ANALYSIS OF THE SUPERSTRUCTURE OF HIGHWAY BRIDGE

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

Roy L. Colby Jr.

1941

Analysis of the Superstructure

of Highway Bridge

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

Roy Lynn Colby Jr.

Candidate for the Degree of

Bachelor of Science

March, 1941

THESIS

Cop. 1

Analysis of the Super Structure of a Bridge located in the west part of the business section in Owosso, Michigan, on M 21.

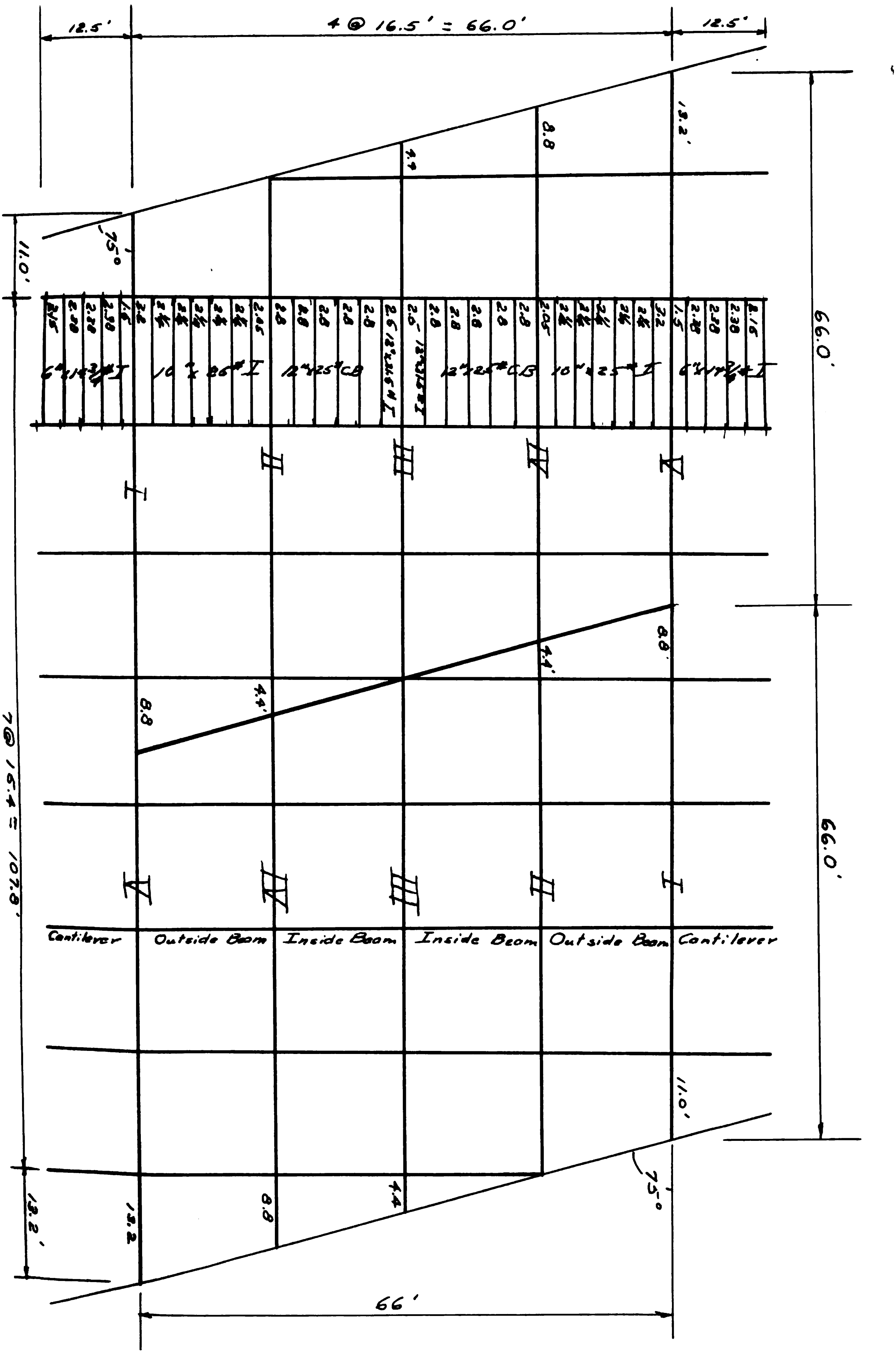
The bridge has two spans at 66 ft. supported on masonary abutments and a masonary pier at the center. It is on a skew of 75 degrees. There are 5 plate girder at 16.5 ft. center to center and built up beams at 90 degrees with the girder. The beams are connected on directly opposites sides of the girders. The stringers, which are I Beams, along with the girders support the 6 in. reinforced concrete deck. The clear distance from curb to curb is 64 ft. There is a 12 ft. sidewalk of 5 in. reinforced concrete slab supported by a cantilever beam on each side of the roadway. The bridge has just recently been redecked and possible other changes took place at that time.

Assumptions:

Girders---------------Freely supported
Beams------ -------- -Continuous
Stringers-------------Freely supported
Concrete Slab---------Continuous

The numbers in parenthesis used through-out the thesis correspond to the specification numbers in the book of specifications used for Design of Highway Bridges adopted by the Michigan State Highway Department in January, 1936.

The fiber stress in the specification is 18000 #/sq. in., but in checking old bridges the Michigan State Highway Department use 24 000 in determining safe loading.



12.5'
4 @ 16.5' = 66.0'
12.5'
13.2'
8.8
4.4
75°
11.0'
66.0'
66.0'
6" x 12 3/4 # I
10" x 26 # I
12" x 25 # CB
12" x 25 # CB
10" x 25 # I
I
II
III
IV
V
8.8'
4.4'
8.8
4.4'
V
IV
III
II
I
Cantilever
Outside Beam
Inside Beam
Inside Beam
Outside Beam
Cantilever
11.0'
75°
13.2'
13.2
8.8
4.4
66'
7 @ 15.4 = 107.8'

INDEX

	pages
Concrete Slab	1 to 15
Stringers	16 to 21
Beams	22 to 43
Cantilever Beam	44 to 53
Girders	54 to 91

CONCRETE SLAB

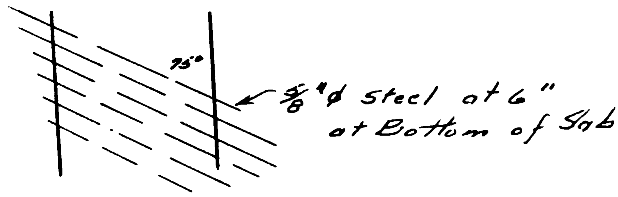
Sidewalk Concrete Slab

Concrete Slab between 6 in. @ 14.75 # I spaced at 2 ft. 4½ in.
Flange wd. 3.45 in.

Clear Dis. $2.38 - \frac{3.45}{12} = 2.09$ ft.

(55) $2.09 + \frac{5}{12} = 2.5$ ft.

2.38 is less than 2.5; therefore span lenght is 2.38 ft.



Dead Load 150 #/ft^3 concrete
5/12 150 = 62.5 #/ft.

(34) Live Load 100 #/sq. ft.

Total lbs. per ft. 100 + 62.5 = 162.5 #/ft.

Total Bending Moment

$$Max\ BM = \frac{w\ l^2}{10} = \frac{162.5\ 2.38^2}{10} = 92.3 \text{ ft. lbs.}$$

Total Shear

$$Max\ V = \frac{w\ l}{2} = \frac{162.5\ 2.38}{2} = 193.5\ \#$$

Check d

$$d = \sqrt{\frac{BM}{K\ b}} = \sqrt{\frac{92.3\ 12}{173\ 12}} = 0.73 \text{ in.}$$

supplied d at center of span
$d = 5 - 1\frac{1}{2} = 3\frac{1}{2}$ in.
supplied d at supports
$d = 1\frac{1}{2}$ in.

Steel-5/8 in. ϕ spaced at 6 in. on 75 degree angle
$p = 0.0111$ steel ration needed

$$A_s = \frac{.6136}{\sin 75} = 0.635 \text{ sq. in.}$$

at supports $p = \frac{.635}{1.5\ 12}$ .0353 supplied
needed .0111

at center of span $p = \frac{.635}{3.5\ 12}$ .0151 supplied
needed .0111

Sidewalk Concrete Slab

Concrete Slab between 6 in. @ 14.74 # I spaced at 2 ft. 4½ in.

Shear

$$v = \frac{V}{7/8\ b\ d} = \frac{193.5}{7/8\ 12\ 1.5} = 12.26\ \#/\text{sq. in.}$$

allowable 50

Bond

$$u = \frac{V}{7/8\ o\ d} = \frac{193.5}{7/8\ 4.07\ 1.5} = 36.2\ \#/\text{sq. in.}$$

allowable 125

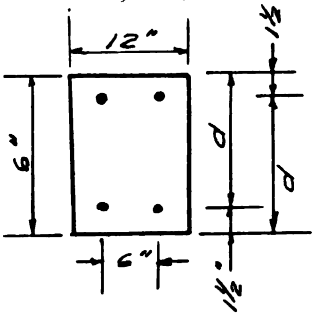
CONCRETE SLAB OF ROADWAY

25000 # Concrete

$f_s = 18000$ #/sq. in.

$f_c = 1000$ #/sq. in.

n = 12
K = 175 used in formulae $d = \sqrt{\frac{M}{K\ b}}$
j = 0.871 say 7/8
p = 0.0111 ratio of steel to concrete cross section



Dis. d supplied equals 6 − 1.5 = 4.5 in.

Steel----5/8 in. Φ spaced at 6 in. on angle of 75 degrees
Top & Bottom

$$A_s = \frac{0.6136}{\sin 75} = 0.635 \text{ sq. in.}$$

$$p = \frac{A_s}{b\ d} = \frac{0.635}{4.5\ 12} = 0.01174 \text{ supplied}$$
needed 0.011

Dead Load of Slab
6 in. Concrete 150 #/ft.³
6/12 150 = 75 #/ft.

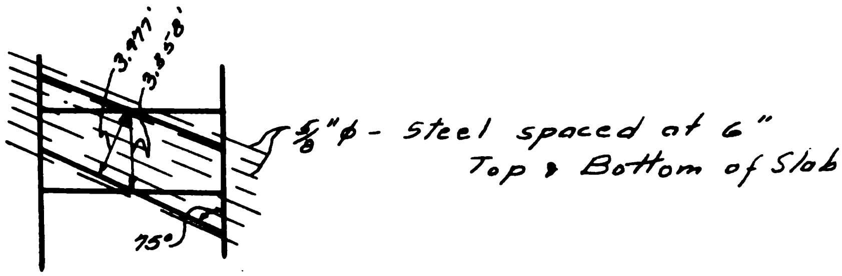
Wearing Surface 25 #/ft.²
25 1 = 25 #/ft.
w = 100 #/ft.

Concrete Slab between the Second Girder and the first 10 in. @ 25 # Stringer

10 in. @ 25 # IBM -flange wd. of 4.66 in.
center to center dis. 2ft. - 0 1/2 in. or 2.04 ft.
wd. of flange of girder 1.19 ft.

(35) $1.25 + .5 = 1.75$ ft.

1.75 is less than 2.04 ft.; therefore Span Length is 1.75 ft.



(41) Effective Width

eff. wd. $= 0.7\ s + 2$ on 75 degree angle
s = center to center of supports on 75 degree angle

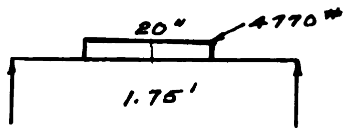
$$s = \frac{2.04}{\sin 75} = 2.11 \text{ ft.}$$

eff. wd. $= 7\ 2.11 + 2 = 3.477$ ft. on 75 degree angle

eff. wd. $= 3.477 \sin 75 = 3.358$ on 90 degree angle

Max Live Load Bending Moment H20 Loading
wheel load = 16000 # 20 in. wd.

Load on a ft. of slab $= \frac{16000}{3.358} = 4770$ #



Continuity = 0.8

$$\text{LLBM} = .8(2385\ .875 - \frac{4770}{2}\ \frac{5}{12}) = 872.8 \text{ ft. lbs.}$$

$$\text{(37) Impack} = \frac{L+20}{6\ L+20} = \frac{1.75+20}{6\ 1.75+20} = 71.4\ \%$$

LLBM Imp. $= 872.8\ 171.4\ \% = 1495$ ft. lbs.

Max. Dead Load Bending Moment $w = 100$ #/ft.

$$\text{DLBM} = \frac{w\ l^2}{10} = \frac{100\ 1.75^2}{10} = 30.7 \text{ ft. lbs.}$$

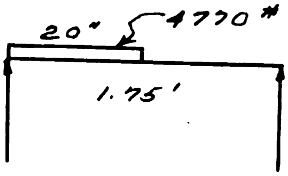
Total BM $= 30.7 + 1495 = 15257$ ft. lbs.

Concrete Slab between the Second Girder and the first 10 in. @ 25 # Stringer

Check d

$$d = \sqrt{\frac{BM}{K\ b}} = \sqrt{\frac{15287\ 12}{175\ 12}} = [illegible].97 \text{ in. supplied } 4.5$$

H20 Live Load Shear H20 Loading



$$LLV = \frac{4770}{1.75}\left(1.75 - \frac{20}{2\ 12}\right) = 2500\ \#$$

$$LLV + Imp. = 2500\ 171.4\% = 4280\ \#$$

Dead Load Shear

$$DLV = \frac{w\ l}{2} = \frac{100\ 1.75}{2} = 87.5\ \#$$

$$\text{Total Shear} = 4280 + 87.5 = 4367.5\ \#$$

Shear $$v = \frac{V}{7/8\ d\ b} = \frac{4367.5}{7/8\ 4.5\ 12} = 92.3\ \#/\text{sq. in.}$$ allowable 75

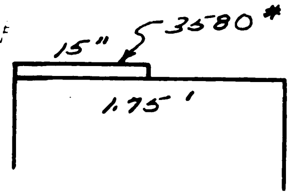
Bond $$u = \frac{V}{7/8\ o\ d} = \frac{4367.5}{7/8\ 4.07\ 4.5} = 272.3\ \#/\text{sq. in.}$$ allowable 250

H15 Live Load Shear H20 Loading

Wheel Load = 12000 15 in. wd.

Eff. wd. = 3.352 ft.

$$\text{Load on a ft. of slab} = \frac{120000}{3.352} = 3580\ \#$$



$$LLV = \frac{3580}{1.75}\left(1.75 - \frac{15}{2\ 12}\right) = 2300\ \#$$

$$LLV + Imp = 2300\ 171.4\% = 3942\ \#$$

$$\text{Total Shear} = 3942 + 87.5 = 4029.5\ \#$$

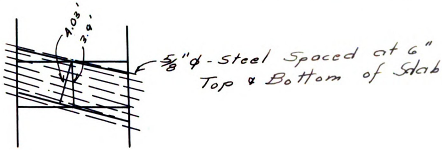
Shear $$v = \frac{V}{7/8\ b\ d} = \frac{4029.5}{7/8\ 12\ 4.5} = 85.2\ \#/\text{sq. in.}$$ allowable 75

Bond $$u = \frac{V}{7/8\ o\ d} = \frac{4029.5}{7/8\ 4.07\ 4.5} = 251\ \#/\text{sq. in.}$$ allowable 250

Concrete Slab between 12 in. @ 25# I Bm. spaced 2ft.-9½in.
12 in. @ 25 # I Bm.-Flange wd. of 6.5 in.
center to center dis. 2.8 ft.

Clear Dis. $2.8 - \frac{6\frac{1}{2}}{12} = 2.25$ ft.

(55) $2.25 + .5 = 2.75$ ft.
2.75 is less than 2.8; therefore Span Length is 2.75 ft.



(41) Effective Width

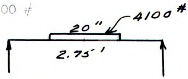
eff. wd. $= 0.7\ s + 2$
s = center to center of supports on 75 degree angle

$s = \frac{2.8}{\sin 75} = 2.9$ ft.

eff. wd. $= .7\ 2.9 + 2 = 4.03$ ft. on 75 degree angle

eff. wd. $= 4.03 \sin 75 = 3.9$ ft. on 90 degree angle

Max. LLBM H20 Loading
wheel load = 16000 # 20 in. wide
Loas on a ft. slab $= \frac{16000}{3.9} = 4100$ #



Continuity = 0.8

LLBM $= .8\left(2050\ 1.375 - \frac{4100}{12}\ \frac{5}{2} \right) = 1572.8$ ft. lbs.

(37) Impact $= \frac{L + 20}{6\ L + 20} = \frac{2.9 + 20}{6\ 2.9 + 20} = 61.2\ \%$

LLBM + Imp. = 1572.8 161.2% = 2540 ft. lbs.

Max. DLBM w = 100 #/ft.

DLBM $= \frac{wl^2}{10} = \frac{100\ 2.75^2}{10} = 75.7$ ft. lbs.

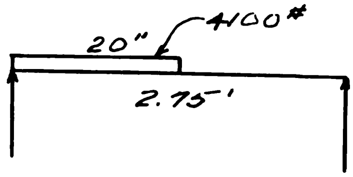
Total BM = 2540 + 75.7 = 2615.7 ft. lbs.

Concrete Slab between 12 in. @ 25" I Bm. spaced 2ft.-9 in.

Check d

$$d = \sqrt{\frac{BM}{K\ b}} = \sqrt{\frac{2615.7\ 12}{175\ 12}} = 3.89 \text{ in. supplied } 4.5 \text{ in.}$$

H20 Live Load Shear H20 Loading



$$LLV = \frac{4100}{2.75}\left(2.75 - \frac{20}{2\ 12}\right) = 2850\ \#$$

$$LLV + Imp. = 2850\ 161.2\% = 4600\ \#$$

Dead Load Shear

$$DLV = \frac{wl}{2} = \frac{100\ 2.75}{2} = 137.5\ \#$$

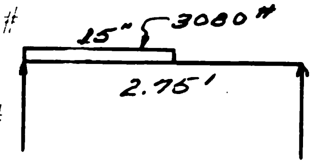
$$\text{Total } V = 4600 + 137.5 = 4737.5\ \#$$

Shear $v = \frac{V}{7/8\ b\ d} = \frac{4737.5}{7/8\ 4.5\ 12} = 100$ #/sq. in.
allowable 75

Bond $u = \frac{V}{7/8\ o\ d} = \frac{4737.5}{7/8\ 4.07\ 4.5} = 295$ #/sq. in.
allowable 250

H15 Live Load Shear H15 Loading
Wheel Load = 12000 # 15 in. wd.
Eff. wd. = 3.9 ft.

Load on a ft. of slab $= \frac{12000}{3.9} = 3080\ \#$



$$LLV = \frac{3080}{2.75}\left(2.75 - \frac{15}{2\ 12}\right) = 2380\ \#$$

$$LLV + Imp = 2380\ 161.2\% = 3840\ \#$$

$$\text{Total } V = 3840 + 137.5 = 3977.5\ \#$$

Shear $v = \frac{V}{7/8\ db} = \frac{3977.5}{7/8\ 4.5\ 12} = 84$ #/sq. in.
allowable 75

Bond $u = \frac{V}{7/8\ o\ d} = \frac{3977.5}{7/8\ 4.07\ 4.5} = 248$ #/sq. in.
allowable 250

Concrete **Slab between** First Girder to First Stringer

10 in. @ 25 # I   Flange wd. of 4.66 in.

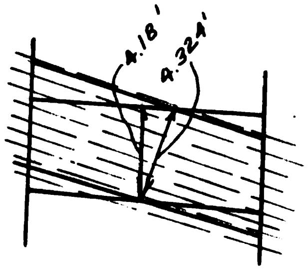
center to center dis.   3 ft. 2 1/2 in.

wd. of flange of girder   1.19 ft.

Clear dis. $3.21 - \dfrac{(1.19 + 4.66/12)}{2} = 2.42$ ft.

(55) $2.42 + .5 = 2.92$ ft.

2.92 is less than 3.21; therefore Span Length is 2.92 ft.



5/8"ϕ-Steel spaced at 6"
Top & Bottom of Slab

(41) Effective Width

eff. wd. = 0.7 s + 2   on 75 degree angle

s = center to center of supports on 75 degree angle

$s = \dfrac{3.21}{\sin 75} = \dfrac{3.21}{.966} = 3.32$ ft.

eff. wd. = .7 3.32 + 2   on 75 degree angle

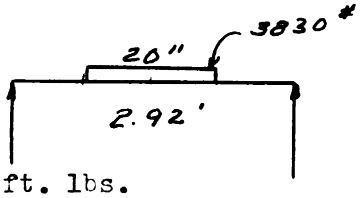
= 4.324 ft.

eff. wd. = 4.324 sin 75 = 4.18 ft. on 90 degree angle

Max. Live Load Bending Moment   H20 Loading

wheel load = 16000 #   20 in. wd.

Load on a ft. slab $= \dfrac{16000}{4.18} = 3830$ #



Continuity = 0.8

LLBM $= .8\left(1915\ 1.46 - \dfrac{3830}{2}\ \dfrac{5}{12}\right) = 1600$ ft. lbs.

(37) Impact $= \dfrac{L + 20}{6\ L + 20} = \dfrac{2.92\ \ 20}{6\ 2.92\ \ 20} = 61$ %

LLBM + Imp. = 1600 161 % = 2575 #

Max. Dead Load Bending Moment   w = 100 #/ft.

DLBM $= \dfrac{w\ l^2}{10} = \dfrac{100\ 2.92^2}{10} = 85.3$ ft. lbs.

Total BM   2575 + 85.3 = 2660.3 ft. lbs.

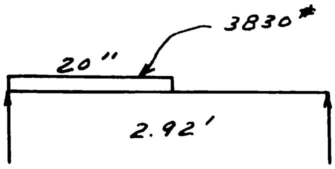
Concrete Slab between First Girder to First Stringer

Check d

$$d = \sqrt{\frac{BM}{K\ b}} = \sqrt{\frac{2060.3\ 12}{173\ 12}} = 3.92 \text{ in.}$$

supplied 4.5

H20 Live Load Shear H20 Loading



$$LLV = \frac{3830}{2.92}\left(2.92 - \frac{20}{2\ 12}\right) = 2740\ \#$$

LLV + Imp. = 2740 161% = 4410 #

Dead Load Shear

$$DLV = \frac{w\ l}{2} = \frac{100\ 2.92}{2} = 146\ \#$$

Total Shear 146 + 4410 = 4556 #

$$\text{Shear} \quad v = \frac{V}{7/8\ b\ d} = \frac{4556}{7/8\ 12\ 4.5} = 96.5\ \#/\text{sq. in.}$$

allowable 75

$$\text{Bond} \quad u = \frac{V}{7/8\ d\ o} = \frac{4556}{7/8\ 4.07\ 4.5} = 284.2\ \#/\text{sq. in.}$$

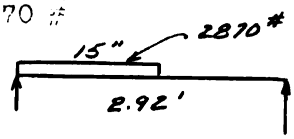
allowable 250

H15 Live Load Shear H15 Loading

Wheel Load = 12000 # 15 in. wd.

Eff. wd. 4.18 ft.

$$\text{Load on a ft. of slab} = \frac{12000}{4.18} = 2870\ \#$$



$$LLV = \frac{2870}{2.92}\left(2.92 - \frac{15}{2\ 12}\right) = 2250\#$$

LLV + Imp. = 2250 161% = 3625 #

Total Shear 3625 + 146 = 3771 #

$$\text{Shear} \quad v = \frac{V}{7/8\ d\ b} = \frac{3771}{7/8\ 4.5\ 12} = 79.7\ \#/\text{sq. in.}$$

$$\text{Bond} \quad u = \frac{V}{7/8\ O\ d} = \frac{3771}{7/8\ 4.07\ 4.5} = 235\ \#/\text{sq.in.}$$

allowable 250

Concrete Slab between 10 in. @ 25 I Beam spaced 2 ft. 3 in.

