Plate Girder Design Using LRFD

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What differentiates a beam from a plate girder? This may seem to be a trivial question. However, it is a necessary and important part of plate girder design when applying the Load and Resistance Factor Design (LRFD) Specification.[1] A beam can be a rolled or a welded shape, but it does not have intermediate stiffeners and its web width-thickness ratio h_c/t_w must not exceed $970/\sqrt{F_{yf}}$, where h_c is twice the distance from the neutral axis to the compression flange (Fig. 1). Plate girders have stiffeners or h_c/t_w is greater than $970/\sqrt{F_{yf}}$, or both.

Making this distinction and the compactness classification early in plate girder design is important when using the LRFD Specification. The significance of these items can be seen in the LRFD Manual flowcharts (Figs. 2 and 3) for the determination of flexural and shear design strength. With these flowcharts, this discussion will focus on the design of plate girders according to LRFD rules. An explanation of plate girder design in the LRFD Specification will include: flexural design strength, shear design strength, flexure-shear interaction, bearing strength under concentrated loads and stiffener design. Application of the LRFD method will show there is actually little difference between it and Allowable Stress Design (ASD)[8] of plate girders.

FLEXURAL STRENGTH

The design flexural strength is calculated to provide an adequate section modulus. The first criterion necessary to separate a beam from a plate girder, $970/\sqrt{F_{yf}}$, relates to flexural design strength. Although there are preliminary steps, LRFD Specification Appendix G2* gives the flexural capacity for a plate girder, whereas Appendix F1.7 is applicable to beams.

Entering the flowchart (Fig. 2) with a trial web and flange plate size, the initial step is to determine whether plastic or elastic design is appropriate. Plastic design is permitted if the section is compact and adequate bracing is provided in accordance with Sect. F1.1, such that "the laterally unbraced length L_b of the compression flange at plastic hinge locations associated with the failure mechanism" does not exceed

$$L_{pd} = \frac{3{,}600 + 2{,}200(M_1/M_p)r_y}{F_y} \qquad \text{(F1-1)}$$

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For elastic design, which is applicable to plate girders, compactness can be verified by Sect. B5 or by the flexural strength provisions of Appendices F1.7 and G2.

At the next decision point of the flowchart, the section is classified as a beam or a plate girder based on the flexural strength criterion for web slenderness, $970/\sqrt{F_{yf}}$. If h_c/t_w does not exceed this value, Appendix F1.7 is used and the section is designed like a beam. As mentioned earlier, a beam may be a rolled or welded shape. Appendix F1.7 takes this into account with F_r. Equations for the limiting buckling moment M_r include a term F_r to incorporate the flange residual compressive stress. For rolled shapes, F_r is 10 ksi, and for welded shapes, F_r is 16.5 ksi. As is indicated by the formulas in Appendix F1.7, the entire cross section is assumed to contribute to the flexural design strength. On the other hand, Appendix G2 is based on a buckled web with the flanges principally resisting the bending.

Appendix G2 is only applicable if the section is a plate girder as defined by $h_c/t_w > 970/\sqrt{F_{yf}}$. Plate girders have three possible modes of failure: compression flange buckling vertically into the web, lateral-torsional buckling or compression flange local buckling.[6] The first mode of failure is avoided by the upper limits on h/t_w given in Appendix G1 based on a/h:

$$\text{for } a/h \le 1.5,\ h/t_w \le 2{,}000/\sqrt{F_{yf}} \qquad \text{(A-G1-1)}$$

$$\text{for } a/h \le 1.5,\ h/t_w \le 14{,}000/\sqrt{F_{yf}\,(F_{yf}+16.5)} \qquad \text{(A-G1-2)}$$

