II/9
Steel Superstructures

Introduction

This chapter presents two designs for short span bridge superstructures constructed entirely of structural steel. Both are designed for prefabrication in the shop in 8 foot sections. This width was chosen to allow ease of handling, transportation and erection.

All-steel designs require considerable fabrication and in short spans are not expected to have the most favorable first costs unless they are mass produced. However, they do have certain other advantages. Among these are the ease and rapidity with which they can be erected to form a complete superstructure; and their low maintenance costs when constructed of corrosion resistant steels.

Design I is for a pre-fabricated all steel sectional multi-cell girder bridge. Top and bottom flanges are continuous horizontal plate separated by closely spaced inclined webs, intermediate diaphragm at mid-span and vertical transverse plates at the end. Elastomeric pads are used for bearings.

Although illustrated for job fabrication in 8 ft sections, it can be made in width multiples of 2 feet without any major revision in design and details. Inclined webs of bent plate are used to reduce both the number of components and the number of welding operations.

Since the girder is a torsionally rigid closed box, it has advantageous load distributing qualities. Thus, the over-all depth for a 50 ft span is less than 27″, compared with the usually required 37″ for a conventional composite rolled beam bridge of identical span. The design is compact, neat in appearance and provides maximum clearance or minimum grades for the bridge approaches. It is a bridge with the majority of surface area enclosed and with exposed surfaces flat and easy to maintain.

Calculations and drawings illustrated are for a 50 ft simple span and a continuous unit of 2—50 ft spans. Average weight of the steel in the simple span is 45 lbs per sq ft of roadway area. Each section weighs approximately 9 tons.

Design II utilizes a stiffened steel plate deck supported on a system of floor beams and girders. By serving as the top flange, the deck acts as an integral part of the girders. Floor beams support the stiffened steel deck and serve as diaphragms, providing transverse stiffness to the structure. Closed ribs stiffen the deck plate. Field splices, either welded or bolted, are provided in the floor beams between girders, underneath longitudinally welded floor plate splices, so that each section to be transported and erected is approximately eight feet in width.

The structure utilizes standard shop fabricating procedures and makes efficient use of material. One aspect of this design that would make it attractive in quantity fabrication is the use of the same size deck plate, deck stiffeners, and floor beams throughout the 40 to 80 foot span range. The system is so designed that at critical points all elements are stressed near the allowable limits, both in bending and in shear. The resulting light weight of the structure, 37 lbs of steel per square ft for the illustrative example, permits further savings in the substructure costs.

Designs were made utilizing USS COR-TEN Steel, ASTM / A36 steel, 12 gage ribs, 3/16″ ribs and girder webs varying from 6 gage to the unstiffened thickness required for an 80′ span. A table on the drawings summarizes the material requirements for simple spans from 40′ to 80′. USS COR-TEN Steel was selected for the illustrative 50′ simple span design because of its high strength and its resistance to corrosion (permitting the structure to be left unpainted.)

Design I

ASSUMPTIONS

This highway bridge is designed for HS-20 live loading. Vertical deflection, limited to $\frac{1}{800}$ of the horizontal span, is the controlling design factor.

It is designed as an orthotropic steel plate deck bridge. The continuous top plate serves as the top flange of the box girder, the roadway deck and as transverse flexural member between webs. The method outlined in the AISC Design Manual for Orthotropic Steel Plate Bridge is used in designing the top plate.

Each 8′ section is assumed to take two-thirds of a lane of live loading, based on a 12′ traffic lane. This same coefficient is used in the stress and deflection calculations.

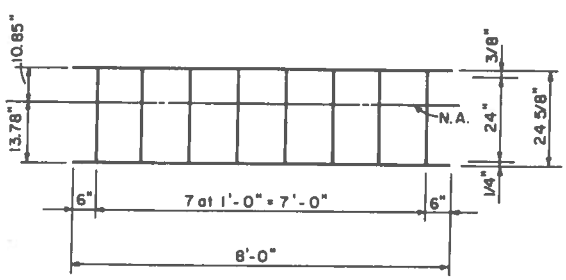
The final design was investigated for load distribution by Guyon-Massonnet theory and charts. Load distribution for the outside 8′ section varies from 0.520 lane to 0.743 lane, with an average of 0.639 lane; this is close to the load distribution in the original design assumption.

While the $\frac{3}{16}''$ inside webs and $\frac{1}{4}''$ bottom plate are less than the thickness required by AASHO Specifications, the inside webs have both sides enclosed and the bottom plate has one side enclosed in the airtight box to protect against corrosion. (See Figures 1 and 2.)

Design Calculations

Assume:
- Live Load distribution = ⅔ Lane/8′ width*
- Weight of structure = 45#/ft.²
- Weight of wearing surface = 25#/ft.²

50 FT SIMPLE SPAN



Top Floor PL 96 × ⅜ (A441)	36.00	15.3
8 Webs 24 × $\frac{3}{16}$ (A 36)	36.00	15.3
Bott. PL 96 × ¼ (A 36)	24.00	10.2
	96.00 in.²	40.8 psf.

$$N.A. = [36.00 \times 0.19 + 36.00\,(0.38 + 12) + 24.00\,(0.38 + 24 + 0.12)] \times \frac{1}{96.00}$$
$$= [7 + 446 + 588] \times \frac{1}{96.00} = \frac{1041}{96.00} = 10.85''$$

$$I_{NA} = 36.00\,(10.85 - 0.19)^2 + 36.00\,(12.38 - 10.85)^2 + 24.00\,(24.50 - 10.85)^2 + \tfrac{1}{12} \times \tfrac{3}{16} \times 8 \times (24)^3$$
$$= 4090 + 80 + 4480 + 1730 = 10370 \text{ in.}^4$$

$$\text{Impact} = \frac{50}{L + 125} = .286$$

$$M_{LL+I} = 628 \times \tfrac{2}{3} \times 1.286 = 538$$

$$M_{DL} = .070 \times 8 \times \frac{(50)^2}{8} = \underline{175}$$
$$713^{kl}$$

$$V_{LL+I} = 58.5 \times \tfrac{2}{3} \times 1.286 = 50.2$$

$$V_{DL} = 0.070 \times 8 \times \frac{50}{2} = \underline{14.0}$$
$$64.2^{k}$$

*See Appendix A, this chapter

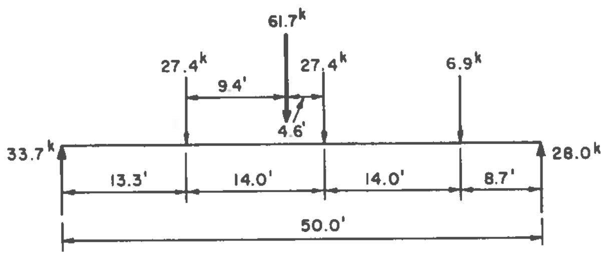
$$Z_{Top} = \frac{10370}{10.85} = 955 \text{ in.}^3 \quad f_t = \frac{713 \times 12}{955} = 8.96 \text{ ksi}$$

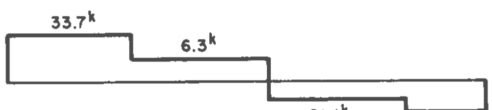
$$Z_{Bot.} = \frac{10370}{13.78} = 753 \text{ in.}^3 \quad f_b = \frac{713 \times 12}{753} = 11.36 \text{ ksi}$$

$$\text{Ave. } \nu = \frac{64.2}{36.00} = 1.78 \text{ ksi}$$

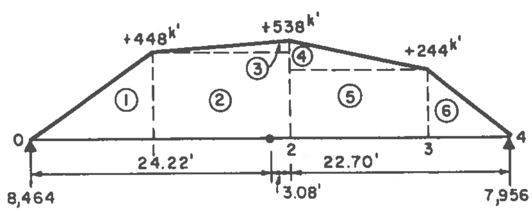
Wt. of 2 End Plates:	$24.5 \times 1/4 \times 2 \times 8' \times 3.4$	$= 333\#/50$ ft.	$\frac{333}{8 \times 50}$	$= 0.8$
Int. Diaphragm:	$20 \times 1/4 \times 8 \times 3.4$	$= 136\#/50$ ft.	$\frac{136}{8 \times 50}$	$= 0.4$
2" — 3/16" bt. PL:	$2 \times 3/16 \times 4 \times 3.4$	$= 5.1\#/\text{ft.}/8'$ width	$\frac{5.1}{8}$	$= 0.6$
5/16" outside PL's	$24 \times 5/16 \times 3.4 \times 1/2$	$= 12.8\#/8'$ width	$\frac{5.1}{8}$	$= 1.6$
Railing PL's 12" × 1/2" × 1'-10" × 32 × 3.4 × /50 × 32				$= 0.7$
1/4 × 3" PL 0.75 × 3.4/32				$= 0.1$
Main Mat'l				$= 40.8$
Total Material				$= 45.0$ psf

$$\text{Allowable deflection} = \frac{1}{800} \text{ span} = \frac{50 \times 12}{800} = 0.75''$$

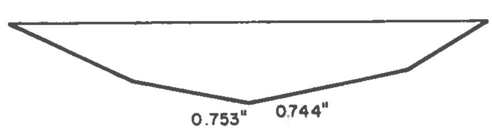




SHEAR DIAGRAM



MOMENT DIAGRAM



DEFLECTION DIAGRAM

2/3 lane/8' section

$16 \times 2/3 \times 2 \times 1.286 = 27.4^k$ wheel load

$4 \times 2/3 \times 2 \times 1.286 = 6.9^k$

$27.4 + 27.4 + 6.9 = 61.7^k$

$$\text{C.G. Load} = \frac{27.4 \times 14 + 6.9 \times 28}{61.7} = \frac{384 + 193}{61.7} = \frac{577}{61.7} = 9.4 \text{ ft.}$$

$$R_A = \frac{6.9 \times 8.7 + 27.4 \times 22.7 + 27.4 \times 36.7}{50.0} = \frac{60 + 622 + 100.6}{50.0} = \frac{1688}{50.0} = 33.7^k$$

$$R_B = \frac{27.4 \times 13.3 + 27.4 \times 273 + 6.9 \times 41.3}{50.0} = \frac{364 + 748 + 285}{50.0} = \frac{1397}{50.0} = 28.0^k$$

L.L. + I Deflection:

