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Design recommendations for plate girders, March 1961

K. Basler

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DESIGN RECOMMENDATIONS
FOR PLATE GIRDERS

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FOREWORD

The numerical values are worked out for girders made of A7 steel (F_y = 33 ksi) with assumed values of E = 30,000,000 psi and ν = 0.3. They are supposed to apply to the AISC Specification. The values added in parenthesis apply to the AASHO Specification.

The nomenclature used here conforms to AISC practice. For convenience the standard symbols as applied in all the Lehigh plate girder publications are added to the right of the list.

NOMENCLATURE

AISC		Lehigh Plate Girder Literature
A_f	Area of compression flange	A_f
A_s	Area of transverse stiffener	A_s
A_w	Area of web	A_w
a	Panel length, measured from centerline to centerline of transverse stiffener	a
b	Width of compression flange	$2c$
h	Web depth, measured from border to border of the web plate.	b
C_1	Stress modification coefficient, depending on moment gradient	C_1
c	Distance between flange centroid and neutral axis.	$\bar{y}_c$
e	Distance between the centerline of the end bearing stiffener to the girder end.	e
I	Moment of inertia	I
I_s	Moment of inertia of transverse stiffeners	I_s
ℓ	Effective lateral buckling length	ℓ
M	Bending moment	M
r	Radius of gyration	r
s	Smaller of the two panel dimensions	s
v'	Shear flow between web and transverse stiffener	v'
t	Flange thickness; Web thickness	d, t
f_b	Bending stress	σ
v	Shearing stress	τ

1. BENDING STRENGTH

1.1 General

The subsequently given allowable stresses f_b are based on the assumption that the bending stress is computed as $f_b = Mc/I$, where M denotes the bending moment, c the distance from the centroid of the cross section to the centroid of the flange, and I the moment of inertia. The computation of I shall be based on the gross section, except for cases where the reduction of area of either flange by rivet holes, calculated in accordance with the provisions of Sec. 19 of the AISC Specifications, exceeds 15 percent of the gross flange area. In such a case the excess shall be deducted. (AASHO: the tensile stress shall be computed from the moment of inertia of the entire net section and the compressive stress from the moment of inertia of the entire cross section. However, in calculating the net moment of inertia the gravity axis of the gross section can be used. In determining the net moment of inertia, all holes on each side of the axis shall be deducted.)

1.2 Tension Flange

The highest permissible tension flange stress is $f_b = 20{,}000$ psi (18,000 psi).

1.3 Compression Flange

a) Lateral Buckling: The permissible stress in psi at the most highly stressed section is given by

$$f_b = 20,000 - \frac{0.56}{C_1}\left(\frac{\ell}{r}\right)^2$$

$$\left(f_b = 18,000 - \frac{0.50}{C_1}\left(\frac{\ell}{r}\right)^2\right)$$ in which

ℓ is the lateral buckling length, which is in general the distance between lateral bracing points. If at points of lateral bracing the compression flange is not rigidly held against lateral displacements, such as the case in a through type girder bridge, the effective lateral buckling length can be computed as $\ell = 2.5\sqrt[4]{EIfg}$, in which EI is the lateral rigidity of the compression flange in k - in^2, f is the distance between adjacent frames yielding elastic lateral supports (measured in inches, and g (measured in inches) is the virtual lateral displacement of either leg of the bracing frame at flange elevation, due to a unit load of 1 kip acting at the location and in the direction of the displacement. In no case can ℓ be taken less than f.

C_1 takes into account the influence of moment gradients. It is always conservative to use unity as a value for C_1 since it corresponds to the case where the entire section of ℓ is subjected to the maximum bending moment. With a constant moment gradient the coefficient C_1 can be taken as $C_1 = 1.75 - 1.05\varkappa + 0.3\varkappa^2$, where $\varkappa$ is the ratio of the smaller to the higher end moment of the considered girder section ℓ. C_1 must not be taken greater than 2.3.

r is the radius of gyration, computed as $r = \sqrt{I/A}$, where I is the moment of inertia of the compression flange, taken about the vertical axis of symmetry, and A is the sum of the compression flange area and one sixth of the web area, $A = A_f + (1/6)A_w$.

