



# DETERMINING THE NEED FOR LATERAL WIND BRACING IN PLATE GIRDER BRIDGES

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A Supplement to Chapter 5, Volume II,  
*USS Highway Structures Design Handbook*

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## COMPOSITE MEDIUM SPAN— WELDED PLATE GIRDER— LOAD FACTOR DESIGN

Until 1979, all plate-girder bridges, that have spans of 125 ft or longer, were required (per Article 1.7.17 of the *AASHTO Standard Specifications for Highway Bridges*) to place lateral wind bracing at the bottom flange level. This was followed by a less arbitrary stipulation in the 1979 *AASHTO Interim Specification*, that called for a more "rational" approach in determining the need for bottom lateral bracing. At present, the 1982 *Interim Specification* offers a sophisticated method for making this determination; the method is based upon rigorous computer solutions for a large number of hypothetical bridges, performed at the University of Maryland.<sup>1</sup>

In each case, the hypothetical bridges were examined both with and without bottom lateral wind bracing. The resulting maximum bottom flange stresses were determined, and these stresses were then compared with stresses developed in a continuous isolated bottom flange on fixed supports that was subjected to similar wind loading. As a result of these tests, the 1982 *AASHTO Interim Specification* now states:

"Bottom lateral wind bracing may or may not be provided at the discretion of the Engineer providing the stresses in the bottom flange due to wind loading are accounted for as specified in Article 1.7.17(B)."

Article 1.7.17(B) states:

"(B) Stresses Due to Wind Loading When Top Flanges are Continuously Supported.

- (1) Flanges. The maximum induced stresses ( $F$ ) in the bottom flange of each girder in the system can be computed from the following:

$$F = RF_{cb}$$

where:

$$R = |0.2272L - 11| S_d^{-2/3}$$

when no bottom lateral bracing is provided

$$R = |0.059L - 0.64| S_d^{-1/2}$$

when bottom lateral bracing is provided

$$F_{cb} = \frac{72M_{cb}}{t_f b_f^2} \text{ (psi)}$$

$$M_{cb} = .08WS_d^2 \text{ (ft-lbs)}$$

$W$  = Wind loading along the exterior flange (lbs/ft)

$S_d$  = Diaphragm Spacing (ft)

$L$  = Span Length (ft)

$t_f$  = Thickness of Flange (inches)

$b_f$  = Width of Flange (inches)"

The latest (1982) provisions furnish a means of eliminating bottom lateral bracing in most plate-girder bridges; a factor that should result in substantial savings. Aside from its added weight, lateral bracing is costly because of two other factors: fabrication and erection. There is yet another positive aspect to this procedure: the girders' fatigue strength will be improved since the elimination of lateral bracing obviates the need for connection attachments.

Illustrative application of the provisions of Article 1.7.17 of the 1982 *Interim Specifications* is given in the calculations of the example that follows. In this case,

<sup>1</sup>Refer to Commentaries—1982 Interim Specifications, pp. 2-9.

the structure investigated is the same as the constant depth design given in Chapter 5, Vol. II of the *USS Highway Structures Design Handbook*. As originally presented, this bridge design met the requirements of AASHTO Specifications prior to 1979, hence lateral bracing was provided. Re-calculation in accordance with the 1982 provisions of Article 1.7.17 will show that the bracing is unnecessary and may be eliminated.

The wind load is defined in AASHTO Article 1.2.14 as acting on the lower half of the exposed profile of the bridge. Stresses caused by such wind loading are considered only in exterior girders. The sum of flange stresses caused by Group II Loading (wind and longitudinal girder dead load bending) and Group III Loading (wind plus dead and live load) are checked against tension and compression limit stresses. The Load Factor Design method is used. For tension, the limit stress is  $F_Y$ . For compression, the limit stress is the lower of the stresses for yielding or buckling. Buckling limit stress for a particular flange geometry is easily obtained by rewriting the existing AASHTO equations for flange local buckling—using the same concept of an effective stress that is used in the development of the effective plastic moment.<sup>2</sup> For Load Factor Design, the equation in Article 1.7.59(B)(1)(a) can be rewritten as  $F_{YE} = (2,200 t/b')^2 \leq F_Y$  where  $F_{YE}$  is the effective compressive limit stress. If lateral torsional buckling stress is *below*  $F_{YE}$ , it should be used in its place.