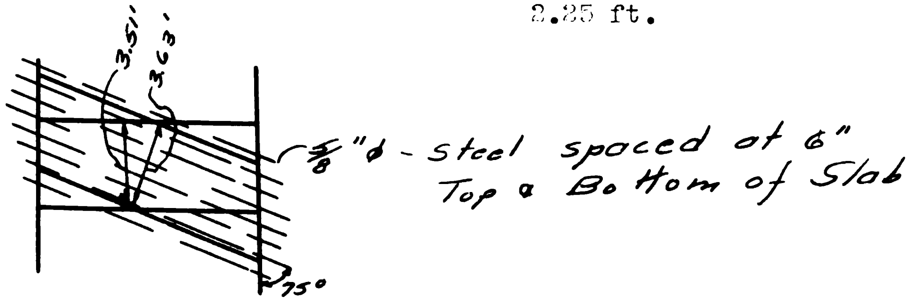
10 in. @ 25# I Bm.-Flange wd. of 4.66 in.

center to center dis. 2.25 ft.

Clear Dis. $= 2.25 - \frac{4.66}{12} = 1.86$ ft.

(55) $1.86 + .5 = 2.36$ ft.

2.36 is more than 2.25; therefore Span Length is 2.25 ft.



(41) Effective Width

eff. wd. $= 0.7\ s + 2$ on 75 degree angle

s = center to center of supports 75 degree angle

$s = \frac{2.25}{\sin 75} = 2.33$ ft.

eff. wd. $= .7\ 2.33 + 2 = 3.63$ ft. on 75 degree angle

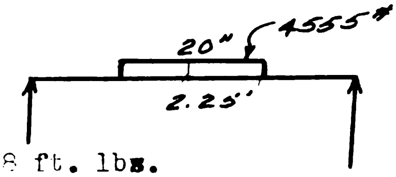
eff. wd. $= 3.63 \sin 75 = 3.51$ ft. on 90 degree angle

Max. LLBM H20 Loading

wheell load 16000 # 20 in. wide

Load on a ft. slab $\frac{16000}{3.51} = 4555$ #

Continuity = 0.8



LLBM $.8\ (2277.5\ 1.125 - \frac{4555}{2}\ \frac{5}{12}) = 1288$ ft. lbs.

(37) Impact $= \frac{L + 20}{6\ L + 20} = \frac{2.25 + 20}{6\ 2.25 + 20} = 66.4\%$

LLBM + Imp. $= 1288\ 166.4\% = 2145$ ft. lbs.

Max. DLBM w = 100 #/ft.

DLBM $= \frac{wl^2}{10} = \frac{100\ 2.25^2}{10} = 50.75$ ft. lbs.

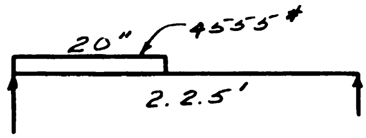
Total BM $= 50.75 + 2145 = 2195.75$ ft. lbs.

Concret e Slab between 10 in. @ 25 I Beam spaced 2 ft. 3 in.

Check d

$$d = \sqrt{\frac{BM}{K\ b}} = \sqrt{\frac{2195.75\ 12}{173\ 12}} = 3.56 \text{ in. supplied } 4.5 \text{ in}$$

H20 Live Load Shear H20 Loading



$$LLV = \frac{4555}{2.25}\left(2.25 - \frac{20}{12\ 2}\right) = 3870\ \#$$

$$LLV + Imp. = 3870\ 166.4\ \% = 4780\ \#$$

Dead Load Shear

$$DLV = \frac{w\ 1}{2} = \frac{100\ 2.25}{2} = 112.5\ \#$$

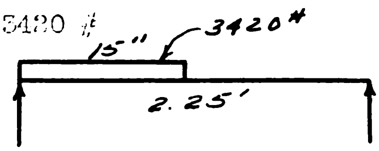
$$\text{Total Shear} = 4780 + 112.5 = 4892.5\ \#$$

Shear $v = \frac{V}{7/8\ b\ d} = \frac{4892.5}{7/8\ 4.5\ 12} = 103.2$ #/sq. in.
allowable 75

Bond $u = \frac{V}{7/8\ 4.07\ 4.5} = 263$ #/sq. in.
allowable 250

H15 Live Load Shear H15 Loading
Wheel Load = 12000 # 15 in. wd.
Eff. wd. = 3.51 ft.

Load on a ft. of slab $= \frac{12000}{3.51} = 3420\ \#$



$$LLV = \frac{3420}{2.25}\left(2.25 - \frac{15}{12\ 2}\right) = 2470\ \#$$

$$LLV + Imp. = 2470\ 166.4\% = 4110\ \#$$

$$\text{Total } V = 4110 + 112.5 = 4222.5\ \#$$

Shear $v = \frac{V}{7/8\ b\ d} = \frac{4222.5}{7/8\ 12\ 4.5} = 89$ #/sq. in.
allowable 75

Bond $u = \frac{V}{7/8\ \Sigma o\ d} = \frac{4222.5}{7/8\ 4.07\ 4.5} = 263$ #/sq. in.
allowable 250

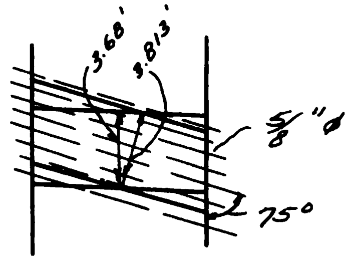
Concrete Slab between Center Girder and first Stringer

12 in. @ 31.5 I BM - Flange wd. of 5 in.
center to center dis. 2.5 ft.
wd. of flange of girder 14 1/4 in. or 1.19 ft.

Clear Dis. $2.5 - \frac{(1.19 \quad 5/12)}{2} = 1695$ ft.

(55) $1.695 + 0.5 = 2195$ ft.

2.195 is less than 2.5 ; therefore Span Length is 2.195
say 2.2 ft.



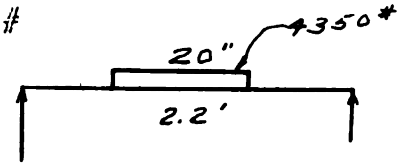
(41) Effective Width

eff. wd. = 0.7 s + 2 on 75 degree angle
s = center to center of supports 75 degree angle
$s = \frac{2.5}{\sin 75} = 2.59$ ft.

eff. wd. = 7 2.59 + 2 = 3.813 on 75 angle

eff. wd. = 3.83 sin 75 = 3.68 on 90 degree angle

Max. LLBM H20 Loading
wheel load = 16000 # 20 in. wide
Load on a ft. slab $= \frac{16000}{3.68} = 4350$ #



Continuity = 0.8

LLBM $= .8 \left(2175\ 1.1 - \frac{4350}{2} \quad \frac{5}{12}\right) = 1190$ ft. lbs.

(37) Impack $= \frac{L + 20}{6L + 20} = \frac{2.2 + 20}{6 \ 2.2 + 20} = 66.9$ %

LLBM + Imp = 1190 166.9 % = 1987 ft. lbs.

Max. DLBM w = 100 #/ft.

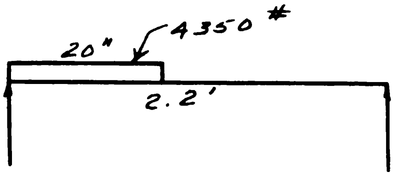
DLBM $= \frac{wl^2}{10} = \frac{100 \ 2.2^2}{10} = 48.5$ ft. lbs.

Total BM 1987 + 48.5 = 2035.5

Concrete Slab between Center Girder and first Stringer

Check d

$$d = \sqrt{\frac{BM}{K\ b}} = \sqrt{\frac{2035.5\ 12}{173\ 12}} = 3.425 \text{ in. supplied } 4.5 \text{ in.}$$



H20 Live Load Shear H20 Loading

$$LLV = 4350\ \left(2.2 - \frac{20}{2\ 12}\right) = 2700\ \#$$

$$LLV + \text{Imp.} = 2700\ 166.9\% = 4500\ \#$$

Dead Load Shear

$$DLV = \frac{w\ l}{2} = 100\ \frac{2.2}{2} = 110\ \#$$

Total Shear 4500 + 110 = 4610 #

Shear $v = \frac{V}{7/8\ b\ d} = \frac{4610}{7/8\ 4.5\ 12} = 97.5$ #/sq. in. allowable 75

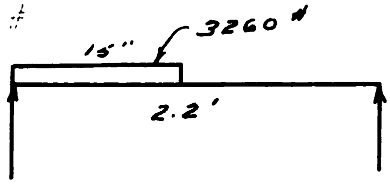
Bond $u = \frac{V}{7/8\ o\ d} = \frac{4610}{7/8\ 4.07\ 4.5} = 289$ #/sq. in. allowable 250

H15 Live Load Shear H15 Loading

Wheel Load 12000 # 15 in. wd.

Eff. wd. 3.68 ft.

Load on a ft. of slab $= \frac{12000}{3.68} = 3260\ \#$



$$LLV = \frac{3260}{2.2}\left(2.2 - \frac{15}{2\ 12}\right) = 2335\ \#$$

$$LLV + \text{Imp.} = 2335\ 166.9\% = 3900\ \#$$

Total Shear 3900 + 110 = 4010 #

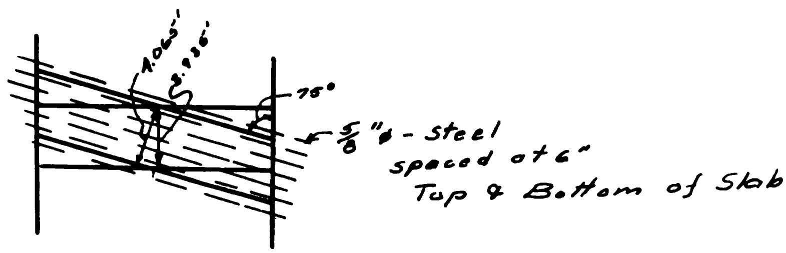
Shear $v = \frac{V}{7/8\ d\ b} = \frac{4010}{7/8\ 4.5\ 12} = 84.9$ #/sq. in. allowable 75

Bond $u = \frac{V}{7/8\ o\ d} = \frac{4010}{7/8\ 4.07\ 4.5} = 250$ #/sq. in. allowable 250

Concrete Slab between Second Girder and first 12 in. @ 25# CB
12 in. @ 25# CB- Flange wd. 6.5 in.
center to center dis. 2ft. 10in.
wd. of flange of girder 1.19 ft.

Clear dis. $= 2.85 - \frac{(1.19 + 6.5)}{2} =$ 1.985 ft.

(55) $1.985 + .5 = 2.485$ ft.
2.485 is less than 2.85; therefore Span Length is 2.485
say 2.49 ft.



(41) Effective Width

eff. wd. $= 0.7\ s + 2$ on 75 degree angle
s = center to center of supports on 75 degree angle
$s = \frac{2.85}{\sin 75} = 2.95$ ft.

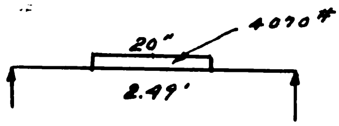
eff. wd. $= .7\ 2.95 + 2$ on 75 degree angle
$= 4.065$ ft.

eff. wd. $= 4.065\ \sin 75 = 3.935$ ft. on 90 degree angle

Max. Live Load Bending Moment H20 Loading
wheel load = 16000 # 20 in. wd.

Load on a ft. slab $= \frac{16000}{3.935} = 4070$ #

Continuity = 0.8



LLBM $= .8\ (2035\ 1.245 - \frac{4070\ 5}{2\ 12}) = 1340$ ft. lbs.

(37) Impact $= \frac{L + 20}{CL + 20} = \frac{2.49 + 20}{6\ 2.49 + 20} = 64.5$ %

LLBM + Imp. = 1340 164.5 % = 2205 ft. lbs.

Max. Dead Load Bending Moment w = 100 #/ft.

DLBM $= \frac{w\ l^2}{10} = \frac{100\ 2.49^2}{10} = 62$ ft. lbs.

Total BM = 2205 + 62 = 2267 ft. lbs.

Concrete Slab betwe n Second Girder and first 12 in. @ 25# CB

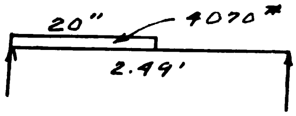
Check d

$$d = \sqrt{\frac{M}{K\ b}} = \sqrt{\frac{2267\ 12}{175\ 12}} = 3.62 \text{ in.}$$

supplied 4.5

H20 Live Load Shear H20 Loading

$$LLV = \frac{4070}{2.49}\left(2.49 - \frac{20}{2\ 12}\right) = 2720\ \#$$



$$LLV + Imp. = 2720\ 164.5\ \% = 4480\ \#$$

Dead Load Shear $W = 100\ \#/ft.$

$$DLV = \frac{w\ l}{2} = \frac{100\ 2.49}{2} = 124.5\ \#$$

$$\text{Total } V = 4480 + 124.5 = 4604.5\ \#$$

$$\text{Shear } v = \frac{V}{7/8\ b\ d} = \frac{4604.5}{7/8\ 4.5\ 12} = 97.4\ \#/sq.\ in.$$

allowable 75

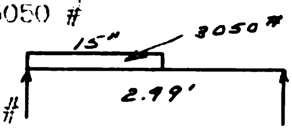
$$\text{Bond} \quad u = \frac{V}{7/8\ \Sigma o\ d} = \frac{4604.5}{7/8\ 4.07\ 4.5} = 287\ \#/sq.\ in.$$

H15 Live Load Shear H15 Loading

Wheel Load = 12000 # 15 in. wd.

Eff. wd. = 3.935 ft.

$$\text{Load on a ft. of slab} = \frac{12000}{3.935} = 3050\ \#$$



$$LLV = \frac{3050}{2.49}\left(2.49 - \frac{15}{2\ 12}\right) = 2285\ \#$$

$$LLV + Imp. = 2265\ 164.5\ \% = 3530\ \#$$

$$\text{Total } V = 3530 + 124.5 = 3654.5\ \#$$

$$\text{Sheae } v = \frac{V}{7/8\ d\ b} = \frac{3654.5}{7/8\ 4.5\ 12} = 77.2\ \#/sq.\ in.$$

allowable 7 5

$$\text{Bond} \quad u = \frac{V}{7/8\ \Sigma o\ d} = \frac{3654.5}{7/8\ 4.07\ 4.5} = 228\ \#/\ sq.\ in.$$

allowable 250

STRINGERS

12 in. @ 25 # CB Stringers spaced at 2 ft. 9½ in.

Center to center of Beams 15.4 ft.

Dead Load

Wearing Surface 25 #/sq. ft.

25 2.8 = 70 #/ft.

Concrete 150 #/ft.^{2}

.5 2.8 150 = 210

Stringer 25

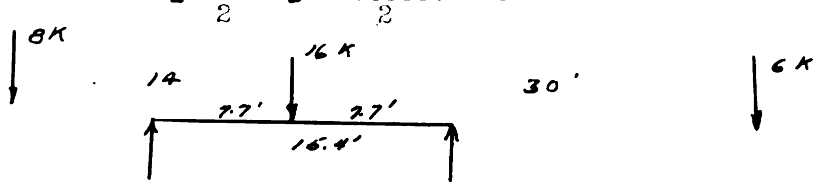
total 305 #/ft.

Max. Dead Load Bending Moment

$$DLBM = \frac{wl^2}{8} = \frac{305\ 15.4^2}{8} = 9040 \text{ ft. lbs.}$$

Max. Dead Load Shear

$$DLV = \frac{wl}{2} = \frac{305\ 15.4}{2} = 2350 \text{ lbs.}$$



Max. Live Load Bending Moment H20 Loading

LLBM of one Traffic Line

$$\frac{32000\ 15.4}{2\quad 2} = 123200 \text{ ft. lbs.}$$

(43) $M' = C\frac{M}{N}$ = LLBM for one stringer

N = Width of Traffic Line(not to exceed 10 ft.) / Spacing of Stringers

$C = 1$ (for concrete surface)

$$N = \frac{10}{2.8} = 3.57$$

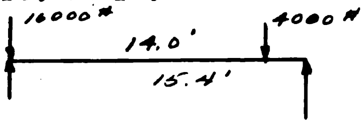
$$M' = \frac{1\ 123200}{3.57} = 34500 \text{ ft. lbs.}$$ LLBM for one stringer

(37) $\text{Impact} = \frac{L+20}{6\ L+20} = \frac{15.4+20}{6\ 15.4+20} = 31.5\ \%$

LLBM + Imp. = 34500 131.5 % = 45400 ft. lbs.

Total BM = 45400 + 9040 = 54440 ft. lbs.

12 in. @ 25 # CB Stringers spaced at 2 ft. 9½ in.



Max. Live Load Shear

$$LLV = 16000 + \frac{4000 \; 1.4}{15.4} = 16364 \;\#$$

$$LLV + Imp. = 16364 \; 131.5 \;\% = 21514 \;\#$$

$$Tot.\ V = 21514 + 2350 = 23864 \;\#$$

Check Section Modulus (I/c)

I/c of 12 in. @ 25 # CB 30.8 in.^{3}

$I/c = M/f$ $\quad f = 18000$ #/sq. in.

$$I/c = \frac{54440 \; 12}{18000} = 36.1 \text{ in.}^3$$

supplied 30.8

$f = 24000$ #/sq. in.

$$I/c = \frac{54440 \; 12}{24000} = 27.0 \text{ in.}^3$$

supplied 30.8

Check Web of Stringer

Back to back of Flange 11.88 in.
Thickness of Web .240 in.

$$v = \frac{V}{A} = \frac{23864}{11.88 \; .24} = 8340 \;\#/\text{sq. in.}$$

allowable 12000

12 in. @ 31.5 # I Bm.

Not only has it a Section Modulus of 36.0 in.^{3}, but is 2 ft. 9½ in. from next stringer and 2 ft. 6 in. from Girder; therefore it is OK

10 in. @ 25 # I Stringers spaced at 2 ft. 3 in.

Center to center of Beams 15.4 ft.

Dead Load

Wearing Surface 25 #/sq. ft.

25 2.25 = 56 #/ft.

Concrete 150 #/ft.^{3}

.5 2.25 150 = 168.75

Stringer 25 #/ft. 25

total 249.75 #/ft.

Max. Dead Load Bending Moment

$$DLBM = \frac{w\,l^2}{8} = \frac{249.75 \; 15.4^2}{8} = 7400 \text{ ft. lbs.}$$

Max. Dead Load Shear

$$DLV = \frac{w\,l}{2} = \frac{249.75 \; 15.4}{2} = 1922 \; \#$$

Max. Live Load Bending Moment H20 Loading

LLBM of one Traffic Line = 123200 ft. lbs.

(43) $M' = C \frac{M}{N}$ = LLBM for one stringer

$N = \frac{\text{Width of Traffic Lane (not to exceed 10 ft.)}}{\text{Spacing of Stringers}}$

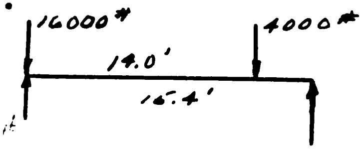
$C = 1$ (for concrete surface)

$$N = \frac{10}{2.25} = 4.45$$

$M' = \frac{1 \; 123200}{4.45} = 27750$ ft. lbs. LLBM for one Stringer

LLBM + Imp. = 131.5 % 27750 = 36500 ft. lbs.

Total BM = 36500 + 7400 = 43900 ft. lbs.



Max. Live Load Shear

$$LLV = 16000 + \frac{4000 \; 1.4}{15.4} = 16364 \; \#$$

LLV + Imp. = 16364 131.5 % = 21514 ft. lbs.

Total V = 21514 + 1922 = 23436 #

10 in. @ 25 # I Stringers spaced at 2 ft. [illegible] in.

Check Section Modulus (I/c)

I/c of 10 in. @ 25 # 24 in^3.

$I/c = M/f$

$f = 18000$ #/sq. in.

$$I/c = \frac{43900 \ 12}{18000} = 29.3 \ in^3.$$

supplied 24

$f = 24000$ #/sq. in.

$$I/c = \frac{43900 \ 12}{24000} = 22.0 \ in^3.$$

supplied 24

Check Web of Stringer

Back to back of Flange 10 in.

Thickness of Web 0.31 in.

$$v = \frac{V}{A} = \frac{23436}{10 \ 0.31} = 7530 \text{ \#/sq. in.}$$

allowable 18000

6 in. @ 14.75 # I Stringer spaced at 2 ft. $4\frac{1}{2}$ in.

Center to center of Beams 15.4 ft.

Dead Load

Concrete 150 #/ft.^{2}

5/12 2.375 150 = 148.5 #/ft.

Stringer 14.75 #/ft. 14.75

total 163.25

Max. Dead Load Bending Moment

$$DLBM = \frac{w\,l^2}{8} = \frac{163.25\ 15.4^2}{8} = 4850 \text{ ft. lbs.}$$

Max. Dead Load Shear

$$DLV = \frac{w\,l}{2} = \frac{163.25\ 15.4}{2} = 1258 \text{ \#}$$

Max. Live Load Bending Moment

(34) Live Load 100 #/sq. ft.

100 2.375 = 237.5 #/ft.

$$LLBM = \frac{w\,l^2}{8} = \frac{237.5\ 15.4^2}{8} = 7050 \text{ ft. lbs.}$$

Max. Live Load Shear

$$LLV = \frac{w\,l}{2} = \frac{237.5\ 15.4}{2} = 1830 \text{ \#}$$

Total Bending Moment

4850 + 7050 = 11900 ft. lbs.

Total Shear

1258 + 1830 = 3038 #

Check Section Modulus (I/c)

I/c of 6 in. @ 14.75 # I 7.9 in.^{3}

I/c = M/f f = 18000 #/sq. in.

$$I/c = \frac{11900\ 12}{18000} = 7.93 \text{ in.}^3$$

supplied 7.9

f = 24000 #/sq. in.

$$I/c = \frac{11900\ 12}{24000} = 5.95 \text{ in.}^3$$

supplied 7.9

6 in. @ 14.75 # I Stringer spaced at 2 ft. 4½ in.

Check Web of Stringer

Back to back of Flange	6 in.
Thickness of Web	0.343 in.

$$v = \frac{V}{A} = \frac{3088}{6 \times 0.345} = 1500 \ \#/\text{sq. in.}$$

allowable 12000

BEAMS

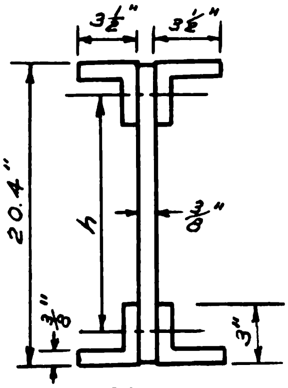
Dead Weight of Inside and Outside Beam #/ft.

3/8 in. Web	20.4 in. wd.		
	3/8 1/12 20.4/1 490	=	25.1 #/ft.
4-Angles	3½ by 3 by 3/8 in. @ 7.9 #		31.6
4-Stiffeners	5 by 2½ by 3/8 in. @ 6.6#		
	4 6.6 1.6/15.4	=	3.0
115 Rivets	12 #/100 Heads		
	12 230/15.4/100	=	1.8
4-Splice Plates	3/8"by .75' by 1.2'		
	$\frac{4\ .75\ 3}{8\ 12}\ \frac{1.2\ 490}{15.4}$	=	1.4
		tot.	62.9 say 63 #/ft.

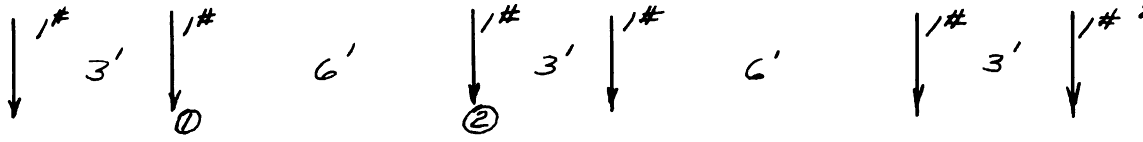
Cross Section of Beam

Gross Section of Flange Area
2-angles 3½ by 3 by 3/8 4.6 sq. in.