$M_1 = 33.7 \times 13.3 = +\ 448^{kl}$

$M_2 = 448 + 6.3 \times 14.0 = 448 + 89 = 537^{kl}$

$M_3 = 28.0 \times 8.7 = +\ 244^{kl}$

$M_2 = 244 + 21.1 \times 14.0 = 244 + 295 = +\ 539^{kl}$

① $448 \times 13.3 \times \frac{1}{2}$	=	2980	$\times \frac{13.3 \times 8}{3}$	=	26,500	
② 448×14.0	=	6270	$\times (13.3 + 7.0)$	=	127,000	
③ $90 \times 14.0 \times \frac{1}{2}$	=	630	$\times (13.3 + 14.0 \times \frac{2}{3})$	=	14,200	
④ $294 \times 14.0 \times \frac{1}{2}$	=	2060	$\times (27.3 + 14.0 \times \frac{1}{3})$	=	66,000	$\frac{397900}{50} = 7958$
⑤ 244×14.0	=	3420	$\times (27.3 + 7.0)$	=	117,300	
⑥ $244 \times 8.7 \times \frac{1}{2}$	=	1060	$\times (41.3 + 8.7 \times \frac{1}{3})$	=	46,900	$16420 - 7958 = 8462$
		16,420			397,900	

$$\text{Deflection at pt. 2} = [8462 \times 27.3 - 2980\,(14.0 + 13.3 \times \tfrac{1}{3}) - 6270 \times 7.0 - 630 \times (14.0 \times \tfrac{1}{3})] \times \frac{1728}{29{,}000 \times 10{,}370}$$

$$= \frac{[231{,}000 - 55{,}000 - 43{,}900 - 2{,}900] \times 1728}{29{,}000 \times 10{,}370}$$

$$= \frac{129{,}200 \times 1728}{29{,}000 \times 10{,}370} = 0.744''$$

$$\text{Deflection at 24.6' from left support} = [8467 \times 24.6 - 2980\,(24.6 - \tfrac{2}{3} \times 13.3) - 448 \times 11.3 \times \frac{11.3}{2} - 90 \times \frac{11.3}{14.0} \times 11.3 \times \tfrac{1}{2} \times \frac{11.3}{3}] \times \frac{1728}{29{,}000 \times 10{,}370}$$

$$= \frac{(208{,}000 - 46{,}800 - 28{,}600 - 1{,}600) \times 1728}{29{,}000 \times 10{,}370} = \frac{131{,}000 \times 1728}{29{,}000 \times 10{,}370} = 0.753''$$

Investigation of web $(24 \times \frac{3}{16})$

(Design Manual for High Strength Steels US Steel Publication ADUCO 02215)

$b/a = 2/50 = .04$

$k = 5.35 + 4\,(b/a)^2 = 5.35 + .01 = 5.36$

$b/t = 24/0.188 = 128$

$$\frac{b/t}{\sqrt{k}} = \frac{128}{\sqrt{5.36}} = \frac{128}{2.32} = 55.2$$

From Table II since $\frac{b/t}{\sqrt{k}} > 41.5$ (point c), use Formula 14 c

$$v_{cr} = \frac{19{,}660{,}000 \times k}{(128)^2} = \frac{19{,}660{,}000 \times 5.36}{(128)^2} = 6{,}440 \text{ psi}$$

For $N = 1.80$

Allowable shearing stress $= \frac{6440}{1.80} = 3{,}570$ psi > 1.78 ksi

Web Buckling Due to Compression

$$S_{cr} = 1.8\,S_y - \frac{\sqrt{S_y^3}}{4770}\left(\frac{h/t}{\sqrt{k}}\right) = 1.8 \times 36{,}000 - \frac{\sqrt{(36{,}000)^3}}{4770}\left(\frac{128}{\sqrt{24}}\right)$$

$S_y = 36{,}000$ $\qquad S_{cr} = 64.80 - \frac{6{,}830}{4770}\left(\frac{128}{4.9}\right) = 64.80 - 37.40 = 27.40$ ksi

$h/t = \frac{24}{.1875} = 128$ $\qquad 8.96 \times \frac{10.85 - .375}{10.85} = 8.64 \text{ ksi} < \frac{27.40}{1.8 \times 0.7} = 21.75$

$k = 24$

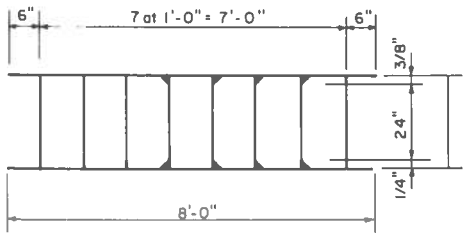
Web Buckling Due to Shear

$$V_{cr} = 1.8\,v_y - n\left(\frac{h/t}{\sqrt{k}}\right)$$

Constant $n = \frac{\sqrt{v_y^3}}{4770}$ $S_y = 36000$ $v_y = 19{,}000$

$$= \frac{\sqrt{(19{,}000)^3}}{4770} = \frac{2{,}620{,}000}{4770} = 550$$

$$V_{cr} = 1.8\,(19{,}000) - 550\left(\frac{128}{\sqrt{24}}\right) = 34{,}200 - 14{,}400 = 19{,}800 \text{ psi} > 17{,}800 \text{ psi}$$
4.9

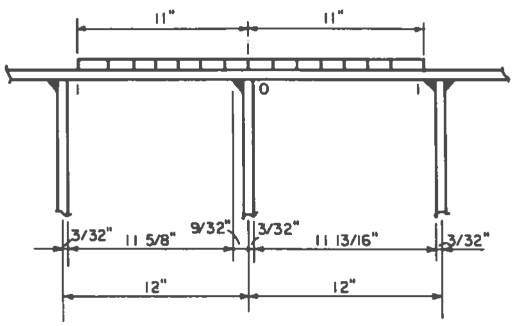


8'-0" Section { Top Floor PL 96 × 3⁄8 (A441); 8 Web PLs 24 × 3⁄16, Bott. PL 96 × 1⁄4 } (A36)

Floor Plate Design (Design Manual for Orthotropic Steel Plate Deck Bridge, AISC)

12^k wheel load $2g \times 2c = 22'' \times 12''$

Unit Pressure p (Incl. 30% Impact) = 59 psi



(a) Beam Moments

$$M_0 = 2\int_{x=0}^{11}\left[-0.5\left(\frac{x}{a}\right) + 0.866\left(\frac{x}{a}\right)^2 - 0.366\left(\frac{x}{a}\right)^3\right] padx$$

$$= 2\,(59)\,(12)^2\left[-\frac{0.5}{2}\left(\frac{11}{12}\right)^2 + \frac{0.866}{3}\left(\frac{11}{12}\right)^3 - \frac{0.366}{4}\left(\frac{11}{12}\right)^4\right]$$

$$= -890\#''$$

$$M_1 = -317\#''$$

$$V_O = R_O + \frac{(-M + M_O)}{a} = \frac{(59)\,(11)\,(6.5)}{12} + \frac{890 - 317}{12} = 351 + 49 = 400\#$$

Bending Moment at edge of plate (3⁄16" from the center of support)

$$M_T = -890 + (400)\,(.188) - \frac{(59)\,(.188)^2}{2} = 890 + 75 - 1 = -816\#''$$

Moment at midspan $M_C = 458\#''$

(b) Plate Moments and Stresses: plate factor Ψ, Fig. 6.3b, Case 3

$2c/a = 12/12 = 1.0$ $\Psi_S = 0.87$ (for moment at support)

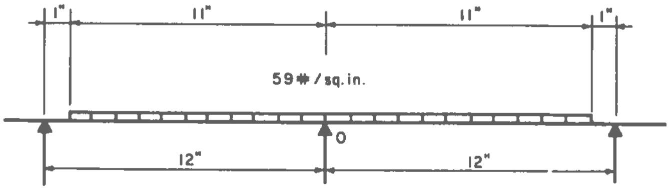
$M_T = 0.87 \times -816 = -710\,\#$ in./in.

3⁄8" plate (A441) $S = \frac{(0.375)^2}{6} = 0.0235$ in.^{3}/in.

$f_{max} = \frac{710}{.0235} = 30{,}200$ psi $\frac{30{,}200}{27{,}000} = 1.12$ (Flexibility of rib will tend to reduce moment)

$\Psi_C = 0.70$ (for moment at mid-span) $M_C = 0.70\,(458) = 320\,\#$ in./in.

$$f = \frac{320}{.0234} = 13{,}700 \text{ psi}$$



1/6 Division

$M_O = -[.0609 + .0840 + .0793 + .0568 + .0271]\ .0271]\ [2]$

$\times 59 \times 2 \times 12$

$= -.6162 \times 59 \times 2 \times 12 = -873\#''$

1/10 Division

$M_O = -[.0417 + .0683 + .0819 + .0849 + .0793 + .0673 + .0512 + .0332 + .0154 \times ⅔]\, 2 \times 59 \times 1.2 \times 12$

$= -.5181 \times 2 \times 59 \times 1.2 \times 12 = -880\#''$

$V = [.9263 + .8351 + .7307 + .6176 + 0.5000 + .3824 + .2693 + .1649 + .0737 \times ⅔$

$+ .0529 + .0866 + .1039 + .1077 + .1005 + .0853 + .0649 + .0421 + .0195 \times ⅔]$

$\times 59 \times 1.2 \times 12$

$= 5.1323 \times 59 \times 1.2 \times 12 = 436\#$

(c) Maximum Deflection—

$t_p \geqq (0.007)\ (12)\ (\sqrt[3]{59}) = 0.328'' < ⅜''$ (p. 160)

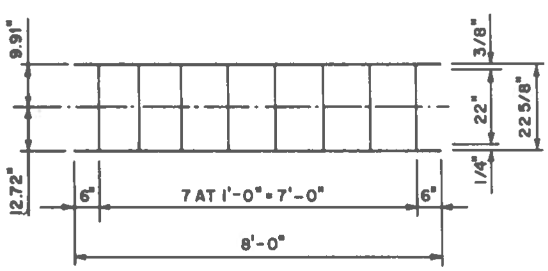
(d) Effect of rib flexibility—

$P = 800\#$ $\quad A = \frac{3}{16} \times 1 = \frac{3}{16} = 0.19$ in.^{2} $\quad \frac{PL}{AE} = \frac{800 \times .24}{0.19 \times 29{,}000{,}000} = .0035''$

$$M_{FE} = \frac{6\,E\,K\,\Delta}{L} = \frac{6 \times E \times ¾\ I/L \times \Delta}{L} = \frac{6 \times 29{,}000{,}000 \times \frac{.0235}{12} \times ¾ \times .0035}{12}$$

$$= \frac{6 \times 29{,}000{,}000 \times .0235 \times 3 \times .0035}{12 \times 12 \times 4} = 74.6\#''$$

$$\frac{74.6}{990} = 8\%$$



For Each 8' Section	Top Floor PL 96 × ⅜ (A441)	36.00	15.3
	8 Webs 22 × $\frac{3}{16}$ (A36)	33.00	14.0
	Bott. PL 96 × ¼ (A36)	24.00	10.2
		93.00 in.^{2}	39.5 psf.

$$N.A. = [36.00 \times 0.19 + 33.00\ (0.38 + 11.00) + 24.00\ (0.38 + 22.00 + 0.12)] \times \frac{1}{93.00}$$

$$= [7 + 376 + 540] \times \frac{1}{93.00} = \frac{923}{93.00} = \frac{923}{93.00} = 9.91''$$

$$I_{NA} = 36.00\ (9.91 - 0.19)^2 + 33.00\ (11.38 - 9.91)^2 + 24.00\ (22.50 - 9.91)^2 + \tfrac{1}{12} \times \tfrac{3}{16} \times 8 \times (22)^3$$

$$= 3{,}400 + 70 + 3{,}800 + 1{,}330 = 8{,}600 \text{ in.}^4$$

$$\text{Deflection} = \frac{109{,}200 \times 1728}{29{,}000 \times 8600} = .757 \approx 0.750''$$

$$S_{Top} = \frac{8600}{9.91} = 868 \text{ in.}^3$$

$$S_{Bot} = \frac{8600}{12.72} = 676 \text{ in.}^3$$

$$M_{(L+I)} = +428^{kl} = 500\,(2/3)\,(1.286)$$

$$M_{(L+I)} = -304^{kl} = 373\,(2/3)\,(1.222)$$

Assume total D.L. = 70 psf.

