Theses
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Live load stresses.

For my live load I took the two passenger engines ~~two~~ given in Merriman to be followed by a train load of 3000 lbs to a foot.

Req. S_2.

$P_1 = \frac{W}{m}$ $16 > \frac{56}{6}$ ∴ took 2 pilots on.

$R = V = \frac{476 + 56 \times 8}{150} = 6.15$

$V \sec\theta = -6.15 \times 1.35 = -8.3$ tons for 2 trusses =

$S_{2} = -8300$ lbs. per truss = min. live load stress in S_2.

Having the train come on from the opposite direction we have

$36 > \frac{176 + 15 \times 1.5}{6}$

∴ put first driver on the panel.

~~R = 1747 + 160 × 4 / 150 = 53.1~~

~~V = 92.5 tons~~ $R = V = \frac{9660 + 176 \times 15 + \frac{15 \times 15 \times 1.5}{2}}{150} = 84.2^{T.}$

$V \sec\theta = 84.2 \times 1.35 = 113.5$ Tons.

= 113500 lbs per truss = max live load stress.

Consider S_3, S_3'

Train from rt.

$$P_1 = \frac{W}{m} \cdot 16 = \frac{104}{6} \text{ nearly}$$

∴ put 2 pilots on.

$$V = -\frac{3536 + 104 \times 3}{150} = -25.6$$

$$V \sec\theta = -25.6 \times 1.35 = -34.4 \text{ tons}$$
$$= -34400 \text{ per truss}$$

Train from left.

$$P_1 = \frac{W}{m} \qquad \cancel{36 > \frac{176 + 15 \times 1.5}{6}}$$

~~∴ put first driver on.~~

$$\cancel{V = 9660 + 176 \times 1.5 + 13}$$

$$36 > \frac{160}{6}$$

∴ put 2 pilots on.

$$V = \frac{7412 + 160 \times 4}{150} = 53.7$$

$$V \sec\theta = 53.7 \times 1.35 = 72.5 \text{ tons}$$
$$= 72500 \text{ per panel}$$

Considering T_1.

Stress in T_1 = maximum wt resting at its base.

$$\text{Max. value} = 20 + \left(\frac{2+7}{25}\right)20 + \left(\frac{17}{25}\right)20 + \frac{10+15+4}{25} \times 8$$

$$= 20 + 7\tfrac{1}{5} + 13\tfrac{3}{5} + 9 = 50 \text{ tons for 2 trusses}$$

or 50000 pounds per truss.

Live load stress in T_2 = V from live load = 72500 lbs per truss.

Live stress for T_3 is similarly = 25600 lbs. per truss.

Train Loads in Chords.

Panel 1.

L_1.

$$P = \frac{n}{m} W = \frac{1}{6} W.$$

Putting second driver on the vertical we have $36 \lessgtr \frac{176 + 40 \times 1.5}{6} = 40-$

This will do as we may consider as much of the wt. of the 2^d driver on the panel as we wish.

$$R = \frac{1}{150}\left(9660 + 40 \times 176 + \frac{\overline{1.5+40}^2}{2}\right) = 119.3 \text{ tons}$$

for two trusses = 119300 lbs per truss.

$R \times 25 = L_1 \times 28$

$L_1 = 106\,200$ lbs per truss

L_2 = the same = 106200 lbs.

Consider lower 3.

Put 1st driver over middle post.

$36 \not\gtrless \frac{4}{6}(152)$ ∴ this will not do.

Put 2d tender wheel

$50 <=> \frac{4}{6}(176 + 7.5)$ ∴ this will not do.

Put 1st pilot of second engine

$88 <=> \frac{4}{6}(176 + 36)$ this won't do

Put 2d pilot of 2d engine.