If the actual computed stress is *above* the limit stress, the designer should consider the following corrective measures in the order given:

- reduce the diaphragm or cross-frame spacing in the overstressed region,
- increase the flange width in the over-stressed region,
- add lateral bracing in the exterior bays of the overstressed span.

## CALCULATIONS

The design from Chapter 5 (Fig. 1) is developed by using the Load Factor method. The lateral wind stress is combined with the vertical dead and live load stresses and factored as specified for Group II and Group III loading, Table 1.2.22:

Group II:  $1.3 (F_{DL} + F_W)$

Group III:  $1.3 (F_{DL} + F_{L+I} + 0.3F_W)^*$

The limit stress for tension flanges is  $F_Y$ . The limit stress for compression flanges is the smallest of:

- $F_Y$  = yielding limit stress, ksi
- $F_{YE} = \left( 2200 \frac{t_f}{b'_f} \right)^2$  = local buckling limit stress, psi
- $F_{YL} = F_Y \left[ 1 - \frac{3F_Y}{4\pi^2 E} \left( \frac{12S_d}{b'_f} \right)^2 \right]$ , ksi  
= lateral torsional buckling limit stress

Terminology for these equations and the relevant dimensions for the aforementioned structure are given below:

- $E$  = modulus of elasticity of steel, ksi  
 $L$  = span length, ft  
 $S_d$  = crossframe spacing, ft  
 $W$  = wind load on bottom flange, kips/ft  
 $b'_f$  = projecting width of bottom flange,  $\frac{b_f}{2}$ , inches  
 $F_{DL}$  = stress produced in bottom flange by vertical dead load, ksi  
 $F_{L+I}$  = live plus impact load stress in bottom flange, ksi  
 $F_Y$  = yield stress of bottom flange, ksi

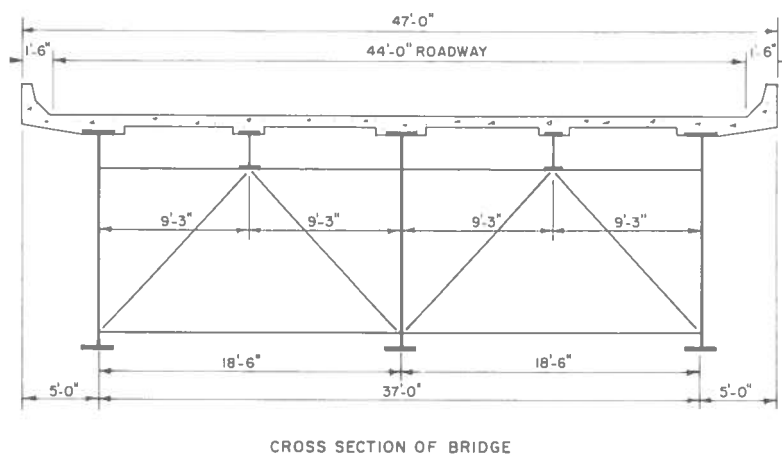


Figure 1.

<sup>2</sup>"Autostress Design of Steel Bridges," G. Haaijer, P.S. Carskaddan, and M.A. Grubb, ASCE Reprint 80-519, October 1980.

\* WL (wind on the live load) in Group III loading is assumed to be zero for the bottom flange because wind on the live load is transferred to and carried directly by the deck slab.

Lateral torsional buckling limit stress, FYL, is applicable only if  $S_d > \frac{20,000A_f}{FY_d}$ , where  $A_f$  is the compression flange area, and  $d$  is the overall depth of the girder. For the structure in the given example, this limit can be expressed as:

$$S_d > \frac{20,000 b_f t_f}{12FY (156 + t_f + t_{tf})}$$

Also, if the ratio of moments at the end of the unbraced length ( $S_d$ ) is less than 0.7, FYL may be increased up to 20 percent, but not to exceed FY.

Analysis is now made on the assumption that the bridge has no lateral bracing. A moving 50 lb per sq ft wind load is uniformly distributed along the fascia.

