Orthotropic Bridge Decks

1. THE ORTHOTROPIC DESIGN CONCEPT

There is a growing interest in this country in the use of orthotropic bridge design and construction, a system now commonly used in Europe.

With conventional bridge structures, the three main elements—longitudinal main girders, transverse floor beam, and lighter longitudinal stringers or stiffeners—all act independently of each other. Usually an 8″ thick concrete floor distributes the applied loads; see Figure 1(A).

In contrast, all elements of the orthotropic structure work together; see Figure 1(B). This new system uses a thin steel deck plate across the entire width and length of the bridge, and this serves as the top flange plate of the (1) longitudinal main girders, (2) transverse floor beams, and (3) lighter longitudinal stiffeners. The deck plate also contributes to the torsional resistance of the stiffeners when it forms a closed section.

Having a common top flange member, all three elements act and load up together in the most efficient manner. The steel deck plate is topped with a light 1½″ thick asphalt wearing surface for complete elimination of the heavy concrete floor.

The combined orthotropic deck structure acts as a single plate or membrane with three separate sectional properties: bending resistance about the x-x axis (transverse to the length of the bridge), bending resistance about the y-y axis (parallel to the bridge), and torsional resistance about the y-y axis. A concentrated load placed upon the deck plate is distributed over a wide area to several adjacent floor beams. The longitudinal stiffeners below this load act as beams on elastic supports. With increasing load, the rather flexible deck and stiffeners spread the load over a greater area. This action has been confirmed by many tests on models as well as actual bridges.

In the tests of the model of one bridge, the computed test load corresponding to maximum allowable design stress was 2.06 tons. The computed ultimate load was 5.6 tons. During testing, measurements indicated there was perfect elastic behavior up to an actual load of 4.1 tons. When loaded above the elastic limit, there was no rapid and unrestrained increase in deflection as is customary in the usual bending of beams; rather the deflections increased linearly just a little faster than the applied load. At a load of 48 tons, a crack started to appear in the stiffener region, and at 56 tons this had spread over the entire depth of the stiffener. This test indicated an apparent factor of safety of 27 to 1.

With optimum use of welding, orthotropic construc-

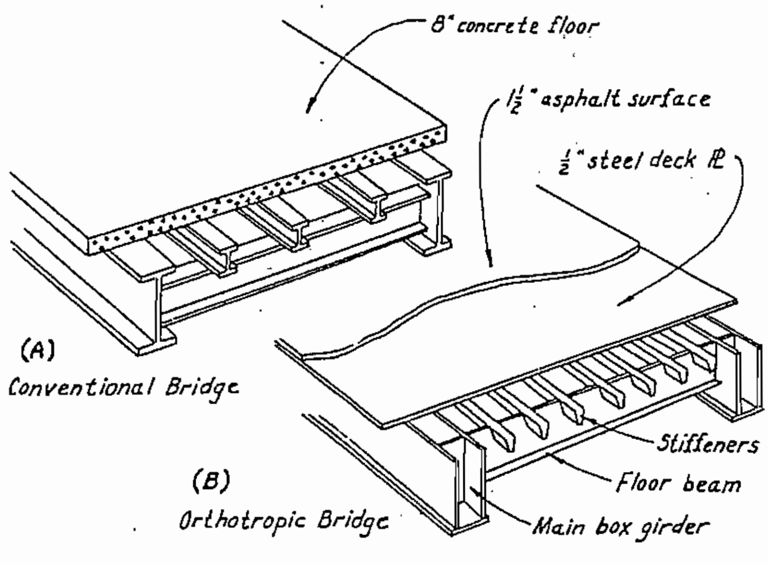


FIGURE 1

tion results in the bridge superstructure usually weighing only half as much as would result from any other design system. This weight saving is such a tremendous advantage on long span bridges, that orthotropic design is rapidly replacing truss design on all European bridges having spans of 400′ or more, and should do the same in this country.