$M_{DL} = -1/8 \times .070 \times (50)^2 = -22^{kl} \times 8'$ width -176^{kl}

$M_{DL} = +0.0700 \times .070 \times (50)^2 = +12^{kl} \times 8'$ width $+96^{kl}$

Total Design Moment = $+428 + 96 = +524^{kl}$
$-304 - 176 = -480^{kl}$

At 20' from outside support:

$$f_{Top} = \frac{524 \times 12}{868} = -7.25 \text{ ksi}$$

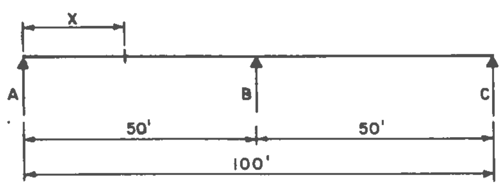
$$f_{Bot} = \frac{524 \times 12}{676} = +9.30 \text{ ksi}$$

At Interior Support:

$$f_{Top} = \frac{480 \times 12}{868} = +6.65 \text{ ksi}$$

$$f_{Bot} = \frac{480 \times 12}{676} = -8.52 \text{ ksi}$$

Ratio of $\frac{W}{t} = \frac{12}{.25} = 48$ (Bottom)



From AISC Booklet
"Moment, Shears and Reactions
Continuous Highway Bridge Tables"

HS 20 Loading

Max. Reaction at A 55.7^k/lane
Max. Reaction at B 68.6^k/lane

Max. shear in AB at B = -62.0^k

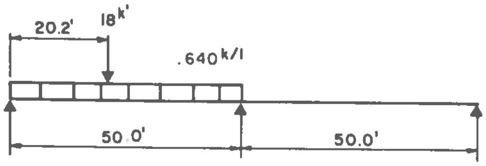
Max. Moment = $+500.7^{kl}$ In AB at X
-373.2^{kl} at B

Impact Coeff. .286 (I)
.222 (VI)

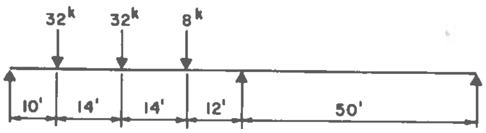
Dist. X = 20.0 ft.

L.L. + I. Deflection —

With lane loading .640k/ft. and conc. load of 18^k

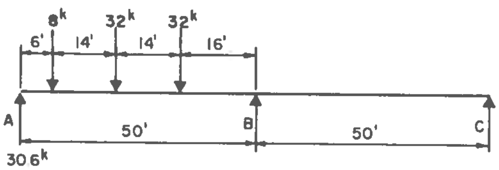


$$M_{20'} = +.0700 \times .640 \times (50.0)^2 + .2064 \times 18 \times 50$$
$$= +112 + 186 = +298^{kl}$$



$$M_{20'} = [0.1527 + (.1216 + .0409 \times .2) + (.0512 + .0331 \times 0.4)\tfrac{1}{4}] \times 32 \times 50.0$$
$$= .2986 \times 32 \times 50.0 = 478^{kl}$$

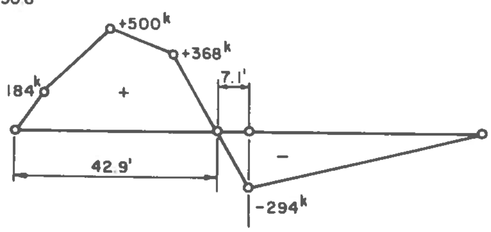
$$M_{20'} = [.2064 + (.0843 + .0373 \times .2) + (.0501 + .0507 \times .2)\tfrac{1}{4}] \times 32 \times 50.0$$
$$= .3133 \times .32 \times 50.0 = +501^{kl}$$



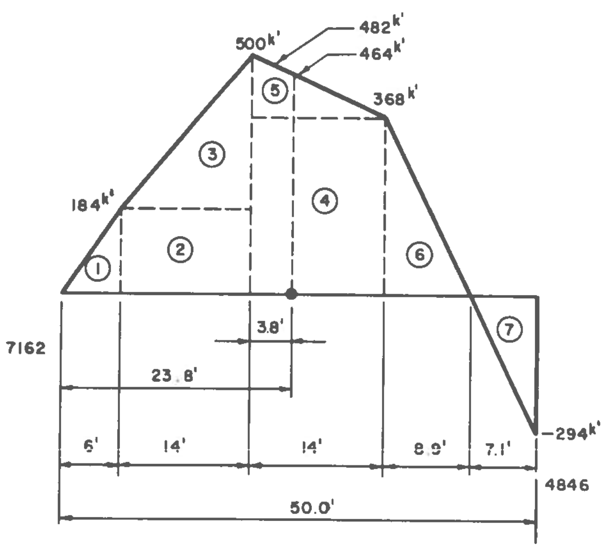
$$R_A = [(.8753 - .1233 \times .2)\tfrac{1}{4} + (.5160) + (.2108 + .0932 \times 0.2)] \times 32$$
$$= [.2126 + .5160 + .2294] \times 32 = 30.6^k$$

$$M_{6'} = 30.6 \times 6 = +184^{kl}$$

$$M_{34'} = 30.6 \times 34 - 8 \times 28 - 32 \times 14$$
$$= 1040 - 224 - 448 = 368^{kl}$$



$\frac{42.9 \times 12}{25} = 20.5''$ MIN. DEPTH



① 184 × 6 × ½	=	552	× (50.0—4.0)	=	25,400
② 184 × 14	=	2,576	× (50—7—6)	=	95,200
③ 316 × 14 × ½	=	2,212	× (50—6—9.3)	=	76,800
④ 368 × 14	=	5,150	× (50—20—7)	=	118,500
⑤ 132 × 14 × ½	=	925	× (50—20—4.7)	=	23,400
⑥ 368 × 8.9 × ½	=	1,636	× (50—34—3.0)	=	21,300
⑦ —294 × 7.1 × ½	=	— 1,043	× (2.4)	=	— 2,500
		12,008			358,100

$$\frac{358,100}{50} = 7,162$$

$$12,008 - 7,162 = 4,846$$

$$[500 - \tfrac{1}{2}(500 - 368) \times \frac{x}{14}] \times x = 1826$$

$$x^2 - 106x + 387 = 0$$

$x = 3.75$
say 3.8 ft.

$$\text{Deflection} = [7162 \times 23.8 - \frac{184 \times 6}{2} \times 19.8 - 184 \times 14 \times (3.8 + 7.0) - 316 \times 14 \times \tfrac{1}{2} \times (3.8 + \frac{14.0}{3})$$

$$- 482 \times 3.8 \times 1.9] \times \frac{1728}{29{,}000 \times I}$$

$$= [170{,}450 - 10{,}920 - 27{,}800 - 18{,}800 - 3{,}480] \times \frac{1728}{29{,}000 \times I}$$

$$= \frac{109{,}200 \times 1{,}728}{29{,}000 \times I} = \frac{6{,}520}{I} = 0.75''$$

$$\text{Req'd } I = \frac{6{,}520}{0.75} = 8{,}700 \text{ in.}^4 \approx 8{,}600 \text{ in.}^4$$

Longitudinal Stiffeners

$b/t = 24/\tfrac{1}{4} = 96$

$$\frac{78{,}640{,}000}{(96)^2} = 8.54 \text{ ksi}$$

$$\frac{8.54}{1.80} = 4.74 \text{ ksi}$$

Use $3\tfrac{1}{2}'' \times \tfrac{5}{16}''$ stiffeners $\quad 42 \times \tfrac{1}{4} = 10.5''$ (Design Manual for High Strength Steels)

$$\begin{array}{llll} 10.5 \times \tfrac{1}{4} & 2.63 & \times 0.125 = & 0.33 \\ 3.5 \times \tfrac{5}{16} & \underline{1.10} & \times 2.00 = & 2.20 \\ & 3.73 \text{ in.}^2 & & \end{array} \Big\} 2.53$$

$$\frac{2.53}{3.73} = 0.68'' \text{ N.A.}$$

$$I = \tfrac{1}{12} \times \tfrac{5}{16} \times (3.5)^3 + 1.10(2.00 - 0.68)^2 + 2.63(0.68 - 0.13)^2$$

$$= 1.1 + 1.9 + 0.8 = 3.8 \text{ in.}^4$$

$$r = \sqrt{\frac{3.8}{3.73}} = \sqrt{1.02} = 1.01$$

$$l/r = \frac{1.44}{1.01} = 143 \qquad f = 9830 \text{ psi (compression)}$$

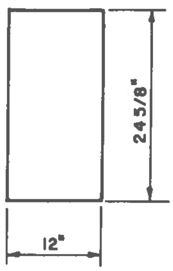
$$\text{Total Force} = 9830(3.73 + 2.63) = 9830(6.36) = 62.5$$

$$\text{Av. stress} = \frac{62.5}{23(\tfrac{1}{4})} = 10.09 \text{ ksi allowable} > 8510 \text{ psi actual OK}$$

Appendix A

Load Distribution

Investigation of Load Distribution by the Guyon-Massonnet Theory
Ref: Paper by P. B. Morice and G. Little, Journal of Institute of Structural Engineers, March 1954



32—Longitudinal Beams at 1 ft. apart
Diaphragm at mid-span
Section of Long. Beam:

1 Top Pl 12 × ⅜	=	4.50
1 Bott. Pl 12 × ¼	=	3.00
2 Webs 24 × 3/32	=	4.50
	A =	12.00 in.²

$I = 1300$ in.^{4}

$Z_{Top} = 119$ in.^{3}

$Z_{Bot.} = 94$ in.^{3}

Diaphragm: Area $= \frac{1}{4} \times 20 = 5.00$ in.^{2}
$J = 1/12 \times \frac{1}{4} \times (20)^3 = 167$ in.^{4}
$S = 167/10 = 16.7$ in.^{3}

Load Distribution: $2a = 50$ ft. Length of bridge
$2b = 32$ ft. Width of bridge

Flexural

$$i = \frac{I}{12''} = \frac{1300}{12} = 108 \text{ in.}^4/\text{in.}$$

$$j = \frac{J}{300} = \frac{167}{300} = 0.56 \text{ in.}^4/\text{in.}$$

$$\theta = \frac{b}{2a} \sqrt[4]{\frac{i}{j}} = \frac{16}{50} \sqrt[4]{\frac{108}{0.56}} = 0.32 \times 3.73 = 1.19 \quad \left(\text{Same as Guyon 1946}\right)$$

Torsion

$$J_o = \tfrac{1}{3} \times (\tfrac{1}{4})^3 \times (20) = .104 \text{ in.}^4$$

$$I_o = \frac{4A^2}{\Sigma \frac{s}{t}} = \frac{4 \times (12 \times 24)^2}{2 \times \frac{24}{.094} + \frac{12}{.375} + \frac{12}{.25}} = \frac{332{,}000}{510 + 32 + 48} = \frac{332{,}000}{590} = 563 \text{ in.}^4$$

$$G_{io} = \frac{563}{12} = 46.9 \text{ in.}^4/\text{in.}$$

$$G_{jo} = \frac{.104}{300} = .0003 \text{ in.}^4/\text{in.} \qquad G_{io} + G_{jo} = 46.9 \text{ in.}^4/\text{in.}$$

$$\alpha = \frac{G(io + jo)}{2E\sqrt{ij}} = \frac{46.9}{\sqrt{108 \times .56}} \times \frac{G}{2E} \qquad \text{For steel } G = .384\,E \text{ (shear modulus)}$$

$$= \frac{46.9}{2 \times 7.8} \times .384 = 1.15 > 1$$

Value of K_l $\left(\begin{array}{l} \text{Full Torsion Curve } \alpha = 1 \\ \theta = 1.04 \end{array} \right)$

