LRFD DESIGN OF PLATE GIRDERS
FOR BUILDINGS

by

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I. INTRODUCTION

A. Problem

Plate girders are built-up beams composed of plate elements—commonly two flange plates and a web plate. They are used when a combination of heavy loads and a long span is such that a standard rolled section is inadequate. They may be built up with bolts or rivets; however, the tendency now is to fasten the flanges to the web by welding. Since the section can be fabricated to any desired geometry, individual elements are designed to result in the most economical section. Plate girders are commonly used in highway and railroad bridges and in building frames. In building plate girder design, generally the AISC Specification(3), which is based on the maximum load carrying capacity of a girder, is used. Recently, a proposed LRFD (Load and Resistance Factor Design) Specification has been issued by AISC(4). The LRFD Specification is the prototype for a new generation of structural steel design codes. This Specification is based upon limit states of strength and serviceability combined with a first-order probability analysis.

In this report the background for the LRFD Specification(4) is discussed, the LRFD Specification provisions for plate girders are summarized and two design examples are presented.

B. Purpose

It is the purpose of this report to illustrate the use of the proposed LRFD Specification(4) for plate girders, and in one
example, to compare the results obtained using these Specifications with those obtained using the current AISC Specification(3).

C. Scope

The design examples in this report are based on the following limitations.

(1) The example problems are for steel plate girders for buildings.

(2) The designs are based on the current AISC Specification(3) and the proposed LRFD Specification(4), respectively.

(3) Only non-hybrid girders are discussed in this report.
II. BACKGROUND

As with most other aspects of steel design, plate girder design procedures are increasingly being based on ultimate strength. Until adoption of the 1961 AISC Specification(2), the basis for plate girder design rules was that elastic buckling should be prevented in plate elements(1). It was thus assumed that either yielding or elastic instability constituted failure. In many cases where plate girder design is based on buckling strength, the existance of post-buckling strength is recognized by using lower factors of safety against web buckling than for the overall strength of the member.

In 1961 design recommendations, which consider post-buckling strength, were introduced for plate girders used in buildings(5,6,7). These are based on the maximum "load-carrying capacity", which includes a considerable reserve load-carrying capacity after the web initially buckles. This reserve capacity has been verified by several large-scale girder tests, and the concept has formed the basis for the AISC Specification since 1961(2).

In the past few years the general limit states approach has been moving toward acceptance by AISC. Termed LRFD (Load and Resistance Factor Design), this general design approach will probably be incorporated in the AISC Specifications in the future. This method is a design procedure that combines the calculation of ultimate or limit states of strength and
serviceability with a probability based approach to safety. Great increases in the use of LRFD will probably occur during the next decade.
III. LRFD DESIGN SPECIFICATIONS

A. Introduction

1. General Approach

The general approach is the result of work by an Advisory Task Force under the direction of T.V. Galambos. Papers by Pinkham and Hansell(10), Galambos and Ravindra(8,9), and Wiesner(11) present the current thinking.

The criteria can be expressed in the following form(10).

\[ \phi R_n \geq \gamma_a \Sigma \gamma_i Q_i \]

The left side of this expression refers to the resistance or capacity of the structure while the right side refers to the load effects on the structure. The resistance side of the expression equals the theoretical or nominal capacity of the member \( R_n \) multiplied by the resistance or undercapacity factor \( \phi \). \( R_n \) can represent moments, shears, axial forces, etc. The resistance factor \( \phi \) is a number less than 1.0 which takes into account the undercapacities present in calculating the theoretical resistance or capacity of a member. Among these uncertainties are such items as variation in material properties (such as yield stress or ultimate tensile stress) and deviations in member thickness, depth, straightness, etc. For plate girders, according to the LRFD Specification, \( \phi \) is 0.9.

On the right side of the equation the sum of the products of the load effects \( Q_i \) and the overload factors \( \gamma_i \) is multiplied by an analysis factor \( \gamma_a \). The subscript \( i \) indicates
load types, such as dead load (DL), live load (LL), wind (W), snow (S), and earthquakes (E). The value of $\gamma_a$ which is larger than 1.0, is selected to estimate the effect of the uncertainties of structural analysis. For example, end connections are frequently treated as either simple (hinged) or rigid (fixed) when actually they are somewhere in between.

2. Load Factors

   The required strength of the structure and its elements must be determined from the appropriate critical combinations (gravity loads only) and the corresponding load factors are:

   \[ 1.4 \, D_n \quad \text{(A1)} \]
   \[ 1.2 \, D_n + 1.6 \, L_n + 0.6 \, (L_r \text{ or } S_n) \quad \text{(A2)} \]

   Where $D_n$, $L_n$, $L_r$ and $S_n$ are nominal load types

   $D_n$ : Dead load due to the self-weight of the structural elements and the permanent features of the structures

   $L_n$ : Live load due to occupancy and moveable equipment

   $L_r$ : Roof live load

   $S_n$ : Snow load

   When wind or earthquake load effects act in the same direction as the dead, live or snow loads, or snow loads act in combinations with live loads, and when wind or earthquake loads act in the opposite direction of the dead load (uplift), those load factors can be investigated in the Specification(4).
3. Limitations

(a). When the web slenderness ratio (ratio of web depth, $h_c$, to thickness, $t_w$) is greater than $970/\sqrt{F_y}$, plate girders shall be distinguished from beams.

(b). The maximum slenderness ratio

For $a/h_c \leq 1.5$

$$\left(\frac{h_c}{t_w}\right)_{\text{max}} = \frac{2.000}{\sqrt{F_y}}$$  \hspace{1cm} \text{(A3)}

For $a/h_c > 1.5$

$$\left(\frac{h_c}{t_w}\right)_{\text{max}} = \frac{14,000}{\sqrt{F_y(F_y+16.5)}}$$  \hspace{1cm} \text{(A4)}

(c). Non-hybrid girders

LRFD Specifications have provisions for hybrid girders and web-tapered girders, but in this report only non-hybrid girders will be discussed, i.e.,

$$F_{yf} = F_{yw} = F_{yst} = F_y$$  \hspace{1cm} \text{(A5)}
B. Design Bending Strength

The design bending strength is \( \phi M_n \), and \( M_n \) is the lowest value obtained according to the limit state of tension-flange yield and buckling.

For tension-flange yield

\[
M_n = S_{xt}R_{pg}F_{yf}
\]  
(B1)

For buckling

\[
M_n = S_{xc}R_{pg}F_{cr}
\]  
(B2)

where

\[
R_{pg} = 1 - 0.0005 \frac{A_w}{A_t} \left[ \frac{h_c}{t_w} - \frac{970}{y_{Fcr}} \right]
\]  
(B3)

The critical stress, \( F_{cr} \), to be used is dependent upon the slenderness parameters \( \lambda \), \( \lambda_p \), \( \lambda_r \) and \( C_{pg} \) as follows:

For \( \lambda \leq \lambda_p \)

\[
F_{cr} = F_{yf}
\]  
(B4)

For \( \lambda_p \leq \lambda \leq \lambda_r \)

\[
F_{cr} = F_{yf} \left[ 1 - \frac{1}{2} \left( \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right]
\]  
(B5)

For \( \lambda \geq \lambda_r \)

\[
F_{cr} = C_{pg}/\lambda^2
\]  
(B6)

In the foregoing, the slenderness parameter must be determined for both the limit state of lateral-torsional buckling and the limit state of flange local buckling and the slenderness parameter which results in the lowest value of \( F_{cr} \) governs.