$104 <=> \frac{4}{6}(176 + 60)$. This will do as we can; as we can, by counting part of the wt of 1st driver on 2d engine, consider this equation to be true.

$$R = \frac{1}{150}\left(9660 + 176 \times 40 + \frac{1.5 \times 40 \times 40}{2}\right) = 119.3$$

= 119300 lbs per truss.

$$\frac{119\,300 \times 50^{\text{lbs ft}} - \frac{1008}{2}^{\text{tons ft.}} - \frac{64 \times 4}{2}^{\text{tons ft.}}}{28} = L_3 = 172\,300 \text{ lbs.}$$

For U_3 take the same loading

$$\frac{75.\times 119300 - 3536000}{28} = 194000 \text{ lbs per}$$

truss. For U_1 take loading for web members.

$P = \frac{W}{m} = \frac{W}{6}$ ∴ put 2d driver on panel pt.

$$R = \frac{9660 + 176 \times 40 + \frac{1}{2} 1.5 \times \overline{40}^2}{150} = 119$$

$V \sec \theta = 119 \times 1.35 = 161.2$ tons

$= 161,200$ lbs per truss.

Wind stresses.

Wind pressure = 30 lbs per sq. ft. which gives 130 lbs per lin. ft for top bracing and 230 lbs per ft for lower bracing. Pressure on train = 10 × 30 = 300 lbs. per linear ft.

∴ 3250 = panel load on upper chord
  5750 = static " " lower "
  7500 = train " " " " "

These give stress in upper bracing as follows.

$CC' = 3250$
$DD' = 1625$
$BC' = 9043$
$CD' = 3014.$

Lower bracing

~~aa' 13200~~

$ab' = 61400$
$bc' = 38300$
$cd' = 17900.$

If the wind were from the opposite side the strut stresses would be the same, but stresses in these diagonals would be 0 and the stresses in B'C, C'D, ~~a~~ a'b etc would the same as those in BC', CD', ab' etc when the wind is represented as above,

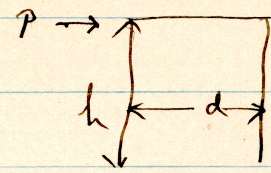
Design of portal strut.

Stress in portal $= \frac{Ph}{4}$ B p 239.

$P = 2\tfrac{1}{2} \times 3250 = 8125$

$h = 450.5''$

$d = 192$



Allowing 20' for train ht. the portal must not be over 30" across. Assume 29" = dist. between c. of g. of flanges.

$\frac{Ph}{4} \times \frac{1}{29} = \frac{8125 \times 450.5}{4} \times \frac{1}{29} = 31600$ = stress in each flange.

To this must added 2000 lbs. of direct compression.

∴ Total stress = 32 600 per flange.

Assume 2 Ls 3" x 3" A 34, area 6.72 □".

This makes an ample area.

I connected the flanges with a solid plate 3/8" x 30".

Design of tension Members.

Bb = 63000 lbs stress
Taking stress at 10000 lbs per sq. in. the area should be 6.3 sq. in.
I used two eye bars $6\frac{1}{2}'' \times \frac{1}{2}''$ area $6\frac{1}{2}\square''$.

Design of ab. = L1
Tension = 157600 lbs.
Area = $15.76\square''$
Used 2 bars $7'' \times 1\frac{1}{8}$ area $15.8''$
Used same for bc. = L2

Design of cd
Tension = 254300
area = $25.43\square''$
Used 2 bars $7\frac{1}{2}'' \times 1\frac{3}{4}$.
area = $26.25\square''$ = L3

In the above cases I neglected to put that a the allowable stress was found as follows.

$$a = 7500(1 + r) \quad \text{where } r = \frac{\text{min stress}}{\text{max stress}}$$

Design of Bc = S2

$$a = 7500\left(1 + \frac{38}{148}\right) = 9500$$

Stress = $1\cancel{48}000$, area = 15.6
Used 2 bars $5\frac{1}{2}'' \times 1\frac{1}{2}''$

$S_3 = Cd$

$a = 7500\left(1+\frac{0}{26}\right) = 7500$

Stress = 26300

$\text{area} = \frac{26300}{7500} = 3.5\ \square''$

Used two bars $5\frac{1}{2}'' \times \frac{3}{8}''$

$S_3'' = Dc$

$a = 7500(1+0) = 7500$

$\text{area} = \frac{19500}{7500} = 3\ \square''$ ~~Used 1 3/4~~

Used a bar $1\frac{3}{4}''$ square.

