Welded Plate Girders for Bridges

1. INTRODUCTION

Plate girders are fabricated for requirements which exceed those of a rolled beam, or a rolled beam with added cover plates. The usual welded plate girder is made of two flange plates fillet welded to a single web plate. Box girders are made of two flange plates and two web plates. They have extremely high torsional strength and rigidity.

Plate girders are proportioned by their moments of inertia. See preceding Section 4.2 on Efficient Plate Girders.

AASHO Specifications govern in the Bridge field, with AWS Specifications generally governing welded joint details. This particular section brings together these two Specifications, with interpretation and supplementary recommendations being added for the designer's guidance.

AASHO (1.6.11) limits the minimum ratio of the depth of beams and plate girders to 1/25 of their length. For continuous spans, the span length shall be considered as the distance between dead-load points of contraflexure.

2. PLATE GIRDER WEBS

AASHO Specifications (1.6.75 & 1.6.80) require that the thickness-to-depth ratio of girder webs be not less than the values indicated in Table 1.

The above ratio of web thickness to clear depth is based on predications of the plate buckling theory, the web plate being subjected to shear throughout its depth and to compressive bending stresses over a portion of its depth.

The plate buckling theory assumes the panel portion of the web to be an isolated plate; however, in the plate girder, the web is part of a built-up member. When the critical buckling stress in the web is reached, the girder does not collapse. The flange plates carry all of the bending moment, the buckled web serves as a tension diagonal, and the transverse stiffeners become the vertical compression members. This has the effect of making the girder act as a truss.

Research at Lehigh University tested, among other things, the effect of the web thickness on the ultimate carrying capacity of the girder; see Figure 1. It was found that the ultimate load carrying capacity of the girder, expressed as the ratio of the ultimate load (P_u) to load causing yield stress (P_y) was directly proportional to the restraint offered by the compression flange. The more torsionally flexible flange (wide and thin) resulted in the lower strengths, and the more torsionally rigid flange (tubular) resulted in higher strengths.

Differences in web slenderness ratio produced little effect on the ultimate load carrying capacity of the girder for the same compression flange.

Although the tubular type of compression flange was used to obtain a torsionally rigid flange, it is not recommended for actual bridge practice. However, the concrete floor slab directly on top of the usual compression flange offers a similarly high torsional restraint, as well as good lateral bracing.

Designers in Europe, as well as Canada, are not held to this (Table 1) fixed ratio of web thickness to web depth. One exception made in the United States is the Quinnipiac River Bridge in New Haven, Conn., which used thin longitudinally stiffened webs. Instead of using an arbitrary ratio of web thickness to depth, the design was based on the elastic stability of the web from information by Mosseiff and Lienhard*. The design also considered safety against yielding based on a yielding criterion obtained from the following Huber-Mises formula:

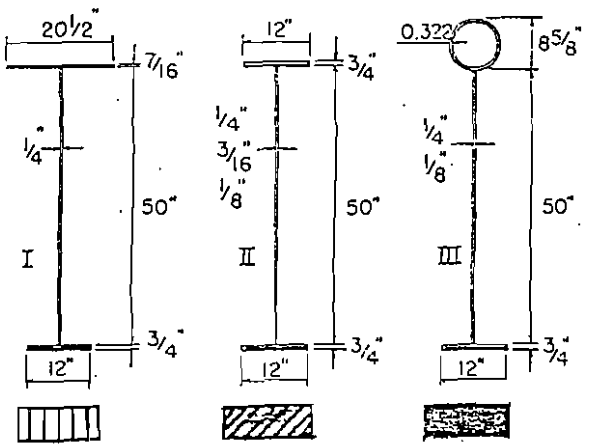
$$\boxed{\sigma_{cr} = \sqrt{\sigma_x^2 - \sigma_x\,\sigma_y + \sigma_y^2 + 3\,\tau_{xy}^2}} \quad \ldots\ldots (1)$$

* "Theory of Elastic Stability Applied to Structural Design" ASCE paper 2120.

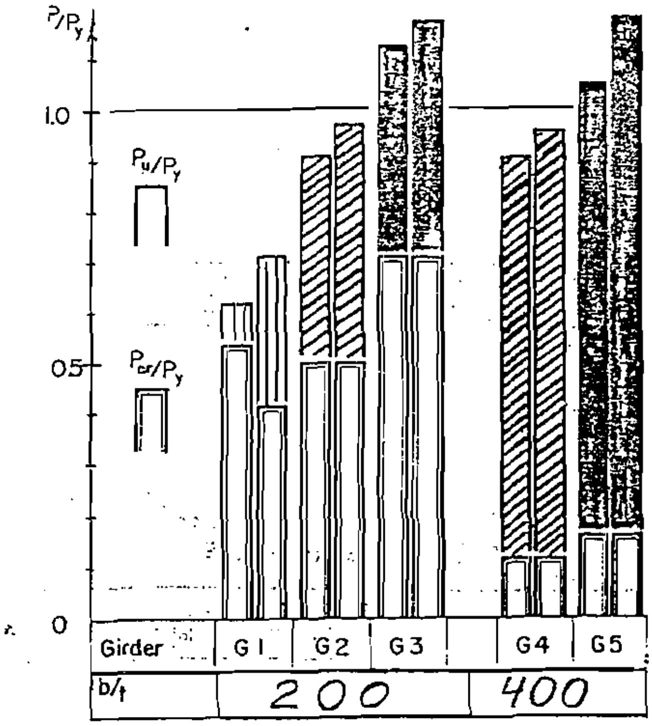
TABLE 1—Minimum Girder Web *Thickness-To-Depth Ratios* *(AASHO 1.6.80, 1.6.75)*

	A-7, A-373 & A-36 Steels	A-441 Steel 46,000 psi yield	A-441 Steel 50,000 psi yield
No stiffeners	1/60	1/52	1/50
Intermediate transverse stiffeners	1/170	1/145	1/140
Longitudinal stiffeners	1/340	1/290	1/280

If the value of σ_{cr} resulting from the above formula is equal to the yield point of the steel in uni-axial tension (what is commonly called the yield strength, σ_y), it is assumed this combination of stresses will just produce yielding in the material. Hence, the use of this formula will give some indication of the factor of safety against yielding.



(a) Cross-sections of test specimens



(b) Comparison: ultimate and critical loads of bending tests

FIG. 1 Effect of web thickness on ultimate carrying capacity of the girder.

3. TRANSVERSE INTERMEDIATE STIFFENERS (AASHO 1.6.80)

Transverse intermediate stiffeners shall preferably be in pairs. They may be either single or double, and be plates or inverted tees. When stiffeners are used on only one side of the web, they shall be welded to the compression flange to give it proper support.

The moment of inertia of the transverse stiffener shall not be less than—

$$\boxed{I = \frac{a_a\ t_w^3\ J}{10.92}} \quad \dots\dots (2)$$

where:

$$\boxed{J = 25\ \frac{d_w^2}{a_r^2} - 20 = 5} \quad \dots\dots (3)$$

I = minimum required moment of inertia of stiffener, in.⁴

a_r = required clear distance between transverse stiffeners, in.

a_a = actual clear distance between transverse stiffeners, in.

d_w = unsupported depth of web plate between flanges, in.

t_w = web thickness, in.

When transverse stiffeners are in pairs, the moment of inertia shall be taken about the centerline of the web plate. When single stiffeners are used, the moment of inertia shall be taken about the face in contact with the web plate.

The width of a plate stiffener shall not be less than 16 times its thickness, and not less than 2″ plus 1/30 of the girder depth.