Load at \ Position on Section	− b	− 3b/4	− b/2	− b/4	0	+ b/4	+ b/2	+ 3b/4	+ b	Σ
0	0.43	0.60	0.92	1.37	1.70	1.37	0.92	0.60	0.43	.987
b/4	0.23	0.33	0.53	0.90	1.38	1.73	1.48	1.08	0.82	.999
b/2	0.12	0.18	0.30	0.57	0.93	1.47	1.80	1.78	1.54	.988
3b/4	0.07	0.10	0.21	0.33	0.60	1.09	1.93	2.50	2.73	1.015
b	0.03	0.06	0.12	0.21	0.43	0.82	1.53	2.73	4.47	.997

Check with Simpson's rule:—

Load at 0 $\quad \frac{1}{3 \times 8}\,[0.43 + 4\,(0.60 + 1.37 + 1.37 + 0.60) + 2\,(0.92 + 1.70 + 0.92) + 0.43]$

$= \frac{1}{24}\,[0.43 + 15.76 + 7.08 + 0.43] = \frac{1}{24}\,[23.70] = .987$

Load at b/4 $\quad \frac{1}{3 \times 8}\,[.23 + 4\,(.33 + .90 + 1.73 + 1.08) + 2\,(0.53 + 1.38 + 1.48) + 0.82]$

$= \frac{1}{24}\,[.23 + 16.16 + 6.78 + 0.82] = \frac{1}{24}\,[23.99] = .999$

Load at b/2 $\quad \frac{1}{3 \times 8}\,[.12 + 4\,(.18 + .57 + 1.47 + 1.78) + 2\,(0.30 + 0.93 + 1.80) + 1.54]$

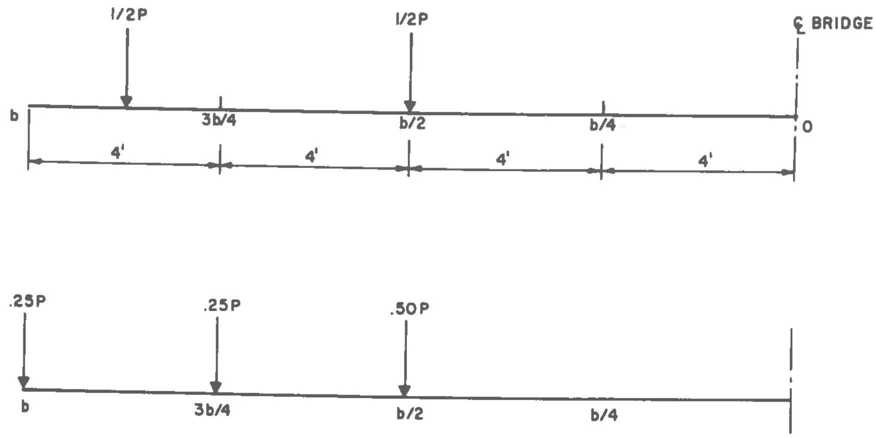
$= \frac{1}{24}\,[.12 + 16.00 + 6.06 + 1.54] = \frac{1}{24}\,[23.72] = .988$

Load at 3b/4 $\quad \frac{1}{3 \times 8}\,[.07 + 4\,(0.10 + 0.33 + 1.09 + 2.50) + 2\,(0.21 + 0.60 + 1.93) + 2.73]$

$= \frac{1}{24}\,[.07 + 16.08 + 5.48 + 2.73] = 1.015$

Load at b $\quad \frac{1}{3 \times 8}\,[.03 + 4\,(.06 + .21 + .82 + 2.73) + 2\,(0.12 + 0.43 + 1.53) + 4.47]$

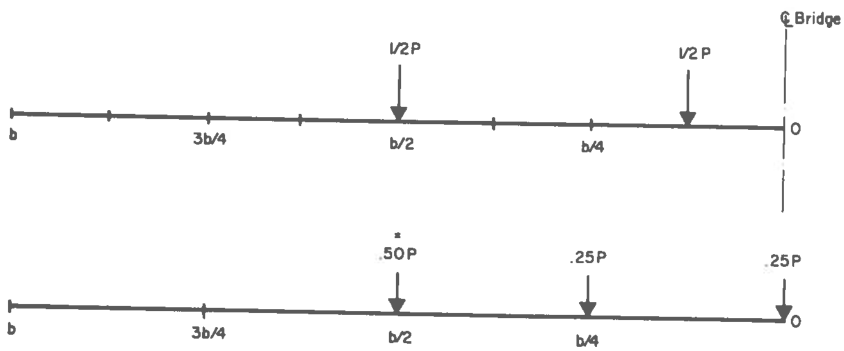
$= \frac{1}{24}\,[.03 + 15.28 + 4.16 + 4.47] = \frac{1}{24}\,[23.94] = .997$



	Position on Section	λ	$-b$	$-3b/4$	$-b/2$	$-b/4$	0	$+b/4$	$+b/2$	$+3b/4$	$+b$
Value of λK_1	Load at b	0.25	0.01	0.02	0.03	0.05	0.11	0.21	0.38	0.68	1.12
	3b/4	0.25	0.02	0.03	0.05	0.08	0.15	0.27	0.48	0.62	0.68
	b/2	0.50	0.06	0.09	0.15	0.29	0.47	0.74	0.90	0.89	0.77
	$\Sigma \lambda K_1$		0.09	0.14	0.23	0.42	0.73	1.22	1.76	2.19	2.57

Ave. wheel load $= \dfrac{2.57 \times 2 + 2.19 \times 4 + 1.76 \times 2}{8} = \dfrac{17.42}{8} = 2.18\text{ P} \qquad \dfrac{2.18}{4}\text{ P} = .545\text{ P}$

$(4\ B_{m-1})$

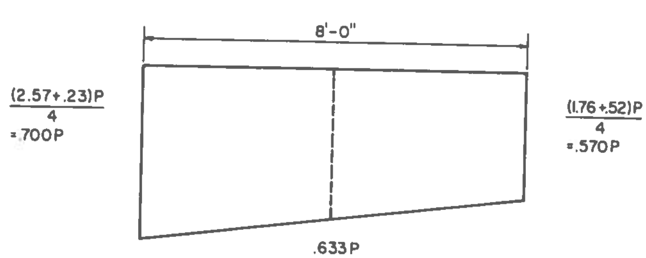


	Position on Section	λ	$-b$	$-3b/4$	$-b/2$	$-b/4$	0	$+b/4$	$+b/2$	$+3b/4$	$+b$
Value of λK_1	Load at b/2	0.50	0.06	0.09	0.15						
	b/4	0.25	0.06	0.08	0.14						
	0	0.25	0.11	0.15	0.23						
	$\Sigma \lambda K_1$		0.23	0.32	0.52						

Ave. wheel load $= \dfrac{.23 + .32 \times 2 + .52}{4} = \dfrac{1.39}{4} = .35\text{ P} \qquad \dfrac{.35\text{P}}{4} = .088\text{ P.}$

With 2 lanes

$$\begin{array}{r} .545\text{ P} \\ +\ \underline{.088\text{ P}} \\ .633\text{ P} \end{array}$$



LOAD DISTRIBUTION FOR 8' EXTERIOR SECTION

Value of K_1 (Full Torsion Curve $\alpha = 1$, $\theta = 1.19$)

Load at	Position on Section	$-b$	$-3b/4$	$-b/2$	$-b/4$	0	$+b/4$	$+b/2$	$+3b/4$	$+b$
0	λ	.32	.50	.88						
	.25	.08	.13	.22						
b/4		.14	.25	.44						
	.25	.04	.06	.11						
b/2		.08	.13	.24				2.10	1.84	1.48
	.50	.04	.07	.12				1.05	.92	.74
3b/4								1.84	2.72	2.88
	.25							.46	.68	.72
b								.48	2.88	5.40
	.25							.12	.72	1.35
	Σ	.16	.26	.45			Σ	1.63	2.32	2.81

$$\text{Ave. Wheel Load} = \frac{.16 + 2\,(.26) + .45}{4} = .283\text{ P}$$

$$\frac{.283}{4} = .071\text{ P}$$

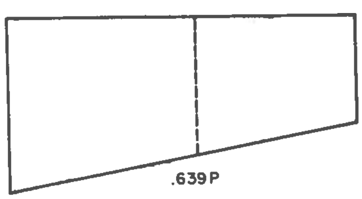
$$\text{Ave. Wheel Load} = \frac{2.81 + 2.32\,(2) + 1.63}{4} = 2.27$$

$$\frac{2.27}{4\,B_{m's}} = .568\text{ P}$$

Two Lanes:

$$\begin{array}{r} .568 \\ .071 \\ \hline .639\text{ P} \end{array}$$

$\frac{2.81 + .16}{4}$ $= .743$ P



$\frac{1.63 + .45}{4}$ $= .520$ P

Design II

GENERAL

In this design, the shape, thickness and spacing of the floor plate ribs and the thickness of the floor plate are selected to take full advantage of the material at critical points along the span. Combined longitudinal compressive stresses in the floor plate, due to the bending in the rib sections and stresses imposed on it as part of the girders, are close to the allowable limit of the material when combined with the transverse stresses in the floor plate spanning across the rib webs. Tensile stresses in the bottoms of the ribs are near the allowable capacity of the material. The depth of these ribs is such that the bottoms of the ribs are very close to the neutral axis of the composite girder sections, so that compressive stresses in the ribs over the floorbeams are not critical. Girder flanges are sized as required within the allowable stress limits of the material.

Design computations were prepared for the four deck systems listed below. The computations for design B are shown in this chapter. Results of designs A, C, and D are represented in the data summary shown in Figure 3. This summary also indicates the major design features of designs A, C, and D, and the estimated quantities of structural steel required for each design.

A. A design was prepared for a structure not meeting the minimum material thickness requirements of AASHO Specifications. This USS COR-TEN Steel design utilizes 12 ga. thickness deck ribs and stiffened web plates for the girders, varying from 6 ga. thickness for 40 foot spans to 5/16 inch thickness for 80 foot spans.

B. A design was prepared for a structure meeting the minimum material thickness requirements of AASHO Specifications for main members, except that the deck ribs are of 3/16 inch material (the minimum thickness recommended by AISC for closed ribs in orthotropic designs). COR-TEN Steel was used for this design. All webs were 5/16 inch plates, stiffened as required.

C. A third COR-TEN Steel design was prepared, similar to B above, except that no web stiffeners are employed. Although this is not the lightest weight design, it may be the most economical because it eliminates stiffeners and their inherent shop fabrication costs.

D. A design was prepared for a 50 foot span using A36 grade steel throughout with unstiffened girder webs. This design provides a basis for relative first cost comparison with the COR-TEN design described under C above.

