准員　守田兵藏君
准員　舟橋喜一君

論說及報告

いんふりうゑんすらいん(Influence line)ニ就テ(承第二百六十卷)

井上清介君

○實例第貳

第八圖ヨリ第拾貳圖ニ至ル五圖ハ悉ク聚合荷重ニ對スル實例ヲ示ス内第八圖ハ肱木形橋ノ略圖ヲ示シ、第九圖ハ車軸荷重ノ圖表、第拾圖ハ第八圖ニ示セル肱木形橋中第二格間諸材ノ法線圖、第拾壹圖ハ第五格間、第拾貳圖ハ第八格間諸材ノ法線ヲ示ス、第壹表ハ肱木形橋全部ノ應力表ナリ

○第拾圖　對角柱材(g')ノ應力算法

Reactions

L. H. R.—a load unity at (9) $= \frac{63}{105} = \frac{3}{5}$.

〃　—〃　〃　〃　〃 (2); $= \frac{63}{105} = \frac{3}{5}$.

L. H. R. — a load unity at (1) $= \frac{84}{105} = \frac{4}{5}$.

Ordinates 9 $= \frac{3}{5} \times \frac{147}{189} = \frac{21}{45}$.　(2) $= \frac{3}{5} \times \frac{147}{189} = \frac{21}{45}$.

(1) $= \frac{4}{5} \times \frac{63}{189} = \frac{12}{45}$.

前記對角柱材(g_1)ハ重ニ張力ヲ受ク、今其最大應張力ヲ求メンニハ格關間(6)ヨリ(15)ニ至ル間ニ聚合荷重、(0)ヨリ(1)迄ニ等布荷重ヲ置ク可キモノトス然ルキハ其格間點(9)ニ於ケル最大力率ハ格間點(6)ヨリ(15)ニ至ル區間ノ荷重ニ依テ生ス、今九圖車軸圖表ニ依テ(9)點ニ最大力率ヲ與フル車軸量ヲ撿スルニ全荷重(11)號ガ(9)點ニ在ルトキニ生スルヲ知ル依テ左ニ其力率ヲ求ムレバ

車軸圖表中34—35間ノ力率ハ $= 38,779.850 + 3.5 \times 388.100 - 8.75 \times 190 = 38,475.900$

$M_9 = \frac{38,475.900}{3} = 12,825.500$ ft. lbs

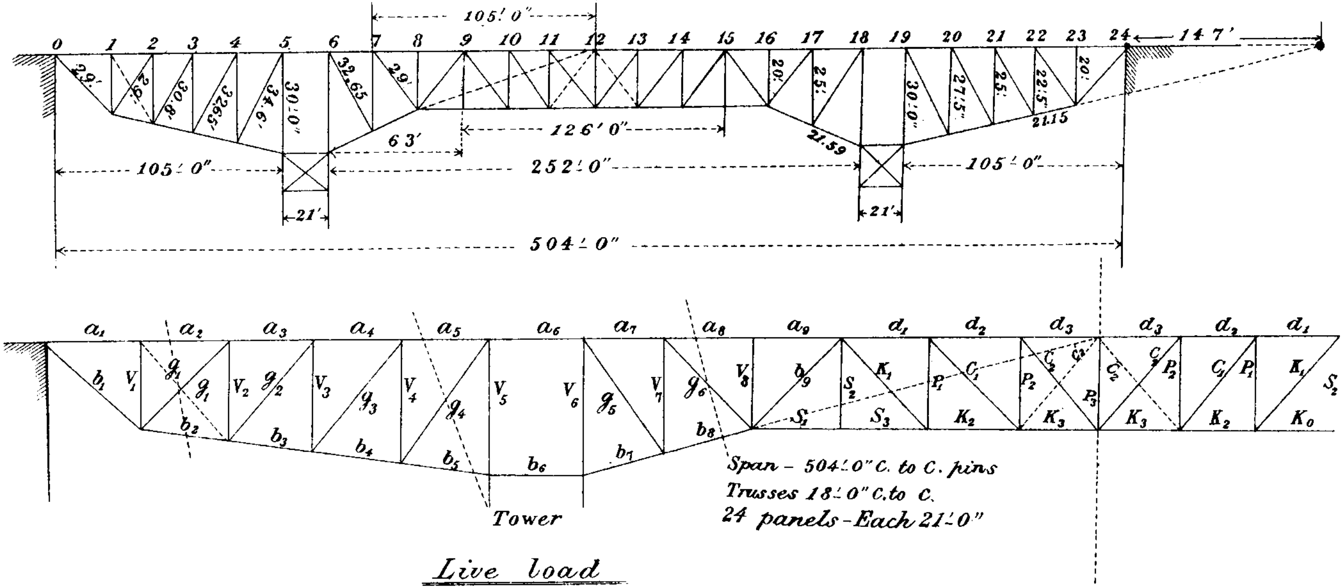
M_9 — a load unity at (9) $= \frac{63 \times 126}{189} = 42$.

V.C.S : $M_9 = \frac{21}{45} : 42$.　$\therefore$ V.C.S $= \frac{1}{90} M_9$

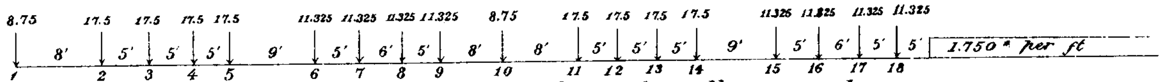
Max. tension in $g_1 = \left[1.750^{\#} \times A_1 + \frac{1}{90} M_9\right] \frac{29}{20}$.

$= \left(1.750 \times 3.82 + 142\,500\right) \frac{29}{20} = 216,500.^{\#}$

第八圖



Live load



Wheel loads are given as thousands of lbs per axle.

Dead loads

870 # per ft for the Suspended span.

1150 # " " " " Cantilever arm.

1.200 # " " " " Anchor arm.

