A DESIGN CHECK OF THE OCQUEOC DECK TRUSS HIGHWAY BRIDGE

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE

F. M. Drilling

1941

A Design Check Of The Ocqueoc Deck Truss Highway Bridge

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

F. M. Drilling

Candidate for the Degree of

Bachelor of Science

June 1941

INTRODUCTION

This thesis is a partial design check of the design of a deck truss highway bridge which was designed and built by the Michigan State Highway Department.

The bridge is located in Presque Isle county in northern Michigan. It spans the Ocqueoc River at station 8032 ⁄ 18 on highway US 23 and is found in Ocqueoc Township about sixteen miles northwest of Rogers City.

The span of the structure is one hundred feet, and its total width is about forty feet. It is equipped with sidewalks and standard railings on each side of the reinforced concrete roadway.

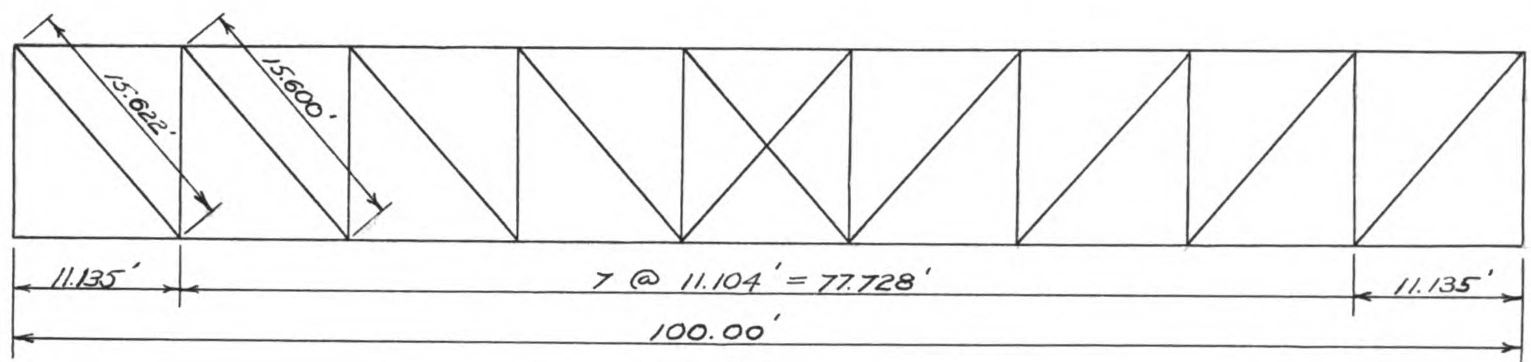
The specifications used in the design were the Michigan State Highway Department Specifications for Bridge Design of 1926, but the design was checked using the revised edition of these specifications which was adopted in 1936. The structure was designated as a Class A bridge and was therefore designed for standard H-15 loading.

It was the object of this thesis to give the author some practical experience in the checking of a design of this type and to better acquaint both the readers and the author with the methods and prodecure used in the design of a highway bridge. As it was impossible to cover the complete design in the time allotted, only the superstructure was checked, and the abuttments and some of the details were not touched upon.

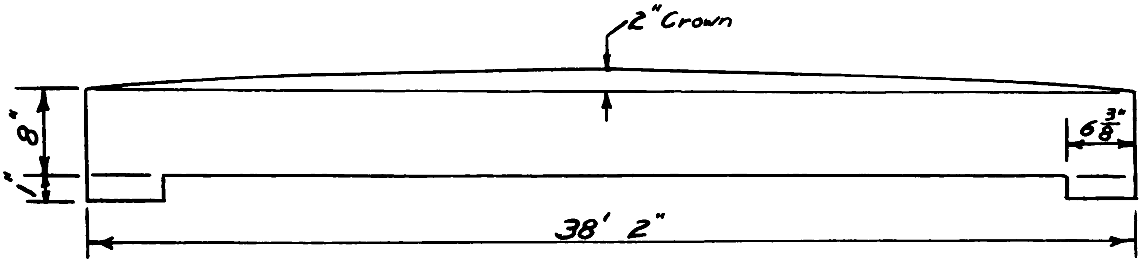
In conclusion, the author wishes to acknowledge his special appreciation to Mr. W. A. Voigt, Bridge Design Engineer of the Michigan State Highway Department, for his kind help in the securing of the plans and specifications necessary for this

project, and to the faculty members of the Civil Engineering Department of Michigan State College, for their cooperation and aid in problems involving the technical points of design.

ELEVATION OF TRUSS



WEIGHT OF SLAB



AREA OF SLAB

Rectangular Part

(8/12)(38 1/6)= 25.45 sq. ft.

Use the crown as triangular

(2/12)(38 1/6)(1/2)= 3.18 sq. ft.

Small rectangles at end under slab

$\frac{(1)(6\ 3/8)(2)}{(144)}$= 0.09 sq. ft.

Total= 28.72 sq. ft.

Use 28.8 sq. ft.

WEIGHT PER FOOT OF SLAB

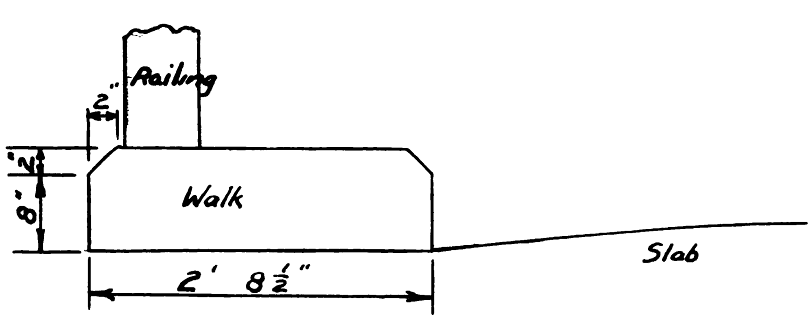
Use w= 150#/cu. ft.

(28.8)(1)(150)= 4320 # per foot

WEIGHT OF SLAB APPLIED TO EACH TRUSS

$\frac{4320}{2}$= 2,160# per foot

WEIGHT OF SIDEWALKS AND RAILINGS



AREA OF ONE WALK

(32.5)(10)-(2)(2)= 2.23 sq. ft.

WEIGHT PER FOOT OF ONE WALK

w= 150#/cu. ft.

(2.23)(1)(150)= 335 #/ft.

TOTAL WEIGHT OF WALKS

(2)(335)= 670#/ft.

WEIGHT PER FOOT APPLIED TO EACH TRUSS

335#/ft.

WEIGHT PER FOOT OF M.S.H.D. STANDARD RAILING APPLIED TO EACH TRUSS

w= 44#/ft.

TOTAL WEIGHT OF RAILING SECTIONS NOT INCLUDING CONCRETE POSTS

(2)(100)(44)= 8,800#

WEIGHT OF CONCRETE RAILING POSTS

WEIGHT OF ONE INTERMEDIATE POST

Size 3'4" x 1'4" x 1'2½"

(3 1/3)(1 1/3)(1 5/24)= 5.4 cu. ft.

(5.4)(150)= 810#

Applied at each intermediate panel point on each truss.

WEIGHT OF ONE END POST

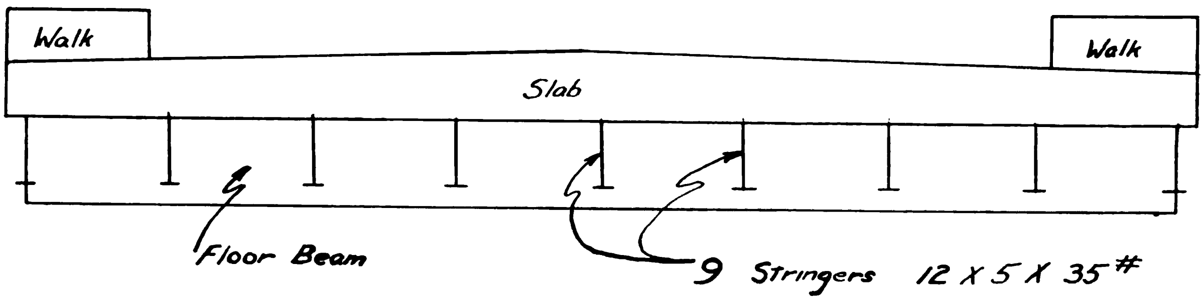
Size 3'8" x 2'2½" x 1'2½"

(3 2/3)(2 5/24)(1 5/24)= 9.8 cu. ft.

(9.8)(150)= 1470#

Applied at each end panel point on each truss

WEIGHT OF STRINGERS



TOTAL WEIGHT PER FOOT OF STRINGERS

(9)(35)= 315# per foot.

WEIGHT PER FOOT OF STRINGERS TRANSFERRED TO EACH TRUSS

$\frac{315}{2}$= 158# per foot.

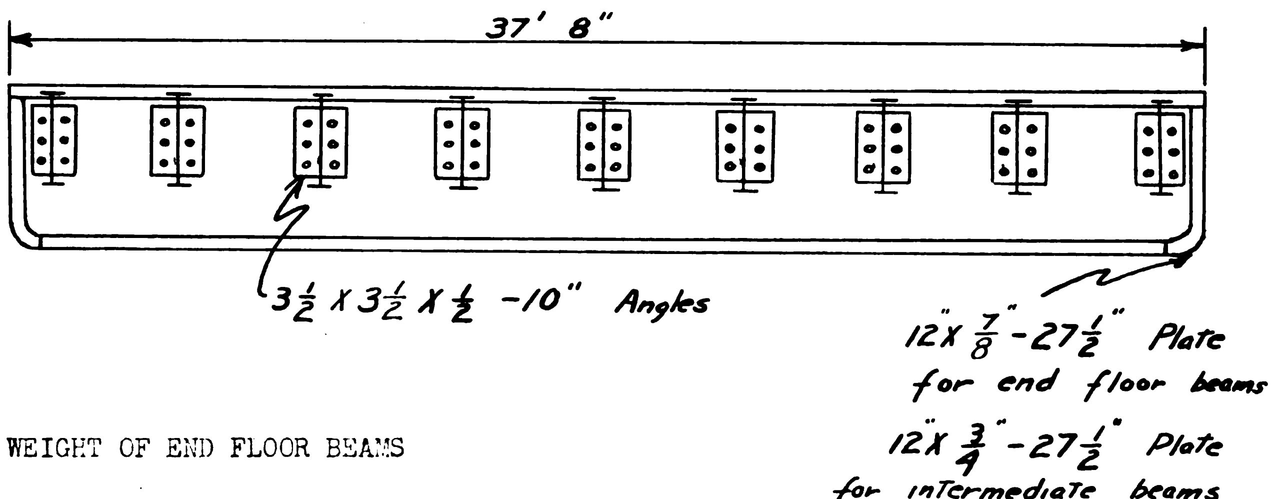
WEIGHT OF STRINGERS TRANSFERRED TO EACH TRUSS AT EACH INTERMEDIATE PANEL POINT

(11.104)(158)= 1,755#

WEIGHT APPLIED AT EACH END PANEL POINT

$\frac{1755}{2}$= 878#

WEIGHT OF FLOOR BEAMS



WEIGHT OF END FLOOR BEAMS

Size--24" x 12" x 110#

Weight of Concrete= $(2)(1)(38)(150)=$ 11,400#

Weight of RI Beam= $(110)(37.67)=$ 4,150#

Weight of 2 plates= $(2)(35.7)\frac{(27.5)}{(12)}=$ 164#

@ 35.7#/ft.

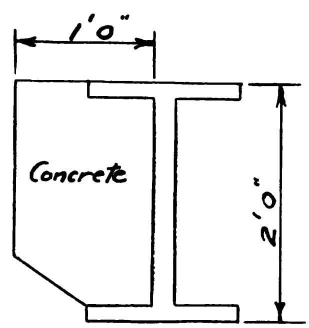
Weight of 18 Hitch Angles

$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$-10" @ 11.1#/ft.= $(18)(11.1)\frac{(10)}{(12)}=$ 167#

Weight of 162 Rivet Heads

@ 21#/100= $(21)(1.62)=$ 34#

Total= 15,915#



End Floor Beam

Weight of end floor beam applied to each truss

$\frac{15,915}{2} = 7,958\#$

WEIGHT OF INTERMEDIATE FLOOR BEAMS Size 24" x 12" x 100#

Weight of RI Beam= $(100)(37.67)=$ 3,767#

Weight of 2 plates= $(2)(30.6)\frac{(27.5)}{(12)}=$ 141#

Weight of 36 Hitch Angles

$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ - 10 @ 11.1#/ft.= $(36)(11.1)\frac{(10)}{(12)}=$ 333#

Weight of 216 Rivet Heads= @ 21#/100= $(21)(2.16)=$ 46#

WEIGHT OF INTERMEDIATE FLOOR BEAMS (con't)

Total= 4,287#

Weight of Intermediate Beam applied to each truss at each panel point.

$$\frac{4287}{2} = 2{,}144\#$$

WEIGHT OF EXPANSION ANGLES

WEIGHT OF ONE EXPANSION ANGLE--E A 2

(2)(1.4+ 1.5) + 33'= 39' of angle

39' of 3$\frac{1}{2}$x3 x 3/8∠@ 7.9#/ft.= 308#

(2)(2'8"+ 10") + 33'= 40' of plate

40' of 8" x 3/4" plate @ 20.4#/ft.= 816#

Total= 816+ 308= 1124#

WEIGHT OF ONE EXPANSION ANGLE--E A 2

(2)(2'8"+ 1'4")+ 33'= 41' of angle

41' of 3$\frac{1}{2}$ x 3 x 3/8"∠@ 7.9#/ft.= 324#

(2)(2'8"+ 10") + 33'= 40' of rod

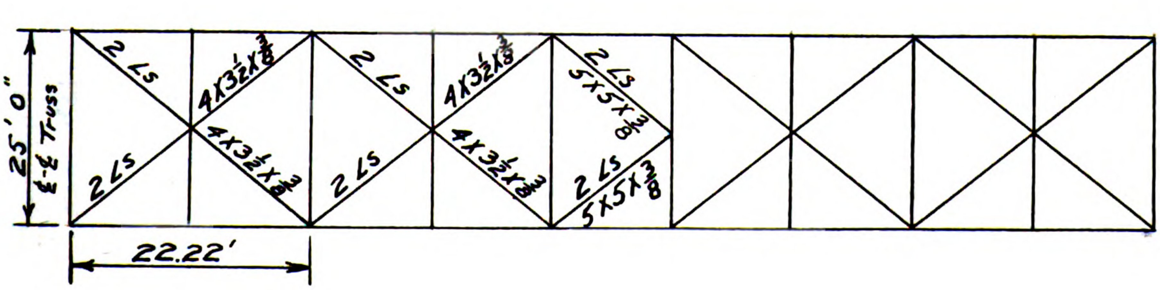
40' of 3/4" square rod @ 1.913#/ft.= 77#

Total= 324 + 77= 401"

TOTAL WEIGHT OF EXPANSION ANGLES APPLIED TO EACH TRUSS AT EACH END PANEL POINT

$$\frac{1124 + 401}{2} = 763\#$$

WEIGHT OF LATERAL BRACING TOP CHORD



$$l = \sqrt{(22.2)^2 + (24)^2} = 33.4 \text{ft.}$$

WEIGHT OF TWO SMALL ANGLES

Size--4" x $3\frac{1}{2}$" x 3/8" -33.4' @ 9.9#/ft.

(2)(9.1)(33.4)= 608#, wt. of angles

WEIGHT OF TWO LARGE ANGLES

Size--5"x5" x 3/8"--16.7' @ 12.3#/ft.

(2)(12.3)(16.7)= 411#, wt. of angles

WEIGHT OF PLATE

Size--36" x 30" x 3/8"

3'-30" x 3/8" @ 38.3#/ft.= (3)(38.3)= 115#

TOTAL WEIGHT OF TOP CHORD LATERAL BRACING

(8)(608) + (2)(411) + (12)(115)= 7,066#

We will assume this weight as being distributed equally between the twenty panel points, except for the end panel points, which take only half the load that the others do.

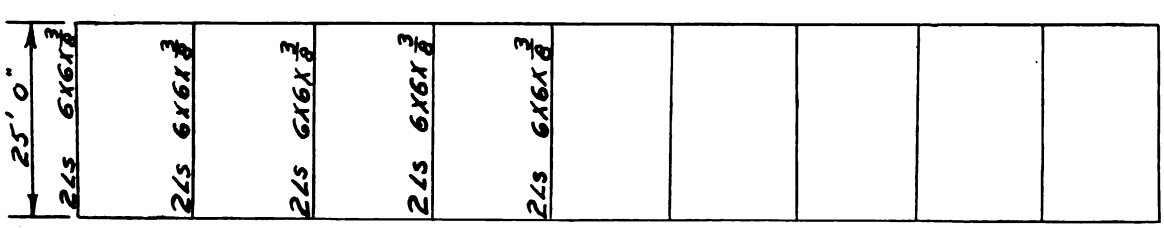
WEIGHT OF TOP CHORD BRACING APPLIED TO EACH TRUSS AT EACH INTERMEDIATE PANEL POINT

$$\frac{7{,}066}{18} = 392\#$$

WEIGHT APPLIED TO EACH TRUSS AT EACH END PANEL POINT

$$\frac{392}{2} = 196\#$$

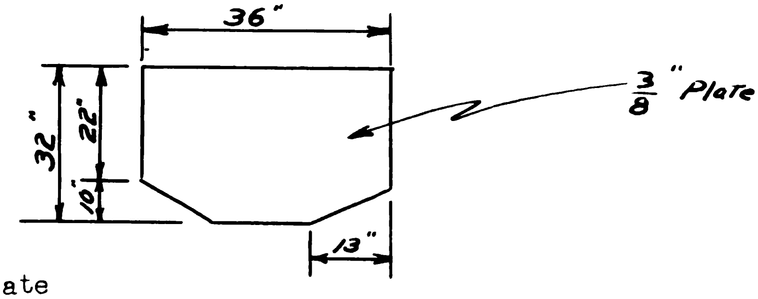
WEIGHT OF LATERAL BRACING BOTTOM CHORD



WEIGHT OF ONE BRACE (two angles + two plates)

Size--6" x 6" x 3/7" -25' 0" @ 14.9#/ft.