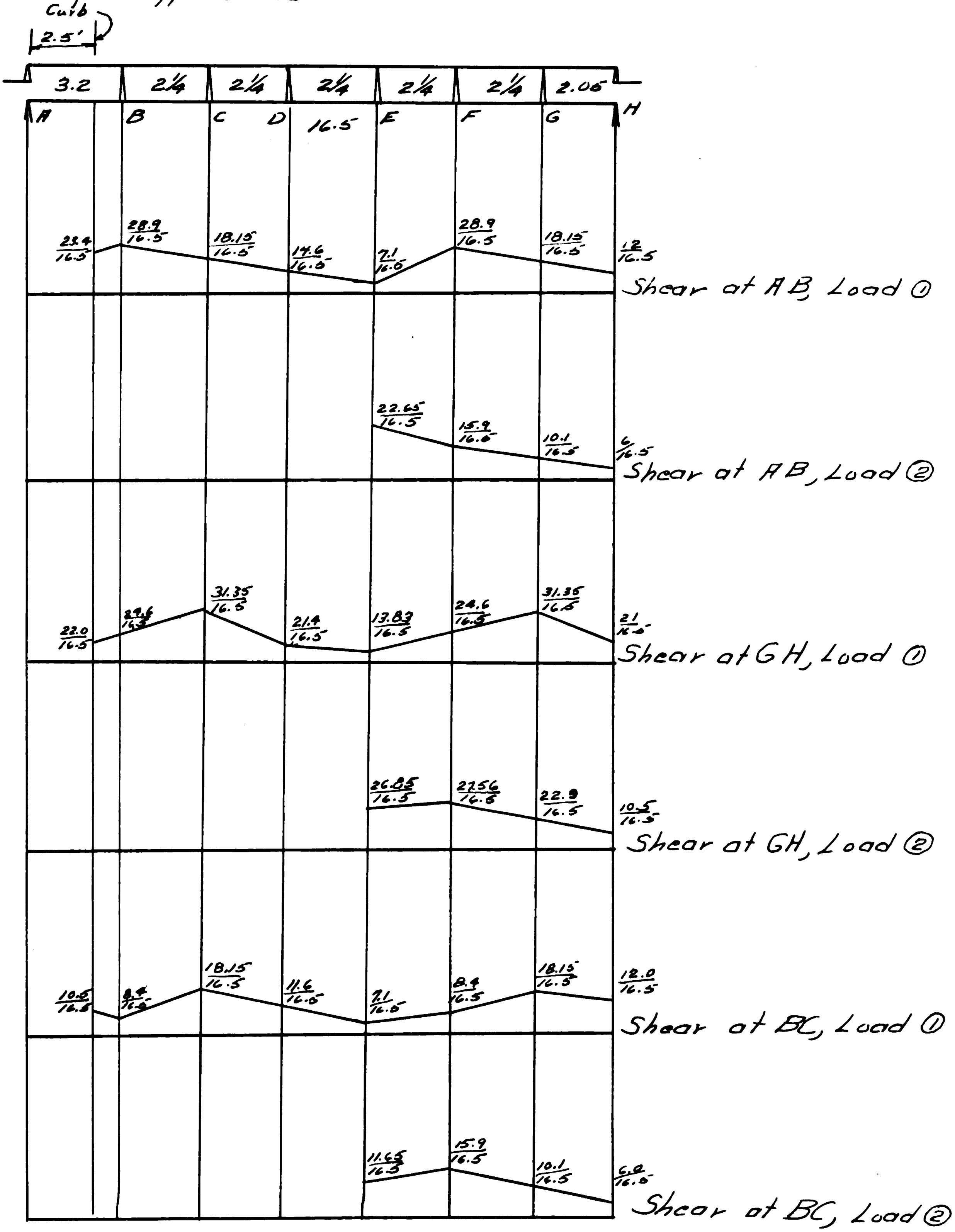
Net Section of Flange Area
-4-3/4 Rivets 3/8 7/8 4 -1.3
3.3 sq. in.

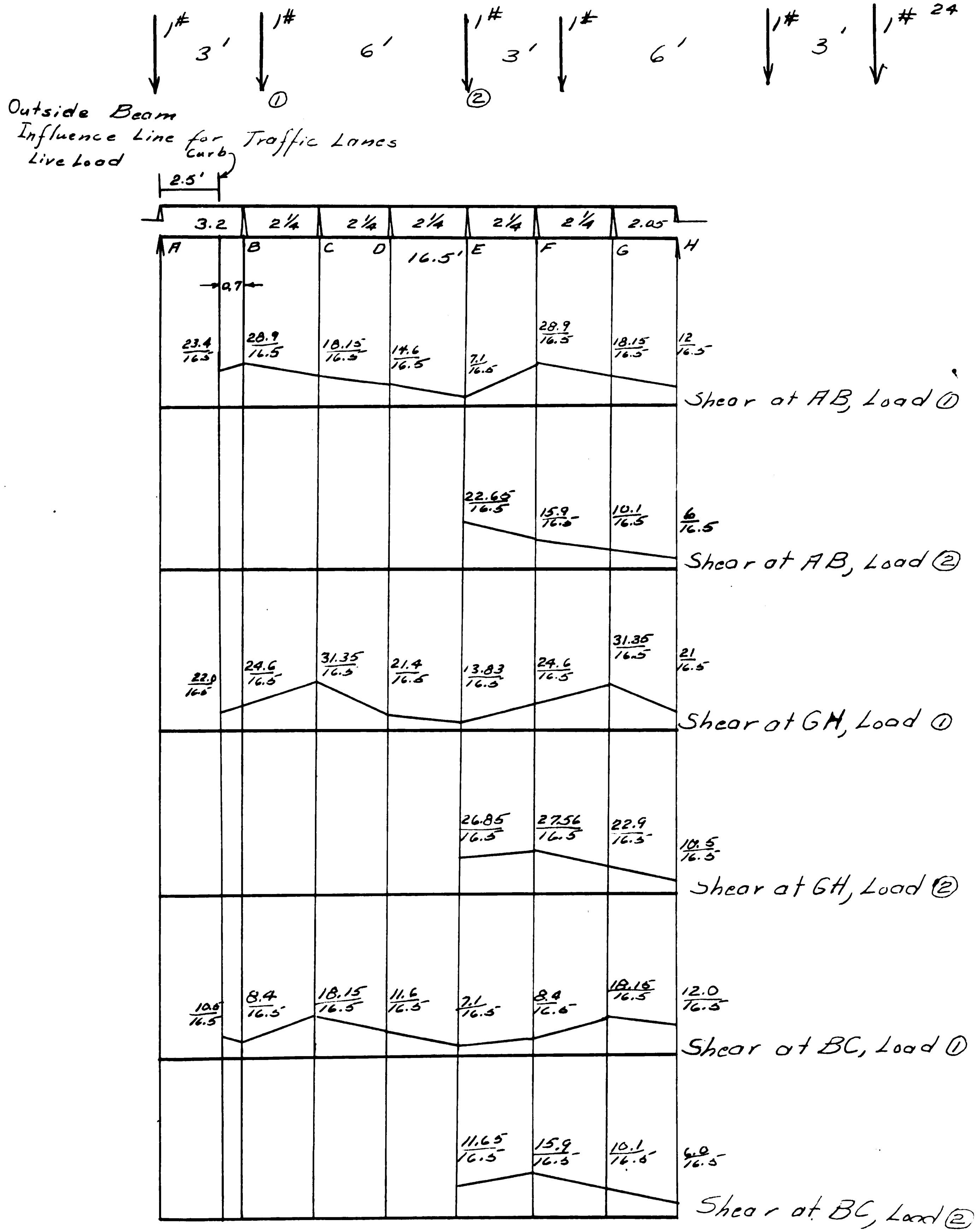


h = 20.4 - 2 .83
h = 18.74 in.



Outside Beam
Influence Lines for Traffic Lanes
Live Load





1#
3'
1#
6'
1#
3'
1#
6'
1#
3'
1#
24
①
②
Outside Beam
Influence Line for Traffic Lanes
Live Load
Curb
2.5'
3.2
2 1/4
2 1/4
2 1/4
2 1/4
2 1/4
2.05
A
B
C
D
16.5'
E
F
G
H
0.7
23.4/16.5
28.9/16.5
18.15/16.5
14.6/16.5
7.1/16.5
28.9/16.5
18.15/16.5
12/16.5
Shear at AB, Load ①
22.65/16.5
15.9/16.5
10.1/16.5
6/16.5
Shear at AB, Load ②
22.0/16.5
24.6/16.5
31.35/16.5
21.4/16.5
13.83/16.5
24.6/16.5
31.35/16.5
21/16.5
Shear at GH, Load ①
26.85/16.5
27.56/16.5
22.9/16.5
10.5/16.5
Shear at GH, Load ②
10.5/16.5
8.4/16.5
18.15/16.5
11.6/16.5
7.1/16.5
8.4/16.5
18.15/16.5
12.0/16.5
Shear at BC, Load ①
11.65/16.5
15.9/16.5
10.1/16.5
6.0/16.5
Shear at BC, Load ②

Outside Beam

Dead Load

Dead Load Conc.

Panel Point B

Concrete 150 #/ft.^{3}

$.5 \dfrac{(3.2 + 2.25)}{2}$ 15.4 150 = 3150 #

Wearing Surface 25 #/sq. ft.

$25 \dfrac{(3.2 + 2.25)}{2}$ 15.4 = 1050

10 in. @ 25 # I stringer

25 15.4 = 385

tot. 4585 #

Panel Points C, D, E, F

Concrete 150 #/ft.^{3}

.5 $2\frac{1}{4}$ 15.4 150 = 2600 #

Wearing Surface 25 #/sq. ft.

25 $2\frac{1}{4}$ 15.4 = 868

10 in. @ 25 # I stringer

25 15.4 = 385

tot. 3853 #

Panel Point G

Concrete 150 #/ft.^{3}

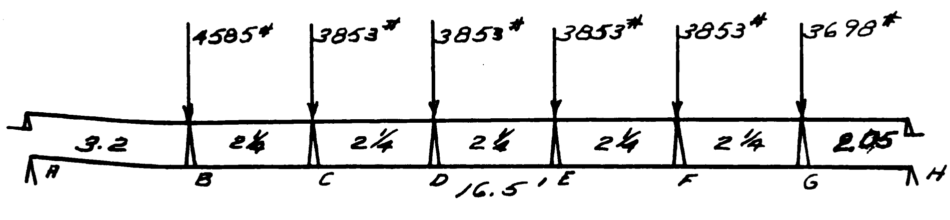
$.5 \dfrac{(2\frac{1}{4} + 2.05)}{2}$ 15.4 150 = 2485 #

Wearing surface 25 #/sq. ft.

$25 \dfrac{(2\frac{1}{4} + 2.05)}{2}$ 15.4 = 828

Stinger 385

tot. 3698 #



Outside Beam

Max. Dead Load Bending Moment

DLBM at E

$$3698\,\frac{14.45\;6.55}{16.5} + \frac{3253\;6.55}{16.5}\,(12.2 + 9.95 + 7.7 + 5.45)$$

$$+ 4615\,\frac{3.2\;6.55}{16.5} - 3698\;4.5 - 3253\;2.25 + \frac{63\;16.5^2}{8} = 56400 \text{ ft. lbs.}$$

DLBM at D

$$3698\,\frac{14.45\;8.8}{16.5} + \frac{3253\;8.8}{16.5}\,(12.2 + 9.95 + 7.7 + 5.45)$$

$$+ 4585\,\frac{3.2\;8.8}{16.5} - 3698\;6.75 - 3253\;2.25 + \frac{63\;16.5^2}{8}$$

$$\text{Max.} \qquad = 59750 \text{ ft. lbs.}$$

Max. Dead Load Shear

DLV at AB

$$3698\,\frac{2.05}{16.5} + \frac{3253}{16.5}\,(4.3 + 6.55 + 8.8 + 11.05) + 4585\,\frac{13.3}{16.5}$$

$$+ \frac{63\;16.5}{2} = 11640$$

DLV at GI

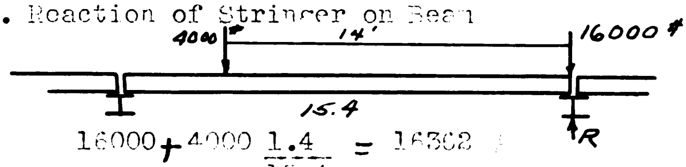
$$3698\,\frac{14.45}{16.5} + \frac{3253}{16.5}\,(12.2 + 9.95 + 7.7 + 5.45) + \frac{4585\;3.2}{16.5}$$

$$\text{Max.} \qquad + \frac{63\;16.5}{2} = 12920 \#$$

Max. Live Load Bending Moment H20 Loading

LLBM is Max. at D when load 1 is 1.5 ft. from curb and equals 12[illegible]/15.5 for 1 load on each wheel

Max. Reaction of Stringer on Beam



$$16000 + 4000\,\frac{1.4}{15.4} = 16362 \#$$

(37) Impact $= \dfrac{L + 20}{6\,L + 20} \qquad \dfrac{16.5 + 20}{6\;16.5 + 20} = 30.7 \%$

Outside Beam

Max. Live Load Bending Moment (cont.)

B.M. at D (cont.)

LLBM at D + Imp. $= 16362\ \frac{122}{16.5}\ 130.7\ \% = 158000$ ft. lbs.

DL BM at D = 59350 ft. lbs.

Total BM at D = 59350 + 158000 = 217350 ft. lbs.

B.M at E

LLBM is Max. at E when load 2 is at E and equals 126.4/16.5 for 1 # load on each wheel

LLBM at E + Imp. $= 16362\ \frac{126.4}{16.5}\ 130.7\ \% = 163800$ ft. lbs.

DLBM at E = 56400 ft. lbs.

(Max.) Total BM at E = 56400 + 163800 = 220200 ft. lbs. Freely supported

Continuity = 0.7

Total BM = 220200 0.7 = 154140 ft. lbs. Continuous Beam

Max. Live Load Shear H20 Loading

LLV is max. at GH with load 1 at C and equals 31.35/16.5 for 1 # load on each wheel

LLV at GH + Imps. $= 16362\ \frac{31.35}{16.5}\ 139.7\ \% = 40500$ #

DLV is max. at GH = 12980 #

Total V = 12980 + 40500 = 53480 #

Outside Beam

Max. Total Shear at the second Panel from End (between first and second stringer)

Dead Load Shear at FG

$$\frac{3698\ 14.45}{16.5} + \frac{3853}{16.5}(12.2 + 9.95 + 7.7 + 5.45) + \frac{4585\ 3.2}{16.5}$$

$$+ \frac{63\ 16.5}{2} - 3698 - 63\ 2.05 = 9093\ \#\ \text{Max.}$$

Dead Load Shear at BC

$$\frac{3698\ 2.05}{16.5} + \frac{3853}{16.5}(4.3 + 6.55 + 8.8 + 11.05) + \frac{4585\ 13.3}{16.5}$$

$$+ \frac{63\ 16.5}{2} - 4585 - 63\ 3.2 = 7053\ \#$$

Live Load Shear is Max. at FG with load 1 at F and equals 24.6/16.5 for 1# load at each wheel.

Impact = 130.7 %

Max. Reaction of Stringer on Beam = 16562 #

$$\text{LLV at FG} + \text{Imp.} = 16562\ \frac{24.6}{16.5}\ 130.7\ \% = 31900\ \#$$

DLV at FG = 9093 #

Total V at FG = 9093 + 31900 = 40993 #

Outside Beam

Check Cross Section of Beam

Gross Section of Flange 4.6 sq. in.

Net Section of Flange 3.3 sq. in.

Web 3/8 by 20.4 in.

h 18.74 in.

Tension Flange

$$A_s = \frac{BM}{f\ h} - 1/8 \text{ of web}$$

f = 18000 #/sq. in.

$$3.3 = \frac{154140\ 12}{18000\ h} - 1/8\ 20.4\ 3/8$$

h = 2[illegible].1 in. supplied 18.74

f = 24000 #/sq. in.

$$3.3 = \frac{154140\ 12}{24000\ h} - 1/8\ 20.4\ 3/8$$

h = 18.1 in. supplied 18.74

Compression Flange

(47) $f = 18000 - 5\ L^2/b^2$

$= 18000 - 5\ (3.2\ 12)^2/7.37^2 = 17864.5$ #/sq. in.

$$A_s = \frac{154140\ 12}{17864.5\ 18.74} - 1/8\ 20.4\ 3/8$$

$A_s = 4.57$ sq. in. supplied 4.6

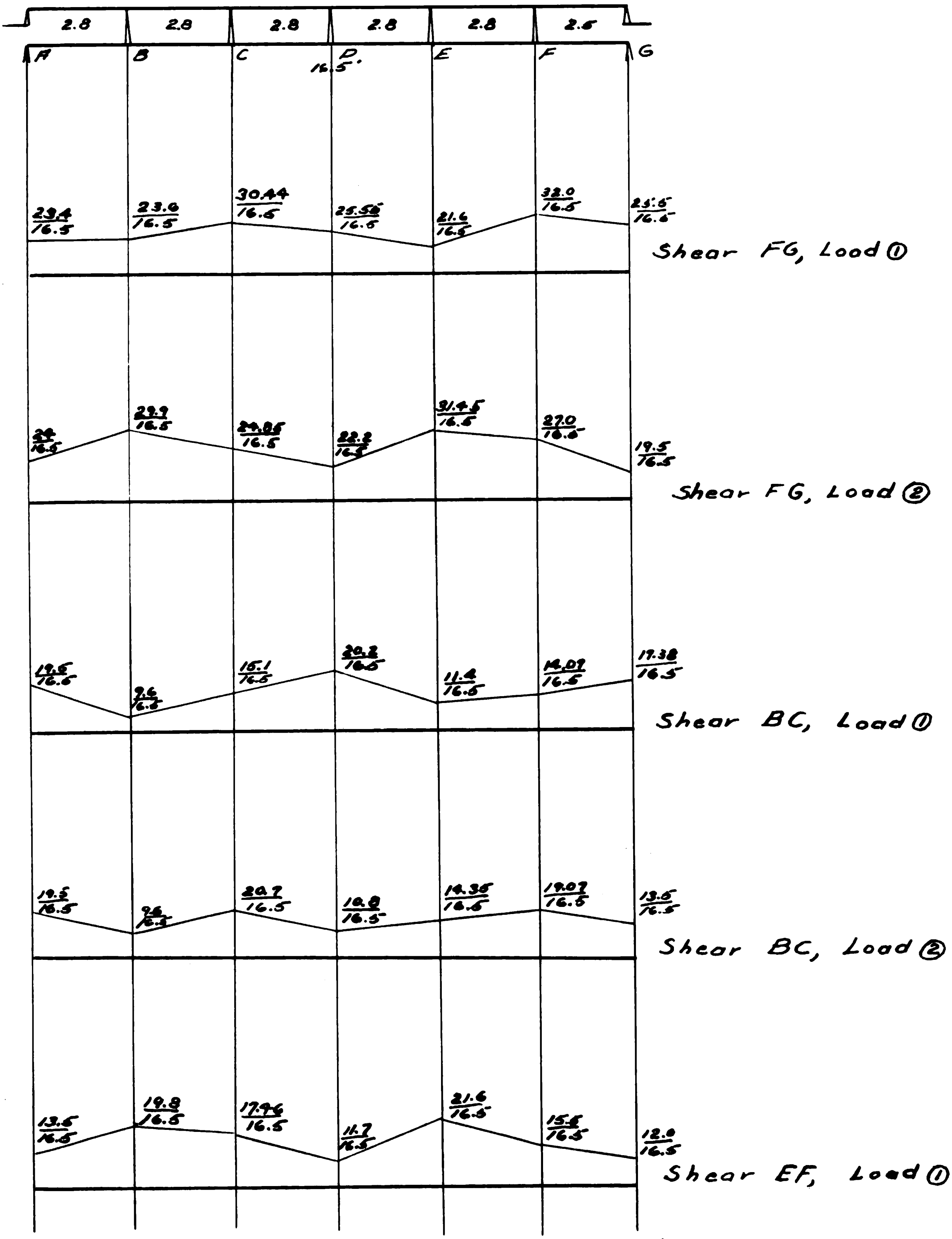
Web

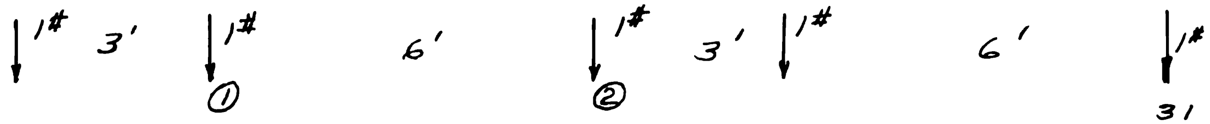
v = 12000 #/sq. in. allowable

$$v = \frac{V}{A} = \frac{55480}{3/8\ 20.4} = 6980 \text{ \#/sq. in.}$$

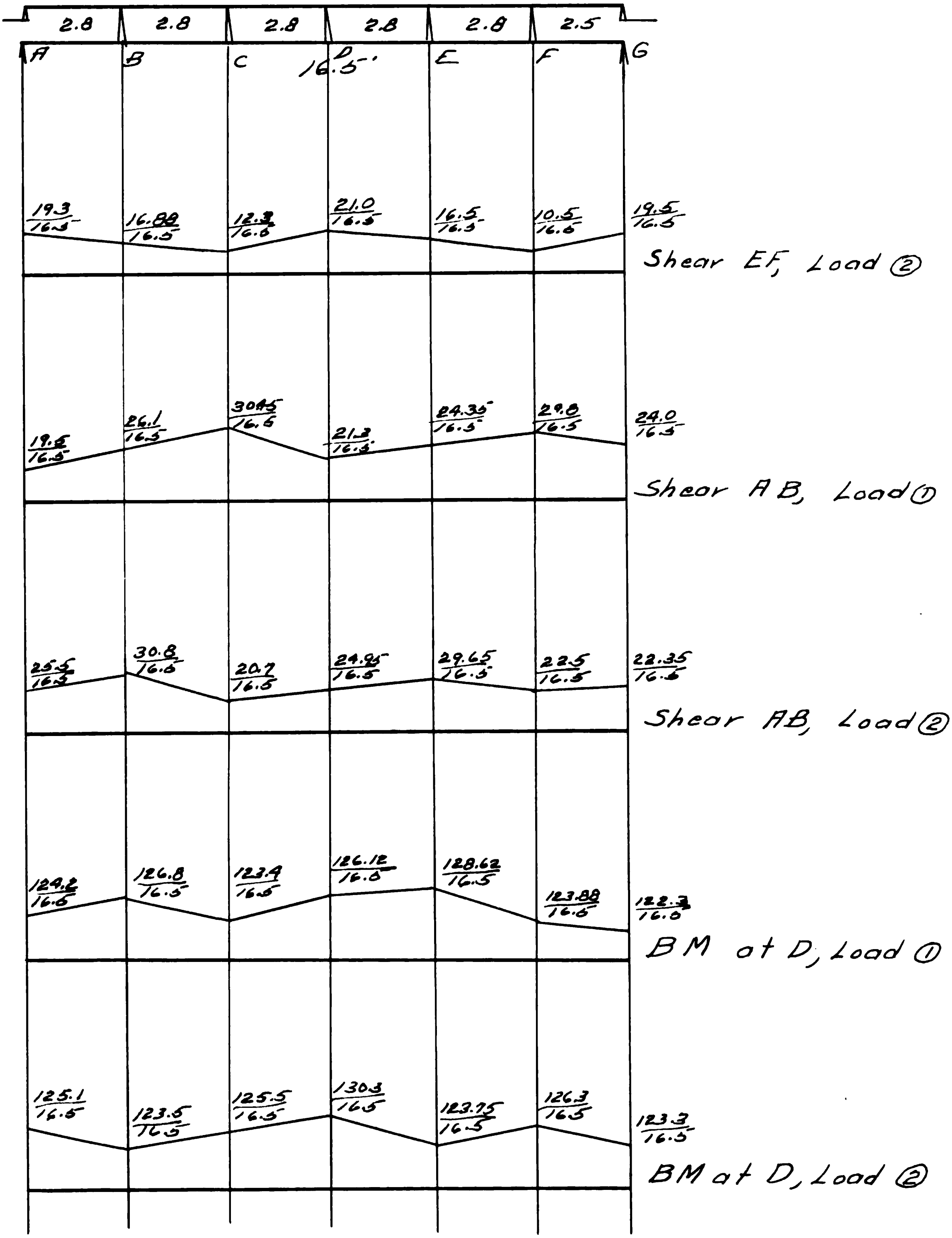
Inside Beam

Live Load
Infuence Lines
for Traffic Lanes





Inside Beam
Live Load Infuence
Lines for Lanes of Traffic



Inside Beam

Dead Load

Conc. Dead Load

Panel Points B, C, D, E

Concrete 150 $\#/ft^3$.