Design of Lateral Bracing.

ab' = Lat. L₁ = 61400

allowable stress 15000

Area = $4.1\ \square''$

Used two $1\frac{7}{16}''$ sq. rds. = $4.12\ \square''$ area

bc' stress = 38300 = Lat L_2

area = $2.55\ \square''$

Used two $1\frac{1}{8}$ sq rds. = $2.53\ \square''$ area

cd' = 17900 lbs stress = Lat L_3

Used one $1\frac{1}{8}''$ sq rd, area 1.27 sq in

$BC' =$ Lat U_{2S} = stress 9430

Used one 1" sq rd.

$CD' =$ Lat U_{3S} = stress 3014

Used same as BC'

$CC' =$ Lat U_{2T} = stress 3250

Used 4 Ls 2½" x 2½". I just took this size as the smallest which could be rivetted well. It exceeds the required area.

$DD' =$ Lat U_{3T} = stress 1625.

Used same.

Design of Posts

$l = 28' \times 12 = 336''$

Try two 10" channels C_3 light. & put them 7" apart.

$r = 3.85$

$P = 7000 - \frac{40 \times 28 \times 12}{3.85} = 3500$ from the

formula $P = 7000 - \frac{40\,l}{r}$ (Cooper)

$T_2 = \begin{matrix} \text{live } 72000 \\ \text{dead } 21000 \end{matrix}$

Cooper considers 2 lbs dead = 1 lb live.

∴ dead stress 21000 = 10500 live

T_2 ∴ = 82500 live load

If my supposition be correct the area of the two bars should be

$82500 \div 3500 = 23.6$ sq. in.

Their area really is 9.8 sq. in. so they are too small.

Try light 12" channels = C_2 light.

$P = 7000 - \frac{40 \times 28 \times 12}{4.63} = 4100$

Area should be $82500 \div 4100 = 20$ sq in

It is 11.8 which makes it too small.

Try two heavy 12" channels

$$P = 7000 - \frac{40 \times 28 \times 12}{4.03} = 3670$$

Area should be $\frac{82500}{3670} = 22.4$

The area is 26 sq. in. which is sufficient.

∴ Made the post of 2 heavy 12" channels, C_2 heavy, and latticed 9' apart back to back, as in fig. H. B p 252.

Design of middle post.

dead load = 10000 ⇌ 5000 live load
live load = 25600
Total ⇌ 30600 live load
Try light 10" channels
P = 3000 as above.

If this is correct the area of the channels should be $\frac{30600}{3500} = 9$ sq. in.
The real area = 9.8 sq. in., which is sufficient.

Design of Upper Chord.

The top chord must be wide enough to allow the entrance of all the members which meet allong its joints. Consider the top of T_2;

Here the width of $T_2 = 9'' + 2 \times 3.47$	= 16
Two bars $1\frac{3}{4}$ wide	= 2.5
Allowance	= 1.5
Inside width	= 20"

Therefore I made my top plate 30" and ½" thick. I connected the top plate with the sides by 3"x 3" angles. The reason for making the top at least ½" thick is that the unsupported length must not exceed 40 times the thickness.

Area of top plate =

Top = 1/2 x 30 = 15

2 angles = 2 x 3" = 6

21 sq. in.

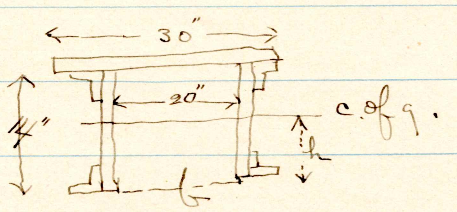
$P = 8000 - 30\frac{l}{r}$ = Cooper's live load formula for columns.

Stress on U_1 = dead 77.5 = live 39
live 161 = " 161
= live 200

$l = 12\sqrt{\overline{28}^2 + \overline{25}^2} = 37.4' \times 12$

As a trial take the following:

Top plate $30'' \times \frac{1}{2}''$	=	15 sq in
2 Ls $3'' \times 3''$	=	6 "
2 side plates $14'' \times \frac{1}{2}$	=	14 "
2 Ls $3'' \times 4''$	=	6"
		41 sq. in.