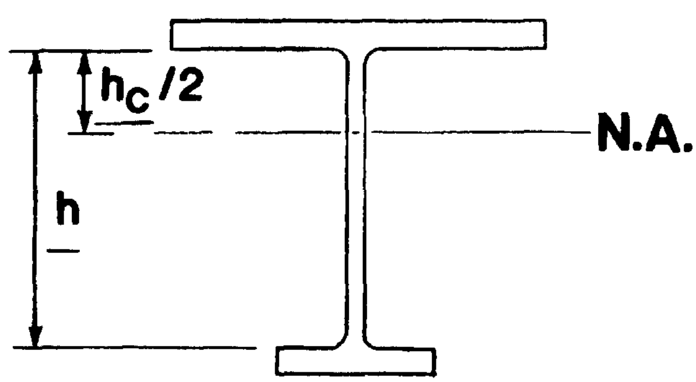

Figure 1

* *All references to Appendices and Sections are from the LRFD Specification.*

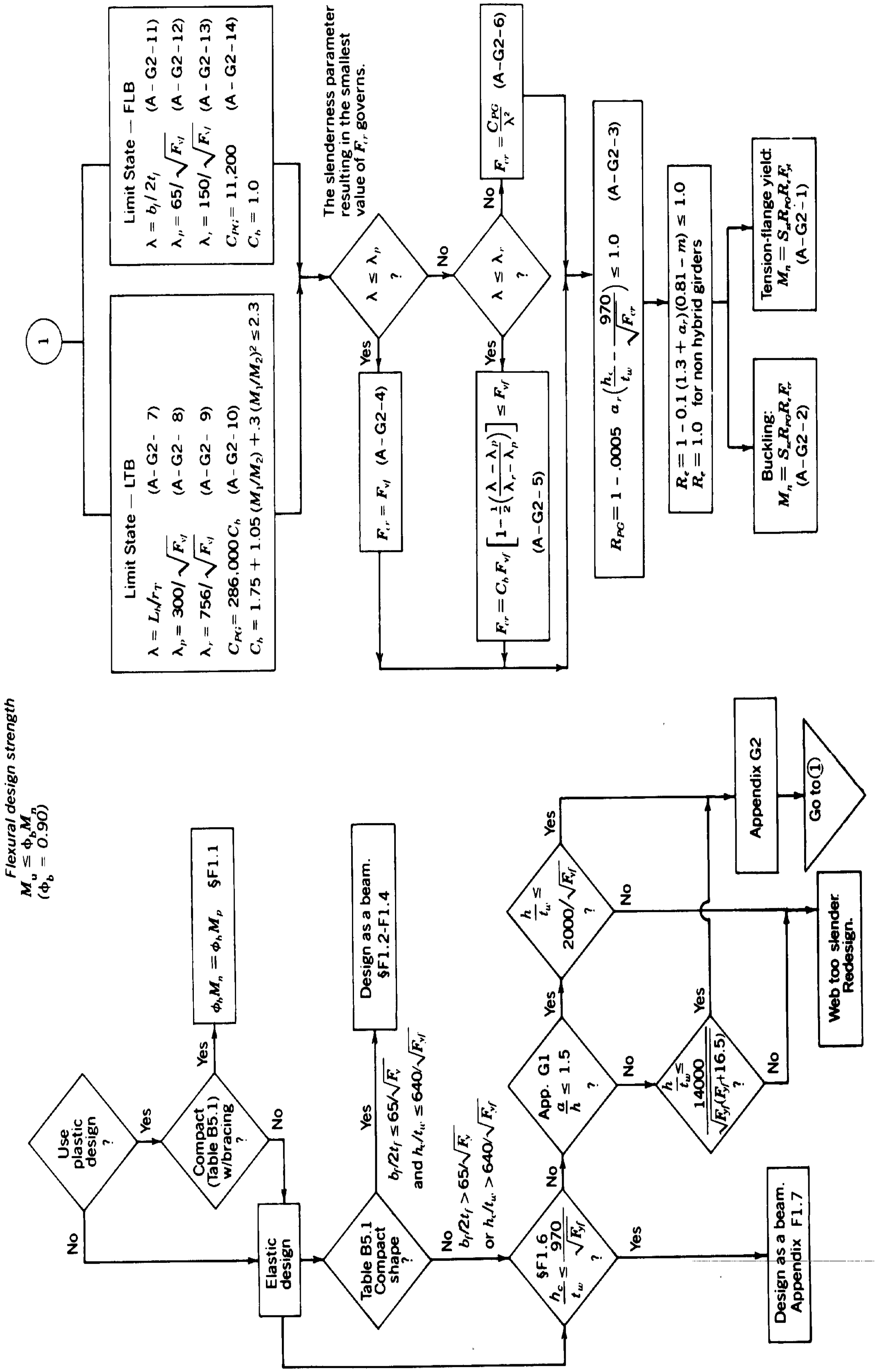
Flexural design strength
$M_u \le \phi_b M_n$
(ϕ_b = 0.90)
Use plastic design ?
Yes
No
Compact (Table B5.1) w/bracing ?
Yes
No
$\phi_b M_n = \phi_b M_p$ §F1.1
Elastic design
Table B5.1 Compact shape ?
Yes
$b_f/2t_f \le 65/\sqrt{F_y}$ and $h_c/t_w \le 640/\sqrt{F_{yf}}$
Design as a beam. §F1.2-F1.4
No
$b_f/2t_f > 65/\sqrt{F_y}$ or $h_c/t_w > 640/\sqrt{F_{yf}}$
§F1.6 $\frac{h_c}{t_w} \le \frac{970}{\sqrt{F_{yf}}}$?
Yes
Design as a beam. Appendix F1.7
No
App. G1 $\frac{a}{h} \le 1.5$?
Yes
No
$\frac{h}{t_w} \le 2000/\sqrt{F_{yf}}$?
Yes
No
$\frac{h}{t_w} \le \frac{14000}{\sqrt{F_{yf}(F_{yf}+16.5)}}$?
Yes
No
Appendix G2
Go to 1
Web too slender. Redesign.
1
Limit State — LTB
$\lambda = L_b/r_T$ (A-G2- 7)
$\lambda_p = 300/\sqrt{F_{yf}}$ (A-G2- 8)
$\lambda_r = 756/\sqrt{F_{yf}}$ (A-G2- 9)
$C_{PG} = 286{,}000\,C_b$ (A-G2-10)
$C_b = 1.75 + 1.05\,(M_1/M_2) + .3\,(M_1/M_2)^2 \le 2.3$
Limit State — FLB
$\lambda = b_f/2t_f$ (A-G2-11)
$\lambda_p = 65/\sqrt{F_{yf}}$ (A-G2-12)
$\lambda_r = 150/\sqrt{F_{yf}}$ (A-G2-13)
$C_{PG} = 11{,}200$ (A-G2-14)
$C_b = 1.0$
The slenderness parameter resulting in the smallest value of F_{cr} governs.
$\lambda \le \lambda_p$?
Yes
No
$F_{cr} = F_{yf}$ (A-G2-4)
$\lambda \le \lambda_r$?
Yes
No
$F_{cr} = C_b F_{yf}\left[1-\frac{1}{2}\left(\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right)\right] \le F_{yf}$ (A-G2-5)
$F_{cr} = \frac{C_{PG}}{\lambda^2}$ (A-G2-6)
$R_{PG} = 1 - .0005\,a_r\left(\frac{h_c}{t_w} - \frac{970}{\sqrt{F_{cr}}}\right) \le 1.0$ (A-G2-3)
$R_e = 1 - 0.1\,(1.3 + a_r)(0.81 - m) \le 1.0$
$R_e = 1.0$ for non hybrid girders
Buckling: $M_n = S_{xc} R_{PG} R_e F_{cr}$ (A-G2-2)
Tension-flange yield: $M_n = S_{xt} R_{PG} R_e F_{yt}$ (A-G2-1)