Referring to the elevation view of the exterior girder (Fig. 2), and the cross-section of the exterior girder (Fig. 3), the first section to investigate is the lower flange transition 34 ft from Support 1, the end bearing.

#### Section 34 ft Rt. of Support 1

Limit stress for tension: FY = 50 ksi

$$\begin{aligned} L &= 273.0 \text{ ft} \\ S_d &= 24.82 \text{ ft} \\ t_f &= 0.75 \text{ inches} \\ b_f &= 18 \text{ inches} \\ t_{tf} &= 0.6875 \text{ inches} \\ F_{DL} &= 20.00 \text{ ksi (from Chapter 5 design)} \\ F_{L+I} &= 7.94 \text{ ksi (from Chapter 5 design)} \end{aligned}$$

Half of the wind load is applied in the plane of the bottom flange.

$$W = \frac{\left(16.67 + \frac{0.75}{12}\right) \left(\frac{50}{1000}\right)}{2} = 0.418 \text{ kips/ft}$$

$$\begin{aligned} R &= [0.2272L - 11] S_d^{-\frac{2}{3}} \\ &= [(0.2272)(273) - 11] 24.82^{-\frac{2}{3}} = 5.997 \\ M_{cb} &= 0.08WS_d^2 = (0.08)(0.418)(24.82)^2 = 20.60 \text{ ft-kips} \\ F_{cb} &= \frac{72M_{cb}}{t_f b_f^2} = \frac{(72)(20.6)}{(0.75)(18)^2} = 6.10 \text{ ksi} \\ F_W &= RF_{cb} (5.997)(6.10) = 36.58 \text{ ksi} \\ \text{Total factored stress} &= \end{aligned}$$

Group II:

$$1.3(F_{DL} + F_W) = 1.3(20.00 + 36.58) = 73.56 \text{ ksi} > 50 \text{ ksi}$$

Group III:

$$1.3(F_{DL} + F_{L+I} + 0.3F_W) = 1.3[20.0 + 7.94 + (0.3)(36.58)] = 50.59 \text{ ksi} > 50 \text{ ksi}$$

Since, at this location, the stress in the bottom flange is tension, the limit stress is FY. The flange stress exceeds the limit stress for both Group II and Group III loadings. Corrective options outlined above will not be considered until other critical sections along the girder have been investigated.

(Investigation of all sections will not be illustrated because the calculations would be repetitive.)

The next illustration is that of the flange transition section, 129 ft from the end bearing. At this location the bottom flange is again in tension and, as shown below, the Group II and Group III stresses are well below the limit stress.

#### Section 129 ft Rt. of Support 1

Limit stress for tension: FY = 50 ksi

$$\begin{aligned} L &= 273 \text{ ft} \\ S_d &= 24.82 \text{ ft} \\ t_f &= 1.875 \text{ inches} \\ b_f &= 18 \text{ inches} \\ t_{tf} &= 0.8125 \text{ inches} \\ F_{DL} &= 18.60 \text{ ksi (from Chapter 5 design)} \\ F_{L+I} &= 10.85 \text{ ksi (from Chapter 5 design)} \end{aligned}$$

$$W = \frac{\left(16.67 + \frac{1.875}{12}\right) (0.050)}{2} = 0.421 \text{ k/ft}$$

$$\begin{aligned} R &= 5.997 \text{ (Same as for Section 34' Rt. of Support 1)} \\ M_{cb} &= 0.08WS_d^2 = (0.08)(0.421)(24.82)^2 \\ &= 20.75 \text{ ft-kips} \end{aligned}$$

$$F_{cb} = \frac{72M_{cb}}{t_f b_f^2} = \frac{(72)(20.75)}{(1.875)(18)^2} = 2.46 \text{ ksi}$$

$$F_W = RF_{cb} (5.997)(2.46) = 14.75 \text{ ksi}$$

Total factored stress =

Group II:

$$1.3(F_{DL} + F_W) = 1.3(18.60 + 14.75) = 43.35 \text{ ksi} < 50 \text{ ksi}$$

Group III:

$$1.3(F_{DL} + F_{L+I} + 0.3F_W) = [1.3(18.60 + 10.85 + (0.3)(14.75))] = 44.04 \text{ ksi} < 50 \text{ ksi}$$

Since  $L$  and  $S_d$  are the same as in the previous calculation,  $R$  is the same as for the previous section and therefore is not recalculated. Wind load,  $W$ , varies from that of the previous section only because of the slightly different wind area resulting from a different flange thickness. (For all practical purposes,  $W$  could be treated as constant for the entire girder.)