(2)(14.9)(25)= 745# Weight of angles



Area of Plate

(36)(32)-(13)(10)= 1152-130= 1022 sq. in.

Weight of plate $\frac{1022}{12} = 85.16"$

12" x 3/8" Plate weighs 15.3#/ft. $(15.3)\left(\frac{85.16}{(12)}\right) = 109\#$

Total weight of one brace (2 angles + 2 plates)

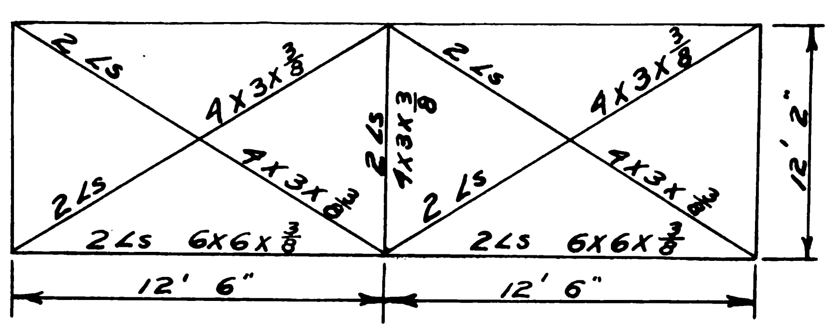
745 + (2)(109)= 963#

WEIGHT OF BOTTOM CHORD BRACING APPLIED TO EACH TRUSS AT EACH PANEL POINT

$\frac{963}{2} = 482\#$

WEIGHT OF CROSS FRAMES

(Intermediate and End)



$d = \sqrt{(12.5)^2 + (12.17)^2} = 17.42'$

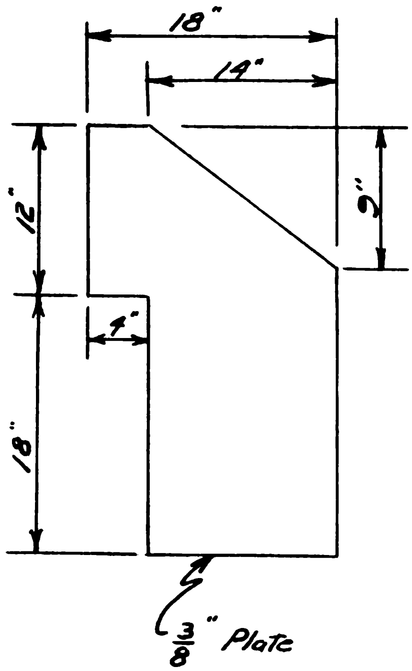
l= (8)(17.42) + (2)(12.17)= 139.36 + 24.34= 163.7' of angle

4 x 3 x 3/8

WEIGHT OF ONE CROSS FRAME

163.7' of 4 x 3 x 3/8" angle @ 8.5#/ft.= 1390#

50' of 6 x 6 x 3/8" @ 14.9#/ft.= 745#



Area of 3/8" plates

$(4)\ (18)(30)-(9)\left(\frac{14}{2}\right)-(4)(18) + (2)$

$(18)(24) + (2)\ (18)(18) = 3132$ sq. in.=

21.8 sq. ft.

Weight of 21.8' of 12" x 3/8' plate @

15.3#/ft= 334#

4∠s - 7 x 4 x 3/8 - 16" @ 13.6#/ft.=

$(2)\left(\frac{16}{12}\right)(13.6)= 72.6$#

4∠s - 4 x 3½ x 3/8 - 12" @ 9.1#/ft.

(2)(1)(9.1)= 36.4#

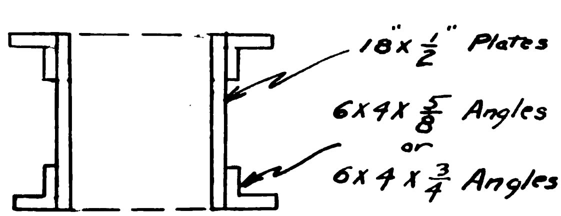
Total= 1390 + 745 + 334 + 72.6 +

36.4 = 2578#

WEIGHT OF ONE CROSS FRAME APPLIED TO EACH TRUSS AT EACH PANEL POINT

$\frac{2578}{2} = 1,289$#

WEIGHT OF UPPER CHORD MEMBERS OF TRUSS

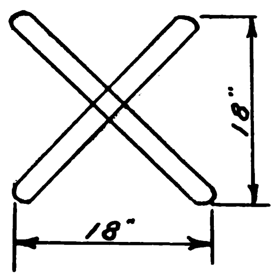


Length of member= 11.104 ft.

WEIGHT OF ONE MEMBER IN OUTER THREE PANELS

2 plates--18" x ½" - 11.104' @ 30.6#/ft.= 680#

4∠s - 6 x 4 x 5/8 - 11.104' @ 20.0#/ft.= 889#



$L = 18\sqrt{2} = 25.5"$, length of lacing bar

No. of Double Lacings required on one edge of member--

$\frac{(11.104)(12)}{(18)} = 7.4$ -- Use 7 as lacing does not go clear to the end of the member.

$2\frac{1}{2} \times \frac{1}{2}$ Lacing Bars

Length of $2\frac{1}{2}"$ x $\frac{1}{2}"$ bars needed in one member.

$(7)(2)(2)\frac{(25.5)}{(12)} = 59.5$ ft. of bar

Lacing bars--$2\frac{1}{2}"$ x $\frac{1}{2}"$ - 59.5' @ 4.25#/ft.= 253#

Total= 680 + 889 + 253= 1822#

WEIGHT OF ONE MEMBER IN MIDDLE THREE PANELS

2 plates--18" x $\frac{1}{2}"$ - 11.104' @ 30.6#/ft.=	680#
4$\angle$s--6 x 4 x 3/4" - 11.104' @ 23._#/ft.=	1048#
Lacing bars--$2\frac{1}{2}"$ x $\frac{1}{2}"$ - 59.5 @ 4.25#/ft.=	253#
Total=	1981#

TOTAL WEIGHT OF UPPER CHORD MEMBERS OF ONE TRUSS

(6)(1822) + (3)(1981)= 10,932 + 5,943= 16,875#

WEIGHT OF LOWER CHORD MEMBERS OF TRUSS

2 plates--18" x $\frac{1}{2}"$

6 x 4 x 7/16" angles in outer three panels

6 x 4 x 3/4" angles in middle three panels

$2\frac{1}{2}"$ x $\frac{1}{2}"$ lacing bars--25.5" long

Length of member= 11.104 ft.

WEIGHT OF ONE MEMBER IN OUTER THREE PANELS

2 plates--18" x $\frac{1}{2}"$ - 11.104 ft. @ 30.6#/ft.=	680#
4$\angle$s - 6" x 4" x 7/16" - 11.104 ft @ 14.3#/ft.=	635#
Lacing bars--$2\frac{1}{2}"$ x $\frac{1}{2}"$ - 59.5 ft. @ 4.25#/ft.=	253#
Total=	1568#

WEIGHT OF ONE MEMBER IN MIDDLE THREE PANELS

2 plates--18" x ½" - 11.104' @ 30.6#/ft.= 680#

4 ∠--6 x 4 x 3/4# - 11.104' @ 23.6#/ft.= 1048#

Lacing bars--2½" x ½" - 59.5' @ 4.25#/ft.= 253#

Total 1981#

TOTAL WEIGHT OF LOWER CHORD MEMBERS OF ONE TRUSS

(6)(1568) + (3)(1981)= 9,408 + 5943= 15,351#

WEIGHT OF VERTICAL MEMBERS OF TRUSS

At outer panel points

1 plate--11" x ½" and 4 angles 6" x 4" x ½"

At panels next to end

1 plate--11" x 3/8" and 4 angles 6" x 4" x 7/16"

At middle six panel points

1 plate--11" x 3/8" and 4 angles 5" x 3½" x 3/8"

Length of members= 10.958'

WEIGHT OF ONE MEMBER AT OUTER PANEL POINT

1 plate--11" x ½" - 10.958' @ 18.7#/ft.=	205 #
4 angles-- 6" x 4" x 7/16" - 10.958' @ 14.3#/ft.=	710#
Total=	915#

WEIGHT OF ONE MEMBER AT PANEL POINTS NEXT TO END

1 plate--11" x 3/8" - 10.958' @ 14.0#/ft.=	154#
4 ∠ -- 6" x 4" x 7/16" - 10.958' @ 14.3#/ft.=	628#
Total=	781#

WEIGHT OF ONE MEMBER AT MIDDLE SIX PANEL POINTS

1 plate--11" x 3/8" - 10.958' @ 14.0#/ft.=	154#
4 ∠ - 5 x 3½ x 3/8 - 10.958' @ 10.4#/ft.=	456#
Total=	610#

TOTAL WEIGHT OF VERTICAL MEMBERS OF ONE TRUSS

(2)(915) + (2)(781) + (6)(610)= 7,052#

WEIGHT OF DIAGONAL MEMBERS OF TRUSS

$$l=\sqrt{(11.104)^2 + (10.958)^2} = 15.600'$$

In outer panels

1 plate-11" x 5/8" and 4 ∠s--5" x 3½" x 3/4"

In panels next to end panels

1 plate-11" x ½" and 4∠s--5"x 3½"x 9/16"

In middle five panels

1 plate-11" x 3/8" and 4∠s--5"x 3½"x 3/8"

2 diagonals in Center panel

WEIGHT OF ONE MEMBER IN OUTER PANEL

1 plate-11" x 5/8" - 15.6 ft. @ 23.4#/ft=	365#
4∠s--5 x 3½ x 9/16 - 15.6' @ 19.8#/ft.-	1236#
Total=	1601#

WEIGHT OF ONE MEMBER IN PANELS NEXT TO END

1 plate-11" x ½" - 15.6 ft. @ 18.7#/ft.=	292#
4∠s--5 x 3½ x 9.16 - 15.6' @ 15.2#/ft.=	950#
Total	1242#

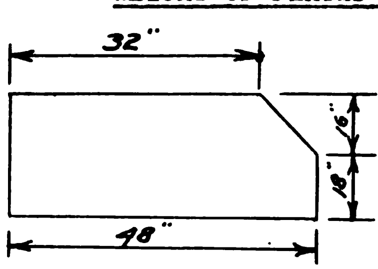
WEIGHT OF ONE MEMBER IN MIDDLE FIVE PANELS

1 plate-11" x 3/8" - 15.6' @ 14.0#/ft.=	218#
4∠s--5" x 3½" x 3/8" - 15.6' ¢ 10.4#/ft.=	649#
Total=	867#

TOTAL WEIGHT OF DIAGONAL MEMBERS OF ONE TRUSS

(2)(1601) + (2)(1242) + (6)(867)= 10,888#

WEIGHT OF PLATES AT TRUSS JOINTS



20 plates per truss

AREA OF ONE PLATE

$$\frac{(48)(34) - \frac{(16)(16)}{2}}{144} = \frac{1632-128}{144} = 10.45 \text{ sq. ft.}$$

WEIGHT OF PLATES ON ONE TRUSS

Total area of $\frac{1}{2}$" plate

(20)(10.45)= 209 sq. ft.

Equivalent to 209 ft. of plate 12" wide

12" x $\frac{1}{2}$" plate - 209' @ 20.4#/ft.= 4,270#

WEIGHT OF TRUSSES AND CROSS FRAMES

WEIGHT OF ONE TRUSS

Upper chord members--	16,875#
Lower chord members--	15,351#
Vertical members--	7,052#
Diagonal members--	10,888#
Plates at joints--	4,270#
Total=	54,436#

WEIGHT OF CROSS FRAMES APPLIED TO ONE TRUSS

(1289)(10)= 12,890#

TOTAL WEIGHT OF ONE TRUSS AND CROSS FRAMES APPLIED TO IT

Truss weight--	54,436#
Cross frame weight--	12,890#
Estimated weight such as rivets, welds, fillers, small angles, small tie plates on truss members, etc.	5,000
Total=	72,326#

Use total= 72,400

WEIGHT APPLIED TO EACH UPPER CHORD INTERMEDIATE PANEL POINT

Take 2/3 of weight as being applied to upper chord

$$(2/3)\left(\frac{72,400}{(9)}\right) = 5,360\#$$

WEIGHT APPLIED AT UPPER CHORD END POINTS

$$\frac{5,360}{2} = 2680\#$$

WEIGHT APPLIED TO EACH LOWER CHORD INTERMEDIATE PANEL POINT

$$(1/3)\left(\frac{72,400}{(8)}\right) = 3,020\#$$

WEIGHTS APPLIED TO TRUSS DUE TO DEAD LOADS

WEIGHT APPLIED AT EACH UPPER CHORD INTERMEDIATE PANEL POINT

Slab--11.104' @ 2160#/ft.=	24,000#
Walk--11.104' @ 335#/ft.=	3,720#
Railing section--11.104' @ 44#/ft.=	490#
One intermediate railing post=	810#
Stringers=	1,744#
Floor Beam=	2,144#
Lateral Bracing	392#
Truss and cross frames=	5,360#
Total=	38,671#
Use total=	38,700#

WEIGHT APPLIED AT EACH UPPER CHORD END PANEL POINT

Slab--5.568' @ 2,160#/ft.=	12,030#
Walk--5.568' @ 335#/ft.=	1,865#
Railing section--5.568' @ 44#/ft.=	245#
One end railing post=	1,470#
Stringers=	878#
End floor beam and concrete around it=	7,948#
Expansion Angles=	763#

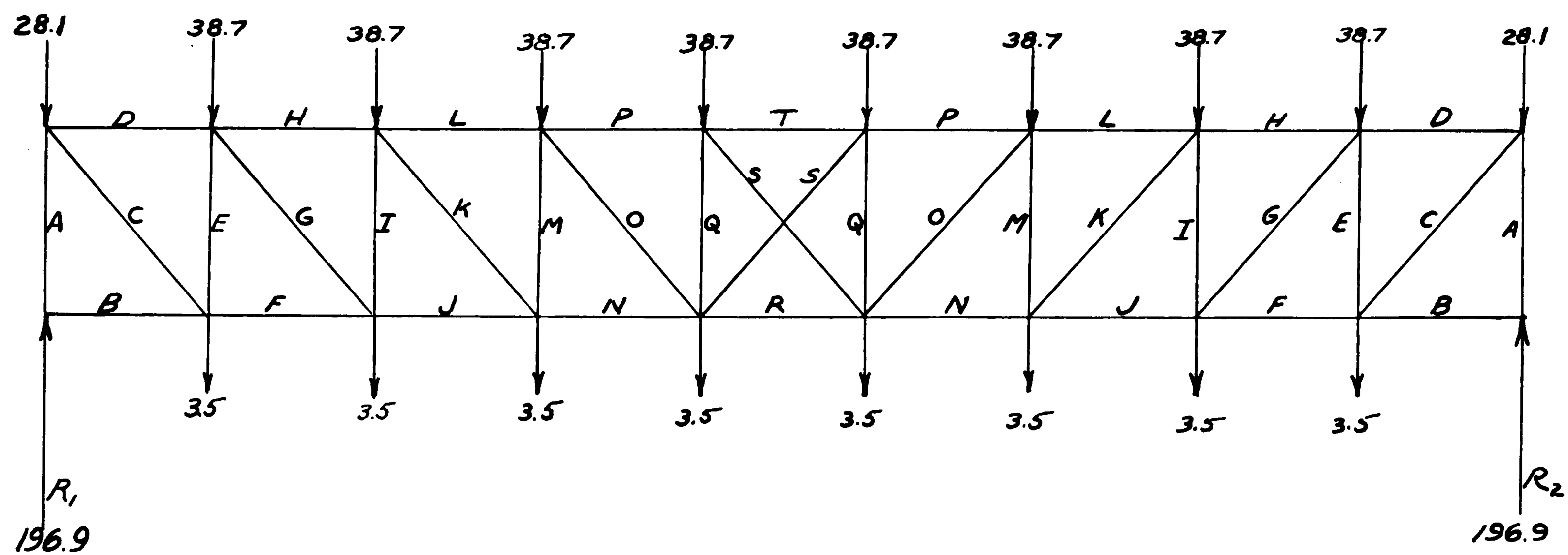
WEIGHT **APPLIED** AR EACH UPPER CHORD END PANEL POINT (con't)

Lateral Bracing=	196#
Truss and cross frames=	2,680#
Total=	28,085#
Use total=	28,100#

WEIGHT **APPLIED** AT LOWER CHORD INTERMEDIATE PANEL POINTS

Bottom chord lateral bracing=	482#
Truss and cross frames=	3,020#
Total=	3,502#
Use total=	3,500#