Stresses

Bar	Live stress.	Dead stress.	Longitudinal force and stress	Bar	Live stress	Dead stress	Longitudinal force and stress
A_1	192.400 ten 121.700 comp	ten 4 500.	77.600 ten 49.600 comp	g_6	ten 209.200	ten 82.000	
A_2	192 400 ten 184 200 comp	″ 31.200	77.600 ten 37.200 comp	V_1	comp 101.900	comp 17.000	
A_3	342.000 ten 184.200 comp	″ 31.200	ten 77.600	V_2	comp 168.300	″ 51.500	
A_4	461.700 ten 119.400 comp	ten 74.400	ten 77.600	V_3	″ 168.000	″ 64.800	
A_5	ten 559.200	″ 128 300.	″ 77.600	V_4	″ 196.000	″ 88.900	
A_6	″ 641.300	″ 190.700.	″ 77.600	V_5	″ 263.000	″ 103.300	
A_7	″ 380.800	″ 133.400	″ 69.600	V_6	″ 192.800	″ 90.000	
A_8	″ 187.800	″ 70.200	″ 60.900	V_7	″ 182.200	comp 83.700	
A_9	″ 187.800	″ 70 200	″ 52.200	V_8	″ 57.700	″ 24.200	
b_1	266 000 comp 167.400 ten	comp 6.200		d_1	″ 148.000	″ 47.900	comp 45.000
b_2	344.400 comp 95.300 ten	″ 31 500		d_2	″ 262.300	″ 76.700	″ 36.000
b_3	465.500 comp 120.200 ten	″ 751.00		d_3	″ 292.100	″ 86.300	″ 27.000
b_4	comp 564.300	″ 129.200		K_1	ten 206.500	ten 63.900	
b_5	″ 64 .400	″ 192.300		K_2	″ 148.000	″ 47.900	
b_6	″ 641.300	″ 190.700		K_3	″ 262.300	″ 76.700	
b_7	″ 659.200	″ 196.200		P_1	comp 142.500	comp 40.700	
b_8	″ 391.500	″ 137.100		P_2	″ 81.100	″ 22.400	
b_9	″ 259·300	″ 96.700		P_3	″ 57.700	″ 27.700	
g_1	ten 216.500	ten 37.200.		c_1	ten 117.300	ten 39 700	
g_1'	ten 66.600	comp 41.200		c_2	″ 67.400	″ 13.200	
g_2	″ 207.500	ten 63.000		c_2'	″ 33.300	comp 13.200	
g_3	″ 212.500	″ 84.200.		Stre	sses in S_1	S_2 and S_3	are
g_4	″ 221.500	ten 103.200		those	due to	Erection	and Wind
g_5	″ 201.900	ten 91.000.			only.		

第八圖附屬第一表

本例ニ於ケル反應力、縱線等ノ算例幷ニ算式ノ說明ハ已ニ前段ニ於テ詳述シタル例ニ等シケレバ玆ニ省ク(以下倣之)

○第拾圖下臥材(b_2)ノ應力算法

本材ハ對角柱扣材(g'_1)ガ荷重ノ爲メ働ヲ受クルトキ應張力、同本材ガ働ヲ受クルトキ應壓力ヲ受ク、今扣材ガ働キヲ受クル場合ニ於ケル材ノ應張力算法ヲ左ニ示セバ

(b_2)材ガ應張力ヲ受クル場合、格間點(4)ノ最大力率ハ同點(0)ヨリ(5)ニ至ル區間ニ於テ車軸量(14)號ガ(4)點ニ在ルトキニ生ズ故ニ此力率ヲ本例ニ用ユレバ

$$M=\frac{11,877.725+710.3}{5}=2,517.500 \text{ ft lbs}$$

$$M-\text{a load unity at }(4)=\frac{4}{5}\times 21=\frac{84}{5}.$$

$$H.C.S : M=\frac{21}{25} : \frac{84}{5}.\ 1/1 \text{ in action} \qquad H.C.S=\frac{M}{20}$$

$$\text{Max. tension in } b_2=\frac{2,517.500}{20}\times\frac{21.15}{20}=126.800^{\#}.$$

$$\text{Dead compression in } b_2=31,500^{\#}$$

$$\therefore \text{ Actual stress}=126.800-31.500=95.300^{\#} \text{ ten.}$$

前記下臥材(b_2)ガ應壓力ヲ受クル場合ハ(9)點ノ最大力率ハ格間點(6)ヨリ(15)ニ至ル區間ニ於テ車軸量(11)號ガ(9)點ニアルトキ

$M=12{,}825{,}500^{\text{ft lbs}}$ ……………………………………(算法前ニ出ス)

$\text{H. C. S.} : M=\frac{42}{37.5}=42$, ($g_1$ in action)　$\text{H. C. S}=\frac{M}{37.5}=342{,}000.$

Max. compression in $b_2=342{,}000\times\frac{21.15}{21}=344.400^{\#}.$

○第拾圖垂直材(V_2)ノ應力算法

本材ハ重ニ壓力ヲ受ク、其最大應壓力ハ格間點(6)ヨリ(15)ニ聚合荷重、(0)ヨリ(2)點ニ等布荷重ヲ置クトキニ生スルモノトス、聚合荷重ノ配置又(9)點ノ最大力率等ハ對角柱材(g_1)ノ例ニ等シ

$\text{V. C. S.} : M=\frac{21}{45}:42.$　$\text{V. C. S.}=\frac{M}{90}.$　$M=12{,}825.500^{\text{ft lbs}}$

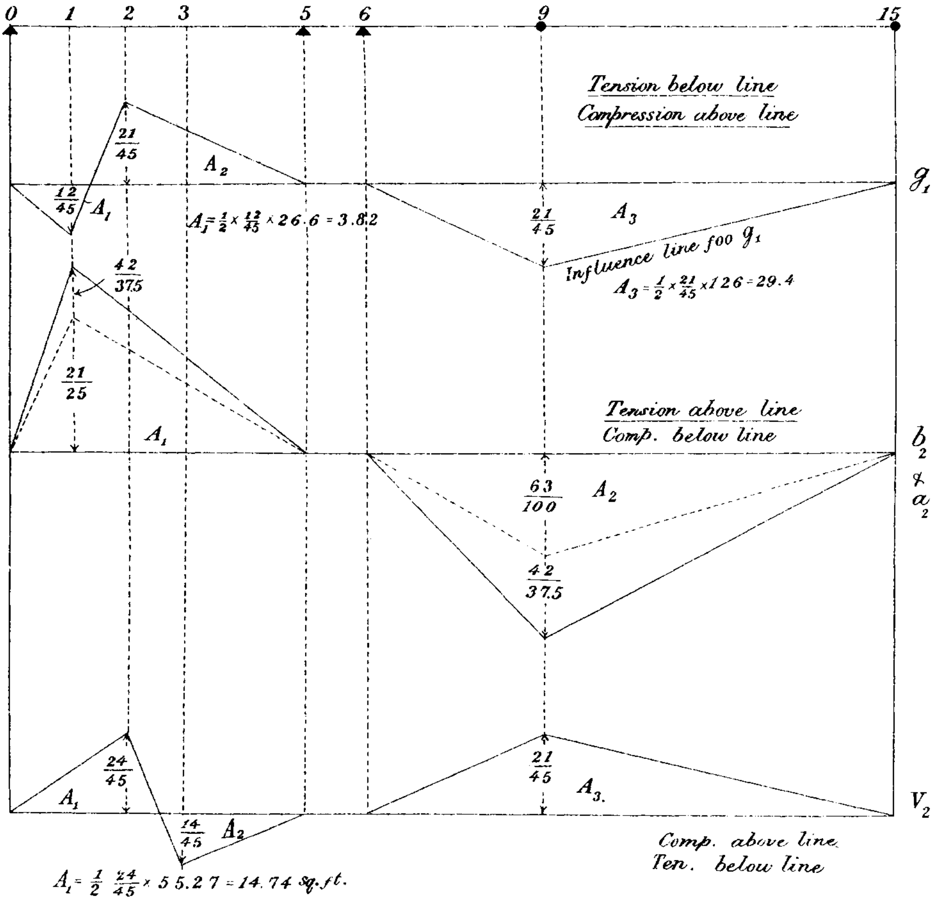
$$\text{Max. comp. in } V_2=\frac{12{,}825.500}{90}+A\times 1'750.$$