The distance between transverse stiffeners shall not exceed—

1. 12 feet
2. the clear unsupported depth of the web (d_w)
3. $$\boxed{a_r = \frac{12{,}000}{\sqrt{\tau}}\ t_w} \quad \dots\dots (4)$$

where:

τ = average unit shear stress in the web's cross-section at the point considered, psi

4. LONGITUDINAL STIFFENERS (AASHO 1.6.81)

The longitudinal stiffener shall lie along a line 1/5 d_w

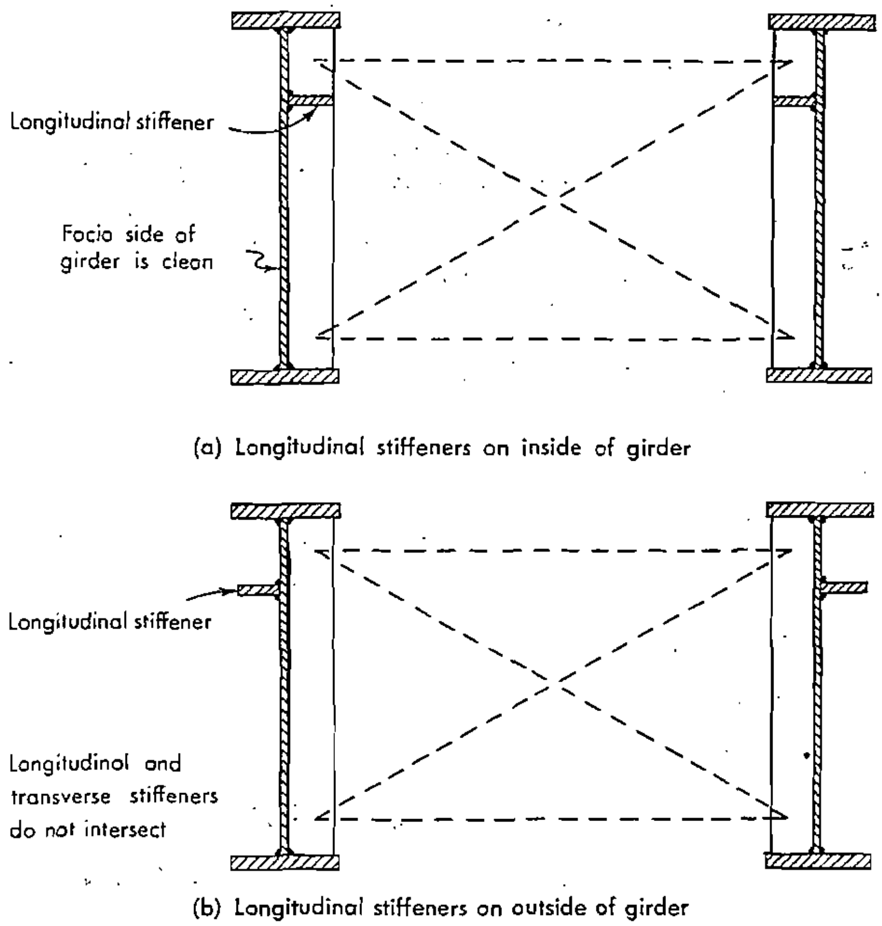


FIG. 2 Placing longitudinal stiffeners on outside of girder and transverse stiffeners inside saves fabricating time.

from the compression flange. Its moment of inertia shall not be less than—

$$\boxed{I = d_w\, t_w^3\ (2.4\ \frac{a_r^2}{d_w^2} - 0.13)} \quad \ldots\ldots\ldots\ldots (5)$$

These stiffeners do not necessarily have to be continuous, but may be cut where they intersect transverse intermediate stiffeners if they lie on the same side of the web.

5. BEARING STIFFENERS

Transverse stiffeners shall be used over the end bearings or along the length of the girder where concentrated loads must be carried, and shall be designed to transmit the reactions to the web. They shall extend as nearly as practicable to the outer edge of the flange, but not to exceed 12 times their thickness. (AASHO 1.6.17)

Some bridges have longitudinal stiffeners on the inside of the girders, others on the outside. If the longitudinal stiffeners are on the inside, along with the transverse stiffeners, it leaves the outside of the girder smooth; Figure 2(a). This, of course, means the longitudinal stiffener must be cut into short lengths and then inserted between the transverse stiffeners. This results in increased welding time and production costs.

Some states have used longitudinal stiffeners on the outside and transverse on the inside; Figure 2(b). This method saves on fabricating time and also allows the use of automatic welding techniques to join the longitudinal stiffeners to the girder web, thereby substantially increasing welding speed.

6. WELDING OF STIFFENERS

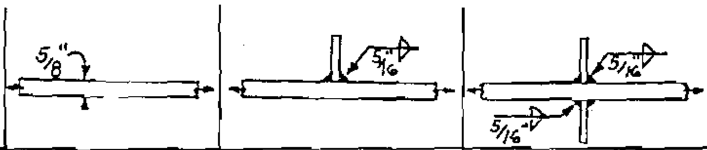
AASHO (2.10.32) will allow the welding of stiffeners or attachments transverse to a tension flange if the bending stress is 75% or less than the allowable.

AWS Bridge (225 c) will allow the welding of stiffeners or attachments transverse to a tension flange if the bending stress in the flange is held to within those of the fatigue formulas (1), (3), or (5) for the welding of attachments by fillet welds; see Section 2.9, Table 1.

Figure 3 illustrates the effect of transverse attachments welded to a plate when tested from tension to an equal compression ($K = -1$).*

* "Fatigue Tests of Welded Joints in Structural Steel Plates", Bull. 327, University of Illinois, 1941.

$K = \frac{min}{max.} = -1$



100,000 CYCLES	25,800 psi	25,400 psi	22,900 psi
2,000,000 CYCLES	22,800 psi	18,900 psi	13,100 psi

FIG. 3 Effect of transverse attachments on fatigue strength of member.

Some engineers have felt this reduction in fatigue strength is due to the transverse fillet welds; however, it is caused by the abrupt change in section due to the attachment. It is believed these plates would have failed at about the same value and location if they had been machined out of solid plate without any welding. This same problem exists in the machining of stepped shafts used in large high-speed turbines and similar equipment.

Figure 4 illustrates the effect of welding transverse stiffeners to tension flanges.* Tests, again at the University of Illinois, were made from tension to zero tension in bending (K = 0) and at 2 million cycles.

Eliminating the weld between the stiffener and the tension flange increased the fatigue strength of the beam. In addition, leaving the weld off the lower quarter portion of the web in the tension region gave a further increase in fatigue strength.

Later tests at the University of Illinois** took into consideration not only the bending stress in the flange, but also the resulting principal tensile stress in the web at critical locations, such as the termination of the connecting fillet weld of the stiffener. See Figure 5.

It was discovered that the fatigue failure in the stiffener area did not necessarily occur at the point of maximum bending stress of the beam. Failure started at the lower termination of the fillet weld connecting the stiffener to the web. When the bottom of the stiffener was also welded to the tension flange, failure started at the toe of the fillet weld connecting the stiffener to the beam flange. After the flange had failed, the crack would progress upward into the web. Here, the failures usually occurred in the maximum moment section of the beam.

This test indicated fairly good correlation when the results were considered in terms of the principal tensile stresses (including the effect of shear) rather than simply the bending stress. The angle of the fatigue failure in the web generally was found to be about

* "Flexural Strength of Steel Beams", Bull. 377, University of Illinois, 1948.

** "Fatigue in Welded Beams and Girders" W. H. Munse & J. E. Stallmeyer, Highway Research Board, Bull. 315, 1962, p 45.

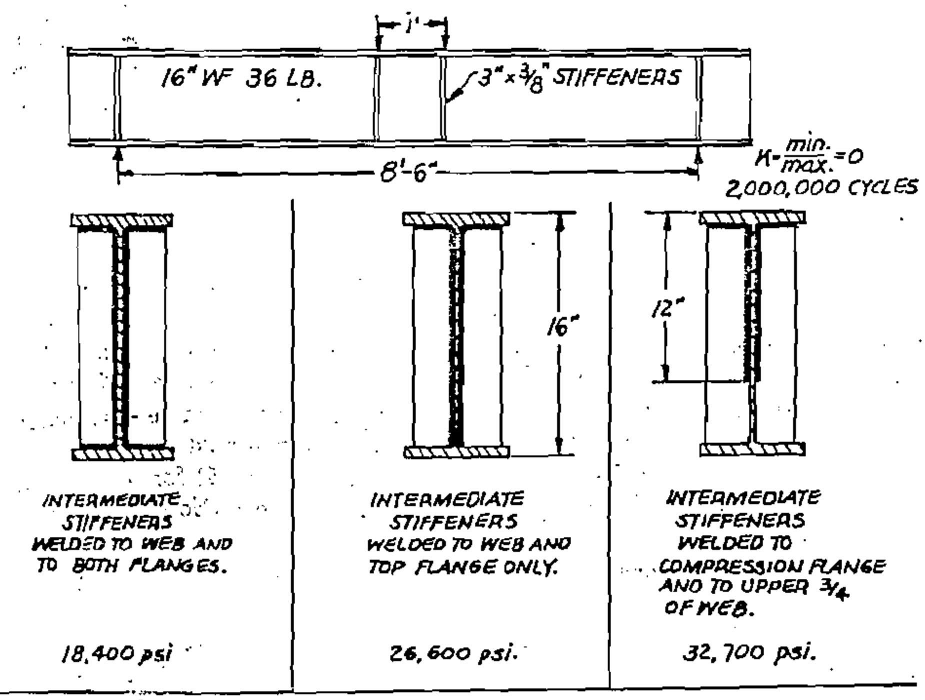
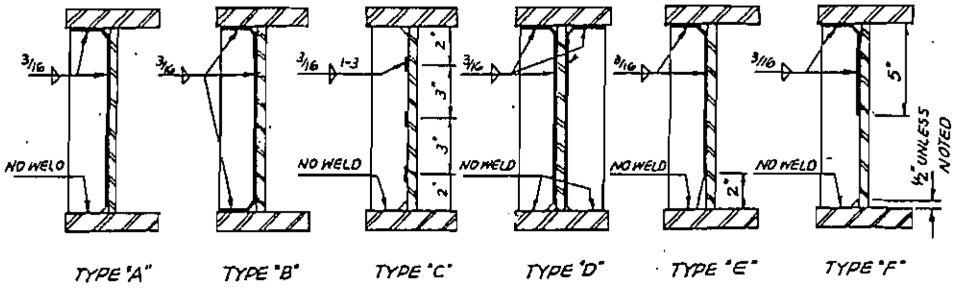
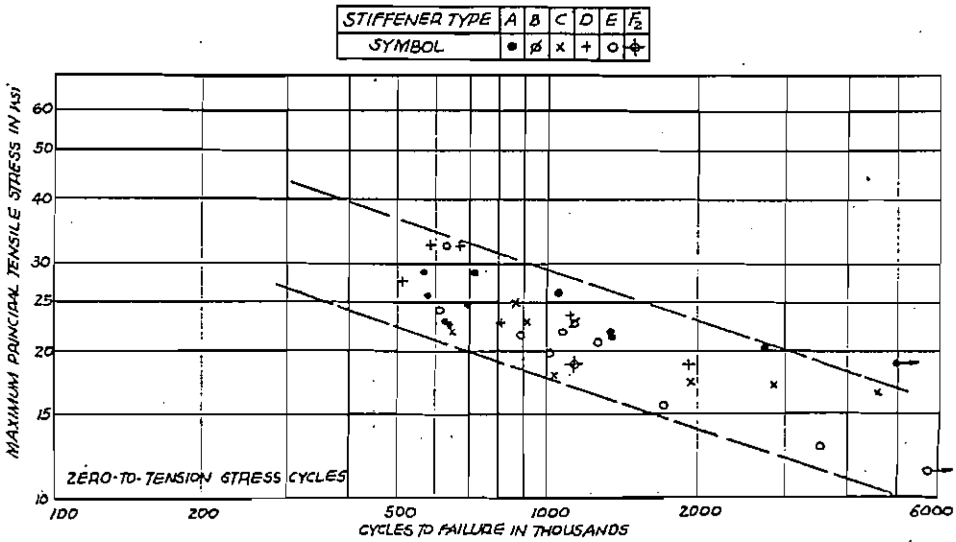