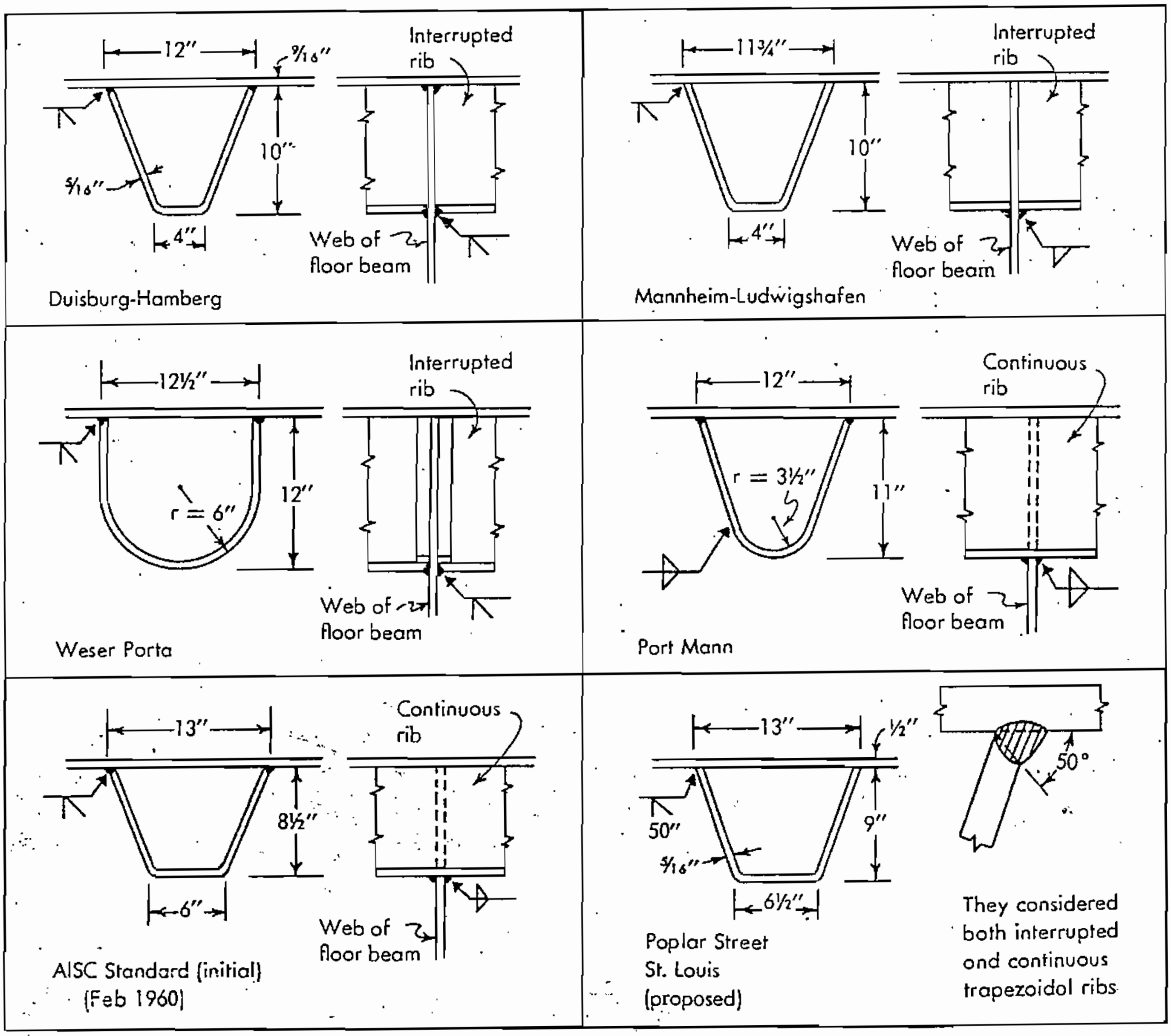
AISC has published an excellent design manual on "Orthotropic Steel Plate Deck Bridges" by Roman Wolchuk (1963). It contains theory, methods of design, and suggested details of orthotropic bridges.

This type of bridge design would be impractical without the extensive use of welding. The miles of welded joints afford a good opportunity to subassemble the sections for automatic downhand welding and modern fabricating methods. Since numerous identical deck sections are required, they may be set up in a jig and automatically submerged-arc welded with minimum time and cost.

2. JOINING LONGITUDINAL STIFFENERS TO DECK PLATE

In European orthotropic bridge design, longitudinal stiffeners are commonly of trapezoidal cross-section for torsional rigidity. American design interest appears to favor this approach; see Figure 2. Although not too clear on the sketch of the Port Mann bridge, the edge

FIG. 2—Typical Hollow Trapezoidal Ribs and Connecting Welds



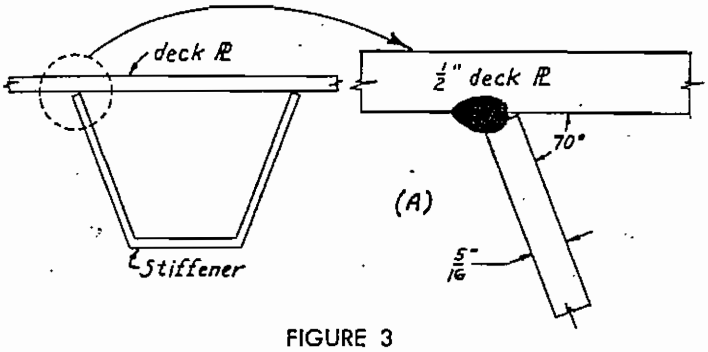


FIGURE 3

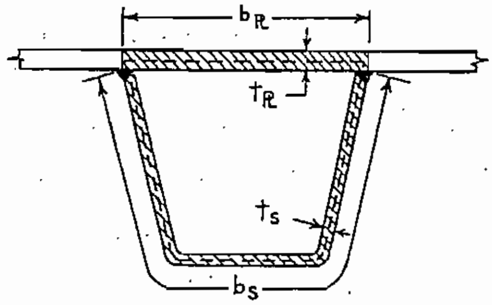


FIGURE 4

of the stiffener was cut square without any bevel. It was shown in tests by the fabricator that a single pass made with the automatic submerged-arc welder would produce a sound weld with throat greater than stiffener thickness; see Figure 3.