DEAD LOADS APPLIED TO TRUSS (in kips)

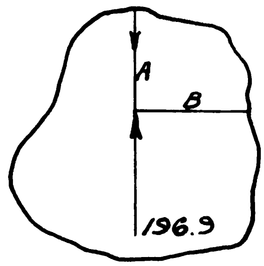


$$R_1 = R_2 = (4)(38.7) + 28.1 + (4)(3.5)$$

$$R_1 = R_2 = 196.9$$

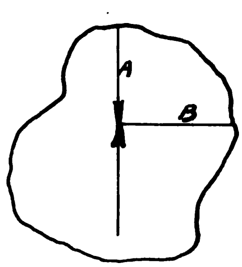
DEAD LOAD STRESSES IN MEMBERS

MEMBER "A"



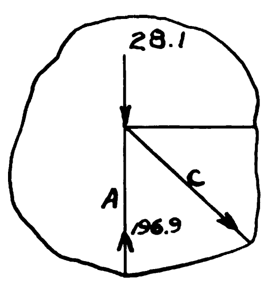
$\Sigma F_v = 0 \quad V_a = S_a = 196$ kips compression

MEMBER "B"



$\Sigma F_h = 0 \quad H_b = S_b = 0$

MEMBER "C"

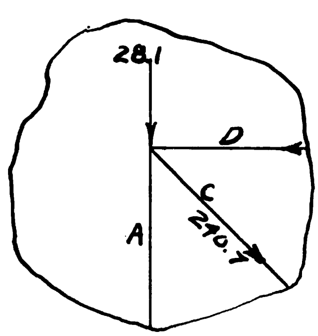


$\Sigma F_v = 0 \quad V_c + 28.1 = 196.9$

$V_c = 196.9 - 28.1 = 168.8$

$S_c = (168.8)\dfrac{(15.622)}{(10.958)} = 240.7$ kips tension

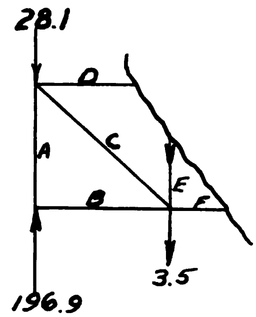
MEMBER "D"



$\Sigma F_h = 0 \quad H_d = S_d = (240.7)\dfrac{(11.135)}{(10.958)} =$

171.5 kips compression

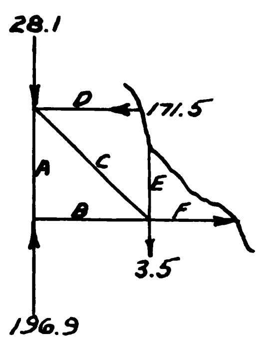
MEMBER "E"



$\Sigma F_v = 0 \quad V_e + 28.1 + 3.5 = 196.9$

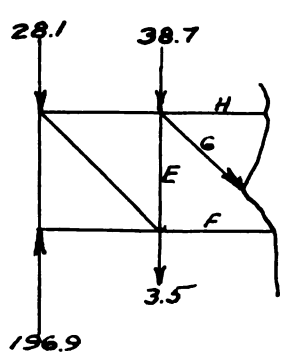
$V_e = S_e = 196.9 - 31.6 = 165.3$ kips compression

MEMBER "F"



$\Sigma F_h = 0 \quad H_f = S_f = 171.5$ kips tension

MEMBER "G"

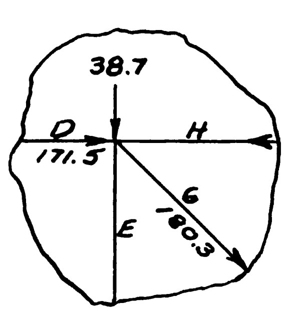


$\Sigma F_v = 0 \quad V_g + 28.1 + 28.7 + 3.5 = 196.9$

$V_g = 126.6$

$S_g = (126.6)\dfrac{(15.600)}{(10.958)} = 180.3$ kips tension

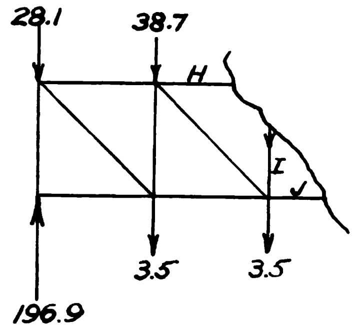
MEMBER "H"



$\Sigma F_h = 0 \quad H_h = S_h = 171.5 + (180.3)\dfrac{(11.104)}{(15.600)}$

$S_h = 171.5 + 128.4 = 299.9$ kips compression

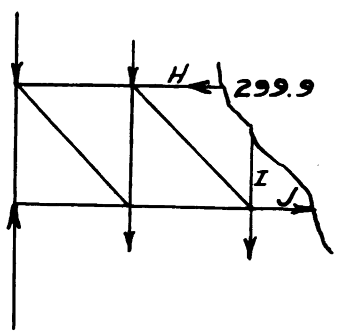
MEMBER "I"



$\Sigma F_v = 0 \quad V_i + 28.1 + 38.7 + (2)(3.5) = 196.9$

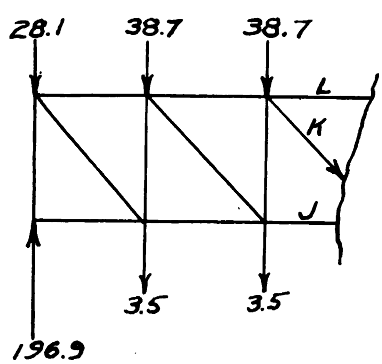
$V_i = S_i = 123.1$ kips compression

MEMBER "J"



$\Sigma F_h = 0 \quad H_j = S_j = 299.9$ kips tension

MEMBER "K"

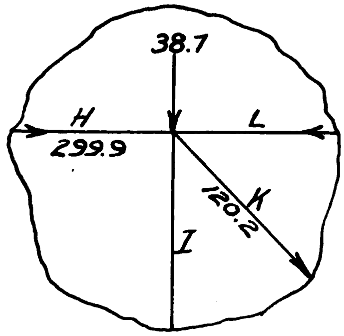


$\Sigma F_v = 0 \quad V_k + 28.1 + (2)(38.7) + (2)(3.5) = 196.9$

$V_k = 84.4$

$S_k = (84.4)\frac{(15.600)}{(10.958)} = 120.2$ kips tension

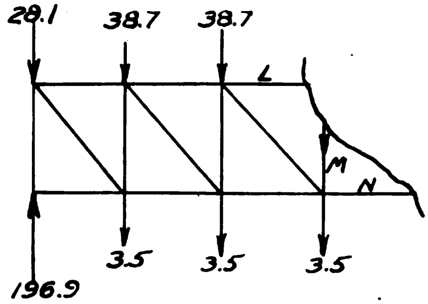
MEMBER "L"



$\Sigma F_h = 0 \quad H_l = S_l = 299.9 + (120.2)\frac{(11.104)}{(15,600)}$

$S_l = 299.9 + 85.6 = 385.5$ kips compression

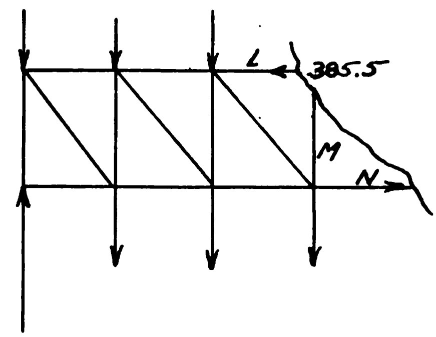
MEMBER "M"



$\Sigma F_v = 0 \quad V_m + 28.1 + (2)(38.7) + (3)(3.5) = 196.9$

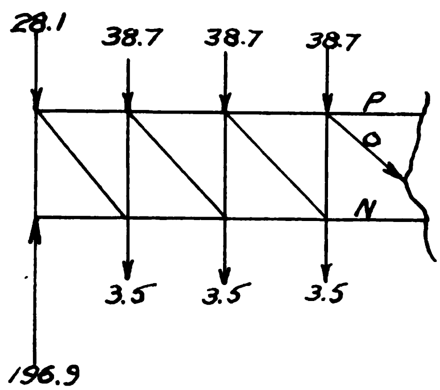
$V_m = S_m = 80.9$ kips compression

MEMBER "N"



$\Sigma F_h = 0 \quad H_n = S_n = 385.5$ kips tension

MEMBER "O"

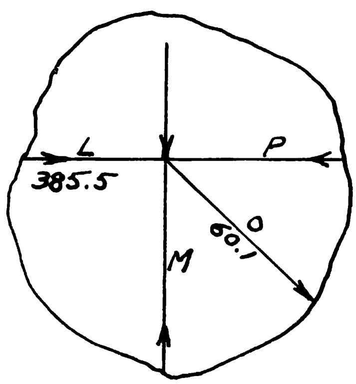


$\Sigma F_v = 0 \quad V_o + 28.1 + (3)(38.7) + (3)(3.5) = 196.9$

$V_o = 42.2$

$S_o = (42.2)\frac{(15.600)}{(10.958)} = 60.1$ kips tension

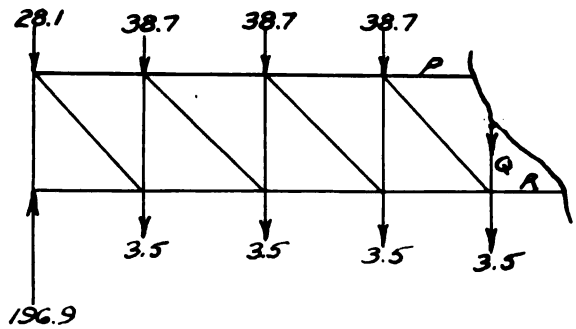
MEMBER "P"



$\Sigma F_h = 0 \quad H_p = S_p = 385.5 + (60.1)\frac{(11.104)}{(15.600)}$

$S_p = 385.5 + 42.8 = 428.3$ kips compression.

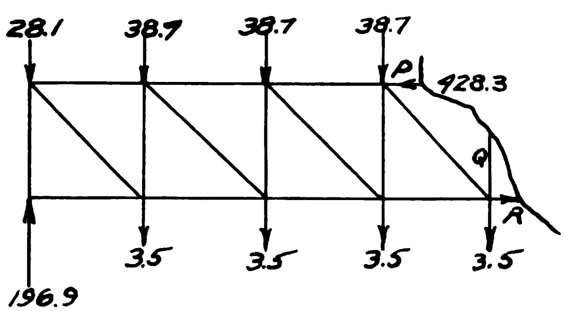
MEMBER "Q"



$\Sigma F_v = 0 \qquad V_q + 28.1 + (2)(28.7) + (4)(3.5) = 196.9$

$V_q = S_q = 38.7$ kips compression

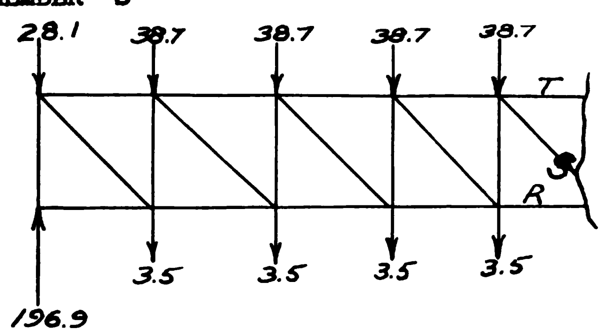
MEMBER "R"



$\Sigma F_h = 0$

$H_r = S_r = 428.3$ kips tension

MEMBER "S"



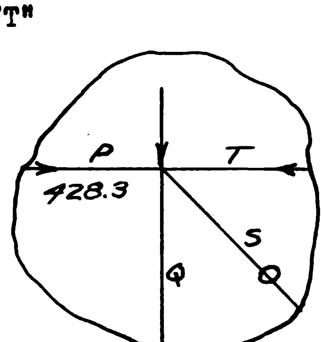
$\Sigma F_v = 0$

$V_s = 196.9 - 28.1 - (4)(38.7) -$

$(4)(3.5)$

$V_s = 0 \qquad S_s = 0$

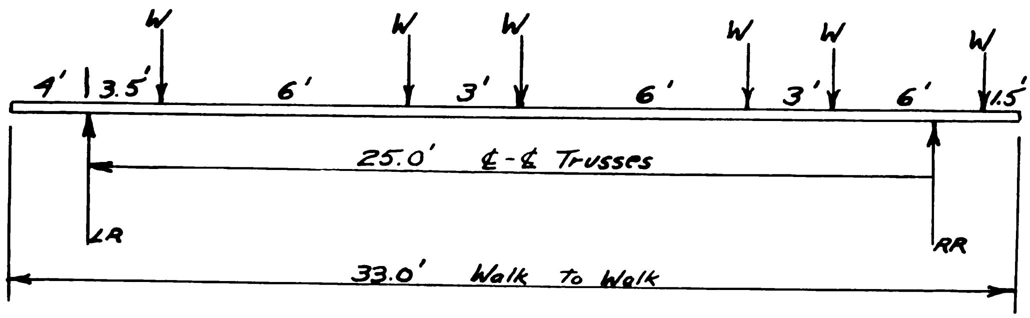
MEMBER "T"



$\Sigma F_h = 0$

$H_t = S_t = 428.3$ kips compression

MAXIMUM AMOUNT OF TRUCK LOADING APPLIED TO ONE TRUSS



W= load applied to the slab by each wheel

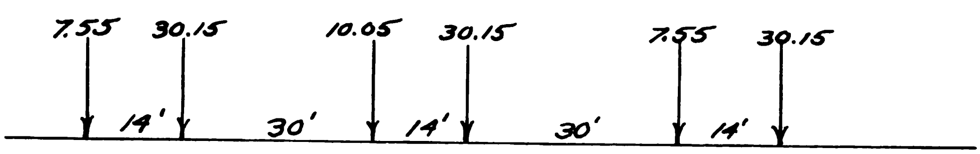
$M_1 = 0 \quad W(3.5 + 9.5 + 12.5 + 18.5 + 21.5 + 27.5) = R(25.0)$

$RR = \frac{93\,W}{25} = 3.72\,W$

Since three lanes of trucks are used in obtaining the maximum loading, 90% of the total load is used in the design.

$(90\%)(3.72\,W) = 3.35\,W$

As H-15 loading is being used, the maximum stresses in the truss will be produced by loading the truss with moving loads as shown below.



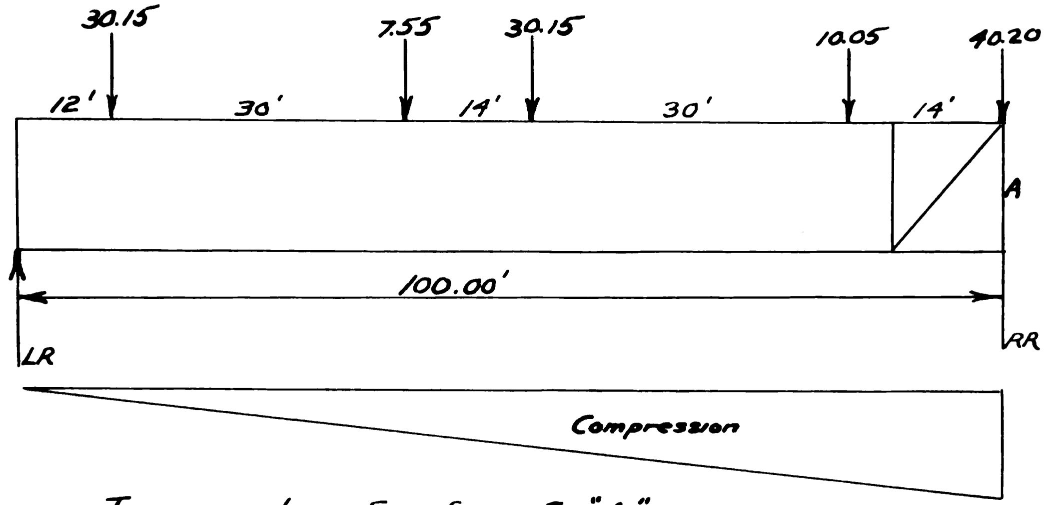
$(3.35)(2.25) = 7.55$ kips

$(3.35)((9.00) = 30.15$ kips

$(3.35)(3.00) = 10.05$ kips

$(3.35)(12.00) = 40.20$ kips

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "A"

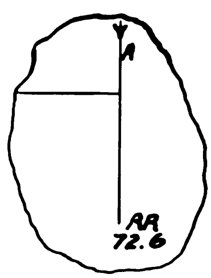


INFLUENCE LINE FOR SRESS IN "A"

LIVE LOAD STRESS DUE TO TRUCK TRAIN LOADING

$$RR = 40.20 + (30.15)\frac{(56 + 12)}{(100)} + (10.05)\frac{(86)}{(100)} + (7.55)\frac{(42)}{(100)}$$

$$RR = 40.2 + 20.5 + 8.7 + 3.2 = 72.6$$



$\Sigma F_v = 0 \quad S_a = 72.6$ kips

Impact Coefficient--$I = \frac{+20}{6 + 20} = \frac{120}{620} =$

19.35%

IMPACT STRESS

(19.35%)(7216)= 14.1 kips

LIVE LOAD STRESS DUE TO SIDEWALK LOADING

Width of walk= 20"= 5/3'

$$P = \left(40 + \frac{3000}{(L)}\right)\frac{(55 - W)}{(50)} = \left(40 + \frac{3000}{(100)}\right)\frac{(55 - 5/3)}{(50)} = 74.7\#/\text{sq.ft.}$$