FIG. 4 Effect of welded intermediate stiffener on tension flange.



(a) Details of various stiffener types

FIG. 5 Effect of stiffener type on fatigue strength of member.



(b) Sigma-n diagram for maximum principal tensile stress at failure section.

20% less than the computed angle of the principal stress.

AASHO Specifications (2.10.32) state that transverse intermediate stiffeners shall fit sufficiently tight to exclude water after painting.

Some inspectors interpret a tight fit to be one in which the stiffeners must be forced into position. Many fabricators feel this is an unnecessary deterrent since it takes extra time to force the edges of the flanges apart to allow the stiffeners to be inserted.

There are two general methods of fitting these stiffeners to the plate girder (Fig. 6):

1. Use a stiffener that does not fit too tight. Push it tightly against the tension flange. Weld it to the girder web and to the compression flange.

With this method, the fitting of the stiffener will comply with the above AASHO specs; yet it is not welded to the tension flange, nor is it a problem to insert. An alternate method is to—

2. Use a stiffener which is cut short about 1". Fit it against the compression flange and weld it to the web. If it is a single stiffener, also weld it to the compression flange. It is not welded to the tension flange. Experience indicates the 1" gap at the lower tension

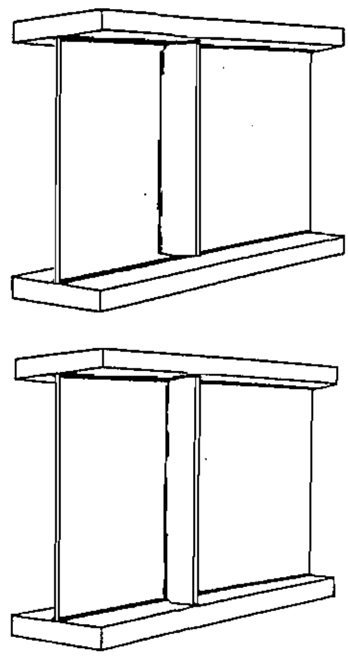


FIG. 6 Fit of stiffeners to girder.

flange will present no maintenance problem. Although this does not comply with the above AASHO requirement, many girders for highway bridges are fitted with stiffeners in this manner.

Plate girder research at Lehigh University* has indicated the stiffener does not have to contact the tension flange to develop the ultimate capacity of the girder. They recommended the stiffeners be cut short as described in the alternate method above (2). The distance between the lower and tension flange and the stiffener is set at 4 times the web thickness; see their recommendations in Figure 7.

There is no clear-cut answer as to whether continuous or intermittent fillet welds should be used to attach the stiffener to the web. The latest research at Illinois on stiffeners indicated that fatigue failures occurred at the terminations of fillet welds, regardless of whether they were continuous or intermittent. Naturally, a continuous weld will have fewer terminations, hence fewer areas for potential fatigue cracks.

Where large, intermittent fillet welds are specified, ⅜" for example, replacement with ¼" continuous fillet welds made by automatic welding equipment achieves a considerable saving in cost. Where small intermittent fillet welds are specified, ¼" possibly, savings from the introduction of continuous welds and automatic equipment become questionable.

With thin, deep web plates, a smaller size weld may tend to reduce distortion. In this case, automatic welding would be of benefit, provided this substitution of continuous welds for intermittent welds does not increase weld length to any major extent.

7. FLANGE-TO-WEB WELDS

These welds hold the flanges to the web of the plate girder. They are located in areas of bending stresses and must transfer longitudinal shear forces between flanges and web. Some restraining action may develop with thick flange plates, but any resulting transverse residual stress should not reduce the weld's load-carrying capacity. This being parallel loading, the actual contour or shape of the fillet weld is not as critical as long as the minimum throat dimension is maintained.

Shop practice today usually calls for submerged-arc automatic welding equipment to make these welds. For the usual thickness of web plate, the two fillet welds penetrate deeply within the web and intersect as in Figure 8(b), giving complete fusion even though simple fillet welds are called for, as in (a). A few

* "Strength of Plate Girders", Bruno Thurliman, AISC Proceedings 1958; "Plate Girder Research", Konrad Basler & Bruno Thurliman, AISC Proceedings, 1959.

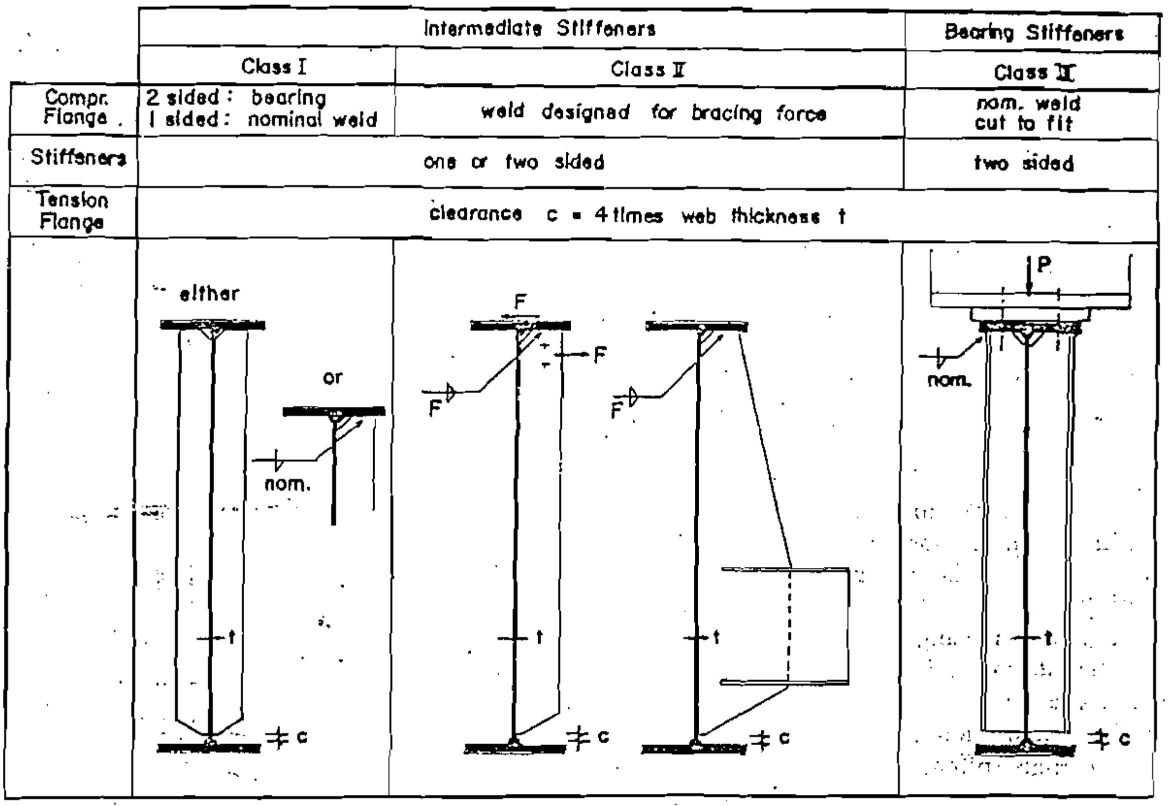


FIG. 7 Summary of design recommendations relative to girder stiffeners.