$$=142.500+14.74\times 1.750=168.300^{\#}.$$

○第拾圖上臥材(A_2)ノ應力算法

本材ハ下臥材(b_2)ニ用ヒタル法線ニ依テ其應力ヲ算出シ得ベシ、然レ𪜈荷重ノ關係ハ下臥材ノ場合ト全ク反對ニシテ、張力ニ對スル有効荷重ハ格間點(6)ヨリ(15)迄、壓力ニ對スル荷重ハ(0)ヨリ(5)迄ナリトス、而シテ對角柱材ノ扣材カ働キヲ受クルトキ本材ハ壓力ヲ受ケ對角柱本材カ働キヲ受クルトキ、本材ハ張力ヲ受クルモノトス

For tension load —— 6—15 actual loads.



0
1
2
3
5
6
9
15
Tension below line
Compression above line
21/45
A2
12/45
A1
A1 = 1/2 × 12/45 × 26.6 = 3.82
21/45
A3
g1
Influence line foo g1
A3 = 1/2 × 21/45 × 126 = 29.4
42/375
21/25
A1
Tension above line
Comp. below line
b2 & a2
63/100
A2
42/375
24/45
A1
14/45
A2
A1 = 1/2 24/45 × 55.27 = 14.74 sq.ft.
21/45
A3.
V2
Comp. above line
Ten. below line

第拾圖

第九圖

	Moments in thousands of foot pounds	Sum of loads in thousands of pounds		Loads in thousands of lbs & Distances in ft.		Sum of distances in ft.
1st Engine	0	8.75	(1)	8.75	8'	0
	70.00	26.25	(2)	17.5	5'	8'
	201.25	43.75	(3)	17.5	5'	13'
	420.00	61.25	(4)	17.5	5'	18'
	726.25	78.75	(5)	17.5	9'	23'
	1.435.00	90.075	(6)	11.325	5'	32'
	1.885.375	101.400	(7)	11.325	5'	37'
	2.493.775	112.725	(8)	11.325	5'	43'
	3.057.400	124.050	(9)	11.325	8'	48
2nd Engine	4.049.80	132'80	(10)	8.75	8'	56
	5 112.200	150.30	(11)	17.5	5'	64
	5.863.70	167.80	(12)	17'5	6'	69
	6.702.70	185.30	(13)	17.5	5'	74
	7.629.20	202.80	(14)	1.75	9'	79
	9.454.40	214.125	(15)	11 325	5'	88
	10.525.025	225.45	(16)	11.325	6'	93
	11.877.725	236.775	(17)	11.325	5'	99
	13.061.600	248.100	(18)	11.325	75'	104
	14.302.00 14.922.35	256.850	(19)	8.75	5'	111.5
	16,206.60	265·60	(20)	8.75	5'	116.5
	17.534.60	274.35	(21)	8.75	5'	121.5
	18.906.35	283.10	(22)	8.75	5'	126.5
	20.321.85	291.85	(23)	8.75	5'	131.5
	21.781.10	300.60	(24)	8.75	5'	136.5
	23.284.10	309.35	(25)	8.75	5'	141.5
	24.830.85	318.10	(26)	8.75	5'	146.5
	26.421.35	326.85	(27)	8.75	5'	151.5
	28.055.60	335.60	(28)	8.75	5'	156.5
	29.733.60	344.35	(29)	8.75	5'	161.5
	31.455.35	353.10	(30)	8.75	5'	166.5
	33.220.85	361.85	(31)	8.75	5'	171.5
	35.030.10	370·60	(32)	8.75	5'	176.5
	36.883.10	379.35	(33)	8.75	5'	181.5
	38.779.85	388.10	(34)	8.75	5'	186.5
	40.720.35	396.85	(35)	8.75		191.5
				1.750lbs per ft.		

$S : M = \frac{68}{100} : 42$ (g_1 in action).　　　$S = \frac{3}{200}M.$

Mx. tension in $\underline{\underline{A_2}} = \frac{3 \times 12,825.500}{200} = 192.400^{\#}.$

For comp. load——o－5. actual loads.

$S : M = \frac{42}{37.5} : \frac{126}{5}$. ($g_1'$ in action)　　　$S = \frac{M}{22.5}$

本例ノ力率Mハ車軸圖表中第七車軸ガ(2)點ニアルトキニ等シ、

$M = \frac{\begin{array}{r}11877.725\\+23.68\end{array}}{12114.525} . \frac{2}{5} \times 12114.525 = 4845.500^{\text{lbs}}$

$\left\{\begin{array}{l}\text{本例ノ力率ガ下臥材}b_2\text{ニ用ヒタルト}\\\text{相異セルハ扣材ガ働ヲ受クル故ナリ}\\\text{ト知ルベシ}\end{array}\right\}$

Max. comp in $A_2 = \frac{4845.500}{22.5} = 215.400^{\#}$

Dead tension$= 31.200.^{\#}$

$\therefore$ Actual stress$= 215.400 - 31.200 = 184.200^{\text{comp.}}$

○第拾壹圖對角柱材(g_4)ノ應力算法

A actual loads－o－5, as for moment at 4.

