Description of a Design for a Single Track Railroad Bridge of 190 Feet Span.

Written as a Thesis, by

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Design for a Single Track Railroad Bridge of 190 Feet Span.

Descriptive Paper.

Accompanying this paper are drawings of a single track railroad bridge of 190 feet span.

The bridge is designed to be built with wrought iron tension members of at least 50000 lbs ultimate strength, and with cast iron compression members of at least 80000 lbs ultimate strength in short lengths.

In designing, the attempt has been to obtain as much of architectural effect as possible, without the addition of passive material or dead weight, as is sometimes done in the ornamentation of wrought iron bridges by elaborate castings.

The principal dimensions of the bridge are here given:—

Length, 192 feet from centre to centre of end pins
Height, 18 " " " " " " chord"
Length of panel, 12 feet, number of panels, 16; width of bridge from centre to centre of chords, 17 feet.

The bridge is of the simple rectangular truss form, the ties having a compass of one panel, the end post being inclined, necessitating the substitution of a vertical suspension bolt for the first post.

The Floor.

The floor is designed to sustain a heavy traffic, being calculated to bear the most severe action of a Baldwin Consolidation Locomotive, carrying 80 000 lbs on 15 feet of wheel base.

The form of the floor proper is thought to be such as to prevent the possibility of the contact of any part of a train of cars with the truss, in any ordinary case of derailment.

Spruce ties are used for lightness, the ties not having to resist any bending action, except in event of derailment.

The general form and details of the floor are shown by the drawings.

Bill No. 1. contains a minute statement of dimensions and estimate of weight of the floor, longitudinal bearers and cross bearers.

The stresses upon the bearers are given upon the stress sheet.

The I beams of the longitudinal bearer are of sufficient area to resist the bending action of a weight of 10 000 lbs. in the middle of one or more of their unsupported lengths, in addition to the stress of direct compression, figured upon the diagram.

The Lateral Bracing

The lower bracing is proportioned to resist a uniformly distributed stress of 300 lbs. per foot of length, acting in the same manner as a rolling load.

This stress might be caused by a wind pressure of 20 lbs. to the square ~~foot~~ acting upon an area of 15 feet.

In the case of a strong lateral wind, acting upon a train of cars moving over the bridge, the lower bracing would have to bear the whole action of the wind upon the side of the train and floor, in addition to half of its pressure upon the truss proper.

No addition has been made to the area of the cross bearers or chord links on account of this stress.

The upper bracing is proportioned to resist a uniformly distributed stress of 100 lbs. per foot of length acting as a dead pressure over the whole length of the truss.

This stress might be caused by a force of 50 lbs. per square foot acting over two feet of area.

The stresses from wind are figured upon the stress sheet. The dimensions of the members of the bracing are marked upon the same.

Dimensions, sectional area, unit stress, limit of safe stress as found by Gordon's formula, and approximate weight of members of the lateral bracing are to be found upon Bill No. 2.

The Truss Proper.

The truss proper is designed to sustain a dead load of 900 lbs. per foot of length, considered as uniformly distributed, half upon the upper and half upon the lower chord; and also, to resist the worst action of a rolling load of 1200 lbs. per foot of length, considered as uniformly distributed over a part or all of the lower chord.

The stresses as found under these suppositions are marked upon the stress sheet.

The principal dimensions of the members of the truss are placed upon the diagram.

Bill No. 3. contains a statement of the dimensions, sectional area and unit stress, together with approximate weight, of members.

This approximate weight is found by adding to the weight of a piece of uniform section and given length, a certain percentage to allow for connections.

Bill No. 4 contains the dimensions, weight &c of end posts and suspension bars, together with weight of end bracing and a summary of weights constituting the entire weight of the bridge.

The Chord Links

The chord links are flat bars of uniform thickness of 1 inch and varying depth.

The standard form of eye is shown upon the detail sheet. This form must be somewhat altered where a bar is to be joined to another of greater depth than itself.

With the standard eye, the weight of a rectangular bar and two eyes is found by adding to the weight of a bar of uniform section and of length equal to the distance between centres of pins, the weight of a bar of the same section and of length equal to $6\,2/3$ times the depth of the bar.

