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 serviceabilty 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(ϕ). R_n can represent moments, shears, axial forces, etc. The resistance factor ϕ 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, ϕ 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 (γ_i) is multiplied by an analysis factor γ_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 γ_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 :

$$1.4\ D_n \tag{A1}$$

$$1.2\ D_n + 1.6\ L_n + 0.6\ (L_r \text{ or } S_n) \tag{A2}$$

,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 $970/\sqrt{Fy}$, plate girders shall be distinguished from beams.

(b). The maximum slenderness ratio

For $a/h_c \leq 1.5$

$$\left(\frac{h_c}{t_w}\right)_{max} = \frac{2{,}000}{\sqrt{F_{yf}}} \tag{A3}$$

For $a/h_c > 1.5$

$$\left(\frac{h_c}{t_w}\right)_{max} = \frac{14{,}000}{\sqrt{F_{yf}\left(F_{yf}+16.5\right)}} \tag{A4}$$

(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.,

$$Fy_f = Fy_w = Fy_{st} = Fy \tag{A5}$$

B. Design Bending Strength

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

For tension-flange yield

$$M_n = S_{xt} R_{pg} F_{yf} \tag{B1}$$

For buckling

$$M_n = S_{xc} R_{pg} F_{cr} \tag{B2}$$

,where

$$R_{pg} = 1-0.0005 \frac{A_w}{A_f} \left[\frac{h_c}{t_w} - \frac{970}{\sqrt{F_{cr}}} \right] \tag{B3}$$

The critical stress, F_{cr}, to be used is dependent upon the slenderness parameters λ, λ_p, λ_r and C_{pg} as follows:

For $\lambda \leq \lambda_p$ $$F_{cr} = F_{yf} \tag{B4}$$

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

For $\lambda \geq \lambda_r$ $$F_{cr} = C_{pg}/\lambda^2 \tag{B6}$$

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:

$$\lambda = \frac{L}{r_T} \tag{B7}$$

$$\lambda_p = \frac{146}{\sqrt{F_{yf}}} \tag{B8}$$

$$\lambda_r = \frac{757\sqrt{C_b}}{\sqrt{F_{yf}}} \tag{B9}$$

$$C_{pg} = 286{,}000\ C_b \tag{B10}$$

For the limit state of flange local buckling:

$$\lambda = \frac{b_f}{2t_f} \tag{B11}$$

$$\lambda_p = \frac{65}{\sqrt{F_{yf}}} \tag{B12}$$

$$\lambda_r = \frac{147}{\sqrt{F_{yf} - 10}} \tag{B13}$$

$$C_{pg} = 11{,}200 \tag{B14}$$

C. Design Shear Strength

The design shear strength is ϕV_n and the plastic shear strength is $V_p = 0.6 A_w F_{yw}$. V_n is determined as follows:

For $\frac{h_c}{t_w} \le \frac{187\sqrt{k}}{\sqrt{F_{yw}}}$

$$V_n/V_p = 1 \tag{C1}$$

For $\frac{h_c}{t_w} \ge \frac{187\sqrt{k}}{F_{yw}}$

$$V_n/V_p = C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h_c)^2}} \tag{C2}$$

except that for end-panels

$$V_n/V_p = C_v \tag{C3}$$

The web plate buckling coefficient k is given as

$$k = 5 + \frac{5}{(a/h_c)^2} \tag{C4}$$

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

For $\frac{187\sqrt{k}}{F_{yw}} \le \frac{h_c}{t_w} \le \frac{234\sqrt{k}}{F_{yw}}$

$$C_v = \frac{187\sqrt{k}}{h_c/t_w\sqrt{F_{yw}}} \tag{C5}$$

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

$$C_v = \frac{44{,}000}{(h_c/t_w)^2 F_{yw}} \tag{C6}$$

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:

$$\frac{M_u}{M_n} + 0.625\frac{V_u}{V_n} \leq 1.375\phi \tag{D1}$$

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 ϕR_n given in Sections K1.2* through K1.6* as applicable.

If the concentrated load, tension or compression, exceeds the criteria for ϕR_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 ϕR_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 0.75h 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 $h_c/t_w > 425/\sqrt{F_{yw}}$, except that stiffeners may be omitted in those portions of the girders where $V_u/V_p \leq \phi C_v$, 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,

$$I_{st} \geq a t_w^3 J \tag{F1}$$

$$,\text{where} \quad J = \left[\frac{2.5}{(a/h_c)^2} - 2 \right] \geq 0.5$$

And the stiffener area Ast must be

$$A_{st} \geq \left[0.15\ D A_w (1 - C_v) \frac{V_u}{\phi V_n} - 18 t_w \right] \tag{F2}$$

,where D = 1 for stiffeners in pairs

= 1.8 for single angle stiffeners

= 2.4 for single plate stiffeners

G. Design Aids

To simplify the designer's work in computing nominal shear strength V_n, the V_n/V_p 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 $DA_wV_u/\phi V_n$ and $18t_w$ 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

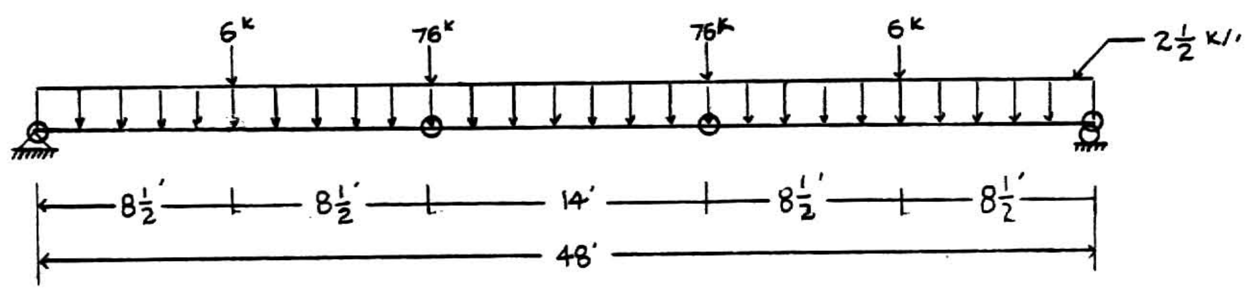
2.1 Use A36 steel

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

1. Loads for LRFD Spec.

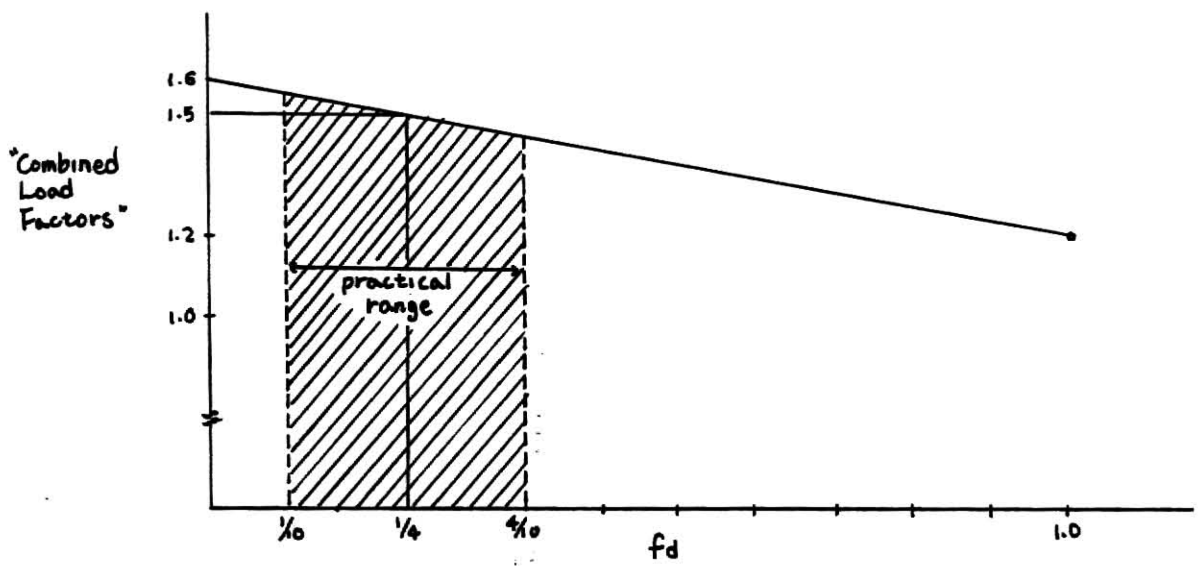
The the loads in the original example are not based on specified DL and LL. Moreover the load factors for DL and LL are different from each other in the LRFD Spec.. If the DL is assumed to be 1/4 of the total and LL is 3/4 of the total, the factored load is obtained by multiplying the working loads of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