Max. moment — — load 1′4 at (4)

$M = 2,517,500.^{\text{ft lbs}}$　　　Moment－a load unity at $(4) = \frac{4}{5} \times 21 = \frac{84}{5}.$

$V.C.S : M = \frac{4}{5} : \frac{84}{5}.$　　　$V.C.S = \frac{M}{21}.$

Max. tension in $g_1 = \left(A_2 \times 1.750 + \frac{M}{21}\right)\frac{1}{h} = \left(33.1 \times 1750 + \frac{2,517.500}{21}\right)\frac{34.5}{27.5}$

$= \left(117.980 + 57.920\right)\frac{34.5}{27.5} = 221.500.$#

○第拾壹圖　下臥材(b_5)ノ應力算法

本材ハ單ニ壓力ノミヲ受ケ張力ヲ受ケサルモノトス

For comp. load 6−15.　Max. with load 11 at (9)

H.C.S : M : 2,1 : 42　　H.C.S $= \frac{2.1}{42}$M.

Max. comp. in $(b_5) = \frac{12,825.500 \times 2.1 \times 21.15}{42 \times 21} = 646.400$#.

○第拾壹圖上臥材(A_6)ノ應力算法

本材ノ應力ハ前記下臥材(b_5)ノ水平分力ニ等シ

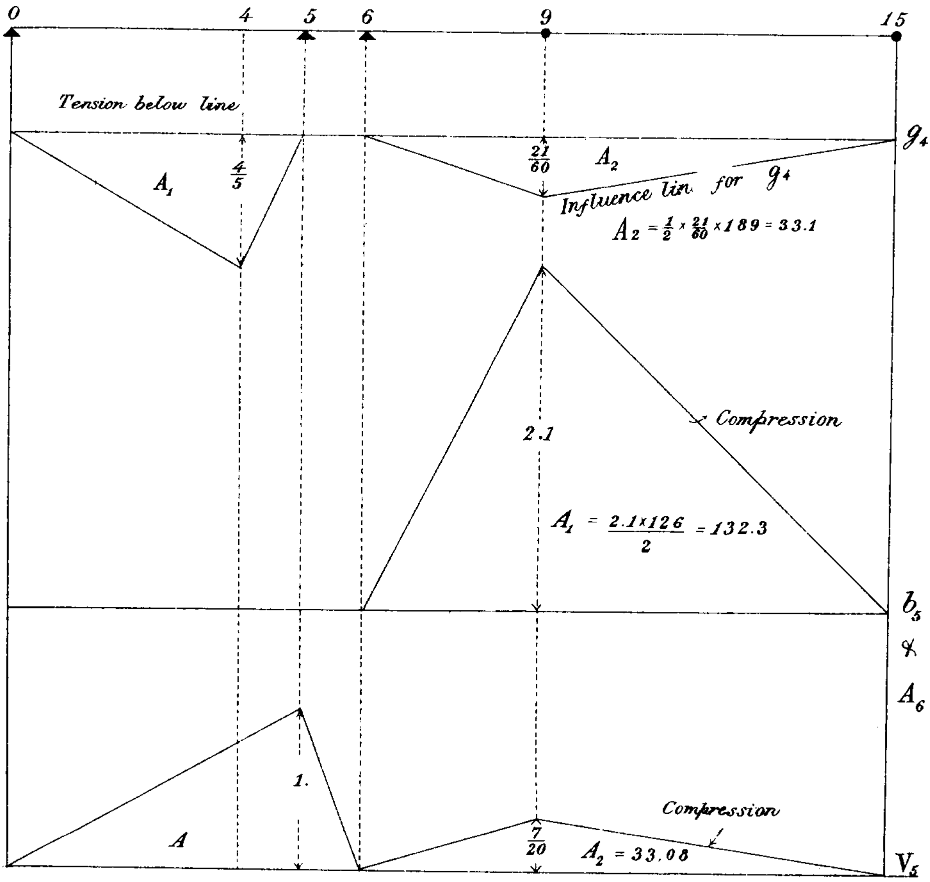
Max. tension in $(A_6) = \frac{12,825,500 \times 21}{42} = 641.300$#

○第拾壹圖垂直材(V_5)ノ應力算法

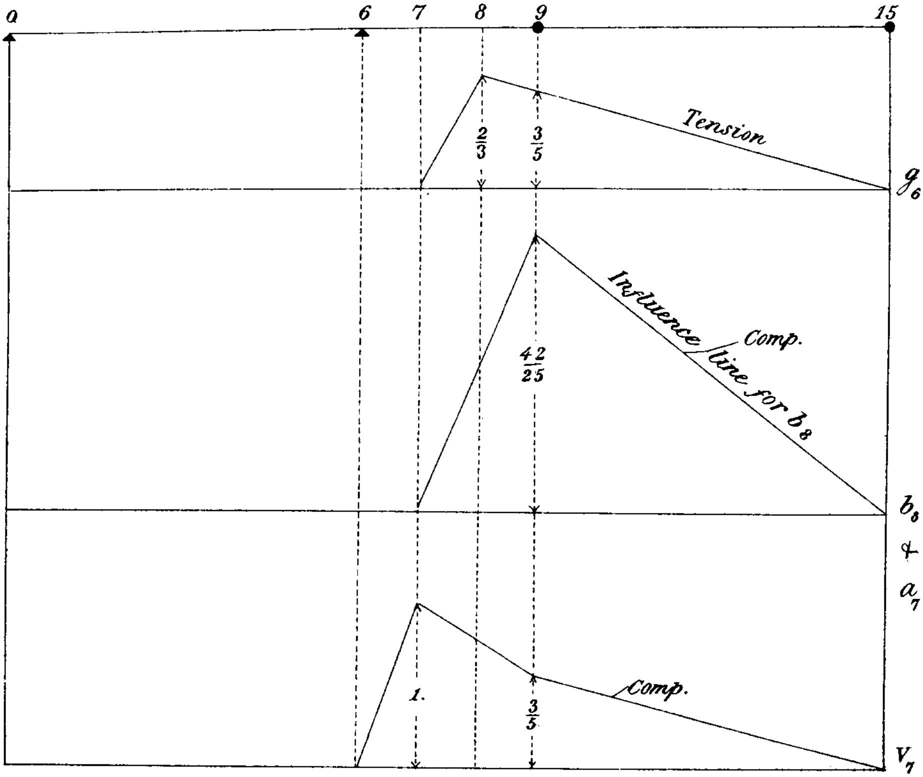
Actual load 0−6 as for moment

M = 3,589,400 ft lbs.

Moment−a load unity at $(5) = \frac{105}{6}$.