For the limit state of lateral-torsional buckling:

\[
\lambda = \frac{L}{F_t}
\]  
(B7)

\[
\lambda_p = \frac{146}{\sqrt{F_{yf}}}
\]  
(B8)
\[ \lambda_r = \frac{757.6 E_b}{\sqrt{F_{yt}}} \]  
(B9)

\[ C_{pg} = 286,000 \text{ Gb} \]  
(B10)

For the limit state of flange local buckling:

\[ \lambda = \frac{bf}{2t_t} \]  
(B11)

\[ \lambda_p = \frac{65}{\sqrt{F_{yt}}} \]  
(B12)

\[ \lambda_r = \frac{147}{\sqrt{F_{yt} - 10}} \]  
(B13)

\[ C_{pg} = 11,200 \]  
(B14)
C. Design Shear Strength

The design shear strength is \( \psi V_n \) and the plastic shear strength is \( V_p = 0.6 A_w F_{yw} \). \( V_n \) is determined as follows:

For \( \frac{h_c}{t_w} \leq \frac{187 \sqrt{k}}{F_{yw}} \)

\[ V_n/V_p = 1 \]  \hspace{1cm} (C1)

For \( \frac{h_c}{t_w} > \frac{187 \sqrt{k}}{F_{yw}} \)

\[ V_n/V_p = C_v + \frac{1 - C_v}{1.15 \frac{1}{1 + (a/h_c)^2}} \]  \hspace{1cm} (C2)

except that for end-panels

\[ V_n/V_p = C_v \]  \hspace{1cm} (C3)

The web plate buckling coefficient \( k \) is given as

\[ k = 5 + \frac{5}{(a/h_c)^2} \]  \hspace{1cm} (C4)

The coefficient \( k \) shall be taken as 5.0 if \( a/h_c \) exceeds 3.0 or \( \frac{[260/(h_c/t_w) \frac{F_{yw}}{F_{yw}}}] \). The shear coefficient \( C_v \) is determined as follows:

For \( \frac{187 \sqrt{k}}{F_{yw}} \leq \frac{h_c}{t_w} \leq \frac{234 \sqrt{k}}{F_{yw}} \)

\[ C_v = \frac{187 \sqrt{k}}{h_c/t_w \cdot F_{yw}} \]  \hspace{1cm} (C5)

For \( \frac{h_c}{t_w} > \frac{234 \sqrt{k}}{F_{yw}} \)

\[ C_v = \frac{44,000}{(h_c/t_w)^2 F_{yw}} \]  \hspace{1cm} (C6)
D. Interaction between Bending and Shear

When stiffeners are required and if $V_n/0.75M_n \geq V_u/M_u \geq 0.6V_n/M_n$ then an interaction must be checked as following:

$$\frac{M_u}{M_n} + 0.625 \frac{V_u}{V_n} \leq 1.375\phi$$

(D1)
E. Bearing Stiffeners

Bearing stiffeners shall be placed in pairs at unframed ends of beams and girders. They shall be placed in pairs at points of concentrated load in the interior of beams, girders or columns if the load exceeds the nominal strength $\varphi R_n$ given in Sections K1.2* through K1.6* as applicable.

If the concentrated load, tension or compression, exceeds the criteria for $\varphi R_n$ of Subsections K1.2* or K1.3* respectively, stiffeners need not extend more than one-half the depth of the web except as follows.

If concentrated compressive loads are applied to both flanges and if the load exceeds the compressive strength of the $\varphi R_n$ given in Sections K1.4* or K1.6*, the stiffeners shall be designed as axially compressed members (columns) according to Section E2* with an effective length equal to 0.75$h$ and for a cross section comprised of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of flexural members.

When the load normal to the flange is tensile, the stiffeners shall be welded to the loaded flange. When the load normal to the flange is compressive, the stiffeners shall either bear on or be welded to the loaded flange.

* Sections marked with an asterisk are the sections in the LRFD Specification(4).
F. Transverse Stiffeners

Transverse stiffeners are required in plate girders when \( h_c/t_w > 425/\sqrt{F_y w} \), except that stiffeners may be omitted in those portions of the girders where \( V_u/V_p \leq \alpha C_v \), where \( C_v \) is determined for \( k=5 \).

The moment of inertia \( I_{st} \) of a transverse stiffener about an axis in the web center must be,

\[
I_{st} \geq a t_w^3 j
\]

\[ j = \left( \frac{2.5}{(\alpha/h_c)^2} - 2 \right) \geq 0.5 \]  \hspace{1cm} \text{(F1)}

And the stiffener area \( A_{st} \) must be

\[
A_{st} \geq \left[ 0.15 D A_w (1 - C_v) \frac{V_u}{\delta V_n} - 18 t_w \right]
\]

\[ \text{where } D = 1 \text{ for stiffeners in pairs} \]
\[ = 1.8 \text{ for single angle stiffeners} \]
\[ = 2.4 \text{ for single plate stiffeners} \]

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G. Design Aids

To simplify the designer's work in computing nominal shear strength \( V_n \), the \( V_n/Vp \) values can be plotted as in Fig. 1 and 2 for various values of \( a/h_c \) and \( h_c/t_w \) by using Eq. C1 through C6.

Also, to assist the designer in computing \( A_{st} \), the quantity \( 0.15(1-C_V) \) for various values of \( a/h_c \) and \( h_c/t_w \) can be tabulated as in Table 1. The tabulated value should be multiplied by \( D_{aw}V_u/\bar{V}_n \) and 18\( t_w \) subtracted from it to obtain \( A_{st} \).
IV. DESIGN EXAMPLES

A. EXAMPLE 1

A.1 Problem

1. Reference

Lambert Tall & Others, "Structural Steel Design", Second Ed.
Ronald Press Co. (1974), Ch.8 Plate Girders, Example 8.2

2. Given Conditions

2.1 Use A36 steel
2.2 Headroom depth limit 6 ft.
2.3 Lateral support at ends & at 76 kips concentrated loads (= O)
2.4 Use LRFD Specification
2.5 Loading condition

Original girder loading and supports
A.2 Solution


The loads in the original example are not based on specified DL and LL. Moreover, the load factors for DL and LL are different from each other in the LRFD Spec. If the DL is assumed to be 1/4 of the total and LL is 3/4 of the total, the factored load is obtained by multiplying the working loads of the original example by the numerical values of Eq. (A2),

\[ \frac{1}{4}(1.2) + \frac{3}{4}(1.6) = 1.5 \]

If \( f_d \) be the dead load fraction of the total load, the combined load factor for DL+LL only is,

\[
\text{Combined load factor} = 1.2f_d + 1.6(1-f_d) \quad \text{(Eq. A2)}
\]

\[
= 1.6 - 0.4f_d
\]

Combined load factor and its practical range

The "combined load factor" can vary from 1.6, when all the load is LL, to 1.2, when all the load is DL. The practical range
of \( f_d \) may be, say, from 1/10 to 4/10, and the corresponding range of the "combined load factor" is from 1.56 to 1.44. Thus the "combined load factor" of 1.5 proposed for the LRFD example is in the center of the suggested practical range.