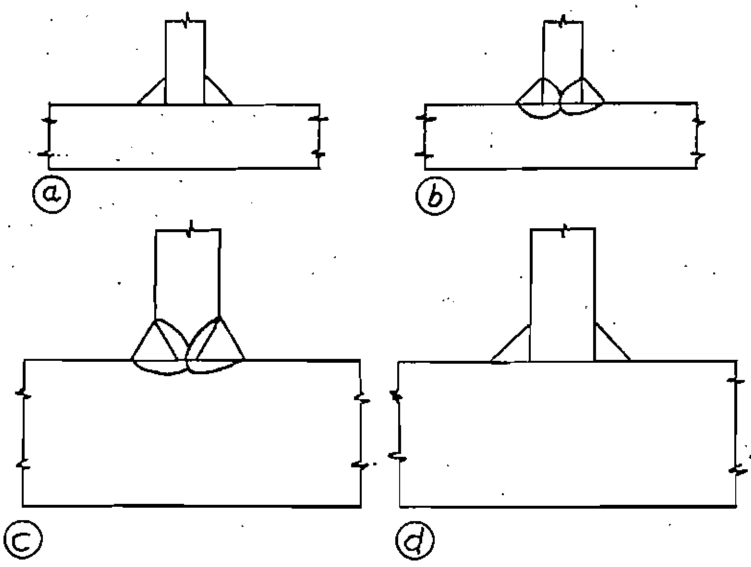


FIG. 8 Flange-to-web welds.

states recognize this penetration and are now detailing this weld with complete fusion. This proves no problem on the normal web thickness. In the future, however, if the same detail is shown on much thicker web plates, the fabricator will have to use a double-bevel edge preparation to obtain the intersection (c), even though detail (d) is sufficient.

It should not be necessary to detail groove welds for this joint from a design standpoint. Selection of a groove T-joint design should be based on a cost comparison with fillet welds. The grooved T-joint requires about ½ the amount of weld metal compared with fillet welds (assuming full-strength welds). However, the grooved joint has the extra cost of preparing the double bevel.

In respect to the physical performance of either the fillet or the grooved T-joint design, tests have been made, by A. Neumann, of these welds under fatigue bending from 0 to tension, K = 0, at 2 million cycles.*

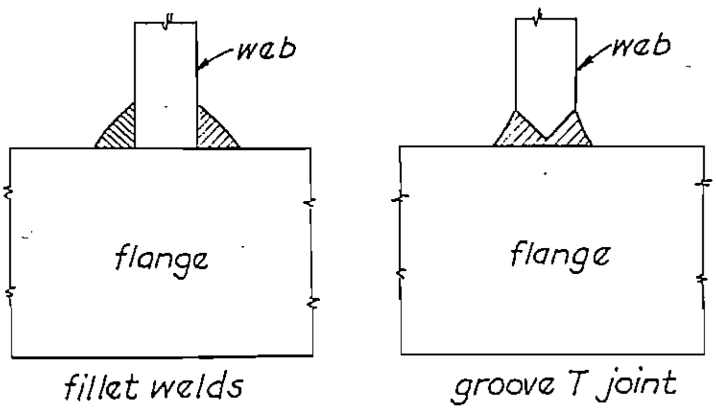


FIG. 9 Both weld types showed same fatigue strength.

No difference was indicated for the fatigue strength of the beam using either joint design, with both types demonstrating a fatigue strength in the beam of 22,000 to 24,000 psi (bending stress); Figure 9.

Fillet Weld Minimum Size

From a design standpoint, these welds may be quite small. Their actual size is usually established by the minimum allowable leg size for the thickness of the flange plate. Table 2 lists the minimum size of fillets for various plate thicknesses as established by AWS Specifications. Leg size increases to take care of the faster cooling rate and greater restraint that exists in thicker plates.

TABLE 2—Minimum Fillet Weld Sizes *For Various Plate Thicknesses (AWS)*

THICKNESS OF THICKER PLATE TO BE JOINED	MINIMUM LEG SIZE OF FILLET WELD*
THRU ½ inch	3/16 in.
Over ½ in. thru ¾ in.	¼ in.
Over ¾ in. thru 1½ in.	5/16 in.
Over 1½ in. thru 2¼ in.	⅜ in.
Over 2¼ in. thru 6 in.	½ in.
Over 6 in.	⅝ in.

* Need not exceed the thickness of the thinner plate

On thicker plates, with multiple pass welds, it is desirable to get as much heat input into the first pass as possible. This means higher welding currents and slower welding speeds. Low-hydrogen electrodes are better for manual welding in this work. The low-hydrogen characteristics of a submerged-arc welding deposit gives this welding method a similar advantage.

* "Discussion at the Symposium on Fatigue of Welded Structures" The British Welding Journal, August, 1960.

TABLE 3—Allowable Shear Forces On Fillet Welds For Various Fatigue Loadings

100,000 CYCLES	600,000 CYCLES	2,000,000 CYCLES
$f = \dfrac{8800\ \omega}{1 - \dfrac{K}{2}}$ lb/in.	$f = \dfrac{7070\ \omega}{1 - \dfrac{K}{2}}$ lb/in.	$f = \dfrac{5090\ \omega}{1 - \dfrac{K}{2}}$ lb/in.

but shall not exceed $f = 8{,}800\ \omega$ (E60 or SAW 1 welds)
$f = 10{,}400\ \omega$ (E70 or SAW 2 welds)

Where:
$K = \dfrac{\text{MINIMUM}}{\text{MAXIMUM}}$ (shear (V) applied to girder)
ω = leg size of fillet

Determination of Combined Stress

The combined stresses in a fillet weld between the girder web and flanges is seldom considered for the following reasons:

1. The maximum bending stress for a simply supported girder does not occur at the same region as the maximum shear force. For a continuous girder, however, the negative moment and shear force are high in the same region near the support, and perhaps the combined forces in this fillet weld should be checked.

2. The maximum bending stress in the outer surface of flange is always designed for something less than the allowable (Bridge code = 18,000 psi). The weld lies inside of the flange and is stressed at a lower value. Ex: If the weld is in an area of 15,000 psi bending stress, this additional normal stress would reduce, theoretically, the allowable shear force for the weld from $f = 8800\ \omega$ to $f = 7070\ \omega$, or about 80% of what it would be if just horizontal shear were considered (E60 or SAW-1 welds).

3. Usually these welds must be larger than design requirements because of the minimum weld size specifications listed above.

Nevertheless, if desirable to determine the combined stresses, it can be theoretically shown that the axial normal stress from the bending, applied to the fillet weld, would increase the maximum shear stress applied to the throat. For a given applied normal stress (σ), the resulting maximum value for the allowable force (f) which may be applied to the fillet weld of a given leg size (ω) under parallel loading is expressed by the formula:—

$$\boxed{f = \omega \sqrt{8800^2 - \frac{\sigma^2}{8}} \quad \text{(E60 or SAW-1 welds)}} \qquad \text{(6a)}$$

$$\boxed{f = \omega \sqrt{10{,}400^2 - \frac{\sigma^2}{8}} \quad \text{(E70 or SAW-2 welds)}} \qquad \text{(6b)}$$

This formulation still permits the maximum shear stress resulting from the combined shear stresses to be held within the allowable of $\tau = 12{,}400$ psi (E60 or SAW-1 welds) or 14,700 psi (E70 or SAW-2 welds).

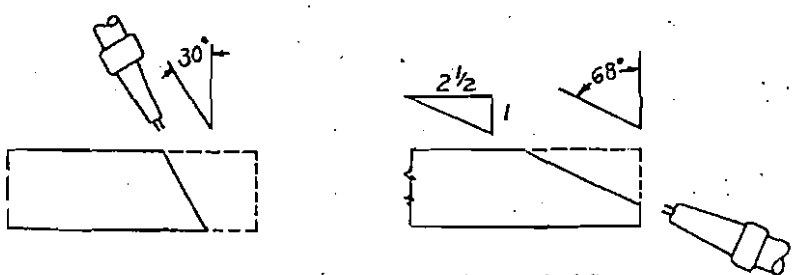
Allowable Fatigue Strength

Table 3 contains the formulas for establishing the allowable shear force that may be applied to fillet welds under various conditions of fatigue loading.