0
4
5
6
9
15
Tension below line
g_4
A_1
$\frac{4}{5}$
$\frac{21}{60}$
A_2
Influence line for g_4
$A_2 = \frac{1}{2} \times \frac{21}{60} \times 189 = 33.1$
Compression
2.1
$A_1 = \frac{2.1 \times 126}{2} = 132.3$
b_5 & A_6
1.
Compression
A
$\frac{7}{20}$
$A_2 = 33.08$
V_5

第拾壹圖


0
6
7
8
9
15
Tension
2/3
3/5
g_6
Influence line for b_8
Comp.
42/25
b_8 & a_7
1.
3/5
Comp.
V_7
第拾貳圖

$S:M=1:\frac{105}{6}$.　　　$S=\frac{2}{35}M$.

Max. comp. in $V_5=\frac{3.589.400\times 2}{35}+A_2\times 1.750.$

$=205.100+33,08\times 1.750=26.300.$*

○第拾貳圖對角柱材(g_6)ノ應力算法

$46.33\times\frac{1}{315}+158,02\times\frac{1}{210}+89.25\times\frac{1}{210}<43.75\times\frac{2}{63}.$

∴ Max with load (3) at (8).

Uniform load $\frac{51}{126}\times\frac{3}{5}\times\frac{51}{2}=6.19=A$　　　$6.19\times 1.750=10.830.$

$\frac{8}{21}\times 8\ 75=3.33$

$\frac{16}{21}\times 17.5=13.36$

$\frac{17.50}{34.19}=$ load on 8.

$1.435.00$ ……………………(See axel load. no 6)

$90.08\times 128=11530.20$

$12.965.20$

$$\begin{array}{r} 14302.00 \\ 248,1 \times 5 = 12653.10 \\ \hline 26955.10 \\ 12963.20 \\ \hline 13,989.90 \end{array}$$ ....(See axel load no 18–19)

$\dfrac{13.9899}{126} = 111.03$ load on 9 i.e. (Reaction-loads. between 9 and 15)

$$\frac{34.19 \times 4}{5} = 217.35$$

$$\frac{111.03 \times 63}{105} = 66.62$$

$$\frac{35 \times 97.5}{105} = 32.50$$

$$\frac{11.325 \times 65}{105.} = \frac{7.01}{133.48}$$

$$\begin{array}{r} 133.480 \\ 10.830 \\ \hline 144.310 \end{array}$$

Max. ten. $= 144.310 \times \dfrac{29}{20} = 209.200^{\#}$

○下臥材(b_2)ノ應力算法

Actual loads 7–15 as for moment at 9.

Max. with load 6 at 9

$$\begin{array}{r} 14.302,00 \\ 248,10 \times 49 = 12.156\,90 \\ 2\,100.9 \\ \hline 28.559.80 \end{array}$$ ....(See axel load no 18–19)

$M \quad \frac{28,559.8}{4} = 7,139.950$

$H.C.S. : S = M : \frac{42}{25} : \frac{126}{4}$

$H.C.S. = \frac{4M}{75}.$

○第拾貳圖上臥材(A_7)ノ應力算法

Max. comp. in $b_8 = \frac{4 \times 7139.900 \times 21.59}{75 \times 21} = 391.500.$#

本材ノ應力ハ下臥材(b_8)ノ水平分力ニ等シ

$\frac{4 \times 7139.950}{75} = 380,800.$ tension

○第十二圖垂直材(V_7)ノ應力算法

Load 6–15 actual loads

Max. with load 4 at 7

Load on (7.)	Load on (8)
$\frac{3}{21} \times 8.75 = 1.25$	$\frac{5}{21} \times 17.5 = 4.18$
$\frac{64}{21} \times 17.5 = 53.3$	$\frac{62}{21} \times 11.325 = 335$

$$\frac{9}{21}\times 11.325=\frac{4.85}{59.40}$$

Load on (9.)

$$\frac{13}{21}\times 11.325= 7.13$$

$$\frac{17}{21}\times 8.75= 7.08$$

$$\frac{38}{21}\times 17.5 = \frac{31.70}{45.91}$$

Load on (11)

$$\frac{56}{21}\times 11.325=30.3$$

$$\frac{7\times 14\times 1.75}{21} = \frac{8.15}{38.45}$$

$$\frac{4}{21}\times 8.75=\frac{1.67}{39.35}$$

Load on (10)

$$\frac{46}{21}\times 17.5= 38.3$$

$$\frac{26}{21}\times 11.325=\frac{14.0}{523}$$

Load on (12)

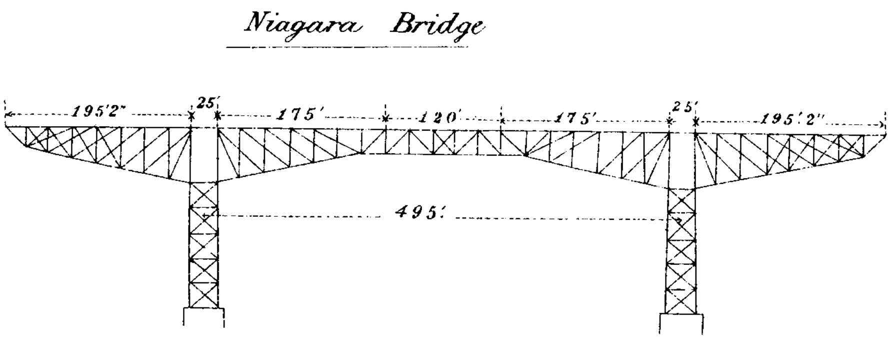
$$21\times 1.75=36.75$$

Max. compression in V_7 =

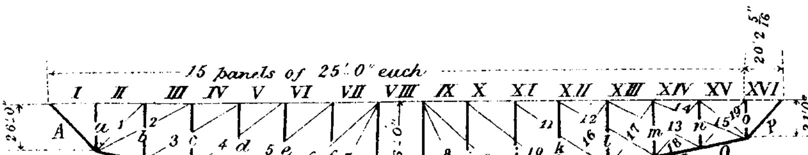
$$\left\{\begin{array}{rl} & 59.400 \\ \frac{4}{5}\times 39.35= & 31.500 \\ \frac{3}{5}\times 45.91= & 27.600 \\ \frac{1}{2}\times 52.3 = & 26.150 \\ \frac{2}{5}\times 38.45= & 15.350 \\ \frac{3}{5}\times 36.75= & \frac{22.,200}{182.200.\#} \end{array}\right\}$$

○第八圖ニ示ス肱木形橋諸材ノ應力ハ第拾圖ヨリ第拾貳圖ニ至ル三實例ニ於テ示シタル同

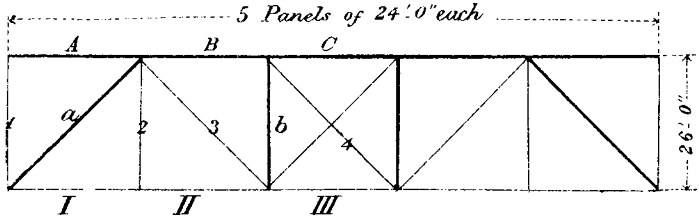
第十三圖 (a)