$S_a = RR = (74.7)(5/3)(100/2) = 6.3$ kips

LIVE LOAD STRESSES DUE TO RAILING LOADING

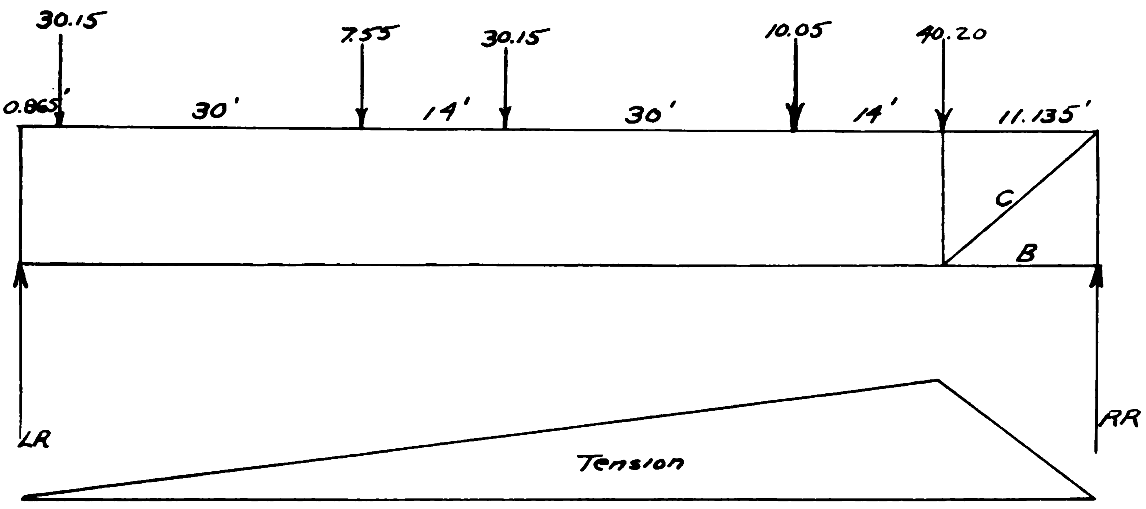
W= 100#/lineal foot

$S_a = RR = (100)(50) = 5.0$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "A"

72.6 + 14.1 + 6.3 + 5.0= 98.0 kips compression

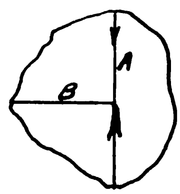
<u>MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBERS "B" AND "C"</u>



INFLUENCE LINE FOR STRESS IN "C"

LIVE LOAD STRESS IN MEMBER "B"

$\Sigma F_h = 0 \quad S_b = 0$



LIVE LOAD STRESS IN "C" DUE TO TRUCK TRAIN LOADING

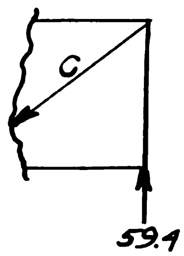
$$RR = \frac{(30.15)(0.865) + (7.55)(30.865) + (30.15)(44.865) +}{100}$$

$$\frac{(10.05)(74.865) + (40.20)(88.865)}{100}$$

RR= 59.4 kips

$\Sigma F_v = 0 \quad V_c = 59.4$

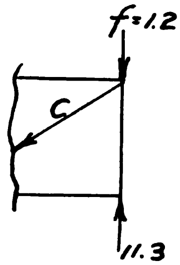
$S_c = (59.4)\frac{(15.622)}{(10.958)} = 84.8$ kips



IMPACT STRESS

(19.35%)(84.8)= 16.4 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES



P= 74.7#/sq. ft. of sidewalk

w=100 #/ft. of railing

$RR = (74.7)(5/3)\frac{(100)}{(2)} + (100)\frac{(100)}{(2)} = 11.3$ kips

$f = (74.7)(5/3)\left(\frac{11.135}{(2)}\right) + (100)\left(\frac{11.135}{(2)}\right) = 1.2$ kips

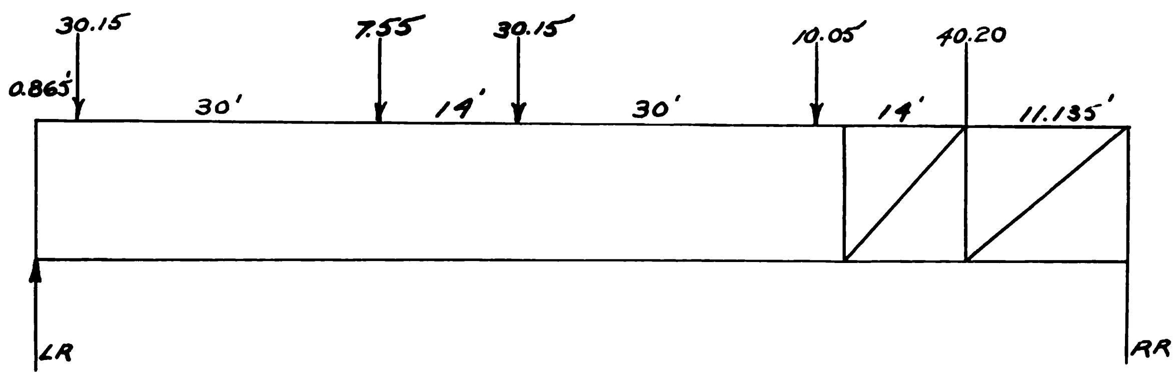
$\Sigma F_V = 0$

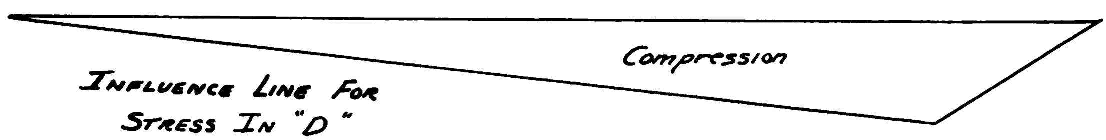
$V_c = 11.3 - 1.2 = 10.1 \qquad S_c = (10.1)\frac{(15.622)}{(10.958)} = 14.4$ kips

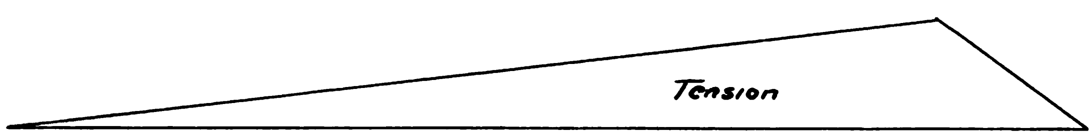
TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "C"

$64.8 + 16.4 + 14.4 = 115.6$ kips tension

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBERS "D" AND "F"







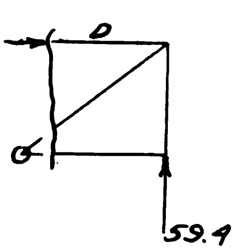
Influence Line For Stress In "F"

LIVE LOAD STRESSIN "D" DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(30.15)(0.865) + (7.55)(30.865) + (30.15)(44.865) +}{100}$$

$$\frac{(10.05)(74.865) + (40.20)(88.865)}{100}$$

RR= 59.4 kips



$\Sigma M_o = 0 \qquad (S_d)(10.958) = (59.4)(11.135)$

$S_d = 60.4$ kips

IMPACT STRESS $(19.35\%)(60.4) = 11.7$ kips

SIDEWALK AND RAILING LIVE LOAD STRESSES

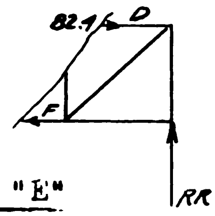
$RR = 11.3 \quad S_d = (11.3 - 1.2)\dfrac{(11.135)}{(10.958)} = 10.3$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "D"

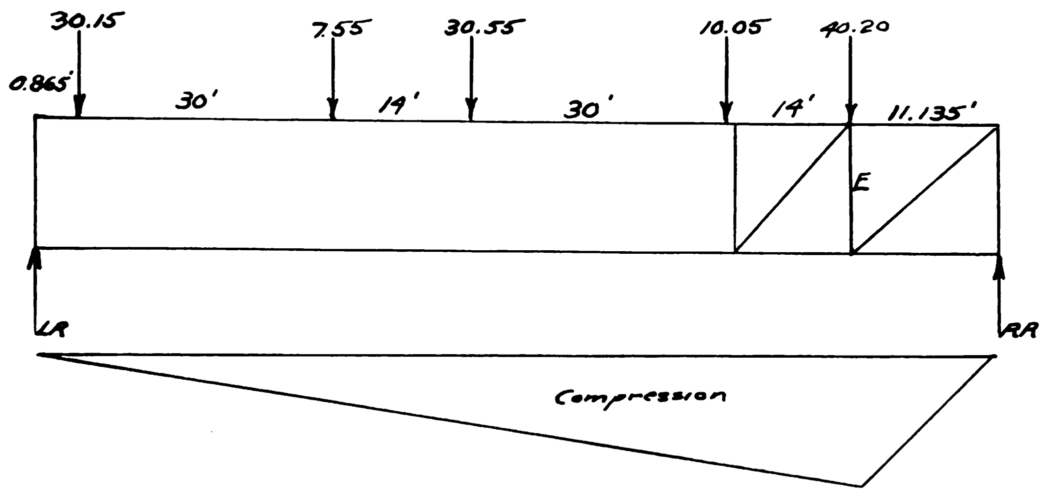
$60.4 + 11.7 + 10.3 = 82.4$ kips compression

TOTAL LIVE LOAD AND IMPACT STRESSIN MEMBER "F"

$\Sigma F_h = 0 \quad S_f = 82.4$ kips tension



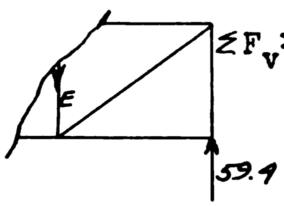
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "E"



INFLUENCE LINE FOR STRESS IN "E"

LIVE LOAD STRESS IN "E" DUE TO TRUCK TRAIN LOADING

RR= 59.4 kips



$\Sigma F_v = 0 \quad S_e = 59.4$ kips

IMPACT STRESS $(19.35\%)(59.4) = 11.5$ kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "E"

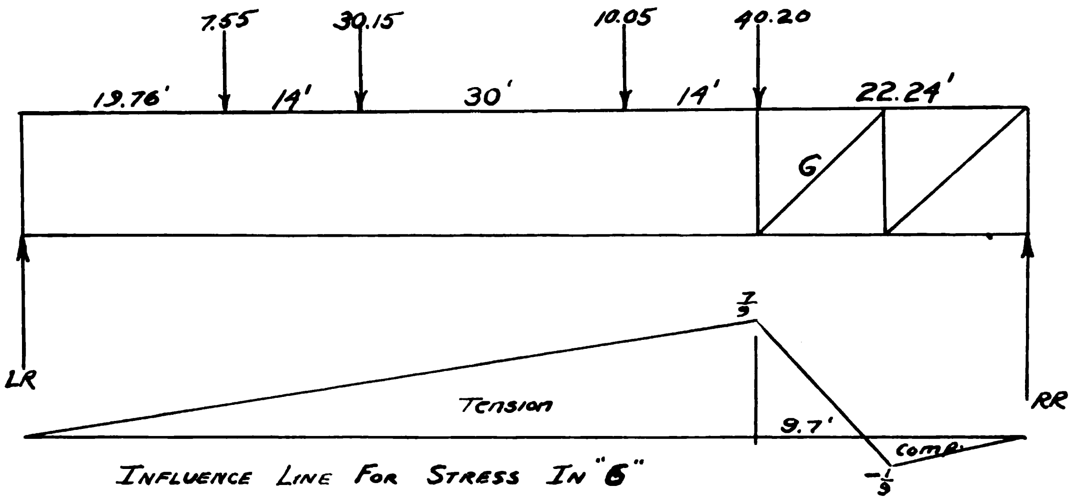
$RR = 11.3 \quad f = 1.2$

$\Sigma F_v = 0 \quad S_e = 11.3 - 1.2 = 10.1$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "F"

$59.4 + 11.5 + 10.1 = 81.0$ kips compression

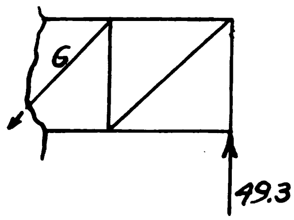
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "G"



INFLUENCE LINE FOR STRESS IN "G"

Loaded Length= $(7/8)(11.104') + 11.135 + (6)(11.104) = 87.5'$

LIVE LOAD STRESS IN "G" DUE TO TRUCK TRAIN LOADING



$RR = 49.3$

$\Sigma F_v = 0 \quad V_g = 49.3$

$$S_g = (49.3)\frac{(15.600)}{(10.958)}$$

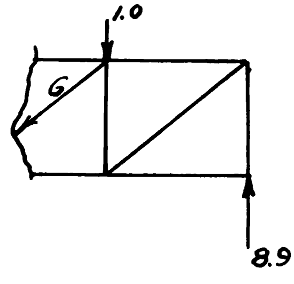
$S_g = 70.2$ kips

IMPACT STRESS $(19.35\%)(70.2) = 13.6$ kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "G"

$$\underset{\text{(sidewalk)}}{P} = \left(40 + \frac{3000}{(L)}\right)\frac{(55 - W)}{(50)} = \left(40 + \frac{3000}{(87.5)}\right)\frac{(55 - 5/3)}{(50)} = 79.2\#/\text{sq.ft.}$$

w(railings)= 100#/ft.



$W_{Tot} = (79.2)(5/3) + 100 = 232.0\#/\text{ft.}$

$$RR = \frac{(232)(87.5)}{(100)}\frac{(87.5)}{(2)} = 8.9 \text{ kips}$$

$f = (232)(9.7)(9.7) = 1.0$ kips

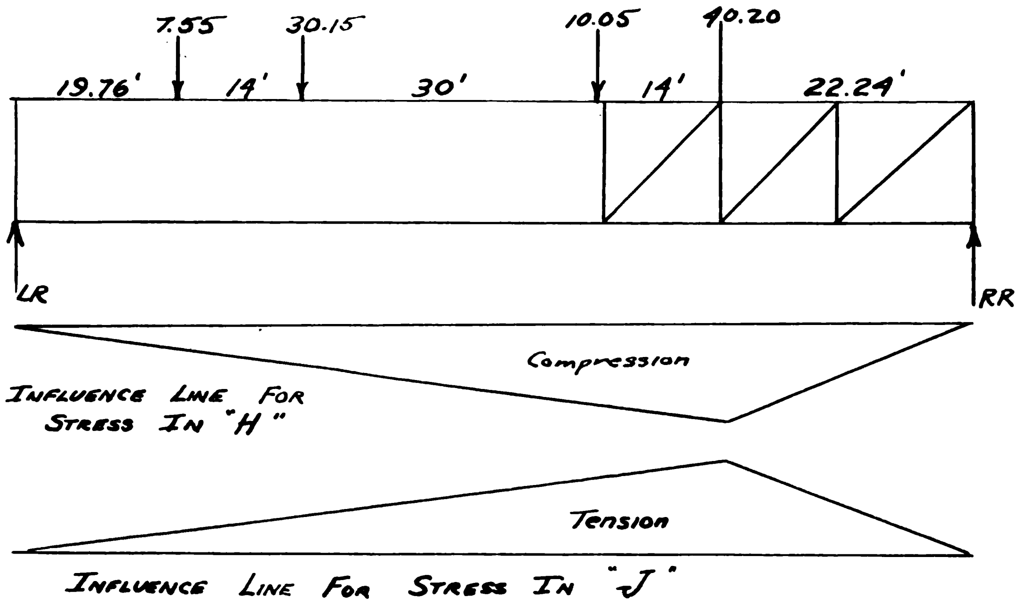
$\Sigma F_V = 0 \quad V_g = 8.9 - 1.0 = 7.9$

$S_g = \frac{(7.9)(15,600)}{(10,958)} = 11.3$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "G"

70.2 + 13.6 + 11.3 = 95.1 kips tension

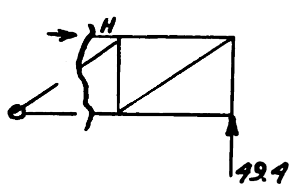
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBERS "H" AND "J"



LIVE LOAD STRESS IN MEMBER "H" DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(7.55)(19.76) + (30.15)(33.76) + (10.05)(63.76) +}{100}$$

$$\frac{(40.20)(77.76)}{(100)} = 49.4$$



$\Sigma M_o = 0 \quad (49.4)(22.24) = H_h(10.958)$

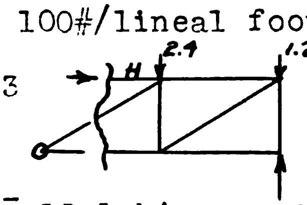
$H_h = S_h = 100.3$ kips

IMPACT STRESS $(19.35\%)(100.3) = 19.4$ kips

SIDEWALK AND RAILING LIVE LOAD STRESSES

P(sidewalk) = 74.7#/sq.ft w(railing) = 100#/lineal foot

$RR = (74.7)(5/3)\frac{(100)}{(2)} + \frac{(100)(100)}{(2)} = 11.3$

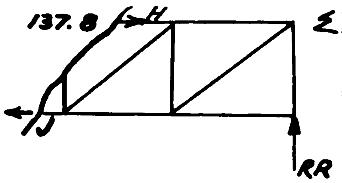


$\Sigma M_o = 0 \quad S_h = \frac{(10.1)(22.24) - (2.4)(11.104)}{(10.958)} = 18.1$ kips