ASSUMPTIONS

Floor beams span continuously through the girder webs and cantilever beyond the outside girders to support the safety curb and bridge railing. For calculating moments and shears, the floor beams were assumed to be on unyielding supports. Fatigue stresses were considered on the basis of 2,000,000 cycles of load. However, in lieu of a detailed grid analysis of the floor system, the floor beams are selected to provide a minimum transverse stiffness equivalent to that furnished by concrete slab and stringer designs, in order to insure the applicability of the AASHO distribution factors.

Additionally, the live load was apportioned to each girder by assuming the floor beams continuous over fixed supports, and positioning the truck wheels to produce maximum reactions. This gave a larger load to the interior girders while the AASHA method gave a larger load to the exterior girders. The interior girders are designed for 0.843 times the standard truck loading and the exterior girders are designed for 0.625 times the standard truck loading.

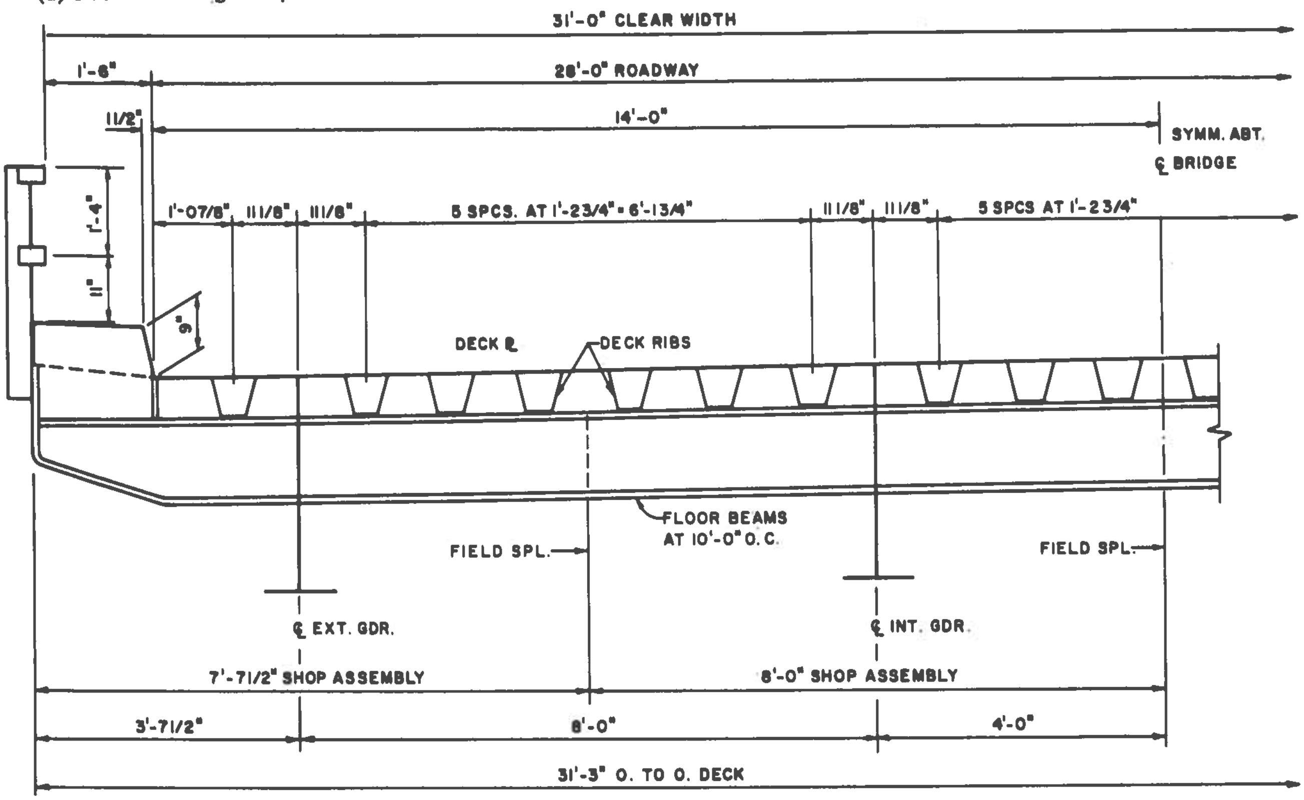
Where the girder spacing is less than 1/3 of the girder span, the entire cross-sectional area of the steel deck may be assumed to participate as the top flange of the girders provided it is adequately stiffened to prevent buckling. This has been confirmed by test measurements of stresses in existing European steel plate deck bridges and the assumption is used in this design. Adequate lateral support against flange buckling is assumed to be furnished by the floor beams spaced at 10 ft. o.c. reducing the max. L/r value to $120/2.58 = 46.5$.

The safety curb is designed to support wheel loads without exceeding 75 percent of the yield strength of the material. The same allowable stress increase is allowed in the floor beams when the wheels are located on the curb and bridge railing loads are applied. Fatigue stresses are not considered critical for this loading condition. Bridge railings are designed for the loads specified in AASHO 1.2.11 (1).

DESIGN FOR 50 FOOT SPAN

Requirements:

(a) Roadway—28′-0″ curb to curb + 2 1′-6″ safety curbs
= 31′-0″ clear between railings.
(b) Span 50′-0″ c. to c. bearings (simple span)
(c) Live Load—HS20 Impact Allowed
(d) Deck Surfacing—20 psf



Material:

Deck, Curbs, Ribs, Floor Beams, Girders—USS COR-TEN
Railings & Posts—ASTM A 36 Galvanized
Field Bolts—ASTM A 325
Bearing—Elastomeric Neoprene Pads

Design of Deck:

Spans

Deck plate—7⅜″ ± c. to c. rib webs.

Deck ribs—10′-0″ c. to c. floor beams. For moments and shears assume five spans continuous over unyielding supports (effect of deflections of floor beams and girders on rib moments and shears is negligible).

Live load distribution

Deck plate—For contact area between deck plate and tire use recommendations outlined in AISC Design Manual for Orthotropic Steel Bridges, Sect. 3.4.1.2. For effective width of deck plate supporting wheel concentrations refer to Sect. 6.2.1.2 of the above Manual.

Deck Ribs—AASHO 1.3.6 specifies that for open steel grid floors the wheel loads shall be distributed, normal to the main bars, a width equal to 1¼ inches per ton of axle load plus twice the distance center to center of main bars. For 16 ton axle and 7⅜″ rib web spacing this width is (16 × 1¼) + (2 × 7⅜) = 20 + 14¾ = 34¾″ or 2.35 ribs.

(d) Deck Surfacing—20 psf.

Deck Plate

Wheel load = 12,000# I = 30%
Contact Area = 2g × 2c = 22" × 12" = 264 in²

Tire pressure = $\frac{12{,}000\# \times 1.30}{264}$ = 59 psi

$M_{MAX.}$ (1" strip) = $\frac{1}{10}WL^2$ =
$\frac{59 \times (7.38)^2}{10}$ = 322#"

2c/a = 12/7.38 = 1.63

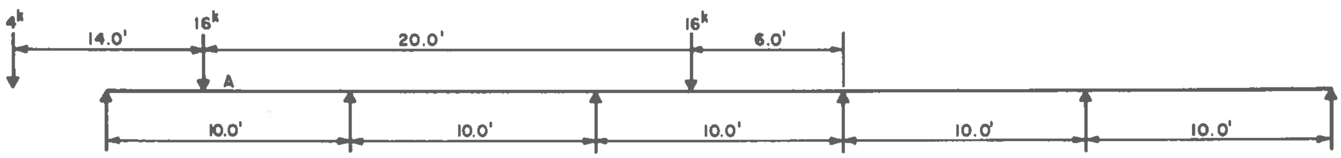
Ψs = 0.96

Av. mom. across effective PL width =
0.96 × 322 = 309#"/inch

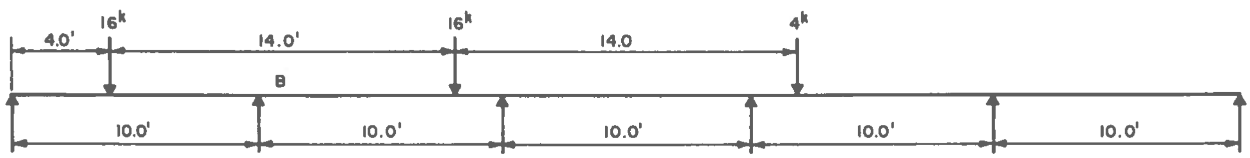
For $\frac{5}{16}$" PL; S = $\frac{(.3125)^2}{6}$ = 0.0163 in³/in.

Max. Transverse PL stress =
309/.0163 = 18,960 psi

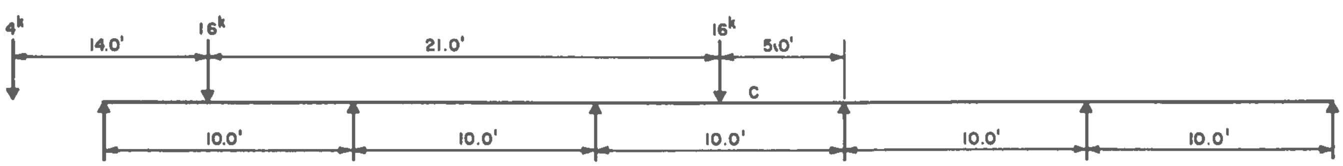
(2) DECK RIBS:—L. L. MOMS. & SHEARS.



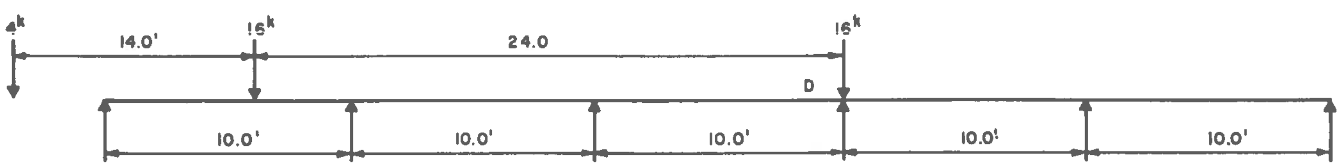
MAX. POS. L. L. MOM. = 16 x 10 x .213 = 34.0$^{k'}$ AT A.



MAX. NEG. L. L. MOM. = (16 x 10 x .121) + (4 x 10 x .005) = 19.6$^{k'}$ AT B.