The torsional resistance of any closed tubular section, as indicated by Figure 4, is:

$$\boxed{R = \frac{4\,[A]}{\int \frac{dt}{ds}} = \frac{4\,[A]}{\frac{t_{℞}}{b_{℞}} + \frac{t_s}{b_s}}}$$

where:

$[A]$ = area enclosed by the trapezoid

$t_{℞}$ = thickness of deck plate

t_s = thickness of stiffener

$b_{℞}$ = width of deck plate within region of stiffener

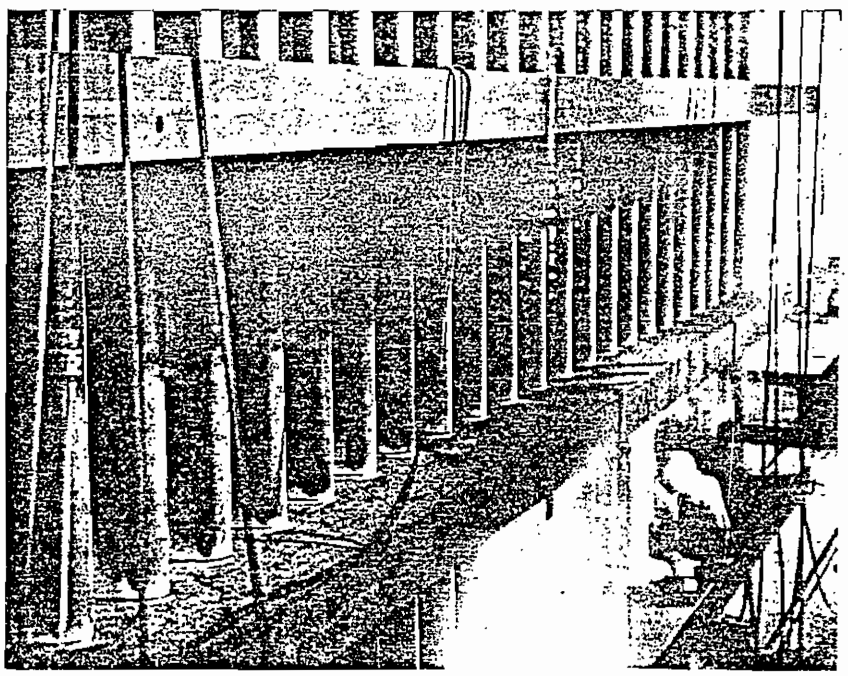
b_s = undeveloped width of stiffener

The Design Manual for Orthotropic Steel Plate Deck Bridges multiplies this torsional resistance (R) by a reduction factor (μ) which has been determined by testing of various shapes of stiffeners. This factor is affected by the shape of the stiffener.

Stiffeners can be readily formed to the trapezoidal shape on a press brake. Because of the tonnage required, it might be more economical to purchase a special mill-rolled section for the stiffeners; see Figure 5. Thus the outer portions of the plate width which become webs of the built-up trapezoid section are rolled thinner, and the central portion is left thicker for the lower flange. This places the material where required, further reducing the bridge weight and tonnage of steel required. The plate could be rolled to the final trapezoid section, thus eliminating the braking operation Lengths of this section would nest and present no problem in shipping.

Another refinement would be to provide slightly greater thickness at web extremities so as to give more bearing against the deck plate and greater throat to the connecting weld.

In designing the Port Mann Bridge in British Columbia, Canada, engineers specified orthotropic deck construction for maximum weight reduction and dollar economy. Deck plate is stiffened by longitudinal trough-shaped stringers formed by press-brake. Welding of stringers to transverse beams is done by a progressive assembly technique . . . for near continuous-flow production.



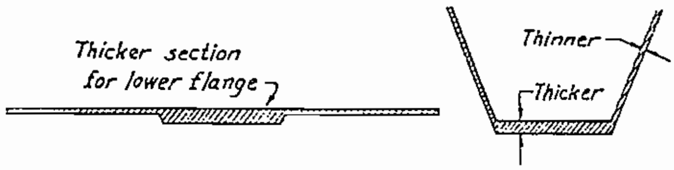


FIGURE 5

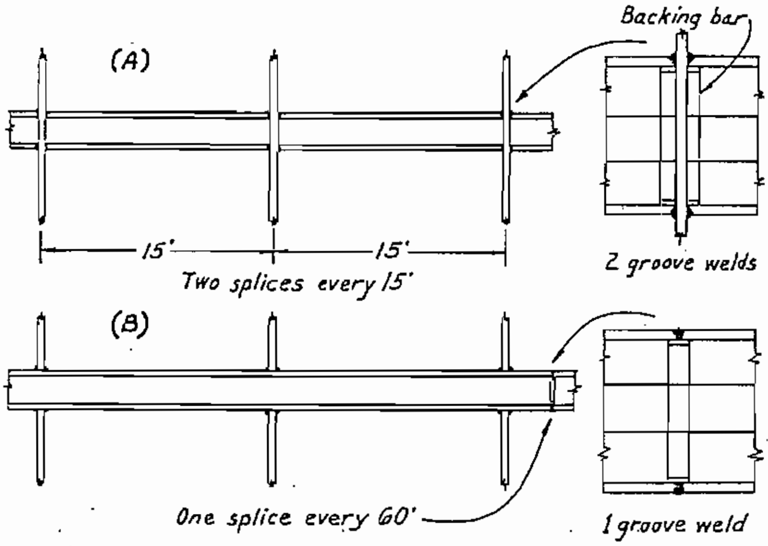


FIGURE 6

3. FIELD SPLICE OF LONGITUDINAL STIFFENERS

There are two basic methods for detailing the intersection of longitudinal stiffeners and transverse floor beams; see Figure 6.

(A) Following the common European practice, the floor beam webs run continuous and stiffeners are cut to fit between the beams. The stiffeners are thus limited to about 15′ in length, and the main bending stresses of the structure in the stiffeners must be transferred transversely through the web of each floor beam by means of groove welds (T joint). There might be a question of the possibility of a lamination in the web opening up because of the transverse force applied through it. This method requires a large number of field groove welds to be made in the vertical and overhead position. There are 2 welds at each beam per stiffener.