$0.5 \; 2.8 \; 15.4 \; 150 = 3240 \; \#$

12 in. @ 25 # CB stringer

$25 \; 15.4 = 385$

Wearing Surface 25 #/sq. ft.

$25 \; 2.8 \; 15.4 = 1078$

total $4703 \; \#$

Panel Point F

Concrete 150 $\#/ft^3$.

$0.5 \left(\frac{2.5+2.8}{2}\right) 15.4 \; 150 = 3060 \; \#$

12 in. @ 31.5 # I Bm. stringer

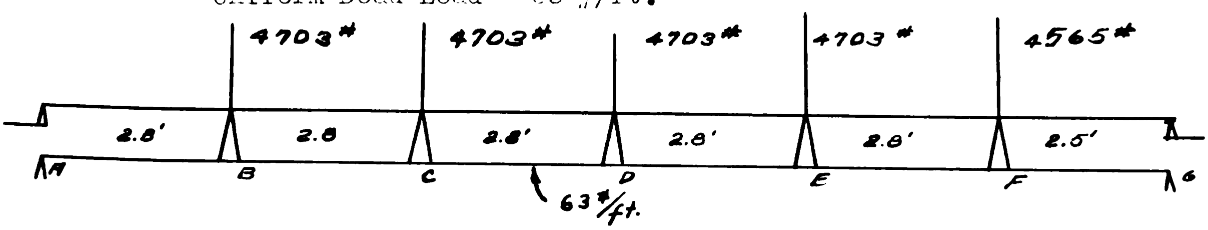
$15.4 \; 31.5 = 485$

Wearing surface 25 #/sq. ft.

$25 \; 15.4 \left(\frac{2.5+2.8}{2}\right) = 1020$

total $4565 \; \#$

Uniform Dead Load 63 #/ft.



Max. Dead Load Bending Moment

DLBM at D

$$\frac{4565 \; 14.0 \; 8.1}{16.5} + \frac{4703 \; 8.1}{16.5}(11.2+8.4+5.6+2.8) - 4703 \; 2.8$$

$$- 4565 \; 5.6 + \frac{63 \; 16.5^2}{8} = 59100 \text{ ft. lbs.}$$

Max. Dead Load Shear

DLV at FG

$$\frac{4565 \; 14}{16.5} + \frac{4703}{16.5}(11.2+8.4+5.6+2.8) + \frac{63 \; 16.5}{2} = 12370 \; \# \text{ Max.}$$

~~DLV at AB~~

$$\cancel{\frac{4565 \; 2.5}{16.5}} \quad \cancel{4703}$$

Inside Beam

Max. Dead Load Shear(cont.)

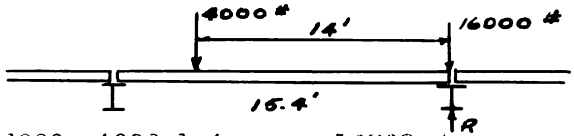
DLV at AB

$$\frac{4565\ 2.5}{16.5} + \frac{4703}{16.5}(5.3+8.1+10.9+13.7) + \frac{63\ 16.5}{2} = 11020\ \#$$

Max. Live Load Bending Moment H20 Loading

LLBM is max. at D with Load 2 at D and equals $\frac{130.3}{16.5}$ for 1# load on each wheel

Max. Reaction of Stringer on Beam



$$16000 + 4000\ \frac{1.4}{15.4} = 16362\ \#$$

(37) Impact $= \frac{L+20}{6\ L+20} = \frac{16.5+20}{6\ 16.5+20} = 30.7\ \%$

LLBM + Imp. $= 16362\ \frac{130.3}{16.5}\ 130.7\ \% = 168570$ ft. lbs.

Max. DLBM $=$ 59100 ft. lbs.

Total BM $= 59100 + 168570 = 227670$ ft. lbs. freely supported

Continuity at center of Beam .7

Total BM $= 0.7\ 227670 = 159370$ ft. lbs. Continuous Beam

Max. Live Load Shear H20 Loading

LLV is max. at FG with Load 1 at F and equals $\frac{32}{16.5}$ for 1# load on each wheel

LLV at FG

$$16362\ \frac{32}{16.5} = 317000\ \#$$

LLV + Imp. $=$ 31700 130.7 % 41400 #

DLV at FG $=$ 12370

Total V $= 12370 + 41400 = 53770$ #

Inside Beam

Max. Total Shear at the second Panel from End (between first and second stringer)

Dead Load Shear at BC

$$\frac{4565\ 2.5}{16.5} + \frac{4703}{16.5}(5.3 + 8.1 + 10.9 + 13.7) + \frac{63\ 16.5}{2}$$

$$- 4703 - 63\ 2.8 = 6141\ \#$$

Dead Load Shear at EF

$$\frac{4565\ 14}{16.5} + \frac{4703}{16.5}(11.2 + 8.4 + 5.6 + 2.8) + \frac{63\ 16.5}{2}$$

$$- 4565 - 63\ 2.5 = 8647\ \# \qquad \text{Max.}$$

Live Load Shear is Max. at EF with load 1 at E and equals 21.6/16.5 for 1 # Load at each wheel.

Impact = 130.7 %

Max. Reaction of Stringer on Beam = 16362 #

$$\text{LLV at EF + Imp.} = 16362\ \frac{21.6}{16.5}\ 130.7\ \% = 27900\ \#$$

DLV at EF = 8647 #

Total V at EF = 27900 + 8647 = 36547 #

Inside Beam

Check Cross Section of Beam

Gross Section of Flange 4.6 sq. in.

Net Section of Flange 3.3 sq. in.

Web 3/8 by 20.4 in.

h 18.74 in.

Tension Flange

$A_s = \frac{BM}{f\ h} - 1/8$ of web

$f = 12000$ #/sq. in.

$3.3 = \frac{159370\ 12}{12000\ h} - 1/8\ 20.4\ 3/8$

$h = 24.8$ in. supplied 18.74 in.

$f = 24000$

$3.3 = \frac{159370\ 12}{24000\ h} - 1/8\ 20.4\ 3/8$

$h = 18.7$ in. supplied 18.74

Compression Flange

(47) $f = 18000 - 5\ L^2/b^2$

$= 18000 - 5(2.8\ 12)^2/7.37^2 = 17896.8$ #/sq. in.

$A_s = \frac{159370\ 12}{17896.8\ 18.74} - 1/8\ 20.4\ 3/8$

$A_s = 4.74$ sq. in. supplied 4.6

Web

$v = 12000$ #/sq. in. allowable

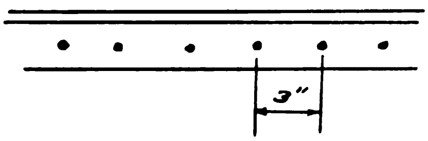
$v = \frac{V}{A} = \frac{53770}{3/8\ 20.4} = 7020$ #/sq. in.

Rivets in Beam from end to first Stiffener

Max. V of Each Beam

Inside Beam 53770 # Max.
Outside Beam 55420

Flange Angle Rivets



Inside Beam:
V was Max. at FG
Load on top of Flange
DL at F = 4565 #
Max. Reaction of Stringer 16362 # (H20)
LL + Imp = 16362 130.7 % = 21380 #
Total Load = 21380 + 4565 = 25945 #

Average Distance between stringers $= \frac{2.5 + 2.8}{2} = 2.65$ ft.

$w = \frac{25945}{2.65} = 9800$ #/ft.

Bearing of 3/4 Rivet on 3/8 Web   Bearing = 27000 #/sq. in.
3/4 3/8 27000 = 7600 #

Double Shear   shear = 13500 #/sq. in.
2 .443 13500 = 11960 #

$$HI = \frac{V}{h} \times \frac{A_f}{A_f + A_w/8} = \frac{53770}{18.74} \times \frac{4.6}{4.6 + 0.957} = 2375 \#$$

$$\text{Pitch} = \frac{R}{\sqrt{HI^2 + w^2}} = \frac{7600}{\sqrt{2375^2 + (9800/12)^2}} = 3.03 \text{ in.}$$

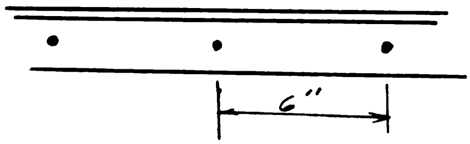
supplied 3 in.

Rivets in Beam (from Stiffener to Stiffener)

Max. V of Each Beam (between frist and second stringer)

Inside Beam	30547 #	
Outside Beam	40993	Max.

Flange Angle Rivets



Outside Beam

V was Max. at FG

Load on top of Flange

DL at F = 3853 #

Max. Reaction of Stringer = 16362 # (T20)

LL + Imp. = 16362 130.7 % = 21380 #

Total Load = 21380 + 3853 = 25233 #

Average Dist. between stringers = 2.25 Ft.

$$w = \frac{25233}{2.25} = 11230 \#/ft.$$

Bearing of 3/4 Rivet on 3/8 Web	7600 #
Double Shear of 3/4 Rivet	11960

$$HI = \frac{V}{h} \times \frac{A_f}{A_f + A_w/8} = \frac{40993}{13.74} \times \frac{4.6}{4.6 + 0.957} = 1807 \#$$

$$Pitch = \sqrt{\frac{R^2}{HI^2 + w^2}} = \sqrt{\frac{7600^2}{1807^2 + (11230/12)^2}} = 3.74 \text{ in.}$$

supplied 6 in.

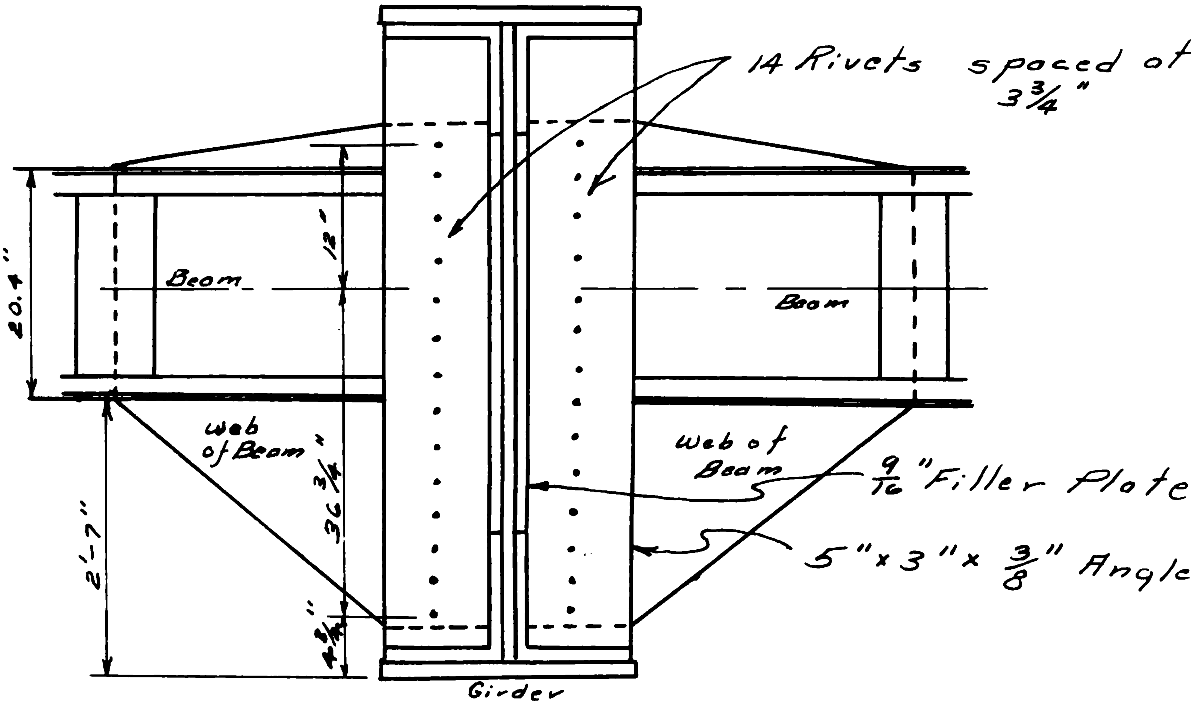
Intermediate Stiffeners (Loacated 3.1 ft. from each end of Beam)

(127)(h) 60 3/8 = 22.5 in. which is greater than clear distance of 14.4 in.

No stiffeners needed.

Connection of Beam to Girder

Test Connection of Angles to Web of Beam



Inside Beam has greatest Bending Moment and also the greatest Shear next the Center Girder (Shear is the Reaction at the End of Beam)

BM = 227670 ft. lbs. bending moment at center of Freely Supported

Continuity at end of beam = 5/6

BM = 227670 5/6 = 190250 ft. lbs. (at support)

V = 53770 #

Shear on each Rivet $= \dfrac{53770}{14} = 3840$ #

Bearing of 3/4 Rivet on 3/8 Web Bearing 27000 #/sq. in.

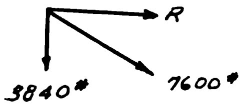
3/8 3/4 27000 = 7600 #

Double Shear of 3/4 Rivet shear = 13500 #/sq. in.

2 .443 13500 = 11960 #

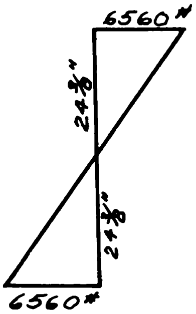
Connection of Beam to Girder
Test Connection of Angles to Web of Beam (cont.)

Reaction for bending moment



$$R = \sqrt{7600^2 - 3840^2} = 6560\ \#$$

Bending Moment of Rivets

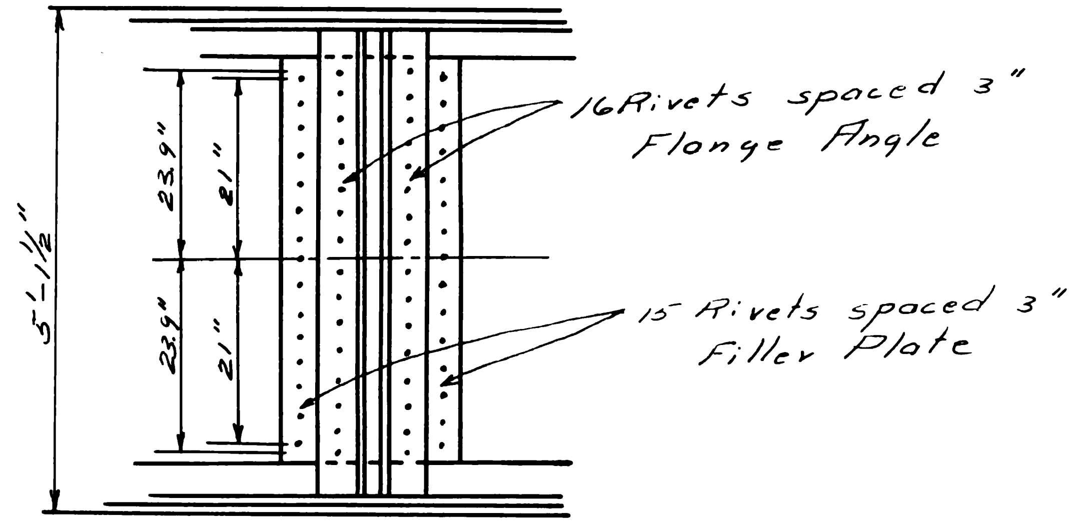


$$= \frac{2\ 6560}{24\,3/8}\left(1.875^2 + 5.625^2 + 9.375^2 + 13.125^2 + 16.875^2 + 20.625^2 + 24.375^2\right)$$

= 860000 in. lbs.
or 71600 ft. lbs. supplied
120250 ft. lbs. needed

Connection of Beam to Girder

Test Connection of Angles to Web of Girder



Rivets in Flange Angle

Bearing on 3/4 Web and on 2-9/16 Filler Plates
bearing = 27000 #/sq. in.

27000 (3/4 + 2 9/16) 3/4 = 56000 #

Double Shear of 3/4 Rivet v = 13500 #/sq. in.
2 .443 13500 = 11960 #

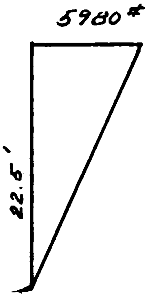
BM = 190250 ft. lbs.
V = 53770 # One Beam
V = 2 53770 = 107540 # Two Beams (one on each side of girder
Number of Rivets needed for Shear
$= \frac{107540}{11960}$ = 9 Rivets Supplied 32 16 in each angle

(92)(a) Rivets in Tension fiber stress = ½ 13500 = 6750 #/sq. in.

Rivets take the Tension while Compression is taken by angles bearing on web of Girder
Assumed neutral Axis ½ Length of Angle
Fiber Stress per 3/4 Rivet = 6750 .443 = 2990 #
2 rows of rivets = 2 2990 = 5980 #

Connection of Beam to Girder

Test Connection of Angles to Web of Girder (cont.)



BM of Rivets

$$\frac{2\ 5980}{22.5}\left(22.5^2+19.5^2+16.5^2+13.5^2+10.5^2+7.5^2+4.5^2+1.5^2\right)$$

= 815000 in. lbs.
or 67000 ft. lbs. Supplied
190250 ft. lbs. Needed

Test Connection of Filler Plate to Web of Girder

(99) Rivets in Filler (Same stress as those in Angle)

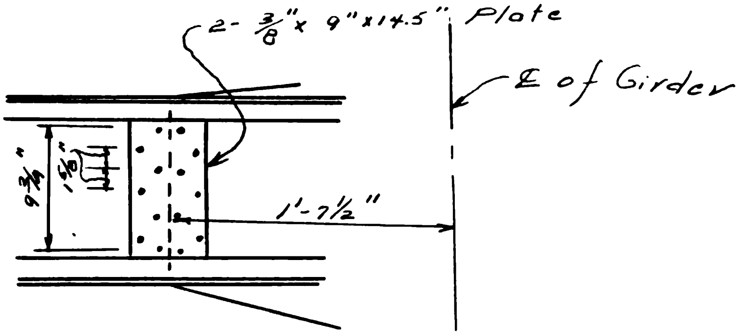
Bearing of 3/4 Rivet on 3/4 Web of Girder
bearing = 27000 #/sq. in.
3/4 3/4 27000 = 15200 #

Double Shear of 3/4 Rivet = 11960 #

V = 107540 #

Number of Rivets $= \frac{107540}{11960} =$ 9 rivets needed
supplied 30
15 on each end of filler

Beam Web Splice



(127)(f)

Web 3/8 by 20.4 in.

Gross Sectio = 7.65 Sq. in.

Net Section

5-3/4 Rivets

5 7/8 3/8 = 1.65 sq. in.

Net Sec. = 7.65 - 1.65 = 6.0 sq. in.

Web Splice Plates 2-3/8 by 14.5 in.

Gross Section

2 14.5 3/8 = 10.88 sq. in.

net Section

2 4 7/8 3/8 = 2.62 sq. in.

Net Sec. = 10.88 - 2.62 = 8.26 sq. in.

needed 6.0

Web will take Shear v = 12000 #/sq. in.

12000 3/8 20.4 = 91800 #

Shear taken by one rivet

$$\frac{91800}{7} = 13114.3 \#$$

Bearing on one Rivet bearing = 27000 #/sq. in.

3/8 3/4 27000 = 7600 #

Double Shear on one Rivet shear = 13500 #/sq. in.

2 .4418 13500 = 11960 #

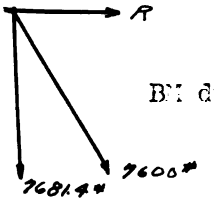
13114.3 is greater than 7600; therefore there is nothing left to

resulsed bending moment

The actual Max. V of Beams is 53770 #

Shear taken by one Rivet = 53770/7 = 7681.4 #

Reaction to BM



$$R = \sqrt{7600^2 + 7681.4^2} = 1400 \#$$

BM due to Rivets

$$\frac{2\ 1400}{47/8}\left[(15/8)^2 + (\ )^2 + (47/8)^2\right] = 21350 \text{ in. lbs.}$$

Beam Web Splice

Bending Moment possible of the web

1/6 20.4 3/8 18000 18.74 = 322500 in. lbs.

BM the rivets will take is 21350 in. lbs.

Max. Bending Moment actually taken by web at this point

Max. Total BM at center 227670 ft. lbs. freely supported

Continuity at end of Beam equals 5/6

5/6 227670 12 = 2275000 in. lbs.

$$= \frac{1/6 \text{ web}}{\text{Gross Sec.}} \; 2275000 = \frac{1/6 \; 20.4 \; 3/8}{4.6} \times 2275000$$

= 473000 in. lbs.

BM the rivets will take is 21350 in. lbs.

Cantilever Beam under Sidewalk

Dead Load of Beam

Top angles 2-3½ by 3 by 3/8 @ 7.9 #/ft. 2 7.9		15.8 #/ft.
Lower angles 2-3½ by 3 by 3/8 @ 7.9 #/ft. 2 7.9 $\frac{11.2}{11}$		16.1
Web 3/8 in.		
uniform load	.7 $\frac{3\ 480}{8\ 12}$	10.7
		tot. 42.6 #/ft.

load varing from 0 to 2.3 $\frac{3\ 480}{8\ 12}$ 35.8 # over 11 ft. of Beam

Live Load (concentrated at panel point.)

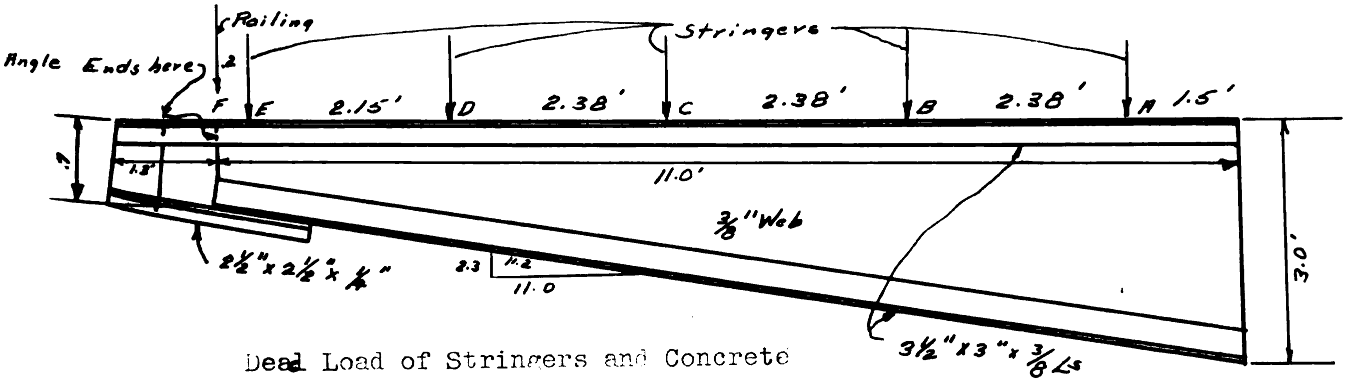
(34) 100 #/ft.²

Panel Point A	
$(\frac{1\ 2.38}{2})$ 15.4 100	2600 #
Panel Points B and C	
$(\frac{2.38\ \ 2.38}{2})$ 15.4 100	3660
Panel Points D	
$(\frac{2.38\ \ 2.15}{2})$ 15.4 100	3500
Panel Point E	
$(\frac{2.15\ \ .2}{2})$ 15.4 100	1820

Dead Load at End of Beam

1 angle-3½ by 3 by 3/8	7.9 #/ft.
1 angle-2½ by 2½ by ¼	4.1
	tot. 12.0 #/ft.