$$1/4(1.2) + 3/4(1.6) = 1.5$$

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

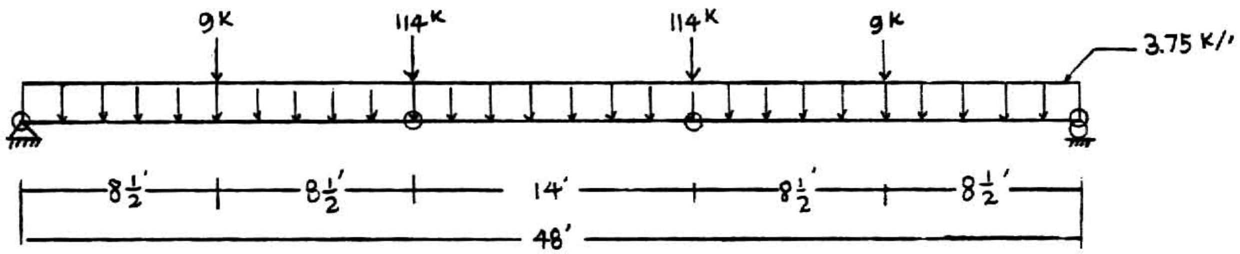
$$\text{Combined load factor} = 1.2f_d + 1.6(1-f_d) \qquad \text{(Eq.A2)}$$

$$= 1.6 - 0.4f_d$$



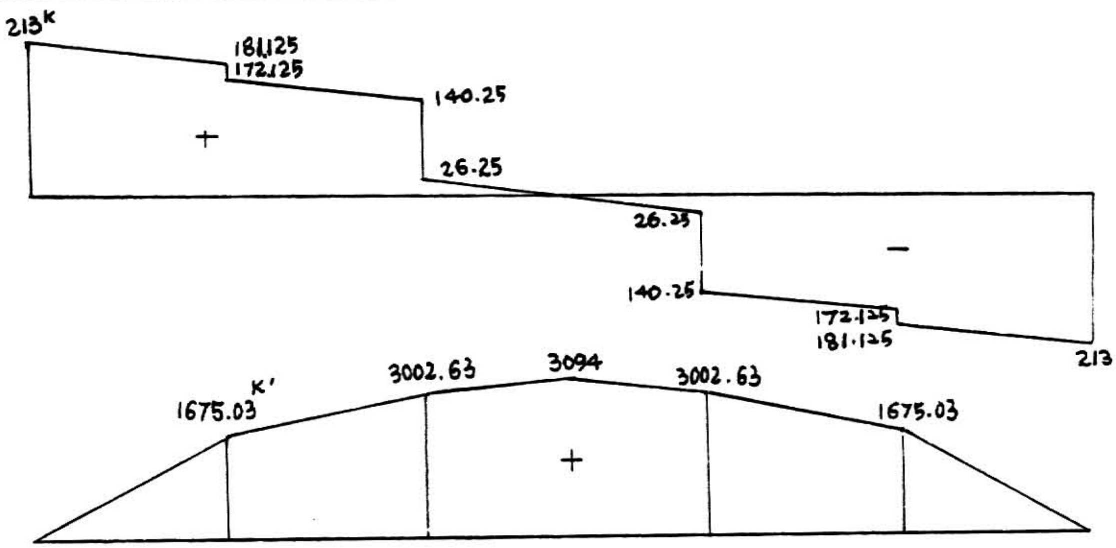
Combined load factor and its practical range

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

2. Shears and Moments



Shear and moment diagrams

3. Section

3.1 Trial Section

flange 2 Pl.'s 3/4 X 20 (A_f = 15 in^2)

web Pl. 5/16 X 70 (A_w = 21.875 in^2)

3.2 Check for width to thickness ratio

flange Pl. : $b_f/2t_f$ = 20/(2X0.75) = 13.3 < $95/\sqrt{Fy}$ = 15.8 O.K

web Pl. : h_c/t_w = 70/0.312 = 224

< $14{,}000/\sqrt{Fy_w(Fy_w+16.5)}$ = 322 O.K (Eq.A4)

3.3 Section Properties

I_x = 46500 in^2, S_x = 1300 in^3

4. Design Bending Strength

4.1 Buckling

(1) Limit state of flange local buckling

$\lambda = b_f/2t_f = 13.33$

$\lambda_p = 65/\sqrt{Fy_f} = 10.83$

$\lambda_r = 147/\sqrt{Fy_f - 10} = 28.33$

Since $\lambda_p \leq \lambda \leq \lambda_r$

$$F_{cr} = 36\left[1 - \frac{1}{2}\left(\frac{13.33 - 10.83}{28.83 - 10.83}\right)\right] = 33.5 \text{ ksi} \quad \text{(Eq.B5)}$$

(2) Limit state of lateral-torsional buckling

$\lambda_p = 146/\sqrt{Fy_f} = 24.33$

$\lambda_r = 757\sqrt{C_b}/\sqrt{Fy_f} = 126.17\ C_b$

a. Center Span

$\lambda = L/r_T = 14X12/5.18 = 32.43$

,where L = unbraced length = 14 ft

$r_T = \sqrt{I_f/(A_f + A_w/6)}$ = 5.18 in

$\lambda_r = 126.17\sqrt{C_b} = 126.17$,where C_b=1

Since $\lambda_p \leq \lambda \leq \lambda_r$

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

b. End Spans

$\lambda = L/r_T = 17X12/5.18 = 39.38$,where L=17 ft

$\lambda_r = 126.17\sqrt{C_b} = 166.91$,where $C_b = 1.75$

Since $\lambda_p \le \lambda \le \lambda_r$

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

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

(3) Rpg

$Rpg = 1 - 0.0005 \times 21.875/15\ [\ 224 - 970/\sqrt{33.5}\ ] = 0.9588$ (Eq.B3)

(4) M_n

$M_n = 1300 \times 0.9588 \times 33.5 = 3480$ k-ft (Eq.B2)

4.2 Tension-flange Yielding

$M_n = 1300 \times 0.9588 \times 36 = 3739$ k-ft (Eq.B1)

===> The smaller M_n(Buckling) governs.

4.3 Bending Strength

Design bending strength $\phi M_n = 0.9 \times 3480 = 3132$ k-ft

Max. factored bending strength $M_u = 3094.5$ k-ft

$\phi M_n > M_u$ O.K

5. Bearing Stiffeners

5.1 Web Crippling (Sect. K1.4*)

(1) At 114 kip load points

$\phi R_n = 54 t_w^2 \sqrt{F_y} = 29.89$ kips < 114 kips

===> Stiffeners are required, and should be extended at least one-half of the web depth.

(2) At ends

$\phi R_n = 13.44$ kips < 213 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 114 kips load points

Try 2 Pl.'s 5/16 X 5

width to thickness ratio, $b/t \leqq 95/\sqrt{Fy}$ O.K

$I = (1/12)(5/16)10^3 = 26.04\ in^4$

$A = 2(5)(5/16) + 25(5/16)^2 = 5.56\ in^2$

$r = \sqrt{I/A} = 3.116\ in$

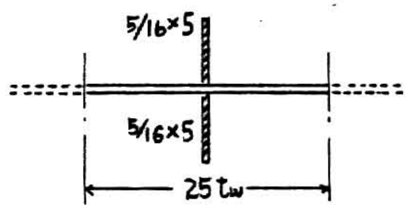
$\lambda_c = (KL/r)\sqrt{Fy/\pi^2E} = (0.75X70/3.116)\sqrt{36/\pi^2 29000} = 0.189 < \sqrt{2}$

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

$F_{cr} = 1.76F_a = 36.58$ ksi > 36 ksi ===> Use $F_{cr} = 36$ ksi

$P_n = AgF_{cr} = 200.2$ kips

$\phi P_n = 0.85 X 200.2 = 170.17$ kips > 114 kips O.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, $b/t \leqq 95/\sqrt{Fy}$ O.K

$I = (1/12)(1/2)16^3 = 171\ in^4$

$A = 2(1/2)(8) + 12(5/16)^2 = 9.2\ in^2$

$r = \sqrt{I/A} = 4.31\ in$

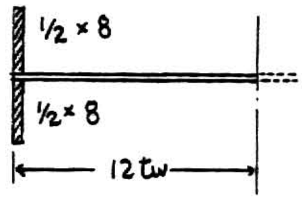
$\lambda_c = (KL/r)\sqrt{Fy/\pi^2E} = 0.137 < \sqrt{2}$

$F_a = 21.06$ ksi

$F_{cr} = 37.06$ ksi > 36 ksi ===> $F_{cr} = 36$ ksi

P_n = 331.2 kips

ϕP_n = 281.52 kips > 213 kips O.K



Use 2 Pl.'s 1/2 X 8 bearing both flanges

6. Intermediate Stiffeners

6.1 Check

(1) $h_c/t_w = 224 > 425/\sqrt{Fy_w} = 70.8$

===> Stiffeners are needed.

(2) At midspan

$V_u/Vp = 26.25/472.5 = 0.056 < \phi C_v = 0.9X\ 0.122 = 0.11$

===> Stiffers are not needed.

(3) At endspans

$V_u/Vp = 213/472.5 = 0.451 > \phi C_v = 0.11$

===> Stiffeners are needed.