Figure 2

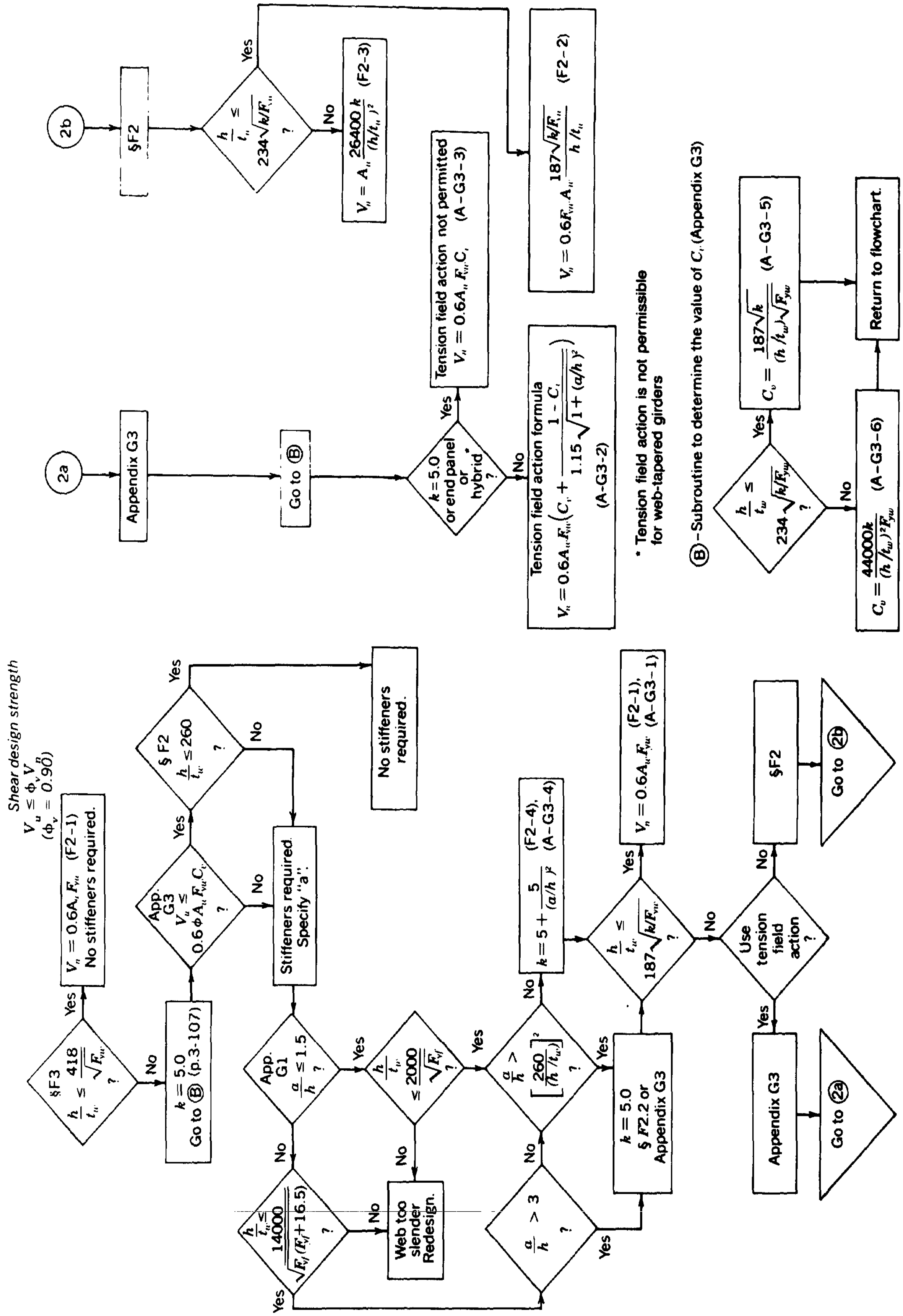
Shear design strength
§F3
No stiffeners required.
Stiffeners required. Specify "a".
No stiffeners required.
Web too slender Redesign.
k = 5.0 § F2.2 or Appendix G3
Appendix G3
Go to 2a
§F2
Go to 2b
Use tension field action ?
Appendix G3
Go to B
Tension field action not permitted
Tension field action formula
* Tension field action is not permissible for web-tapered girders
B -Subroutine to determine the value of C_v (Appendix G3)
Return to flowchart.

Figure 3

If the appropriate criterion is satisfied, Appendix G2 can be used, otherwise a new trial size must be selected. Once these limitations are met, the slenderness parameters (λ, λ_p, λ_r) can be determined for the other two possible failure modes, lateral-torsional buckling (LTB) and flange local buckling (FLB), as shown in the flowchart. Next, F_{cr} is calculated for each of these limit states and the smaller value governs. With this resulting F_{cr}, the nominal flexural strength, M_n is determined for the limit states of tension flange yield and compression flange buckling. The flexural capacity is the lesser value of M_n for these limit states multiplied by the bending resistance factor ϕ_b of 0.90. For both limit states M_n is dependent on two reduction coefficients, R_{PG} and R_e. The plate girder reduction factor R_{PG} accounts for the strength reduction due to elastic web buckling. The hybrid girder factor R_e is equal to 1.0 for homogeneous girders. Application of R_e will be discussed subsequently.

After all the necessary values have been determined, the flexural design strength can be compared to the maximum moment in the panel in question caused by the factored loads. Once the trial section modulus has been deemed satisfactory, the need for bearing and intermediate stiffeners should be determined.

Hybrid Sections

The use of two steel grades in a bending member affects three parameters when determining the flexural design strength: $970/\sqrt{F_{yf}}$, M_p and R_e. The $970/\sqrt{F_{yf}}$ limit is the demarcation where bend-buckling of the web may begin to affect flexural strength. This limit is written in terms of the flange yield stress because stability of the web due to bending is dependent on the flange strain.[3] Thus, the limit indicates the same web dimensions would be required for a hybrid section as for a homogeneous girder made entirely of the higher grade steel. Recent research substantiates the elastically derived $970/\sqrt{F_{yf}}$ limit also applies to partially yielded hybrid girder webs. A study by Dawe and Kulak[4] verifies this boundary from that perspective, as they found that web plastification can occur up to $800/\sqrt{F_{yw}}$. For a hybrid girder with an A36 web and 50 ksi flanges:

$$800/\sqrt{F_{yw}} = 133 \simeq 970/\sqrt{F_{yf}} = 137$$

If the hybrid section does not exceed $970/\sqrt{F_{yf}}$, the shape is designed like a beam. In this case, the advantage of hybrid sections is evident only in the inelastic and plastic ranges of the beam curve. Figure 4 demonstrates graphically this point. Note all three curves are the same beyond L_r (elastic lateral-torsional buckling range). For unbraced lengths less than L_r, the difference between the hybrid and homogeneous curves is a result of the M_p calculation. As Fig. 5 shows, fully yielded stress blocks in the web and flanges are used in computing M_p for a compact hybrid beam.