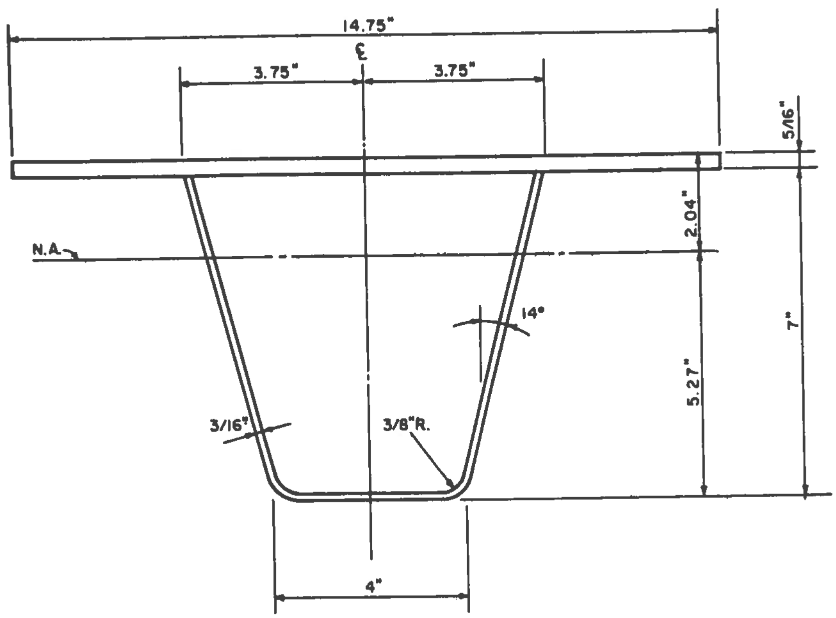


MAX. POS. L. L. MOM. AT ℄ GDR. SPAN = 16 x 10 x .183 = 29.2$^{k'}$ AT C



MAX. L. L. SHEAR = 16 x 1.034 = 16.55^{k} AT D.

Deck PL $\frac{5}{16}$": Formed Rib PL $\frac{3}{16}$"



Section Properties:

Wgt = 27.9 #/L.F.
Area = 8.19 Sq."
I = 54.6 in.^{4}
SM_T = 26.8 in.^{3}
SM_B = 10.4 in.^{3}
r = 2.58 in.

$$\text{DL (2.35 ribs)} = \quad 2.35 \times 27.9 = 65.5$$
$$+ \text{ surfacing} = 2.35 \times \frac{14.75}{12} \times 20 = \underline{57.8}$$
$$\text{DL} = 123.3\#/\text{L.F.}$$
say 125#/L.F.

Max. Pos. Mom. @ A
$M_{DL} + M_{LL+I} = .125 \times .077 \times (10.0)^2 + 34.0 \times 1.30 = 45.2^{k'}$

Max. Stresses

$$\text{Compr. (Top)} = \frac{45.2 \times 12}{2.35 \times 26.8} = 8.62_{ksi}$$

$$\text{Tens. (Bot.)} = \frac{45.2 \times 12}{2.35 \times 10.4} = 22.2_{ksi}$$

Max. Neg. Mom. @ B
$M_{DL} + M_{LL+I} = .125 \times .107 \times (10.0)^2 + 19.6 \times 1.30 = 26.7^{k'}$

Max. Stresses

$$\text{Tens. (Top)} = \frac{26.7 \times 12}{2.35 \times 26.8} = 5.08_{ksi}$$

$$\text{Compr. (Bot.)} = \frac{26.7 \times 12}{2.35 \times 10.4} = 13.1_{ksi}$$

Max. Pos. Mom. @ C
$M_{DL} + M_{LL+I} = .125 \times .033 \times (10.0)^2 + 29.2 \times 1.30 = 38^{k'}$

Max. Stresses

$$\text{Compr. (Top)} = \frac{38.4 \times 12}{2.35 \times 26.8} = 7.31_{ksi}$$

$$\text{Tens. (Bot.)} = \frac{38.4 \times 12}{2.35 \times 10.4} = 18.8_{ksi}$$

Max. Shear @ D
$V_{DL} + V_{LL+I} = .125 \times 5.0 + 16.55 \times 1.30 = 22.15^k$
Assume shear on 3 webs only
Shear per web $= 22.15/3 = 7.38^k$/web
Web A $= .1875 \times 6.44/\cos 14° = 1.24$ in^2
D/t $= 6.44/.1875 \times \cos 14° = 35.4$
Allow. V $= 15.0_{ksi}$

$V = 7.38/1.24 = 5.95_{ksi} < 15.0$

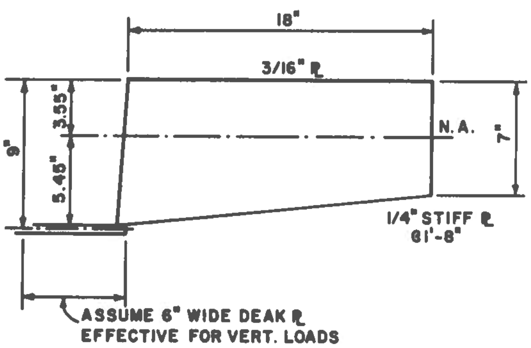
Provide $\frac{5}{16}$" Brg. Stiff. over Floor Beam Webs
Max. Rib reaction $= 7.38 \times 2 + 2/3 \times .63 = 15.18^k$
Brg. under stiff $= 15.18^k/.3125 \times 2.90 = 16.8_{ksi} < 40.0$

Deck Rib Splices

Use full penetration butt welds for Rib splices located approximately 2.5' from floor beam where max. stresses are below the allow. fatigue stress of approx. 16,000 psi.

Curbs

Loads — 500#/L.F. horizontal at top of curb
Check curb section for supporting a 16^k wheel load plus impact without exceeding 75% of yield stress (37.5_{ksi}).



$I = 122$ in^4
$Z_{Top} = 34.3$ in^3
$Z_{Bot} = 21.2$ in^3

Section ok for horiz. load by observation.

$M_{LL+I} = 34.0 \times 1.30 = 44.2$
$M_{DL} = .035 \times .077 \times (10.0)^2 = .3$
$= 44.5^{k'}$

Max. stresses

$$\text{Top (Compr.)} = \frac{44.5 \times 12}{34.3} = 15.6_{ksi} < 37.5$$

$$\text{Bot. (Tens.)} = \frac{44.5 \times 12}{21.2} = 25.2_{ksi} < 37.5$$

Railing and Posts (A 36 steel)
Top Rail—Vert. load = 100#/L.F.
Mom. $= .10 \times (10.0)^2/8 = 1.25^{k'}$
Top Rail—Horiz. load = 150#/L.F.
Mom. $= .15 \times (10.0)^2/8 = 1.88^{k'}$
Tubing 4 × 2 × .250# $Z_{XX} = 2.345$ $Z_{YY} = 1.532$

$$\text{Max. stress} = \frac{1.25 \times 12}{1.532} + \frac{1.88 \times 12}{2.345}$$
$= 19.38_{ksi} < 20.0$

Intermediate Rail
Horiz. load = 300#/L.F.
Mom. $= .30 \times (10.0)^2/8 = 3.75^{k'}$

$$Z \text{ reqd} = \frac{3.75 \times 12}{20} = 2.25 \text{ in}^3$$
Use Tubing 4 × 2 × .250

Posts

Mom. @ top of curb $= 1.50 \times 2.25 + 3.0 \times .92 = 6.14^{k'}$

$$Z \text{ reqd} = \frac{6.14 \times 12}{20} = 3.69 \text{ in}^3$$

Use Tubing $4 \times 4 \times .250$: $Z = 3.994 \text{ in}^3$

H. S. Bolts reqd @ top of curb (¾" single shear)

$$N = \frac{1.5 \times 3.33 + 3.0 \times 2.0}{1.08 \times 5.96} = 1.71$$

Use 2 Bolts

Floor Beams

Dead Loads

Railing & Posts $= 22\#/\text{ft} \times 10 + 75\#/\text{post} \approx 300\#$

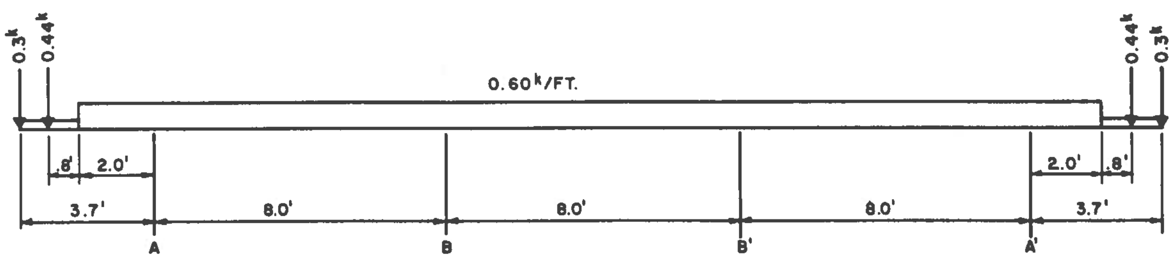
Curb $= 35\#/\text{ft} \times 10 \times 1.15$ (contongency) $\approx 400\#$

Deck (Including surfacing) $= 49\#/\text{ft}^2 \times 10 \times 1.14 = 558\#/\text{ft}$

Fl Bm. wgt. $= \underline{26\#/\text{ft}}$

$584\#/\text{ft}$

Use $\underline{600}\#/\text{ft}$



Dead Load Moments & Shears

$M_A = -0.3 \times 3.7 - 0.44 \times 2.8 - 0.6 \times (2.0)^2/2 = -3.54^{k'}$

$V_A = -1.94^k$

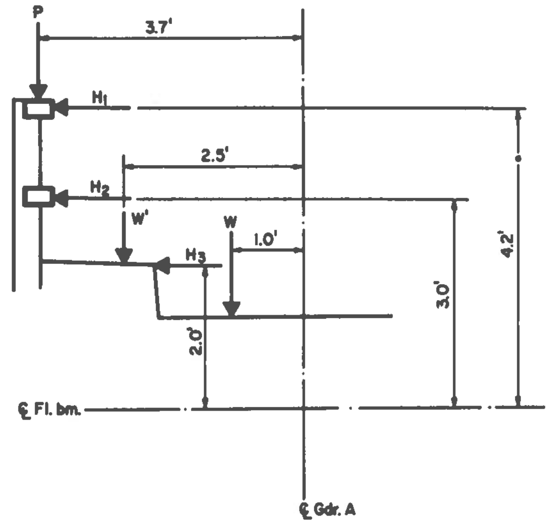
$M_{AB} = 0.6 \times .080 \times (8.0)^2 - .475 \times 3.54 = +1.40^{k'}$

$M_B = -06 \times .100 \times (8.0)^2 + .1875 \times 3.54 = -3.18^{k'}$

$V_{AB} = +2.45^k$ $V_{BA} = -2.35$

$M_{BB'} = 0.6 \times .025 \times (8.0)^2 + .1875 \times 3.54 = +1.62^{k'}$

$V_{BB'} = +2.40^k$



Loads on Fl. Bm. Cantilever

$P = .10^{k/\prime} \times 10 = 1.0^k$ $M = 1.0 \times 3.7 = 3.7^{k'}$
$H_1 = .15^{k/\prime} \times 10 = 1.5^k$ $M = 1.5 \times 4.2 = 6.3^{k'}$
$H_2 = .30^{k/\prime} \times 10 = 3.0^k$ $M = 3.0 \times 3.0 = 9.0^{k'}$
$H_3 = .50^{k/\prime} \times 10 = 5.0^k$ $M = 5.0 \times 2.0 = 10.0^{k'}$
$W = 16.55^k$ $M = 16.55 \times 1.0 = 16.6^{k'}$
$W' = 16.55^k$ $M = 16.55 \times 2.5 = 41.4^{k'}$

Loading (1) $DL + (LL\,(W) + 1) + P + H_2 + H_3$

Loading (2) $DL + (LL\,(W') + 1) + P + H_1 + H_2$

Max. Mom. @ A:

Loading (1) At basic allow. stresses

$M_{DL} = \quad -\ 3.5$
$M_{LL} = \quad -16.6$
$M_I = .30 + 16.6 = -\ 5.0$
$M_P + H_2 + H_3 = \quad \underline{-29.0}$
$\quad -54.1^{k'}$

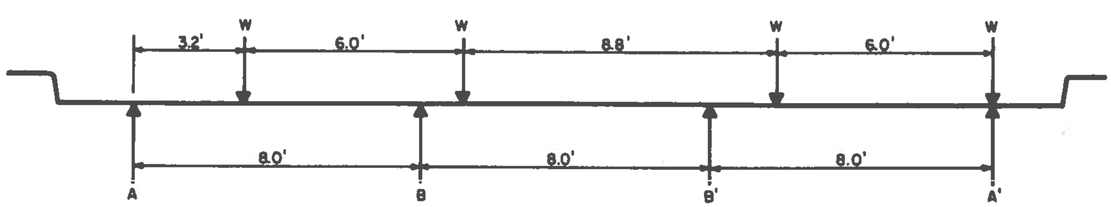
$V = -\ 1.94$
$V = -16.55$
$V = -\ 4.97$
$V = \underline{-\ 1.00}$
$V = -24.46^k$

$$Z\ reqd = \frac{54.1 \times 12}{27.0} = 24.1\ in^3$$

Loading (2) At 75% yield stress

$M_{DL} =$	$-\ 3.5$	$V = -\ 1.94$
$M_{LL} =$	-41.4	$V = -16.55$
$M_I = .30 \times 41.4 =$	-12.4	$V = -\ 4.97$
$M_P + H_1 + H_2 =$	$\underline{-19.0}$	$V = \underline{-\ 1.00}$
	$= -76.3^{k'}$	$V = -24.46^k$

$$Z\ reqd = \frac{76.3 \times 12}{37.5} = 24.4\ in^3$$



$M_{AB} = 8.0 \times W\,(.2042 - .0206 + .0086) = 1.538\,W$
$W = 16.0 \times 1.034 = 16.55^k$

$M_{DL} = \quad +\ 1.4$
$M_{LL} = 1.538 \times 16.55 = +\ 25.4$
$M_I = .30 \times 25.4 = +\ \underline{7.6}$
$= +\ 34.4^{k'}$

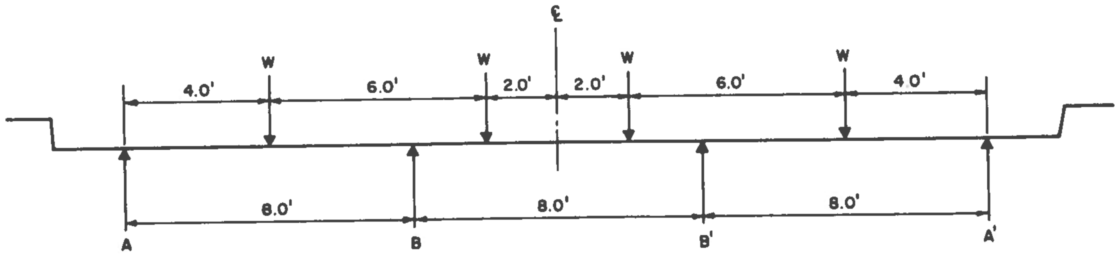
For full penetration butt welded splice at ℄ span AB with 2,000,000 loading cycles

$PL = 1.4/34.4 = .041$

$$F_r = \frac{16000}{1 - (.8 \times .041)} = \frac{16000}{.967} = 16{,}500 \text{ psi}$$

$$Z \text{ reqd} = \frac{34.4 \times 12}{16.55} = 24.9 \text{ in}^3$$

Max. Mom. @ B



$$M_B = 8.0 \times W(-.1000 - .0705 - .0405 + .0250) = -1.488\,W$$

$$\begin{aligned} M_{DL} &= & &= -3.2 \\ M_{LL} &= -1.488 \times 16.55 & &= -24.6 \\ M_I &= .30 \times (-24.6) & &= -7.4 \\ & & &= -35.2^{k'} \end{aligned}$$

If Floor Beams are butt welded to webs of girders

$$F_r = \frac{16000}{1 - \left(.8 \times \frac{3.2}{35.2}\right)} = \frac{16000}{.927} = 17{,}250 \text{ psi}$$

$$Z \text{ reqd} = \frac{35.2 \times 12}{17.25} = 24.5 \text{ in}^3$$

$M_{BB'}$—(Not critical by observation)

Max. Shear

$V_{BA} = 2.35 + (16.55 \times 1.328 \times 1.30) = 30.91^k$

A concrete slab and steel stringer bridge with 8'-0" stringer spacing requires a 7" slab. The expression for a floor beam of equal stiffness may be written as follows:

$$K \frac{E_s I_s}{L_s} = K \frac{E_c I_c}{L_c}$$

which can be reduced to

$$I_s = \frac{E_c I_c}{E_s}$$

Therefore, for a 10'-0" floor beam spacing

$$I_s = \frac{10 \times 12 \times (7)^3}{12 \times 10} = 316 \text{ in}^4 \text{ (Minimum floor beam)}$$

For Floor Beams Use 16 B 26

$I = 298.1 \text{ in}^4$

$Z = 38.1 \text{ in}^3 > 24.9$

$$\text{Max. web shear} = \frac{30.91}{15.65 \times .250} = 7.90_{ksi}$$

Floor Beam Bolted Field Splice

16 B 26	I =	298.1 in⁴
Less 4—⅞" holes	=	−70.7
Net I	=	227.4 in⁴

$$Z = \frac{227.4}{7.83} = 29.04 \text{ in}^3$$

$$\text{Allow M} = \frac{29.04 \times 27}{12} = 65.3^{k'}$$

Max. Mom. @ splice = $34.4^{k'}$

Allow Girder Mom. $= \dfrac{38.1 \times 27}{12} = 85.7^{k'}$

75% allow. = $64.3^{k'}$

Splice Design Mom. = $64.3^{k'}$

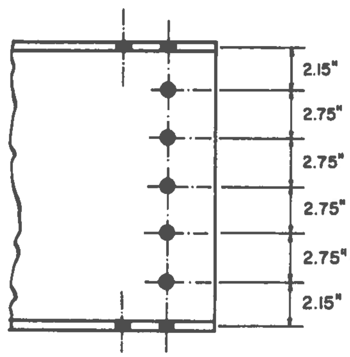
Design shear = $.75 \times .25 \times 15.65 \times 15.0 = 44.02^{k}$

Av. Flange stress $= \dfrac{64.3 \times 12 \times 7.65}{298.1} = 19.80_{ksi}$

Flg. force = $19.8 \times 5.5 \times .345 = 37.6^{k}$

¾" H.S. Bolts—Double shear = 11.92^{k}

Flange Bolts reqd $= \dfrac{37.6}{11.92} = 3.15$



Mom. of Inertia of bolts

$$\begin{aligned} 2 \times (2.75)^2 &= 15.1 \\ 2 \times (5.5)^2 &= 60.5 \\ 8 \times (7.65)^2 &= \underline{468.2} \\ & 543.8 \end{aligned}$$

Stress/Flg. Bolt $= \dfrac{64.3 \times 12 \times 7.65}{543.8} = 10.85^{k} < 11.82$

Stress/web Bolt

From Mom. $= \dfrac{64.3 \times 12 \times 5.5}{543.8} = 7.80^{k}$

From Shear = $44.02/5 = 8.80^{k}$

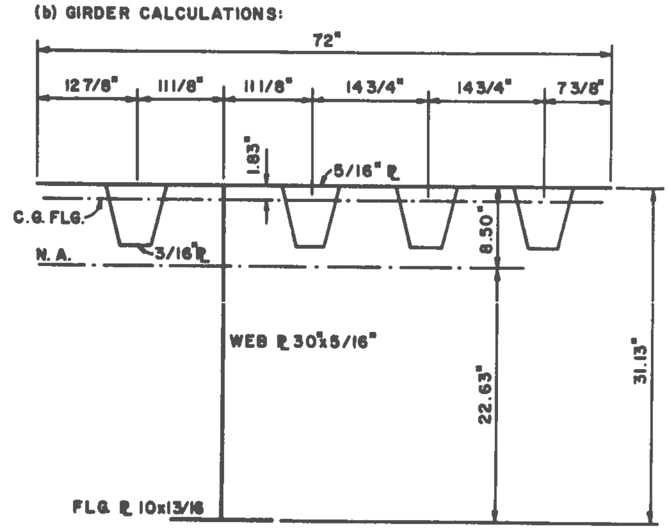
Max. stress = $\sqrt{(7.80)^2 + (8.80)^2} = 11.77^{k} < 11.92$

Min. Splice Material o.k. by observation

Ea. Flange Splice—$\left\{\dfrac{\text{1-PL } 5\tfrac{1}{2} \times \tfrac{5}{16}}{\text{2-PLs } 2\tfrac{1}{2} \times \tfrac{5}{16}}\right\}$ 4-¾" H.S. Bolts

Web Splice—2-PLs 5½ × $\tfrac{5}{16}$ × 1′-0½″—5-¾" H.S. Bolts

(b) GIRDER CALCULATIONS:



ASSUMED EXTERIOR GIRDER SECTION

Exterior Girder Properties

Top Flg. A	=	36.75
Web A	=	9.38
Bot. Flg. A	=	8.13
Totl. Area	=	54.26 in²

$I = 6990 \text{ in}^4$
$Z_T = 822 \text{ in}^3$
$Z_B = 308 \text{ in}^3$

Dead Loads

Railing—$.030^{k/l} \times 11.7/8.0 = .044$

Curb—$.035^{k/l} \times 10.8/8.0 = .047$

Deck PL—$.0128 \times 10 \times 5.0/8.0 = .080$

Ribs—$.0121 \times 6 \times 4.0/8.0 = .036$

$.0121 \times 1 \times 8.9/8.0 = .013$

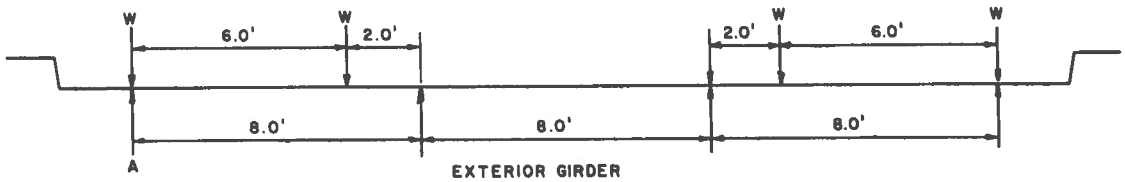
Fl. Bins—$.026 \times 11.8 \times 1/10 \times 5.9/8 = .023$

Flange & Web— $= .060$

Surfacing—$.020 \times 10 \times 5.0/8.0 = \underline{.125}$

Total DL $= .428^{k}/\text{ft}$

Use $\underline{.43^{k}/\text{ft}}$



EXTERIOR GIRDER

Location of Wheels for Max. Ext. Gdr. LL

Simple Spans—$A = 1.0\,W + 2/8\,W = 1.25\,W$

or .625 Lanes

Cont. spans—$A = W(1.000 + .164 + .022) = 1.186\,W$

or .593 Lanes

AASHO—$A = W\left\{\dfrac{8.0}{4.0 + 8/4}\right\} = 1.333\,W$

or .667 Lanes

Max. Mom.