TOTAL LIVE LOAD STRESS IN MEMBER "H"

100.3 + 19.4 + 18.1 = 137.8 kips compression

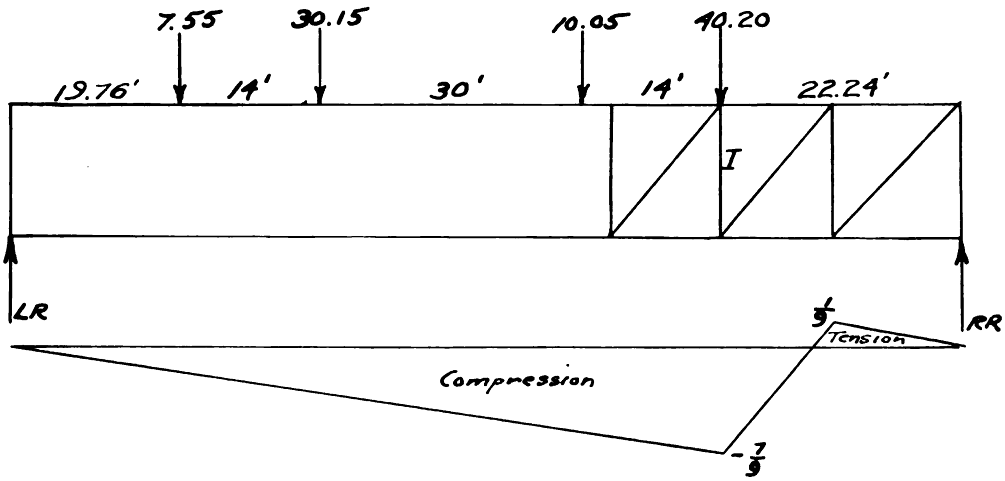
TOTAL LIVE LOAD STRESS IN MEMBER "J"



$\Sigma F_h = 0$

$S_j = 137.8$ kips tension

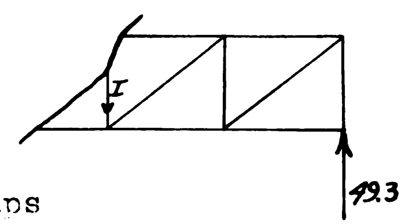
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "I"



Loaded Length = 87.5'

LIVE LOAD STRESS IN "I" DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(7.55)(19.76) + (30.15)(33.76) + (10.05)(63.76)}{100} + \frac{(40.20)(77.76)}{100}$$



$\Sigma F_v = 0$

$V_i = S_i = 49.3$ kips

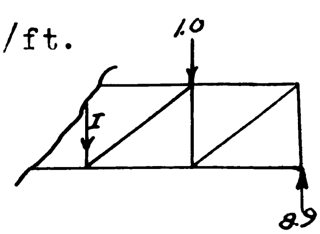
IMPACT STRESS (19.35%)(49.3) = 9.6 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "I"

P(sidewalk) = 79.2#/ft. w(railings) = 100#/ft.

w(total) = (79.2)(5/3) + 100 = 232.0#/ft.

$$RR = \frac{(232)(87.5)}{(100)} \frac{(87.5)}{(2)} = 8.9 \text{ kips}$$



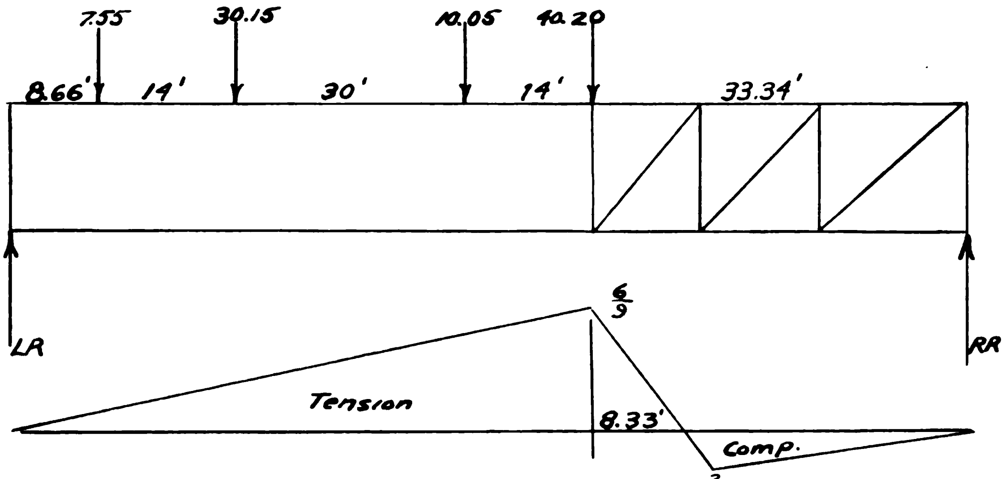
$\frac{(.232)(9.7)}{(11.104)}\frac{(9.7)}{(2)} = 1.0$ kips

$\Sigma F_V = 0 \quad S_i = 8.9 - 1.0 = 7.9$ kips

TOTAL LIVE LOAD STRESS IN MEMBER "I"

$49.3 + 9.6 + 7.9 = 66.8$ kips compression

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "K"



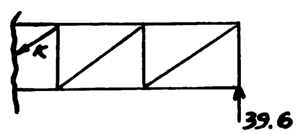
INFLUENCE LINE FOR STRESS IN "K"

Loaded Length = $11.135 + (5)(11.104) + (6/8)(11.104) = 75.0'$

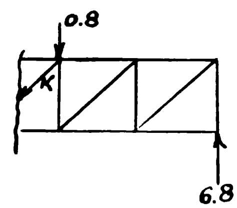
LIVE LOAD STRESS IN "K" DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(7.55)(8.66) + (30.15)(22.66) + (10.05)(52.66) +}{100}$$

$$\frac{(40.20)(66.66)}{100} = 39.6 \text{ kips}$$



$\Sigma F_V = 0 \quad V_k = 39.6 \quad S_k = (39.6)\frac{(15.600)}{(10.958)} = 56.4$ kips



IMPACT STRESS $(19.35\%)(56.4) = 10.9$ kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "K"

$P(\text{sidewalk}) = (40 + \frac{3000}{(75)})(\frac{55 - 5/3}{(50)}) = 85.3\#/\text{sq.ft.}$

$w(\text{railing}) = 100\#/\text{ft.} \quad w(\text{total}) = (85.3)(5/3) + 100 = 242\#/\text{ft.}$

$$RR = \frac{(.242)(75)}{100}(75/2) = 6.8 \text{ kips}$$

$$f = \frac{(.242)(8.33)}{(11.104)}\frac{(8.33)}{(2)} = 0.8 \text{ kip}$$

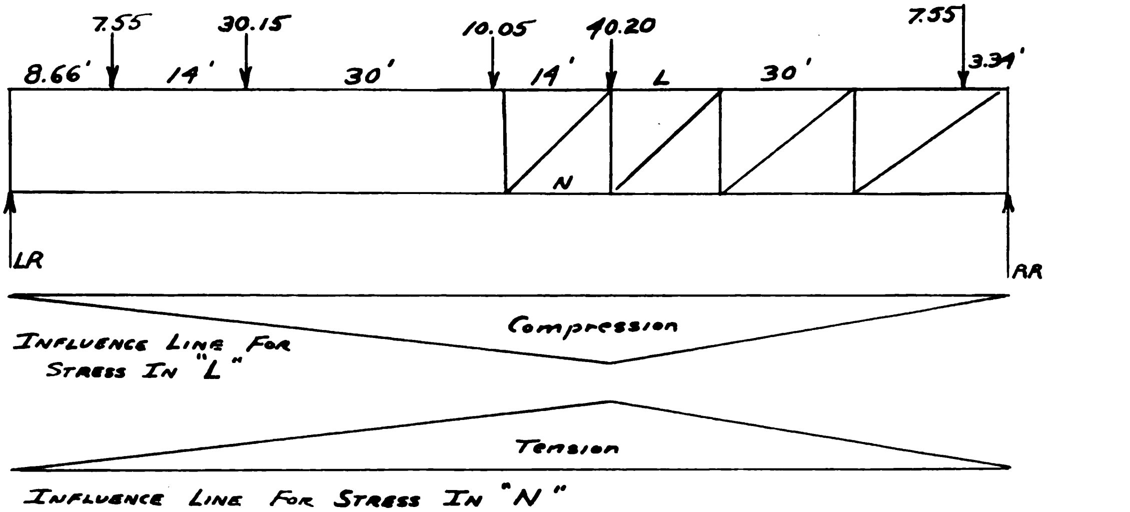
$\Sigma F_V = 0 \quad V_K = 6.8 - 0.8 = 6.0$

$$S_K = (6.0)\frac{(15.600)}{(10.958)} = 8.6 \text{ kips}$$

TOTAL LIVE LOAD STRESS IN MEMBER "K"

$56.4 + 10.9 + 8.6 = 75.9$ kips tension

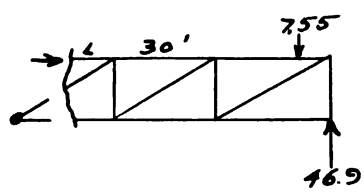
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBERS "L" AND "N"



LIVE LOAD STRESS IN "L" DUE TO TRUCK TRAIN LOADING

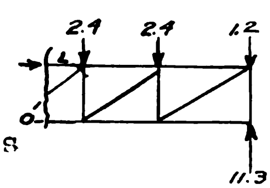
$$RR = \frac{(7.55)(8.66) + (30.15)(22.66) + (10.05)(52.66) +}{100}$$

$$\frac{(40.20)(66.66) + (7.55)(96.66)}{100}$$



$RR = 46.9$ kips $\quad \Sigma M_o = 0 \quad S_L = 122.0$ kips

IMPACT STRESS $(19.35\%)(122.0) = 23.6$ kips



SIDEWALK AND RAILING LIVE LOAD STRESSES IN "L"

$RR = 11.3$ kips $\quad f_1 = 2.4$ kips $\quad f_2 = 1.2$ kips

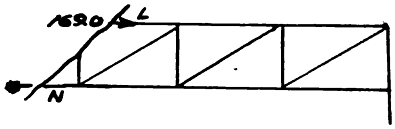
$\Sigma M_o = 0 \quad (S_1)(10.958) + (2.4)(11.10 + 22.21) = (10.1)(33.34)$

S_l = 23.4 kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "L"

122.0 + 23.6 + 23.4 = 169.0 kips compression

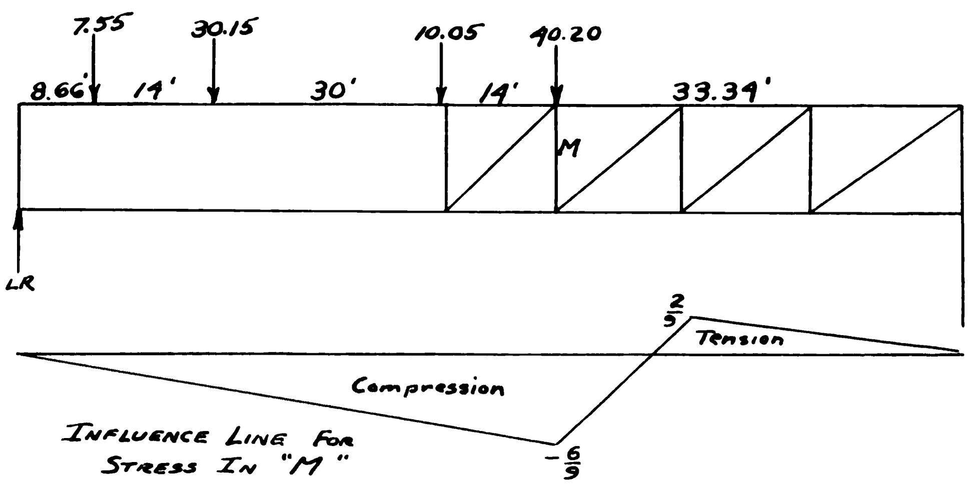
TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "N"



$\Sigma F_h = 0$

S_n = 169.0 kips tension

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "M"

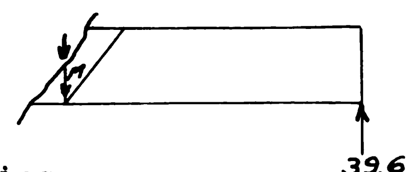


Loaded Length- 75.0'

LIVE LOAD STRESS IN "M" DUE TO TRUCK TRAIN LOADING

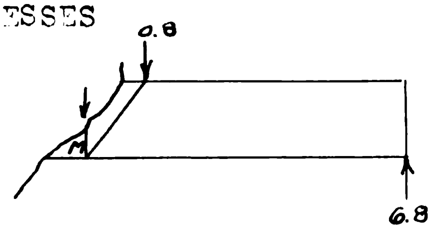
RR= 39.6 kips

$\Sigma F_v = 0$ $V_m = S_m =$ 39.6 kips



IMPACT STRESS (19.35%)(39.6)= 7.7 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES



P(sidewalk)= 85.3#/sq. ft.

w(railing)- 100#/ft.

W(total)-- 242#/ft.

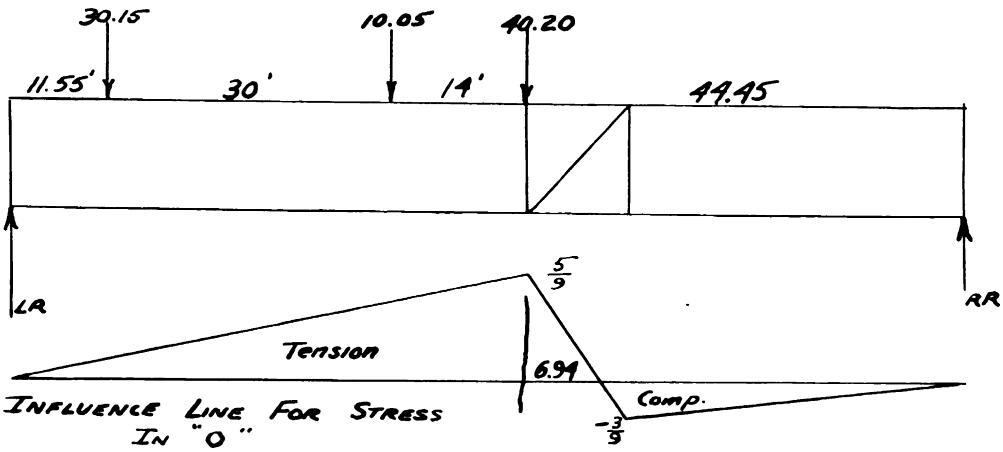
RR= 6.8 kips

f- 0.8 kip $\Sigma F_v = 0$ S_m = 6.8 - 0.8= 6.0 kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "M"

39.6 + 7.7 + 6.0 = 53.3 kips compression

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "O"



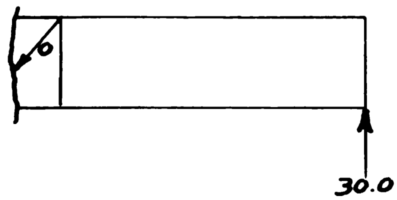
INFLUENCE LINE FOR STRESS IN "O"

Loaded Length- 11.135 + (4)(11.104) + (5/8)(11.104)= 62.5 ft.