When the $970/\sqrt{F_{yf}}$ limit is exceeded, the hybrid girder design criteria must be satisfied. The hybrid girder reduction factor R_e accounts for the strength reduction due to web

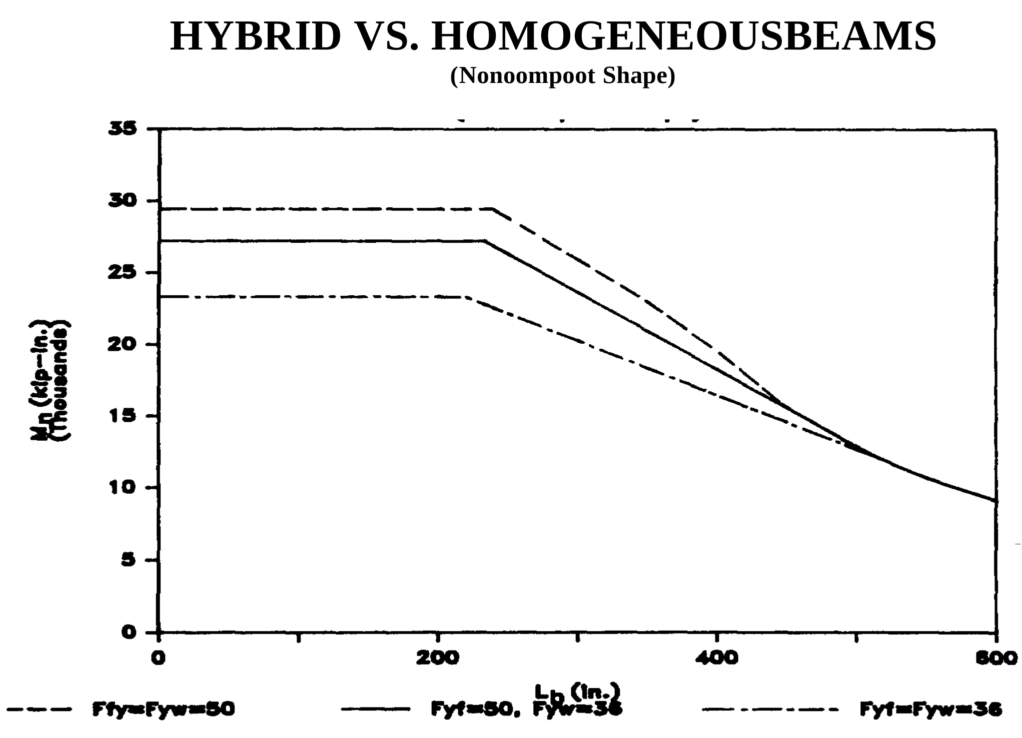

Figure 4

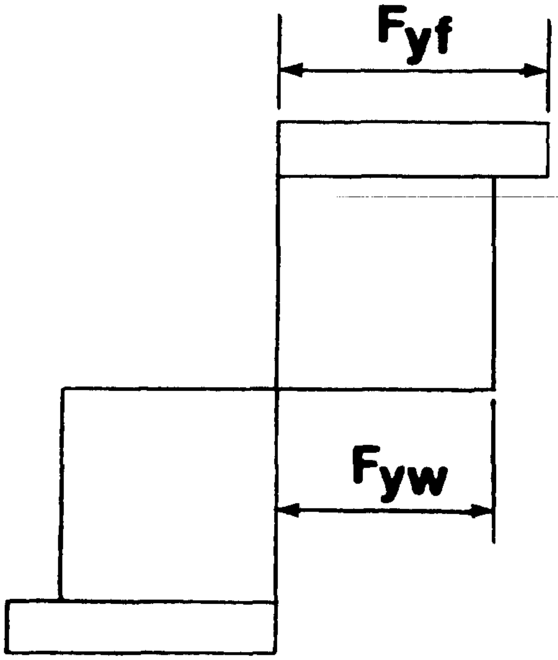

Figure 5

yielding and is only important in hybrid girder design. The formula for R_e given in the flowchart and in the LRFD Specification is a conservative approximation of the hybrid girder reduction factor formula given in the ASD Specification.[8] As mentioned, the plate girder reduction factor R_{PG} reduces strength due to slender web buckling. In the LRFD Specification, these reduction factors are multiplicable in the determination of flexural design strength for hybrid girders. To demonstrate the validity of including *both* R_e and R_{PG} in this M_n calculation, a plot based on web local buckling is shown in Fig. 6. Beyond λ_r, the curve for the hybrid and the homogeneous 50 ksi steel shape run nearly parallel. This would be expected, because beyond λ_r the strength contribution of the web diminishes substantially. However, the slightly higher level of the 50 ksi homogeneous curve indicates that the web is still contributing some strength.

SHEAR STRENGTH

The need for intermediate stiffeners is based on the shear capacity of the section or the value of h/t_w, whichever governs. For shear strength calculations, h represents the clear web depth between flanges (Fig. 1), which equals h_c only for doubly symmetric sections. If the design depends on tension field action, Appendix G3 is appropriate; if not, Sect. F2 should be employed.