Req. the c. of g.

Take moments about b.

$30 \times \frac{1}{2} \times 14 + 14 \times \frac{1}{2} \times 2 \times 7 + 2 \times 3 \times 13\frac{1}{2} + 6 \times \frac{1}{2} = 41h$

$\therefore h = \frac{210 + 98 + 81 + 3}{41} = 9.5$

Req. value of I for the beam.

I_g for bottom Ls $= 2.74$

distance of c. of g. from back of L $= .9''$

I_g for top Ls $= 2.62$

dist. of c. of g. from back $= .86''$

I_g for top $= \frac{30 \times (\frac{1}{2})^3}{12} = \frac{5}{16}$

I_g for side plates $= \frac{\frac{1}{2} \times 14 \times 14 \times 14}{12} = 114\frac{1}{3}$

I_g for whole member $= 2(2.62) + 2(2.74) + \frac{5}{16} + 114\frac{1}{3} +$

$15 \times \overline{4.5}^2 + 14 \times \overline{2.5}^2 + 6 \times \overline{3.5}^2 + 6 \times \overline{8.5}^2 = 1024.36$

$Ar^2 = I$

$\therefore r = 5$

$P = 8000 - \frac{30 \times 37.4 \times 12}{5} = 5300$

" my section can hold

$5300 \times 41 = 217000$ lbs which is sufficient the stress being but 200000 lbs.

Design of ~~U~~ U_3

Try same section as U_1

here $l = 25 \times 12$

Allowable stress $= 8000 - \frac{30 \times 25 \times 12}{5} = 6200$

$6200 \times 41 = 264000$ which is too large, so I reduced the size of the side plates to 3/8", the live load stress being but $194000 + \frac{1}{2}\,92500 =$ 241000 live load stress.

Design of U_2

From the above we see that the section of U_1 will stand a live load stress of 264000.

The live load stress in U_2 = is but $171300 + \frac{1}{2}\,82500 = \cancel{242600}$

$= 212600$ lbs.

3/8" will be more than is required in the side plates, but that is the specified min thickness it will have to be taken.

Strains and Compositions

Member	Dead	Live	~~Max~~ Min.	~~Min~~ Max.	Composition
U_1	-77.5	-161.2	-77.5	-238.7	30" x ½" top plate } ~~two~~ 3"x4" Ls 14" x ½ side " } two 3"x3" Ls
U_2	-82.5	-171.3	-82.0	-254.3	30" x ½" top plate } two 3"x4" Ls 14" x 3/8" side " } two 3"x3" Ls
U_3	-92.5	-194.0	-92.5	-286.5	30" x ½" top plate } two 3"x4" Ls 14" x ½" side " } " 3"x3" Ls
L_1	51.4	106.2	51.4	157.6	two bars 7" x 1 1/8
L_2	51.4	106.2	51.4	157.6	"
L_3	82.0	172.3	82.0	254.3	two bars 7½ x 1¾
T_1	13.0	50.	13	63.0	one bar 6½ by 1"
T_2	-21.5	-72.	-21.5	-93.5	2 channels 12" wt per ft. 44 lbs put 9" apart.
T_3	-10.0	-25.6	-10	-35.6	2 channels 10" wt = 16.5 per ft - 7" apart
S_2	46.6	-8.3 113.5	38.3	160.1	2 bars 5½" x 1¾"
S_3	14.9	-34.4 72.5	0 ~~-19.5~~	97.4	2 bars 5½ x 3/8
S'_3			0 ~~+9.5~~	19.5	1 bar 1¾ sq.
Lat L_1		61.4			two 1½ sq. rods.
Lat L_2		38.3			" 1 1/8" " "
" L_3		17.9			one 1 1/8" " "
" U_{2S}		9.1			" 1" " "
" U_{2S}		3.0			" 1" " "
" U_{2T}		3.3			4 Ls 2½ x 2½
" U_{3T}		1.6			"