The bottom flange transition section 219 ft from the end bearing, is now illustrated. At this location the bottom flange is a compression flange. Local buckling is determined to be the one that governs the limit stress.

#### Section 219 ft Rt. of Support 1

Limit Stress: FY = 50 ksi

$$\begin{aligned} L &= 273 \text{ ft} \\ S_d &= 24.82 \text{ ft} \\ t_f &= 1.25 \text{ inches} \\ b_f &= 26 \text{ inches} \\ t_{tf} &= 0.8125 \text{ inches} \\ F_{DL} &= -15.90 \text{ ksi (from Chapter 5 design)} \\ F_{L+I} &= -10.00 \text{ ksi (from Chapter 5 design)} \end{aligned}$$

$$W = \frac{\left(16.67 + \frac{1.25}{12}\right) (0.050)}{2} = 0.419 \text{ kips/ft}$$

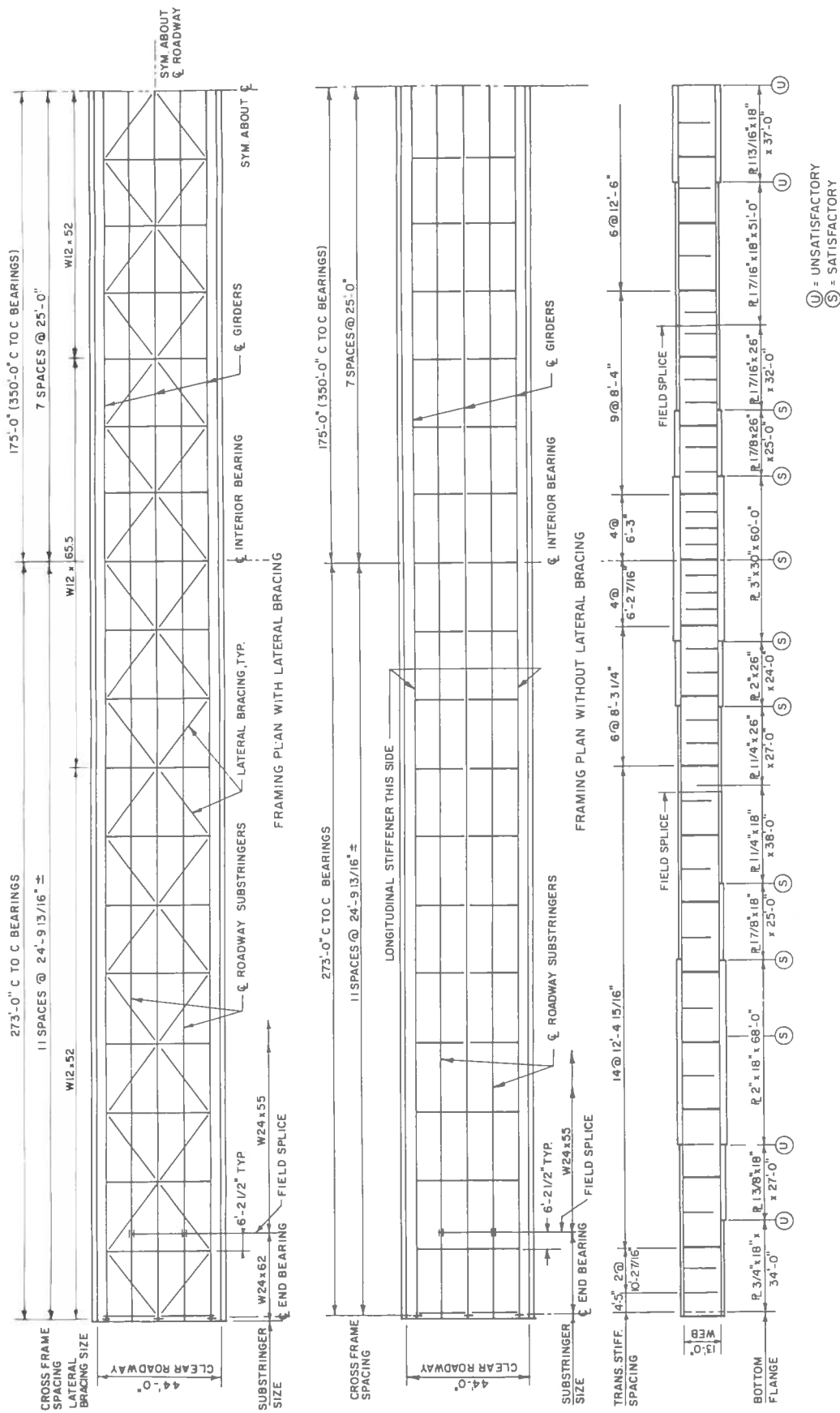


Figure 2.

$$R = 5.997 \text{ (Same as for Section 34' ft Rt. of Support 1)}$$

$$M_{cb} = (0.08)(0.419)(24.82)^2 = 20.65 \text{ ft-kips}$$

$$F_{cb} = \frac{(72)(20.65)}{(1.25)(26)^2} = 1.76 \text{ ksi}$$

$$F_w = (5.997)(1.76) = 10.55 \text{ ksi}$$

$$FYE = \frac{\left[ 2,200 \left( \frac{1.25}{26/2} \right) \right]^2}{1000} = 44.75 \text{ ksi}$$

Governs

$$\text{Limiting } S_d = \frac{(20,000)(26)(1.25)}{(12)(50)(156 + 1.25 + 0.8125)}$$

$$= 6.85' < 24.82$$

$$FYL = 50 \left[ 1 - \frac{(3)(50)}{(4)(\pi^2)(29,000)} \left( \frac{24.82 \times 12}{26/2} \right)^2 \right]$$

$$= 46.56 \text{ ksi}$$

Total factored stress =

Group II:

$$1.3(F_{DL} + F_w) = 1.3(15.90 + 10.55) = 34.39 \text{ ksi} < 44.75 \text{ ksi}$$

Group III:

$$1.3(F_{DL} + F_{L+I} + 0.3 F_w) = 1.3[15.90 + 10.00 + (0.3)(10.55)] = 37.78 \text{ ksi} < 44.75 \text{ ksi}$$

$$FYE = \frac{\left[ 2,200 \left( \frac{3}{30/2} \right) \right]^2}{1000} = 193.60 \text{ ksi}$$

$$\text{Limiting } S_d = \frac{(20,000)(30)(3)}{(12)(50)(156 + 3 + 2.875)}$$

$$= 18.53' < 25$$

$$FYL = 50 \left[ 1 - \frac{(3)(50)}{(4)(\pi^2)(29,000)} \left( \frac{25 \times 12}{30/2} \right)^2 \right]$$

$$= 47.38 \text{ ksi Governs}$$

$$W = \frac{\left( 16.67 + \frac{3}{12} \right)(0.050)}{2} = 0.423 \text{ kips/ft}$$

$$R = |(0.2272)(350) - 11| 25^{-3/4} = 8.014$$

$$M_{cb} = (0.08)(0.423)(25)^2 = 21.15 \text{ ft-kips}$$

$$F_{cb} = \frac{(72)(21.15)}{(3)(30)^2} = 0.56 \text{ ksi}$$

$$F_w = (8.014)(0.56) = 4.49 \text{ ksi}$$

$$L = 350 \text{ ft}$$

$$S_d = 25 \text{ ft}$$

$$t_f = 3 \text{ inches}$$

$$b_f = 30 \text{ inches}$$

$$t_{ff} = 2.875 \text{ inches}$$

$$F_{DL} = -23.60 \text{ ksi (from Chapter 5 design)}$$

$$F_{L+I} = -9.22 \text{ ksi (from Chapter 5 design)}$$

Total factored stress =

Group II:

$$1.3(F_{DL} + F_w) = 1.3(23.60 + 4.49) = 36.51 \text{ ksi} < 47.38$$

Group III:

$$1.3(F_{DL} + F_{L+I} + 0.3 F_w) = 1.3[23.60 + 9.22 + (0.3)(4.49)]$$

$$= 44.42 \text{ ksi} < 47.38 \text{ ksi}$$

At Support 2, FYL is the controlling limit stress. But, since it exceeds the factored stress produced by loads by a substantial margin, once again the increase of 20 percent need not be looked at.

