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SIDOR's Design Aids for Cold-formed Steel Beams and Columns

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SIDOR'S DESIGN AIDS FOR COLD-FORMED STEEL BEAMS AND COLUMNS

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SUMMARY

This paper shows how to take full advantage of Beam Tables in the design when the value C_b is other than 1,0 and of Column Tables for shapes which may be subject to torsional or torsional-flexural buckling and also to combined axial and bending stresses.

INTRODUCTION

The new second edition of SIDOR's "MANUAL DE PROYECTO DE ESTRUCTURAS DE ACERO" (4) has been prepared to assist engineers, manufacturers, educators, and others, in the use and design of structural steel. The emphasis is on cold formed and welded structural members.

The first of three volumes contains the authorized spanish edition (metric and ISO units) of the AISI Design Specification, Commentary, and Supplementary Information (abstract) according to AISI's COLD FORMED STEEL DESIGN MANUAL, 1977 Edition. The Tentative Criteria for Structural Applications of Steel Tubing and Pipe is also included.

For solving problems related to cold formed structural members, as defined by the AISI's Specification (1,2,3), Part 4 of the Manual's Volume II provides Tables of Properties for designing and Dimensions for detailing of the shapes most commonly used in local practice. It also provides Tables for compression and flexural members, and Tables for Connections. For a given shape, the allowable moments for beams are at the left page, and in front, at the right page, the allowable concentric loads are given for columns.

The third volume provides discussions and examples on the behavior and design criteria of cold formed structural members.

BEAM TABLES

For each shape, the characteristic length, L_u, which is the maximum length between lateral supports or the maximum span length of a simply supported beam to prevent possible lateral buckling are given. Thus, if the effective length, $K_a L_m$, is less than L_u, the local buckling controls design. Otherwise, when $K_a L_m$ is greather than L_u, the lateral buckling determines the beam capacity. The value $K_a L_m = 0$ gives the maximum moment without local buckling reduction.

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For closed box-type sections used as beams subjected to bending about the major axis, the characteristic length adopted is proposed by the ICHA's Manual (6) which is a modification of Section 5.3 of the AISI Specification (1,2,3):

$$L_u = 2{,}500B/F_y$$

where B is the width of the compressed flange.

In the case of hat sections used as continuous beams, the required information at negative moment is given according to the AISI's interative procedure for laterally unbraced compression flanges (Section 3, Supplementary Information (1)).

L_u is omitted when the section is laterally stable.

Highlights of the effective length concept is the direct use of tables for any length between lateral bracing, L_m, when C_b is other than 1,0. Introducing the following variables:

$$K_a = 1/\sqrt{C_b}$$

$$r_a = \sqrt{d\ I_{yc}/S_{xc}}$$

It can be demonstrated (5,6,4) that the AISI equations included in Sections 3.3 and 3.6.1.1 of the Specification for the maximum compression stress, F_b, that is Eqs. 3.3-1 and 3.3-2 for symmetrical I-shaped sections and symmetrical channel-shaped sections, Eqs. 3.3-3 and 3.3-4 for point symmetrical Z-shaped sections, and Eq. 3.6.1-3 for hat shaped beams when I_y is less than I_x can be written as:

$$F_b = \frac{1}{1.50}\left[1 - \frac{(K_a L_m/r_a)^2}{1.8\ C_c^2}\right] F_y \qquad \text{(Eq.3.3-1,Modified)}$$

$$F_b = \frac{1}{1.67}\left[(\pi^2 E)/(K_a L_m/r_a)^2\right] \qquad \text{(Eq. 3.3-2,Mod.)}$$

$$F_b = \frac{1}{1.50}\left[1 - \frac{(K_a L_m/r_a)^2}{0.9\ C_c^2}\right] F_y \qquad \text{(Eq. 3.3-3,Mod.)}$$

$$F_b = \frac{1}{3.33}\left[(\pi^2 E)/(K_a L_m/r_a)^2\right] \qquad \text{(Eq. 3.3-4,Mod.)}$$

$$F_b = \frac{1}{1.92}\left[(\pi^2 E)/(K_a L_m/r_a)^2\right] \qquad \text{(Eq. 3.6.1-3,Mod.)}$$

where

$$C_c = \sqrt{2\pi^2 E/F_y} \qquad \text{(Eq. 3.6.1-5)}$$

$$C_b = 1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)^2 \qquad \text{(Eq. 3.3-5)}$$