Disregarding tension field action initially, enter the flowchart with values for h/t_w, F_y, A_w and V_u. The need for stiffeners can be resolved immediately. Stiffeners are not required if $h/t_w \leq 418/\sqrt{F_{yw}}$. The nominal shear strength V_n is then $0.6F_{yw}A_w$. This is a practical limit that basically applies to the design of rolled beams, as all A36 beams and most 50 ksi beams have $h/t_w \leq 418/\sqrt{F_{yw}}$. This limit is derived from the h/t_w limitation that gives the largest shear capacity without tension field action:

$$418/\sqrt{F_{yw}} = 187\sqrt{k/F_{yw}} \text{ with } k = 5.0 \text{ (no stiffeners)}$$

If the required shear strength V_u exceeds $\phi_v V_n$, a larger beam

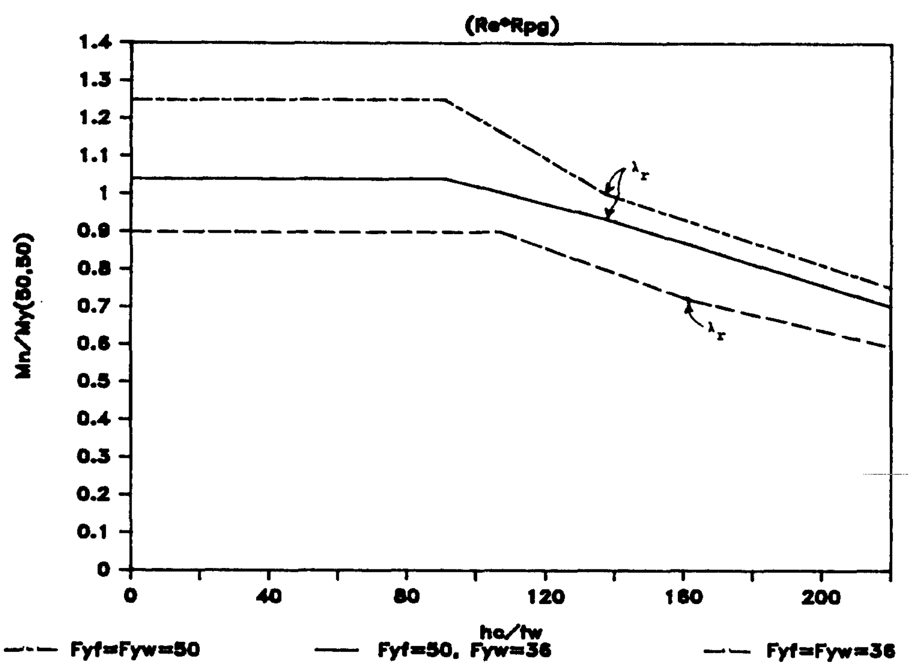

Figure 6

size must be selected. For larger values of h/t_w, stiffeners are not required unless the required shear strength V_u exceeds the design shear strength $\phi_v V_n$ equal to $\phi_v 0.6 A_w F_{yw} C_v$ (Appendix G3) or to the equivalent values in Sect. F2 (explained in a later paragraph). The value of C_v is determined in the subroutine labelled Ⓑ in the flowchart, with the web plate buckling coefficient k equal to 5.0.

The formula for k is

$$k = 5 + \frac{5}{(a/h)^2} \quad \text{(F2-4 and A-G3-4)}$$

This formula is approximately an average (Fig. 7) of the two formulas given in the ASD Specification:[8]

$$\text{for } a/h < 1.0, k = 4 + \frac{5.34}{(a/h)^2}$$

$$\text{for } a/h > 1.0, k = 5.34 + \frac{4.00}{(a/h)^2}$$

The value $k = 5$ may be assumed since in the initial design phase, there are no intermediate stiffeners (stiffener spacing "a" is large). If the design shear strength is found to be sufficient, one final criterion must be met for an unstiffened member: h/t_w must be less than 260.

If stiffeners are needed, the required stiffener spacing "a" must be determined by trial and error as in the current ASD Specification. Also, based on this "a," the value of h/t_w must be checked against the same uppermost limits, given in Appendix G1, as were checked for flexural strength. If these limits are exceeded, a new section must be selected. Next, the required shear strength V_u is compared to $\phi_v V_n$ calculated from the appropriate formula using Sect. F2 for no tension field action and Appendix G3 otherwise. Alternatively, tables are included in the LRFD Manual to assist with this calculation.[1] These tables will be explained later.

The decision to use tension field action (Appendix G3) must depend on the following. Tension field action is not permitted if the section is an end panel, hybrid girder, $h/t_w \leq 187\sqrt{k/F_{yw}}$, web-tapered or if k equals 5.0. The latter implies the maximum a/h allowed for tension field action is the smaller of 3.0 or $[260/(h/t_w)]^2$. For tension field action, therefore, the required stiffener spacing "a" must be calculated from this criterion initially, followed by the calculation of $\phi_v V_n$ with $\phi_v = 0.90$ and Formula A-G3-2:

$$V_n = 0.6 A_w F_{yw}\left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \quad \text{(A-G3-2)}$$

Further understanding of the shear strength calculation may be gained by referring to Fig. 3 and comparing Sect. F2 to Appendix G3. The primary difference lies in Formula A-G3-2. This formula applies to $h/t_w > 187\sqrt{k/F_{yw}}$ and contains the extra term to account for the additional tension field strength (post-buckling) provided over the elastic Sect. F2 equations.