Cantilever Beam under Sidewalk



Dead Load of Stringers and Concrete

Panel Point A

5 in. concrete

$$\frac{5}{12}\,150\,\left(\frac{1+2.38}{2}\right)\,15.4 = 1625\ \#$$

6 in. @ 14.75 I

$$14.75\ 15.4 = 227.5$$

tot. 1852.5

Panel Points B and C

5 in. concrete

$$\frac{5}{12}\,150\,\left(\frac{2.38+2.38}{2}\right)\,15.4 = 2290$$

6 in. @ 14.75 I = 227.5

tot. 2517.5

Panel Point D

5 in. concrete

$$\frac{5}{12}\,150\,\left(\frac{2.38+2.15}{2}\right)\,15.4 = 2180$$

6 in. @ 14.75 I = 227.5

tot. 2407.5

Panel Point E

5 in. concrete

$$\frac{5}{12}\,150\,\left(\frac{2.15}{2}+0.2\right)\,15.4 = 1134$$

6 in. @ 14.75 I = 227.5

tot. 1361.5

Railing Loading

Dead Load 15#/ft.

15 15.4 = 231 #

(25) Verticle Live Load 100 #/ft.

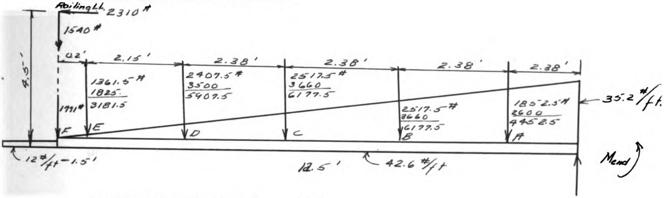
100 15.4 = 1540 #

Total Verticle Load 231 + 1540 = 1771 #

(25) Horizonal Live Load 150 #/ft.

150 15.4 = 2310 #

Cantilever Beam under Sidewalk



Bending Moment at End Max.

$$4452.5\ 1.5 + 6177.5\ 3.88 + 6177.5\ 6.26 + 5907.5\ 8.64 + 3181.5\ 10.8 + 1771\ 11 + 1540\ 11 + 2310\ 4.5 + 42.6\ 11\ \frac{11}{2} + 12\ 1.5\ \left(\frac{1.5}{2} + 11\right) + \frac{35.2}{2}\ 11\ \frac{11}{3} = 189,727 \text{ ft. lbs.}$$

Shear at End Max.

$$4452.5 + 6177.5 + 6177.5 + 5907.5 + 3181.5 + 1540 + 1771 + 12\ 1.5 + 42.6\ 11 + \frac{35.2\ 11}{2} = 29887.0\ \#$$

Bending Moment at A

$$6177.5\ 2.38 + 6177.5\ 4.76 + 5907.5\ 7.14 + 3181.5\ 9.3 + 1771\ 9.5 + 1540\ 9\ \ 2310\ 4.5 + 12\ 1.5\ \left(\frac{1.5}{2} + 9.5\right) + 42.6\ 9.5\ \frac{9.5}{2} + \frac{9.5\ 35.2}{11}\ \frac{9.5}{2}\ \frac{9.5}{3} = 160,261.5 \text{ ft. lbs.}$$

Shear at A

$$4452.5 + 6177.5 + 6177.5 + 5907.5 + 3181.5 + 1771 + 1540 + 12\ 1.5 + 42.6\ 9.5 + \frac{35.2}{11}\ \frac{9.5\ 9.5}{2} = 29774.5\ \#$$

Bending Moment at D

$$3181.5\ 2.15 + 1771\ 2.35 + 1540\ 2.35 + 2310\ 4.5 + 12\ 1.5\left(\frac{1.5}{2} + 2.35\right) + \frac{35.2}{11}\ \frac{2.35\ 2.35}{2}\ \frac{2.35}{3} + 42.6\ 2.35\ \frac{2.35}{2} = 27010.9 \text{ ft. lbs.}$$

Shear at D

$$5907.5 + 3181.5 + 1771 + 1540 + 12\ 1.5 + 42.6\ 2.35 + \frac{35.2}{11}\ \frac{2.35\ 2.35}{2} = 12526.8\ \#$$

Cantilever Beam under Sidewalk

Bending Moment at F

$$2310 \ 4.5 + 12 \ 1.5 \ \frac{1.5}{2} = 10413.5 \text{ ft. lbs.}$$

Shear at F

$$1540 + 1771 + 12 \ 1.5 = 3329 \ \#$$

Check Cross Section at End of Cantilever Beam

BM = 189727 ft. lbs   V = 29887 #

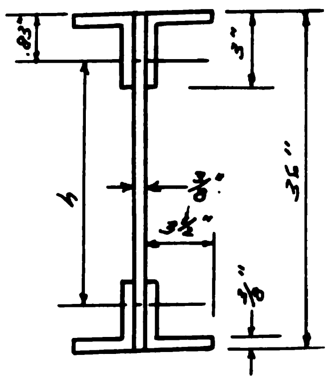
Cross Section

2-angles @ 2.3 sq. in.   4.6 sq. in.

Net Section

-4 Rivets holes 4 7/8 3/8 = -1.3 sq. in.

3.3 Net.



$h = 36 - 2 \ .83 = 34.34$ in.

Tension Flange

$f = 18000$ #/sq. in.

$$A = \frac{BM}{f \ h} - 1/8 \text{ web}$$

$$3.3 = \frac{189727 \ 12}{18000 \ h} - 1/8 \ 3/8 \ 36$$

$h = 35.4$ in.   supplied 34.34

Compression Flange

(47) $$f = 18000 - 5 \frac{(L)^2}{(b)^2}$$

$$f = 18000 - 5 \frac{12^2}{(7/12)^2} = 15880 \ \#/\text{sq. in.}$$

$$A = \frac{189727 \ 12}{15880 \ 34.34} - 1/8 \ 3/8 \ 36$$

$A = 2.49$ sq. in.   supplied 4.6

Web

$v = 12000$ #/sq, in.   allowable

$$v = \frac{V}{A} = \frac{29887}{3/8 \ 36} = 2220 \ \#/\text{sq. in.}$$

Cantilever Beam under Sidewalk

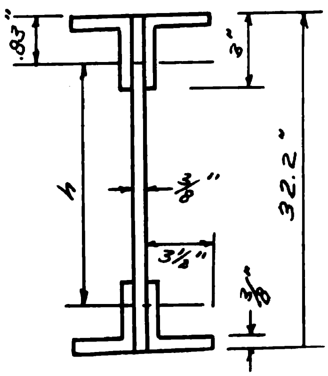
Check Cross Section at Panel Point A

BM = 160261.5 ft. lbs. $\quad$ V = 29774.5 #

Gross Section $\quad$ 4.6 sq. in.
Net Section $\quad$ 3.3 sq. in.

Back to back of Flange angles

dis. $\quad 3 - \frac{2.3}{11}\ 1.5 = 2.68$ ft. or 32.2 in.



h = 32.2 - 2 .83
= 30.54 in.

Tension Flange
f = 18000 #/sq. in.

$$A = \frac{BM}{f\ h} - 1/8 \text{ web}$$

$$3.3 = \frac{160261.5\ 12}{18000\ h} - 1/8\ 3/8\ 32.2$$

h = 22.3 in. $\quad$ supplied 30.54

Compression Flange
f = 15880 #/sq. in.

$$A = \frac{160261.5\ 12}{15880\ 30.54} - 1/8\ 3/8\ 32.2$$

A = 2.47 sq. in. $\quad$ supplied 4.6

Web $\quad$ v = 12000 #/sq. in. $\quad$ allowable

$$v = \frac{V}{A} = \frac{29774.5}{3/8\ 32.2} = 2380 \text{ \#/sq. in.}$$

Cantilever Beam under Sidewalk

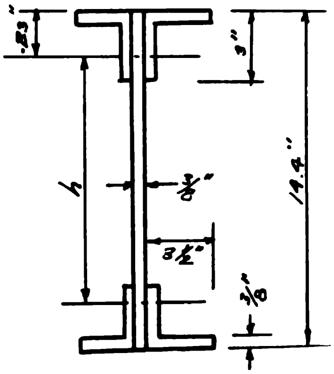
Check Cross Section at Panel Point D

BM = 27010.9 ft. lbs   V = 12526.8 #

Cross Section   4.5 sq. in.
Net Section   3.3 sq. in.

Back to back of Flange angles

dis. $3 - \frac{2.3}{11}\ 8.64 = 1.2$ ft. or 14.4 in.



h = 14.4 - 0.83 2
= 12.54 in.

Tension Flange

f = 18000 #/sq. in.

$A = \frac{BM}{f\ h} - 1/8$ web

$3.3 = \frac{27010.9\ 12}{18000\ h} - 1/8\ 14.4\ 3/8$

h = 4.4 in.   supplied 12.54

Compression Flange

f = 15880 #/sq. in.

$A = \frac{27010.9\ 12}{15880\ 12.54} - 1/8\ 3/8\ 14.4$

A = 0.951 sq. in.   supplied 4.6

Web

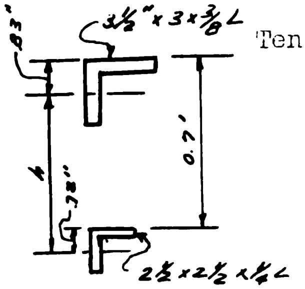
v = 12000 #/in.   allowable

$v = \frac{V}{A} = \frac{12526.8}{14.4\ 3/8} = 2320$ #/sq. in.

Cantilever Beam under Sidewalk

Check Cross Section at Panel Point F

BM = 10413.5 ft. lbs.   V 3329 #



Tension Flange

f = 18000 #/sq. in.

Gross Section angle-3/8 by 3½ by 3 @ 2.3 sq. in.

$A = \frac{BM}{f\ h} = \frac{10413.5\ 12}{18000\ 8.29}$

A = .84 sq. in.   supplied 2.3

h = .7.62 + .72 - .83 = 8.29"

Compression Flange

f = 15880 #/sq. in.

Gross Section angle-2½ by 2½ by ¼ @ 1.19 sq. in.

$A = \frac{10413.5\ 12}{15880\ 8.29} = 0.952$ sq. in.

supplied 1.19

No web present here; therefore Flanges must take shear

Total A of Flanges   1.19 + 2.3 = 3.49 sq. in.

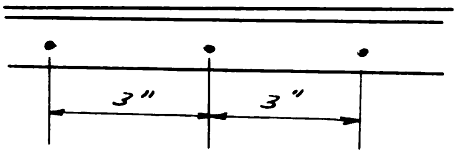
$v = \frac{V}{A} = \frac{3329}{3.49} = 953$ #/sq. in.

The Flanges have enough extra Area over what is needed for Compression and Tension to take care of this Shear.

Rivets in Cantilver Beam (form the end to first stringer)

Shear at End = 29887 #
h = 30.54 in. at Panel Point A

Flange Angle Rivets



Load on top of Flange
DL + LLV = 4452.5 #

Average Distance between stringers $= \frac{2.38 + 1.5}{2} = 1.94$ ft.

$$w = \frac{4452.5}{1.94} = 2295 \ \#/\text{ft.}$$

Bearing of 3/4 Rivet on 3/8 Web 7600 #
Double Shear of 3/4 Rivet 11960

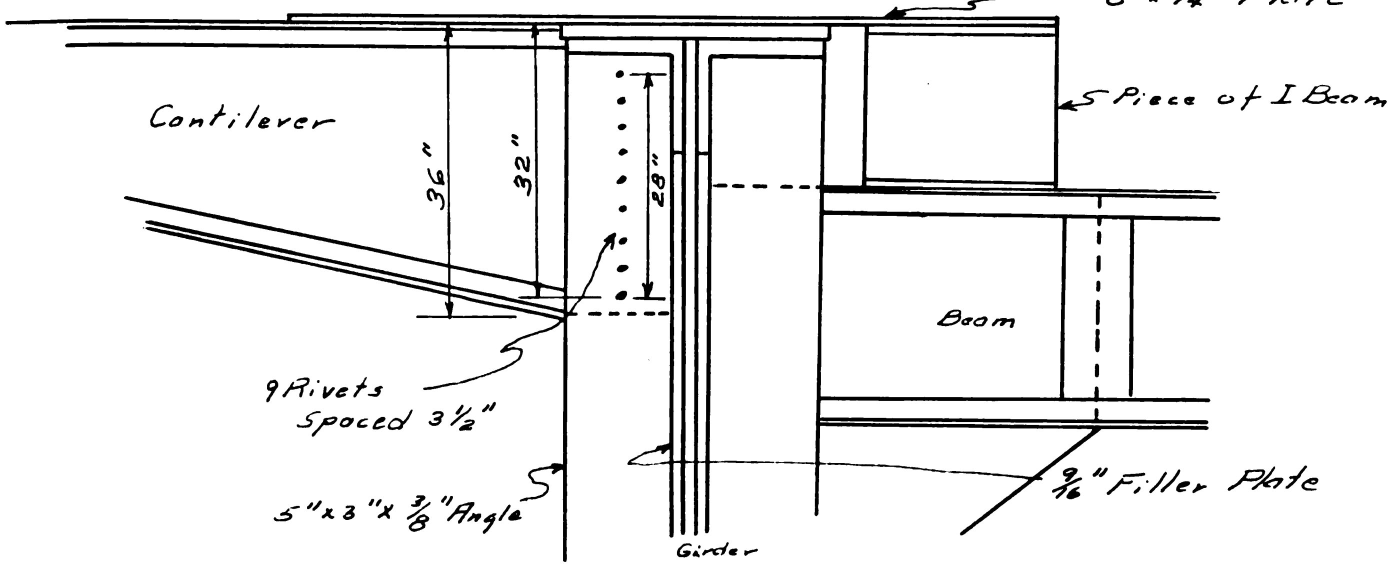
$$HI = \frac{V}{h} \times \frac{A_f}{A_f + A_w/8} = \frac{29887}{30.54} \times \frac{4.6}{4.6 + 1.51} = 735 \ \#$$

$$\text{Pitch} = \frac{R}{\sqrt{HI^2 + w^2}} = \frac{7600}{\sqrt{735^2 + (2295/12)^2}} = 8 \text{ in.}$$ supplied 3 in.

The Rivets in the rest of the Flange are 6 in. (no need to test)

Connection of Cantilever to Girder

Test Connection of Angles to Web of Cantilever



Total V of Cantilever and Beam (V is the Reaction of Beams)

53420 + 29887 = 83307 #

Bearing on 3/8 web by 3/4 River

27000 3/8 3/4 = 7600 #

Double Sheer of 3/4 Rivet

2 .443 13500 = 11960 #

Sheer Taken by each Rivet

$$\frac{29887}{9} = 3321 \#$$

BM at end of Cantilever = 189787 ft. lbs.

Reaction of Rivets for Bending Moment

$$R = \sqrt{7600^2 - 3320^2} = 6840 \#$$

$$BM = \frac{2\ 6840}{14}(14^2 + 10.5^2 + 7^2 + 3.5^2) = 359000 \text{ in. lbs. supplied}$$

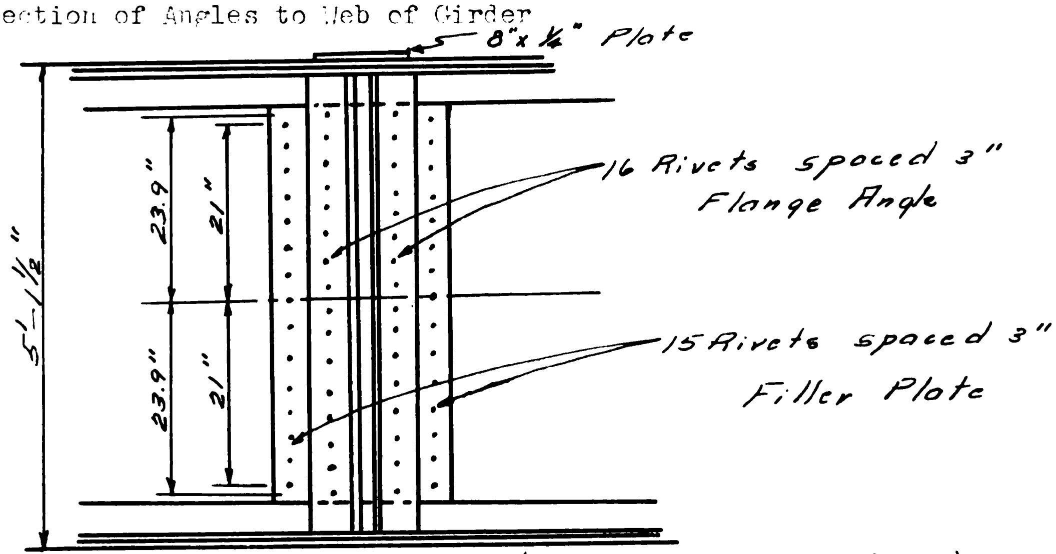
or 29900 ft. lbs.

189787 needed

The difference in BM supplied and that needed is offset by having a 1/4 by 8 in. Plate on top of cantilever and attach to beam on other side of Girder.

Connection of Cantilever to Girder

Test Connection of Angles to Web of Girder



Total V of Cantilever and Beam (V is the Reaction of beams)

53420 + 29887 = 83307 #

Bearing on 3/4 web and 2 9/16 bearing 27000 #/sq. in.

27000 (3/4 + 2 9/16) = 56000 #

Double Shear of 3/4 Rivet

2 .443 13500 = 11960 #

Number of Rivets for Shear = $\frac{83307}{11960}$ = 7 Rivets
supplied 32
16 in each angle
or 30 in Filler Plate

Bending Moment at end of Cantilever = 189727 ft. lbs.

1/4 by 8 in. Plate takes Tension while the bearing of angle on web of girder takes the compression

Assumed Neutral Axis at neutral axis of the cantilever at the end.

BM = 8 ¼ 16000 36 = 1150000 in. lbs.
or 96000 ft. lbs. supplied
189727 ft. lbs. needed

GIRDERS

Girders

Cross Section of Girders

2 angles-9/16 by 61/4 by 61/4 in. — 13.4 sq. in.

1 CP-9/16 by 141/4 in. — 8.02

total — 21.42 sq. in.

(83) 4 Holes 4 9/16 7/8 = 1.97 sq. in. angles

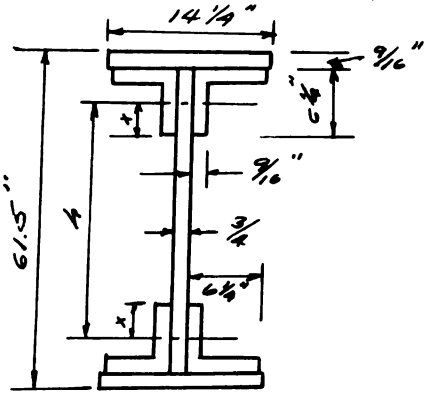
2 Holes 2 7/8 9/16 = 1.0 sq. in. CP

(85) $x = 1 - \frac{s^2}{4\,h\,g}$

$x = 1 - \frac{6^2}{4\ \ 31/4\ \ 7/8} = -2.164$ or 0 to be deducted

Net Section 21.42 - 1.97 - 1.0 = 18.45 sq. in.

Find h (h dist. from center of gravity of the flanges)



$21.45\,x = \frac{2\ \ 25\ \ 9\ \ 25}{4\ \ 16\ \ 8} + \frac{2\ \ 182\ \ 9\ \ 191}{58\ \ 16\ \ 32} + \frac{57\ \ 9\ \ 209}{4\ \ 16\ \ 32}$

x = 5.84 in.

h = 61.5 - 2 (6.88 - 5.84) = 59.34 in.

Compression of extreme fiber of rolled sections and built up girder

(46) $= 18000 - 5\,(L/b)^2$

$= 18000 - \frac{5\,(15.4\ \ 12)^2}{(14.25)^2} = 17157.5$ #/sq. in.

Weight of the Girder #/ft.

2-CP 9/16 in. by 14½ in. 60 ft. long @ 27.3 #
27.3 60/66 = 24.8 #/ft.

3/4" Web 5ft. 1in. @ 153 # 153

4-angles 6½ by 6½ by 9/16 in. @ 22.8 # 91.2

Concrete over Girder (girder II, III, IV)

$$\left(\frac{2.2+2.0\text{[illegible]}}{2}\right) . 0.5\ 150 + 1.5 \left(\frac{23/4+27/8}{2\ 12}\right) 150 = 205.9$$

total 474.9 #/ft.

8-Int. Stiffeners 3/8 by 3 by 5 in. @ 9.8 #
5 8 9.8 = 196 #

1000 Rivets @ 12 #/100 heads 12 20 = 240

12-End Stiffeners 3/8 by 3 by 5 in. @ 9.8 #
12 5 9.8 = 294

Sway Bracing 2-3½ by 3 by ½ in. angles
@ 4.5 22.5 2 4.5 = 202.5

20-Filler Plates 9/16 by 3/1 in. @ 5.74 #
4 20 5.74 = 458

2-CP Splice 5/8 by 14½ in. @ 30.3 #
4.9 30.3 2 = 298

total 1688.5 #

$$w = 474.9 + \frac{1688.5}{66} = 501 \text{ say } 510 \text{ \#/ft.}$$

Girder I

Dead Load

Conc. Dead Load

Max. Reaction of Beam between Girder I and II — 11840 #

Max. reaction of Canilever Beam — 29887

Panel Point -C one beam and one canilever — 41727 #

" " -D one beam and one canilever — 41727

" " -E one beam and one canilever — 41727

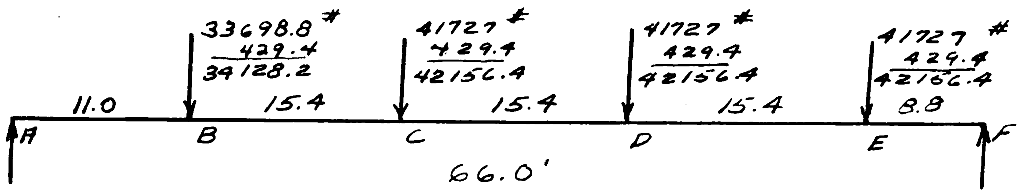
" " -B $11840 + \frac{12.1 \quad 11840}{15.4 \quad 2} = 10570$ Beam

$$\frac{29887.0}{2} + \frac{29887.0}{2 \; 15.4} \left(11 - \frac{11 \; 4.4}{16.5 \; 2} \right) = 23128.8 \quad \text{canilever}$$

totol DL at B $23128.8 + 10570 = 33698.8$ #

wt. of connection of beam and canilever to girder = 429.4#

Uniform Dead Load same as Girder V — 640.9 #/ft say 641



Max. Dead Load Bending Moment

DLBM at C

$$42156.4 \; \frac{26.4}{66} \left(8.8 + 24.2 + 39.6 \right) + 34128.2 \; \frac{55 \; 26.4}{66} - 34128.2 \; 15.4 = 1451 \text{ KF Max.}$$

$$+ \frac{641 \; 66^2}{8} = \frac{35}{1486} \text{ KF Total}$$

DLBM at D

$$42156.4 \; \frac{24.2}{66} \left(26.4 + 41.8 + 57.2 \right) + 34128.2 \; \frac{11 \; 24.2}{66} - 42156.4 \; 15.4 = 1430.4 \text{ KF}$$

$$+ \frac{641 \; 66^2}{8} = \frac{35}{1465.4} \text{ KF Total}$$

Max. Dead Load Shear

DLV at EF

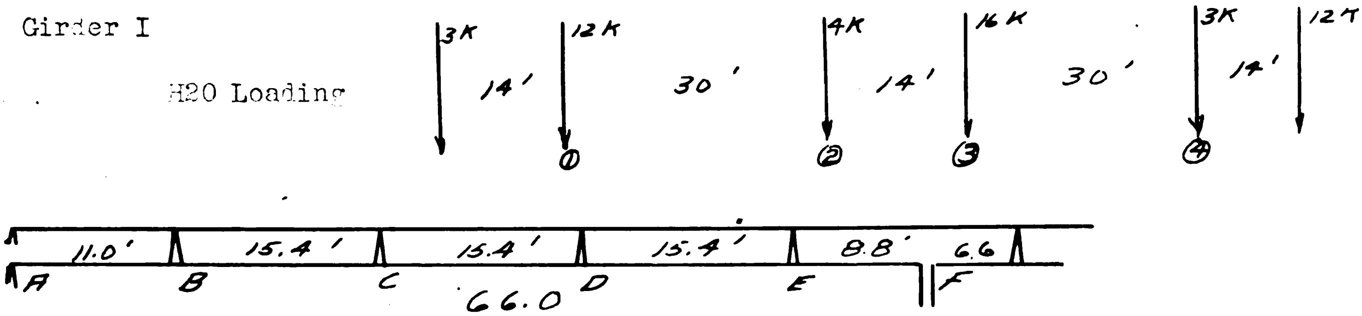
$$34128.2 \; \frac{11}{66} + \frac{42156.4}{66} \left(26.4 + 41.8 + 57.2 \right) + \frac{641 \; 66}{2} = 109030 \text{ \# Max.}$$

DLV at AB

$$\frac{42156.4}{66} \left(8.8 + 24.2 + 39.6 \right) + 34128.2 \; \frac{55}{66} + \frac{641 \; 66}{2} = 95850 \text{ \#}$$

Girder I

H20 Loading



Max. Live Load Bending Moment one wheel of a train of trucks

LLBM at C

Load 1 at C

$$16\,\frac{2.2\ \ 8.8\ \ 26.4}{15.4\ \ 66} + 4\,\frac{9.6\ \ 26.4}{66} + 12\,\frac{39.6\ \ 26.4}{66} + 3\,\frac{53.6\ \ 26.4}{66} - 3\ \ 14$$

$$= 236\ \text{KF}$$

Load 2 at C

$$3\,\frac{2.2\ \ 8.8\ \ 26.4}{15.4\ \ 66} + 16\,\frac{24.2\ \ 26.4}{66} + 4\,\frac{39.6\ \ 26.4}{66} = 220\ \text{KF}$$

Load 3 at C

$$12\,\frac{2.2\ \ 8.8\ \ 26.4}{15.4\ \ 66} + 3\,\frac{9.6\ \ 26.4}{66} + 16\,\frac{39.6\ \ 26.4}{66} + 4\,\frac{53.6\ \ 26.4}{66} - 4\ \ 14$$

$$= 301\ \text{KF}$$

LLBM at D

Load 1 at D

$$3\,\frac{27.8\ \ 24.2}{66} + 12\,\frac{41.8\ \ 24.2}{66} + 4\,\frac{0.8\ \ 57.2\ \ 24.2}{15.4\ \ 66} - \frac{0.8\ \ 4\ \ 15.4}{15.4} = 215.8\ \text{KF}$$

Load 2 at D

$$16\,\frac{10.2\ \ 41.8}{66} + 4\,\frac{24.2\ \ 41.8}{66} + 12\,\frac{54.2\ \ 41.8}{66} - 12\ \ 30 = 217\ \text{KF}$$

Load 3 at D

$$4\,\frac{27.8\ \ 24.2}{66} + 16\,\frac{41.8\ \ 24.2}{66} + 3\,\frac{0.8\ \ 57.2\ \ 24.2}{15.4\ \ 66} - 3\,\frac{0.8\ \ 15.4}{15.4} = 282\ \text{KF}$$

Max. Live Load Shear one wheel of train of trucks

LLV at AB

Load 2 at B

$$12\,\frac{3.6\ \ 8.8}{15.4\ \ 66} + 3\,\frac{11}{66} + 16\,\frac{41}{66} + 4\,\frac{55}{66} = 14.13\ \text{K}$$

Load 3 at B

$$12\,\frac{11}{66} + 3\,\frac{25}{66} + 16\,\frac{55}{66} = 16.45\ \text{K}$$

Girder I

Max. Live Load Shear -cont.