(B) An alternate method would be to have the trapezoid stiffeners run continuous throughout the length of the structure, with webs of the floor beams cut out to fit around the stiffeners. This would eliminate any questions as to the safe transfer of main bending stresses.

This method would greatly reduce the required field welding. For example, the stiffeners could be shop fabricated into 60′ lengths; this would require just a single groove weld in the field every 60′. This would be a single groove butt joint in contrast to the 2 groove welds at each floor beam required by Method A. The critical field welding thus would be only ⅛ of that required by Method A.

In a translation of a German paper, "Fatigue Tests on Hollow Rib Connections" by H. Hansch and G. Muller, results of fatigue testing three different details of longitudinal stiffeners were summarized:

1. The longitudinal stiffeners were interrupted at the transverse floor beam webs and joined by fillet welds to the webs of the floor beam.

2. The longitudinal stiffeners were interrupted at the floor beam webs, but were welded with single bevel groove welds to the webs of the floor beams.

3. The longitudinal stiffeners ran continuously through the floor beam webs.

The results showed the continuous stiffener (1) to have the highest fatigue strength, $\sigma = 28{,}000$ psi, when tested with a stress range of

$$K = \frac{\min}{\max} = + .2$$

The shape of the closed tubular longitudinal stiffener tested had no appreciable effect upon the test results. Cold forming of the stiffeners had no effect. They recommend that the designer place the field splice of the stiffeners in low-stressed regions.

4. SHOP FABRICATED SUBASSEMBLIES

It is possible to fabricate nearly the entire deck of the bridge, in sections, under optimum shop conditions and thereby minimize the amount of field welding. This includes deck sections lying between the main box girders, and any sections to be cantilevered out from the box girder.

The deck unit which is to rest between the main box girders can be made initially in three sections. For an average bridge, each of these prefabricated sections, 9′ wide by 60′ long, would weigh about 8¾ tons; see Figure 7.

Three of these sections would be laid out, still upside down, and tack welded together; see Figure 8(A). This work would preferably be done on the final means of transport, in some cases a barge. Each longitudinal joint of the top deck plate can be made with a two-pass weld; one pass on each side using a submerged-arc automatic welder. This joint is a simple square-butt joint without any backing bar, and requires no beveling of plate edges. After making the first pass, the four floor beams are manually welded in place. Each beam consists of a bottom flange plate and a web plate having trapezoidal cutouts along the top edge to fit around each stiffener.

With the transverse floor beam welded in place,

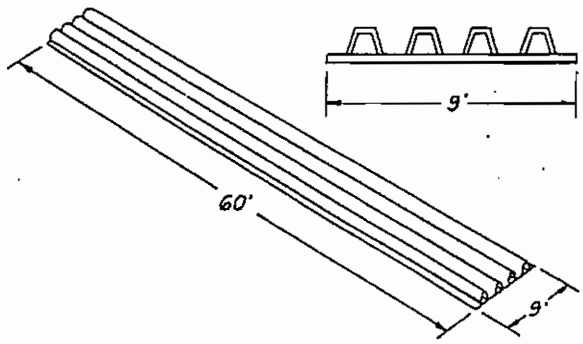


FIGURE 7

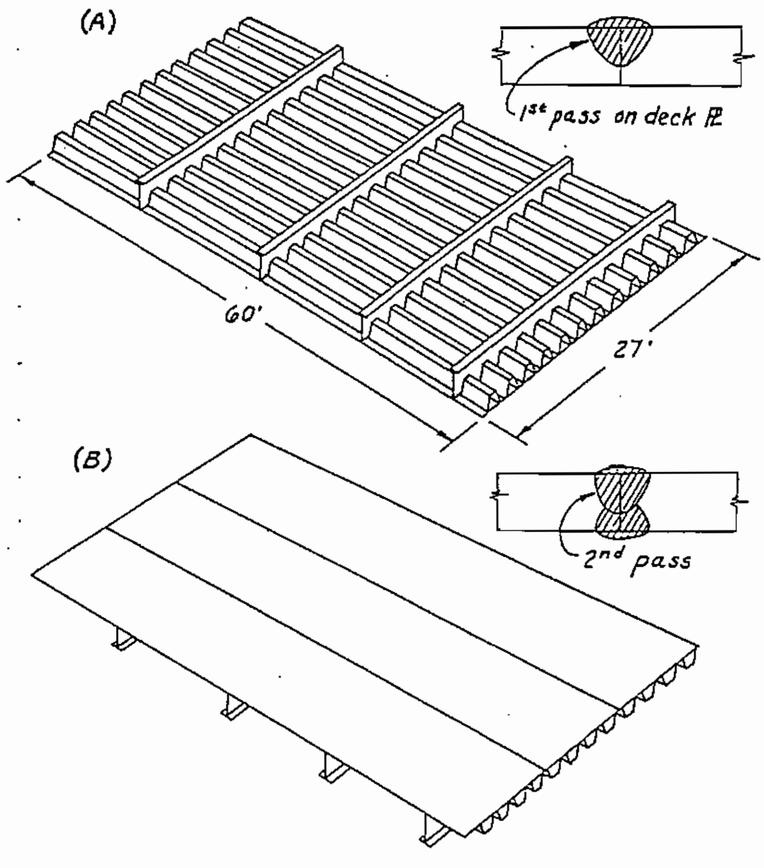


FIGURE 8

the entire unit can be turned over without undue strain on the incomplete butt weld. A second pass is taken to complete the automatic welding of the longitudinal joints, all in the downhand position; see Figure 8(B). The result is a complete deck unit, 27′ × 60′, weighing about 29 tons, to be hoisted from the barge into position between the two main box girders.