COLUMN TABLES

Tables for allowable concentric loads on columns, P, are presented for individual channel and Z-sections, both for stiffened and unstiffened flanges, for I or box-shaped members made by connecting two channels, and for equal leg angles with unstiffened legs and for its common combination as T,X, and closed boxes. Tables are also given for hat sections.

In each table the possible buckling modes (flexural, flexural-torsional, torsional) of the section are considered. For this reason the allowable concentric load is determined by the effective length, KL, with respect to the appropiate radius of gyration. The maximum allowable load, P_{max}, is given by KL = 0.

BEAM-COLUMNS

Members subjected to both axial compression and bending shall be proportioned to meet the requirements prescribed in AISI's Section 3.7. To facilitate the use of Beam and Column Tables, AISI 1980's equations are written in terms of allowable working loads and not using conventional allowable working stress.

The working loads over the section are denoted as p and m while the allowable loads or resistance capacity of sections are denoted as P, P_{max}, M_x and M_y (tabulated values).

Doubly-symmetric shapes

Doubly-symmetric shapes or shapes which are not subjected to torsional-flexural buckling should be designed in accordance with any one of the Eqs. 3.7.1-1 to 3.7.1-3 as applicable (1,2,3).

Table 1.- Combined Axial Load and Bending

Condition	Interaction Formulas
A. $p/P > 0.15$	1. $\frac{p}{P} + \frac{C_{mx}}{\left[1 - p/P_x^E\right]} \frac{m_x}{M_x} + \frac{C_{my}}{\left[1 - p/P_y^E\right]} \frac{m_y}{M_y} \leq 1$ 2. $\frac{p}{P_{max}} + \frac{m_x}{M_x} + \frac{m_y}{M_y} \leq 1$
B. $p/P \leq 0.15$	$\frac{p}{P} + \frac{m_x}{M_x} + \frac{m_y}{M_y} \leq 1$

Biaxial bending of symmetrical sections may be computed using Formula B with p = 0. For simple design procedure the above interaction formulas becomes,

$$p + \frac{C_{mx}}{(1-p/P_x^E)} \left(\frac{P}{M_x}\right) m_x + \frac{C_{my}}{(1-p/P_y^E)} \left(\frac{P}{M_y}\right) m_y \leqslant P \quad \text{(Form.A1-Mod.)}$$

$$\left(\frac{P}{P_{max}}\right) p + \left(\frac{P}{M_x}\right) m_x + \left(\frac{P}{M_y}\right) m_y \leqslant P \quad \text{(Form.A2-Mod.)}$$

$$p + \left(\frac{P}{M_x}\right) m_x + \left(\frac{P}{M_y}\right) m_y \leqslant P \quad \text{(Form.B-Mod.)}$$

A preliminary selection can be made using the equivalence axial load, $P_{equiv.}$,

$$P_{equiv} = p + B_x m_x + B_y m_y \leqslant P$$

where

$B_x = QA/(S_x)_{efective}$ and $B_y = QA/(S_y)_{efective}$

Singly-symmetric and Point-symmetric Shapes

The load carrying capacity of these shapes should be determined on the basis of tabular loads for axially loaded members which may be subject to torsional-flexural buckling, P^{FT}, and flexural buckling, P^F. Torsional loads, P^T, are given in the rarely cases in which the design governs. According to Sections 3.7.2 and 3.7.3 of the AISI Specification the following formulas are valid when the X axis is the axis of symmetry.

a. Flexural buckling

The allowable load is calculated using Eqs. A1-Mod., A2-Mod., or B-Modified with $m_x = 0$ and replacing P with P_y^F.