Niagara Bridge
195'2"
25'
175'
120'
175'
25'
195'2"
495'

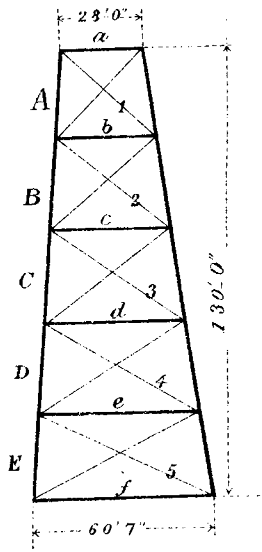
第十三圖 (b)


15 panels of 25'0" each
20'2 5/16"
I II III IV V VI VII VIII IX X XI XII XIII XIV XV XVI
26'0"
56'0"
21'0"
A B C D E F G H I J K L M N O P
a b c d e f g h i j k l m o

第十三圖 (C)


5 Panels of 24'.0" each
A
B
C
1
a
2
3
b
4
26'.0"
I
II
III

第十四圖


23'0"
a
A
1
b
B
2
c
C
3
d
130'0"
D
4
e
E
5
f
60'7"

一ノ方法ニ依テ格間點(0)ヨリ(9)ニ至ル九格間內諸材ノ應力ヲ算知シ得ベク又(9)ヨリ(15)ニ至ル六格間內ノ諸材ハ肱木形橋ノ範圍ヲ離レ普通桁構法線法ニ依テ算知シ得ベシ

○本論ノ法線法ハ結構中ノ壹材每ニ法線ヲ畫キ其材ノ張力壓力ノ區別ヲ圖上ニ明載シ又應力計算上尤モ必要ナル荷重ノ配置ヲ精確ニ寫出スル等ノ利益アリテ此ノ法ニ依ルトキハ重復、誤算等ノ患ナク普通ノ算法ニ優ル數等ナリトス、特ニ之ヲ肱木形橋ニ應用スルトキハ一層其利便ヲ感スベシ

○第拾參圖ハ「みしがんせんとらる」鐵道(Michigan Central Railroad)線路中「ないやがら」ニ架スル肱木形橋ノ略圖、第十四圖ハ同橋梁附屬ノ鐵塔ヲ示ス、第十三、十四、二圖ニ附屬スル圖表ハ共ニ結構ノ應力並ニ構造ノ大躰ヲ示ス是等ハ本論ニ關係ナキモノナリト雖モ其構造本論ニ於ケル肱木形橋ニ類似セルヲ以テ參考ノ爲メ特ニ編中ニ加フ

○本論ニ就テハ編中許多ノ誤解、違算アランコトヲ恐ル、幸ニ敎示ヲ賜ハランコトヲ乞フ(完)

Strain Sheets.

Cantilever Bridge, Niagara Falls.

This bridge consists of two cantilever spans of 395'.$2\frac{5}{16}$" each and one ordinary Pratt truss span of 120'-0" susqended from the former, all double track, deck.

Cantilever Spans.

Each Cantilever span contains fifteen panels of 25'-0" each and one of 20'$2\frac{5}{16}$", the latter being at the end of the shore arm.

Height, center to center of chords over pier, 56′–0″; at first vertical post, shore end, 21′–0″; at first vertical post, river end, 26′–0″.

The span rests upon an iron pier underneath posts G and H (see figure 13), the end of the shore arm is anchored to the masonry while the Pratt truss span is suspended from the end of the river arm.

Loads.

The total dead load of each cantilever span is about 1,930,000 pounds, which, for the computation of strains, was assumed to be concentrated at the panel points as nearly as possible in accordance with the actual condition in the bridge.

The moving load assumed on each track consisted of two 10 wheel locomotives with 72,000 pounds on drivers, with a wheel base of 12 feet, followed by a uniform load of 2,000 pounds per foot.

Fixed Span.

Length for computation, 120′–0″. Height for same, 26′–0″. Five panels of 24′–0″ each.

Trusses, 28′–0″ apart ceutre to centre.

Tracks, 13′–0″ apart centre to centre.

Floor beams, web plate $48 \times \frac{3}{8}''$; each flange composed of two angles $6'' \times 6'' \times \frac{1}{2}''$, one plate $14 \times \frac{5}{8}'' \times 22'$ and one $14 \times \frac{5}{8}'' \times 15'$.

Stringers two under each track, spaced 6′–6″ apart centre to centre; web plate, $30 \times \frac{3}{8}''$; each flange