This weight includes the weight of a section of both pins of the thickness of the link.

The Compression Members.

The compression members of the truss proper are double hollow cylindrical pillars, as shown in section on the detail sheet.
They are to be cast separately, the projecting rims to be planed, the posts to be bolted together through lugs at their ends, and keyed by steel keys driven into double dovetails cut out from their contact faces.
The proportions of bolts and lugs, together with number and size of keys could best be determined by experiment.
The outer diameter of the upper chord struts is uniform, the struts being transformed into square boxes at the end, these boxes to have planed but joints, to be covered on top by a square, flat, casting, having a boss for the attachment of the laterals cast in a piece with it.
The chord boxes are to have bolted to them below, by four bolts which also secure the top casting, another square, flat casting, having a tenon cast in a piece with it, to fit into the vertical post.
The following table shows the safe working stress per square inch for all of the cast iron compression members of the truss, as found by Gordon's formula, taking for the safe stress on any strut, one sixth

of its breaking stress.

The formula used is

$$\frac{P}{S} = \frac{1}{6}\left[\frac{80000.}{1+\frac{1}{400}\left(\frac{l}{h}\right)^2}\right]$$

$\frac{P}{S}$ being the safe working stress per square inch of section; l, the length, and h, the outer diameter of the strut.

Working Stress of Cast Iron Struts

Length of Strut in feet	Length of Strut in inches $= l$	Outer diameter in inches $= h$	$\frac{l}{h}$	$1+\frac{1}{400}\left(\frac{l}{h}\right)^2$	$\frac{80000}{1+\frac{1}{400}\left(\frac{l}{h}\right)^2}$ lbs.	$\frac{1}{6}\left[\frac{80000}{1+\frac{1}{400}\left(\frac{l}{h}\right)^2}\right]$ lbs.
12	144	11	13.1	1.429	55984	9331
20.5	246	10	24.6	2.513	31835	5306
17	204	9	22.67	2.285	44084	7347
17	204	8	25.5	2.626	30465	5077.5
17	204	7	29.14	3.123	25617	4269.5
17	204	6	34	3.890	20566	3428
17	204	5	40.8	5.162	15498	2583
17	204	4	51	7.503	10663	1777
17	204	3	68	12.56	6369	1061.5

The actual inch stress of the principal struts is made much less than that above tabulated (See Bill, 3.)

The sectional Area of the Compression Members

Upon inspection of Bill No. 3. it will appear that the unit stress to which any compression member is subjected is always less than the calculated safe working stress for the member, this stress being one sixth of the breaking stress as found by Gordon's formula.

It is also apparent that the difference between these two quantities is greater at the end of the chord than at the middle, and greater for the middle vertical posts than for the extreme ones.

There are four reasons why this decrease in stress is necessary.

First:- In calculating, by Gordon's formula, the safe stress, the outer diameter of the pillar has been used instead of the mean diameter. The substitution of the latter would give a smaller value to this safe limit.

Second:- In using Gordon's formula, no account has been taken of the thickness of the ring of the pillar. Manifestly, the outer diameter being constant, the pillar of a thin shell will bear a less unit stress than that of a thick one.

Third:- In order to exert a stress upon the centre segments of the top chord, and the end vertical posts, equal to the calculated total stress, the whole truss must be loaded with its maximum load, which, if this maximum has been assumed sufficiently great, will rarely take place; whereas the centre web members are strained almost always up to, and sometimes over the calculated value for the maximum load by the passage of a single locomotive, followed by an ordinary train.