For plate girders, whose compression flange has the shape of a rectangle, r can be expressed in terms of the flange width b, in which case an alternate form of the lateral buckling equation is:

$$f_b = 20,000 - \frac{1.12}{C_1}\left(6 + \frac{A_w}{A_f}\right)\left(\frac{\ell}{b}\right)^2$$

$$\left(f_b = 18,000 - \frac{1}{C_1}\left(6 + \frac{A_w}{A_f}\right)\left(\frac{\ell}{b}\right)^2\right)$$

If the permissible stress obtained by the foregoing formulas is less than

$$f_b = \frac{12,000,000}{\frac{\ell d}{A_f}} \qquad \text{when } \frac{\ell d}{A_f} > 600$$

$$f_b = 20,000 \qquad \text{when } \frac{\ell d}{A_f} < 600$$

$$\left(\begin{array}{ll} f_b = \dfrac{10,800,000}{\frac{\ell d}{A_f}} & \text{when } \dfrac{\ell d}{A_f} > 600 \\ f_b = 18,000 & \text{when } \dfrac{\ell d}{A_f} < 600 \end{array}\right)$$

the latter shall govern. ℓ is as defined above and d is the depth of the member.

b) Torsional Buckling: If the compression flange is a rectangular plate having a width b and thickness t and the ratio b/t exceeds 12 plus the ratio of lateral buckling length to flange width ℓ/b, an adjusted slenderness ratio ℓ/b, equal to b/t - 12, shall be substituted in the term ℓ/b as a safeguard against torsional buckling in computing permissible compression stress according to the second set of formulas above.

c) <u>Vertical Buckling</u>: In order to prevent the compression flange from buckling into the web, the web depth-to-thickness ratio must not exceed 360 (340) and transverse loads should be introduced through stiffeners.

d) <u>Web Participation</u>: In slender-web girders the participation of the compression portion of the web in carrying the bending moment is less than assumed by ordinary beam theory. An increase of the compression flange stress beyond the value computed from Mc/I will cover this deficiency. To allow for this increase the permissible flange stress, computed as equal to Mc/I, must be reduced by an amount dependent upon the web depth-thickness ratio $\beta = h/t$ and the ratio of web area to compression flange area A_w/A_f.

Up to a web depth-to-thickness ratio

$$\beta_o = 170\sqrt{\frac{20{,}000}{f_b}} \qquad \left(\beta_o = 170\sqrt{\frac{18{,}000}{f_b}}\right)$$

the permissible bending stress f_b as determined by the paragraphs (a) and (b) requires no reduction. If the web slenderness ratio exceeds this limit in the amount of $\Delta\beta = \frac{h}{t} - \beta_o$, the reduction of the bending stress f_b in percent is

$$0.05 \cdot \frac{A_w}{A_f} \cdot \Delta\beta$$

2. SHEAR STRENGTH

2.1 Shear Stress

The unit shear stress v acting in a panel is computed as the shear force divided by the gross cross-sectional area of the web. Permissible shear stresses v are given in Table 1. They depend on the ratio of longitudinal spacing of transverse stiffeners a to web depth h, and on the web depth-to-thickness ratio h/t. The analytical expression from which the table values were obtained are*:

$$v = 11{,}500\left(C + \frac{1 - C}{1.15\sqrt{1+(a/h)^2}}\right) \qquad \text{for } C < 1$$
$$v = 11{,}500\, C \qquad \text{for } 1 < C < 1.13$$

$$\text{with} \quad C = \frac{1400}{(h/t)^2}\,k \qquad \text{when } C < 0.8$$
$$C = \frac{33.5}{h/t}\sqrt{k} \qquad \text{when } C > 0.8$$

$$\text{where} \quad k = 4.00 + \frac{5.34}{(a/h)^2} \qquad \text{when } a/h < 1$$
$$k = 5.34 + \frac{4.00}{(a/h)^2} \qquad \text{when } a/h > 1$$

2.2 Intermediate Stiffeners

a) Strength: When a portion of the shear applied to the girder web is resisted by tension field action, transverse stiffeners must sustain a certain axial force. For all possible values of a/h and h/t these stiffeners will have sufficient strength if their gross area A_s is:

* (AASHO : replace 11,500 by 10 400)

For double sided arrangement (sum of both areas) $A_s = 0.0005\ h^2$

For single sided, equal leg angle $A_s = 0.0009\ h^2$

For single sided, rectangular plate $A_s = 0.0012\ h^2$

If the permissible shear strength is not fully utilized, the required area A_s may be reduced in the same proportion as the maximum computed shear stress v, in either adjacent panel, is smaller than the permissible shear stress v_{all} in the same panel.

b) Stiffness: The moment of inertia of the stiffeners I_s (figured with a common axis at the centerline of web for stiffeners in pairs, and with an axis at the interface between stiffener and web for single stiffeners) must in all cases be:

$$I_s > 0.00000016\ h^4$$

c) Connectors: Using ordinary working stresses, rivets and welds connecting the stiffeners to the web shall be proportioned for the following total shear transfer v' per unit length of transverse stiffener (pounds per inch):

$$v' = 30\ h$$

$$(v' = 27\ h)$$

where h is the length of the stiffeners measured in inches.

d) Details: Transverse stiffeners may be stopped short at the tension flange. The clearance, however, should not exceed 4 times the web thickness ($3\frac{1}{2}$ times the web thickness). The stiffeners must stay the compression flange. In case of stiffeners in pairs simple bearing is sufficient; if single stiffeners are placed at one side only, provisions against the uplift tendency due to torsion in the flange plate must be made.