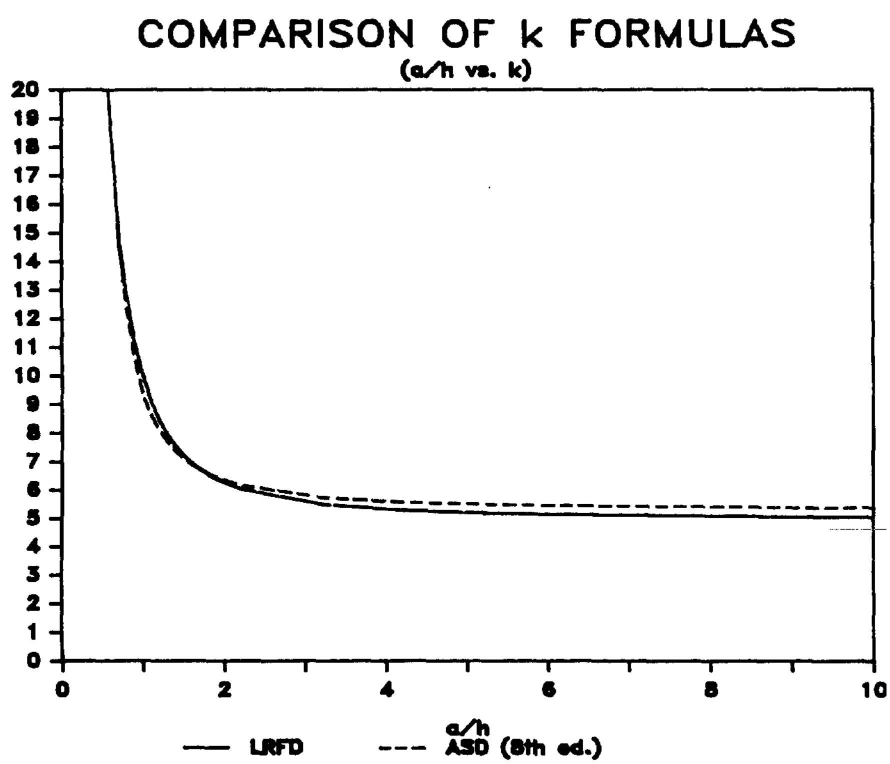

Figure 7

The inclusion of this term

$$\frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}$$

along with some of the other plate girder criteria originated in the AASHTO Specification.[7] The remaining two shear strength formulas in Appendix G3 are obtained by simply dropping this tension field action term (Formula A-G3-3) or by setting C_v to a maximum value of 1 for shear yielding (Formula A-G3-1). The following will demonstrate that the Formulas in Sect. F2 are merely algebraic simplifications of Appendix G3 formulas:

For $h/t_w \leq 187\sqrt{k/F_{yw}}$ (shear yielding)

$$V_n = 0.6F_{yw}A_w \qquad \text{(F2-1) and (A-G3-1)}$$

where $C_v = 1$

For $187\sqrt{k/F_{yw}} \leq h/t_w \leq 234\sqrt{k/F_{yw}}$

(inelastic buckling)

$$V_n = 0.6F_{yw}A_w \frac{187\sqrt{k/F_{yw}}}{h/t_w} \qquad \text{(F2-2)}$$

$$= 0.6F_{yw}A_wC_v \qquad \text{(A-G3-3)}$$

with $$C_v = \frac{187\sqrt{k/F_{yw}}}{h/t_w} \qquad \text{(A-G3-5)}$$

For $h/t_w > 234\sqrt{k/F_{yw}}$ (elastic buckling)

$$V_n = A_w \frac{26{,}400k}{(h/t_w)^2} \qquad \text{(F2-3)}$$

$$= 0.6F_{yw}A_wC_v \qquad \text{(A-G3-3)}$$

with $$C_v = \frac{44{,}000k}{(h/t_w)^2 F_{yw}} \qquad \text{(A-G3-6)}$$

From the above comparison, it is evident the formulas in Appendix G3 and Sect. F2 are consistent. The flowchart directs you specifically to Appendix G3 for tension field action and to Sect. F2 for no tension field action.

Other Shear Strength Design Aids

Tables 10 and 11 in the LRFD Specification contain values for $\phi_v V_n/A_w$ for plate girders. Tables 10-36 (Fig. 8) and 10-50 do not include the tension field action formula and are based on Sect. F2. For tension field action design, Tables 11-36 (Fig. 9) and 11-50 are applicable and founded on Appendix G3, including the required stiffener areas as a percentage of web area.

The LRFD formula for the required stiffener area,

$$\frac{F_{yw}}{F_{yst}}\left[0.15Dht_w(1-C_v)\frac{V_u}{\phi_v V_n} - 18t_w^2\right] \geq 0.0 \qquad \text{(A-G4-2)}$$

is not directly proportional to D or (F_{yw}/F_{yst}) as it was in the ASD Specification,[8] because of the additional $18t_w^2$ term. The stiffener area percentages tabulated are based on $(V_u/\phi_v V_n) = 1$, $D = 1$, and $(F_{yw}/F_{yst}) = 1$. The table values cannot be directly modified for other cases. They will be more conservative if $V_u < \phi_v V_n$ or $F_{yw} < F_{yst}$, while a less conservative value is tabulated when D is greater than 1 or $F_{yw} > F_{yst}$. The value of D is 1 only for stiffener pairs, otherwise it is 1.8 for single-angle stiffeners and 2.4 for single-plate stiffeners.

FLEXURE-SHEAR INTERACTION

For tension field action design, when $0.6V_n/M_n \leq V_u/M_u \leq V_n/0.75M_n$, an interaction check is necessary. The requirement is:

$$\frac{M_u}{M_n} + 0.625\frac{V_u}{V_n} \leq 1.375\phi \qquad \text{(A-G5-1)}$$

where $\phi = 0.90$, and M_u and V_u are the largest values within the panel (between stiffeners). The values of V_n and M_n are the nominal shear and flexural strengths discussed previously.

BEARING STRENGTH

The need for bearing stiffeners can be determined by checking the bearing strength of the web at unframed girder ends and at concentrated load points. According to the LRFD Specification Section K1, the factored load must not exceed ϕR_n, where ϕR_n is defined for the following criteria:

Local web yielding: $\phi = 1.0$

—at end of member ($\leq d$)

$$R_n = (2.5k + N)\, F_{yw}t_w \qquad \text{(K1-3)}$$

—at concentrated load point

$$R_n = (5k + N)\, F_{yw}t_w \qquad \text{(K1-2)}$$

Web crippling: $\phi = 0.75$

—at end of member ($< d/2$)

$$R_n = 68t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \qquad \text{(K1-5)}$$

—at concentrated load point

$$R_n = 135t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \qquad \text{(K1-4)}$$

Sidesway web buckling $\phi = 0.85$ (see LRFD Spec. Commentary K1.5)

—loaded flange not restrained against rotation and $(d_c/t_w)/(\ell/b_f) < 1.7$

$$R_n = \frac{12{,}000t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{\ell/b_f}\right)^3\right] \qquad \text{(K1-7)}$$

—loaded flange restrained against rotation and $(d_c/t_w)/(\ell/b_f) < 2.3$

$$R_n = \frac{12{,}000t_w^3}{h}\left[1 + 0.4\left(\frac{d_c/t_w}{\ell/b_f}\right)^3\right] \qquad \text{(K1-6)}$$