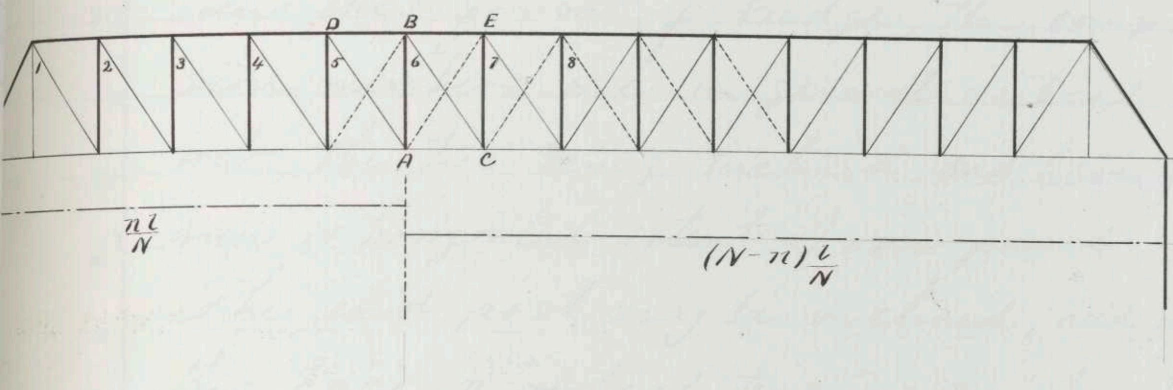
The end segments of the chord are strained in a greater proportion to the maximum, by the daily load, than the centre ones.

Fourth:- The compression members are all liable to diminution of area by oxidation, and the smaller the member, the greater, the proportional wear.

All of these considerations point to the use of a greater margin of safety for the end segments of the chord than for those of the centre, for the centre web members than for the end.

(Note. There has been no attempt to calculate exactly the required area in any case, as the number of considerations involved would make the calculation consume much more than the time at hand.

The proportioning of the members is undeniably rough, and might be greatly improved. No attempt has been made to perform even a tithe of the work necessary in designing an actual bridge of this pattern, but I have no hesitation in saying that I believe the proportions of the compression members to be more nearly adapted to the requirements of a complete theory, than those obtainable by the use of Gordon's formula, with a constant factor of safety throughout.


D
B
E
1
2
3
4
5
6
7
8
A
C
nl/N
(N-n) l/N

Method of Computing the Stresses upon the Members of a Single Intersection, Rectangular Truss, Through Bridge

In this form of bridge, the compression members are in general vertical in the web, the ties being inclined and having a run or horizontal extent of one panel.
The end post may be inclined, and in this case a vertical tension member is substituted for the first post from the end.

Notation

Let w be the total weight of a panel length of the bridge, considered as uniformly distributed, one half along the top and one half along the bottom chord, the web members being considered as devoid of weight.
Let w' be the weight of a panel length of the rolling load, this load being considered as uniformly distributed over a part or the whole of the truss, but always extending to one abutment.
This will be called the loaded abutment, as always bearing the greater load, while its opposite will be called the unloaded abutment.

Let l be the length of the truss.
" N be the whole number of panels in the truss
" n be the number of panels between any intersection or panel point and the nearest abutment
Let k be the depth of the truss, H_n the horizontal stress in the segment of the chord next the n^{th} panel point. and V_n the vertical stress in the n^{th} post.

Horizontal Stresses

The greatest bending moment at any panel point occurs when the rolling load covers the whole truss. Consider the chord segment AC to be severed, the truss will then tend to turn about B, its moment being $(w+w')\frac{N}{2}.\frac{nl}{N} - (w+w')n\frac{nl}{2N} = \frac{(w+w')l}{2N}.n(N-n)$. Now this moment is balanced by the moment $H_n k$ of the chord stress, therefore

$$H_n = \frac{(w+w')l}{2Nk}.n(N-n) \qquad (1.)$$

Vertical Stress from Weight of Truss.

Consider an even number of panels, and a centre post. The stress on the centre post is $\frac{1}{2}w$

On the tie next the centre, $\frac{1}{2}w$,
On the 1st post from the centre, w,
On the 2nd tie " " ", $\frac{3}{2}w$,
" " " post " " ", $2w$,

And if n' is the number of any post from the centre the stress on that post will be $n'w$. (The centre post is an exception)
The stress on the n^{th} tie from the centre is $n'w - \frac{1}{2}w$

Now $n' = \frac{1}{2}N - n$, therefore for the n^{th} post from the abutment;
Vertical stress from weight of truss $= w(\frac{1}{2}N - n)$ (2.),
and for the tie on the centre side of that post, vertical component of stress $= w(\frac{1}{2}N-n)-\frac{1}{2}w$ (3.)

Vertical Stress from Rolling Load.