For preliminary design the relation to be satisfied is the following:

$p(1 + B_y e)\ P_y^F$

In the above equation,

For $e > 0$, $B_y = QA/(S_y)_{efect.}$, where S_y is the compressed section modulus for the effective section.

For $e < 0$, $B_y = QA/(S_y)_{efect.}$, where S_y is the tension section modulus for the effective section.

b. Flexural-Torsional buckling (Axis of symmetry)

The load p shall be less than or equal to load ${}^eP_x^{FT}$ indicated in Table 2.

Table 2.- Allowable load for Flexural-Torsional Buckling Mode, ${}^{e}P_x^{FT}$

Eccentricity of the axial load e	Allowable load ${}^{e}P_x^{FT}$	Remarks
A. Load applied on side of centroid opposite from shear center. (+e)	1. ${}^{+}P_x^{FT} = \alpha\, P_{max}$ if ${}^{+}P_x^{FT} / P_{max} > 0.5$ 2. ${}^{+}P_x^{FT} = {}^{+}P_x^{FT}$ if ${}^{+}P_x^{FT} / P_{max} \leqslant 0.5$ where $\alpha = 1- \frac{P_{max}}{4\ {}^{+}P_x^{FT}}$	${}^{+}P_x^{FT} = 0.5\left[\phi_2 - \sqrt{\phi_2 - 4\phi_1}\right]$ $\phi_1 = P_x^{FT} . P_y^E$ $\phi_2 = P_x^{FT} + P_y^E - \frac{C_{TF}.e.P_x^{FT}.P_y^E}{j\ .\ \eta^-.P_y^E}$ $\eta^- = 1- \sqrt{1+(r_o/j)^2(P^T/P_x^E)}$
B. Load applied between the shear center and the centroid. (-e)	1. Except for T-or unsymmetric I-sections ${}^{-}P^{FT} = P_x^{FT} + \frac{e}{x_o}({}^{x_o}P_y^F - P_x^{FT})$ if $P_y^F > P_x^{FT}$ * 2. For T-and unsymmetric I-sections ${}^{-}P_x^{FT} = P_x^{FT} + \frac{e}{x_o}({}^{x_o}P^i - P_x^{FT})$ if $P_y^F > P_x^{FT}$ *	For ${}^{x_o}P_y^F$ see case a.Flexural buckling For ${}^{x_o}P^i$ to use the lower value between ${}^{x_o}P_y^F$ and ${}^{x_o}P_x^{FT}$ (See C. below)
C. Load applied on side of shear center opposite from centroid. (-e)	1. For T-and unsymmetric I-sections ${}^{-}P_x^{FT} = \alpha\ {}^{-}P_x^{FT}$ if ${}^{-}P_x^{FT} / P_{max} > 0.5$ ${}^{-}P_x^{FT} = {}^{-}P_x^{FT}$ if ${}^{-}P_x^{FT} / P_{max} \leqslant 0.5$ where $\alpha = 1- \frac{P_{max}}{4.{}^{-}P_x^{FT}}$	${}^{-}P^{FT} = 0.5\left[\phi_4 - \sqrt{\phi_4^2 + 4\phi_3}\right]$ $\phi_3 = \frac{P_x^E.P_y^E.j.\eta^+}{C_{TF}.x_o + j.\ \eta^+}$ $\phi_4 = \frac{j.\eta^+(P_y^E+P_x^E) - C_{TF}.P_y^E(e-x_o)}{j.\eta^+ + C_{TF}.x_o}$ $\eta^+ = 1+ \sqrt{1+(r_o/j)^2(P^T/P_x^E)}$

* If $P_y^F \leqslant P_x^{FT}$, P shall be determining as P_y^F with $e = x_o$

C_{TF} is a coefficient defined in AISI Spec.,Section 3.7.2

DESIGN EXAMPLES

In the following, the application of design method is demostrated in two examples. To facilitate comparison Example 1 is similar to Example 4.9 from Ref. 7, and Example 2 is similar to Example 22 from Ref. 1. In both examples the conversions from metric units to U.S. customary units are only approximate.