TABLE 10-36

$\dfrac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Section F2

For 36 ksi Yield-stress Steel, Tension Field Action Not Included

h/t_w	Aspect Ratio *a/h*: Stiffener Spacing to Web Depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Over 3.0
60	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
70	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
80	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	18.9	18.2	17.9	16.9
90	19.4	19.4	19.4	19.4	19.4	19.4	19.4	18.5	17.8	17.2	16.8	16.2	15.9	14.7
100	19.4	19.4	19.4	19.4	19.4	19.2	17.6	16.6	16.0	15.5	14.9	13.8	13.2	11.9
110	19.4	19.4	19.4	19.4	18.4	17.4	16.0	14.8	13.7	12.8	12.3	11.4	10.9	9.8
120	19.4	19.4	19.4	18.1	16.9	16.0	14.0	12.5	11.5	10.8	10.3	9.6	9.2	8.3
130	19.4	19.4	18.2	16.7	15.6	14.1	11.9	10.6	9.8	9.2	8.8	8.2	7.8	7.0
140	19.4	18.8	16.9	15.5	13.5	12.1	10.3	9.2	8.4	7.9	7.6	7.0	6.7	6.1
150	19.4	17.6	15.7	13.5	11.8	10.6	8.9	8.0	7.3	6.9	6.6	6.1	5.9	5.3
160	18.9	16.5	14.1	11.9	10.4	9.3	7.9	7.0	6.5	6.1	5.8	5.4		4.6
170	17.8	15.5	12.5	10.5	9.2	8.2	7.0	6.2	5.7	5.4	5.1			4.1
180	16.8	13.9	11.1	9.4	8.2	7.3	6.2	5.5	5.1	4.8	4.6			3.7
200	14.9	11.2	9.0	7.6	6.6	5.9	5.0	4.5	4.1					3.0
220	12.3	9.3	7.5	6.3	5.5	4.9	4.2							2.5
240	10.3	7.8	6.3	5.3	4.6	4.1								2.1
260	8.8	6.6	5.3	4.5	3.9	3.5								1.8
280	7.6	5.7	4.6	3.9										
300	6.6	5.0	4.0											
320	5.8	4.4												

Figure 9

TABLE 11-36

$\dfrac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix G

For 36 ksi Yield-stress Steel, Tension Field Action Included[b]
(*Italic* values indicate gross area, as percent of ($h \times t_w$) required for pairs of intermediate stiffeners of 36 ksi yield-stress steel with $V_u/\phi V_n = 1.0$.)[a]

h/t_w	Aspect Ratio *a/h*: Stiffener Spacing to Web Depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Over 3.0[c]
60	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
70	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
80	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.1	18.6	18.3	16.9
90	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.0	18.5	18.2	17.8	17.3	16.8	14.7
100	19.4	19.4	19.4	19.4	19.4	19.3	18.6	18.1	17.6	17.2	16.6	15.6	14.9	11.9
110	19.4	19.4	19.4	19.4	19.1	18.7	17.9	17.2	16.3	15.6	15.1	14.0	13.3	9.8
120	19.4	19.4	19.4	19.0	18.5	18.1	17.0	16.0	15.1	14.4	13.9	12.8	12.0	8.3
130	19.4	19.4	19.1	18.6	18.1	17.4	16.1	15.1	14.2	13.5	12.9	11.8	11.0	7.0
140	19.4	19.3	18.7	18.2	17.4	16.6	15.4	14.4	13.5	12.8	12.2	11.0	10.2	6.1
150	19.4	19.0	18.4	17.5	16.7	16.0	14.8	13.8	12.9	12.2	11.6	10.4	9.6	5.3
160	19.3	18.7	17.9	17.0	16.2	15.5	14.3	13.3	12.4	11.7	11.1	9.9		4.6
170	19.1	18.4	17.4	16.6	15.8	15.1	13.9	12.9	12.0	11.3	10.7			4.1
										0.3	*0.4*			
180	18.9	18.0	17.1	16.2	15.5	14.8	13.6	12.6	11.7	11.0	10.4			3.7
							0.2	*0.7*	*1.1*	*1.3*	*1.5*			
200	18.4	17.3	16.4	15.6	14.9	14.2	13.1	12.0	11.2					3.0
				0.1	0.9	*1.4*	*2.1*	*2.5*	*2.8*					
220	17.8	16.9	16.0	15.2	14.5	13.8	12.7							2.5
			1.1	*2.0*	*2.6*	*3.0*	*3.6*							
240	17.4	16.5	15.7	14.9	14.2	13.5								2.1
		1.5	*2.7*	*3.4*	*3.9*	*4.3*								
260	17.1	16.2	15.4	14.6	14.0	13.3								1.8
	1.3	*3.0*	*4.0*	*4.6*	*5.0*	*5.4*								
280	16.8	16.0	15.2	14.4										
	2.7	*4.2*	*5.0*	*5.6*										
300	16.6	15.8	15.0											
	3.9	*5.2*	*5.9*											
320	16.4	15.6												
	4.9	*6.0*												

[a]For area of single-angle and single-plate stiffeners, or when $V_v/\phi V_n < 1.0$, see Formula A-G4-2.
[b]For end-panels and all panels in hybrid and web-tapered plate girders use Table 10-36.
[c]Same as for Table 10-36.
Note: Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

Figure 8

Sidesway web buckling need only be checked when the flanges are subject to concentrated loads and are not restrained against relative movement by stiffeners or lateral bracing.

If any of the appropriate ϕR_n values are exceeded by an end reaction or concentrated load, bearing stiffeners are required.

STIFFENER DESIGN

Bearing Stiffeners

Bearing stiffeners are nearly always required at unframed girder ends and also often at concentrated load points. Where required, stiffeners should be placed in pairs, extending as near to the outer flange edges as possible.[7] Based on this width, a trial thickness can be determined according to the compactness requirements given in Sect. B5. The compressive strength and the surface bearing must be checked, as applicable.