Results of an investigation of the entire exterior girder are summarized on the girder elevation in Figure 2. (While all control sections have been checked, not all calculations are given here since they are essentially the same as those already illustrated.) Note that sections at field splices are *not* checked because they are not control sections for strength. The criterion for Group II loading [ $1.3(F_{DL} + F_w) \leq F_Y, F_{YE}, F_{YL}$ ] and for Group III loading [ $1.3(F_{DL} + F_{L+I} + 0.3 F_w) \leq F_Y, F_{YE}, F_{YL}$ ] is satisfied without lateral bracing at all sections except for two flange transition sections adjacent to each end bearing, and at the three sections near the middle of the interior span.

As stated earlier, a number of measures can be taken to lower wind stresses. Two such, are very effective solutions. Solution 1 calls for widening the bottom flange of the girder in regions where overstressing occurs. The initial measure should be to replace the overstressed flange with a wider plate of the same area as the original plate, making certain that in doing so all

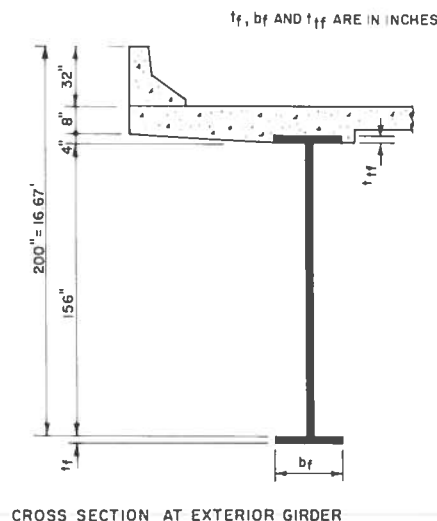


Figure 3.

Since FYL is not the governing limit stress, the possibility of a 20 percent increase is not checked here. But, this provision should be investigated when FYL would otherwise yield the lowest limit stress, and when this limit stress is lower than the actual stress.

The bottom flange at Support 2 is now illustrated:

Section at Support 2

$$\text{Limit Stress: } F_Y = 50 \text{ ksi}$$

[illegible]

## ELEVATION OF EXTERIOR GIRDER



girder performance criteria for Group I loading are still satisfied. With the wider flange of the same area as the original flange,  $F_{DL}$  and  $F_{L+I}$  will remain approximately the same and  $F_w$  will be reduced. The girder weight will not be changed. If the limit stress cannot be satisfied with this wider, equivalent-area plate, then a wider, greater-area plate becomes necessary. A greater-area plate will reduce all three stresses— $F_{DL}$ ,  $F_{L+I}$ , and  $F_w$ . It will add weight to the girder, but probably far less weight than that of the eliminated lateral bracing.

In solution 2, the crossframe spacing is reduced in regions where overstressing occurs. Since wind stresses are inversely proportional to the square of the crossframe spacing, they can be substantially reduced by modest adjustments in the framing. Weight will be added in the form of more crossframes but the overall weight should be lowered to a greater degree by eliminating lateral bracing.

Lateral bracing offers another solution. With lateral bracing, Article 1.7.17 specifies a different formula for the parameter  $R$  which yields lower values of  $R$  and therefore lower wind stresses. But, of the three solutions, this last is the least attractive. For this bridge, it would mean using lateral bracing in all three spans to accommodate 7 sections overstressed out of 23. And it is not even certain that lateral bracing would eliminate all overstressing. *Solution 3 should be considered only when Solutions 1 and 2 fail.*

Solutions 1 and 2 as applied to the design from Chapter 5 will now be illustrated.