Example 1.- Determine the allowable uniform load, q, of an IC 200x15,9 (mm) shape as simply supported beam laterally braced at both ends and midspan. Use the value of C_b determined by the AISI 1980 Eq. 3.3-5.

Given:
Simple span length = 10 ft F_y = 36 ksi $A = 3.13$ in.^{2} $r_y = 0.673$ in.
Flange flat width ratio = 12.3 $d = 7.87$ in. $S_x = 6.47$ in.^{3}

Solution:

With $M_1 = M_2 = 0$, $C_b = 1.75$ then $K_a = 1/\sqrt{1.75} = 0.756$ and $K_aL_m = 0.756 \times 5 = 3.78$ft

From Beam Tables K_aL_m = 3,00 ft M_r = 134 in.-kips
K_aL_m = 4,00 ft M_r = 134 in.-kips

Then M_r = 134 in.-kips. The allowable uniform load is

$\frac{qL^2}{8}(12) = 134$ $q = 0.893$ kips/ft

Example 2.- Determine the allowable axial load for the hat section 100x13,5 (mm) if the eccentricity of the axial load is e = +2 in. at both ends. Assume $K = C_m = C_{TF} = 1,0$ and $F_y = 36$ ksi.

Given:

$A = 17,2$ cm^2 (2.67 in.^{2}) $Q = 0.981$ $L = 4,5$m (14.76 ft)
$r_x = 7,29$ cm (2.87 in.) $r_y = 3,99$ cm (1.57 in.) $(S_y)_{ef} = 50,4$cm^3 (3.07 in.^{3})
$r_o = 11,9$ cm (4.68 in.) $j = 12,5$ cm (4.92 in.) $J = 0,918$ cm^4 (0.0221 in.^{4})
$C_w = 6430$ cm^6 (23.94 in.^{6})

Solution:

I. Flexural beam-column behavior (Axis Y-Y)

1. First approximation

$B_y = QA/(S_y)_{ef} = 0,335$

$p(1 + B_y e) \leq P_y^F = p(1 + 0,335 \times 5) \leq P_y^F$. Solving this expression for p leads to $p = 0,374\ P_y^F$

With KL_y = 4.5m, from Colun Tables, P_y^F = 13,7 tf(30.2 kips), p=5,12tf(11.3 kips

2. Check interation formulas
Using $KL_b/r_b = 113$, $P_y^E = A\ F'_{ey} = 14,4$tf (31.75 kips)
With KL=0 from the Columns Tables, we obtain P_{max} = 22,3 tf (49.2 kips)
From Eq. Al-Mod., $2,39p \leq 13,7$tf. Solving this expression for p leads to
$p = 5,73$ tf (12.63 kips) $\geq$ 5,12 tf (11.29 kips) No check.

3. Second approximation

   In order to solve Eq. A1-Mod., a value must be assumed for p. Assume p = 5,75tf (12.68 kips)

   From Eq. A1-Mod., p = 5,50tf(12.13 kips) < 5,75 tf (12.68 kips)

   From Eq. A2-Mod., p = 9,07tf(20.0 kips)

   The lower value of p obtained from the first relation governs for the flexural buckling mode.

II. Torsional-Flexural Buckling (Axis X-X)

Since the axial load is located on the side of centroid opposite from that of the shear center, the allowable load for torsional buckling is calculated in accordance with one of the equations A.1 and A.2 of Table 2.