LIVE LOAD STRESS IN "L" DUE TO TRUCK TRAIN LOADING

$$RR= \frac{(30.15)(11.55) + (10.05)(41.55) + (40.20)(55.55)}{100}$$

= 30.0 kips



$\Sigma F_V= 0 \qquad V_O=30.0$

$$S_O= (30.0)\frac{(15.800)}{(10.958)}= 42.7 \text{ kips}$$

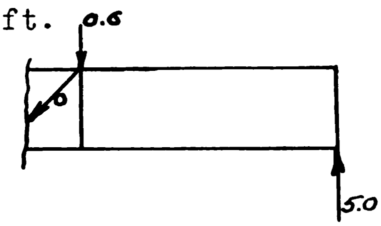
IMPACT STRESS (19.35%)(42.7)= 8.3 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES

$$P(\text{sidewalk})= \left(40 + \frac{3000}{62.5}\right) \frac{(55- 5/3)}{(50)}= (88)\frac{(16)}{(15)}= 93.9\#/\text{sq.ft.}$$

w(railing)= 100#/ft.

w(total)= (93.9)(5/3) + 100= 256#/ft.



$$RR= \frac{(.256)(62.5)}{(100)}\frac{(62.5)}{(2)}= 5.0 \text{ kips}$$

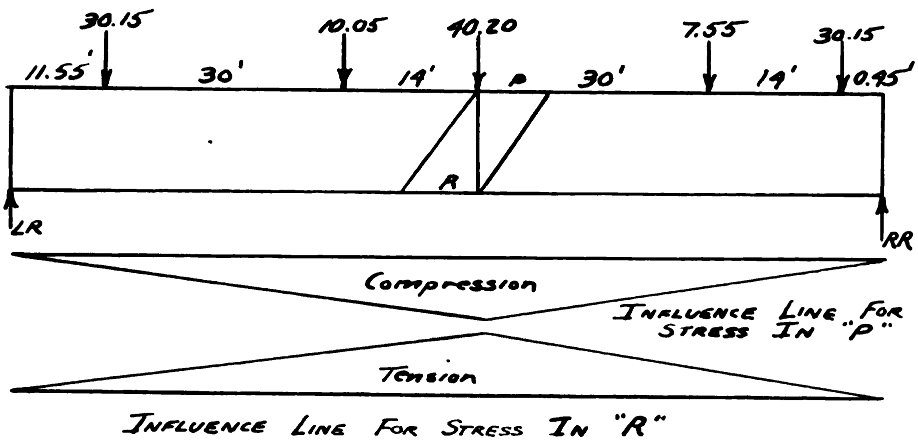
$$f= \frac{(.256)(6.94)}{(11.104)}\frac{(6.94)}{(2)}= 0.6 \text{ kip}$$

$\Sigma F_V= 0 \qquad V_O= 5.0 - 0.6= 4.4 \qquad S_O= \frac{(4.4)(15.800)}{(10.958)}= 6.2$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "O"

42.7 + 8.3 + 6.2 = 57.2 kips tension

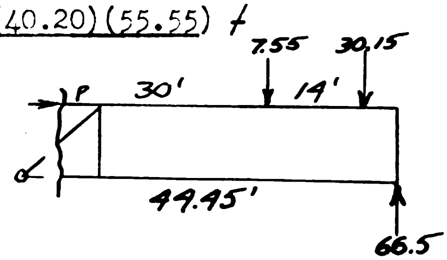
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBERS "P" AND "R"



LIVE LOAD STRESS IN "P" DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(30.15)(11.55) + (10.05)(41.55) + (40.20)(55.55)}{100} +$$

$$\frac{(7.55)(85.55) + (30.15)(99.55)}{100}$$



RR= 66.5 kips

$\Sigma M_o = 0 \quad (66.5)(44.45) = (30.15)(44) + (7.55)(30) + (S_p)(10.958)$

$S_p = -128.0$ kips

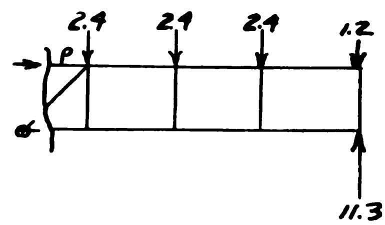
IMPACT STRESS (19.35%)(128.0)= 24.8 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES

RR= 11.3 kips $\quad \Sigma M_o = 0 \; (S_p)(10.958) + (7.2)(22.22) = (10.1)(44.45)$

f_1= 1.2 kips $\quad S_p$= 26.4 kips

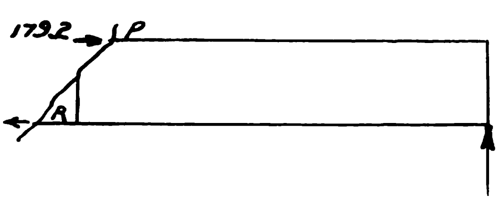
f_2= 2.4 kips



TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "P"

128.0 + 24.8 + 26.4= 179.2 kips compression

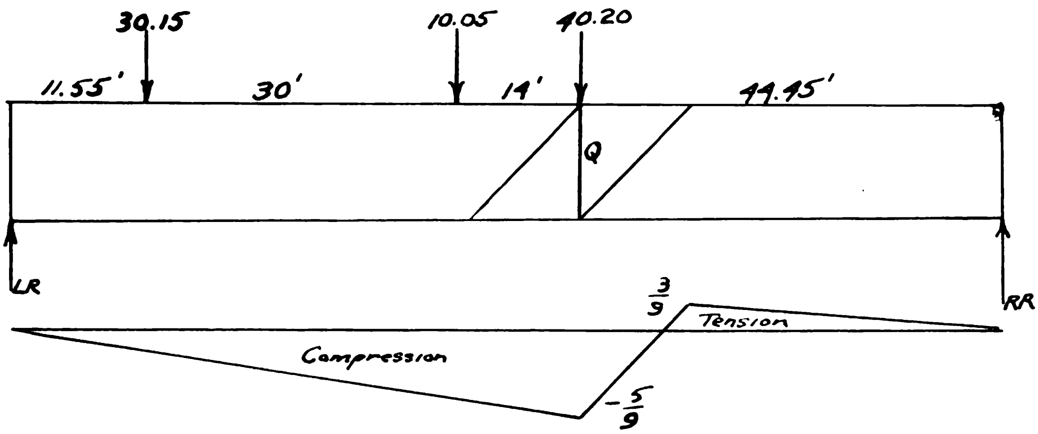
TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "R"



$\Sigma F_h = 0$

S_r= 179.2 kips tension

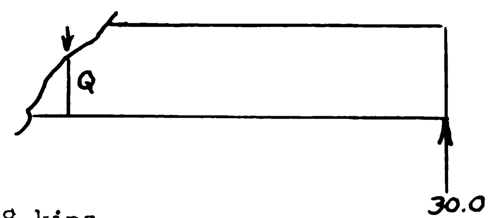
MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "Q"



INFLUENCE LINE FOR STRESS IN "Q"

Loaded length 62.5'

LIVE LOAD STRESS IN "Q" DUE TO TRUCK TRAIN LOADING



RR= 30.0 kips

$\Sigma F_v = 0$

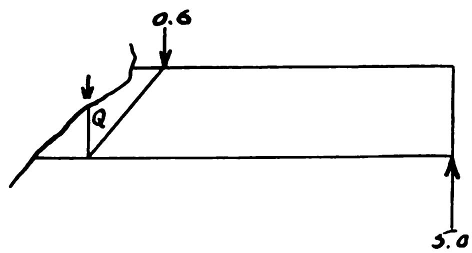
$S_q = 30.0$ kips

IMPACT STRESS (19.35%)(30)= 5.8 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "Q"

P(sidewalk)= 93.9#/sq. ft. w(railing)= 100#/ft.

w(total)= 256#/ft. f= 0.6 kip



RR= 5.0 kips

$\Sigma Fv = 0$ $S_q = 5.0 - 0.6$

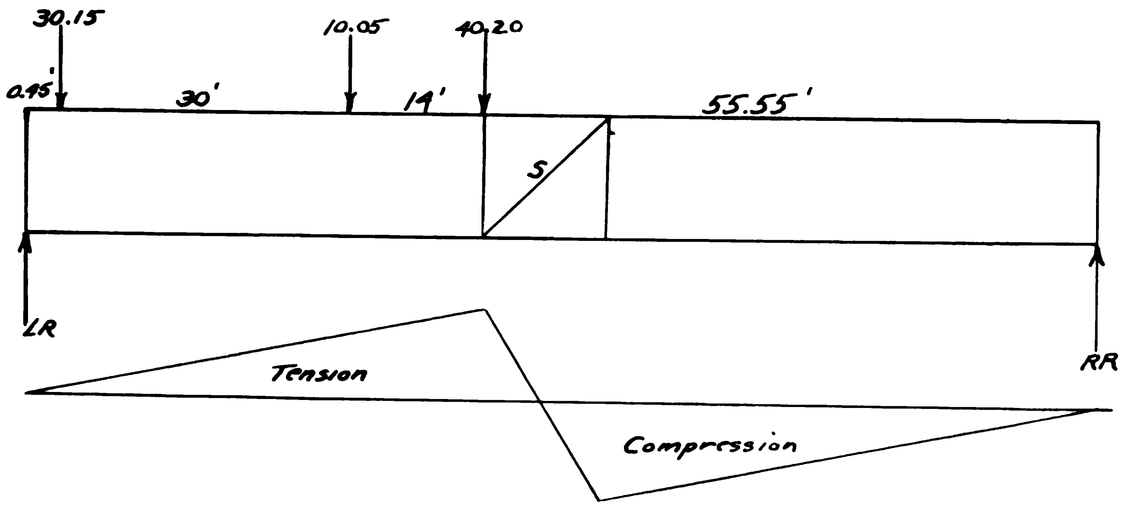
$S_q = 4.4$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN "Q"

30.0 + 5.8 + 4.4= 40.2 kips compression

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "S"

(illustration con't on next page)

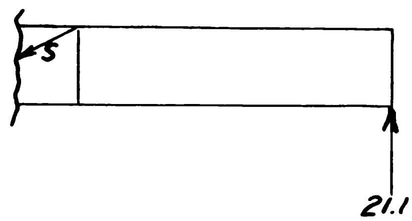


INFLUENCE LINE FOR STRESS IN "S"

Loaded Length- $100/2 = 50'$

LIVE LOAD STRESS DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(30.15)(0.45) + (10.05)(30.45) + (40.20)(44.45)}{100} = 21.1 \text{ kips}$$



$\Sigma F_v = 0 \qquad V_s = 21.1$

$$S_s = (21.1)\frac{(15.600)}{(10.958)} = 30.1 \text{ kips}$$

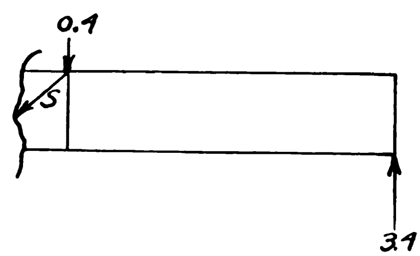
IMPACT STRESS $\quad (19.35\%)(30.1) = 5.8$ kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "S"

$$P(\text{sidewalk}) = \left(40 + \frac{3000}{(50)}\right)\left(\frac{55 - 5/3}{(50)}\right) = 107\#/\text{sq. ft.}$$

Use $P = 100\#/\text{sq. ft.}$ $\quad w(\text{railing}) = 100\#/\text{ft.}$

$w(\text{total}) = (100)(5/3) + 100 = 267\#/\text{ft.}$



$$RR = \frac{(.267)(50)}{(100)}(25) = 3.4 \text{ kips}$$

$$f = (.267)\frac{(11.104)}{(2)}(1/4) = 0.4 \text{ kips}$$

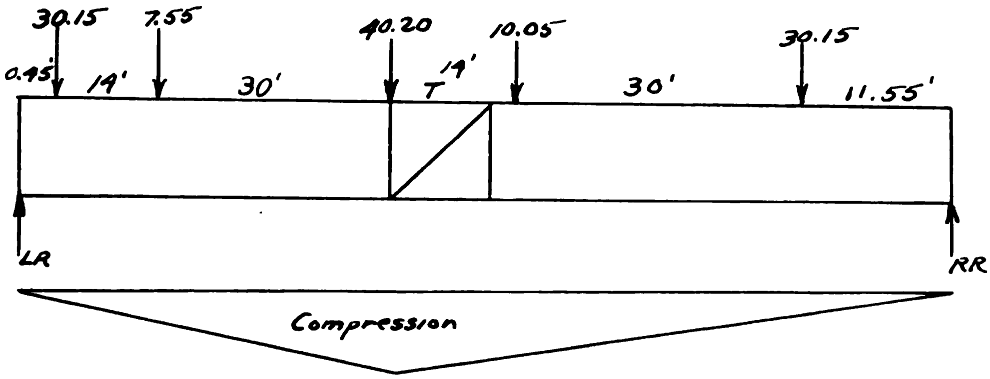
$\Sigma F_v = 0 \qquad V_s = 3.4 - 0.4 = 3.0 \qquad S_s = (3.0)\dfrac{(15.600)}{(10.958)} = 5.3$ kips

TOTAL LIVE LOAD AND IMPACT STRESSES IN MEMBER "S"

$30.1 + 5.8 + 4.3 = 40.2$ kips tension or compression

Since a reversal of stress takes place in one passage of the load, the design stress must be increased by 50%. (con't) (con't) (150%)(40.2)= 60.3 kips tension or compression

MAXIMUM LIVE LOAD AND IMPACT STRESSES IN MEMBER "T"

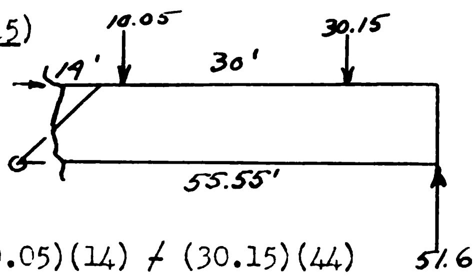


INFLUENCE LINE FOR STRESS IN "T"

LIVE LOAD STRESS IN MEMBER "T" DUE TO TRUCK TRAIN LOADING

$$RR = \frac{(30.15)(0.45) + (7.55)(14.45) + (40.20)(44.45) +}{100}$$

$$\frac{(10.05)(58.45) + (30.15)(88.45)}{100}$$



RR= 51.6 kips

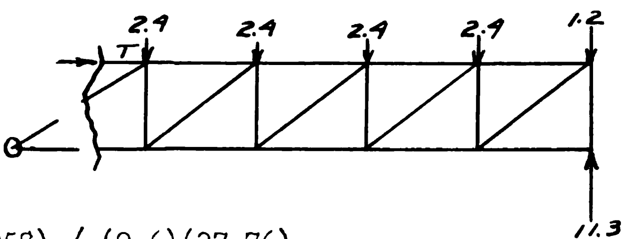
$\Sigma M_o = 0$

$(51.6)(55.55) = (S_t)(10.958) + (10.05)(14) + (30.15)(44)$

$S_t = 127.4$ kips

IMPACT STRESS (19.35%)(127.4)= 24.7 kips

SIDEWALK AND RAILING LIVE LOAD STRESSES IN "T"



RR= 11.3 kips

$f_1 = 2.4$ kips

$f_2 = 1.2$ kips $\Sigma M_o = 0$

$(10.1)(55.55) = (S_t)(10.958) + (9.6)(27.76)$

$S_t = 26.9$ kips

TOTAL LIVE LOAD AND IMPACT STRESS IN MEMBER "T"

127.4 + 24.7 + 26.9= 179.0 kips compression

TOTAL STRESSES IN TRUSS MEMBERS IN KIPS

MEMBER	KIND OF STRESS	DEAD LOAD STRESS	LIVE LOAD AND IMPACT STRESS	TOTAL STRESS
A	Compression	196.9	98.0	294.9
B	None	0.0	0.0	0.0
C	Tension	240.7	115.6	356.3
D	Compression	171.5	82.4	253.9
E	Compression	165.3	81.0	246.3
F	Tension	171.5	82.4	253.9
G	Tension	180.3	95.1	275.4
H	Compression	299.9	137.8	437.7
I	Compression	123.1	66.8	189.9
J	Tension	299.9	137.8	437.7
K	Tension	120.2	75.9	196.1
L	Compression	385.5	169.0	554.5
M	Compression	80.9	53.3	134.2
N	Tension	385.5	169.0	554.5
O	Tension	60.1	57.2	117.3
P	Compression	428.3	179.2	607.5
Q	Compression	38.7	40.2	78.9
R	Tension	428.3	179.2	607.5
S	Tension or Compression	0.0	60.3	60.3
T	Compression	428.3	179.0	607.3

UNIT STRESSES IN LOWER CHORD MEMBERS IN OUTER THREE PANELS

(Members "B", "F", and "J")

NET AREA OF SECTION

2 plates 18" x $\frac{1}{2}$" - (2)(1)($\frac{1}{2}$)= 16.00sq. in.

4 angles 6 x 4 x 7/16 - (1)$\left(\frac{7}{16}\right)$= 14.97sq. in.

TOTAL= 30.97sq.in.

ALLOWABLE STRESS IN TENSION= 18,000#/sq. in

TOTAL STRESSES IN MEMBERS "B", "F", AND "J"

Member B= 0.0

Member F= 253,900# tension

Member J= 437,700# tension

MAXIMUM UNIT STRESS IN MEMBER "B"

$s = \frac{0.0}{30.97} = 0.0$ #/sq. in.

MAXIMUM UNIT STRESS IN MEMBER "F"

$s_F = \frac{253,900}{30.97} = 8.200$#/sq. in tension

MAXIMUM UNIT STRESS IN MEMBER "J"

$s_J = \frac{437,700}{30.97} = 14,150$#/sq. in. tension

UNIT STRESSES IN LOWER CHORD MEMBERS IN MIDDLE THREE PANELS (Members "N" and "R")

NET AREA OF SECTION

2 plates 18" x $\frac{1}{2}$" - (2)(1)($\frac{1}{2}$)= 16.00 sq. in.

4 angles 6 x 4 x 3/4 - (1)(3/4)= 24.76 sq. in.

TOTAL= 40.76 sq. in.

ALLOWABLE STRESS IN TENSION= 18,000#/sq. in.

TOTAL STRESSES IN MEMBERS "N" AND "R"

Member "N" = 554,500# tension

Member "R"= 607,500# tension

MAXIMUM UNIT STRESS IN MEMBER "N"

$$S_n = \frac{554,500}{40.76} = 13,600\#/\text{sq. in. tension}$$

MAXIMUM UNIT STRESS IN MEMBER "R"

$$S_r = \frac{607,500}{40.76} = 14,900\#/\text{sq. in. tension}$$

UNIT STRESSES IN DIAGONAL MEMBERS IN OUTER TWO PANELS

(Members "C" and "G")

NET AREA OF SECTION FOR MEMBER "C"

1 plate 11" x 5/8" - (2)(1)(5/8)= 5.63 sq. in.

4 angles 5"x $3\frac{1}{2}$" x 3/4" - (1)(3/4)= 20.24 sq. in.

TOTAL= 25.87 sq. in.