The Port Mann bridge deck panels were fabricated and welded in the shop as units 65′ wide, the width of the deck lying in between the main longitudinal girders, and 25′ long, the distance between the transverse floor beams. These panels weighed between 32 and 36 tons, depending upon the deck plate thickness. In Europe, panels up to 58′ × 18′ have been fabricated and transported by barges to the site. The Save River bridge had prefabricated panels weighing 27.5 tons. The Mannheim-Ludwigshafen bridge was erected in panels 18.5′ wide and 60′ long. The Severin bridge in Cologne was erected in panels 62.8′ wide and 47 to 54′ long.

5. FIELD ERECTION

The entire superstructure probably would be erected in units, starting from a pier support and cantilevering out. A traveling crane could place the individual units. For any given segment of the span, the main longitudinal box girders would be put into position first. The field splice of the top flange deck plate should be welded because the 1½″ thick asphalt floor to be applied leaves little room for splice plates and bolts. The erection bolts probably should be on the girder webs. The girder's bottom flange may vary from ¾″ to 3 or 4″ thick plate, and could be spliced by field welding because field bolting of this thick plate would be costly.

Transverse shrinkage of the weld on the ½″ deck plate within this box girder is estimatel at about .03″, and shrinkage of the groove weld of a 3″ bottom flange plate at about .10″. Under this condition, a suggested procedure is to weld the bottom flange to about ⅔ completion, then weld the top deck simultaneous with welding the remaining ⅓ of the bottom flange. In this manner, both flanges should pull in together evenly.

The next step would be erection of the subassembled deck unit between these two main box girders.

6. FIELD WELDING

With a deck unit raised into place, the ends of each floor beam would be field welded to the main box girders. The two longitudinal joints and one transverse joint of the ½″ deck plate should be welded in a single pass with a submerged-arc tractor. Plates should be partially beveled at the top and a backing bar used so that full-penetration welds can be made in the downhand position; see Figure 9.

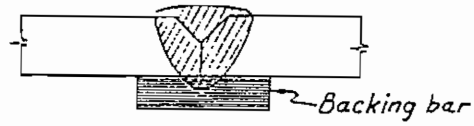


FIGURE 9

Longitudinal stiffeners would be field spliced by manually groove welding the butt joint using a light backing bar placed on the inside of the trapezoid, very similar to pipe welding. The upper edge of the stiffener could be notched at this joint so a backing bar can run continuously across the deck to facilitate automatic welding of the deck plate transverse joint. Under these conditions, the joints of deck plate and stiffeners should be offset at least 2″, as shown in Figure 10, so each deck unit can be lowered down without interference of the backing bars.

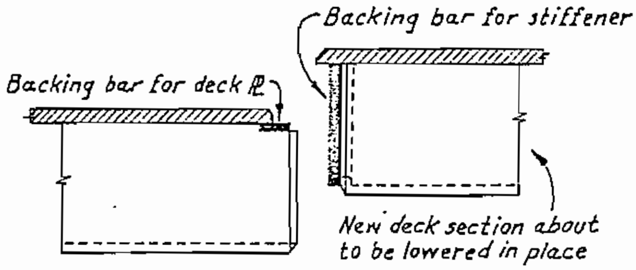


FIGURE 10

If there is any doubt about the fit-up of multiple stiffeners for field splicing, ends of the stiffeners can be left unwelded to the deck plate for about a foot. This will permit them to be individually aligned horizontally for welding.

If specific dimensions of the stiffener indicates a possible problem in acessibility for the weldor in making the field splices, the deck plate can be left short by about 10″ from each end of the section; see Figure 11. This would also allow the back of the joints on the inside of the trapezoid stiffener to be root gouged and a root or back pass made. A 20″ wide deck plate section would then be inserted, and two transverse groove welds made. This would double the length of transverse welds for splicing the deck plates; however, all of this welding would be automatic, single pass work. Ends of the stiffeners would then be overhead welded to this deck insert; as shown in Figure 11.

An alternate way to field splice the trapezoidal stiffeners is to place the bevel on the inside and a backing bar on the outside; the weldor then makes all the splices while working from the top of the deck.