![Girder loading and support for LRFD example](image)

Girder loading and support for LRFD example

2. Shears and Moments

![Shear and moment diagrams](image)

Shear and moment diagrams

3. Section

3.1 Trial Section

flange 2 Pl.'s 3/4 x 20 (\( A_f = 15 \text{ in}^2 \))

web Pl. 5/16 x 70 (\( A_w = 21.875 \text{ in}^2 \))

3.2 Check for width to thickness ratio
flange Pl. : \( b_f/2t_f = 20/(2 \times 0.75) = 13.3 < 95/\sqrt{F_y} = 15.8 \) O.K

web Pl. : \( h_c/t_w = 70/0.312 = 224 \)

\[
< 14,000/\sqrt{F_{yw}(F_{yw}+16.5)} = 322 \quad \text{O.K (Eq. A4)}
\]

3.3 Section Properties

\[ I_x = 46500 \text{ in}^2, \quad S_x = 1300 \text{ in}^3 \]

4. Design Bending Strength

4.1 Buckling

(1) Limit state of flange local buckling

\[
\lambda = \frac{b_f}{2t_f} = 13.33
\]

\[
\lambda_p = \frac{65}{\sqrt{F_{yf}}} = 10.83
\]

\[
\lambda_f = \frac{147}{\sqrt{F_{yf}}-10} = 28.33
\]

Since \( \lambda_p \leq \lambda \leq \lambda_f \),

\[
F_{cr} = 36 \left[ 1 - \frac{1}{2} \left( \frac{13.33 - 10.83}{28.33 - 10.83} \right) \right] = 33.5 \text{ ksi} \quad \text{(Eq. B5)}
\]

(2) Limit state of lateral-torsional buckling

\[
\lambda_p = \frac{146}{\sqrt{F_{yf}}} = 24.33
\]

\[
\lambda_f = \frac{757}{\sqrt{C_b}/\sqrt{F_{yf}}} = 126.17 \text{ C_b}
\]

a. Center Span

\[ \lambda = L/r_T = 14 \times 12/5.18 = 32.43 \]

,where \( L = \) unbraced length = 14 ft

\[ r_T = \sqrt{lt/(A_t + A_w/6)} = 5.18 \text{ in} \]

\[ \lambda_f = 126.17 \sqrt{C_b} = 126.17 \]

,where \( C_b = 1 \)

Since \( \lambda_p \leq \lambda \leq \lambda_f \),

\[
F_{cr} = 36 \left[ 1 - \frac{1}{2} \left( \frac{32.43 - 24.33}{126.17 - 24.33} \right) \right] = 34.57 \text{ ksi} \quad \text{(Eq. B5)}
\]

b. End Spans

\[ \lambda = L/r_T = 17 \times 12/5.18 = 39.38 \]

,where \( L = 17 \text{ ft} \)
\[ \lambda_r = 126.17 \sqrt{C_b} = 166.91 \quad \text{, where } C_b = 1.75 \]

Since \( \lambda_p \leq \lambda \leq \lambda_r \)

\[ F_{cr} = 36 \left[ 1 - \frac{1}{2} \left( \frac{32.43 - 24.33}{126.17 - 24.33} \right) \right] = 34.10 \text{ ksi} \quad \text{(Eq. B5)} \]

The smallest \( F_{cr} \) (flange local buckling) governs.

(3) Rpg

\[ R_{pg} = 1 - 0.0005 \times 21.875/15 \left[ 224 - 970/\sqrt{33.5} \right] = 0.9588 \quad \text{(Eq. B3)} \]

(4) \( M_n \)

\[ M_n = 1300 \times 0.9588 \times 33.5 = 3480 \text{ k-ft} \quad \text{(Eq. B2)} \]

4.2 Tension-flange Yielding

\[ M_n = 1300 \times 0.9588 \times 36 = 3739 \text{ k-ft} \quad \text{(Eq. B1)} \]

\[ \Rightarrow \text{ The smaller } M_n \text{ (Buckling) governs.} \]

4.3 Bending Strength

Design bending strength \( \varphi M_n = 0.9 \times 3480 = 3132 \text{ k-ft} \)

Max. factored bending strength \( M_u = 3094.5 \text{ k-ft} \)

\[ \varphi M_n > M_u \quad \text{O.K} \]

5. Bearing Stiffeners

5.1 Web Crippling (Sect. K1.4*)

(1) At 114 kip load points

\[ \varphi R_n = 54t_w^2 \sqrt{F_y} = 29.89 \text{ kips} < 114 \text{ kips} \]

\[ \Rightarrow \text{ Stiffeners are required, and should be extended at least one-half of the web depth.} \]

(2) At ends

\[ \varphi R_n = 13.44 \text{ kips} < 213 \text{ kips} \]

\[ \Rightarrow \text{ Stiffeners are required, and should be extended at least one-half of the web depth.} \]
5.2 Stiffener Design (Sect. E.2*)

(1) At 114 kips load points

Try 2 Pl.'s 5/16 X 5

width to thickness ratio, \( b/t \leq 95/\sqrt{F_y} \) 0.K

\[
I = \frac{(1/12)(5/16)10^3}{26.04 \text{ in}^4}
\]

\[
A = 2(5/16) + 25(5/16)^2 = 5.56 \text{ in}^2
\]

\[
r = \sqrt{I/A} = 3.116 \text{ in}
\]

\[
\lambda_c = \frac{(KL/r)\sqrt{F_y/\pi^2E}}{0.75 \times 70/3.116 \sqrt{36/\pi^229000}} = 0.189 < \sqrt{2}
\]

\[
F_a = (1-\lambda_c^2/4)F_y/[5/3+3/8(\lambda_c/\sqrt{2})-1/8(\lambda_c/\sqrt{2})^3] = 20.785 \text{ ksi}
\]

\[
F_{cr} = 1.76F_a = 36.58 \text{ ksi} > 36 \text{ ksi} \Rightarrow \text{Use } F_{cr} = 36 \text{ ksi}
\]

\[
P_n = Ag_{Fcr} = 200.2 \text{ kips}
\]

\[
\omega_{P_n} = 0.85 \times 200.2 = 170.17 \text{ kips} > 114 \text{ kips} 0.K
\]

Use 2 Pl.'s 5/16 X 5 bearing both flanges.