LLV at EF

Load 1 at E

$$12\,\frac{13.2}{66} + 3\,\frac{43.2}{66} + 12\,\frac{57.2}{66} = 14.75 \text{ K}$$

Load 2 at E

$$16\,\frac{\cancel{14}\ 57.2}{15.4\ 66} + 4\,\frac{57.2}{66} + 12\,\frac{27.2}{6\cancel{6}} + 3\,\frac{13.2}{66} = 10.28 \text{ K}$$

Load 3 at E

$$16\,\frac{57.2}{66} + 4\,\frac{43.2}{66} + 12\,\frac{13.2}{66} = 18.89 \text{ K Max.}$$

Bending Moment and Shear Totals

Max. LLBM (1 wheel) 391 KF
Max. LLV (1 wheel) 18.89 K

Impact 20.6%
Max. Reaction at Girder I is same as Girder V $\frac{27}{16.5}$

Total LLBM $391\,\frac{27}{16.5}$ 120.6 % = 593 KF

Total LLV $18.89\,\frac{27}{16.5}$ 120.6 % = 37.5 K

Max. DLBM 1486 K
Max. DLV 107030 #

Total Max. BM 1486 + 593 = 2079 KF
Total Max. V 107030 + 37500 = 144530 #

Girder I

Dead Load Shear at BC

$$\frac{42156.4}{66}(8.8+24.2+39.6)+34128.2\,\frac{55}{66}+641\,\frac{66}{2}$$
$$-\,34128.2-11\;641\;=\;54570.8\;\#$$

Dead Load Shear at ED

$$34128.2\,\frac{11}{66}+\frac{42156.4}{66}(26.4+41.8+57.2)+\frac{641\;66}{2}$$
$$-\,42156.4-8.8\;641\;=\;59232.8\;\#\qquad \text{Max.}$$

Live Load Shear at BC H20 Loading 1 wheel of train of trucks

Load 2 at C

$$3\,\frac{2.2\;8.8}{15.4\;66}+16\,\frac{25.6}{66}+4\,\frac{39.6}{66}\;=\;8.68\text{ K}$$

Load 3 at C

$$12\,\frac{2.2\;8.8}{15.4\;66}+3\,\frac{9.6}{66}+16\,\frac{39.6}{66}+4\,\frac{53.6}{66}-4\,\frac{14}{15.4}\;=\;9.86\text{ K}$$

Live Load Shear at DE

Load 2 at D

$$12\,\frac{11.8}{66}+4\,\frac{41.8}{66}+16\,\frac{55.8}{66}+16\,\frac{14}{15.4}\;=\;3.69\text{ K}$$

Load 3 at D

$$4\,\frac{27.8}{66}+16\,\frac{41.8}{66}+3\,\frac{.2\;57.8}{15.4\;66}-3\,\frac{.8}{15.4}\;=\;13.3\text{ K}\qquad\text{Max.}$$

Total Shear at DE

Max. Reaction at Girder I of 1# load on each wheel across Girder
(Reaction same as Girder V) 27/16.5

Impact = 20.6 %

$$\text{Total LLV} = 13.3\,\frac{27}{16.5}\,120.6\,\% \;=\; 26.3\text{ K}$$

DLV at DE = 59232.8 #

Total V at DE = 26300 + 59232.8 = 85532.8 #

Girder I

Check Girder Cross Section

Gross Section of Flange 21.42 sq. in.
Net " " " 18.45
Web 3/4 in. by 60 3/8 in.
h equals 58.34 in.

Tension Flange
f = 18000

$A = \frac{BM}{h\ f} - 1/8$ of web

$18.45 = \frac{2079000\ 12}{h\ 18000} - 1/8\ 3/4\ 603/8$

h = 57.8 in. supplied 58.34 in.

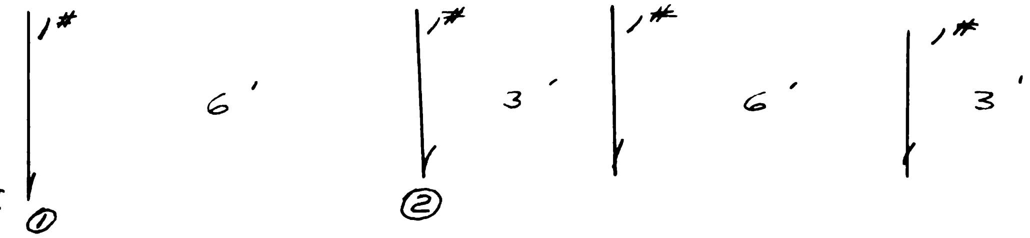
Compression Flange
f = 17157.5 #/sq. in.

$A = \frac{2079000\ 12}{18000\ 58.34} - 1/8\ 3/4\ 603/8$

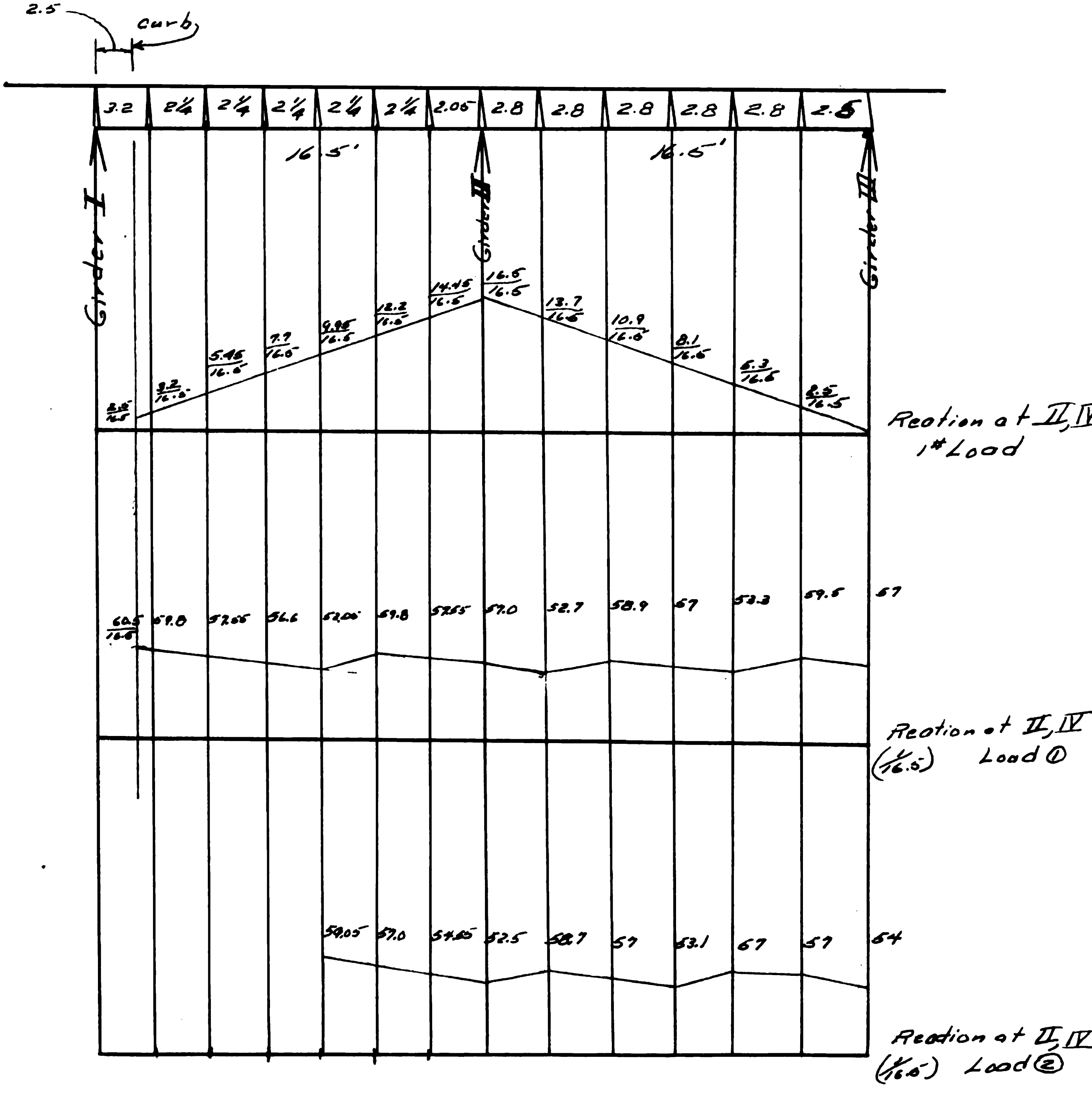
A = 18.12 sq. in. supplied 21.42

Web
v = 12000 allowable

$v = \frac{V}{A} = \frac{144530}{3/4\ 603/8} =$ 3190 #per sq. in.



Max. Reaction at Girder II & Girder IV
Influence Lines
for Traffic Lanes



Girder II

Dead Load

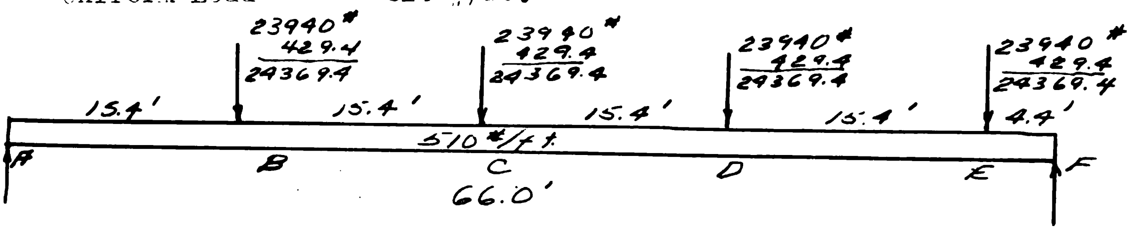
Conc. Dead Load

Max. Reaction of Beam between Girder	I and II	12920
" " " " " "	II and III	11020
Panel Points B, C, D, and E ------------------	tot.	23940

wt. of connections of two beams to girder

4-3/8 by 3 by 5 angles	4 5 9.8	=	1 6
2- back plates 9/16	$\frac{2\ 4\ 1\ 490\ 9}{16\ 12}$	=	134
2- triangles of web of beam 3/8 (lower side)	$2\ \frac{3\ \ 15\ 2.3}{8\ 12\ 12\ 2}\ 490$	=	44
2- triangles of web of beam 3/8 (upper side)	$2\ \frac{3\ 15\ .28\ 490}{8\ 12\ 12\ 2}$	=	5.37
		tot.	429.4 #

Uniform Load 510 #/ft.



Max. Dead Load Bending Moment

DLBM at C

$$\frac{24369.4\ 30.8}{66}\ (4.4 + 19.8 + 35.2 + 50.6) - 24369.4\ 15.4 + \frac{510\ 66^2}{8} = 902.9\ \text{KF}$$

Max. Dead Shear

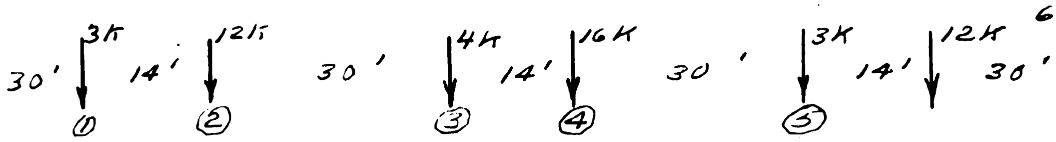
DLV at AB

$$\frac{24369.4}{66}\ (4.4 + 19.8 + 35.2 + 50.6) + \frac{510\ 66}{2} = 57445.7\ \#$$

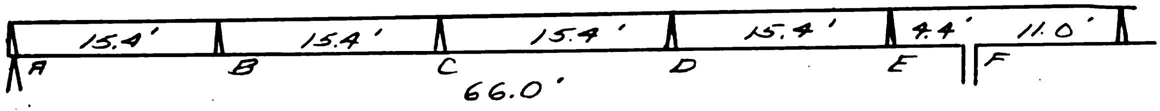
DLV at EF

$$\frac{24369.4}{66}\ (15.4 + 30.8 + 46.2 + 61.6) + \frac{510\ 66}{2} = 73630\ \#$$

Girder II



H 20 Loading



Max. Live Load Bending Moment one wheel of a train of trucks

LLBM at C

Load 2 at C

$$3\,\frac{16.8\ \ 35.2}{66} + 12\,\frac{30.8\ \ 35.2}{66} + 4\,\frac{60.8\ \ 35.2}{66} + 16\,\frac{2.2\ \ 61.6\ \ 35.2}{15.4\ \ 66}$$

$$-\ 4\ \ 30 - 16\,\frac{2.2\ \ 30.8}{15.} = 238.5\ \text{KF}$$

Load 3 at C

$$12\,\frac{0.8\ \ 35.2}{66} + 4\,\frac{30.8\ \ 35.2}{66} + 16\,\frac{44.8\ \ 35.2}{66} + 3\,\frac{2.2\ \ 61.6\ \ 35.2}{15.4\ \ 66}$$

$$-\ 16\ \ 14 - 3\,\frac{2.2\ \ 30.8}{15.4} = 230.8\ \text{KF}$$

Load 4 at C

$$4\,\frac{16.8\ \ 35.2}{66} + 16\,\frac{30.8\ \ 35.2}{66} + 3\,\frac{60.8\ \ 35.2}{66} + 12\,\frac{2.2\ \ 61.6\ \ 35.2}{15.4\ \ 66}$$

$$-\ 3\ \ 30 - 12\,\frac{2.2\ \ 30.8}{15.4} = 304.5\ \text{KF Max.}$$

Load 5 at C

$$16\,\frac{0.8\ \ 35.2}{66} + 3\,\frac{30.8\ \ 35.2}{66} + 12\,\frac{44.8\ \ 35.2}{66} + 3\,\frac{2.2\ \ 61.6\ \ 35.2}{15.4\ \ 66}$$

$$-\ 12\ \ 14 - 3\,\frac{2.2\ \ 30.8}{15.4} = 175.3\ \text{KF}$$

Max. Live Load Shear

LLV at AB

Load 2 at B

$$3\,\frac{1.4\ \ 50.6}{15.4\ \ 66} + \frac{12\ \ 50.6}{66} + \frac{4\ \ 20.6}{66} + \frac{16\ \ 6.6}{66} = 12.24\ \text{K}$$

Load 3 at B

$$\frac{4\ \ 50.6}{66} + \frac{16\ \ 36.6}{66} + \frac{3\ \ 6.6}{66} + \frac{12\ \ 3.6\ \ 4.4}{66} = 12.43\ \text{K}$$

Girder II

Max. Live Load Shear -cont.

LLV at AB

Load 4 at B

$$4\ \frac{1.4}{15.4}\ \frac{50.6}{66} + \frac{16\ 50.6}{66} + \frac{3\ 30.6}{66} + \frac{12\ 6.6}{66} = 14.28\ \text{K}$$

LLV at EF

Load 3 at E

$$3\ \frac{17.6}{66} + \frac{12\ 31.6}{66} + \frac{4\ 61.6}{66} + 16\ \frac{1.4}{15.4}\ \frac{61.6}{66} = 11.62\ \text{K}$$

Load 4 at E

$$\frac{16\ 61.6}{66} + 4\ \frac{47.6}{66} + \frac{12\ 17.6}{66} + 3\ \frac{3.6}{66} = 21.2\ \text{K Max.}$$

Bending Moment and Shear Totals

Max. LLBM (1 wheel) H20 304.5 KF
Max. LLV (1 wheel) H20 21.2 K

(37) Impact 20.6 %

(35) 4 Lanes 80%

Max. Reaction at Girder II of 1 Load each wheel across Girder $\frac{60.5}{16.5}$

Total LLBM $304.5\ 120.6\%\ 80\%\ \frac{60.5}{16.5} = 1075$ KF

Total LLV $21.2\ 120.6\%\ 80\%\ \frac{60.5}{16.5} = 75$ K

Max. DLBM ~~11/20.7~~ 902.9 KF
Max. DLV 73630 #

Total Max. BM 1075 + 902.9 = 1977.9 KF

Total Maz. V at EF 75000 + 73630 = 148630 #

Girder II

Dead Load Shear at DE

$$\frac{24369.4}{66}(15.4 + 30.8 + 46.2 + 61.6) + \frac{510\ 66}{2}$$
$$- 24369.4 - 4.4\ 510 = 47016.6\ \#\quad \text{Max.}$$

Dead Load Shear at BC

$$\frac{24369.4}{66}(4.4 + 19.8 + 35.2 + 50.6) + \frac{510\ 66}{2}$$
$$- 24369.4 - 15.4\ 510 = 25222.3\ \#$$

Live Load Shear at DE H20 Loading 1 wheel of train of trucks

Load 3 at D

$$3\ \frac{2.2}{66} + 12\ \frac{16.2}{66} + 4\ \frac{46.2}{66} + 16\ \frac{61.6}{66} - \frac{16\ 14}{15.4} = 6.25\ K$$

Load 4 at D

$$\frac{12\ 2.2}{66} + \frac{4\ 32.2}{66} + \frac{16\ 46.2}{66} + \frac{3.8\ 61.6}{15.4\ 66} - \frac{3\ .8}{15.4} = 13.544\ K\ \text{Max.}$$

Live Load Shear at BC H20 Loading 1 wheel of train of trucks

Load 3 at C

$$3\ \frac{2.2\ 4.4}{15.4\ 66} + \frac{16\ 21.2}{66} + \frac{4\ 35.2}{66} + \frac{12\ 55.2}{66} - 12 = 7.3\ K$$

Load 4 at C

$$\frac{12\ 2.2\ 4.4}{15.4\ 66} + 3\ \frac{5.2}{66} + 16\ \frac{35.2}{66} + 4\ \frac{49.2}{66} - \frac{4\ 14}{15.4} = 8.23\ K$$

Total Shear at DE

Max. Reaction at Girder II of 1# load on each wheel across Girder 60.5/16.5

Impact 20.6 %
4 Lanes 80 %

$$\text{Total LLV} = 13.544\ \frac{60.5}{16.5}\ 120.6\ \%\ 80\ \% = 48\ K$$

DLV at DE = 47016.6 #

Total V at DE = 48000 + 47016.6 = 95016.6 #

Girder II

Check Girder Cross Section

Gross Section of Flange 21.42 sq. in.
Net " " " 18.45
Web 3/4 in. by 60 3/8 in.
h equals 58.34 in.

Tension Flange
$f = 18000$

$$A = \frac{M}{h\ f} - 1/8 \text{ of web}$$

$$18.45 = \frac{1977900\ 12}{18000\ h} - 1/8\ 3/4\ 60 3/8$$

$h = 54.8$ in. supplied 58.34 in.