$M_{DL} = .43 \times (50)^2/8 = 134.3$

$M_{LL} = .667 \times 627.9 = 418.8$

$M_{I} = .286 \times 418.8 = \underline{119.8}$

$672.9^{k'}$

Max. Shear

$V_{DL} = .43 \times (50)^2/2 = 10.7$

$V_{LL} = .667 \times 58.5 = 39.0$

$V_{I} = .286 \times 39.0 = \underline{11.2}$

60.9^{k}

Max. Bot. Flg. stress $= \dfrac{672.9 \times 12}{308} = 26.2_{\text{ksi}} < 27.0$

Max. Top Flg. stress

Gdr. stress $= \dfrac{672.9 \times 12}{822} = 9.82$

Long. Rib compr. $= \underline{7.31}$

Max. Long. $= 17.13$

Max. Transv. $= 18.96$

Max. Compr. $= \sqrt{(17.13)^2 + (18.96)^2} = 25.6_{\text{ksi}} < 27.0$

Web Stresses

For $\frac{5}{16}$" PL: $D/t = (30 - 50)/.3125 = 94.4 > 58$

$t = \dfrac{D\sqrt{f_v}}{7500}$

Allow. $f_v = \left[\dfrac{7500}{D/t}\right]^2$

Allow. $f_v = \left[\dfrac{7500}{94.3}\right]^2 = 6320$ psi

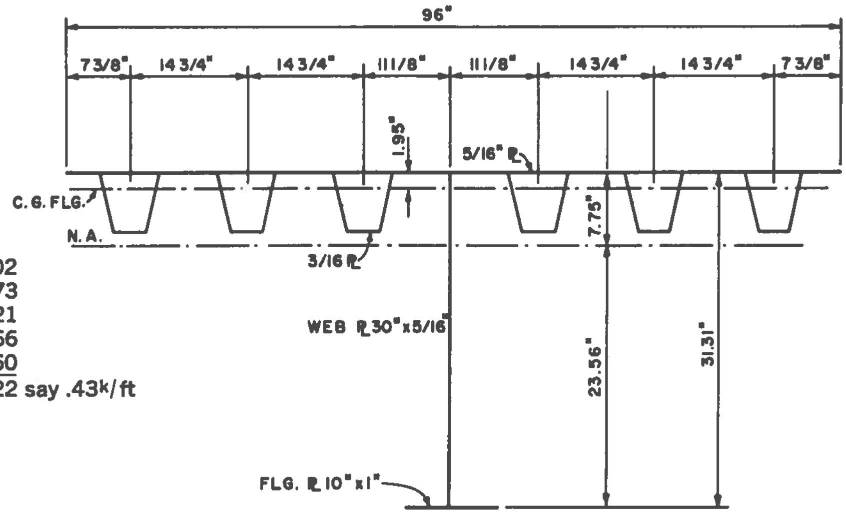
Allow. $V = 9.38 \times 6.32 = 59.3^{k}$

Interior Girder Properties

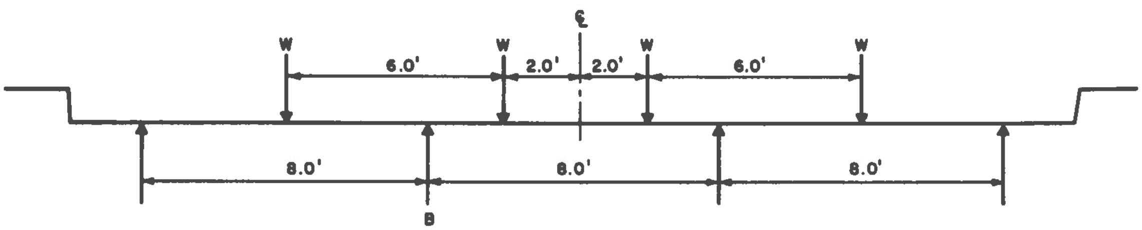
Top Flg. A $= 51.38$
Web A $= 9.38$
Bot. Flg. A $= \underline{10.00}$
Tot. Area $= 70.76 \text{ in}^2$
$I = 8570 \text{ in}^4$
$Z_T = 1106 \text{ in}^3$
$Z_B = 364 \text{ in}^3$

Dead Load

Deck PL	$- .0128 \times 8.0$	$= .102$
Ribs	$- .0121 \times 6$	$= .073$
Fl. Bm.	$- .026 \times 8.0/10$	$= .021$
Flg. & Web	$-$	$= .066$
Surfacing	$- .020 \times 8$	$= \underline{.160}$
Total Gdr. DL		$= .422$ say $.43^k/\text{ft}$



ASSUMED INTERIOR GIRDER SECTION.



Location of Wheels for Max. Int. Gdr. LL
Simple Spans—$B = 4/8\,W + 6/8\,W + 2/8\,W = 1.50\,W$
or .75 Lanes
Cont. Spans—$B = W\,(.725 + .8505 + .2605 - .1500)$
$= 1.686\,W$ or .843 Lanes

AASHO — $B = W\,(8.0/5.5) = 1.454\,W$
or .727 Lanes

Max. Mom.
$M_{DL} = .43 \times (50)^2/8 = 134.3$
$M_{LL} = .843 \times 627.9 = 529.3$
$M_I = .286 \times 529.3 = \underline{151.4}$
$= 815.0^{k'}$

Max. Shear
$V_{DL} = .43 \times 50/2 = 10.7$
$V_{LL} = .843 \times 58.5 = 49.3$
$V_I = .286 \times 49.3 = \underline{14.1}$
$= 74.1^k$

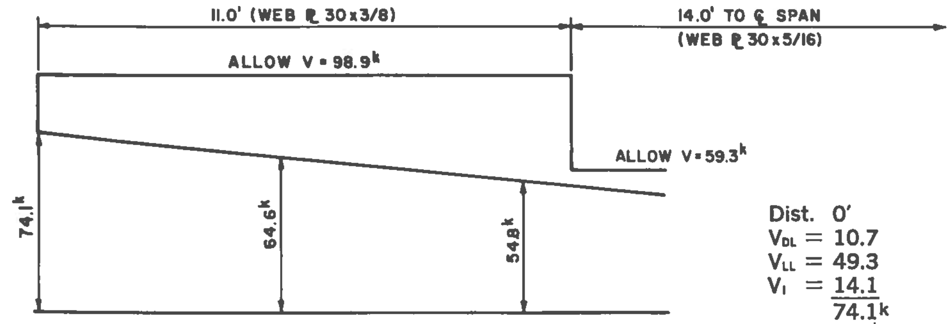
Interior Girder

Max. Bot. Flg. stress $= \dfrac{815.0 \times 12}{364} = 26.8_{\text{ksi}} < 27.0$

Max. Top Flg. stress
Gdr. stress $= \dfrac{815.0 \times 12}{1106} = 8.85$
Long. Rib compr. $= \underline{7.31}$
Max. Long. $= 16.16$
Max. Transv. $= 18.96_{\text{ksi}}$

Max. Compr. $= \sqrt{(16.16)^2 + (18.96)^2} = 25.0_{\text{ksi}} < 27.0$

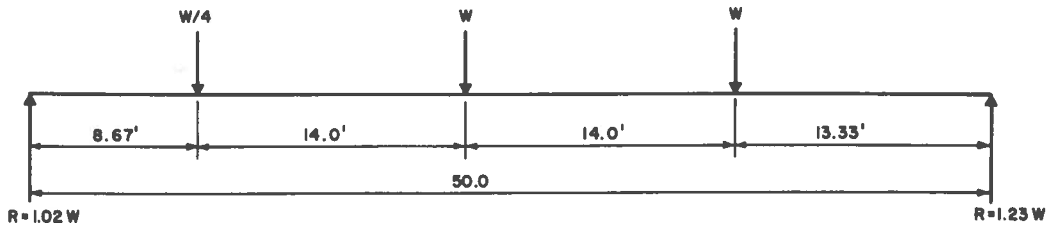
Web Stresses
For $30 \times \frac{5}{16}$ PL Allow. $V = 59.3^k$
(see Ext. Gdr. Calculations)
For $30 \times \frac{3}{8}$ PL Allow. $V = 98.9^k$



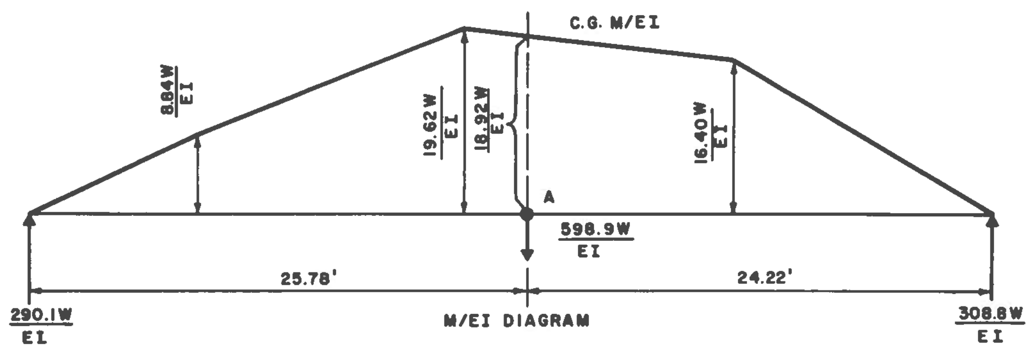
MAX SHEARS

Dist.	0′	5′	10′
V_{DL} =	10.7	8.6	6.4
V_{LL} =	49.3	43.3	37.2
V_I =	14.1	12.7	11.2
	74.1^k	64.6^k	54.8^k

Live Load Deflections



LOADING FOR MAXIMUM MOM. AND DEFL.



M/EI DIAGRAM

$$\frac{M_A \times EI}{W} = 308.8 \times 24.22 = 7479$$
$$-\frac{16.40}{2} \times 13.33 \times 15.34 = -1677$$
$$-16.40 \times 10.89 \times 5.44 = -972$$
$$-\frac{2.52}{2} \times 10.89 \times 3.63 = -\underline{50}$$
$$4780$$

$$\text{Max. Defl. (@ A)} = \frac{4780W}{EI} \times 1728 = \frac{8{,}259{,}800W}{EI}$$

$$\text{For } E = 29{,}000_{ksi}, \text{ LL Defl.} = \frac{284W}{I}$$

$$\text{Total I for Bridge} = 2(6990 + 8570) = 31{,}120 \text{ in}^4$$

$$\text{Avg. LL \& I Defl.} = \frac{284 \times 16.0 \times 4 \times 1.286}{31{,}120} = 0.750''$$

$$\text{Max. Allow. LL Defl.} = \frac{50 \times 12}{800} = 0.750''$$

DL Deflections

$$\text{For Ext. Girders} = \frac{5 \times .43 \times (50)^4 \times 1728}{384 \times 29000 \times 6990} = 0.298''$$

$$\text{For Int. Girders} = \frac{5 \times .43 \times (50)^4 \times 1728}{384 \times 29000 \times 8570} = 0.244''$$

Estimate of Cost:

Bridge Railing and posts— 102 Lin. Ft.	@ \$5.50 = \$ 561
Bridge Deck & Girders (COR-TEN)— 54,880 Lbs.	@ \$0.22 = \$12,074
	Total Cost = \$12,635