第二表（I）

Member	Strain	Compsition.	Sectional area	Strain per sq. inch
a	100.100	Two 12″ channels, 105-lbs Per yd	21.00	4.770
b	234.000	,, 15″ ,, 155″ ,, ,,	31.00	7.550
c	225.500	Two 15″ channels 155″ ,, ,,	31.00	7.274
d	293.000	two 15″ channels 150* per yd. 2 plates $12\times{}^{1}/_{2}''$	42.00	6.976
e	297.000	Two 15″ channels 150* per yd 2 plates $12\times{}^{5}/_{8}''$	45.00	6.600
f	352.000	,, ,, ,, 205* ″ ″ ″ $12\times{}^{5}/_{8}''$	56.00	6.286
g	683.000	4 angles $4\times4\times{}^{5}/_{8}''$, 2 plates $20\times{}^{3}/_{4}''$ (steel)	50.00	13.660
h	658.000	4 ″ $4\times4\times{}^{5}/_{8}''$. 2 plates $20\times{}^{3}/_{4}''$ (″)	50.00	13.160
i	331.000	2–15″ channels. 150* per yd 2 plates $12\times{}^{5}/_{8}''$	51.00	6.490
j	300.000	″ ″ ″ 150* ″ ″ 2 ″ $12\times{}^{5}/_{8}''$	45.00	6.667
k	270.000	″ ″ ″ 205 ″ ″	41.00	6.585
l	242.000	2–15″ channels 150* per yd	30.00	8.033
m	184.000	″ 15″ ″ 150* ″ ″	30.00	6.067
n	102.000	″ 15″ ″ 140* ″ ″	28.00	3.643
o	205.900	″ 15″ ″ 150* ″ ″	30.00	6.863
1	191.000	2 bars $7\times1{}^{1}/_{2}''$	21.00	9.095
2	297.000	2 ″ $6\times1{}^{1}/_{4}''$. 2 bars $6\times1{}^{1}/_{8}''$	28.50	9.790
3	363.000	4 ″ $7\times1{}^{1}/_{4}''$,	35.00	10.371
4	330.000	4 ″ $7\times1{}^{1}/_{4}''$,	35.00	9.429
5	403.000	2 ″ $7\times1{}^{1}/_{2}''$, 2 bars $7\times1{}^{3}/_{8}''$	40.25	10.012
6	397.000	2 ″ $7\times1{}^{1}/_{2}''$, 2 ″ $7\times1{}^{3}/_{8}''$	40.25	9.863
7	394.000	2 ″ $7\times1{}^{1}/_{2}''$, 2 ″ $7\times1{}^{3}/_{8}''$	40.25	9.784
8	374.000	2 ″ $7\times1{}^{1}/_{4}''$, 2 ″ $7\times1{}^{3}/_{8}''$	36.75	10.177
9	397.000	4 ″ $7\times1{}^{3}/_{8}''$	38.50	10.312
10	379.000	2 ″ $7\times1{}^{1}/_{4}''$, 2 bars $7\times1{}^{3}/_{8}''$	36.75	10.313
11	356.000	2 ″ $6\times1{}^{1}/_{4}''$, 2 ″ $6\times1{}^{3}/_{8}''$	31.50	11.302
12	295.000	2 ″ $6\times1{}^{1}/_{4}''$, 2 ″ $6\times1{}^{3}/_{8}''$	31.50	9.365
13	282.000	2 ″ $6\times1{}^{1}/_{4}''$, 2 ″ $6\times1{}^{1}/_{8}''$	28.50	9.895
14	313.000	2 ″ $6\times1{}^{1}/_{4}''$, 2 ″ $6\times1{}^{1}/_{8}''$	28.50	10.[illegible]83
15	165.000	2 ″ $7\times1{}^{3}/_{8}''$	19.25	8.572
16	0	2 ″ $1{}^{1}/_{4}''$ square	3.13	0
17	0	2 ″ $1{}^{1}/_{2}''$ ″	4.50	0
18	0	2 ″ 2″ ″	8.00	0
19	79.000	2 ″ $6\times1{}^{1}/_{4}$ ″	15.00	5.267
A	418.000	4 angle $4\times4\times{}^{5}/_{8}''$, 2 web plates $24\times{}^{5}/_{8}''$ (steel)	50.00	8.036
B	666.000	4 ″ $4\times4\times{}^{5}/_{8}''$, 2 ″ ″ $24\times{}^{5}/_{8}''$ (″)	50.00	13.320
C	966.000	4 ″ $4\times4\times{}^{5}/_{8}''$, 2 ″ ″ $24\times{}^{5}/_{8}''$, 2 web plates $16\times{}^{5}/_{8}''$ (steel)	70.00	13.800

Member	Strain	Composition	Sectional area	Strain per sq. inch
D	1.191.000	4 angles $4\times4\times{}^5/_8''$, 2 web plates $24\times{}^3/_4''$,………		
		2 web plates $16\times{}^5/_8''$, 4 bars $3\times{}^1/_2''$ (steel)………	82,00	14.524
E	1.461.000	4 angles $4\times4\times{}^5/_8''$. 2 web plates $24\times{}^3/_4''$,………		
		2web plates $16\times{}^5/_8''$. 1bottom plate $22{}^1/_2\times{}^1/_2''$(steel)	98.50	14,833
F	1.716.000	4 angles $4\times4\times{}^5/_8''$, 4 web plates $24\times{}^1/_2''$, 2 web		
		Plates $16\times{}^5/_8''$, 1 bottom plate $22{}^1/_2\times{}^1/_2''$ (steel)	110.50	51,530
G	1.874.000	4 angles $4\times4\times{}^5/_8''$, 4 web plates $24\times{}^1/_2''$, 2 web		
		Plates $16\times{}^5/_8''$, 1 bottom plate $22\frac{1}{2}\times{}^5/_8''$ (steel)	116.13	16,137
H	1,837.800	Same as panel G………	116,13	15.826
I	1.875.000	" " G………	116,13	16.146
J	1.766.000	" " G………	116.13	15.207
K	1.576.000	" " F………	110,50	14,263
L	1,377.000	" " E………	98.50	13.980
M	1.187.000	4 angles $4\times4\times{}^5/_8''$. 2 web plates $24\times{}^3/_4''$, 2		
		web plates $16\times{}^5/_8''$, 1 bottom plate $22\frac{1}{2}\times{}^3/_8''$ (steel)	92,87	12.781
N	969.000 c 60.000 t	Same as panel C………	70.00 gross 56.25 net	13.843 c 1.067 t
O	744.000 c	4 angles $4\times4\times{}^5/_8''$, 2 web plates $24\times{}^1/_2''$, 2 web	64.00 gross	11.625 c
	72.000 t	Plates $16\times{}^5/_8''$ (steel)………	51.75 net	1.392 t
P	457.000 c 249.000 t	Same as panel O………	64.00 gross 51.75 net	7.141 c 4.811 t
I	291.000	4 bars $6\times1{}^1/_4''$ ………	30.00	9.700
II	291.000	4 " $6\times1{}^1/_4''$ ………	30.00	9.700
III	399.500	4 " $8\times1{}^3/_4''$ ………	44.00	9.080
IV	648.000	2 " $8\times1{}^3/_8''$, 4 bars $8\times1{}^1/_2''$………	70.00	9.257
V	943.000	10 " $8\times1{}^3/_8''$ ………	110.00	8.573
VI	1.163.000	4 " $8\times1{}^3/_8''$, 2 bars $8\times1{}^1/_2''$, 4 bars $8\times1{}^3/_4''$.	124.00	9.371
VII	1.427.000	4 " $8\times1{}^3/_8''$, 2 bars $8\times1{}^1/_2''$, 6 bars $8\times1{}^3/_4''$	152.00	9.388
VIII	1.837.800	10 " $8\times1{}^3/_4''$, 4 " $8\times1{}^1/_2''$………	188.00	9.776
IX	1.540.000	10 " $8\times1{}^3/_4''$, 2 " $8\times1{}^1/_2''$, 4 angles $5\times3\frac{1}{2}''\times\frac{5}{8}''$,		
		2 web plates $18\times{}^3/_4''$………	164.00 net	9.590
X	1,345,000	2 bars $8\times1{}^3/_8''$, 8 bars $8\times1{}^1/_2''$, 2 bars $8\times1{}^5/_8''$.		
		4 angles $4\times3{}^1/_2\times{}^5/_8''$, 2 web plates $18\times{}^3/_4''$ ……	144.00 net	9.340
XI	1,159,000	2 bars $8\times1{}^3/_8''$, 6 bars $8\times1{}^1/_2''$, 2 bars $8\times1{}^3/_4''$.		
		4 angles $5\times3{}^1/_2\times{}^5/_8''$. 2 web plates $18\times{}^3/_4''$ ……	122.00 "	9.500
XII	86.000 c	2 bars $8\times1{}^3/_8''$. 6 bars $8\times1{}^1/_2''$, 4 angles $4\times3{}^1/_2''$	45.75 gross	1.880 c
	946.000 t	$\times{}^5/_8''$. 2 web plates $18\times{}^3/_4''$ ………	94,00 net	10.064 t
XIII	193.000 c	2 bars $8\times1{}^3/_8''$. 4 bars $8\times1{}^1/_2''$. 4 angles $5\times3\frac{1}{2}''$	49.25 gross	3.919 c
	727.000 t	$\times{}^5/_8''$, 2 web plates $18\times{}^3/_4''$………	70.00 net	10.386 t
XIV	265.000 c	4 bars $8\times1{}^3/_8''$, 4 angles $4\times3\frac{1}{2}\times{}^5/_8''$, 2 web	45.75 gross	5.792 c
	438.000 t	Plates $18\times{}^3/_4''$ ………	44.00 net	9.955 t
XV	270.000 c	2 bars $8\times1{}^5/_8''$, 4 angles $5\times3\frac{1}{2}\times{}^5/_8''$. 2 web	49.25 gross	5.482 c
	315.000 t	Plates $18\times{}^3/_4''$ ………	26.00 net'	12.116 t
XVI	173.000 c	2 bars $8\times1{}^3/_8''$, 4 angles $3{}^1/_2''\times3\frac{1}{2}\times{}^5/_8''$	44.50 gross	3.888 c
	315.000 t	2 web plates $18\times{}^3/_4''$………	22.00 net	14.319 t