Compression Flange
$f = 17157.5$ #/in.^{2}

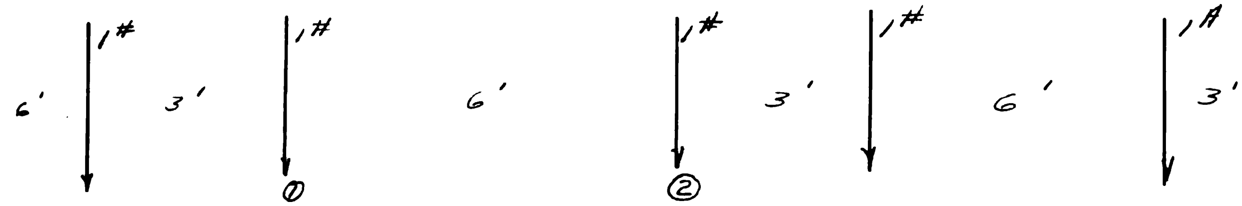
$$A = \frac{1977900\ 12}{17157.5\ 58.34} - 1/8\ 3/4\ 60 3/8$$

$A = 18.04$ sq. in. supplied 21.42

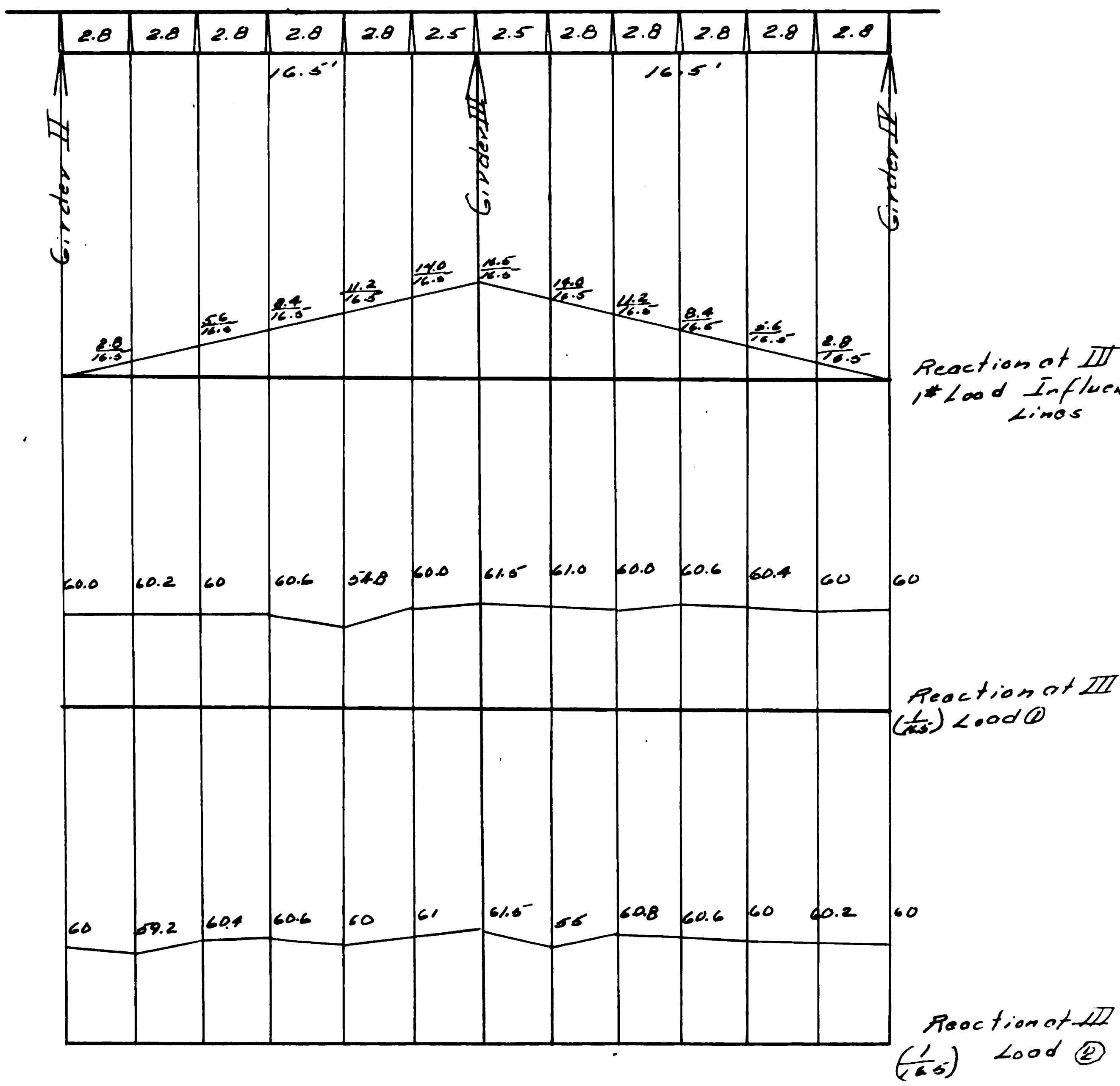
Web

$v = 12000$ allowable

$$v = \frac{V}{A} = \frac{148630}{60 3/8\ 3/4} = 5790 \text{ \#/in.}^2$$



Max. Reaction at Girder III
Influence Lines for Traffic Lanes.

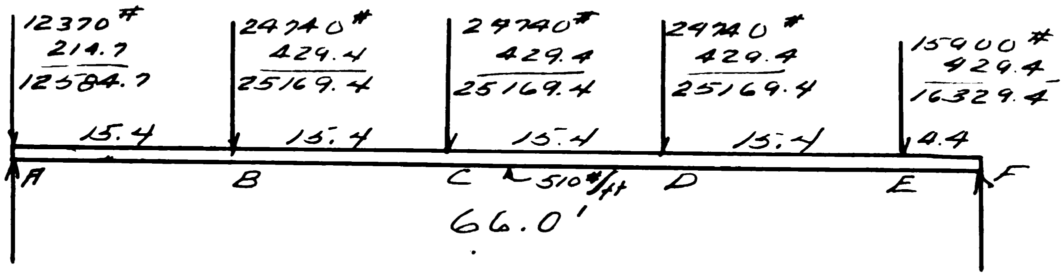


Dead Load
Conc. Dead Load
Ind. Reaction of Beam between Girder III and II, and Girder III and IV
12370#

Pan 1 Point-A	one beam	12370#	
" " -B	two beam	24740	
" " -C	" "	24740	
" " -D	" "	24740	
" " -E	½ 24740 + $\frac{4.4}{15.4}$ 24740½	=	15900

wt, of connection of two beams to girder 429.4#

Uniform Load 510 # per ft.



Max. Dead Load Bending Moment

DLBM at C

$$\frac{16329.4 \; 4.4 \; 30.8}{66} + \frac{25169.4 \; 30.8}{66} (19.8 + 35.2 + 50.6)$$

$$- 25169.4 \; 15.4 + \frac{510 \; 66^2}{8} = 915.9 \text{ KF}$$

Max. Dead Load Shear

DLV. at AB

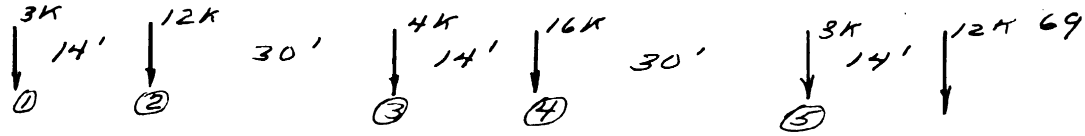
$$\frac{16329.4 \; 4.4}{66} + \frac{25169.4}{66} (19.8 \; 35.2 \; 50.6) + \frac{510 \; 66}{4} = 58.1? \text{ K}$$

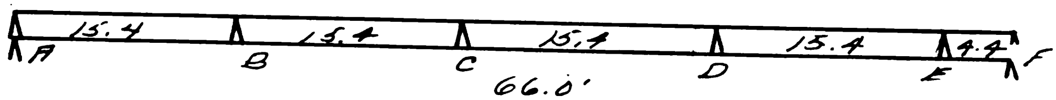
DLV. at EF

$$\frac{25169.4}{66} (15.4 \; 30.8 \; 46.2) + 16329.4 \frac{61.6}{66} + \frac{510 \; 66}{2} = 67.07 \text{ K}$$

Girder III

H 20 Loading





Max. Live Load Bending Moment one wheel of a train of trucks

LLBM at C

Load 2 at C

$$4\,\frac{52}{66}\,30.8 + 12\,\frac{35.2}{66}\,30.8 + 3\,\frac{49.2}{66}\,30.8 - 3\ 14 = 233\text{KF}$$

Load 3 at C

$$16\,\frac{21.2}{66}\,30.8 + 4\,\frac{35.2}{66}\,30.8 + 12\,\frac{65.2}{66}\,30.8 - 12\ 30 = 229\text{ KF}$$

Load 4 at C

$$3\,\frac{5.2}{66}\,30.8 + 16\,\frac{35.2}{66}\,30.8 + 4\,\frac{49.2}{66}\,30.8 + 4\ 14 = 306\text{ KF}$$ Max.

Max. Live Load Shear one wheel of a train of trucks

LLV. at AB

Load 2 at B

$$16\,\frac{6.6}{66} + 4\,\frac{20.6}{66} + 12\,\frac{50.6}{66} + 3\,\frac{14\ 50.6}{15.4\ 66} = 12.25\text{ K}$$

Load 3 at B

$$3\,\frac{6.6}{66} + 16\,\frac{36.6}{66} + 4\,\frac{50.6}{66} = 12.24\text{ K}$$

Load 4 at B

$$12\,\frac{0.66}{66} + 3\,\frac{20.6}{66} + 16\,\frac{50.6}{66} + 4\,\frac{1.4\ 50.6}{15.4\ 66} = 14.63\text{ K}$$

Max V at AB

Girder III

Max. LLV - cont.
LLV. at EF

Load 2 at E

$$3\frac{3.6}{66} + 12\frac{17.6}{66} + 3\frac{47.6}{66} + 12\frac{61.6}{66} = 16.74 \text{ K}$$

Load 3 at E

$$3\frac{17.6}{66} + 12\frac{31.6}{66} + 4\frac{61.6}{66} = 1?.7 \text{ K}$$

Load 4 at E

$$3\frac{3.6}{66} + 12\frac{17.6}{66} + 4\frac{47.6}{66} + 16\frac{61.6}{66} = 21.15 \text{ K} \quad \text{Max.}$$

Load 6 at E

$$4\frac{3.6}{66} + 16\frac{17.6}{66} + 3\frac{47.6}{66} + 12\frac{61.6}{66} = 17.?5 \text{ K}$$

BENDING MOMENT and SHEAR TOTALS

Max. LLBM (1 wheel) H20 306 KF

Max. LLV (1 wheel) H20 21.15 K

(37) Impact $\dfrac{L + 20}{6L + 20} = \dfrac{66 + 20}{6\ 66 + 20} = 20.6\%$

(33) 4 Lanes 80%

Max. Reaction at Girder III of 1 Load each wheel across Girder $\dfrac{61.5}{16.5}$

Total LLBM = $306\ 120.6\ 80\%\ \dfrac{61.5}{16.5} = 1100$ KF

Total LLV = $21.15\ 120.6\ 80\%\ \dfrac{61.5}{16.5} = 76$ K

Max. DLBM 913.9 KF
Max. DLV 67.07 K

Total Max. BM 1100 + 913.9 = 2013.9 KF

Total Max. V at EF 76 + 67.07 = 143.07 K

Girder III

Dead Load Shear at DE

$$\frac{25169.4}{66}(15.4+30.8+46.2)+16329.4\,\frac{61.6}{66}+\frac{510\ 66}{2}$$
$$-\ 16329.4-510\ 4.4 = 48496.6\ \#\quad \text{Max.}$$

Dead Load Shear at BC

$$16329.4\,\frac{4.4}{66}+\frac{25169.4}{66}(19.8+35.2+50.6)+\frac{510\ 66}{2}$$
$$-\ 25169.4-15.4\ 510 = 25157.6\ \#$$

Live Load Shear at BC H20 Loading 1 wheel of train of trucks

Load 3 at C

$$16\,\frac{21.2}{66}+4\,\frac{35.2}{66}+12\,\frac{65.2}{66}-12 = 7.17\text{ K}$$

Load 4 at C

$$3\,\frac{5.2}{66}+16\,\frac{35.2}{66}+4\,\frac{49.2}{66}-4\,\frac{1.4}{15.4} = 8.12\text{ K}$$

Live Load Shear at DE H20 Loading 1 wheel of train of trucks

Load 3 at D

$$3\,\frac{2.2}{66}+12\,\frac{16.2}{66}+4\,\frac{46.2}{66}+16\,\frac{60.2}{66}-\frac{16\ 14}{15.4} = 5.71\text{ K}$$

Load 4 at D

$$12\,\frac{2.2}{66}+4\,\frac{32.2}{66}+16\,\frac{46.2}{66} = 13.52\text{ K}\quad \text{Max.}$$

Total Shear at DE
Max. Reaction at Girder III of 1# load on each wheel across Girder
61.5/16.5

Impact 20.6 %
4 Lanes 80 %

$$\text{Total LLM} = 13.52\,\frac{61.5}{16.5}\ 120.6\ \%\ 80\ \% = 48.7\text{ K}$$

DLV at DE = 48496.6 #

Total V at DE = 48700 + 48496.6 = 97196.6 #

Girder III

Check Girder Cross Section

Gross Section of Flange 21.42 sq. in.
Net " " " 18.45
Web 3/4 in. by 60 3/8 in.
h equals 58.34 in.

Tension Flange
$f = 18000$ #/in

$A_s = \frac{BM}{h\ f} - 1/8$ of web

$18.45 = \frac{2013900\ \ 12}{18000\ \ h} - 1/8\ 603/8\ 3/4$

$h = 55.6$ in. supplied h of 58.34 in.

Compression Flange

$f = 18000$ #/in.^{2}

$A = \frac{2013900\ 12}{17137.5\ \ 58.34} - 1/8\ \ 603/8\ 3/4$

$A = 18.44$ in.^{2} supplied 21.45 in.^{2}

Web
$v = 12000$#/in.^{2} allowable

$v = \frac{V}{A}$

$v = \frac{143070}{603/8\ \ 3/4} = 3160$ #/in.^{2}

Girder IV

Dead Load

Conc. Dead Load

Max. Reaction of Beam between Girder III and IV — 11020 #
" " " " " " IV " V — 12920
2 940

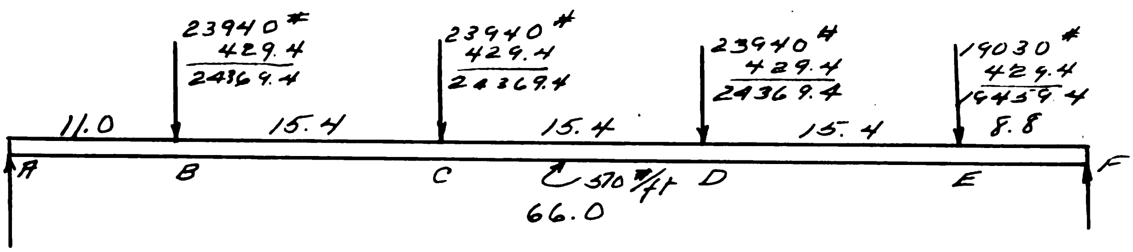
Panel Point -B two beams 23940
" " -C two beams 23940
" " -D two beams 23940
" " -E

$$\frac{11020}{2} + \frac{11020 \; 2\;7.7}{2\;15.4} = 8420$$

$$\frac{12920}{2} + \frac{12920 \quad 9.9\;2}{15.4\;2} = 10610$$

total DL at E ---------- - 19030

Uniform Dead Load 510 #/ft.



Max. Dead Load Bending Moment

DLBM at C

$$\frac{19459.4\;8.8\;26.4}{66} + \frac{24369.4\;26.4\;(24.2+39.6+55)}{66}$$

$$- 24369.4\;15.4 + \frac{510\;66^2}{8} = 87?.9 \;\; KF$$

DLBM at D

$$\frac{24369.4\;24.2\;(11+26.4+41.8)}{66} + \frac{19459.4\;24.2\;57.2}{66}$$

$$- 19459.4\;15.4 + \frac{510\;66^2}{8} = 830.9 \;\; KF$$

Max. Dead Load Shear

DLV at B

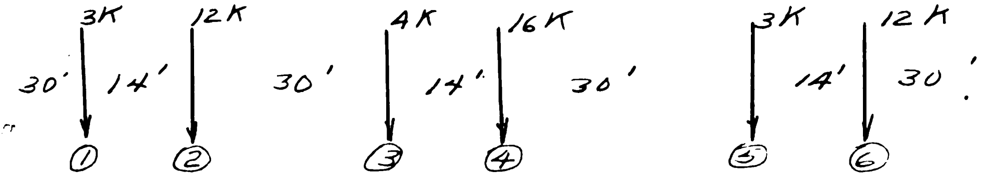
$$\frac{19459.4\;8.8}{66} + \frac{24369.4\;(24.2+39.6+55)}{66} + \frac{510\;66}{2} = 63.02 \;\; K$$

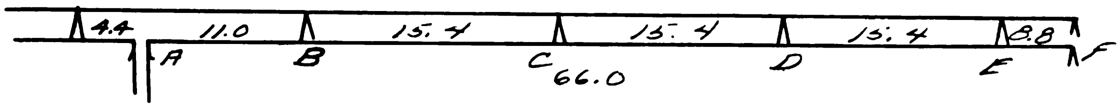
DLV at EF

$$\frac{24369.4\;(11+26.4+41.8)}{66} + \frac{19369.4\;57.2}{66} + \frac{510\;66}{2} = 62.53 \;\; K$$

Girder IV

H 20 Loading





Max. Live Load Bending Moment one wheel of a train of trucks

LLBM at C

Load 2 at C

$$3\,\frac{12.4\;39.6}{66} + 12\,\frac{26.4\;39.6}{66} + 4\,\frac{56.4\;39.6}{66} - 30\;4 = 228\ \text{KF}$$

Load 3 at C

$$4\,\frac{39.6\;26.4}{66} + 16\,\frac{25.6\;26.4}{66} + 12\,\frac{0.2\;55\;26.4}{15.4\;66} - 12\,\frac{0.2}{15.4}\,15.4 = 231.1\ \text{KF}$$

Load 4 at C

$$4\,\frac{12.4\;39.6}{66} + 16\,\frac{26.4\;39.6}{66} + 3\,\frac{56.4\;39.6}{66} - 3\;30 = 295\ \text{KF Max.}$$

LLBM at D

Load 4 at D

$$16\,\frac{41.8\;24.2}{66} + 4\,\frac{27.8\;24.2}{66} + 12\,\frac{2.2\;11\;24.2}{15.4\;66} = 293\ \text{KF}$$

Load 5 at D

$$4\,\frac{2.2\;11\;24.2}{15.4\;66} + 16\,\frac{11.8\;24.2}{66} + 3\,\frac{41.8\;24.2}{66} + 12\,\frac{55.8\;24.2}{66}$$

$$- 12\;14 = 195\ \text{KF}$$

Load 6 at D

$$12\,\frac{41.8\;24.2}{66} + 3\,\frac{27.8\;24.2}{66} + 16\,\frac{2.2\;11\;24.2}{15.4\;66} = 277.5\ \text{KF}$$

Girder IV

Max. Live Load Shear one wheel of a train of trucks

LLV at B

Load 2 at B

$$3\,\frac{1.4}{15.4}\,\frac{55}{66} + 12\,\frac{55}{66} + 4\,\frac{[illegible]5}{66} + 16\,\frac{11}{66} = 14.[illegible]5 \text{ K}$$

Load 3 at B

$$4\,\frac{55}{66} + 16\,\frac{41}{66} + 3\,\frac{11}{66} = 1[illegible].76 \text{ KF}$$

Load 4 at B

$$12\,\frac{11}{66} + 3\,\frac{25}{66} + 16\,\frac{55}{66} + 4\,\frac{1.4}{15.4}\,\frac{55}{66} = 16.76 \text{ K}$$

LLV at EF

Load 3 at E

$$3\,\frac{13.2}{66} + 12\,\frac{27.2}{66} + 4\,\frac{57.3}{66} = 9 \text{ K}$$

Load 4 at E

$$3\,\frac{3.6}{15.4}\,\frac{11}{66} + 12\,\frac{13.2}{66} + 4\,\frac{43.2}{66} + 16\,\frac{57.2}{66} = 19\text{K} \quad \text{Max.}$$

Load 5 at E

$$4\,\frac{13.2}{66} + 16\,\frac{27.2}{66} + 3\,\frac{57.2}{66} = [illegible] \text{ K}$$

Bending Moment and Shear Totals

Max. LLBM at C (1 wheel) H20 295 KF
Max. LLV at F (1 wheel) H20 19 K

(27) Impact 30.6 %
(3) 4 Lanes 90 %

Max. Reaction at Girder IV of 1# Load each wheel across Girder $\frac{60.5}{16.5}$

Total LLBM $295\ 130.6\%\ 90\%\ \frac{60.5}{16.5} = 1047$ KF

Total LLV $19\ 130.6\%\ 90\%\ \frac{60.5}{16.5} = [illegible]$ K

Max. DLBM [illegible]73.9 KF
Max. DLV 62.53 K

Total Max. BM 1047 + [illegible]73.9 = 1930.9 KF

Total Max. V [illegible] + [illegible] = [illegible] K

Girder IV

Dead Load Shear At BC

$$19459.4\,\frac{8.8}{66} + \frac{24369.4}{66}\,(24.2 + 39.6 + 55) + 510\,\frac{66}{2}$$
$$- 24369.4 - 11\;510 = 33040.6\ \#$$

Dead Load Shear at DE

$$\frac{24369.4}{66}\,(11 + 26.4 + 41.8) + 19562.4\,\frac{57.2}{66} + 510\,\frac{66}{2}$$
$$- 19459.4 - 510\;8.8 = 38582.3\ \# \quad \text{Max.}$$

Live Load Shear at BC H20 Loading 1 wheel of train of trucks

Load 3 at C

$$16\,\frac{25.6}{66} + 4\,\frac{39.6}{66} + .8\,\frac{12}{15.4}\,\frac{55}{66} - .8\,\frac{12}{15.4} = 8.5\ K$$

Load 4 at C

$$3\,\frac{9.6}{66} + 16\,\frac{39.6}{66} + 4\,\frac{53.6}{66} - 4\,\frac{14}{15.4} = 9.76\ K$$

Live Load Shear at DE H20 Loading 1 wheel of train of trucks

Load 3 at D

$$3\,\frac{2.2}{15.4}\,\frac{11}{66} + 12\,\frac{11.8}{66} + 4\,\frac{41.8}{66} + 16\,\frac{55.8}{66} - 16\,\frac{14}{15.4} = 3.74\ K$$

Load 4 at D

$$12\,\frac{2.2}{15.4}\,\frac{11}{66} + 4\,\frac{27.8}{66} + 16\,\frac{41.8}{66} = 12.1\ K \quad \text{Max.}$$

Total Shear at DE

Max. Reactio at Girder IV of 1/ load on each wheel across Girder 60.5/16.5

Impact 20.6 %

4 Lanes 80 %

$$\text{Total LLV} = 12.1\,\frac{60.5}{16.5}\,120.6\ \%\ 80\ \% = 42.8\ K$$

DLV at DE = 38582.3 #

Total V at DE = 42800 + 38582.3 = 81382.3 #

Girder IV

Check Girder Cross Section

Gross Section of Flange 21.42 Sq. in.
Net " " " 18.45
Web 3/4 in. by 60 3/8 in.
h equals 58.34 in.

Tension Flange

$f = 18000$

$A = \dfrac{BM}{h\ f} - 1/8$ of web

$18.45 = \dfrac{1920000\ 12}{18000\ \ h} - 1/8\ 3/4\ 60\ 3/8$

$h = 52$ in. supplied 58.34

Compression Flange

$f = 17157.5$

$A = \dfrac{1920900\ 12}{17157.5\ 58.34} - 1/8\ 3/4\ 60\ 3/8$

$A = 17.44$ supplied 21.45 sq. in.

Web

$v = 12000$ allowable

$v = \dfrac{V}{A} = \dfrac{124730}{3/4\ 60\ 3/8} = 2760$ #/ sq. in.

Girder V

Dead Load

Conc. Dead Load

Max. Reaction of Beam between Girder V and IV — 11840 #

Max. Reaction of Cantilever Beam — 29887

Panel Point-B	one beam and one cantilever	41727 #
Panel Point-C	one beam and one canilever	41727
Panel Point-D	one beam and one cantilever	41727

Panel Point-E $\frac{1}{2}\,11840 + \frac{12.1}{15.4}\,\frac{11840}{2} = 10580$ Beam

$\frac{1}{2}\,29887 + \frac{(13.2+1.47)\ 29887}{15.4\ \ 15.4\ \ 2} = 29173.5$ Cantilever

Total DL at E $10580 + 29173.5 = 39753.5$ #

wt. of connection of beam and cantilever to girder 429.4 #

Uniform Load #/ft.

2 CP	24.8 #/ft.
web	153.0
4 angles	91.2

concrete over girder

$\left(\frac{3.2}{2} + .5\right) .5 = 1.05$ sq. in.

$(1.6 + .5)\frac{5}{12} = 0.9$

$.5\ 7/12 = 0.3$

tot. 2.25 sq. in.

conc. load 1688.5 #

$w = \frac{1688.5}{66} + 24.8 + 153 + 91.2 + 2.25\ 150 = 640.9$ #/ft.

say 641

Max. Dead Bending Moment

DLBM at C

$$40182.9\,\frac{13.2}{66}\,22 + \frac{42156.4}{66}\,22\,(28.6+44+59.4) + \frac{641\ 66^2}{8} - 42156.4\ 15.4$$

$= 1416667.0$ ft. lbs.