#### Solution 1:

The girder elevation (Fig. 2) shows that the overstressed locations are points of 18-inch bottom flange width. With other bottom flange widths of 26 and 30 inches, 22 inches would seem a good choice for a revised width of the 18-inch flange. The section 34 ft from Support 1 is investigated with a  $\frac{7}{8}$  x 22-inch bottom flange. No change is made to the top flange.

#### Section 34' Rt. of Support 1 (Revised Solution 1)

Limit Stress:  $F_Y = 50$  ksi

$$\begin{aligned} t_f &= 0.875" \\ b_f &= 22" \\ t_{tf} &= 0.6875" \\ F_{DL} &= 16.81 \text{ ksi} \\ F_{L+I} &= 6.82 \text{ ksi} \\ F_{cb} &= \frac{(20.60)(12)}{(0.875)(22)^2/6} = 3.50 \text{ ksi} \\ F_w &= (5.997)(3.50) = 20.99 \text{ ksi} \end{aligned}$$

Group II:

$$1.3(F_{DL} + F_w) = 1.3(16.81 + 20.99) = 49.14 \text{ ksi} < 50 \text{ ksi} - \text{Governs}$$

Group III:

$$1.3(F_{DL} + F_{L+I} + 0.3F_w) = 1.3[16.81 + 6.82 + (0.3)(20.99)] = 38.91 \text{ ksi} < 50 \text{ ksi}$$

It is noted that the area of the  $\frac{7}{8}$  x 22-inch revised flange is greater than the area of the original  $\frac{3}{4}$  x 18-inch flange. An equivalent-area 22-inch wide flange would not have worked, as evidenced by the closeness of the Group II stress, above, to the limit stress.

The next section, 61 ft from Support 1, is now revised.

#### Section 61' Rt. of Support 1 (Revised-Solution 1)

Limit Stress:  $F_Y = 50$  ksi

$$\begin{aligned} t_f &= 1.25" \\ b_f &= 22" \\ t_{tf} &= 0.6875" \\ F_{DL} &= 21.07 \text{ ksi} \\ F_{L+I} &= 9.07 \text{ ksi} \\ F_{cb} &= \frac{(20.60)(12)}{(1.25)(22)^2/6} = 2.45 \text{ ksi} \\ F_w &= (5.997)(2.45) = 14.69 \text{ ksi} \end{aligned}$$

Group II:

$$1.3(F_{DL} + F_w) = 1.3(21.07 + 14.69) = 46.49 \text{ ksi} < 50 \text{ ksi} - \text{Governs}$$

Group III:

$$1.3(F_{DL} + F_{L+I} + 0.3F_w) = 1.3[21.07 + 9.07 + (0.3)(14.69)] = 44.91 \text{ ksi} < 50 \text{ ksi}$$

Again the revised flange area is larger than the original flange area.

All other sections of the positive-moment shipping piece in Span 1 were satisfactory as originally designed. However, it would not be good practice to mix 18-inch and 22-inch bottom flange widths. The flanges of the remaining three sections are therefore changed to 22-inch width equivalent-area plates with no investigation needed.

Solution 1 is completed by making similar flange width changes to the sections 138 ft and 175 ft from Support 2. The revised Exterior Girder elevation is shown in Figure 4; and a steel weight summary and comparison with the original design is given below. *Solution 1 yields 6.2 percent savings in steel relative to the original design, gained by widening and thickening the bottom flange in certain regions to meet loading Group II and III limit stresses, and thereby eliminating lateral bracing.*

#### STEEL WEIGHT SUMMARY

(Original)

Interior Girder	593,674
Field Splices	3,335
Exterior Girders	1,005,978
Field Splices	5,872
Stringer & Splices	102,672
Lateral Bracing	129,207
Crossframes	89,364
	<hr/>
	1,930,102 #

$$\text{Weight Per Square Foot of Slab} = \frac{1,930,102}{47' \times 896'} = 45.8 \#$$

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## STEEL WEIGHT SUMMARY

(Revised—Solution 1)

Interior Girder	593,674
Field Splices	3,335
Exterior Girders	1,013,378
Field Splices	5,872
Stringer & Splices	102,672
Lateral Bracing	—
Crossframes	89,364
	<u>1,808,295 #</u>

Savings = 118,807 # (6.2%)

Weight Per Square Foot of Slab =  $\frac{1,808,295}{47' \times 896'} = \underline{\underline{42.9 \#}}$

### Solution 2:

Solution 2 accomplishes the same thing by reducing the crossframe spacing in the region where the overstress occurs. Figure 5 shows an adjusted framing plan in which the spacing of crossframes has been decreased in the vicinity of the overstressed locations. The number of crossframe lines has increased from 37 to 43.