----

(2) At ends

Try 2 Pl.'s 1/2 X 8

width to thickness ratio, \( b/t \leq 95/\sqrt{F_y} \) 0.K

\[
I = \frac{(1/12)(1/2)16^3}{171 \text{ in}^4}
\]

\[
A = 2(1/2)(8) + 12(5/16)^2 = 9.2 \text{ in}^2
\]

\[
r = \sqrt{I/A} = 4.31 \text{ in}
\]

\[
\lambda_c = \frac{(KL/r)\sqrt{F_y/\pi^2E}}{0.137 < \sqrt{2}
\]

\[
F_a = 21.06 \text{ ksi}
\]

\[
F_{cr} = 37.06 \text{ ksi} > 36 \text{ ksi} \Rightarrow F_{cr} = 36 \text{ ksi}
\]

20
\[ P_n = 331.2 \text{ kips} \]
\[ \varphi P_n = 281.52 \text{ kips} > 213 \text{ kips} \]

Use 2 Pl.’s 1/2 x 8 bearing both flanges

6. Intermediate Stiffeners

6.1 Check

(1) \[ \frac{h_c}{t_w} = 224 > 425/\sqrt{F_{yw}} = 70.8 \]

\[ \implies \text{Stiffeners are needed.} \]

(2) At midspan

\[ \frac{V_u}{V_p} = \frac{26.25}{472.5} = 0.056 < \varphi C_v = 0.9 \times 0.122 = 0.11 \]

\[ \implies \text{Stiffers are not needed.} \]

(3) At endspans

\[ \frac{V_u}{V_p} = \frac{213}{472.5} = 0.451 > \varphi C_v = 0.11 \]

\[ \implies \text{Stiffeners are needed.} \]

6.2 Spacing

(1) At midspan

\[ \varphi V_n \geq V_u \]

\[ V_n \geq \frac{V_u}{\varphi} = \frac{26.25}{0.9} = 29.17 \text{ kips} \]

\[ V_n/V_p = 29.17/(0.6 \times 21.9 \times 36) = 0.062 \]

From Fig.2, with \[ V_n/V_p = 0.062 \& \frac{h_c}{t_w} = 224 \]

\[ a/h_c = \infty \implies \text{Stiffeners are not needed.} \]

(2) At endspans
\[ V_n \geq \frac{V_u}{\phi} = 213/0.9 = 236.7 \text{ kips} \]

\[ V_n/V_p = 0.5 \]

From Fig. 1 with \( V_n/V_p = 0.5 \) & \( h_c/t_w = 224 \)

Use \( a/h_c = 0.55 \) \[ \therefore a = 38 \text{ in} \]

(3) For span BC

At B, \( V_u = 201.13 \text{ kips} \)

\[ V_n \geq \frac{V_u}{\phi} = 223.5 \]

\[ V_n/V_p = 0.472 \]

From Fig. 2, with \( V_n/V_p = 0.472 \) & \( h_c/t_w = 224 \)

Use \( a/h_c = 1.9 \) \[ \therefore a = 133 \text{ in} \]

\[ \Rightarrow \text{Stiffener is needed at center.} \]

Resulting stiffener arrangement

6.3 Stiffener Design

(1) At points B & G

\[ A_{st} \geq \left[ 0.15 D A_w (1-C_v) V_u/\phi V_n - 18 t_w \right] \]  

(Eq. F2)
where \( D = 2.4 \) (for single plate stiffeners)

\[
V_u = 201.13 \text{ kips}
\]

\[
V_n/V_p = 0.53 \quad (\text{Fig. 1, with } a/h_c = 0.55)
\]

\[
V_n = 250.71 \text{ kips}
\]

\[
0.15(1-C_Y) = 0.0714 \quad (\text{Table 1})
\]

Min. \( A_{st} = [0.0714 \times 2.4 \times 21.9 \times 201.13 / (0.9 \times 250.71) - 18 \times 0.3125] \]

\[
= -2.284 \text{ in}^2
\]

\[
J = [2.5 / (a/h_c)^2 - 2] = [2.5 / 0.55^2 - 2] = 6.264
\]

Min. \( I_{st} = at_w^3j = 38 \times 0.3125^3 \times 6.264 = 7.264 \text{ in}^4 \) \quad (Eq. F1)

Try 1 Pl. 3/8 X 4

- width to thickness ratio, \( b/t < 95/\sqrt{F_y} \quad \text{OK}\)

\[
I = (1/3)(3/8)^4 = 8 \text{ in}^4 > 7.264 \text{ in}^4
\]

\[
\Rightarrow \text{Use 1 Pl. 3/8 X 4, bearing on comp. flange and cut 1 in short of tension flange.}
\]

(2) At points C & F

\[
A_{st} > [0.15DA_{w}(1-C_Y)V_u/V_n - 18t_w] \quad (Eq. F2)
\]

where \( D = 2.4 \)

\[
V_u = 166.19 \text{ kips}
\]

\[
V_n/V_p = 0.65 \quad (\text{Fig. 2, with } a/h_c = 1.186)
\]

\[
V_n = 307.48 \text{ kips}
\]

\[
0.15(1-C_Y) = 0.1188 \quad (\text{Table 1})
\]

Min. \( A_{st} = [0.1188 \times 2.4 \times 21.9 \times 166.19 / (0.9 \times 307.48) - 18 \times 0.3125] \]

\[
= -1.875 \text{ in}^2
\]

\[
J = [2.5 / (a/h_c)^2 - 2] = -0.233 < 0.5 \quad \Rightarrow \text{Use } j=0.5
\]

Min. \( I_{st} = at_w^3j = 83 \times 0.3125^3 \times 0.5 = 1.266 \text{ in}^4 \) \quad (Eq. F1)

Try 1 Pl. 3/16 X 3

- width to thickness ratio, \( b/t < 95/\sqrt{F_y} \quad \text{OK} \)
\[ I = \frac{1}{12} \cdot \frac{3}{4} \cdot 3^3 = 1.687 \text{ in}^4 > 1.266 \text{ in}^4 \quad 0. K \]

\[ \Rightarrow \text{Use 1 Pl. } 3/16 \times 3, \text{ bearing on comp. flange and cut } 1 \text{ in short of tension flange.} \]

7. Interaction between Bending and Shear

7.1 At points B & G

\[ V_u = 201.13 \text{ kips} \]

\[ M_u = 655.7 \text{ k-ft} \]

\[ \frac{V_u}{M_u} = 0.307 \text{ ft}^{-1} \]

\[ V_n = 250.71 \text{ kips (Fig. 1, with } \frac{a}{h_c} = 0.55 \text{ & } h_c/t_w = 224 \text{ )} \]

\[ M_n = 3480 \text{ k-ft (By the procedure of III. B. Design Bending Strength)} \]

\[ \frac{V_n}{0.75M_n} = 0.096 \]

\[ \frac{V_u}{M_u} > \frac{V_n}{0.75M_n} \quad \Rightarrow \text{Check not needed.} \]