ALLOWABLE STRESS IN TENSION= 18,000#/sq. in.

TOTAL STRESSES IN MEMBERS "C" AND "G"

Member "C"= 356,300# tension

Member "G"= 275,400# tension

MAXIMUM UNIT STRESS IN MEMBER "C"

$$S_c = \frac{356,300}{25.87} = 13,800\#/\text{sq. in. tension}$$

MAXIMUM UNIT STRESS IN MEMBER "G"

$$S_g = \frac{275,400}{20.14} = 13,700\#/\text{sq. in. tension}$$

UNIT STRESSES IN DIAGONAL MEMBERS IN CENTER FIVE PANELS

(Members "K", "O", and "S")

NET AREA OF SECTION

1 PLATE 11" x 3/8" - (2)(1)(3/8)= 3.38 sq. in.

4 angles 5" x $3\frac{1}{2}$" x 3/8" - (1)(3/8)= 10.68 sq. in.

TOTAL= 14.06 sq. in.

ALLOWABLE STRESS IN TENSION= 18,000#/sq. in.

TOTAL STRESSES IN MEMBERS

Member "K"= 196,100# tension

Member "O"= 117,300# tension

Member "S"= 60,300# tension

MAXIMUM UNIT STRESS IN MEMBER "K"

$S_k = \frac{196,100}{14.06} = 14,000\#/\text{sq. in.}$ tension

MAXIMUM UNIT STRESS IN MEMBER "O"

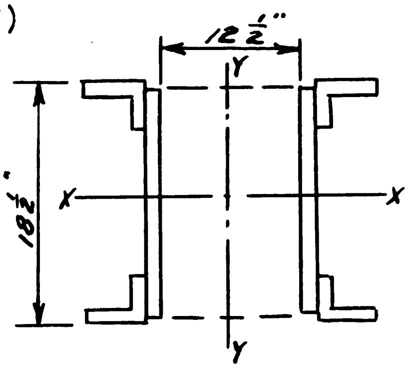
$S_o = \frac{117,300}{14.06} = 8,350\#/\text{sq. in.}$ tension

MAXIMUM UNIT STRESS IN MEMBER "S"

$S_s = \frac{60,300}{14.06} = 4,290\#/\text{sq. in.}$ tension

UNIT STRESSES IN UPPER CHORD MEMBERS IN OUTER THREE PANELS

(Members "D", "H", and "L")



GROSS AREA OF SECTION

2 plates 18" x ½"= 18.00 sq. in.

4 angles 6" x 4" x 5/8"= 23.44 sq. in.

TOTAL= 41.44 sq. in.

MOMENT OF INERTIA OF SECTION

I_x--2 plates= $(2)(1/12)(\frac{1}{2})(18^3) = 486$ in.^{4}

4 angles= $(4)\ 21.1 + (5.86)(9.25 - 2.03)^2 = 1306$ in.^{4}

Total $I_x = 1792$ in.^{4}

I_y--2 plates= $(2)(18)(\frac{1}{2}^3) + (18.00)(6.50^2) = 765$ in.^{4}

4 angles= $(4)\ 7.5 + (5.86)(6.75 + 1.03)^2 = 1447$ in.^{4}

Total $I_y = 2212$ in.^{4}

LEAST RADIUS OF GYRATION OF SECTION

$r = \sqrt{I/A} = \sqrt{\frac{1792}{41.44}} = 6.56"$

UNSUPPORTED LENGTH OF MEMBERS

$L = (11.1)(12) = 133"$

ALLOWABLE STRESS IN COMPRESSION FOR MEMBERS "D", "H", AND "L"

$$L/r = \frac{133}{6.56} = 20.3 \qquad S_c = 15,000 - 1/4(L/r)^2 = 15,000 - (1/4)(20.3^2)$$

$$S_c = 14,897\#/\text{sq. in.}$$

TOTAL STRESSES IN MEMBERS

Member "D" = 253,900# compression

Member "H" = 437,700# compression

Member "L" = 554,500# compression

MAXIMUM UNIT STRESS IN MEMBER "D"

$$S_d = \frac{253,900}{41.44} = 6,130\#/\text{sq. in. compression}$$

MAXIMUM UNIT STRESS IN MEMBER "H"

$$S_h = \frac{437,700}{49.44} = 10,560\#/\text{sq. in compression}$$

MAXIMUM UNIT STRESS IN MEMBER "L"

$$S_l = \frac{554,500}{41.44} = 13,380\#/\text{sq. in. compression}$$

UNIT STRESSES IN UPPER CHORD MEMBERS IN MIDDLE THREE PANELS

(Members "P" and "T")

GROSS AREA OF SECTION

2 plates 18" x ½" = 18.00 sq. in.

4 angles 6" x 4" x 3/4" = 27.76 sq. in.

TOTAL = 45.76 sq. in.

MOMENT OF INERTIA OF SECTION

I_x--2 plates $= (2)(1/12)(\frac{1}{2})(18^3) =$ 486 in.^{4}

4 angles $= (4)\ 24.5 + (6.94)(9.25 - 2.08)^2 =$ 1524 in.^{4}

TOTAL = 2010 in.^{4}

I_y--2 plates $= (2)(18)(\frac{1}{2}^3) + (18.00)(6.50^2) =$ 765 in.^{4}

4 angles $= (4)\ 8.7 + (6.94)(6.75 + 1.08)^2 =$ 1736 in.^{4}

TOTAL = 2501 in.^{4}

LEAST RADIUS OF GYRATION OF SECTION

$$r = \sqrt{I/A} = \sqrt{\frac{2010}{45.76}} = 6.61 \text{ in.}$$

UNSUPPORTED LENGTH OF MEMBERS

$$L = (11.1)(12) = 133''$$

ALLOWABLE STRESS IN COMPRESSION FOR MEMBERS "P" AND "T"

$$L/r = \frac{133}{6.61} = 20.1 \qquad S_c = 15,000 - (1/4)(20.1)^2 = 14,899\#/\text{sq. in.}$$

TOTAL STRESSES IN MEMBERS

Member "P" = 607,500# compression

Member "T" = 607,300# compression

MAXIMUM UNIT STRESS IN MEMBER "P"

$$S_p = \frac{607,500}{45.76} = 13,280\#/\text{sq. in. compression}$$

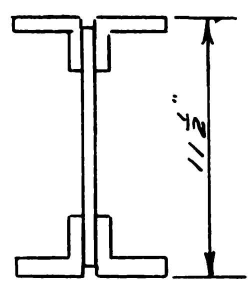
MAXIMUM UNIT STRESS IN MEMBER "T"

$$S_t = \frac{607,300}{45.76} = 13,275\#/\text{sq. in.}$$

<u>UNIT STRESSES IN VERTICAL MEMBERS AT END AND FIRST INTERMEDIATE</u>

<u>PANEL POINTS</u> (Members "A" and "E")

GROSS AREA OF SECTION OF MEMBER "A"



1 plate 11" x $\frac{1}{2}$" = 5.50 sq. in.

4 angles 6" x 4" x $\frac{1}{2}$" = 19.00 sq. in.

TOTAL = 24.50 sq. in.

MOMENT OF INERTIA OF SECTION OF MEMBER "A"

I_y -- Plate = $(1/12)(11)(\frac{1}{2})^3$ = 0.115 in.^{4}

4 angles = $(4)\ 17.4 + (4.75)(0.25 + 1.99)^2$ = 165 in.^{4}

TOTAL = 165.1 in.^{4}

LEAST RADIUS OF GYRATION OF SECTION "A"

$$r = \sqrt{\frac{165.1}{24.5}} = 2.59 \text{ in.}$$

UNSUPPORTED LENGTH OF MEMBER "A" AND "E"

$$L = (10.96)(12) = 131 \text{ in.}$$

ALLOWABLE STRESS IN COMPRESSION OF MEMBER "A"

$L/r = \frac{131}{2.59} = 50.6 \quad S_c = 15{,}000 -)1/4)(50.6)^2 = 14{,}360\#/\text{sq. in.}$

TOTAL STRESSES IN MEMBERS

Member "A" = 294,900# compression

Member "E" = 246,300 # compression

GROSS AREA OF SECTION OF MEMBER "E"

1 plate 11" x 3/8" =		4.13 sq. in.
4 angles 6" x 4" x 7.16" =		16.72 sq. in.
	TOTAL =	20.85 sq. in.

MOMENT OF INERTIA OF SECTION OF MEMBER "E"

I^y--Plate = $(1/12)(11)(3/8)^3 = 0.048 \text{ in.}^4$

4 angles = $(4)\ 15.5 + (4.18)(0.188 + 1.96)^2 = 139.2 \text{ in.}^4$

TOTAL = 139.3 in.^4

LEAST RADIUS OF GYRATION OF MEMBER "E"

$r = \sqrt{\frac{139.3}{20.85}} = 2.58 \text{ in.}$

ALLOWABLE STRESS IN COMPRESSION OF MEMBER "E"

$L/r = \frac{131}{2.58} = 50.8 \quad S_c = 15{,}000 - (1/4)(50.8)^2 = 14{,}355\#/\text{sq. in.}$

MAXIMUM UNIT STRESS IN MEMBER "A"

$S_a = \frac{294{,}900}{24.50} = 12{,}030\#/\text{sq. in. compression}$

MAXIMUM UNIT STRESS IN MEMBER "E"

$S_e = \frac{246{,}300}{24.50} = 11{,}800\#/\text{sq in. compression}$

<u>UNIT STRESSES IN VERTICAL MEMBERS AT MIDDLE SIX PANEL POINTS</u>

(Members "I", "M", and "Q")

GROSS AREA OF SECTION

1 plate 11" x 3/8" = 4.13 sq. in.

4 angles 5" x $3\frac{1}{2}$" x 3/8"= 12.20 sq. in.

TOTAL= 16.33 sq. in.

MOMENT OF INERTIAL OF SECTION

I_y--plate= $(1/12)(11)(3/8)^3$= 0.048 in.^{4}

4 angles= $(4)\ 7.8 + (3.05)(0.188 + 1.61)^2$= 70.8 in.^{4}

TOTAL= 70.9 in.^{4}

LEAST RADIUS OF GYRATION OF SECTION

$r = \sqrt{\frac{70.9}{16.33}} = 2.08$ in.

ALLOWABLE STRESS IN COMPRESSION OF MEMBERS "I", "M", AND "Q"

$L/r = \frac{131}{2.08} = 63.0 \qquad S_c = 15{,}000 - (1/4)(63.0)^2 = 14{,}010\#/\text{sq. in.}$

TOTAL STRESSES IN MEMBERS

MEMBER "I"= 189,900 # compression

Member "M"= 134,200# compression

Member "Q"= 78,900# compression

MAXIMUM UNIT STRESS IN MEMBER "I"

$S_i = \frac{189{,}900}{16.33} = 11{,}620\#/\text{sq. in.}$ compression

MAXIMUM UNIT STRESS IN MEMBER "M"

$S_m = \frac{134{,}200}{16.33} = 8{,}220\#/\text{sq. in.}$ compression

MAXIMUM UNIT STRESS IN MEMBER "Q"

$S_q = \frac{78{,}900}{16.33} = 4{,}840\#/\text{sq. in.}$ compression

UNIT COMPRESSIVE STRESS IN DIAGONAL MEMBERS IN CENTER PANEL

(Member "S")

GROSS AREA OF SECTION

1 plate 11" x 3/8"=	4.13 sq. in.
4 angles 5" x $3\frac{1}{2}$" x 3/8"=	12.20 sq. in.
TOTAL=	16.33 sq. in.

MOMENT OF INERTIA OF SECTION

Iy--plate = $(1/12)(11)(3/8)^3 =$ 0.05 in.^{4}

4 angles = $(4)\ 7.8 + (3.05)(1.80)^2 =$ $\underline{70.85}$ in.^{4}

TOTAL— 70.9 in.^{4}

LEAST RADIUS OF GYRATION OF SECTION

$$r = \sqrt{\frac{70.9}{16.33}} = 2.08 \text{ in.}$$

UNSUPPORTED LENGTH OF MEMBER

$(15.6)(12) = 187$ inches

ALLOWABLE STRESS IN COMPRESSION OF MEMBER "S"

$$L/r = \frac{187}{2.08} = 90.0$$

$S_c = 15{,}000 - (1/4)(90.0^2) = 12{,}975\#/\text{sq. in.}$

TOTAL STRESS IN COMPRESSION IN MEMBER "S"

S— 60,300# compression

MAXIMUM UNIT STRESS IN COMPRESSION IN MEMBER "S"

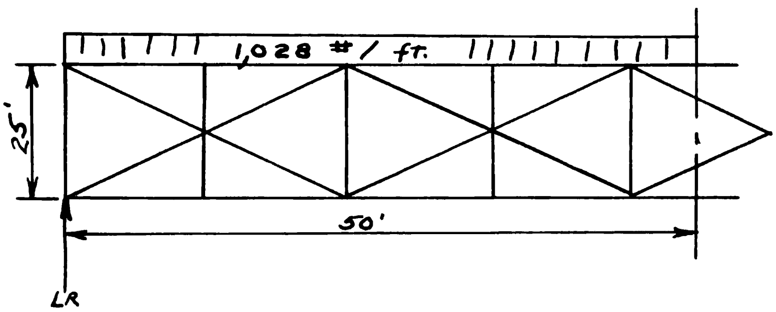
$$S_s = \frac{60{,}300}{16.33} = 3{,}690\#/\text{sq. in. compression}$$

MAXIMUM ALLOWABLE STRESSES AND MAXIMUM POSSIBLE STRESSES IN

IN TRUSS MEMBERS

MEMBER	KIND OF STRESS	ALLOWABLE STRESS IN LBS. PER SQ. IN.	POSSIBLE STRESS IN LBS. PER SQ. IN.
A	Compression	14,360	12,030
B	None	18,000	0
C	Tension	18,000	13,800
D	Compression	14,897	6,130
E	Compression	14,355	11,800
F	Tension	18,000	8,200
G	Tension	18,000	13,700
H	Compression	14,897	10,560
I	Compression	14,010	11,620
J	Tension	18,000	14,150
K	Tension	18,000	14,000
L	Compression	14,897	13,380
M	Compression	14,010	8,220
N	Tension	18,000	13,600
O	Tension	18,000	8,350
P	Compression	14,899	13,280
Q	Compression	14,010	4,840
R	Tension	18,000	14,900
S	Tension	18,000	4,290
S	Compression	12,975	3,690
T	Compression	14,899	13,275

MAXIMUM STRESSES IN LATERAL BRACING



Height of structure as seen in elevation= 19.5 ft.

Length of diagonals= 33.4 ft.

WIND FORCES AND LATERAL VIBRATION FORCES APPLIED TO STRUCTURE

w_1= 30#/sq. ft. W_1= (30)(19.5)(1)(1.5)= 878#/ft.

W_2= 150#/lin. ft. W= 878 + 150= 1028#/ft.

MAXIMUM STRESS IN A DIAGONAL

LR= (1028)(50)= 51,400# Max shear= V_{max}= 51,300#

S_{max}= $(51,400)\frac{(33.4)}{(25.0)}$= 68,700# tension

NET AREA PROVIDED IN A DIAGONAL

2 angles $4 \times 3\frac{1}{2} \times 3/8 - (2)(1)(3/8) = 4.59$ □ in.

ALLOWABLE TENSILE STRESS IN A DIAGONAL

S= 18,000#/sq. in.

MAXIMUM TENSILE STRESS IN A DIAGONAL

$S = \frac{68,700}{4.59} = 15,000$#/sq. in.

Since the shear in the center panels is less that that in the outside panels and the same area is provided in these panels, the maximum unit stresses in the bracing angles in the center panels are less than the above value.

NECESSARY DEPTH OF SLAB AND AMOUNT OF REINFORCING STEEL

$fc^1 = 2,500\#/sq. in.$

$fc = (0.4)(2,500) = 1,000\#/sq. in.$

$fs = 18,000\#/sq. in.$

MINIMUM EFFECTIVE DEPTH OF SLAB PROVIDED= $6\frac{1}{2}$ in.

AREA OF STEEL PROVIDED PER FOOT OF LENGTH

$\frac{1}{2}$" square rods spaced at 4"

$A = (0.25)(12/4) = 0.75$ sq. in.

MAXIMUM LIVE MOMENT IN SLAB

S= 4.583 ft, spacing of stringers

$B = 0.7\,S + 2 = (0.7)(4.583) + 2.00$

B= 5.2 ft., effective width of slab

Maximum wheel load= 12,000# $w = \frac{12,000}{5.2} = 2,310\#/ft.$

$M = 1/4\,wl = (1/4)(2310)(4.583) = 2,650$ ft. lb.

IMPACT MOMENT IN SLAB

$I = \frac{4.58 + 20}{(6)(4.58) + 20} = 51.7\%$ $(51.7\%)(2,650) = 1,370$ ft. lb.

MAXIMUM DEAD MOMENT IN SLAB

Use average total depth of slab as 9"

$w = (9/12)(150) = 112.5\#/ft.$ $M = (1/10)(112.5)(4.583)^2 = 236$ ft. lb.

TOTAL MAXIMUM MOMENT IN SLAB

$2,650 + 1,370 + 236 = 4,256$ ft. lb.

MINIMUM EFFECTIVE DEPTH OF SLAB R= 173

$d = \sqrt{M/Rb} = \sqrt{\frac{(4,256)(12)}{(173)(12)}} = 4.95$ in.