8. FLANGE BUTT JOINTS

In nearly all welded plate girders, the flange is a single plate. These plates are stepped down as less area is required. A smooth transition is made between the two, by reducing either the thickness or width of the larger flange to correspond to that of the smaller.

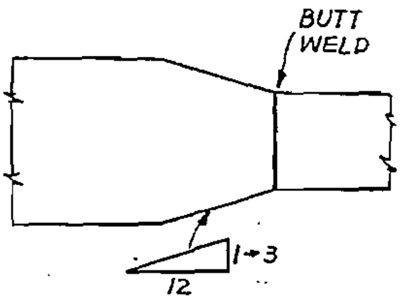
When this transition is made in thickness, the end of the larger flange is beveled by a flame-cutting torch. There is a practical limit to the angle of bevel, but this slope, according to AWS Bridge Specifications, should not be greater than 1″ in 2½″ (an angle of 23°). On the Calcasieu River bridge, this slope was decreased to about 1″ in 6″ (an angle of about 9½°). Transitions also can be made by varying the surface contour of



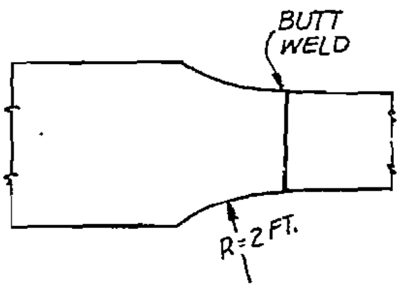
(a) Beveling end of flange plate for groove butt weld

(b) Beveling end of flange plate for transition in thickness.

FIG. 10 Plate bevels made by flame cutting.



(a) Straight-line transition in width



(b) Curved transition in width

FIG. 11 Method of transition in width affects weld's allowable fatigue values.

the groove welds.

The usual method of flame cutting a bevel in the preparation of a welded joint is to cut down through the surface of the plate at the proper angle. Because of the wide angle needed for this transition in thickness, it is often better to flame-cut back from the edge of the plate after the flange plate has been cut to length. See Figure 10.

When the transition is made in width, the end of the wider flange is cut back at an angle, again with the flame-cutting torch. There is no problem in cutting in this manner, and any slope may be used; many times 1 in 12, but usually a maximum slope of 1 in 4. Often this taper may extend back for several feet.

Generally, it is felt that the straight-line transition in width is sufficient, and in the case of fatigue loading the allowable fatigue values for butt groove welds in tension or compression are used. See Figure 11. If a curve tangent to the edge of the narrow flange at the point of termination is used, it may be assumed the flanges have equal widths. Thus, for equal plate thicknesses and with the weld reinforcement removed, the butt groove weld may be assigned the same allowable stress as the flange plate, under any condition of fatigue loading.

Studies at the University of Illinois have indicated a slight advantage in making a transition in width rather than in thickness. This advantage undoubtedly would be greater if the transition in width were made more gradual; however, both methods are sound and acceptable. Fatigue values for these transitions are found in Figure 12.

TABLE 4—Allowable Fatigue Strengths Of Groove Welds in Butt Joints

	100,000 CYCLES	600,000 CYCLES	2,000,000 CYCLES
BUTT WELD IN TENSION (not to exceed 18,000 psi)	$\sigma = \frac{18{,}000 \text{ psi}}{1 - \frac{K}{2}}$	$\sigma = \frac{17{,}000 \text{ psi}}{1 - .7\,K}$	$\sigma = \frac{16{,}000 \text{ psi}}{1 - .8\,K}$
BUTT WELD IN COMPRESSION (not to exceed p)	$\sigma = \frac{18{,}000 \text{ psi}}{1 - \frac{K}{2}}$	$\sigma = \frac{18{,}000 \text{ psi}}{1 - 0.8\,K}$	$\sigma = \frac{18{,}000 \text{ psi}}{1 - K}$

Where:

(p) is the allowable compressive stress for the member involved.

$K = \frac{\text{MINIMUM}}{\text{MAXIMUM}}$ (bending stress or bending moment)

Allowable Fatigue Strengths

Groove welds in butt joints of equal plate thickness, if the reinforcement is finished smooth with the surface, may be allowed the same fatigue strength under any type of fatigue loading as the base metal. For plates of unequal thickness where the transition slope is not greater than 1 in 2½, the formulas found in Table 4 may be used.

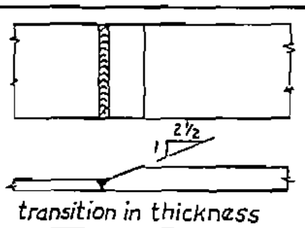
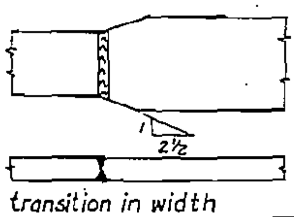
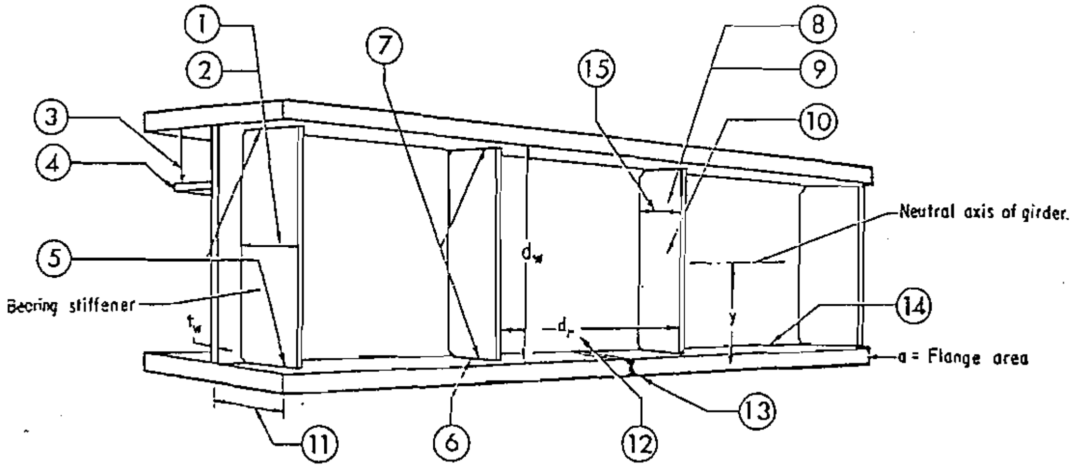
Type of transition of flange section	K=0 N=100,000 ~	K=0 N=2,000,000~
 transition in thickness	34,600 psi	18,500 psi
 transition in width	34,900	19,500

FIG. 12 Making a transition in flange width rather than thickness has a slight advantage in fatigue strength.

FIG. 13—Summary of Bridge Plate-Girder Specifications AWS & AASHO



9. SUMMARY OF BRIDGE SPECIFICATIONS

In order to aid the bridge engineer in designing a welded plate girder, the pertinent AWS and AASHO Specifications have been brought together into a single drawing, Figure 13, and related text, below. The corresponding numbers are included so the engineer may refer back to the original specifications.

This summary can also serve as a checkoff list, so that nothing will be inadvertently omitted.

The following requirements apply:

1. Extend bearing stiffener as near as practical to outer edge of flange. Proportion for bearing. Welds to web must transmit end reaction. (1.6.79)

2. Width of bearing stiffener must not exceed 12 times stiffener thickness (1.6.17).

3. Space (horizontal) longitudinal stiffener ⅕ d_w from compression flange (1.6.81).

4. Dimension longitudinal stiffener for required moment of inertia, using—

$$I = d_w\, t_w^3 \left(2.4\, \frac{d_r^2}{d_w^2} - 0.13\right)$$

about edge of stiffener (1.6.81).

5. Mill or grind bearing stiffener ends for even bearing to flange. Stiffener may be welded without milling to compression flange, or to tension flange if less than 75% tensile strength (2.10.32).

6. Do not weld transverse intermediate stiffener to tension flange if stressed over 75% (2.10.32) or unless stress is within that of fatigue formulas 1, 3 or 5 of Art. 228 (225c).

7. Fit intermediate stiffener tight to flanges to exclude water after painting (2.10.32).

8. Consider placing intermediate stiffeners at points of concentrated load to transmit reactions to the web (1.6.80).

9. Use transverse intermediate stiffener preferably in pairs on opposite sides of web. If only one side of web, weld ends to compression flange and intermittent weld to web (1.6.80, 225c).

10. The minimum moment of inertia of transverse intermediate stiffener shall be (1.6.80)—

$$I = \frac{d_a\, t_w^3\, J}{10.92}$$

where:

$$J = 25\, \frac{d_w^2}{d_r} - 20 \geqq 5$$

d_a = actual distance between stiffeners, in.

d_r = required distance between stiffeners, in.

d_w = web depth, in.

t_w = web thickness, in.