6.2 Spacing

(1) At midspan

$\phi V_n \geq V_u$

$V_n \geq V_u/\phi = 26.25/0.9 = 29.17$ kips

$V_n/Vp = 29.17/(0.6\ X\ 21.9\ X\ 36) = 0.062$

From Fig.2, with $V_n/Vp = 0.062$ & $h_c/t_w = 224$

$a/h_c = \infty$ ===> Stiffeners are not needed.

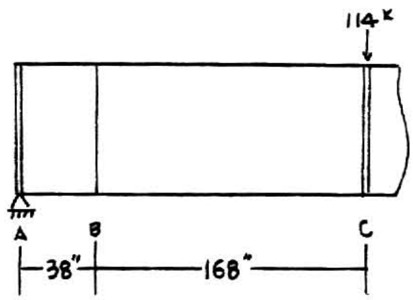
(2) At endspans

$V_n \geq V_u/\phi = 213/0.9 = 236.7$ kips

$V_n/V_p = 0.5$

From Fig. 1 with $V_n/V_p = 0.5$ & $h_c/t_w = 224$

Use $a/h_c = 0.55$ $\therefore a = 38$ in



(3) For span BC

At B, $V_u = 201.13$ kips

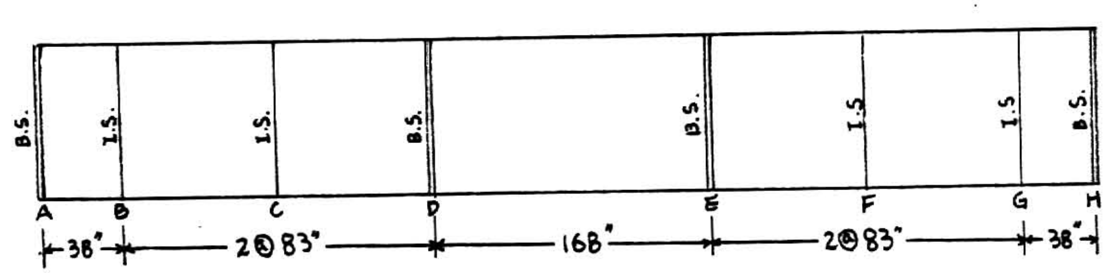
$V_n \geq V_u/\phi = 223.5$

$V_n/V_p = 0.472$

From Fig. 2, with $V_n/V_p = 0.472$ & $h_c/t_w = 224$

Use $a/h_c = 1.9$ $\therefore a = 133$ in

===> Stiffener is needed at center.



Resulting stiffener arrangement

6.3 Stiffener Design

(1) At points B & G

$A_{st} \geq [\ 0.15\ DA_w(1-C_v)V_u/\phi V_n - 18t_w]$ (Eq.F2)

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

$V_u = 201.13$ kips

$V_n/Vp = 0.53$ (Fig. 1, with $a/h_c = 0.55$)

$V_n = 250.71$ kips

$0.15(1-C_v) = 0.0714$ (Table 1)

Min. $A_{st} = [\ 0.0714X2.4X21.9X201.13/(0.9X250.71)-18X0.3125\ ]$

$= -2.284\ in^2$

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

Min. $I_{st} = at_w^3 j = 38X0.3125^3X6.264 = 7.264\ in^4$ (Eq.F1)

Try 1 Pl. 3/8 X 4

width to thickness ratio, $b/t < 95/\sqrt{Fy}$ O.K

$I = (1/3)(3/8)4^3 = 8\ in^4 > 7.264\ in^4$ O.K

===> Use 1 Pl. 3/8 X 4, bearing on comp. flange and cut 1 in short of tension flange.

(2) At points C & F

$A_{st} > [\ 0.15DA_w(1-C_v)V_u/\phi V_n - 18t_w\ ]$ (Eq.F2)

,where $D = 2.4$

$V_u = 166.19$ kips

$V_n/Vp = 0.65$ (Fig. 2, with $a/h_c = 1.186$)

$V_n = 307.48$ kips

$0.15(1-C_v) = 0.1188$ (Table 1)

Min. $A_{st} = [\ 0.1188X2.4X21.9X166.19/(0.9X307.48)-18X0.3125\ ]$

$= -1.875\ in^2$

$j = [\ 2.5/(a/h_c)^2 - 2\ ] = -0.233 < 0.5$ ===> Use $j=0.5$

Min. $I_{st} = at_w^3 j = 83X0.3125^3X0.5 = 1.266\ in^4$ (Eq.F1)

Try 1 Pl. 3/16 X 3

width to thickness ratio, $b/t < 95/\sqrt{Fy}$ O.K

$I = (1/3)(3/16)3^3 = 1.687\ in^4 > 1.266\ in^4$ O.K

===> 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

7.1 At points B & G

$V_u = 201.13$ kips

$M_u = 655.7$ k-ft

$V_u/M_u = 0.307\ ft^{-1}$

$V_n = 250.71$ kips (Fig. 1, with $a/h_c = 0.55$ & $h_c/t_w = 224$)

$M_n = 3480$ k-ft (By the procedure of III. B. Design Bending Strength)

$V_n/0.75M_u = 0.096$

$V_u/M_u > V_n/0.75M_n$ ===> Check not needed.

7.2 At points of C & F

$V_u = 166.19$ kips

$M_u = 1942.86$ k-ft

$V_u/M_u = 0.086\ ft^{-1}$

$V_n = 307.48$ kips (Fig. 2, with $a/h_c = 1.186$ & $h_c/t_w = 224$)

$M_n = 3480$ k-ft (By the procedure of III. B. Design Bending Strength)

$V_n/0.75M_n = 0.118$

$0.6V_n/M_n = 0.053$

$V_n/0.75M_n > V_u/M_u > 0.6\ V_n/M_n$

$M_u/M_n + 0.625\ V_u/V_n = 0.896 < 1.375\phi = 1.238$ O.K

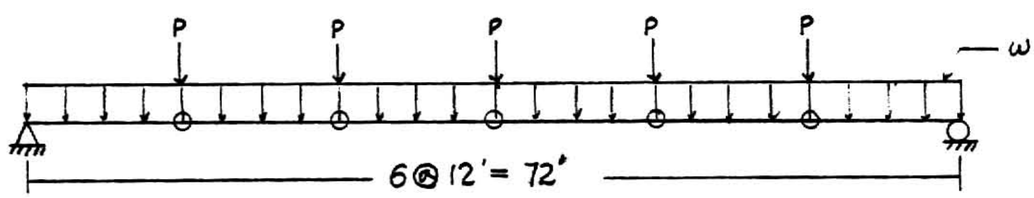
8. Comment

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 (= O)
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



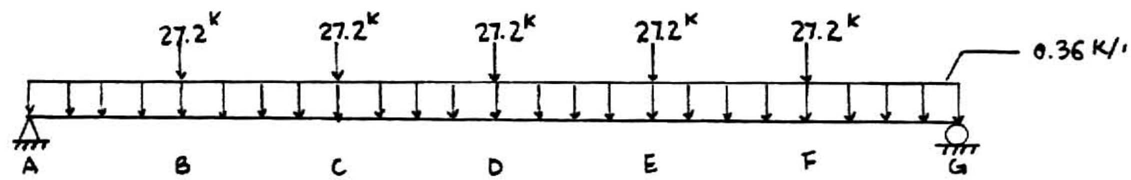
Girder loading and support

B.2 Solution

1. Factored Loads

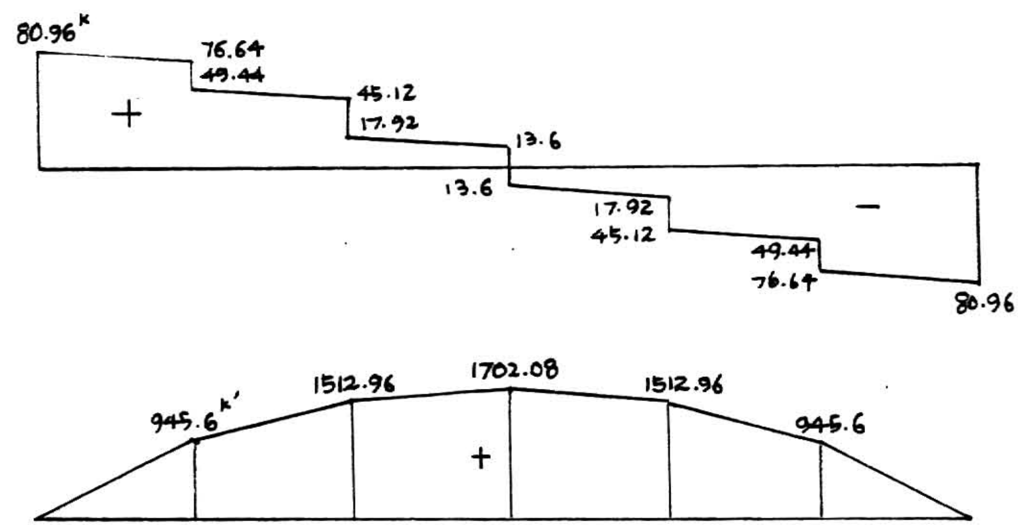
Concentrated load = $1.2D_n + 1.6L_n$ = 1.2(4) +1.6(14) = 27.2 kips