When ϕR_n for web crippling is exceeded, the stiffeners should be designed according to column compression strength Sect. E2. The effective length should be taken as $.75h$ and the cross section should consist of the two stiffeners, plus a $25t_w$ width of the web at interior stiffeners and a $12t_w$ web width at end stiffeners (Sect. K1.8). In addition, the concentrated load should be compared to the design surface bearing strength. According to Sect. J8.1, this is ϕR_n, where

$$R_n = 2.0 F_y A_{pb} \qquad \text{(J8-1)}$$

and ϕ equals 0.75. The projected bearing area A_{pb} is the net area of the stiffener, as only the portions outside the flange-to-web plate welds are considered effective in bearing.[7]

If the compression or outside bearing strength controls over the compactness criterion, the stiffener thickness should be increased and the controlling factor rechecked. Typically, bearing stiffeners extend the full depth of the web with the top of the stiffener bearing on or welded to the compressively-loaded top flange. The exception to this is when the local web yielding criterion controls, and the stiffener "need not extend more than one-half the web depth" (Sect. K1.8).[1]

Intermediate Stiffeners

The design of intermediate stiffeners generally consists of a compactness check (local buckling) and a moment of inertia (stiffness) calculation (Sect. F3 or Appendix G4). The minimum moment of inertia of the transverse stiffener about an axis in the web center for stiffener pairs and about the face in contact with the web plate for single stiffeners[7] is $at_w^3 j$,

where $$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \qquad \text{(A-G4-1)}$$

A minimum stiffener area calculation is also required if the web strength is based on tension field action, due to post-buckling strength considerations. The minimum required stiffener area is:[7]

$$\frac{F_{yw}}{F_{yst}}\left[0.15 Dht_w (1 - C_v)\frac{V_u}{\phi_v V_n} - 18t_w^2\right] \geq 0.0 \qquad \text{(A-G4-2)}$$

This requirement is a result of the truss action analogy for tension field behavior, where the stiffeners resist the compressive forces and the web carries the diagonal tension forces.

The attachment of the shear stiffeners to the flanges is another important consideration. It is not necessary for transverse stiffeners to be in bearing with the tension flange. However, based on previous studies, the distance between the end of the stiffener weld and the near edge of the web-to-flange weld should not be less than $4t_w$ nor exceed $6t_w$ (Fig. 10).[7] On the compression side, the stiffener should be in bearing against the flange. Attachment here is only important if out-of-plane movements in a welded web-to-flange connection may occur.[7] The resulting stiffener length L_{st} is:

$$h - (k - t_f) - (4xt_w) \geq L_{st} \geq h - (k - t_f) - (6xt_w)$$

CONCLUSION

Plate girder design according to LRFD is very similar to the ASD method presented in the 8th Edition *Manual of Steel Construction*.[8] The basic approach is unchanged, with the major changes being the addition of an additional bearing strength formula to check and some new AASHTO criteria for stiffener design.

The LRFD method is facilitated by the flowcharts being introduced in the LRFD Manual. These new design aids demonstrate graphically how the applicable Specification sections interact, and will certainly accelerate the familiarization process for first-time users.

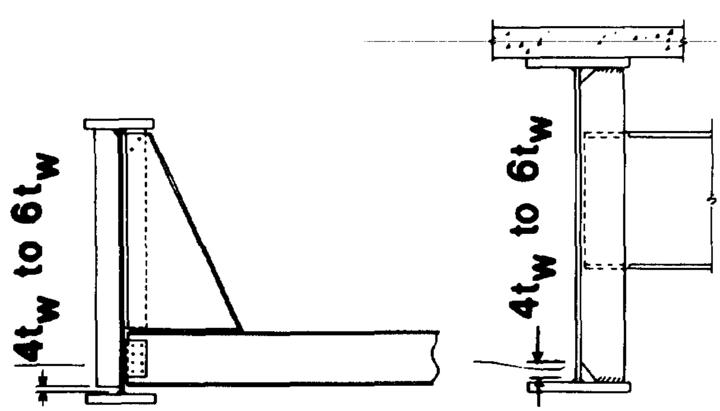

Figure 10

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NOMENCLATURE

A_f	Area of flange, in.^{2}
A_{pb}	Projected bearing area, in.^{2}
A_s	Area of steel cross section, in.^{2}
A_w	Web area, in.^{2}
C_v	Ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material
D	Factor used in stiffener area formula, dependent on the type of transverse stiffeners used
F_{cr}	Critical stress, ksi
F_r	Compressive residual stress in flange, ksi
F_{yf}	Specified minimum yield stress of flange, ksi
F_{yst}	Specified minimum yield stress of stiffener material, ksi
F_{yw}	Specified minimum yield stress of the web, ksi
L_b	Laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or against twist of the cross section, ft
L_{pd}	Limiting laterally unbraced length for plastic analysis, ft
M_n	Nominal flexural strength, kip-ft
M_p	Plastic bending moment, kip-ft
M_r	Limiting buckling moment M_{cr} when $\lambda = \lambda_r$, kip-ft
M_u	Required flexural strength, kip-ft
M_1	Smaller moment at end of unbraced length of beam, kip-in.
N	Length of bearing, in.
R_e	Hybrid girder factor
R_{PG}	Plate girder factor
R_n	Nominal resistance, kips
V_n	Nominal shear strength, kips
V_u	Required shear strength, kips
a	As it applies to plate girders, clear distance between transverse stiffeners
a_r	Ratio of web area to area of *one* compression flange
d_c	Web depth clear of fillets, in.
h	For rolled shapes, clear distance between flanges less the fillet or corner radius; for welded built-up sections, clear distance between flanges, in.
h_c	For rolled shapes, twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius; for welded built-up sections, twice the distance from the neutral axis to the compression flange, in.
ℓ	Largest laterally unbraced length along either flange at the point of load, in.
k	Web plate buckling coefficient in plate girder design
m	Ratio of web yield stress to flange yield stress or critical stress
r_y	Radius of gyration with respect to weak axis, in.
t_f	Flange thickness, in.
t_w	Web thickness, in.
λ_p, λ_r	Limiting slenderness parameters for compact and noncompact elements, respectively
ϕ_b	Resistance factor for flexure, = 0.90
ϕ_v	Resistance factor for shear, = 0.90