τ = average shear stress in web

11. Girder flange shall not extend beyond 12 times its thickness (1.6.17).

12. Distance between stiffeners must not exceed 12′, d_w, or $\dfrac{12{,}000\ t_w}{\sqrt{\tau}}$ (1.6.80)

13. All shop groove butt welds in flange and web plates shall be made before final fitting and welding into girder (404f).

14. Web-to-flange fillet weld leg size $= \dfrac{V\ a\ y}{17{,}600\ I}$.

15. Width of transverse intermediate stiffeners must not exceed 16 times stiffener thickness, or 2″ plus ¹⁄₃₀ of girder depth.

Also, deflection due to live load plus impact shall not exceed 1/800 of the span; for cantilever arms, 1/300 of the span (1.6.10).

MINIMUM WEB THICKNESS (t_w)

	A-7, A-373 A-36	A-441 Low Alloy 46,000 YP	50,000 YP
If no stiffeners (1.6.80)	$t_w = \frac{1}{60} d_w$	$t_w = \frac{1}{52} d_w$	$t_w = \frac{1}{50} d_w$
If trans. int. stiffeners (1.6.75)	$t_w = \frac{1}{170} d_w$	$t_w = \frac{1}{145} d_w$	$t_w = \frac{1}{140} d_w$
If long. and trans. stiffeners	$t_w = \frac{1}{340} d_w$	$t_w = \frac{1}{290} d_w$	$t_w = \frac{1}{280} d_w$

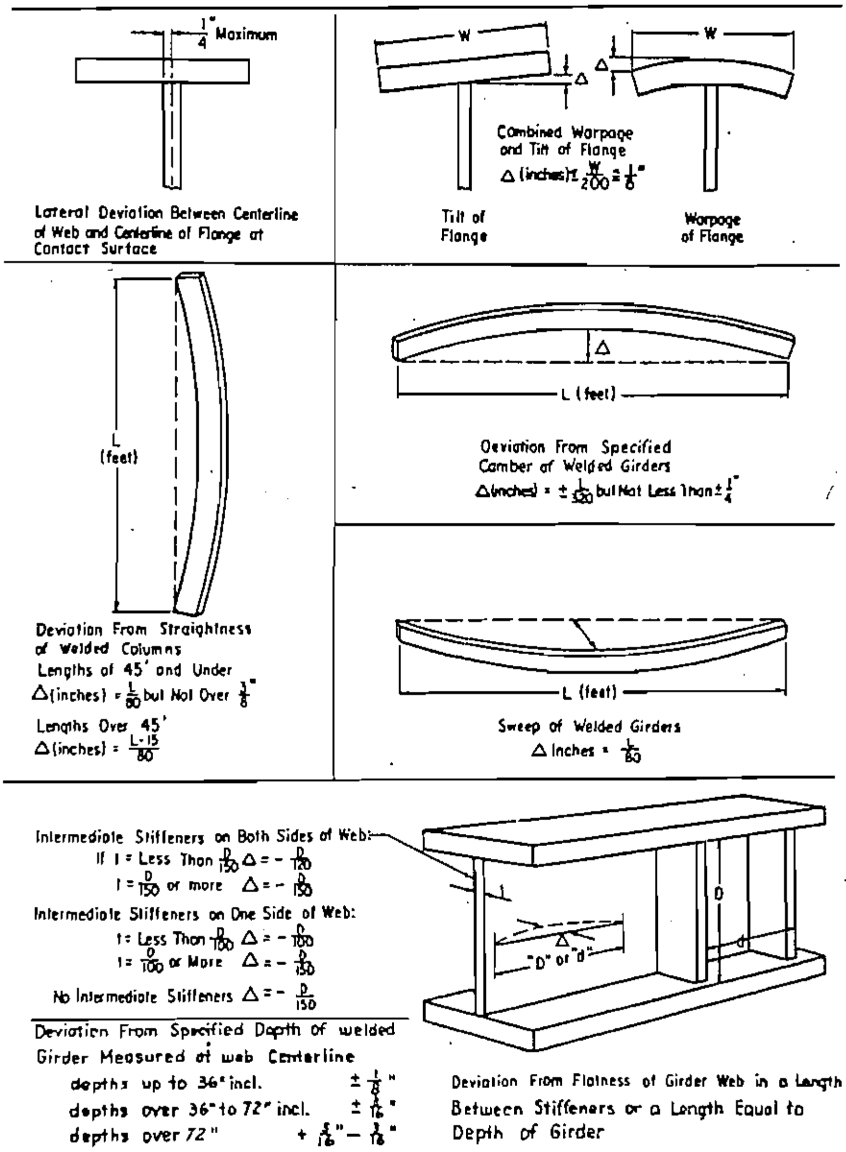
Also, ratio of depth to length of span shall preferably not be less than $\frac{1}{25}$; for lower depth the section shall be increased so that the maximum deflection will not be greater than if this ratio had not been exceeded (1.6.11).

Also, web thickness shall meet requirements given in the above table for the more common steels.

10. DIMENSIONAL TOLERANCES

The dimensional tolerances in Figure 14 have been set up for welded plate girders by the AWS Bridge Specifications.

FIG. 14—Maximum Dimensional Tolerances AWS 407



11. DIAPHRAGMS

Figure 15 illustrates several types of diaphragms used, and represent the extremes in designs and fabrication. Diaphragm (a), although so simple in design that no shop welding is required, must be fitted and welded in the field. Diaphragm (b), although much more complicated, may be mass-produced in the shop: The angles are sheared to length, and the plates are sheared and punched. These are placed into a simple fixture and welded together at low cost. The field erection is simpler, since the diaphragms are put into position, held by an erection bolt, and then welded into place.

12. COVER PLATES

Using A-441 steel (previously A-242), it may be advantageous in some cases to use two plates, a flange plate and a cover plate, to make up the flange. This will permit use of thinner plates and take advantage of the higher allowable stresses. This steel has the following allowable tension in members subject to bending:

THICKNESS	ALLOWABLE
¾" and under	27,000 psi
over ¾" to 1½"	24,000 psi
over 1½" to 4"	22,000 psi

Many methods have been suggested for termination of cover plates. The existence of at least four conditions which affect this makes it impossible to recommend one specific cover plate end which will best meet all conditions.

First, the tensile forces, assumed to be uniformly distributed across the width of the cover plate, should be transferred simply and directly into the corresponding flange of the rolled beam without causing any stress concentration in the beam flange. In general, a large transverse fillet weld across the end of the cover plate does this in the simplest manner.

Second, there must be a very gradual change in the beam section at the end of the cover plate, in order to develop a similar gradual change in bending stress of the beam. Any abrupt change in beam section