7.2 At points of C & F

\[ V_u = 166.19 \text{ kips} \]

\[ M_u = 1942.86 \text{ k-ft} \]

\[ \frac{V_u}{M_u} = 0.086 \text{ ft}^{-1} \]

\[ V_n = 307.48 \text{ kips (Fig. 2, with } \frac{a}{h_c} = 1.186 \text{ & } h_c/t_w = 224 \text{ )} \]

\[ M_n = 3480 \text{ k-ft (By the procedure of III. B. Design Bending Strength)} \]

\[ \frac{V_n}{0.75M_n} = 0.118 \]

\[ 0.6V_n/M_n = 0.053 \]

\[ \frac{V_n}{0.75M_n} > \frac{V_u}{M_u} > 0.6 \frac{V_n}{M_n} \]

\[ \frac{M_u}{M_n} + 0.625 \frac{V_u}{V_n} = 0.896 < 1.375\phi = 1.238 \quad 0. K \]

8. Comment

The results using the LRFD Specification are almost the same as
those obtained using the current the AISC Specification, the only
difference being the saving of one intermediate stiffener at
midspan.
B. EXAMPLE 2

B.1 Given Conditions

1. Use A36 steel
2. No depth limit
3. Lateral supports at all concentrated loads (= 0)
4. Each concentrated load $P = 18$ kips (4 kips-DL, 14 kips-LL)
5. Uniform load $w = 0.3$ kips/ft (Only DL)
6. Use LRFD Specification
7. Loading Condition

![Girder loading and support]
B.2 Solution

1. Factored Loads

Concentrated load = 1.2Dn + 1.6Ln = 1.2(4) + 1.6(14) = 27.2 kips
Uniform load = 1.2(0.3) + 1.6(0) = 0.36 kips/ft  \hfill (Eq. A2)

Factored loading condition

2. Shears and Moments

Shear and moment diagram

3. Section

In order to determine the lightest plate girder section, consider web thicknesses of 1/4 in, 5/16 in, and 3/8 in.
3.1 \( t_w = 1/4 \) in

\( (1) \quad \text{Max. } h_c \)
$$h_c = \frac{2000t_w}{\sqrt{F_y f}} = \frac{2000 \times 0.25}{\sqrt{36}} = 83 \text{ in}$$  \hspace{2cm} \text{(Eq. A3)}

\Rightarrow \text{for } a/h_c = 12 \times 12 / 83 = 1.74 > 1.5 \text{ in} \quad (h_c)_{\text{max}} = 83 \text{ in N.G}

$$h_c = \frac{14,000 t_w}{\sqrt{F_y f (F_y f + 16.5)}} = 80 \text{ in}$$  \hspace{2cm} \text{(Eq. A4)}

\Rightarrow \text{for } a/h_c = 12 \times 12 / 80 = 1.5 > 1.5 \text{ in} \quad (h_c)_{\text{max}} = 80 \text{ in O.K}

(2) Max. \( t_f \)

For \( t_w = 1/4 \text{ in} \), max. fillet weld size = \( 1/4 - 1/16 = 3/16 \text{ in} \) (AISC 1.17.3)

\Rightarrow \text{Max. } t_f = 1/2 \text{ in} \quad \text{(AISC Table 1.17.2.A)}

(3) Max. \( b_f \)

width to thickness ratio, \( b_f / 2 t_f \leq 95 / \sqrt{F_y} = 15.8 \)

Max. \( b_f = 2 t_f \times 15.8 = 2 \times 1/2 \times 15.8 = 15.8 \text{ in} \Rightarrow \text{Use 15 in}

(4) Lightest Section

No acceptable section can be obtained using \( t_w = 1/4 \text{ in} \), since for every value of \( h_c \) the required flange exceeds the width to thickness ratio limit.

3.2 \( t_w = 5/16 \text{ in} \)

(1) Max. \( h_c \)

$$h_c = \frac{2000 t_w}{\sqrt{F_y f}} = 103 \text{ in}$$  \hspace{2cm} \text{(Eq. A3)}

\Rightarrow \text{for } a/h_c = 12 \times 12 / 103 = 1.4 < 1.5 \quad (h_c)_{\text{max}} = 103 \text{ in O.K}

$$h_c = \frac{14,000 t_w}{\sqrt{F_y f (F_y f + 16.5)}} = 80 \text{ in}$$  \hspace{2cm} \text{(Eq. A4)}

\Rightarrow \text{for } a/h_c = 12 \times 12 / 80 = 1.44 < 1.5 \quad (h_c)_{\text{max}} = 100 \text{ in N.G}

(2) Max. \( t_f \)

For \( t_w = 5/16 \text{ in} \), max. fillet weld size = \( 5/16 - 1/16 = 1/4 \text{ in} \) (AISC 1.17.3)

\Rightarrow \text{Max. } t_f = 3/4 \text{ in} \quad \text{(AISC Table 1.17.2.A)}

(3) Max. \( b_f \)

Max. \( b_f = 2 t_f \times 15.8 = 23.7 \text{ in} \Rightarrow \text{Use 23 in}

(4) Lightest Section
If \( h_c = 103 \) in, then
\[
\frac{h_c}{t_w} = 320
\]
\( A_w = 32.2 \) in\(^2\)
\( V_p = 0.6A_wF_{yw} = 696 \) kips
\( V_u/\phi V_p = 0.129 \)

From Fig. 2, with \( h_c/t_w = 330 \) and \( a/h_c = 1.4 \)
\[
\frac{V_n}{V_p} = 0.53 > \frac{V_u}{\phi V_p} = 0.129 \quad \Rightarrow \text{Shear check is O.K}
\]

Req'd \( S = \frac{M_u}{\phi \text{Rpg} F_{cr}} \) \hspace{1cm} (Eq. B2)
\[
= \frac{1702 \times 12}{(0.9 \times 0.7 \times 25)} = 1297 \text{ in}^3
\]

where Rpg & \( F_{cr} \) are assumed values

Req'd \( I = 1297 \times 52.25 = 67756 \) in\(^4\)

Req'd \( I_f = \text{Req'd } I - I_w = 67756 - (1/12)(5/16)103^3 = 39299 \) in\(^4\)

Req'd \( A_f = \frac{I_f}{(2 \times 51.875^2)} = 7.3 \) in\(^2\)

Req'd \( b_f = \frac{A_f}{t_f} = 7.3/0.75 = 9.73 \) in \( \Rightarrow \) 10 in

\( \therefore \) Flange Pl. 3/4 X 10 (\( A_f = 7.5 \) in\(^2\))

Revised Rpg and \( F_{cr} \):

For the limit state of lateral-torsional buckling
\[
\lambda = 65.45 \quad \text{(Eq. B7)}
\]
\[
\lambda_p = 24.33 \quad \text{(Eq. B8)}
\]
\[
\lambda_T = 126.17\sqrt{C_b} \quad \text{(Eq. B9)}
\]

\[
= 166.9 \quad \text{(Assumed } C_b=1.75 \text{ for smallest } F_{cr})
\]