With KL_x = 4.5 m (14,76 ft), P_x^{FT}= 4,84 tf (10.67 kips)

With KL_x/r_x = 61,7, $P_x^E = AF'_{ex}$ = 48,2tf (106.26 kips)

In accordance with Section D.2e of the Commentary on the 1968 Specification,
$\underline{P}^T = A.F_{a2}$, P^T = 5,07tf (11,18 kips).
$\bar{\eta}$ = 0,0466, ϕ_1 = 69,7tf (153.66 kips), ϕ_2= 31,7tf (68.98 kips)
Then $^+P_x^{FT}$= 2,38tf (5,25 kips).

Compariosn of this allowable load with that obtained from flexural beam-column failure (p = 5,50tf) indicates that the torsional-flexure failure mode is critical and hence the allowable load is p = 2,38tf (5.25 kips)

CONCLUSIONS

The convenience of the format adopted in the tables for the design of cold formed members has been reflected by the chilean experience in applying the Manual ICHA (6) since 1976.

In addition, this format will provide an adequate way for the transition to the next generation of structural design specifications, i.e. Load and Resistance Factor Design (LRFD).

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Appendix I. References

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2. American Iron and Steel Institute. Specification for the Design of Cold-Formed Steel Structural Members, September 3, 1980, 47 pp.

3. American Iron and Steel Institute. The New and the Old Specification for the Design of Cold-Formed Steel Structural Members. Report SG 80-1, Sept. 3, 1980, AISI, 110 pp.

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6. Instituto Chileno del Acero. Manual de Diseño de Estructuras de Acero. Segunda Edicion, ICHA, Santiago 1976, 947 pp.

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Appendix II. - Notation

A = Full unreduced cross-sectional area of the member, in^2.

B = Width of compressed flange, in.

B_x, B_y = Bending factor with respect to the X-X axis and Y-Y axis, respectively.

C_b = Bending coefficient dependent on moment gradient

C_c = Column slenderness ratio dividing elastic and inelastic buckling

C_m = End moment coefficient in interaction formula

C_{TF} = End moment coefficient in interaction formula

d = Depth of section, in.

E = Modulus of elasticity of steel, ksi

e = Eccentricity of the axial load with respect to the centroidal axis, in.

F_b = Maximum bending stress in compression that is permitted where bending stress only exits, ksi

F_y = Yield point, ksi

I_{yc} = Moment of inertia of the compressive portion of a section about the gravity axis of the entire section parallel to the web, in^4.

K = Effective length factor

K_a = Bending coefficient dependent upon moment gradient

L = Span length, ft; unbraced length of column, ft.

L_m = Distance between lateral supports of compression flange of a beam, ft.

L_u = Maximum unbraced length of the compression flange at which the allowable bending stress may be taken at 0.60 F_yQ_s, ft.

M = Allowable bending moment permitted if bending stress only exists, kips-in.

M_1 = Smaller end moment, kips-in.

M_2 = Larger end moment, kips-in.

m = Working bending moment, kips-in.

P = Allowable axial load, kips;

Left top subscripted

+ = The point of application of the eccentric load is located at the side of the centroid opposite from that of the shear center ($e > 0$)

- = The point of application of the eccentric load is between the shear center and the centroid ($e < 0$)

x_o = The point of application of the eccentric load is located at the shear center ($e = x_o$)

Right top subscripted

T = Indicate torsional buckling mode

FT = Indicate flexural-torsional buckling mode

F = Indicate flexural buckling mode

E = Indicate flexural buckling mode under Euler load (AF'_e)

Right bottom subscripted

x = The major axis for shapes not subjected to torsional flexural buckling mode

The symmetry axis for shapes subjected to torsional-flexural buckling mode

y = The minor axis for shapes not subjected to torsional-flexural buckling mode

The perpendicular axis to the axis of symmetry for shapes subjected to torsional-flexural buckling mode

p = Applied working axial load, kips

Q = Stress and/or area factor to modify allowable axial stress

r_a = Radius of gyration involving the St. Venant torsional rididity

S_{xc} = Compression section modulus of entire section about major axis

$(S_x)_{ef}$, $(S_y)_{ef}$ = Effective elastic modulus. in.^{3}