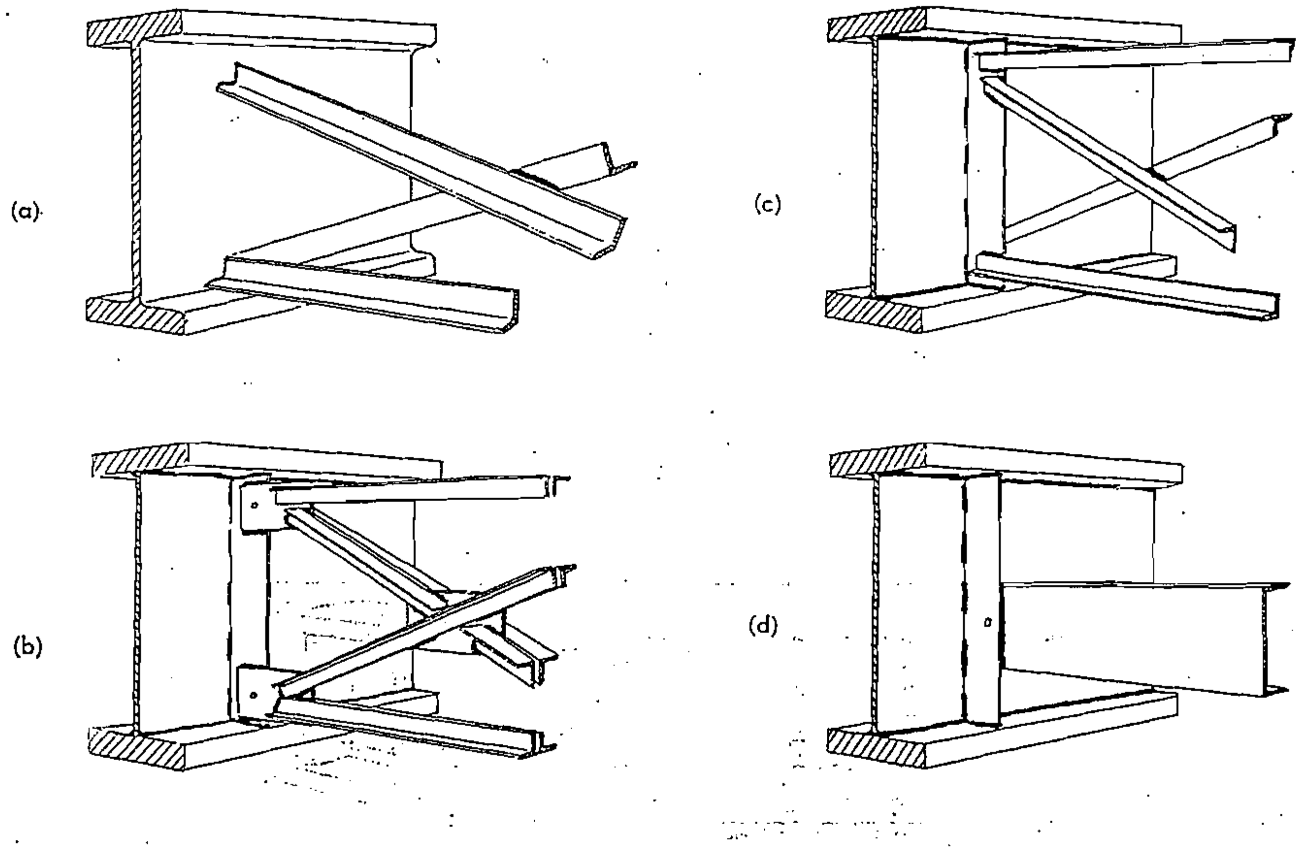


FIG. 15 Diaphragms used in modern bridges: (a) angles cut to length and dropped into place; (b) Shop welded diaphragm, field welded to girder stiffener; (c) angles attached to stiffeners; and (d) channel welded to web and stiffeners.

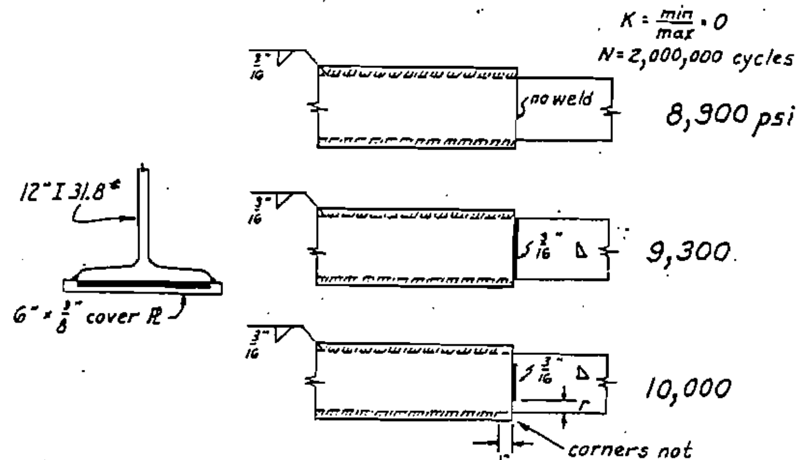


FIG. 16 Cover plates extending beyond width of beam flange.

will reduce the beam's fatigue strength. This would tend to favor a gradual tapered width at the end of the cover plate.

Third, some caution should be exercised relative to terminating the cover plate in the narrow zone of the flange that is in direct line of the beam web. This is a rigid portion with little chance for localized yielding to prevent the build-up of possible high stress concentration.

Fourth, the selected joint should be economically practical to make and answer functional requirements. For example:

1. Continuous welds may be needed to provide a positive seal and prevent moisture from entering underneath the plate and causing connection deterioration.

2. Minimum appearance standards may eliminate some joint designs.

Early fatigue testing at the University of Illinois* on rolled beams with cover plates indicated that:

1. In general, continuous fillet welds were better than intermittent fillet welds for joining cover plates to the beam flange.

2. On cover plates extending beyond the width of the beam flange and connected with longitudinal 3/16" continuous fillet welds, adding a 3/16" fillet weld across the end of the cover plate produced a slight increase in fatigue strength (from 8900 psi to 9300 psi at 2 million cycles). Omitting the welds for a distance at each corner of the cover plate increased this value up to 11,000 psi; see Figure 16.

The intersection of the longitudinal and transverse fillet welds could present a point of weakness if not properly made. This "cross-over" usually results in a very shallow concave weld. By eliminating this weld for 1" back from each corner, the fatigue strength is increased. This does not apply if the cover plate lies within the beam flange, since the weld does not have to "crossover."

* Bull. No. 377, Jan. 1948.

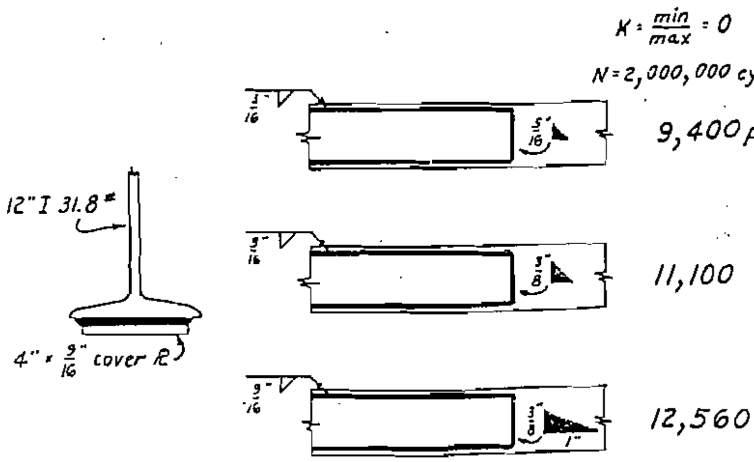


FIG. 17 Cover plates lying within width of beam flange.

no tests made with the transverse fillet weld left off

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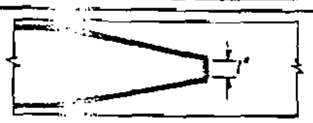
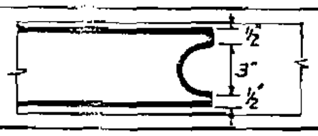
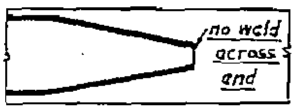
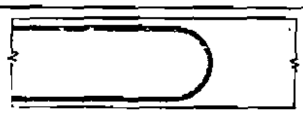
Types of termination of cover plate		K=0 N=100,000~	K=0 N=2,000,000~
(a)		26,500 psi	11,300 psi
(b)		33,000	11,500
(c)		30,700	14,500
(d)		34,700	12,500
(e)		36,500	13,700
(f)		29,000	11,700

FIG. 18 Effect of cover plate termination on fatigue strength. Calculations based on 4" x ½" cover plate and ¼" fillet weld.

3. For cover plates lying within the width of the beam flange, increased fillet weld size across the end of the cover plate produced a gradual increase in fatigue strength. A 5/16" fillet weld had a strength of 9400 psi at 2 million cycles, a ⅜" fillet weld 11,000 psi, and a ⅜" × 1" fillet weld up to 12,600 psi. This particular size of cover plate was not tested with the transverse fillet weld omitted; see Figure 17.

The latest work reported at the University of Florida on steady loading of 18" WF 70# beams with 5" × ⅝" cover plates showed that the beam flange within the cover-plated region was stressed lower when a ⅝" fillet weld was placed across the end of the cover plate as compared to that with no transverse weld. The transverse weld also produced a more uniform distribution of stress across the cover plate as well as the beam flange, and allowed the plate to pick up its share of the beam force in a shorter distance. However, all of these factors occur within the cover-plated region of greater section modulus and lower bending stress, so this is not very serious.

What is more important is the effect the transverse weld and shape of the cover plate's end has on the stress in the beam flange adjacent to where the cover plate is attached. This is the region of lower section modulus and higher bending stress and is much more critical than any region within the cover plate.

The drawing, Figure 18, illustrates variations of cover plate terminations.* The data summarizes recent tests on the fatigue strength of beams with partial cover plates, conducted at the University of Illinois. Although the common method of terminating the cover plate directly across the flange with a transverse fillet weld is satisfactory and acceptable by the AWS Bridge Specifications, this data would seem to indicate that tapering the end of the cover plate and eliminating transverse welds across the end slightly increases the fatigue strength.

* "Fatigue in Welded Beams and Girders", W. H. Munse and J. E. Stallmeyer, Highway Research Board, Bull. 315, 1962, p. 45.