STEEL AREA NECESSARY PER FOOT OF SLAB

$p = .0111$ $A = pbd = (.0111)(12)(4.95) = 0.66$ sq. in.

MAXIMUM FIBRE STRESSES IN STRINGERS

9 Stringers--12" x 5" x 35# Rolled I-Beams

Spacing= 8 @ 4'7" CL-CL= 36'8"

Unsupported length= 11.1' CL-CL of floor Beams

MAXIMUM LIVE MOMENT FOR ONE LANE

$M = (12{,}000)(11.1/2) = 66{,}600$ ft. lb.

MAXIMUM LIVE MOMENT FOR ONE STRINGER

C(reinforcec concrete slab) = 1.0

$N = \frac{\text{Width of traffic lane}}{\text{Spacing of stringers}} = \frac{10.0}{4\ 7/12} = 2.18$

$M' = C\ M/N = 1.0\frac{(66{,}600)}{(2.18)} = 30{,}600$ ft. lb.

IMPACT MOMENT FOR ONESTRINGER

$I = \frac{L + 20}{6L + 20} = \frac{11.1 + 20}{66.6 + 20} = 36.0\ \%$

$M_i = (36.0\%)(30{,}600) = 11{,}000$ ft. lb.

DEAD MOMENT FOR ONE LANE

Using average depth of slab as 9 inches

w(slab) = (9/12)(10)(1)(150) = 1125#/ft.

w(stringers) = $\frac{(9)(35)(10)}{(36\ 2/3)} = 86$#/ft.

TOTAL w = 1211#/ft.

$M = 1/8\ wl^2 = 1/8(1211)(11.1)^2 = 18{,}630$ ft. lb.

DEAD MOMENT FOR ONE STRINGER

$M' = (1.0)\frac{(18{,}630)}{(2.18)} = 8{,}540$ ft. lb.

TOTAL MAXIMUM MOMENT IN ONE STRINGER

$M = 30{,}600 + 11{,}000 + 8{,}540 = 50{,}140$ ft. lb.

ALLOWABLE TENSILE STRESS IN BOTTOM OF STRINGER

$S_t = 18{,}000$#/sq. in.

ALLOWABLE COMPRESSIVE STRESS IN TOP OF STRINGER

$L/b = \frac{(11.1)(12)}{(5.078)} = 26.25$ $S_c = 18{,}000 - 5(L/b)^2 = 14{,}550$#/sq.in.

MAXIMUM COMPRESSIVE AND TENSILE STRESS IN STRINGER

$$S = \frac{Mc}{I} = \frac{(50,140)(12)(4.88)}{(227)} = 12,920\#/\text{sq. in. tension and compression}$$

MAXIMUM SHEARING STRESS IN STRINGERS

MAXIMUM LIVE SHEAR FOR ONE LANE

$$V = 24,000\#$$

MAXIMUM LIVE SHEAR IN ONE STRINGER

$$C = 1.0 \qquad N = 2.18$$

$$V^1 = (1.0)\frac{(24,000)}{(2.18)} = 11,000\#$$

IMPACT SHEAR IN ONE STRINGER

$$(36.0\%)(11,000) = 3,960\#$$

DEAD SHEAR FOR ONE LANE

$$w(\text{total}) = 1211\#/\text{ft.}$$

$$V^1 = (1211)\frac{(11.1)}{(2)} = 3080\#$$

TOTAL MAXIMUM SHEAR IN ONE STRINGER

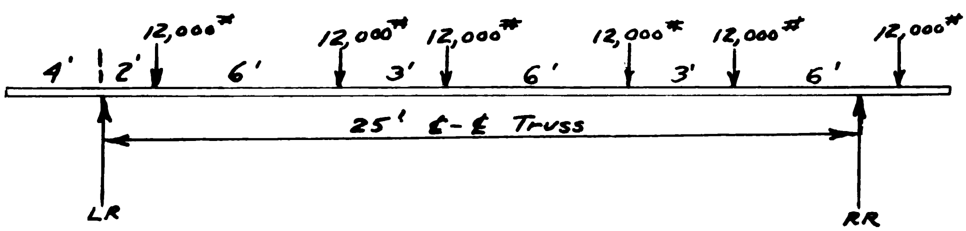
$$V = 11,000 + 3,960 + 3,080 = 18,040\#$$

ALLOWABLE UNIT SHEARING STRESS IN WEB

$$\frac{\text{Clear distance}}{\text{Thickness}} = \frac{9.75}{.428} = 22.8 \text{ which is less than } 60$$

$$v = \frac{18,040}{(12.00)(0.428)} = 3,520\#/\text{sq. in.}$$

MAXIMUM LIVE LOAD AND IMPACT MOMENT IN FLOOR BEAMS



MAXIMUM LIVE MOMENT IN FLOOR BEAM

$$M_1 = 0 \qquad (12,000)(2 + 8 + 11 + 17 + 20 + 26) = (RR)(25)$$

$RR = \frac{(12,000)(84)}{(25)} = 40,300\#$

$LR = (6)(12,000) - 40,300 = 31,700\#$

$M_{max} = (31,700)(11) - (12,000)(3 + 9)$

$M_{max} = 204,700$ ft. lb.

Since three lanes are loaded to obtain this maximum bending moment, only 90% of the total moment is used in the design.

$M = (90\%)(204,700) = 185,000$ ft. lb.

IMPACT MOMENT IN INTERMEDIATE FLOOR BEAMS

$I = \frac{L + 20}{6L + 20} = \frac{25 + 20}{150 + 20} = 26.5\%$

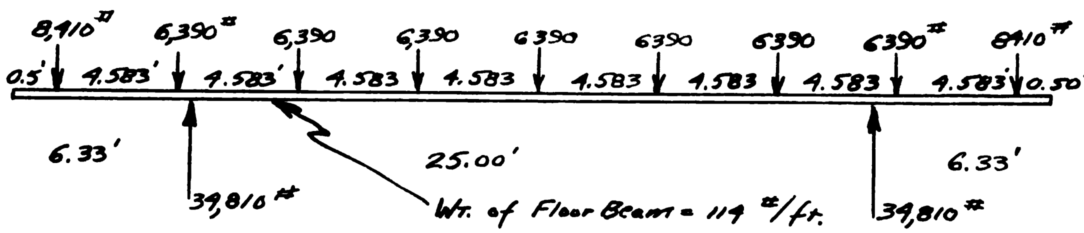
$M = (26.5\%)(184,000)$ $M = 48,700$ ft. lb.

IMPACT MOMENT IN END FLOOR BEAMS

$I = (2)(26.5\%) = 53.0\%$

$M = (53.0\%)(184,000) = 97,500$ ft. lb.

MAXIMUM DEAD MOMENT IN INTERMEDIATE FLOOR BEAMS



Assume weight of slab as uniformly distributed to the stringers.

Assume weight of walks, railing section, and railing posts as being carried entirely by the outer stringers.

WEIGHT APPLIED TO EACH FLOOR BEAM THROUGH EACH INSIDE STRINGER

Weight of slab--$\frac{(4320)}{(8)}(11.1) = 6,000$ lb.

Weight of stringer--$(35)(11.1) = \underline{390}$ lb.

TOTAL= 6,390 lb.

WEIGHT APPLIED TO EACH FLOOR BEAM THROUGH EACH OUTSIDE STRINGER

Weight of slab--$\frac{6000}{2}$= 3,000 lb.

Weight of stringer--(35)(11.1)= 390 lb.

Weight of sidewalk--(335)(11.1)= 3,720 lb.

Weight of railing section--(44)(11.1)= 490 lb.

Weight of railing posts-- 810 lb.

TOTAL= 8,410 lb.

TOTAL WEIGHT OF FLOOR BEAM

$\frac{4,287}{37.67}$= 114#/ft.

MAXIMUM DEAD MOMENT IN INTERMEDIATE FLOOR BEAMS

LR= (3.5)(6,930) + 8,410 + $\frac{(4,287)}{(2)}$= 34,810#

M= 436,000 - 175,600 - 154,200 - 20,200

M= 86,000 ft. lb.

MAXIMUM FIBRE STRESSES IN INTERMEDIATE FLOOR BEAMS

TOTAL MAXIMUM MOMENT IN INTERMEDIATE FLOOR BEAM

Live moment	184,000 ft. lb.
Impact moment	48,700 ft. lb.
Dead moment	86,000 ft. lb.
TOTAL=	318,700 ft. lb.

ALLOWABLE TENSILE STRESS IN BOTTOM OF INTERMEDIATE FLOOR BEAM

S_t= 18,000#/sq. in.

ALLOWABLE COMPRESSIVE STRESS IN TOP OF INTERMEDIATE FLOOR BEAM

L/b= $\frac{(25)(12)}{(12.00)}$= 25 S_c= 18,000 - (5)(L/b)^2

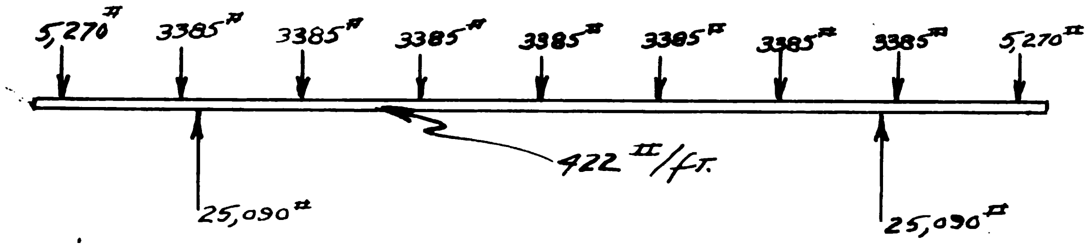
S_c= 14,885#/sq. in.

MAXIMUM COMPRESSIVE AND TENSILE STRESS IN INTERMEDIATE FLOOR BEAM

S= Mc/I= $\frac{(318,700)(12)(10.44)}{(2987.3)}$

S= 13,370 #/sq. in. tension or compression

MAXIMUM DEAD MOMENT IN END FLOOR BEAMS



WEIGHT APPLIED TO EACH END FLOOR BEAM THROUGH EACH INSIDE STRINGER

Weight of slab--$\frac{(4320)}{(8)}\frac{(11.1)}{(2)}$=	3,000 lb.
Weight of stringer--(35)$\frac{(11.1)}{(2)}$=	195 lb.
Weight of expansion angles--$\frac{(1525)}{(8)}$=	190 lb.
TOTAL =	3,385 lb.

WEIGHT APPLIED TO EACH END FLOOR BEAM THROUGH EACH OUTSIDE STRINGER

Weight of slab--$\frac{3000}{2}$ =	1,500 lb.
Weight of stringer--(35)(11.1/2)=	195 lb.
Weight of sidewalk--(335)(11.1/2)=	1,860 lb.
Weight of railing section--(44)(11.1/2)=	245 lb.
Weight of railing post--	1,470 lb.
TOTAL=	5,270 lb.

TOTAL WEIGHT OF END FLOOR BEAM AND CONCRETE AROUND IT

$\frac{15,915}{37.67}$= 422#/ft.

MAXIMUM DEAD MOMENT IN END FLOOR BEAMS

LR= (3.5)(3,385) + 5,270 + $\frac{15,915}{2}$ = 25,090#

M= 314,000 - 93,100 - 96,700 - 75,000 M= 49,200 ft. lbs.

MAXIMUM FIBRE STRESSES IN END FLOOR BEAMS

TOTAL MAXIMUM MOMENT IN END FLOOR BEAM

Live moment =	184,000 ft. lb.
Impact moment =	97,500 ft. lb.
Dead moment =	49,200 ft. lb.
TOTAL =	330,700 ft. lb.

ALLOWABLE TENSILE STRESS IN BOTTOM OF END FLOOR BEAM

$S_t = 18{,}000\#/\text{sq. in.}$

ALLOWABLE COMPRESSIVE STRESS IN TOP OF END FLOOR BEAM

$$L/b = \frac{(25)(12)}{(12.042)} = 24.9$$

$$S_c = 18{,}000 - (5)(24.9)^2 = 14{,}900\#/\text{sq. in.}$$

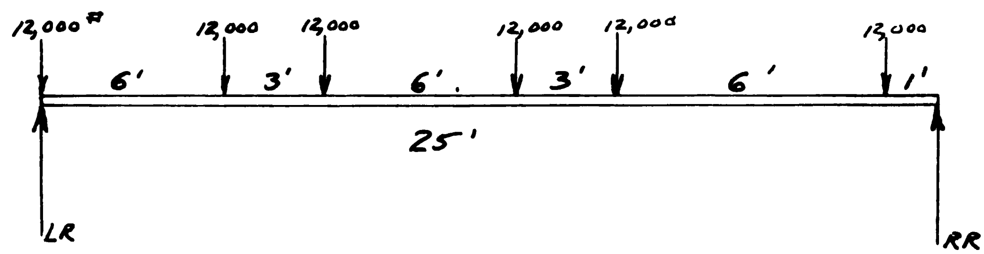
MAXIMUM COMPRESSIVE AND TENSILE STRESS IN END FLOOR BEAM

$$S = \frac{Mc}{I} = \frac{(330{,}700)(12)(10.44)}{(3315.0)}$$

$S = 12{,}520\#/\text{sq. in.}$ tension or **compression**

MAXIMUM SHEARING STRESS IN INTERMEDIATE FLOOR BEAMS

MAXIMUM LIVE SHEAR IN ONE FLOOR BEAM



$$LR = (12{,}000)\left(\frac{25 + 19 + 16 + 10 + 7 + 1}{25}\right) = (12{,}000)\frac{(78)}{(25)}$$

$LR = 37{,}400\#$ $\quad V_{max} = 37{,}400\#$

Since three lanes are loaded to obtain this value, only 90% of the maximum need be used in design.

$V = (90\%)(37{,}400) = 33{,}650\#$

IMPACT SHEAR IN ONE INTERMEDIATE FLOOR BEAM

(26.5%)(33,650)= 8.920#

MAXIMUM DEAD SHEAR IN ONE INTERMEDIATE FLOOR BEAM

V= 24,810 - 8.410 - 6,390 - (114)(6.33)

V= 19,290#

TOTAL MAXIMUM SHEAR IN INTERMEDIATE FLOOR BEAM

33,650 + 8,920 + 19,290= 61,860#

ALLOWABLE UNIT SHEARING STRESS IN WEB OF BEAM

$\frac{\text{Clear distance}}{\text{Thickness}} = \frac{20.875}{0.468} = 44.7$ which is less than 60

v= 12,000#/sq. in.

MAXIMUM UNIT SHEARING STRESS IN WEB OF INTERMEDIATE FLOOR BEAM

$v = \frac{(61,860)}{(24)(0.468)} = 5,500$#/sq. in.

MAXIMUM SHEARING STRESS IN END FLOOR BEAM

MAXIMUM LIVE SHEAR IN END FLOOR BEAM

V= 33,650#

IMPACT SHEAR IN END FLOOR BEAM

(53.0%)(33,650)= 17,840#

MAXIMUM DEAD SHEAR IN END FLOOR BEAM

V= 25,090 - 5,270 - 3,385 - (422)(6.33)

V= 13,760#

TOTAL MAXIMUM SHEAR IN END FLOOR BEAM

33,650 + 17,850 + 13,760= 65,250#

ALLOWABLE UNIT SHEARING STRESS IN WEB OF BEAM

$\frac{\text{Clear distance}}{\text{Thickness}} = \frac{20.875}{0.510} = 41.0$ which is less than 60

v= 12,000#/sq. in.

MAXIMUM UNIT SHEARING STRESS IN WEB OF END FLOOR BEAM

$$v = \frac{65,250}{(24.16)(.510)} = 5,300\#/\text{sq. in.}$$

MAXIMUM FIBRE STRESSES DUE TO BENDING IN

FLOOR BEAMS AND STRINGERS IN POUNDS PER SQUARE INCH

	ALLOWABLE UNIT STRESS IN COMPRESSION	MAXIMUM UNIT STRESS IN COMPRESSION	ALLOWABLE UNIT STRESS IN TENSION	MAXIMUM UNIT STRESS IN TENSION
STRINGERS	14,550	12,920	18,000	12,920
INTERMEDIATE FLOOR BEAMS	14,885	13,370	18,000	13,370
END FLOOR BEAMS	14,900	12,520	18,000	12,520

MAXIMUM SHEARING STRESSES IN WEBS OF FLOOR BEAMS

AND STRINGERS IN POUNDS PER SQUARE INCH

	ALLOWABLE UNIT SHEARING STRESS	MAXIMUM UNIT SHEARING STRESS
STRINGERS	12,000	3,520
INTERMEDIATE FLOOR BEAMS	12,000	5,500
END FLOOR BEAMS	12,000	5,300

CONCRETE BEARING AREA

Size of lead bearing plate--22" x 23"

MAXIMUM DEAD LOAD REACTION

R= 196,900#

MAXIMUM LIVE LOAD AND IMPACT REACTION

R= 98,000#

TOTAL MAXIMUM REACTION

R= 196,900 + 98,000= 294,900#

ALLOWABLE BEARING STRESS IN CONCRETE

$f_c' = 2{,}500\#/\text{sq. in.}$

$s = (0.25)(2{,}500) = 625\#/\text{sq. in.}$

MINIMUM CONCRETE BEARING AREA NECESSARY

$\frac{294{,}900}{625} = 472 \text{ sq. in.}$

CONCRETE BEARING AREA PROVIDED

$22 \times 23 = 506 \text{ sq. in.}$