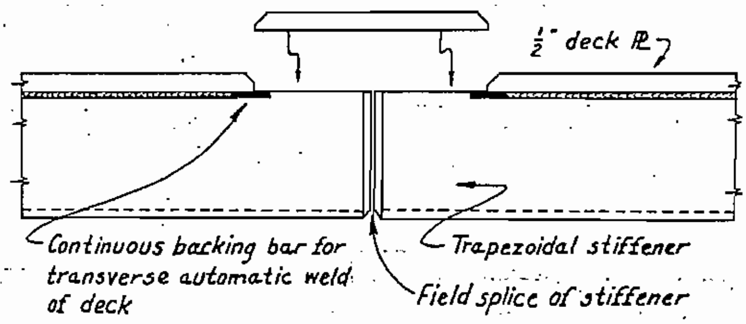


FIGURE 11

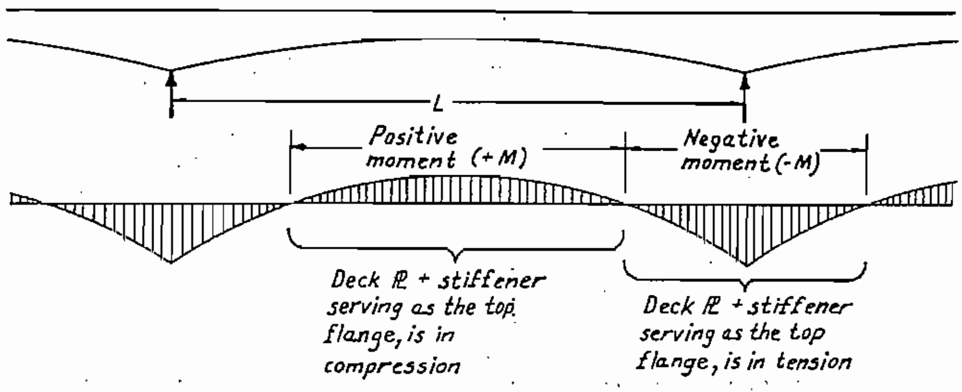


FIGURE 12

7. RADIOGRAPHIC INSPECTION

This type of inspection should be limited to critical joints which the Engineer should select. Fatigue conditions that reduce the allowable stress in design may indicate such a need; for example, groove welded butt joints subject to tension, a wide range of stress, a high stress, and a large number of cycles. As the factors that produce fatigue loading are reduced, the necessity for radiographic inspection is likewise reduced.

If all of the groove welds in the deck plate are made by the submerged-arc automatic process, proper procedures can be established to insure good welding. This should eliminate the need for costly radiographic inspection of these welds, although limited spot checks could be made.

Any field splice in the lower flange of the main box girders in a region of positive moment, might be inspected by radiography.

Field splices in the longitudinal stiffeners must be considered from the type of loading:

1. The stiffener serves along with the deck plate as the top flange of the main structure, and as such is subjected to tension in the negative moment region near the pier supports. However, this comes from the dead load of the structure and any live load spread over a rather large area, thus the range of stress variation and the number of stress cycles would be relatively small; see Figure 12.

2. The stiffener serves along with the deck plate as a short beam between floor beams, and any localized wheel load would produce a wide range in stress and the number of applications could be very high. However, by using Method B to detail the network of floor beams and stiffeners the only critical welds would occur at about every 60′ of bridge length. The influence lines, see Figure 13, show the moment due to concentrated wheel load at given points as the load progresses along the span between floor beams. By locating the field splice of the stiffener at a point about $\frac{1}{10}$ L along the span between supporting floor beams, the bending stress on the weld is rather low and without much fluctuation.

Spot checks of the stiffener field splices by gamma ray inspection, if required, could be made by drilling a small hole in the ½″ deck plate and lowering the capsule down halfway into the interior of the trapezoidal area, with the film wrapped around the outside