第二表（2）

第三表

Member	Strain	Composition	Sectional area	Strain per sq. inch
a	271.000	two 15″ channels. 200*per yd, 2 web plates		
		$15\times{}^{5}/_{8}$″, 1 cover plate $24\times{}^{3}/_{8}$″	67.75	4.000
b	101.000	two 12″ channels, 150*per yd	30.00	3.367
1	209.400	4 bars $5\times1^{1}/_{4}$″	25 00	8.376
2	6.000	2 rods 1″ diameter	1.57	3.822
3	150.000	4 bars $4\times1\ /_{8}$″	18.00	8.333
4	52.000	2 ″ $4\times1^{1}/_{8}$″........................	9.00	5.778
A	19.300	two 12″ channels. 70* per yd	14.00	1.378
B	264.000	″ 15″ ″ 200* per yd 1 top plate $24\times{}^{3}/_{8}$″	49.00	5.388
C	264.000	Same as panel B	49.00	5.388
I	181.900	Two 12 channels, 150* per yd	20.00 net	9.095
II	181.900	″ 12 ″ 150* ″ ″	20.00 ″	9.095
III	264.000	2 bars 5″ × 1″, 4 bars $5\times1^{1}/_{8}$″	32.50 ″	8.123

第四表

Member	Strain	Composition.	Sectional area	Strain per sq. inch
a	247·100	Two 12″ channels, 70* per yd	14.00	17.650
b	181.500	″ 10″ ″ 50″ ″ ″	10,00	18.150
c	96.100	″ 10″ ″ 50″ ″ ″	10.00	9.610
d	87.200	″ 10″ ″ 50″ ″ ″	10.00	8.720
e	81 400	″ 10″ ″ 50″ ″ ″	10.00	8.140
f	74.600 c. 180.200 t.	″ 10″ ″ 50″ ″ ″	10.00 gross 8.00 net	7.460 c. 22.525 t.
1	152,000	2 bars 2″ square	8.00	19·000
2	119.000	2 ″ $1^3/_4$″ ″	6.12	19.444
3	101.000	2 ″ $1^5/_8$″ ″	5.28	19.130
4	90,000	2 ″ 19-16″ ″	4.86	18.520
5	82.600	2 ″ 17-16″ ″	4.14	19 952
A	1.310.900	4 angles $4\times4\times{}^5/_8$″. 2 web plates $26\times{}^1/_2$″.		
		2 web plates $18\times{}^5/_8$″, 1 cover plate $30\times{}^1/_2$″,		
		2 bars $5\times{}^3/_4$″ (steel)	91.00	14.405
B	1.375.900	4 angles $4\times4\times{}^5/_8$″, 2 web plates $26\times{}^5/_8$″,		
		2 web plates $18\times{}^5/_8$″. 1 cover plate $30\times{}^1/_2$″,	97.50	14.112
		2 bars $5\times{}^3/_4$″ (steel)		
C	1·430.200	4 angles $4\times4\times{}^5/_8$″, 2 web plates $26\times11-16$″,		
		2 web plates $18\times{}^5/_8$″, 1 cover plate $30\times{}^1/_2$″,		
		2 bars $5\times{}^3/_4$″ (steel)	100.75	14.196
D	1.474.700	4 angles $4\times4\times{}^5/_8$″, 2 web plates $26\times{}^3/_4$″,		
		2 web plates $18\times{}^5/_8$″, 1 cover plate $30\times\frac{1}{2}$″,		
		2 bars $5\times{}^3/_4$ (steel)	104.00	14.180
E	1.524.300	same as D	104.00	14.657

composed of two angles $5' \times 3\frac{1}{2}'' \times \frac{5}{8}''$.

Loads

Dead load=3,000 pounds per foot. Moving load=same as for cantilever spans.

Towers under Cantilever spans

Each of the two towers upon which the Cantilever spans rest is composed of two bents placed $25'-0''$ apart and braced together.

Height of bent, $130'-0,''$ divided in five stories of nearly equal height. Width, centre to centre of legs at top, $28'-0''$; at base, $60'-7''$.

For the complete determination of the stresses in a double track bent computations are necessary under three different conditions of loading the maximum, wind pressure acting at all times. First, a maximum load on both tracks, which would consist of trains of loaded coal cars, and produces the maximum compression in the legs and top horizontal strut and maximum tension in bottom horizontal brace. Second, a train of empty box cars on the leeward track only producing the minimum compression or maximum tension, if any, in the windward leg. Third a train of loaded box cars, which will weigh about 2,000 lbs. per foot, on the windward track only. Producing maximum tension in all diagonal rods and maximum compression in all horizontal struts except the top one. The preceding computations were made on these assumptions with a wind pressure of 30 lbs per square foot.