Uniform load = 1.2(0.3) + 1.6(0) = 0.36 kips/ft (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) Max. h_c

h_c = $2000t_w/\sqrt{Fy_f}$ = $2000X0.25/\sqrt{36}$ = 83 in (Eq.A3)

===> for a/h_c=12X12/83=1.74 > 1.5 in $(h_c)_{max}$=83 in N.G

h_c = $14,000t_w/\sqrt{Fy_f(Fy_f+16.5)}$ = 80 in (Eq.A4)

===> for a/h_c= 12X12/80=1.8 > 1.5 in $(h_c)_{max}$=80 in O.K

(2) Max. t_f

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

===> Max.t_f = 1/2 in (AISC Table 1.17.2.A)

(3) Max. b_f

width to thickness ratio, $b_f/2t_f \leq 95/\sqrt{Fy}$ = 15.8 O.K

Max. b_f = $2t_f$X15.8 = 2X1/2X15.8 = 15.8 in ===> Use 15 in

(4) Lightest Section

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

3.2 t_w = 5/16 in

(1) Max. h_c

h_c = $2000t_w/\sqrt{Fy_f}$ = 103 in (Eq.A3)

===> for a/h_c=12X12/103=1.4 < 1.5 $(h_c)_{max}$ = 103 in O.K

h_c = $14000t_w/\sqrt{Fy_f(Fy_f+16.5)}$ = 80 in (Eq.A4)

===> for a/h_c=12X12/80= 1.44 < 1.5 $(h_c)_{max}$ = 100 in N.G

(2) Max. t_f

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

===> Max. t_f = 3/4 in (AISC Table 1.17.2.A)

(3) Max. b_f

Max. b_f = $2t_f$X15.8 = 23.7 in ===> Use 23 in

(4) Lightest Section

If h_c = 103 in, then

h_c/t_w = 320

A_w = 32.2 in^2

$Vp = 0.6A_wFy_w$ = 696 kips

$V_u/\phi Vp$ = 0.129

From Fig. 2, with h_c/t_w = 330 and a/h_c = 1.4

$V_n/Vp = 0.53 > V_u/\phi Vp = 0.129$ ===> Shear check is O.K

Req'd S = $M_u/\phi RpgF_{cr}$ (Eq.B2)

= 1702X12/(0.9X0.7X25) = 1297 in^3

,where Rpg & F_{cr} are assumed values

Req'd I = 1297X52.25 = 67756 in^4

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

Req'd A_f = $I_f/(2X51.875^2)$ = 7.3 in^2

Req'd b_f = A_f/t_f = 7.3/0.75 = 9.73 in ===> 10 in

∴ 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

λ = 65.45 (Eq.B7)

λp = 24.33 (Eq.B8)

$\lambda_r = 126.17\sqrt{C_b}$ (Eq.B9)

= 166.9 (Assumed C_b=1.75 for smallest F_{cr})

Since $\lambda p < \lambda < \lambda_r$ F_{cr} = 30.8 ksi <---

For the limit state of flange local buckling

λ = 6.67 (Eq.B11)

λp = 10.83 (Eq.B12)

Since $\lambda p > \lambda$ F_{cr} = 36 ksi

New Rpg = 1 - 0.0005X32.2/7.5 [330 - $970/\sqrt{30.8}$] = 0.667

$S = 1102 \text{ in}^3$

$I = 576.8 \text{ in}^4$

$I_f = I - I_w = 29161 \text{ in}^4$

Req'd $A_f = 5.42 \text{ in}^2$

$A_f = 7.5 \text{ in}^2 >$ Req'd $A_f = 5.42 \text{ in}^2$ O.K

Try flange Pl. 3/4 X 9 ($A_f = 6.75 \text{ in}^2$)

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

$F_{cr} = 36$ ksi (Eq.B4)

$Rpg = 0.638$ (Eq.B3)

$S = 1197 \text{ in}^3$

$I = 62529 \text{ in}^4$

$I_f = 34072 \text{ in}^4$

Req'd $A_f = 6.33 \text{ in}^2$

$A_f = 6.75 \text{ in}^2 >$ Req'd $A_f = 6.33 \text{ in}^2$ O.K

$\therefore$ Use flange plate Pl. 3/4 X 9, web Pl. 5/16 X 59

width to thickness ratio $b_f/2t_f = 6 < 95/\sqrt{F_y}$ 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 $t_w = 3/8$ 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.

$\therefore$ 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

$h_c/t_w = 188.8$

$A_w = 18.44 \text{ in}^2$

$V_p = 0.6 \times 18.44 \times 36 = 398.3$ kips

$V_u/\phi V_p = 0.226$

From the Fig. 2, with $h_c/t_w = 188.8$ & $a/h_c = 2.44$

$V_n/V_p = 0.45 > V_u/\phi V_p = 0.226$ O.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 ($A_w = 18.44 \text{ in}^2$)

$I_x = 21414 \text{ In}^4$, $S_x = 708 \text{ in}^3$

4. Design Bending Strength

4.1 Buckling

(1) Limit state of flange local buckling

$\lambda = b_f/2t_f = 8$

$\lambda_p = 65/\sqrt{F_{yf}} = 10.83$

Since $\lambda_p > \lambda$ $F_{cr} = 36$ ksi (Eq.B4)

(2) Limit State of lateral-torsional buckling

$\lambda_p = 146/ F_{yf} = 24.33$

$\lambda = L/r_T = 48.16$,where $r_T = 2.99$

a. Span AB and FG

$\lambda_r = 126.17\sqrt{C_b} = 126.17\sqrt{1.75} = 166.91$

Since $\lambda_p < \lambda < \lambda_r$ $F_{cr} = 32.99$ ksi <--- (Eq.B5)

b. Span BC and EF

$C_b = 1.75 + 1.05\ (M_1/M_2) + 0.3(M_1/M_2)^2 > 2.3$

$\therefore C_b = 2.3$

$\lambda_r = 126.17\sqrt{2.3} = 191.35$

Since $\lambda p < \lambda < \lambda_r$ $\quad F_{cr} = 33.43$ ksi (Eq.B5)

c. Span CD and DE

$C_b = 2.3$

$\lambda_r = 191.35$

Since $\lambda p < \lambda < \lambda_r$ $\quad F_{cr} = 33.43$ ksi (Eq.B5)

(3) Rpg

$Rpg = 1-0.0005X18.44/9[188.8-970/\sqrt{32.99}] = 0.9796$ (Eq.B3)

(4) M_n

$M_n = 708 \text{ X } 0.9796 \text{ X } 32.99 = 1907$ kip-ft (Eq.B2)

4.2 Tension-flange yielding

$M_n = 708 \text{ X } 0.9796 \text{ X } 36 = 2081$ k-ft (Eq.B1)

4.3 Bending Strength

Design bending strength $\phi M_n = 0.9X1907 = 1716$ k-ft

Max. factored bending strength $M_n = 1702.8$ k-ft

$\phi M_n > M_u$ O.K

5. Bearing Stiffeners

5.1 Web crippling (Sect. K1.4*)

(1) At each concentrated load point

$\phi R_n = 54t_w^2\sqrt{Fy} = 26.89$ kips < 27.2 kips

===> Stiffeners are required, and should be extended at least one-half of the web depth.

(2) At ends

$\phi R_n = 13.44$ kips < 80.96 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

width to thickness ratio, $b/t = 95 < \sqrt{Fy}$ O.K

$I = (1/12)(3/16)4^3 = 1$

$A = 2X0.375 + 25(3/16)^2 = 1.619$

$r = \sqrt{I/A} = 0.784$

$\lambda_c = (0.75X70/0.784)\sqrt{36/\pi^2 29000} = 0.633 < \sqrt{2}$

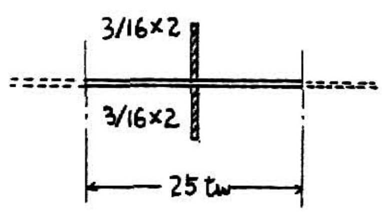
$F_a = (1-0.633^2/4)36/[5/3+3/8(0.633/\sqrt{2})-(0.633/\sqrt{2})^3/8]$

$= 15.26$ ksi

$F_{cr} = 1.76X15.16 = 28.56$ ksi

$P_n = 1.629X26.86 = 43.75$ kips

$\phi P_n = 0.85X43.75 = 37.19$ kips > 27.2 kips O.K



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

(2) At ends

Try 2 Pl.'s 1/4 X 4

width to thickness ratio, $b/t \leqq 95/\sqrt{Fy}$

$I = 10.67$ in^4

$A = 2X1 + 12(5/16)^2 = 3.17$ in^2

$r = 1.83$

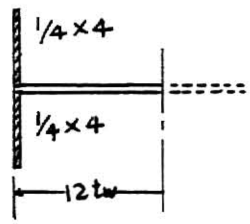
$\lambda_c = 0.271 < \sqrt{2}$

F_a = 20.33 ksi

F_{cr} = 35.78 ksi

P_n = 113.42 kips

ϕP_n = 96.4 kips > 80.96 kips O.K



Use 2 Pl.'s 1/4 X 4, bearing both flanges.