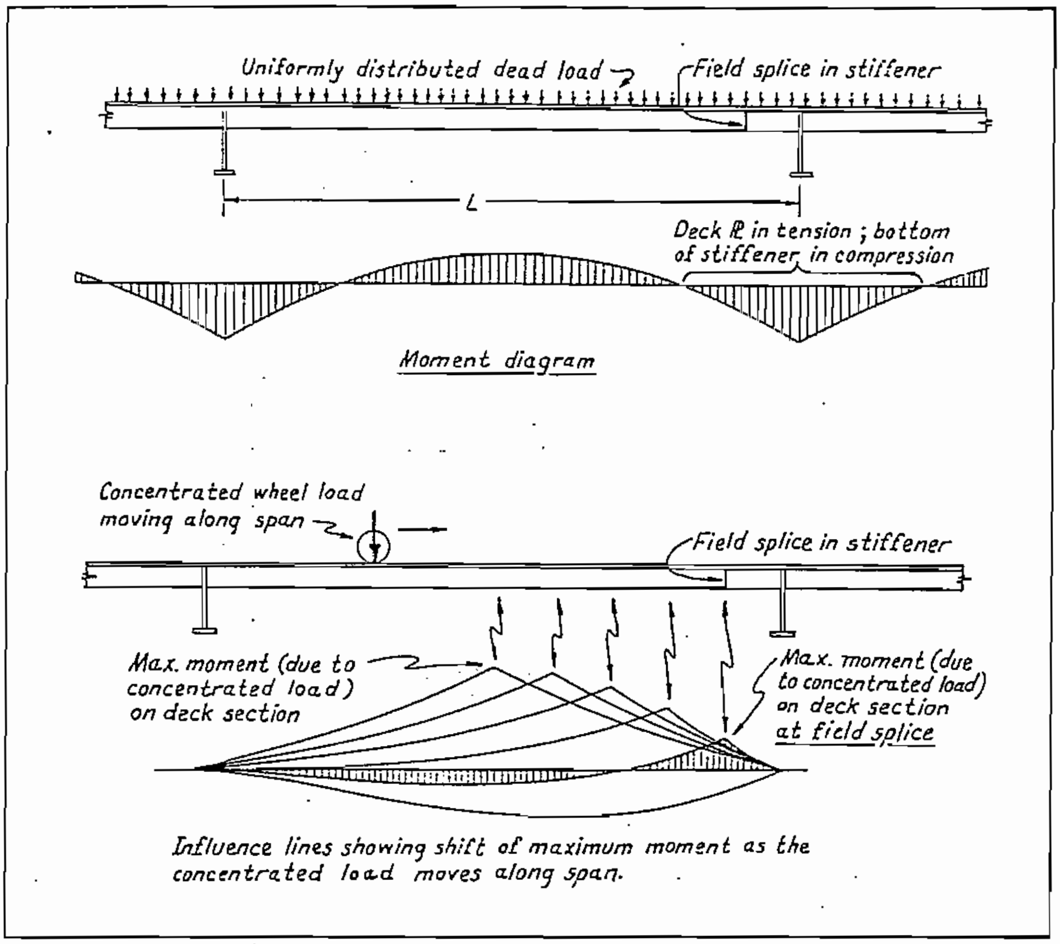


FIGURE 13

of the stiffener. This hole can be filled later by welding, or by tapping it and screwing a pipe plug into it.

8. WELDOR QUALIFICATION

In addition to the standard AWS weldor qualification test, it would be well for those men assigned to field weld the stiffeners to first weld a test joint of this splice in position. This can be given a visual inspection, including sawing of the joint at one or more points and etching to determine if proper fusion was obtained. It might be well to consider weldors who have had some experience in pipe welding.

Problem 1

An orthotropic deck is to be fabricated in units 104″ wide containing 4 trapezoidal stiffeners each 13″ wide and on 11″ centers. The stiffeners are welded to the ⅜″ deck plate along their edges. If these units are 30′ long, estimate the amount of bending or camber due to the shrinkage of the welds; see Figure 14.

To find the properties of this section, select reference axis (x-x) along underneath surface of deck plate. This is almost through the center of gravity of the 2 welds, and the resulting distance to the neutral axis (n) will also be the distance between the neutral axis and the center of gravity of welds (d).

$$I_{NA} = I_x + I_g - \frac{M^2}{A}$$

$$= (279.87) - \frac{(-35.412)^2}{(16.179)} \quad \text{(From Table A)}$$

$$= 279.87 - 77.51$$

$$= 202.36 \text{ in.}^4$$

$$n = \frac{M}{A}$$

$$= \frac{(-35.412)}{(16.179)}$$

$$= -2.19'' \text{ also } = d$$

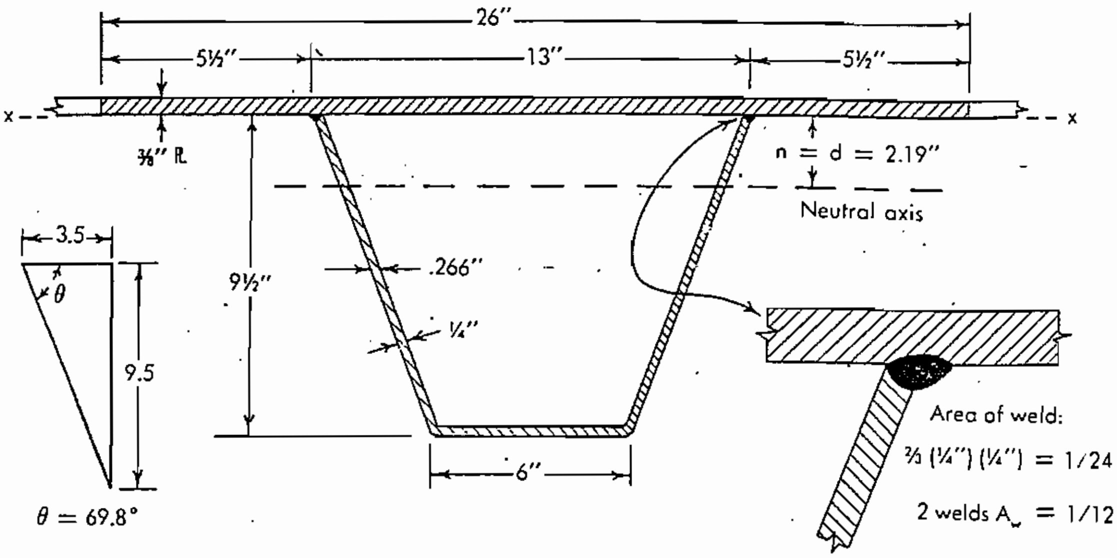


FIGURE 14

TABLE A

	Plate	A	y	M = A y	I_x = M y	I_g
Ⓐ	26" x ⅜"	9.750	+ $\frac{3}{16}$"	+ 1.828	+ .34	.11
Ⓑ	.532" x 9½"	5.054	− 4.75"	− 24.007	+114.03	38.00
Ⓒ	5½ x ¼"	1.375	− 9.625"	−13.233	+127.38	.007
	Total →	16.179		−35.412	279.87	

bending or camber L = 30′ = 360″

$$\Delta = \frac{.005\ A_w\ d\ L^2}{I}$$

$$= \frac{.005\ (\frac{1}{12})\ (2.19)\ (360)^2}{(203.36)}$$

= .585″ (ends would go up this amount)

This means when the 30′ long unit is upside down for welding, the fixture should be curved sufficiently to pull the central section of the unit down by this amount (.585″).

Problem 2

The orthotropic deck used in the Port Mann bridge in British Columbia consists of trapezoidal stiffeners with rounded bottoms spaced on 24″ centers and welded to a ½″ to $\frac{7}{16}$″ deck plate. These deck sections are shop welded into panels about 65′ wide, the width of the bridge in between the main longitudinal girders, and 25′ long; as shown in Figure 15. Estimate the amount of bending or camber due to the shrinkage of the welds.

