches from the post and is fastened to the end post by angle irons similar to those holding the joints of the upper boom and shown in fig. 7. A square plate of 5/8" iron, 11" on a side, is capped over the end of the boom and held in place by the angle irons.

The diagonals are rivetted where they cross each other which makes the bridge stiffer.

For the upper and lower horizontal bracing we proceed as follows.

When a train of cars is on the bridge, the surface acted on by the wind, would be nearly of the area, 192′ × 18′, or considering one panel only the area acted on is 12′ × 18′ = 216 sq. or in round number 200 sq. ft.

A wind causing a pressure of more than 25 lbs to the sq. ft. would be a hurricane, and its effect would be to sweep away the structure from its points of support, or would be likely to overturn it. A train would never be run, when the wind causes such a pressure, so, for calculations regarding horizontal bracing we use a pressure of 25 lbs per. sq. ft. For the lower horizontals, we consider them all ties because of the I beams, and the wind blowing from one side or the other, and for this latter reason we see that the upper horizontals act alternately as struts or ties. We use for the lower the tensile strength of wrought iron, 10,000 lbs per sq. in. and for the upper we consider them as struts, using Gordons formula for the ultimate strength of wrought iron. We calculate for struts because wrought iron gives way easier by crushing. On each point we have acting 12 × 18 × 25 = 5400 lbs.

Sec. i = $\frac{19.2}{15}$ = 1.28 Pull or thrust = shear × sec i.

Increment	Panel	Shear	Pull	Sec. Area	Diam. given	Thrust	Req. Rivets	Given Rivets	Area Reg.	Area Given
5400	1	40500	51,840	5	2"	155,520	4.5	4	4.8	4
	2	35100	44,928	5	2"	134,784	3.7	4	4.2	4
	3	29700	38,016	4	1 3/4"	114,048	3.1	4	3.5	4
	4	24300	31,104	3	1 3/4"	93,312	2.5	3	2.9	3
	5	18900	24,192	2	1 1/2"	72,576	2.	3	2.2	3
	6	13500	17,280	2	1 1/2"	51,840	1.4	3	1.6	3
	7	8100	10,368	1	1 1/8"	31,104	.8	2	.9	3
	8	2700	3,456	1	1 1/8"	10,368	.03	2	.3	3

Ties — Lower Bracing | Struts — Upper Bracing

Gordons formula becomes, calling $h = 1$ and consequently $r^2 = \frac{h^2}{12} = \frac{1}{12}$ also $l^2 = 12^2 + 15^2 = 369$

$$\frac{P}{S} = \frac{36000}{1 + \frac{369}{\frac{36000}{12}}} = \frac{36000}{1 + \frac{369}{3000}} = \frac{36000}{\frac{3369}{3000}}$$

$$= \frac{108}{.003369} = \frac{36}{.001123} \quad \therefore \; S = \frac{P \times .001123}{36}$$

$$\frac{155520 \times .001123}{36} = 4.85$$

$$\frac{134,784 \times .001123}{36} = 4.20 \qquad .65$$

$$\frac{114,048 \times .001123}{36} = 3.55 \qquad .65$$

$$\frac{93,312 \times .001123}{36} = 2.91 \qquad .64$$

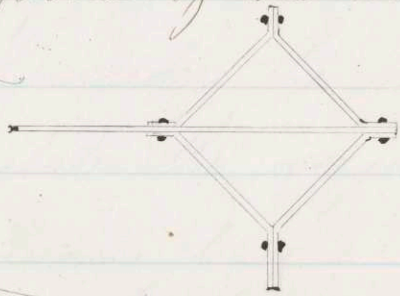
$$\frac{72,576 \times .001123}{36} = 2.26 \qquad .65$$

$$\frac{51,840 \times .001123}{36} = 1.61 \qquad .65$$

$$\frac{31,104 \times .001123}{36} = .96 \qquad .65$$

$$\frac{10,368 \times .001123}{36} = .31 \qquad .65$$

These upper horizontals pass through the upper and lower divisions of the cell that is used for the upper boom and are rivetted by the same rivets that keep the parts of the boom together.

Thus. 

The diagonals 4" wide are widened to 6" where they meet & pass through the parts of the cell, in order to make surface enough for rivets of 1½" diam. Rivets throughout are placed 3" from centre to centre.

The horizontal tie rods are 21.5 ft. in length, fastened as shown in plate of details.

A horizontal, perpendicular to the booms of 4"×1" cross section, and length 14' 8", is put from end post to end post.

The angle irons for the lower boom are cut in the following manner.

First 14½' from end, next 18' and all the rest except the remaining end, which is 14'5

Approximate Weight.

Consider the middle panel, and calculate the weight nearly. 1 cu. in. of iron weighs $\frac{5}{18}$ lbs.

2 I Beams. $l = 17$	=	2267
Stringers ties etc	=	5415
2 Separators	=	150
2 Supporters	=	500
2 Tie rods	$2\times 21.5\times 1\times 12\times\frac{5}{18}$	144
4 diag.	$\cancel{4}\times 20\times 1\times 3\times 12\times\frac{5}{18}$ =	800
flange lower boom	$2\times 14\times 12\times 12\times\frac{5}{18}$ =	1120
Web	$2\times 2\times 20\times 12\times 12\times\frac{5}{18}$	3200
Upper boom	$(72\times 12\times 12+7\times 12\times 12)\frac{10}{18}$ =	
	$(576+56)\,10 = 632 =$	6320
Ang. iron	$(4\times 4\times 12\times 12\times\frac{1}{2}+3\times 12\times 12\times\frac{1}{2}\times 4)\frac{5}{18}=$	
	$7\times 288\times\frac{5}{18}$ =	560
		20476 lbs.

The total dead load used for calculating the size of the pieces is 19,200 per panel. The 20,476 is actually larger than real weight because the dimensions are not taken exactly. Hence the bridge is safe.

H. K. Burrison

C. E.

May 1874.