Since \( \lambda_p < \lambda < \lambda_T \quad F_{cr} = 30.8 \) ksi

For the limit state of flange local buckling
\[
\lambda = 6.67 \quad \text{(Eq. B11)}
\]
\[
\lambda_p = 10.83 \quad \text{(Eq. B12)}
\]

Since \( \lambda_p > \lambda \quad F_{cr} = 36 \) ksi

New \( \text{Rpg} = 1 - 0.0005 \times 32.2 / 7.5 \left[ 330 - 970 / \sqrt{30.8} \right] = 0.667 \)
S = 1102 in³
I = 576.8 in⁴
If = I - Iw = 29161 in⁴

Req'd Af = 5.42 in²

Af = 7.5 in² > Req'd Af = 5.42 in²

Try flange Pl. 3/4 x 9 (Af = 6.75 in²)

\[ F_{cr} = 36 \left[ 1 - \frac{1}{2} \left( \frac{74.23 - 24.33}{166.91 - 24.33} \right) \right] = 29.17 \text{ ksi} \]  \[(\text{Eq. B5})\]

F_{cr} = 36 \text{ ksi} \quad \text{(Eq. B4)}

Rpg = 0.638 \quad \text{(Eq. B3)}

S = 1197 in³
I = 62529 in⁴
If = 34072 in⁴

Req'd Af = 6.33 in²

Af = 6.75 in² > Req'd Af = 6.33 in²

\[ \therefore \text{Use flange plate Pl. 3/4 x 9, web Pl. 5/16 x 59} \]

width to thickness ratio \( b_f/2t_f = 6 < 95/\sqrt{F_y} \) \quad \text{O.K}

In this manner, the lightest section can be found for every \( h_C \). As a result, when the \( h_C \) is 59 inches and \( b_f \) is 12 inches the lightest section was obtained by a computer program (Appendix C). Each section is plotted in Fig. 3.

\[ 3.3 \ tw = 3/8 \text{ in} \]

By the procedure of 3.1 and 3.2, when \( h_C \) is 58 inches and \( b_f \) is 10 inches, the lightest section is obtained from the computer program.
Use flange Pl. 7/8 x 10, web Pl. 3/8 x 58

Finally, when \( t_w \) is 5/16 inches the section is lighter than when \( t_w \) is 3/8 inches.

3.4 Check Shear

\[
\frac{h_c}{t_w} = 188.8 \\
A_w = 18.44 \text{ in}^2 \\
V_p = 0.6 \times 18.44 \times 36 = 398.3 \text{ kips} \\
V_u/\phi V_p = 0.226
\]

From the Fig. 2, with \( \frac{h_c}{t_w} = 188.8 \) & \( a/h_c = 2.44 \)

\[
V_n/\phi V_p = 0.45 > V_u/\phi V_p = 0.226 \quad \text{0.K}
\]

3.5 Section Properties

flange 2 Pl.’s 3/4 X 12 (\( A_f = 9 \text{ in}^2 \))

web Pl. 5/16 X 59 (\( A_w = 18.44 \text{ in}^2 \))

\( I_x = 21414 \text{ In}^4, \quad S_x = 708 \text{ in}^3 \)

4. Design Bending Strength

4.1 Buckling

(1) Limit state of flange local buckling

\[
\lambda = \frac{b_f}{2t_f} = 8 \\
\lambda_p = \frac{65}{\sqrt{F_y f}} = 10.83 \\
\text{Since } \lambda_p > \lambda \quad F_{cr} = 36 \text{ ksi} \quad \text{Eq. B4}
\]

(2) Limit State of lateral-torsional buckling

\[
\lambda_p = \frac{146}{F_y f} = 24.33 \\
\lambda = \frac{L}{r_T} = 48.16 \quad \text{where } r_T = 2.99
\]

a. Span AB and FG

\[
\lambda_T = 126.17 \sqrt{C_D} = 126.17 \sqrt{1.75} = 166.91 \\
\text{Since } \lambda_p < \lambda < \lambda_T \quad F_{cr} = 32.99 \text{ ksi} \quad \text{Eq. B5}
\]

b. Span BC and EF
\[ C_b = 1.75 + 1.05 \left( \frac{M_1}{M_2} \right) + 0.3\left( \frac{M_1}{M_2} \right)^2 > 2.3 \]

\[ \therefore C_b = 2.3 \]

\[ \lambda_r = 126.17 \sqrt{2.3} = 191.35 \]

Since \( \lambda_p < \lambda < \lambda_r \)

\[ F_{cr} = 33.43 \text{ ksi} \quad \text{(Eq. B5)} \]

\( c. \) Span CD and DE

\[ C_b = 2.3 \]

\[ \lambda_r = 191.35 \]

Since \( \lambda_p < \lambda < \lambda_r \)

\[ F_{cr} = 33.43 \text{ ksi} \quad \text{(Eq. B5)} \]

(3) Rpg

\[ R_{pg} = 1 - 0.0005 \times 18.44 / 9(188.8 - 970/\sqrt{32.99}) = 0.9796 \quad \text{(Eq. B3)} \]

(4) \( M_n \)

\[ M_n = 708 \times 0.9796 \times 32.99 = 1907 \text{ kip-ft} \quad \text{(Eq. B2)} \]

4.2 Tension-flange yielding

\[ M_n = 708 \times 0.9796 \times 36 = 2081 \text{ k-ft} \quad \text{(Eq. B1)} \]

4.3 Bending Strength

Design bending strength \( \varphi M_n = 0.9 \times 1907 = 1716 \text{ k-ft} \)

Max. factored bending strength \( M_n = 1702.8 \text{ k-ft} \)

\[ \varphi M_n > M_u \quad \text{O.K} \]

5. Bearing Stiffeners

5.1 Web crippling (Sect. K1.4*)

(1) At each concentrated load point

\[ \varphi R_n = 54 t_w^2 \sqrt{F_y} = 26.89 \text{ kips} < 27.2 \text{ kips} \]

\[ \Rightarrow \text{Stiffeners are required, and should be extended at least one-half of the web depth.} \]

(2) At ends

\[ \varphi R_n = 13.44 \text{ kips} < 80.96 \text{ kips} \]
Stiffeners are required, and should be extended at least one-half of the web depth.

5.2 Stiffener Design (Sect. E.2")

(1) At each concentrated load point

Try 2 Pl.'s 3/16 X 2

\[ \text{width to thickness ratio, } b/t = 95 < \sqrt{F_y} \]  
\[ I = \frac{1}{12}(3/16)4^3 = 1 \]  
\[ A = 2 \times 0.375 + 25 \times (3/16)^2 = 1.619 \]  
\[ r = \sqrt{I/A} = 0.784 \]  
\[ \lambda_c = (0.75 \times 70 / 0.784) \sqrt{36 / \pi^2 29000} = 0.633 < \sqrt{2} \]  
\[ F_a = (1 - 0.633^2 / 4)36 / [(5 / 3 + 3 / 8)(0.633 / \sqrt{2}) - (0.633 / \sqrt{2})^3 / 8] \]  
\[ = 15.26 \text{ ksi} \]  
\[ F_{cr} = 1.76 \times 15.16 = 28.56 \text{ ksi} \]  
\[ P_n = 1.629 \times 26.86 = 43.75 \text{ kips} \]  
\[ \varphi P_n = 0.85 \times 43.75 = 37.19 \text{ kips} > 27.2 \text{ kips} \]  

Use 2 Pl.'s 3/16 X 2, bearing both flanges.