6. Intermediate Stiffeners

6.1 Check

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

===> Stiffeners are needed.

(2) At span AB and FG

V_u/V_p = 0.203 > ϕC_v = 0.154

===> Stiffeners are needed.

(3) At span BC and EF

V_u/V_p = 0.124 < ϕC_v = 0.154

===> Stiffeners are not needed.

(4) At span CD and DE

V_u/V_p = 0.045 < ϕC_v = 0.154

===> Stiffeners are not needed.

6.2 Spacing

(1) At span AB and EF

$V_n \geq V_u/\phi$ = 80.96/0.9 = 89.96 kips

V_n/V_p = 89.96/(0.6X18.44X36) = 0.226

From Fig.1, with $V_n/Vp=0.226$ and $h_c/t_w=188.8$

$a/h_c = 1.7$ $\therefore a = 100.3$ in

===> Stiffeners are needed at center.

(2) At span BC and EF

$V_n \geq V_u/\phi = 49.44/0.9 = 54.93$ kips

$V_n/Vp = 0.138$

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

$a/h_c = \infty$ ===> Stiffeners are not needed.

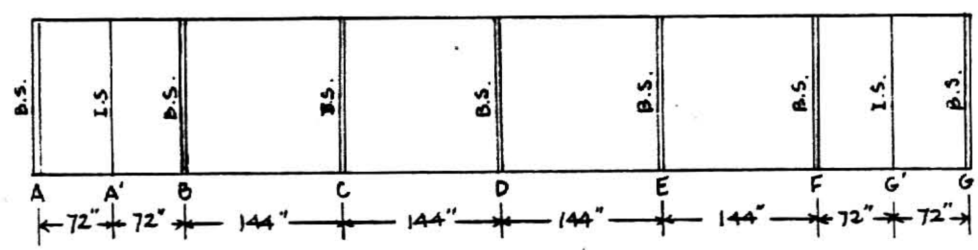
(3) At span CD and DE

$V_n \geq V_u/\phi = 17.92/0.9 = 19.91$ kips

$V_n/Vp = 0.05$

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

$a/h_c = \infty$ ===> Stiffeners are not needed.



Resulting stiffener arrangement

6.3 Design

$A_{st} = [\, 0.15\ DA_w(1 - C_v)V_u/\phi V_n - 18t_w\ ]$ (Eq.F2)

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

$V_u = 78.8$ kips

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

$\therefore V_n = 83.64$ kips

$0.15(1-C_v) = 0.107$

Min. A_{st} = [0.107X2.4X18.44X78.8/(0.9X83.64)-18X0.3125]

= -0.67 in_2

j = [$2.5/(a/h_c)^2 - 2$] = -0.32 < 0.5 ===> Use j=0.5

Min. $I_{st} = a t_w^3 j = 72X0.3125^3X0.5 = 1.1\ in^4$

Try 1 Pl. 3/16 X 3

width to thickness ratio, $b/t = 95/\sqrt{Fy}$ O.K

$I = (1/3)(3/16)^3 = 1.69\ in^4 > 1.1\ in^4$

===> 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 kips

M_u = 479.28 k-ft

$V_u/M_u = 0.164\ ft^{-1}$

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

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

$V_n/0.75M_n$ = 0.058

Since $V_u/M_u > V_n/0.75M_n$ ===> 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

The writer wishes to express his sincere appreciation and gratitude to his major professor, Dr. Peter B. Cooper, who offered continual guidance and suggestions throughout the preparation of this report.

VII. APPENDICES

A. REFERENCES

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B. Notation

A = Cross-sectional area, (in.^{2})

A_f = Flange area, (in.^{2})

A_g = Gross area, (in.^{2})

A_w = Web area, (in.^{2})

C_b = 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 unbracecd segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature.

D_n = 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)

F_a = Allowable compressive stress, (ksi)

F_{cr} = Critical stress, (ksi)

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

F_{yf} = Yield strength of the flange, (ksi)

F_{yw} = Yield strength of the web, (ksi)

I = Moment of inertia, (in.^{4})

I_x = Moment of inertia about major axis, (in.^{4})

K = Effective length factor

L = Unbraced length of member, (in.)

L_n = Live load due to occupancy

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

M_u = Required bending strength, (kip-in.)

P_n = Nominal axial strength, (kips)

R_n = Nominal resistance

Rpg= 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 (=$0.6A_wFy_w$, 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.)

ϕ = Resistance Factor

λ_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),CPG(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:LPRINT
310 LPRINT "                                FLANGE   FLANGE   REQ'D FLANGE   TOTAL SECTION"
320 LPRINT "            WEB DEPTH       WIDTH    AREA       AREA            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)^2)
430  IF I>1 THEN 460
440  BF(I)=INT(RAF(I)/TF)+1
450  IF I=1 THEN 480
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 "    RAF=##.###";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  X(1,I)=12*L/SQR(RT(I))
600  XP(1,I)=24.33
610  XR(1,I)=126.17*SQR(CB)
620  CPG(1,I)=286*1000*CB
630  X(2,I)=BF(I)/(2*TF)
640  XP(2,I)=10.83
650  XR(2,I)=28.83
660  CPG(2,I)=11200
670  FOR K=1 TO 2
680   IF X(K,I)<XP(K,I) THEN 690 ELSE 700
690   F(K)=36:GOTO 720
700   IF X(K,I)>XR(K,I) THEN F(K)=CPG(K,I)/X(K,I)^2 GOTO 720
710   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)))
720  NEXT K
730  IF F(1)<F(2) THEN FCR=F(1) ELSE FCR=F(2)
740  RPG=1-.0005*AW*(HC/TW-970/SQR(FCR))/AF(I)
750 NEXT I
760 NEXT HC
770 LPRINT " ====> OVER WIDTH TO THICKNESS RATIO !!!"
780 END
```

```
TW= .25
TF= .5
MU= 1702
CB= 1.75
RPG= .9
FCR= 33
L=? FEET 12
VU= 80.96

                FLANGE    FLANGE    REQ'D FLANGE  TOTAL SECTION
WEB DEPTH       WIDTH     AREA         AREA           AREA
=========       ======    ======    ============  =============
  HC= 83 ====> OVER WIDTH TO THICKNESS RATIO !!!
```

TW= .3125
TF= .75
MU= 1702
CB= 1.75
RPG= .7
FCR= 25
L=? FEET 12
VU= 80.96

WEB DEPTH	FLANGE WIDTH	FLANGE AREA	REQ'D FLANGE AREA	TOTAL SECTION AREA
=========	======	======	============	=============
HC=103	BF= 9	AF= 6.75	RAF= 6.331	ATT= 45.6875
HC=102	BF= 9	AF= 6.75	RAF= 6.292	ATT= 45.375
HC=101	BF= 9	AF= 6.75	RAF= 6.260	ATT= 45.0625
HC=100	BF= 9	AF= 6.75	RAF= 6.236	ATT= 44.75
HC= 99	BF= 9	AF= 6.75	RAF= 6.218	ATT= 44.4375
HC= 98	BF= 9	AF= 6.75	RAF= 6.207	ATT= 44.125
HC= 97	BF= 9	AF= 6.75	RAF= 6.202	ATT= 43.8125
HC= 96	BF= 9	AF= 6.75	RAF= 6.204	ATT= 43.5
HC= 95	BF= 9	AF= 6.75	RAF= 6.211	ATT= 43.1875
HC= 94	BF= 9	AF= 6.75	RAF= 6.224	ATT= 42.875
HC= 93	BF= 9	AF= 6.75	RAF= 6.243	ATT= 42.5625
HC= 92	BF= 9	AF= 6.75	RAF= 6.266	ATT= 42.25
HC= 91	BF= 9	AF= 6.75	RAF= 6.295	ATT= 41.9375
HC= 90	BF= 9	AF= 6.75	RAF= 6.330	ATT= 41.625
HC= 89	BF= 9	AF= 6.75	RAF= 6.369	ATT= 41.3125
HC= 88	BF= 9	AF= 6.75	RAF= 6.412	ATT= 41
HC= 87	BF= 9	AF= 6.75	RAF= 6.461	ATT= 40.6875
HC= 86	BF= 9	AF= 6.75	RAF= 6.515	ATT= 40.375
HC= 85	BF= 9	AF= 6.75	RAF= 6.573	ATT= 40.0625
HC= 84	BF= 9	AF= 6.75	RAF= 6.635	ATT= 39.75
HC= 83	BF= 9	AF= 6.75	RAF= 6.703	ATT= 39.4375
HC= 82	BF= 10	AF= 7.5	RAF= 6.265	ATT= 40.625
HC= 81	BF= 10	AF= 7.5	RAF= 6.352	ATT= 40.3125
HC= 80	BF= 10	AF= 7.5	RAF= 6.443	ATT= 40
HC= 79	BF= 10	AF= 7.5	RAF= 6.538	ATT= 39.6875
HC= 78	BF= 10	AF= 7.5	RAF= 6.636	ATT= 39.375
HC= 77	BF= 10	AF= 7.5	RAF= 6.739	ATT= 39.0625
HC= 76	BF= 10	AF= 7.5	RAF= 6.847	ATT= 38.75
HC= 75	BF= 10	AF= 7.5	RAF= 6.958	ATT= 38.4375
HC= 74	BF= 10	AF= 7.5	RAF= 7.074	ATT= 38.125
HC= 73	BF= 10	AF= 7.5	RAF= 7.195	ATT= 37.8125
HC= 72	BF= 10	AF= 7.5	RAF= 7.320	ATT= 37.5
HC= 71	BF= 10	AF= 7.5	RAF= 7.449	ATT= 37.1875
HC= 70	BF= 11	AF= 8.25	RAF= 7.261	ATT= 38.375
HC= 69	BF= 11	AF= 8.25	RAF= 7.404	ATT= 38.0625
HC= 68	BF= 11	AF= 8.25	RAF= 7.552	ATT= 37.75
HC= 67	BF= 11	AF= 8.25	RAF= 7.706	ATT= 37.4375
HC= 66	BF= 11	AF= 8.25	RAF= 7.864	ATT= 37.125
HC= 65	BF= 11	AF= 8.25	RAF= 8.028	ATT= 36.8125
HC= 64	BF= 11	AF= 8.25	RAF= 8.197	ATT= 36.5
HC= 63	BF= 12	AF= 9	RAF= 8.133	ATT= 37.6875
HC= 62	BF= 12	AF= 9	RAF= 8.316	ATT= 37.375
HC= 61	BF= 12	AF= 9	RAF= 8.505	ATT= 37.0625
HC= 60	BF= 12	AF= 9	RAF= 8.701	ATT= 36.75