Case I The greater segment loaded — the ties slope upward from the load.
Let the load cover the truss from the right hand abutment to the foot of the n^{th} tie from the left hand abutment. The load covers $(N-n-1)$ panel lengths. The supporting force of the left hand abutment is $w'(N-n-1) . \frac{(N-n-1)}{2N}$
$= \frac{w'}{2N}(N-n-1)^2$ and this is the vertical stress

in the n^{th} post and the vertical component of the stress in the n^{th} tie.

Now consider the load to extend up to the foot of the $(n-1)^{st}$ tie, covering $(N-n)$ panels.

Then, in addition to the load $w'(N-n-1)$ just considered, there will be a load of $\frac{1}{2}w'$ upon the n^{th} panel point and a load of $\frac{1}{2}w'$ upon the $(n+1)^{st}$ panel point.

Of the $\frac{1}{2}w'$ upon the $(n+1)^{st}$ panel point, the $\frac{N-n-1}{N}$ th part is borne by the unloaded abutment, travelling through the n^{th} tie and post and causing an additional stress amounting to $\frac{(N-n-1)w'}{2N}$ in these members.

Of the $\frac{1}{2}w'$ upon the n^{th} panel point, the $\frac{n}{N}^{th}$ part is borne by the loaded abutment, passing through the n^{th} post and tie and causing a <u>counterstress</u> of value $\frac{nw'}{2N}$

Now, the difference between this additional direct and counter stress, added to the direct stress from the load covering $(N-n-1)$ panels, will form the total stress in the n^{th} tie and post.

Hence, the vertical stress from the rolling load on the n^{th} tie and post is $\frac{w'}{2N}\left[(N-n-1)^2+(N-2n-1)\right]$ (4.)

And the stress on the n^{th} post from both dead and live loads is

$$V_n = w(\tfrac{1}{2}N - n) + \frac{w'}{2N}\left[(N-n-1)^2 + (N-2n-1)\right] \quad (5.)$$

For the stress on the n^{th} tie, subtract $\frac{1}{2}w$. It is plain that when $n > \frac{N}{2}$, $2n > N$ and $(N-2n-1)$ becomes negative; that is:—
When less than one half of the truss is covered with the rolling load, the greatest stress in any tie and in the brace connected with its upper end exists when the rolling load extends to, and not beyond the foot of that tie. (The tie in this case is a counter.)

Case II The lesser segment loaded.

The counter ties slope upward from the load.

The greatest stress in the n^{th} counter AE is equal to the difference between the portion of the live load that passes through the tie to the unloaded abutment, and the portion of the dead load which passes through the n^{th} main tie to the loaded abutment.

The first term is greatest when the live load covers n panels and is $w'n.\frac{n}{2N} = \frac{w'n^2}{2N}$

The second term is $w(\frac{1}{2}N-n)-\frac{1}{2}w = w(\frac{N-1}{2}-n)$

Hence for the vertical component of the stress in the n^{th} counter tie, we find

$$v_n = \frac{w'n^2}{2N} - w\left(\frac{N-1}{2}-n\right) \qquad (6.)$$

To find the direct stress in any tie, we multiply the vertical component of the stress in that tie as above found by the ratio $\frac{s}{k}$ $= \frac{\text{length of tie}}{\text{depth of truss}}$.

When the 0 or end post is inclined the stress upon it is the same as upon the tie adjoining the end post in a truss with a vertical end post, or the 0 tie, which does not exist in this case. The 1st post is omitted, its place being supplied by a vertical suspension bar or bolt. whose office is to support a panel length of lower chord, floor and <u>concentrated</u> live load. The stresses in the 0 and 1st segments of the lower chord are equal, in this case, for the truss tends to turn about 1 whichever segment be severed.

Calculation of Stresses upon Truss Proper.