(2) At ends

Try 2 Pl.'s 1/4 X 4

\[ \text{width to thickness ratio, } b/t \neq 95 / \sqrt{F_y} \]  
\[ I = 10.67 \text{ in}^4 \]  
\[ A = 2 \times 1 + 12(5/16)^2 = 3.17 \text{ in}^2 \]  
\[ r = 1.83 \]  
\[ \lambda_c = 0.271 < \sqrt{2} \]
Fa = 20.33 ksi
Fcr = 35.78 ksi
Pn = 113.42 kips
\( \varphi P_n = 96.4 \text{ kips} > 80.96 \text{ kips} \)

Use 2 Pl’s 1/4 x 4, bearing both flanges.

5. Intermediate Stiffeners

6.1 Check

1) \( h_c/t_w = 188.8 > 425/\sqrt{Fy_w} = 70.8 \)

   \( \Rightarrow \) Stiffeners are needed.

2) At span AB and FG

   \( V_u/V_p = 0.203 > \varphi C_v = 0.154 \)

   \( \Rightarrow \) Stiffeners are needed.

3) At span BC and EF

   \( V_u/V_p = 0.124 < \varphi C_v = 0.154 \)

   \( \Rightarrow \) Stiffeners are not needed.

4) At span CD and DE

   \( V_u/V_p = 0.045 < \varphi C_v = 0.154 \)

   \( \Rightarrow \) Stiffeners are not needed.

6.2 Spacing

1) At span AB and EF

   \( V_n \geq V_u/\varphi = 80.96/0.9 = 89.96 \text{ kips} \)

   \( V_n/V_p = 89.96/(0.6 \times 18.44 \times 36) = 0.226 \)
From Fig.1, with $V_n/V_p = 0.226$ and $h_c/t_w = 188.8$

$$a/h_c = 1.7 \quad \therefore a = 100.3 \text{ in}$$

$\implies$ Stiffeners are needed at center.

(2) At span BC and EF

$$V_n \leq V_u/\phi = 49.44/0.9 = 54.93 \text{ kips}$$

$$V_n/V_p = 0.138$$

From Fig.2, with $V_n/V_p = 0.138$ and $h_c/t_w = 188.8$

$$a/h_c = \infty \implies \text{Stiffeners are not needed.}$$

(3) At span CD and DE

$$V_n \leq V_u/\phi = 17.92/0.9 = 19.91 \text{ kips}$$

$$V_n/V_p = 0.05$$

From Fig.2, with $V_n/V_p = 0.05$ and $h_c/t_w = 188.8$

$$a/h_c = \infty \implies \text{Stiffeners are not needed.}$$

Resulting stiffener arrangement

6.3 Design

$$A_{st} = [ \cdot 0.15 DA_w (1 - C_v) V_u/\phi V_n - 18 t_w ]$$

Equation F2

where $D = 2.4$ (for single plate stiffeners)

$V_u = 78.8$ kips

$V_n/V_p = 0.21$ (Fig. 1, with $a/h_c = 1.22$)

$\therefore V_n = 83.64$ kips

$0.15(1-C_v) = 0.107$
Min. \( A_{st} = \left[ 0.107 \times 2.4 \times 18.44 \times 78.8 / (0.9 \times 83.64) - 18 \times 0.3125 \right] \)
\[ = -0.67 \text{ in}^2 \]

\( j = \left( 2.5 / (a/h_c)^2 - 2 \right) = -0.32 < 0.5 \quad \text{==> Use } j=0.5 \)

Min. \( I_{st} = a t_w^3 j = 72 \times 0.3125^3 \times 0.5 = 1.1 \text{ in}^4 \)

Try 1 Pl. 3/16 X 3

width to thickness ratio, \( b/t = 95/\sqrt{F_y} \) 0.K

\( I = (1/3)(3/16)^3 = 1.69 \text{ in}^4 > 1.1 \text{ in}^4 \)

\( \text{==> Use 1 Pl. 3/16 X 3, bearing on comp. flange and cut 1 in short of tension flange.} \)

7. Interaction between Bending and Shear

At points \( A' \) and \( G' \)

\( V_u = 78.8 \text{ kips} \)

\( M_u = 479.28 \text{ k-ft} \)

\( V_u/M_u = 0.164 \text{ ft}^{-1} \)

\( V_n = 83.64 \text{ kips (Fig. 1, with } a/h_c=1.22 \text{ and } h_c/t_w=188.8 \) \)

\( M_n = 1907 \text{ k-ft (By the procedure of III. B. Design Bending Strength)} \)

\( V_n/0.75M_n = 0.058 \)

\( \text{Since } V_u/M_u > V_n/0.75M_n \quad \text{==> Check not needed.} \)

8. Comment

This example provides an optimal design solution to assure the lightest section at the first step of the design procedure.
V. SUMMARY AND CONCLUSION

The proposed LRFD Specification for plate girders for buildings has been summarized and through two design examples the use of this Specification has been illustrated. To simplify design calculations, two design aids have been developed for shear strength calculations, and one aid developed for intermediate stiffener design. Also a computer program was developed to determine the lightest cross section.

The results obtained using the proposed LRFD Specification are almost the same as those based on the current AISC Specification, the only difference being the saving of one intermediate stiffener in the LRFD case. However, based on both design examples, it is concluded that the design procedure of the proposed LRFD Specification is a little simpler than that of the current AISC Specification.
VI. ACKNOWLEDGEMENT

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VII. APPENDICES

A. REFERENCES

B. Notation

A  = Cross-sectional area, (in.²)
Af  = Flange area, (in.²)
Ag  = Gross area, (in.²)
Aw  = Web area, (in.²)
Cb  = Equivalent moment factor
   \[ C_b = 1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 2.3 \]
   where \( M_1 \) is the smaller and \( M_2 \) the larger end-moment in
   the unbraced segment of the beam; \( M_1/M_2 \) is positive when
   the moments cause reverse curvature.

Dn  = Dead load due to the self-weight of the structural elements
      and the permanent features on the structure

E  = Modulus of elasticity of steel (=29,000 ksi)

Fcr = Critical stress, (ksi)

Fy  = Specified yield stress of the type of steel being used,
      (ksi)

Fyf = Yield strength of the flange, (ksi)

Fyw = Yield strength of the web, (ksi)

I  = Moment of inertia, (in.⁴)

Ix  = Moment of inertia about major axis, (in.⁴)

K  = Effective length factor

L  = Unbraced length of member, (in.)