```
HC= 59      BF= 12    AF= 9       RAF= 8.903     ATT= 36.4375
HC= 58      BF= 13    AF= 9.75    RAF= 8.923     ATT= 37.625
HC= 57      BF= 13    AF= 9.75    RAF= 9.140     ATT= 37.3125
HC= 56      BF= 13    AF= 9.75    RAF= 9.365     ATT= 37
HC= 55      BF= 13    AF= 9.75    RAF= 9.597     ATT= 36.6875
HC= 54      BF= 14    AF= 10.5    RAF= 9.684     ATT= 37.875
HC= 53      BF= 14    AF= 10.5    RAF= 9.934     ATT= 37.5625
HC= 52      BF= 14    AF= 10.5    RAF=10.192     ATT= 37.25
HC= 51      BF= 14    AF= 10.5    RAF=10.460     ATT= 36.9375
HC= 50      BF= 15    AF= 11.25   RAF=10.610     ATT= 38.125
HC= 49      BF= 15    AF= 11.25   RAF=10.898     ATT= 37.8125
HC= 48      BF= 15    AF= 11.25   RAF=11.198     ATT= 37.5
HC= 47      BF= 16    AF= 12      RAF=11.399     ATT= 38.6875
HC= 46      BF= 16    AF= 12      RAF=11.723     ATT= 38.375
HC= 45      BF= 17    AF= 12.75   RAF=11.964     ATT= 39.5625
HC= 44      BF= 17    AF= 12.75   RAF=12.316     ATT= 39.25
HC= 43      BF= 17    AF= 12.75   RAF=12.682     ATT= 38.9375
HC= 42      BF= 18    AF= 13.5    RAF=13.174     ATT= 40.125
HC= 41      BF= 18    AF= 13.5    RAF=13.581     ATT= 39.8125
HC= 40      BF= 20    AF= 15      RAF=14.649     ATT= 42.5
HC= 39      BF= 20    AF= 15      RAF=15.113     ATT= 42.1875
HC= 38      BF= 22    AF= 16.5    RAF=16.333     ATT= 44.875
HC= 37 ====> OVER WIDTH TO THICKNESS RATIO !!!
```

TW= .375
TF= .875
MU= 1702
CB= 1.75
RPG= .5
FCR= 20
L=? FEET 12
VU= 80.96

WEB DEPTH	FLANGE WIDTH	FLANGE AREA	REQ'D FLANGE AREA	TOTAL SECTION AREA
HC=124	BF= 8	AF= 7.000	RAF= 5.225	ATT= 60.5
HC=123	BF= 8	AF= 7.000	RAF= 5.075	ATT= 60.125
HC=122	BF= 8	AF= 7.000	RAF= 4.940	ATT= 59.75
HC=121	BF= 8	AF= 7.000	RAF= 4.817	ATT= 59.375
HC=120	BF= 8	AF= 7.000	RAF= 4.707	ATT= 59
HC=119	BF= 8	AF= 7.000	RAF= 4.609	ATT= 58.625
HC=118	BF= 8	AF= 7.000	RAF= 4.521	ATT= 58.25
HC=117	BF= 7	AF= 6.125	RAF= 6.076	ATT= 56.125
HC=116	BF= 7	AF= 6.125	RAF= 5.943	ATT= 55.75
HC=115	BF= 7	AF= 6.125	RAF= 5.823	ATT= 55.375
HC=114	BF= 7	AF= 6.125	RAF= 5.715	ATT= 55
HC=113	BF= 7	AF= 6.125	RAF= 5.620	ATT= 54.625
HC=112	BF= 7	AF= 6.125	RAF= 5.535	ATT= 54.25
HC=111	BF= 7	AF= 6.125	RAF= 5.460	ATT= 53.875
HC=110	BF= 7	AF= 6.125	RAF= 5.395	ATT= 53.5
HC=109	BF= 7	AF= 6.125	RAF= 5.339	ATT= 53.125
HC=108	BF= 7	AF= 6.125	RAF= 5.292	ATT= 52.75
HC=107	BF= 7	AF= 6.125	RAF= 5.253	ATT= 52.375
HC=106	BF= 7	AF= 6.125	RAF= 5.221	ATT= 52
HC=105	BF= 7	AF= 6.125	RAF= 5.197	ATT= 51.625
HC=104	BF= 7	AF= 6.125	RAF= 5.180	ATT= 51.25
HC=103	BF= 7	AF= 6.125	RAF= 5.169	ATT= 50.875