If lateral bracing is attached to the stiffener, a shear connection between the end of the stiffener and the compression flange should be provided. This should be strong enough to transfer one percent of the maximum total compression flange force. Such connections for shear transfer is not needed when the flange is composed of angles.

2.3 End Stiffeners

Provision must be made at the ends of girders, designed on the basis of tension field action, to resist the horizontal component of the tension field stresses with a rigid means of support. This can be accomplished by limiting the smaller of the two dimensions s of the end panels to not more than

$$s = 11{,}000\ t/\sqrt{v}$$

$$(s = 9000\ t/\sqrt{v})$$

where the shearing stress v in the end panel is expressed in pounds per square inches and s and t in inches. When so proportioned the web in the end panel can support the shear by ordinary beam action without the assistance of a tension field.

The bearing stiffeners over the supports of unframed stiffeners can be reinforced to function as beams spanning between the top and bottom flange to resist the horizontal component of a tension field in the end panel. Extending the web a distance e beyond the centerline of the bearing stiffener and adding a vertical flange plate to produce an H-type profile of approximate depth e, the required area of this flange plate can be computed as

$$A = \frac{h}{e} \left(\frac{V}{160,000} - 750 \frac{ht^3}{s^2} \right)$$

with V expressed in pounds and the linear dimensions e, h, s, and t in inches.

$$\text{AASHO: } A = \frac{h}{e} \left(\frac{V}{144,000} - 550 \frac{ht^3}{s^2} \right)$$

3. INTERACTION

3.1 General

The bending stress f_b and the shear stress v must not exceed the permissible values specified in Part 1 and Part 2, respectively. In case the shear stress exceeds 60% of the allowable shear stress v_{all} and, simultaneously, the bending stress in the highest stressed cross section of a given panel is beyond 15,000 lbs/in^2, (13,500 lbs/in^2) an interaction check is indicated, where the permissible coexistent stresses are determined by the following inequality:

$$f_b < 27{,}000 - 12{,}000 \frac{v}{v_{all}}$$

$$\left(f_b < 24{,}500 - 11{,}000 \frac{v}{v_{all}}\right)$$

3.2 Continuous Girders

Over an interior support of a continuous girder the above given interaction condition must apply to the compression flange stress only and not to the stress in the tension flange.

TABLE I

PERMISSIBLE SHEAR STRESSES IN PLATE GIRDERS

(in kips per square inch)

Slenderness ratios h/t: web depth to web thickness	Aspect ratios 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
Under 70	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0
70	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0
80	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	12.5	12.2	12.0	11.7	11.5	11.1
90	13.0	13.0	13.0	13.0	13.0	13.0	12.2	11.6	11.3	11.2	11.1	10.8	10.7	10.0
100	13.0	13.0	13.0	13.0	12.6	11.8	11.3	11.1	10.8	10.6	10.4	10.1	9.9	8.6
110	13.0	13.0	13.0	12.6	11.4	11.3	10.8	10.5	10.1	9.8	9.6	9.1	8.8	7.1
120	13.0	13.0	12.4	11.4	11.2	10.8	10.3	9.8	9.4	9.0	8.8	8.2	7.9	6.0
130	13.0	13.0	11.5	11.2	10.9	10.5	9.8	9.4	8.8	8.5	8.2	7.6	7.1	5.1
140	13.0	12.2	11.3	11.0	10.6	10.0	9.3	8.9	8.3	8.0	7.6	7.0	6.6	4.4
150	13.0	11.5	11.0	10.7	10.2	9.6	8.9	8.5	8.0	7.6	7.2	6.6	6.1	3.9
160	12.1	11.3	10.8	10.4	9.9	9.3	8.6	8.1	7.6	7.2	6.9	6.3	5.8	3.4
170	11.5	11.1	10.6	10.1	9.6	9.0	8.4	7.9	7.3	6.9	6.6	5.9	5.4	3.0
180	11.4	10.8	10.3	9.9	9.4	8.9	8.1	7.7	7.0	6.7	6.4	5.7	5.2	
200	11.1	10.5	9.9	9.5	9.0	8.5	7.8	7.3	6.7	6.3	6.0	5.2	4.8	
220	10.8	10.2	9.6	9.2	8.7	8.3	7.6	7.0	6.4	6.0	5.7			
240	10.6	10.0	9.4	9.0	8.5	8.0	7.4	6.8	6.2					
260	10.3	9.8	9.2	8.8	8.4	7.9								
280	10.1	9.6	9.0	8.7	8.2									
300	10.0	9.5	8.9	8.5	8.1									
320	9.8	9.3	8.8	8.4										
340	9.7	9.2	8.7	8.3										
360	9.6	9.1	8.6											

Intermediate stiffeners not required