In order to find the property of this built-up section, it is necessary to know the properties of the arc of a circle which forms the round bottom portion.

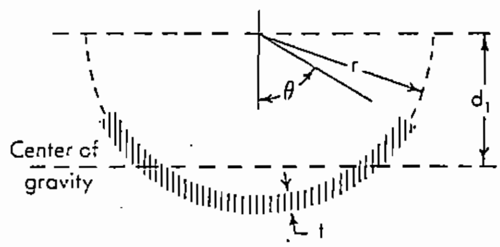


FIGURE 16

It can be shown that the following is true:

$$A = 2\ t\ r\ \theta$$

$$d_1 = \frac{r \sin \theta}{\theta}$$

$$I_g = t\ r^3 \left[\theta + \tfrac{1}{2} \sin 2\,\theta - \frac{2 \sin^2 \theta}{\theta} \right]$$

(about center of gravity)

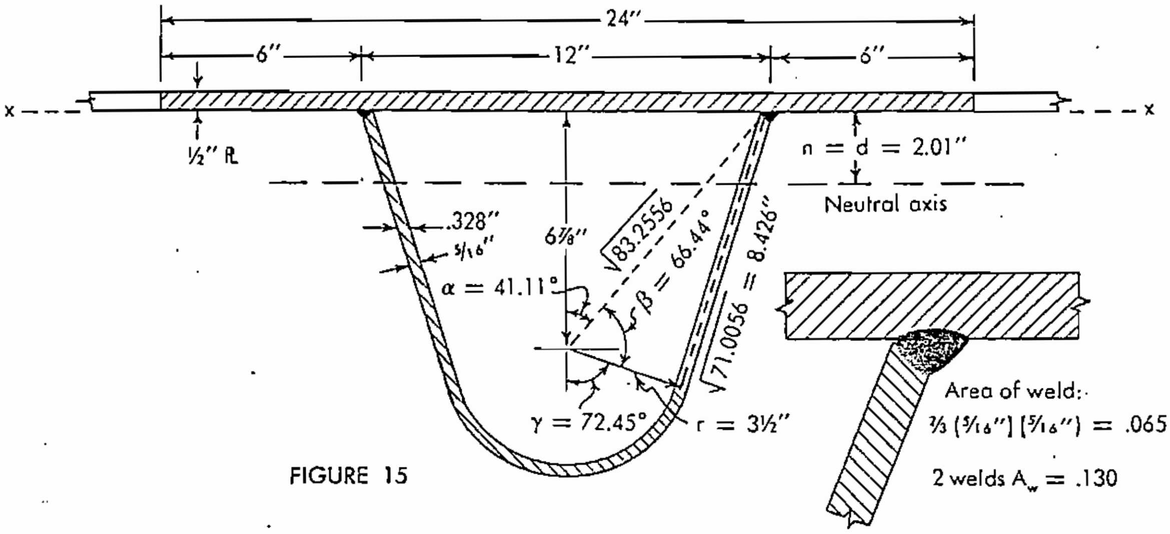


FIGURE 15

TABLE B

	Plate	A	y	M = A y	I_x = M y	I_g
(A)	24" x ½"	12.00	+ .25	+ 3.00	+ .75	.25
(B)	.656" x 6⅞"	4.51	−3.4375	−15.50	+ 53.29	17.73
(C)	round bottom	2.76	−9.515	−26.26	+249.87	1.46
	Total →	19.27		−38.76	323.35	

In this example:

$$t = \tfrac{5}{16}''$$

$$r = 3\tfrac{1}{2}''$$

$$\theta = 72.45^\circ \text{ or } 1.263 \text{ radians}$$

$$A = 2\ (\tfrac{5}{16})(3\tfrac{1}{2})(1.263)$$

$$= 2.76 \text{ in.}^2$$

$$d_t = \frac{(3\tfrac{1}{2})(.9535)}{1.263}$$

$$= 2.64''$$

$$I_g = (\tfrac{5}{16})(3\tfrac{1}{2})\left[1.263 + \tfrac{1}{2}(.575) - \frac{2(.9535)^2}{1.263}\right]$$

$$= 1.46 \text{ in.}^4$$

These values will now be used in finding the properties of the built-up section. To find these properties, select reference axis (x-x) along the underneath surface of the deck plate. This is almost through the center of gravity of the 2 welds, and the resulting distance to the neutral axis (n) will also be the distance between the neutral axis and the center of gravity of welds (d).

$$I_{NA} = I_x + I_g - \frac{M^2}{A}$$

$$= (323.35) - \frac{(-38.76)^2}{(19.27)} \quad \text{(From Table B)}$$

$$= 323.35 - 77.96$$

$$= 245.39 \text{ in.}^4$$

$$n = \frac{M}{A}$$

$$= \frac{(-38.76)}{(19.27)}$$

$$= -2.01'' = d$$

bending or camber $\qquad L = 25' = 300''$

$$\Delta = \frac{.005\ A_w\ d\ L^2}{I}$$

$$= \frac{.005(.130)(2.01)(300)^2}{(245.39)}$$

$$= .48'' \text{ (ends would go up this amount)}$$

This means when the 25′ long unit is upside down for welding, the fixture should be curved sufficiently to pull the central section of the unit down by this amount or about ½".