HC=102	BF= 7	AF= 6.125	RAF= 5.165	ATT= 50.5
HC=101	BF= 7	AF= 6.125	RAF= 5.167	ATT= 50.125
HC=100	BF= 7	AF= 6.125	RAF= 5.175	ATT= 49.75
HC= 99	BF= 7	AF= 6.125	RAF= 5.188	ATT= 49.375
HC= 98	BF= 7	AF= 6.125	RAF= 5.207	ATT= 49
HC= 97	BF= 7	AF= 6.125	RAF= 5.232	ATT= 48.625
HC= 96	BF= 7	AF= 6.125	RAF= 5.261	ATT= 48.25
HC= 95	BF= 7	AF= 6.125	RAF= 5.296	ATT= 47.875
HC= 94	BF= 7	AF= 6.125	RAF= 5.335	ATT= 47.5
HC= 93	BF= 7	AF= 6.125	RAF= 5.380	ATT= 47.125
HC= 92	BF= 7	AF= 6.125	RAF= 5.429	ATT= 46.75
HC= 91	BF= 7	AF= 6.125	RAF= 5.482	ATT= 46.375
HC= 90	BF= 7	AF= 6.125	RAF= 5.540	ATT= 46
HC= 89	BF= 7	AF= 6.125	RAF= 5.603	ATT= 45.625
HC= 88	BF= 7	AF= 6.125	RAF= 5.670	ATT= 45.25
HC= 87	BF= 7	AF= 6.125	RAF= 5.741	ATT= 44.875
HC= 86	BF= 7	AF= 6.125	RAF= 5.817	ATT= 44.5
HC= 85	BF= 7	AF= 6.125	RAF= 5.897	ATT= 44.125
HC= 84	BF= 7	AF= 6.125	RAF= 5.982	ATT= 43.75
HC= 83	BF= 7	AF= 6.125	RAF= 6.071	ATT= 43.375
HC= 82	BF= 8	AF= 7.000	RAF= 5.489	ATT= 44.75
HC= 81	BF= 8	AF= 7.000	RAF= 5.595	ATT= 44.375

HC= 80	BF= 8	AF= 7.000	RAF= 5.706	ATT= 44
HC= 79	BF= 8	AF= 7.000	RAF= 5.820	ATT= 43.625
HC= 78	BF= 8	AF= 7.000	RAF= 5.938	ATT= 43.25
HC= 77	BF= 8	AF= 7.000	RAF= 6.060	ATT= 42.875
HC= 76	BF= 8	AF= 7.000	RAF= 6.186	ATT= 42.5
HC= 75	BF= 8	AF= 7.000	RAF= 6.317	ATT= 42.125
HC= 74	BF= 8	AF= 7.000	RAF= 6.452	ATT= 41.75
HC= 73	BF= 8	AF= 7.000	RAF= 6.591	ATT= 41.375
HC= 72	BF= 8	AF= 7.000	RAF= 6.735	ATT= 41
HC= 71	BF= 8	AF= 7.000	RAF= 6.883	ATT= 40.625
HC= 70	BF= 9	AF= 7.875	RAF= 6.606	ATT= 42
HC= 69	BF= 9	AF= 7.875	RAF= 6.767	ATT= 41.625
HC= 68	BF= 9	AF= 7.875	RAF= 6.934	ATT= 41.25
HC= 67	BF= 9	AF= 7.875	RAF= 7.105	ATT= 40.875
HC= 66	BF= 9	AF= 7.875	RAF= 7.281	ATT= 40.5
HC= 65	BF= 9	AF= 7.875	RAF= 7.463	ATT= 40.125
HC= 64	BF= 9	AF= 7.875	RAF= 7.650	ATT= 39.75
HC= 63	BF= 9	AF= 7.875	RAF= 7.843	ATT= 39.375
HC= 62	BF= 10	AF= 8.750	RAF= 7.733	ATT= 40.75
HC= 61	BF= 10	AF= 8.750	RAF= 7.939	ATT= 40.375
HC= 60	BF= 10	AF= 8.750	RAF= 8.152	ATT= 40
HC= 59	BF= 10	AF= 8.750	RAF= 8.371	ATT= 39.625
HC= 58	BF= 10	AF= 8.750	RAF= 8.598	ATT= 39.25
HC= 57	BF= 11	AF= 9.625	RAF= 8.593	ATT= 40.625
HC= 56	BF= 11	AF= 9.625	RAF= 8.835	ATT= 40.25
HC= 55	BF= 11	AF= 9.625	RAF= 9.084	ATT= 39.875
HC= 54	BF= 11	AF= 9.625	RAF= 9.342	ATT= 39.5
HC= 53	BF= 11	AF= 9.625	RAF= 9.609	ATT= 39.125
HC= 52	BF= 12	AF=10.500	RAF= 9.693	ATT= 40.5
HC= 51	BF= 12	AF=10.500	RAF= 9.978	ATT= 40.125
HC= 50	BF= 12	AF=10.500	RAF=10.273	ATT= 39.75
HC= 49	BF= 13	AF=11.375	RAF=10.422	ATT= 41.125
HC= 48	BF= 13	AF=11.375	RAF=10.738	ATT= 40.75
HC= 47	BF= 13	AF=11.375	RAF=11.067	ATT= 40.375