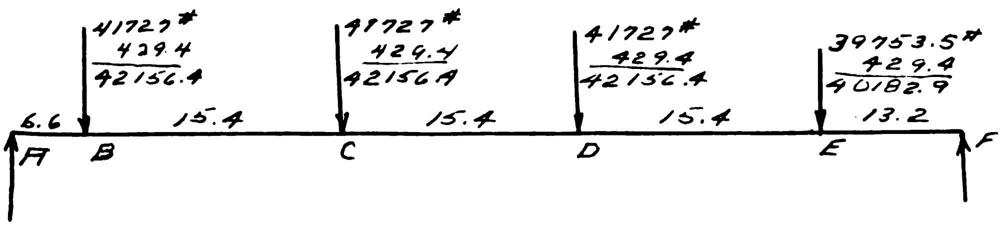
DLBM at D

$$\frac{42156.4}{66}\,(6.6+22+37.4)\,28.6 + \frac{40182.9\ 52.8}{66}\,28.6 - 40182.9\ 15.4 + \frac{641\ 66^2}{8} = 1545000 \text{ ft. lbs.}$$

Max. Dead Load Shear

DLV at AB

$$40182.9\,\frac{13.2}{66} + \frac{42156.4}{66}\,(28.6+44+59.4) + \frac{641\ 66}{2} = 113502.8\ \#$$

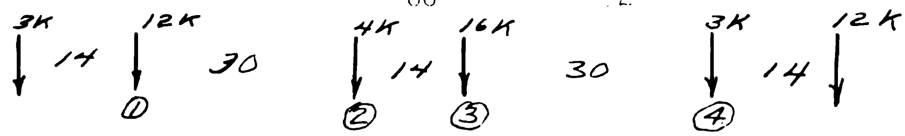


Girder V

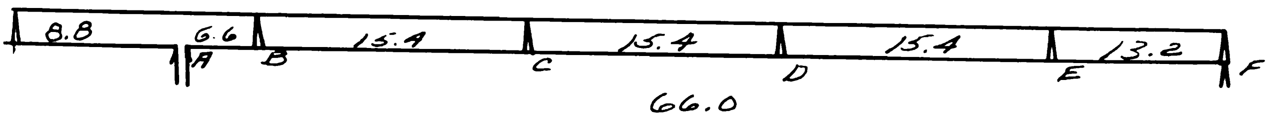
Max. Dead Load Shear - cont.

DLV at EF

$$\frac{48156.4}{66} (6.6 + 22 + 37.4) + 40102.9 \frac{52.[illegible]}{66} + 641 \frac{66}{2} = 9[illegible]6.4$$



H 20 Loading



Max. Live Load Bending Moment one wheel of a train of trucks

LLBM at C

Load 1 at C

$$4 \frac{14\ 22}{66} + 12 \frac{44\ 22}{66} + 3 \frac{58\ 22}{66} - 3\ 14 = 144 \text{ KF}$$

Load 2 at C

$$16 \frac{30\ 22}{66} + 4 \frac{44\ 22}{66} = 204 \text{ KF}$$

Load 3 at C

$$3 \frac{14\ 22}{66} + 16 \frac{44\ 22}{66} + 4 \frac{58\ 22}{66} - 4\ 14 = 270 \text{ KF}$$

LLBM at D

Load 2 at D

$$16 \frac{51.4\ 28.6}{66} + 4 \frac{37.4\ 28.6}{66} + 12 \frac{7.4\ 28.6}{66} - 16\ 14 = 286 \text{ KF}$$

Load 3 at D

$$16 \frac{28.6\ 37.4}{66} + 4 \frac{23.4\ 28.6}{66} = 300 \text{ KF} \quad \text{Max.}$$

Load 4 at D

$$16 \frac{7.4\ 28.6}{66} + 3 \frac{37.4\ 28.6}{66} + 12 \frac{51.4\ 28.6}{66} - 12\ 14 \neq 199 \text{ KF}$$

Girder V

Max. Live Load Shear one wheel of a train of trucks

LLV at AB

Load 1 at B

$$16\,\frac{15.4}{66} + 4\,\frac{29.4}{66} + 12\,\frac{59.4}{66} + 3\,\frac{1.4\;59.4}{15.4\;66} = 16.3\ K$$

Load 2 at

$$3\,\frac{15.4}{66} + 16\,\frac{45.4}{66} + 4\,\frac{59.4}{66} = 15.3\ K$$

Load 3 at B

$$12\,\frac{15.4}{66} + 3\,\frac{29.4}{66} + 16\,\frac{59.4}{66} + 4\,\frac{1.4\;59.4}{66\;15.4} = 18.86\ K\ \text{Max.}$$

LLV at EF

Load 2 at E

$$3\,\frac{9.8}{66} + 12\,\frac{22.8}{66} + 4\,\frac{52.8}{66} = 7.75\ K$$

Load 3 at E

$$3\,\frac{3.6\;6.6}{15.4\;66} + 12\,\frac{8.8}{66} + 4\,\frac{38.8}{66} + 16\,\frac{52.8}{66} = 16.83\ K$$

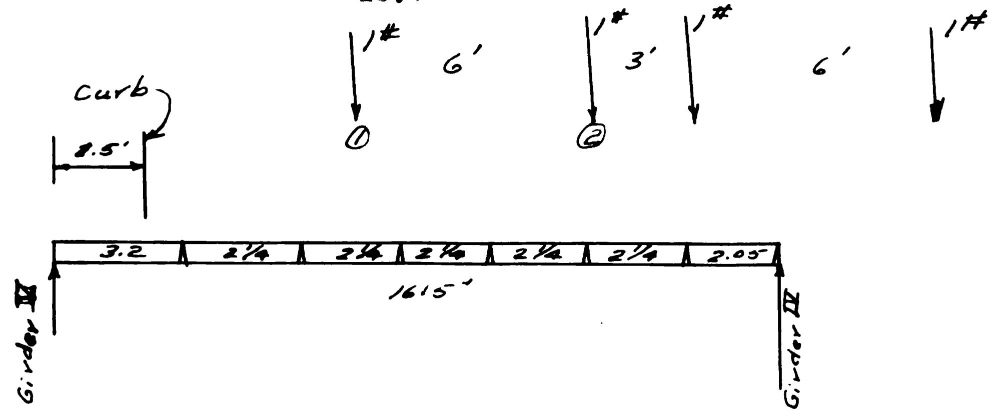
Load 4 at E

$$4\,\frac{8.8}{66} + 16\,\frac{22.8}{66} + 3\,\frac{52.8}{66} = 8.45\ K$$

Max. Reaction at Girder V of 1# Load on each wheel across Girder

// and Max. when wheel 1 is closest to curb

$$16.5\,(14 + 5 + [illegible]) = \frac{27}{16.5}$$



Girder V

Dead Load Shear at DE

$$\frac{42156.4}{66}(6.6+22+37.4)+\frac{40182.9\ 52.8}{66}+\frac{641\ 66}{2}$$

$$-40182.9-13.2\ 641 = 46762.3\ \#$$

Dead Load Shear at BC

$$40182.9\frac{13.2}{66}+\frac{42156.4}{66}(28.6+44+59.4)+\frac{641\ 66}{2}$$

$$-42156.4-6.6\ 641 = 67115.8\ \#\quad \text{Max.}$$

Live Load Shear at BC   H20 Loading   1 wheel of train of trucks

Load 2 at C

$$16\frac{30}{66}+4\frac{44}{66}+12\frac{.8}{15.4}\ .\frac{59.4}{66}-.8\frac{12}{15.4} = 8.406\ K$$

Load 3 at C

$$\frac{3\ 14}{66}+\frac{16\ 44}{66}+\frac{4\ 58}{66}+\frac{4\ 14}{15.4} = 11.18\ K\quad \text{Max.}$$

Live Load Shear at DE   H20 Loading   1 wheel of train of trucks

Load 2 at D

$$3\frac{2.2\ 6.6}{15.4\ 66}+12.0\frac{7.4}{66}+4\frac{37.4}{66}+16\frac{51.4}{66}-\frac{16\ 14}{15.4} = 1.95\ K$$

Load 3 at D

$$12\frac{2.2\ 6.6}{15.4\ 66}+\frac{4\ 23.4}{66}+\frac{16\ 37.4}{66} = 10.63\ K$$

Total Shear at BC

Max. Reaction at Girder V of 1# load on each wheel actoss Girder 27/16.5

Impact = 20.6 %

$$\text{Total LLV} = 11.18\frac{27}{16.5}\ 120.6\ \% = 22.1\ K$$

DLV at BC = 67115.8 #

Total V at BC = 67115.8. + 22100 = 89215.8 #

Girder V

Bending Moment and Shear Totals

Max. LLBM (1 wheel) 300 KF
Max. LLV (1 wheel) 18.86 K

(37) Impact $= \frac{L + 20}{6 L + 20} = \frac{66 + 20}{6\ 66 + 20} = 20.6\ \%$

Max. Reaction at Girder V of 1 # Load on each wheel across Girder
27/ 16.5

Total LLBM = 300 120.6 % 27/16.5 = 592 KF

Total LLV = 18.86 120.6 % 27/16.5 = 37.15 K

Max. DLBM 1545 KF
Max. DLV 113502.8 #

Total Max. BM 1545 + 592 = 2137 KF
Total Max. V 37150 + 113502.8 = 150652.8 #

Check Girder Cross Section

Gross Section of Flange 21.42 sq. in.
Net " " " 18.45
Web 3/4 in. by 60 3/8 in.
h equals 58.34 in.

Tension Flange f = 18000 #/sq. in.

$A = \frac{BM}{f\ h} - 1/8$ of web

$18.45 = \frac{2137000\ 12}{18000\ \ h} - 1/8\ 3/4\ 603/8$

h = 58.7 supplied 58.34 in.

Compression Flange f = 17157.5 #/sq. in.

$A = \frac{2137000\ 12}{17157.5\ 58.34} - 1/8\ 3/4\ 603/8 = 18.45$ sq. in.
supplied 21.42

Web v = 12000 #/sq. in. allowable

$v = \frac{V}{A} = \frac{150652.8}{603/8\ 3/4} = 3330$ #/sq. in.

Rivets in Girder First Panel (between end and first Beam)

Flange Angle Rivets

Girder II

Max. V 148630 #

Girder II has uniform load of concrete on top of flange of 205.9 #/ft.

Due to the fact that Girder II is in contact with concrete slab, there is a live load applied to top of Flange of Girder which is the Max. LLV (H20) of Slab between Girder and 12 in. @ 25 # CB Stringer and is equal to 4480 # on a ft. of slab.

Total $w = 4480 + 205.9 = 4685.9$ #/ft.

$$HI = \frac{V}{h} \times \frac{A_f}{A_f + A_w/8}$$

$$= \frac{148630}{58.34} \times \frac{21.42}{21.42 + 5.66/8} = 2020 \#$$

Bearing = 15200 #

Double Shear = 11960 #

$$\text{Pitch} = \frac{R}{\sqrt{HI^2 + w^2}} = \frac{11960}{\sqrt{2020^2 + (4685.9/12)^2}} = 5.72 \text{ in.}$$

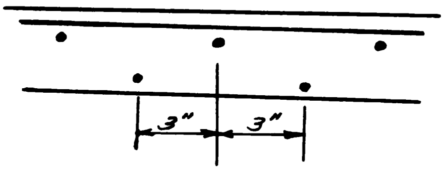
supplied 3 in.

Rivets in Girder First Panel (between end and first beam)

Max. V of Each Girder

Girder I	144580 #	
Girder II	148630	
Girder III	145070	
Girder IV	134730	
Girder V	150652.8	Max.

Flange Angle Rivets



Girder V

Girder V has a uniform load of concrete on top of flange of
2.25 150 = 337.5 #/ft.

No live load on flange, because of direct contact of Girder with Slab, because the curb keeps wheels at a distance of 2.5 ft. from center line of Girder

$$HI = \frac{V}{h} \times \frac{A_f}{A_f + A_w/8}$$

$$= \frac{150652.8}{53.34} \times \frac{21.42}{21.42 + 5.66} = 2040 \ \#$$

Bearing of 3/4 Rivet on 3/4 Web Bearing = 27000 #/sq. in.
3/4 3/4 27000 = 15200 #

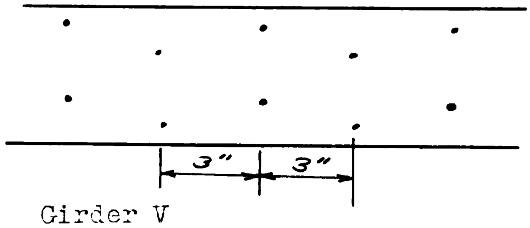
Double Shear of 3/4 Rivet shear 13500 #/sq. in.
2 .443 13500 = 11960 #

$$\text{Pitch} = \frac{R}{\sqrt{HI^2 + w^2}} = \frac{11960}{\sqrt{2040^2 + (337.5/12)^2}} = 4.85 \text{ in.}$$

supplied 3 in.

Rivets in Girder   First Panel (between end and first Beam)

Cover Plate Rivets



Girder V

$V = 150652.8$ #

$h = 58.34$ in.

$$HI = \frac{V}{h} \times \frac{A_c}{A_f + A_w/8} = \frac{150652.8}{58.34} \times \frac{8.02}{21.48 + 5.66} = 765 \text{ \#}$$

Bearing of 1 Rivet on 9/16 in. Flange

9/16 3/4 27000 = 11400 #

Shear one Rivet 3/4

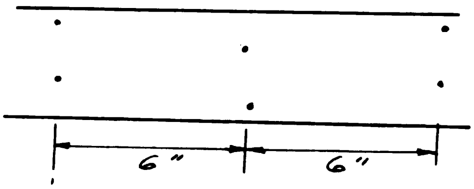
.443 13500 = 5980 #

$$\text{pitch} = \frac{R\ 2}{V/h} = \frac{2\ 5980}{765} = 15.6 \text{ in.}$$

supplied 3 in.

Rivets in Girder   Second Panel (between first and second Beam)

Cover Plate Rivets



Girder III

$V = 97196.6$ #

$h = 58.34$ in.

$$HI = \frac{V}{h} \times \frac{A_c}{A_f + A_w/8} = \frac{97196.6}{58.34} \times \frac{8.02}{21.48 + 5.66} = 494 \text{ \#}$$

$$\text{Pitch} = \frac{2\ R}{V/h} = \frac{2\ 5980}{494} = 24.2 \text{ in.}$$

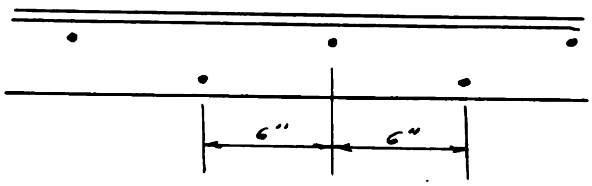
supplied 6 in.

Rivets in Girder   Second Panel (between first and second Beam)

Max. V of Each Girder between first and second Beam

Girder I	85532.8 #	
Girder II	95016.9	
Girder III	97196.6	Max.
Girder IV	81382.3	
Girder V	89815.8	

Flange Angle Rivets



Girder III

Girder III has a uniform load of concrete on top of flange of 205.9 #/ft.

Due to fact that Girder III is in contact with surface there is a Live Load applied to top of Flange of Girder, which is the Max. LLv (H20) of slab between Girder III and 12 in. @ 31.5 # I Bm. Stringer of 3500 # on a ft. of slab.

Total w = 205.9 + 4500 = 4705.9 #/ft.
h = 58.34 in.

$$HI = \frac{V}{h} \times \frac{A_f}{A_f + A_w/8}$$

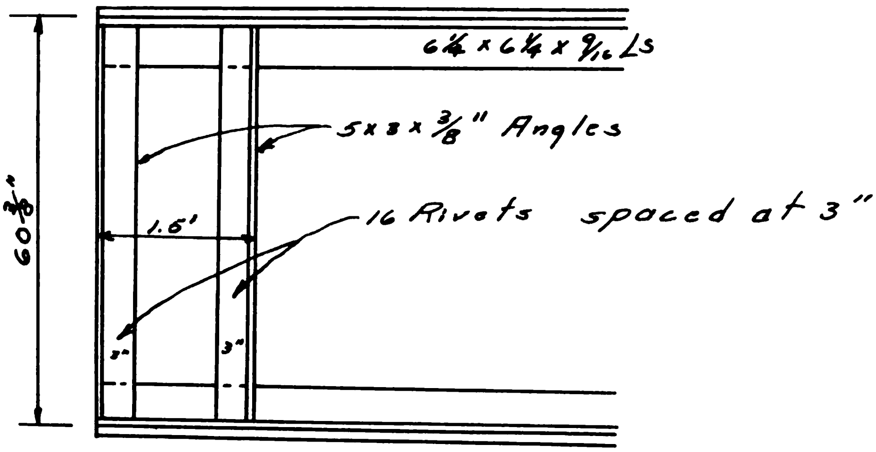
$$= \frac{97196.6}{58.34} \times \frac{21.48}{21.48 + 5.66} = 1320 \#$$

Bearing = 15800 #
Double Shear = 11960 #

$$\text{Pitch} = \frac{P}{\sqrt{HI^2 + w^2}} = \frac{11960}{\sqrt{1320^2 + (4705.9/12)^2}} = 8.7 \text{ in.}$$

supplied 6 in.

End Stiffeners (127)(c)



Girder V has the Max. V which is the Reaction at End
V = 150652.8 #

Double Shear of 3/4 Rivets shear = 13500 #/sq. in.
2 .443 13500 = 11960 #

Bearing of 3/4 Rivet on 3/4 Web of Girder bearing 27000 #
3/4 3/4 27000 = 15190

Number of Rivets $= \frac{150652.8}{11960} =$ 12.6 Rivets
supplied 32
16 in pair of angles

Area $= \frac{150652.8}{27000} =$ 5.57 sq. in. Needed

Supplied 4-5 by 3 by 3/8 in. Angles

A = 4 3/8 (5 - 3/8) = 6.95 sq. in.

Intermediate Stiffeners (128)(b)

Clear Distance between Flange Angles

$60\,3/8 - 2(6\tfrac{1}{4}) = 47.87$ in.

60 times the thickness of web $60 \cdot 3/4 = 45$ in.

47.87 is more than 45; therefore need stiffeners

Outstanding Leg of angle shall width of not more than 16 times its thickness $16 \cdot 3/8 = 6$ in. supplied 5 in.

and not less than (2 in. + 1/30 of depth of web)

$2 + 1/30 \cdot 60\,3/8 = 4$ in. supplied 5 in.

Intervals between stiffeners should not more than 72 in. nor that given by formulae.

$$d = \left(\frac{255000\ t}{s}\right)\sqrt[3]{\frac{s\ t}{a}}$$

d = clear distance between stif.

t = thickness of web

s = Shear, in pounds per sq. in. in the gross vertical sect. of the web at the point considered

a = clear depth of web between flanges of side plates

Girder I V = 14450 # EF s = 3190 #/sq. in.

V = [illegible]8.8 DE s = 1940

EF $d = \frac{255000 \cdot 3/4}{3190}\sqrt[3]{\frac{3190 \cdot 3/4}{47.87}} = 221$ in.

supplied [illegible] in.

DE $d = \frac{255000 \cdot 3/4}{1940}\sqrt[3]{\frac{1940 \cdot 3/4}{47.87}} = 318$ in.

supplied [illegible]5 in.

Intermediate Stiffeners (cont.)

Girder II   V = 148480 # EF   s = 3720 #/sq. in.
V = 95016.6 DE   s = 2100

EF $d = \frac{255000\ 3/4}{3720} \sqrt[3]{\frac{3720\ 3/4}{47.87}}$ = 197 in.
supplied 48 in.

DE $d = \frac{255000\ 3/4}{2100} \sqrt[3]{\frac{21000\ 3/4}{47.87}}$ = 281 in.
supplied 60 in.

Girder III   V = 145070 # EF   s = 3160 #/sq. in.
V = 97196.6 DE   s = 2150

EF $d = \frac{255000\ 3/4}{3160} \sqrt[3]{\frac{3160\ 3/4}{47.87}}$ = 225 in.
supplied 47 in.

DE $d = \frac{255000\ 3/4}{2150} \sqrt[3]{\frac{2150\ 3/4}{47.87}}$ = 286 in.
supplied 60 in.

Girder IV   V = 124730 # EF   s = 2760 #/sq. in.
V = 81382.3 DE   s = 1800

EF $d = \frac{255000\ 3/4}{2760} \sqrt[3]{\frac{2760\ 3/4}{47.87}}$ = 240 in.
supplied 87 in.

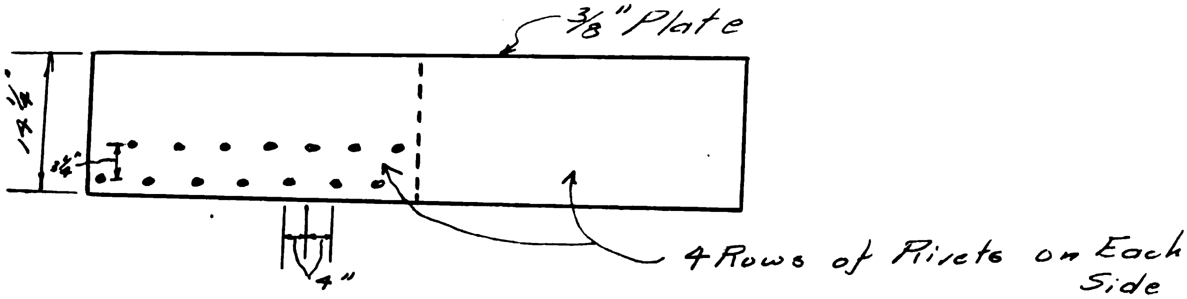
DE $d = \frac{255000\ 3/4}{1800} \sqrt[3]{\frac{1800\ 3/4}{47.87}}$ = 322 in.
supplied 83 in.

Girder V   V = 150652.8 # AB   s = 3530 #/sq. in.
V = 89215.8 BC   s = 1880

AB $d = \frac{255000\ 3/4}{3530} \sqrt{\frac{3530\ 3/4}{47.87}}$ = 215 in.
supplied 50 in.

BC $d \ \frac{255000\ 3/4}{1880} \sqrt[3]{\frac{1880\ 3/4}{47.87}}$ = 315"
supplied 83"

Girder Cover Plate Splice



Cover Plate 9/16 by 14½ in.
Gross Section = 8.02 sq. in.
Net section

(83) 2-3/4 Rivets
$x = 1 - \frac{3^2}{4 \; 3\frac{1}{2} \; 7/8} = 0.211$ of 2 Holes

2 7/8 9/16 + .21 2 7/8 9/16 = 1.21 sq. in.
Net Sec. = 8.02 - 1.21 = 6.81 sq. in.

Cover Plate Splice 5/8 by 14½ in.
Gross Section = 8.9 sq. in.
Net Section

(83) 2-3/4 Rivets
$x = 1 - \frac{3^2}{4 \; 3\frac{1}{2} \; 7/8} = 0.21$ of 2 Holes

2 7/8 5/8 = 1.1 sq. in.
0.21 2 7/8 5/8 = 0.22 sq. in.
tot. 1.32 sq. in.

Net Sec. = 8.9 - 1.32 = 7.58 sq. in. supplied

(127) (e) Net Sec. of Splice ≥ 110 % of Net Sec. of CP
110 % 6.81 = 7.5 sq. in. supplied 7.58

Tension Side
Bearing of one Rivet Bearing 27000 #/sq. in.
3/4 27000 9/16 = 11400 #
Single Shear of one Rivet Shear 13500 #/sq. in.
.4418 13500 = 5980 #

Number of Rivets = $\frac{6.81 \; 18000}{5980}$ = 20.5 supplied 28

Girder Cover Plate Splice

Compression side

$f = 17157.5$ #/sq. in.

Number of Rivets $= \dfrac{8.0 \; 17157.5}{5000} =$ 25 Rivets

supplied 28

Conclusion:

Although, it may be satisfactory for the bridge to have H20 Loading, the safety factor is cut down considerably in a few places. The Girders and Stringers will carry this loading without sacrificing any of their safety factor, but in the Beams and Slab this loading will raise stresses beyond that allowed in the specifications.

The greatest failure occurred in carrying bending moment, by continuous action of beams, into the connection of the beams to the girder. The bending moment of the beam could be taking care of within the beam itself if the fiber stress in the flange area were raised to about 30,000 #/sq. in.; therefore, by having a freely supported beam, the connection would more than satisfactory carry the maximum reaction at the end of the beams.

It may be possible that this bridge should have been analyzed with H15 Loading instead of H20, but according to specification number 21 the bridge seems in that class of bridges which should be designed with H20 Loading.

Pocket has: 1 Suppl.