The section 34 ft from the end bearing is reinvestigated, this time with a crossframe spacing of 14.6 ft. The calculations below show that the stresses are now satisfactory.

### Section 34 ft Rt. of Support 1 (Revised—Solution 2)

Limit stress for tension:  $F_Y = 50$  ksi

$S_d = 14.60$  ft

$R = [0.2272(273) - 11] 14.60^{-3/4} = 8.545$

$M_{cb} = (0.08)(0.418)(14.60)^2 = 7.13$  ft-kips

$F_{cb} = \frac{(72)(7.13)}{(0.75)(18)^2} = 2.11$  ksi

$F_W = (8.545)(2.11) = 18.05$  ksi

Total factored stress =

Group II:

$1.3(F_{DL} + F_W) = 1.3(20.00 + 18.05) = 49.47$  ksi < 50 ksi

Group III:

$1.3(F_{DL} + F_{L+I} + 0.3F_W) = 1.3[20.00 + 7.94 + (0.3)(18.05)] = 43.36$  ksi < 50 ksi

A new calculation of stresses is also performed at the middle of the center span. Again the results, given below, show that the design is satisfactory.

### Section 175 ft Rt. of Support 2 (Revised)

$S_d = 16.67$  ft      Limit stress for tension:  
 $F_Y = 50$  ksi

$R = [0.2272(350) - 11] 16.67^{-3/4} = 10.5$

$M_{cb} = (0.08)(0.421)(16.67)^2 = 9.36$  ft-kips

$F_{cb} = \frac{(72)(9.36)}{(1.8125)(18)^2} = 1.15$  ksi

$F_W = (10.505)(1.15) = 12.05$  ksi

Total factored stress =

Group II:

$1.3(F_{DL} + F_W) = 1.3(18.70 + 12.05) = 39.97$  ksi < 50 ksi

Group III:

$1.3(F_{DL} + F_{L+I} + 0.3F_W) = 1.3[18.70 + 11.12 + (0.3)(12.05)] = 43.47$  ksi < 50 ksi

A complete investigation indicates that wind stresses in the entire structure have been reduced to allowable values by shortening the crossframe spacing in selected regions.

For comparison, weight summaries are given below for the original structure of Chapter 5 and the revised structure — Solution 2 — with lateral bracing eliminated. A 6.0 percent savings in structural steel weight is achieved.

For the particular structure in this example, with its girder-substringer construction and, of necessity, heavy crossframes, Solution 1 proved the most economical means of eliminating lateral bracing. However, in multi-girder stringer-type bridges having lighter crossframes, Solution 2 would probably be more economical. Clearly, for any given bridge, both solutions should be investigated. *It should be noted that one advantage of Solution 2 is the fact that the girder design needn't be adjusted.*

## STEEL WEIGHT SUMMARY

(ORIGINAL DESIGN WITH LATERAL BRACING)

Interior Girder	593,674
Field Splices	3,335
Exterior Girders	1,005,978
Field Splices	5,872
Stringer & Splices	102,672
Lateral Bracing	129,207
Crossframes	89,364
	<u>1,930,102 #</u>

Weight Per Square Foot of Slab =  $\frac{1,930,102}{47' \times 896'} = \underline{\underline{45.8 \#}}$

## STEEL WEIGHT SUMMARY

(REVISED—Solution 2)

Interior Girder	593,674
Field Splices	3,335
Exterior Girders	1,005,978
Field Splices	5,872
Stringer & Splices	102,672
Lateral Bracing	—
Crossframes	100,369
	<u>1,811,900 #</u>

Weight Per Square Foot of Slab =  $\frac{1,811,900}{47' \times 896'} = \underline{\underline{43.0 \#}}$

SAVINGS = 115,202 # (6.0%)



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