Ln  = Live load due to occupancy

Mn  = Nominal bending strength, (kip-in.)

Mu  = Required bending strength, (kip-in.)
\( P_n \) = Nominal axial strength, (kips)
\( R_n \) = Nominal resistance
\( R_{pg} \) = Plate girder reduction factor
\( S \) = Elastic section modulus, (in.\(^3\))
\( S_n \) = Snow load
\( S_x \) = Section modulus about major axis, (in.\(^3\))
\( S_{xc} \) = Section modulus referred to compression flange, (in.\(^3\))
\( S_{xt} \) = Section modulus referred to tension flange, (in.\(^3\))
\( V_n \) = Nominal shear strength, (kips)
\( V_p \) = Plastic shear strength \( = 0.6A_wF_yw \), (kips)
\( V_u \) = Required shear strength, (kips)
\( W_n \) = Wind load
\( a \) = Clear distance between transverse stiffeners, (in.)
\( b \) = Compression element width, (in.)
\( b_f \) = Flange width, (in.)
\( d \) = Overall depth of member, (in.)
\( d_C \) = Web depth clear of fillets, (in.)
\( h \) = Web depth, (in.)
\( h_C \) = Twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius, (in.)
\( J \) = Factor defined by formula (E1)
\( l \) = Largest laterally unbraced length along either flange at the point of load, (in.)
\( r \) = Radius of gyration, (in.)
\( r_T \) = Radius of gyration of compression flange plus one third of
the compression portion of the web, (in.)

\( t_f \) = Flange thickness, (in.)

\( t_w \) = Web thickness, (in.)

\( \phi \) = Resistance Factor

\( \lambda_c \) = Column slenderness parameter
C. Computer Program and Output

10 REM **********************************************************************
20 REM **** THIS PROGRAM IS TO FIND THE LIGHTEST SECTION OF THE PLATE-*****
30 REM **** GIRDER FOR GIVEN TW AND TF. REFER TO IV. A. EXAMPLE 2. *********
40 REM **********************************************************************
50 DIM AF(20), RAF(20), X(2,20), XP(2,20), XR(2,20), CPB(2,20), F(2), RT(20), BF(20)
60 PRINT "WEB THICKNESS, TW=";
70 INPUT TW
80 LPRINT "        TW=";TW
90 PRINT "FLANGE THICKNESS, TF=";
100 INPUT TF
110 LPRINT "        TF=";TF
120 PRINT "REQUIRED BENDING STRENGTH, MU=? K-FT";
130 INPUT MU
140 LPRINT "        MU=";MU
150 PRINT "CB=";
160 INPUT CB
170 LPRINT "        CB=";CB
180 PRINT "RPG=";
190 INPUT RPG
200 LPRINT "        RPG=";RPG
210 PRINT "CRITICAL STRESS, FCR=? KSI";
220 INPUT FCR
230 LPRINT "        FCR=";FCR
240 PRINT "LATERALLY UNBRACED LENGTH, L=? FEET";
250 INPUT L
260 LPRINT "        L=? FEET";L
270 PRINT "REQUIRED SHEAR STRENGTH, VU=? KIPS";
280 INPUT VU
290 LPRINT "        VU=";VU
300 LPRINT:PRINT
310 LPRINT "              FLANGE FLANGE FLANGE TOTAL SECTION"
320 LPRINT "WEB DEPTH WIDTH AREA Req'D FLANGE AREA"
330 LPRINT "-------------------------------------------------
340 MAX=INT(332*TW)
350 MIN=INT(VU/(19.44*TW))+1
360 FOR HC=MAX TO MIN STEP-1
370 LPRINT USING "    HC####";HC,
380 FOR I=1 TO 20
390 AS=12*MU/(.9*RPG*FCR)
400 AI=AS*(HC/2+TF)
410 IIF=AI-(1/12)*TW*HC^3
420 RAF(I)=IIF/(2*(HC/2+TF)^2)
430 IF I=1 THEN 460
440 BF(I)=INT(RAF(I)/TF)+1
450 IF I=1 THEN 460
460 IF AF(I-1)*RAF(I) THEN BF(I)=BF(I-1)-1 ELSE BF(I)=BF(I-1)+1
470 IF BF(I)=BF(I-2) THEN 510
480 WT=BF(I)/(2*TF)
490 IF WT>15.8 THEN 770
500 AF(I)=TF*BF(I);GOTO 560
510 LPRINT "    BF=";BF(I-2);
520 LPRINT USING "    AF####";AF(I-2),
530 LPRINT USING "  RAFFF###.###";RAF(I-1),
540 ATT=AW+2*AF(I-2)
550 LPRINT "   ATT=";ATT;GOTO 760
560 IY=(1/12)*TF*BF(I)^3
570 AW=TW*HC
580 RT(I)=IY/(AF(I)+AW/6)
590 XI(I,1)=12*L/SQR(RT(I))
600 XP(I,1)=24.33
610 XR(I,1)=126.17*SQR(CB)
620 CPG(I,1)=286*1000*CB
630 X(2,1)=BF(I)/(2*TF)
640 XP(2,1)=10.83
650 XR(2,1)=28.83
660 CPG(2,1)=11200
670 FOR K=1 TO 2
680 IF X(K,I)<XP(K,I) THEN 590 ELSE 700
690 IF X(K,I)<XR(K,I) THEN F(K)=CPG(K,I)/X(K,I)^2;GOTO 720
700 IF X(K,I)<XR(K,I) THEN F(K)=36*(1-5*(X(K,I)-XP(K,I))/(XR(K,I)-XP(K,I)))
710 NEXT K
720 IF F(1)<F(2) THEN FCR=F(1) ELSE FCR=F(2)
730 RPG=1-.0005*AW*(HC/TW-970/SQR(FCR))/AF(I)
740 NEXT I
750 NEXT HC
760 LPRINT " ===== OVER WIDTH TO THICKNESS RATIO !!!"
780 END
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MU = 1702
CB = 1.75
RPG = .9
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HC = 27 ===> OVER WIDTH TO THICKNESS RATIO !!!
Fig. 1 Shear strength without tension field action
Fig. 2 Shear strength with tension field action

$F_{yw} = 36 \text{ ksi}$
Fig. 3 Result of computer output (Example 2)
LRFD DESIGN OF PLATE GIRDERs FOR BUILDINGS

by

HYOSEOP HAN

B.E., Hanyang University, Korea, 1984

AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1985
ABSTRACT

In this report provisions for the design of plate girders for buildings using the proposed LRFD (Load and Resistance Factor Design) Specification, which is based on the ultimate strength design concept, are summarized. The use of these provisions is demonstrated with two design examples. In one design example, the results are compared with those obtained using the current AISC Specification, which is based on the allowable strength design concept. It is concluded that the two design approaches yield approximately the same results, and that the LRFD approach is a little simpler to use.