HC= 46	BF= 14	AF=12.250	RAF=11.275	ATT= 41.75
HC= 45	BF= 14	AF=12.250	RAF=11.629	ATT= 41.375
HC= 44	BF= 14	AF=12.250	RAF=11.997	ATT= 41
HC= 43	BF= 15	AF=13.125	RAF=12.265	ATT= 42.375
HC= 42	BF= 15	AF=13.125	RAF=12.664	ATT= 42
HC= 41	BF= 15	AF=13.125	RAF=13.080	ATT= 41.625
HC= 40	BF= 16	AF=14.000	RAF=13.413	ATT= 43
HC= 39	BF= 16	AF=14.000	RAF=13.867	ATT= 42.625
HC= 38	BF= 17	AF=14.875	RAF=14.251	ATT= 44
HC= 37	BF= 17	AF=14.875	RAF=14.748	ATT= 43.625
HC= 36	BF= 18	AF=15.750	RAF=15.188	ATT= 45
HC= 35	BF= 18	AF=15.750	RAF=15.737	ATT= 44.625
HC= 34	BF= 19	AF=16.625	RAF=16.238	ATT= 46
HC= 33	BF= 20	AF=17.500	RAF=16.953	ATT= 47.375
HC= 32	BF= 20	AF=17.500	RAF=17.604	ATT= 47
HC= 31	BF= 22	AF=19.250	RAF=18.997	ATT= 50.125
HC= 30	BF= 23	AF=20.125	RAF=20.133	ATT= 51.5
HC= 29	BF= 25	AF=21.875	RAF=21.775	ATT= 54.625
HC= 28	BF= 26	AF=22.750	RAF=23.126	ATT= 56

HC= 27 ====> OVER WIDTH TO THICKNESS RATIO !!!

(X 10^{-2})

Slenderness ratio (h_c/t_w)	Aspect ratio (a/h_c) 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	>3.0
60	0	0	0	0	0	0	0	0	0	0	0	0	0	0
80	0	0	0	0	0	0	0	0	0	0.05	0.39	0.93	1.23	1.93
100	0	0	0	0	0	0.22	1.39	2.15	2.67	3.00	3.54	4.37	4.81	5.83
120	0	0	0	1.06	1.98	2.69	4.21	5.39	6.15	6.67	7.04	7.62	7.93	8.63
140	0	0.49	1.98	3.02	4.55	5.65	7.08	7.94	8.50	8.88	9.15	9.57	9.80	10.32
160	0.39	2.30	4.11	5.82	7.00	7.84	8.93	9.59	10.02	10.31	10.52	10.85	11.42	11.42
180	2.01	4.31	6.40	7.75	8.68	9.34	10.21	10.73	11.07	11.30	11.46	12.17	12.17	12.17
200	3.54	6.34	8.03	9.13	9.88	10.42	11.21	11.54	11.81	12.71	12.71	12.71	12.71	12.71
220	5.53	7.85	9.24	10.15	10.77	11.21	11.79	13.11	13.11	13.11	13.11	13.11	13.11	13.11
240	7.04	8.99	10.16	11.44	11.82	13.41	13.41	13.41	13.41	13.41	13.41	13.41	13.41	13.41
260	8.22	9.88	10.88	11.53	11.97	13.64	13.64	13.64	13.64	13.64	13.64	13.64	13.64	13.64
280	9.15	10.58	11.44	12.00	13.83	13.83	13.83	13.83	13.83	13.83	13.83	13.83	13.83	13.83
300	9.91	11.15	11.90	13.98	13.98	13.98	13.98	13.98	13.98	13.98	13.98	13.98	13.98	13.98
320	10.52	11.62	14.10	14.10	14.10	14.10	14.10	14.10	14.10	14.10	14.10	14.10	14.10	14.10
340	11.04	14.21	14.21	14.21	14.21	14.21	14.21	14.21	14.21	14.21	14.21	14.21	14.21	14.21

Table 1. $0.15(1-C_v)$ values for 36 ksi yield stress steel

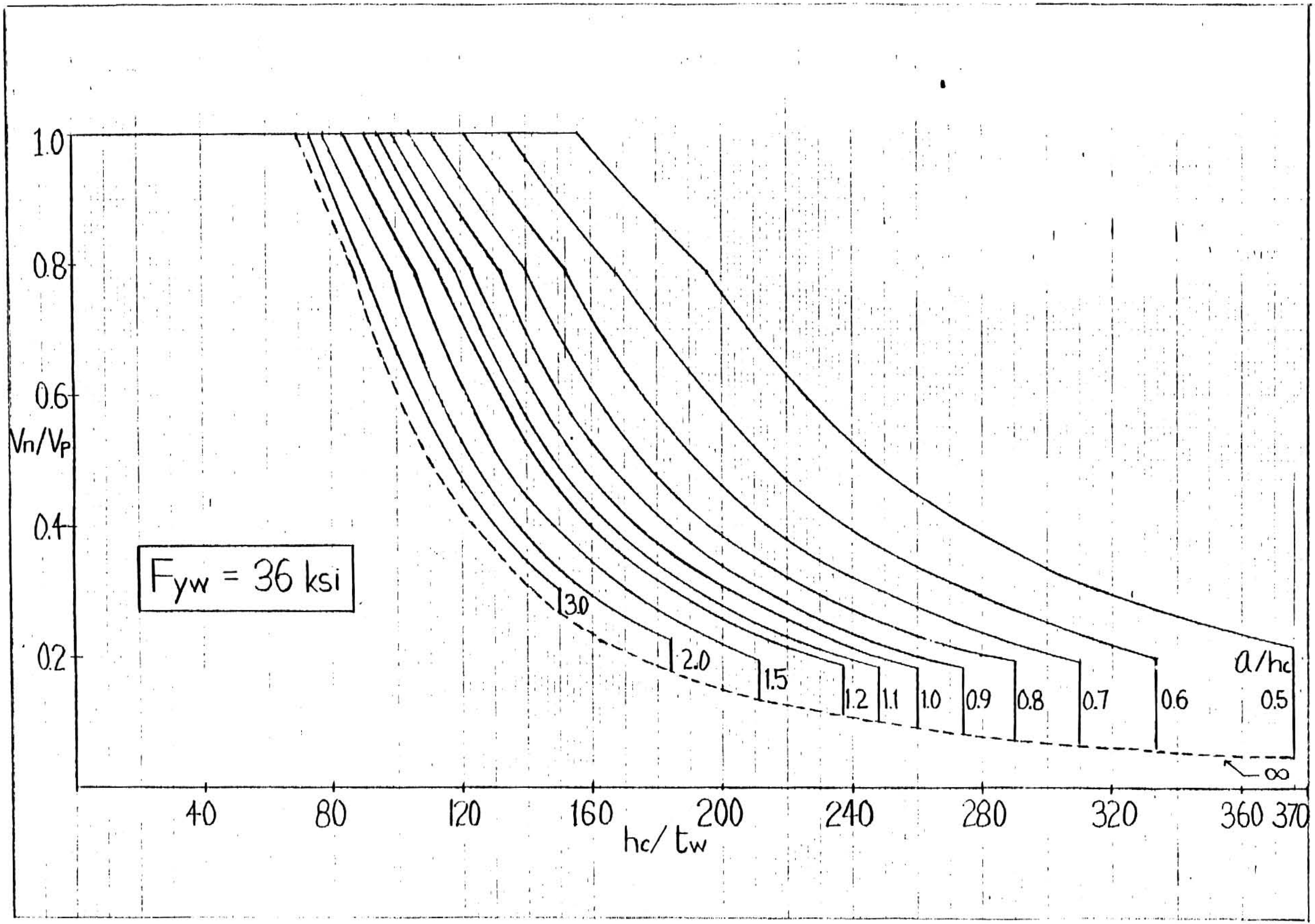

Fig. 1 Shear strength without tension field action

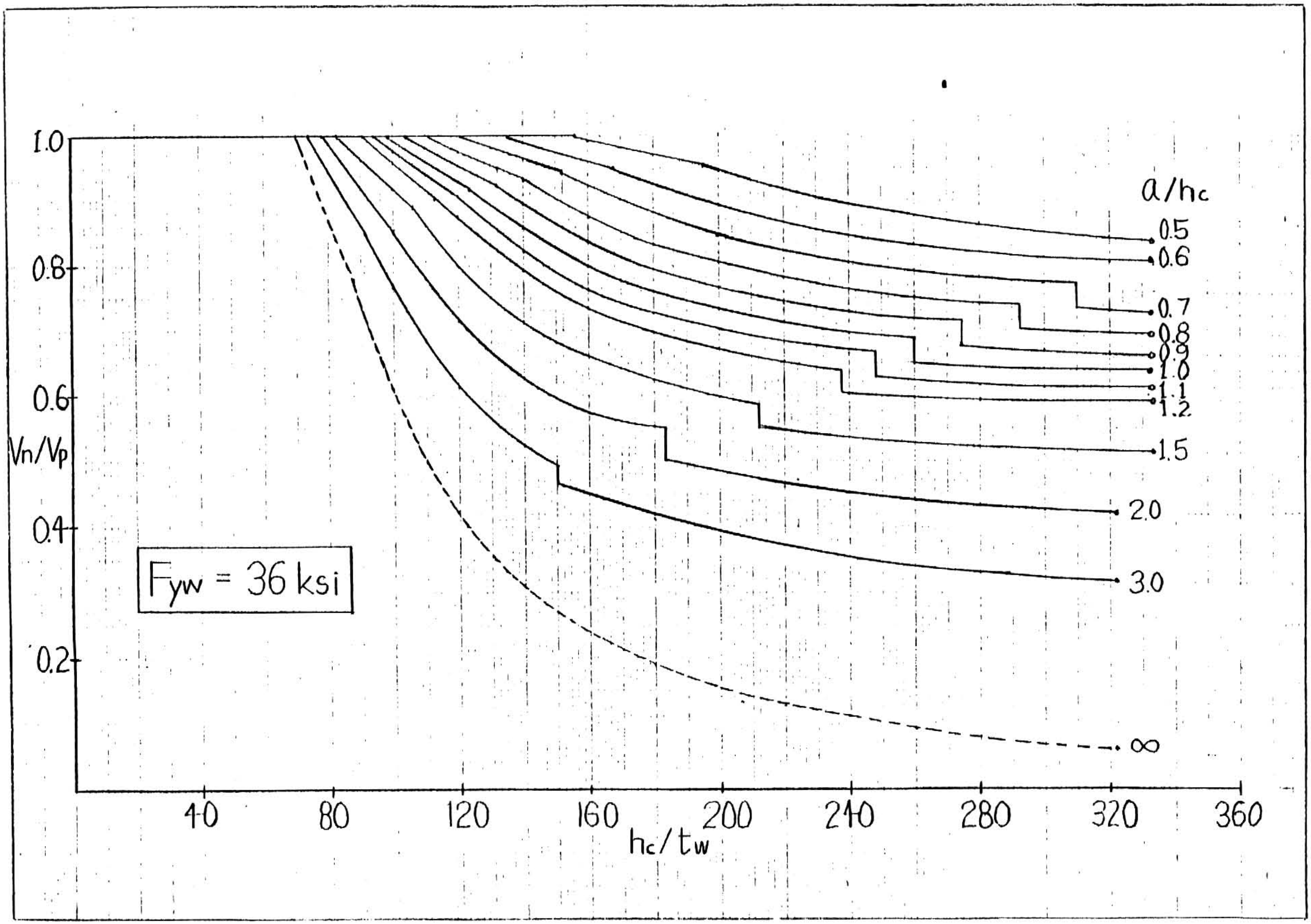

Fig. 2 Shear strength with tension field action

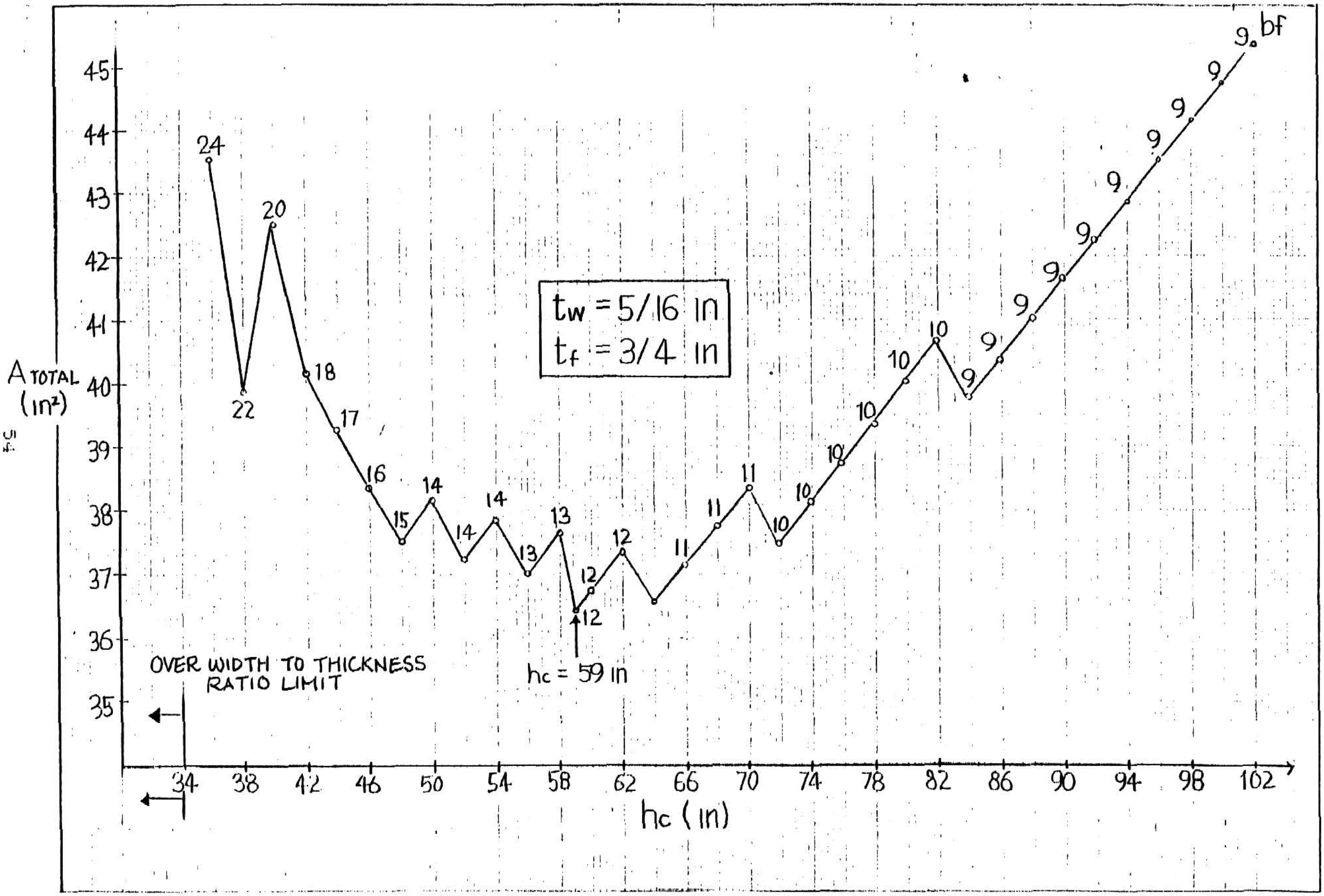

Fig. 3 Result of computer output (Example 2)

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.