Data. $l = 192$. $k = 18$. $N = 16$. $\frac{l}{N} = 12$

$s = \sqrt{12^2 + 18^2} = 21.63 \quad \frac{s}{k} = \frac{21.63}{18} = 1.2$

$$\left.\begin{array}{l} w = 900 \times 12 = 10800 \\ w' = 1200 \times 12 = \underline{14400} \\ w + w' = 2100 \times 12 = 25200 \end{array}\right\} \quad \frac{l}{k}\frac{(w+w')}{2N} = \frac{192}{18} \cdot \frac{25200}{32} = 8400$$

Calculation of Horizontal Stress.

n	$N-n$	$n(N-n)$	H_n
0	16	0	0
1	15	15	126000
2	14	28	235200
3	13	39	327600
4	12	48	403200
5	11	55	462000
6	10	60	504000
7	9	63	529200
8	8	64	537600

Calculation of Vertical Stress.

$$\frac{w'}{2N} = \frac{14400}{32} = 450$$

n	$\frac{1}{2}N-n$	$w(\frac{1}{2}N-n)$	$N-n-1$	$(N-n-1)^2$	$\frac{w'}{2N}(N-n-1)^2$	$N-2n-1$	$\frac{w'}{2N}(N-2n-1)$	Total vertical stress on post and tie from live load	V_n
0	8	86400	15	225	101250	15	6750	108000	194400
1	7	75600	14	196	88200	13	5850	94050	169650
2	6	64800	13	169	76050	11	4950	81000	145800
3	5	54000	12	144	64800	9	4050	68850	122850
4	4	43200	11	121	54450	7	3150	57600	100800
5	3	32400	10	100	45000	5	2250	47250	79650
6	2	21600	9	81	36450	3	1350	37800	59400
7	1	10800	8	64	28800	1	450	29250	40,050
8	0	5400	7	49	22050	−1	—	22050	27450

Stress in Ties — Counter Ties

n	$T_n\frac{k}{s} = V_n - \frac{1}{2}w$	T_n	n^2	$\frac{w'n^2}{2N}$	$\frac{N-1}{2}-n$	$w(\frac{N-1}{2}-n)$	v_n	t_n
0	189000	226800 *						
1	164250	197100						
2	140400	168480						
3	117450	140940						
4	95400	114480						
5	74250	89100	25	11250	2.5	27000		
6	54000	64800	36	16200	1.5	16200		
7	34650	41580	49	22050	.5	5400	16650	19980
8		* End post.	64	—	−.5	−5400		

Bills of Dimensions and Weights.

Bill No. 1.

Half Panel Weight of Floor, Floor Beavers and Cross Beavers

Rails, spikes and joints, 4 yds, at 65 lbs per yd	260	
Half of cross tie 6"x8"x5' = 5/3 cu. ft. - at 30 lbs = 50 lbs		
per half tie. 3 ties in 4 ft. 9 ties in 12' - 9 x 50	450	
Guard timber 10"x10"x12' = 8 1/3 cu. ft - at 30 lbs	250	
Wedge shaped blocks, spiked to tie, 5/12 cu. ft, at 30 lbs		
12.5 lbs each - 9 blocks - 9 x 12.5	112	5
Top long'l covering planking 2"x4'x12' = 8 cu. ft. at 30	240	
Side " " plank 11"x2"x12' = 11/6 cu. ft. at 30	55	
Cushion plank (on floor beaver) 2"x10"x12' = 5/3 cu. ft - 30 -	50	
Guard timber bolts		
8 - 3/4" bolts each 1'10" long, weighing with head		
nut and washer 4 lbs each	32	
1 - 1" bolt 3'2" long, with fastenings	11	
Small strut 1'10" x 3" x 8" = 11/36 cu. ft. at 30	9	1
Spikes - 4 1/2 x 1/2" at 3/8 lb. each, 24 in one tie } 4.5 lbs per half tie }		
3 spikes in wedge block 6" x 9/16 at 4/7 lb each } 1.7 lbs per half tie }		
Total 6.2 lbs per half tie 6.2 x 9	55	8
	1527	4

Longitudinal Floor Beaver

2 - 9" light beams, 23 1/3 lbs each, 46 2/3 lbs. pr. ft. x 12'	560	
1 - 2" bolt 12.5 long at 10.6 per ft.	132	5
	692	5

Bill No. 1. (continued)

(2.)