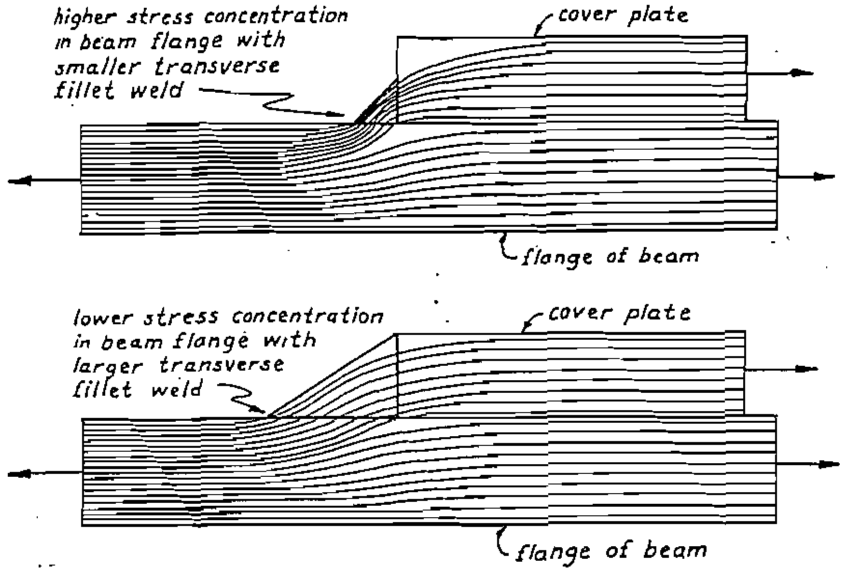


FIG. 19 Effect of transverse fillet weld size on fatigue strength.

It should be noted that a small ¼" fillet weld was used across the end of the ½" thick cover plate. The results might have been different if a larger transverse weld had been used. Most states require continuous welds on cover plates and across their ends, thereby limiting the selection to termination types *a* or *b*. Since the data indicates that tapering has little effect, final selection between *a* or *b* would have to be made on the basis of some other factor such as appearance, or lower dead weight.

In summary, it would appear that the short section of the transverse weld across the end of the cover plate directly over the web of the beam (1) is restrained and (2) when tested under severe fatigue loading may reduce the fatigue strength of the connection unless it is made large. A large transverse fillet weld, especially in this central section, would more uniformly transfer this force through the surface of the beam flange into the end of the cover plate. See Figure 19.

Summary of Cover Plate Specifications (AWS Art. 225)

The AWS Bridge Specifications limit the thickness of cover plates to 1½ times the thickness of the flange to which it is attached (225 e 1).

For partial-length cover plates, their end shall extend beyond the "theoretical end" (theoretical cut-off point) which is determined by the allowable stresses from fatigue formulas (1), (3), or (5) of Section 2.9, Table 1.

The ends of the cover plate shall extend beyond this "theoretical end" a sufficient distance to allow "terminal development" (transfer of cover plate bending force into the beam flange) by either of the following two methods:

A. With *square ends* and a continuous transverse fillet weld across the end and along both edges of the cover plate, the minimum terminal development length measured from the actual end of the cover plate to the theoretical end or cut-off point shall be 1½ times the width of the cover plate.

B. With *tapered ends* having no transverse weld across the end but welds along both tapered edges, tapered beyond the terminal end to a width not greater than ⅓ the width, but not less than 3", the terminal development length shall be 2 times the width of the cover plate.

Normally the inner end of the terminal development length will lie at the theoretical cut-off point; see Figure 20, (A) and (B). However, the cover plate may be extended farther so that the distance between the actual and the theoretical cut-off point exceeds the required terminal development length. In this case only the required terminal development length shown in (A) and (B) shall be used for the length of connecting weld when determining weld size, rather than the actual length between the actual and theoretical cut-off point; see (A′) and (B′).

Fillet welds between terminal developments along the cover plated length, shall be continuous and be designed to transfer the horizontal shear forces:

$$\boxed{f = \frac{V\,a\,y}{2\,I}} \qquad (7)$$

(for each weld, there are 2 welds along the edge of the cover plate)

Fillet welds within the terminal development zone (between the inner end of the terminal development and the actual end of the cover plate) shall be continuous and be designed to transfer the cover plate portion of the bending force in the beam at the inner

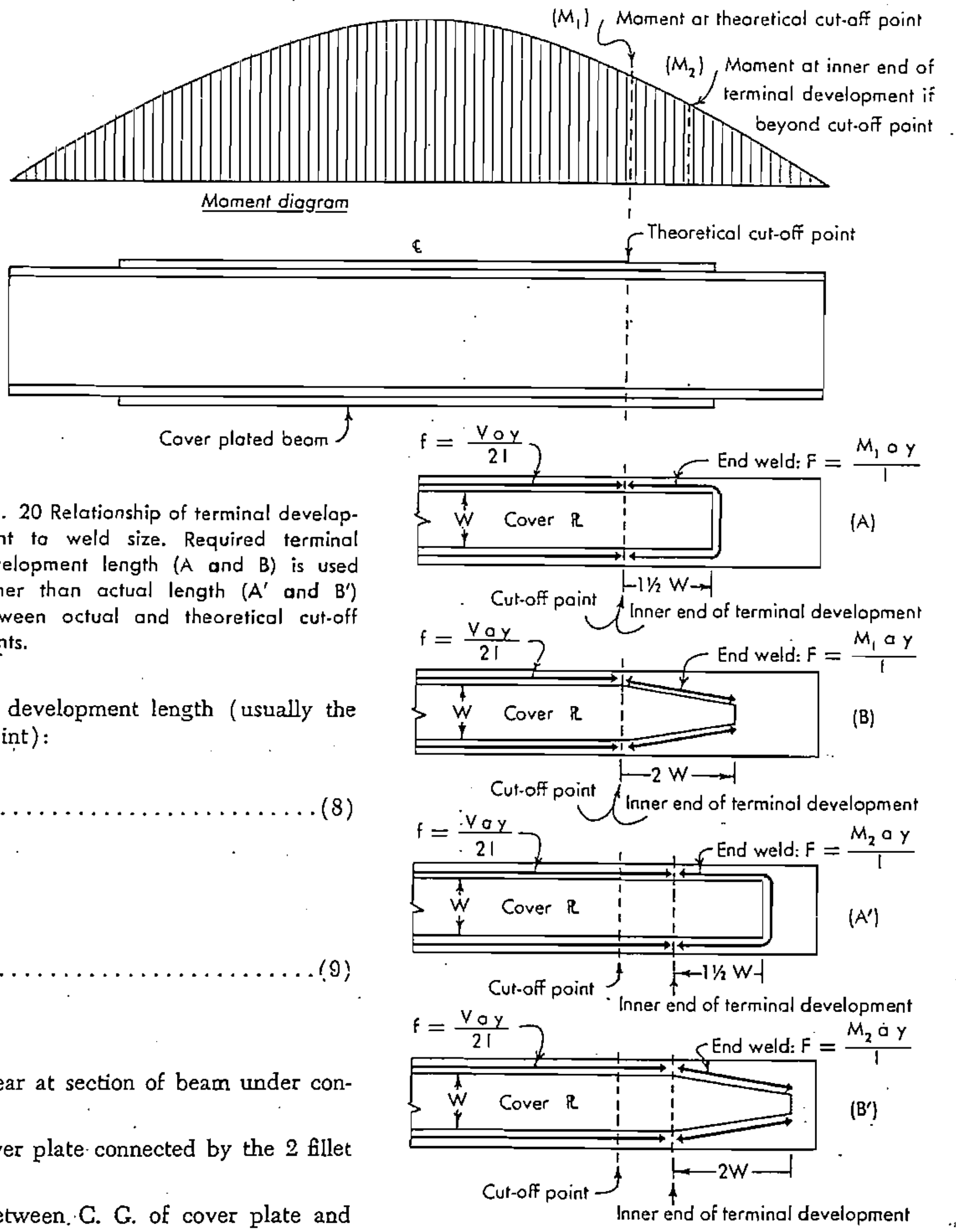


FIG. 20 Relationship of terminal development to weld size. Required terminal development length (A and B) is used rather than actual length (A′ and B′) between actual and theoretical cut-off points.

end of the terminal development length (usually the theoretical cut-off point):

$$\boxed{F = \sigma A_{\text{℞}}} \qquad \text{(8)}$$

$$\sigma = \frac{M\ y}{I}$$

$$\therefore \boxed{F = \frac{M\ a\ y}{I}} \qquad \text{(9)}$$

where:

V = vertical shear at section of beam under consideration

a = area of cover plate connected by the 2 fillet welds

y = distance between C. G. of cover plate and the N.A. of the total section

I = moment of inertia of the total section

M_1 = moment applied to beam at the section of the theoretical cut-off point

M_2 = moment applied to beam at the section of the inner end of the terminal development

The allowable to be used for these fillet welds would come from formulas (10), (14), or (18) of Table 1, Section 2.9, and shall conform to the minimum fillet weld size of Table 2.

AASHO (1.6.74) specifies that the length of any cover plate added to a rolled beam shall not be less than—

$$(2d + 3) \text{ feet}$$

where

d = depth of beam (feet)