Half Panel Weight of Floor, Floor Bearers and Cross Bearers

Brought forward on Long'l Floor Bearer.	692	5
2 castings at 8 lbs each	16	
6 - 3/4" connecting bolts with collars at 5 lbs. each	30	
1 - 1" bolt 7" long (to fasten down to cross b.) with nut + wash's	3	
2 spikes 4 1/2 x 1/2		75
Cushion block 1" x 9" x 4" = 1/4 cu. ft. at 30	7	5
	749	75
Cross Bearers		
2 - 6" beams, 13 1/3 lbs, per. ft. each - 26 2/3 x 8.5	227	
1 - 3" bolt, 9.5 long at 23.8 pr. ft.	226	1
1 - Casting 50 lbs	50	
2 - Connecting bolts with collars, at 5 lbs	10	
Nut and block for end of 3" bolt	35	
	548	1
Supporting box and bolts for cross bearers Estimated at	100	
Total weight of panel length of floor &c borne by one truss	2923	25
Total weight of floor &c borne by one half of the truss	23386	

Bill No. 2.

Lower Latural Tie Bolts

number of member	diameter of bolt	calculated total stress	calculated area for 10000 lbs working stress	actual area	weight per ft.	Total weight	
1	2¼	38 000	3.8	3.9	13.4	268	
2	2⅛	33000	3.3	3.5	12.0	240	
3	2	29000	2.9	3.1	10.6	212	
4	1¾	25 000	2.5	2.4	8.10	162	
5	1⅝	21000	2.1	2.0	6.99	139	8
6	1½	16 000	1.6	1.8	5.95	119	
7	1⅜	15 000	1.5	1.5	5.00	100	
8	1¼	12000	1.2	1.2	4.13	82	6
Ties all calculated for weight as 20' long						1323	4

number of member	diameter of bolt	calculated total stress	calculated area for 10000 lbs. working stress	actual area	weight per foot	Total weight	
2	1⅛	10 000	1.0	1.	3.35	73	8
3	1⅛	9000	.9	1.	3.35	70	4
4	1	7000	.7	.8	2.65	55	6
5	⅞	6000	.6	.6	2.03	42	6
6	⅞	5000	.5	.6	2.03	42	6
7	¾	3000	.3	.4	1.49	31	4
8	⅝	2000	.2	.3	1.03	21	6
Ties all calculated for weight as 21' long, except no. 2. which, take 22'.						358	0

Bill No. 2. (continued)

Upper Lateral Struts

number of member	diameter	thickness	area of one strut	area of both struts	calculated total stress	calculated inch stress	limit of safe stress	wt. per foot	Total weight	
1	4"	1/4"	3.39	6.78	8400	1239	1777	21.20	169	6
2	3	1/4	3.39	6.78	7200	1062	1777	21.20	169	6
3	3	1/4	2.41	4.82	6000	1245	1061.5	15.06	120	5
4	4	3/8	4.27		4800	1124	1777	13.36	106	9
5	4	3/8	4.27		3600	843	1777	13.36	106	9
6	4	1/4	2.95		2400	814	1777	9.22	73	8
7	3	1/4	2.16		1200	556	1061.5	6.75	54	0
8	3	1/4	2.16		0	0	1061.5	6.75	27	0
									828	3

Summary of weights of Lateral Bracing

Lower Ties	1323.4
Upper "	358.0
" Struts	828.3
	2509.7

Take 25 lbs. as average weight of bosses or projections on upper chord castings for connection of laterals – 8 of them } 200.

Total weight to form item Lateral Bracing 2709.7

Bill No. 3.

Lower Chord Links

number of member	no. of pieces in member	dimensions of each piece inches		calculated total stress	calculated area for 10000 lbs working stress	actual area	total weight of member	
1	4	3¼	1	126000	12.6	13	650	
2	4	3¼	"	126000	12.6	13	650	
3	6	4	"	235200	23.52	24	1200	
4	8	4⅛	"	327600	32.76	33	1650	
5	8	5⅛	"	403200	40.32	41	2050	
6	9	5¼	"	462000	46.20	47.25	2362	5
7	8	6⅜	"	504000	50.40	51	2550	6
8	9	5⅞	"	529200	52.92	53.875	2693	6
weight of wrought iron piece = $\frac{10}{3}$ x length in ft. x sectional area in sq in. For chord links = $\frac{10}{3}$ x 12 x a = 40 a add 25 % for connections making wt. of link = 50 a							13806	1

Diagonal Tie Bars.

number of member	no. of pieces in member	dimensions of each piece		calculated total stress	calculated area for 10000 lbs. working stress	actual area	total weight of member	
1	2	5	2	197100	19.71	20	1600	
2	"	4¼	2	168480	16.84	17	1360	
3	"	4	1¾	140940	14.09	14	1120	
4	"	4	1½	114480	11.45	12	960	
5	"	3	1½	89100	8.91	9	720	
6	"	3¼	1	64800	6.48	6.5	520	
7	"	2¼	1	41580	4.16	4.5	360	
weight of tie = $\frac{10}{3}$ x 21.63 x sectional area = 72 add 11 % for connections making wt = 80 x a							6640	

Bill No. 8. (continued) (6.)

Upper Chord Struts

number of members	diameter	thickness	area of one strut	area of both struts	calculated total stress	calculated inch stress	limit of safe stress	total weight of member
2	11"	1/2	19.85	39.70	235200	5924	9331	1985
3	"	5/8	23.73	46.46	327600	7051	"	2325
4	"	3/4	27.51	55.02	403200	7328	"	2750
5	"	13/16	29.36	58.72	462000	7868	"	2935
6	"	7/8	31.19	62.38	504000	8080	"	3120
7	"	15/16	33.00	66.00	529200	8018	"	3300
8	"	1	34.78	69.56	537600	7729	"	3480
Weight of cast iron piece = $\frac{25}{8}$ x sectional area x length. For this strut, weight = $\frac{25}{8}$ x 12 x a = 37.5 a. Add 33 o/o for connections making w = 50 a								19895

Vertical Posts

number of members	diameter	thickness	area of one strut	area of both struts	calculated total stress	calculated inch stress	limit of safe stress	total weight of member
2	9"	1/2		31.20	145800	4673	7347	2184
3	9	3/8		24.82	122850	4950	"	1736
4	8	1/2		27.12	100800	3717	5077.5	1897
5	8	3/8		21.52	79650	3701	"	1505
6	7	1/2		25.14	59400	2363	4269.5	1764
7	7	3/8		18.33	40050	2185	"	1288
8	6	3/8		14.25	27450	1949	3428	1004
w = 17 a x $\frac{25}{8}$ = 53 add 32 o/o for connections making w = 70 a								11378

Bill No. 4.

End Post, End Panel Suspension Bars and End Bracing.

End Post

Diam 10", thickness 3/4", sectional area 49.15, calculated total stress 226 800, calc. inch stress 4614, limit of safe stress (Gordon) 5306. Weight | 4428

End Suspension Bars

2 bars 1 1/2 x 1" sectional area 3" calculated stress 26000, calculated area for 10000 lbs inch stress 2.6. Weight | 195

Counter Ties

areas		
2.45	196	
1.57	125	
.88	70	4
	392	0

End Bracing - Entrance Arch

1 Quadrantal cast cylinder 12' long, outer diam. 4" thickness 1/4". wt. per ft run 9.22. Total weight | 110 | 6

Circular castings as follows

no.	diam.	circum.	sectional area	volume
1	2.3	7.3	5	36.5
2	1.1	3.5	4	28.0
2	.6	1.7	3	10.2

	74.7 cu.in	
Add for fastenings - buttons, entrance plate &c	25.3	
making	100. cu.in	25
Total weight of 1/2 of entrance arch		136

Bill No. 4. (continued) (8)

Summary of Weights

Making up the weight of the Quarter Bridge.

Floor.	23386	
Lateral Bracing	2709	7
Chord Links	13806	1
Main Ties	6640	
Chord Struts	19895	
Vertical Posts	11378	
End Post	4428	
End Suspension Bar	195	
Counter Ties	392	
End Bracing	136	
Upper Chord - castings - 8 averaging 100	800	
Lower Chord - " - 9 " 250	2250	
Total weight of Quarter Bridge Being weight borne by ½ of one truss.	86016	6
Average panel weight	10752	1
Average weight per foot of length.	895	2