American Iron and Steel Institute

Appendix 1

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

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PREFACE

This Appendix provides alternative design procedures to portions of the *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G, and Appendices A through C (herein referred to as the main *Specification*). The Direct Strength Method detailed in this Appendix requires determination of the elastic buckling behavior of the member, and then provides a series of nominal strength [resistance] curves for predicting the member strength based on the elastic buckling behavior. The procedure does not require effective width calculations, nor iteration, and instead uses gross properties and the elastic buckling behavior of the cross-section to predict the strength. The applicability of these provisions is detailed in the General Provisions of this Appendix.

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APPENDIX 1: Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

1.1 GENERAL PROVISIONS

1.1.1 Applicability

The provisions of this Appendix are applicable for determination of the nominal axial (P_n) and flexural (M_n) strengths of cold-formed steel members. Sections 1.2.1 and 1.2.2 present a method applicable to all cold-formed steel beams and columns. Those members meeting the geometric and material limitations of Section 1.1.1.1 for columns and section 1.1.1.2 for beams have been pre-qualified for use, and the calibrated safety factor, Ω, and resistance factor, ϕ, given in 1.2.1 and 1.2.2 apply. Other beams and columns shall be permitted to use the provisions of Sections 1.2.1 and 1.2.2, but the standard Ω and ϕ factors for rational analysis (Section A1.1(b) of the main *Specification**) apply.

Currently, the Direct Strength Method provides no explicit provisions for members in tension, shear, combined bending and shear, web crippling, combined bending and web crippling, or combined axial load and bending (beam-columns). Further, no provisions are given for structural assemblies or connections and joints. As detailed in main *Specification* Section A1.1, the provisions of the main *Specification*, when applicable, shall be used for all cases listed above.

For members or situations to which the main *Specification* is not applicable, obvious extensions to the Direct Strength Method of this Appendix may exist. Users who choose to employ such extensions to the Direct Strength Method are subject to the same provisions as any other rational analysis procedure as detailed in Section A1.1(b) of the main *Specification*: (1) applicable provisions of the main *Specification* must be followed when they exist, and (2) increased Ω factors and reduced ϕ factors are employed for strength when rational analysis is conducted.

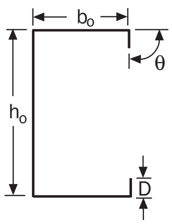
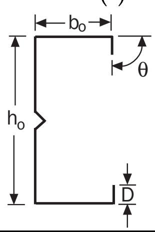
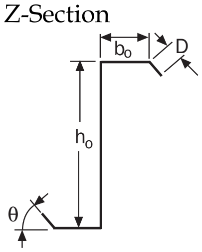
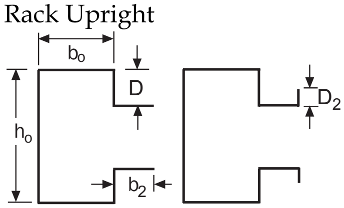
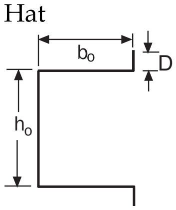
Note:

* The *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G and Appendices A through C is herein referred to as the main *Specification*.

1.1.1.1 Pre-qualified Columns

Unperforated columns that fall within the geometric and material limitations given in Table 1.1.1-1 shall be permitted to be designed using the safety factor, Ω, and resistance factor, ϕ, defined in Section 1.2.1.

Table 1.1.1-1 Limits for Pre-qualified Columns[1]

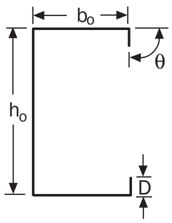
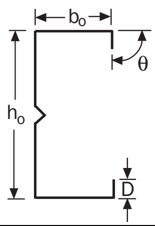
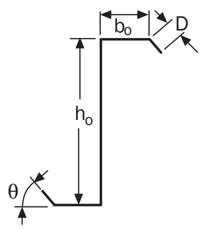
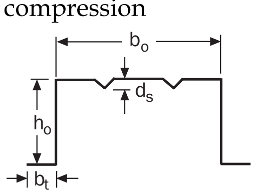
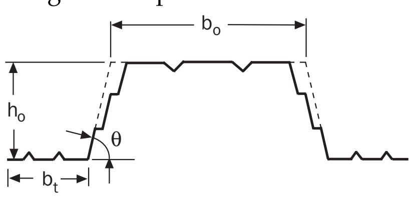
Lipped C-Section 	$h_o/t < 472$ $b_o/t < 159$ $4 < D/t < 33$ $0.7 < h_o/b_o < 5.0$ $0.05 < D/b_o < 0.41$ $\theta = 90$ deg $E/F_y > 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm²)]
Lipped C-Section with Web Stiffener(s) 	$h_o/t < 489$ $b_o/t < 160$ $6 < D/t < 33$ $1.3 < h_o/b_o < 2.7$ $0.05 < D/b_o < 0.41$ one or two intermediate stiffeners $E/F_y > 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm²)]
Z-Section 	$h_o/t < 137$ $b_o/t < 56$ $0 < D/t < 36$ $1.5 < h_o/b_o < 2.7$ $0.00 < D/b_o < 0.73$ $\theta = 50$ deg $E/F_y > 590$ [$F_y < 50$ ksi (345 MPa or 3520 kg/cm²)]
Rack Upright 	$h_o/t < 51$ $b_o/t < 22$ $5 < D/t < 8$ $2.1 < h_o/b_o < 2.9$ $1.6 < b_2/D < 2.0$ (b_2 = stiffener parallel to b_o) $D_2/D = 0.3$ (D_2 = second lip parallel to D) $E/F_y = 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm²)]
Hat 	$h_o/t < 50$ $b_o/t < 20$ $4 < D/t < 6$ $1.0 < h_o/b_o < 1.2$ $D/b_o = 0.13$ $E/F_y > 428$ [$F_y < 69$ ksi (476 MPa or 4850 kg/cm²)]

(1) $r/t < 10$, where r is the centerline bend radius
b_o = overall width
D = overall lip depth
t = base metal thickness
h_o = overall depth

1.1.1.2 Pre-qualified Beams

Unperforated beams that fall within the geometric and material limitations given in Table 1.1.1-2 shall be permitted to be designed using the safety factor, Ω, and resistance factor, ϕ, defined in Section 1.2.2.

Table 1.1.1-2 Limitations for Pre-qualified Beams[1]

C-Sections 	$h_o/t < 321$ $b_o/t < 75$ $0 < D/t < 34$ $1.5 < h_o/b_o < 17.0$ $0 < D/b_o < 0.70$ 44 deg < θ < 90 deg $E/F_y > 421$ [$F_y < 70$ ksi (483 MPa or 4920 kg/cm²)]
Lipped C-Sections with Web Stiffener 	$h_o/t < 358$ $b_o/t < 58$ $14 < D/t < 17$ $5.5 < h_o/b_o < 11.7$ $0.27 < D/b_o < 0.56$ θ = 90 deg $E/F_y > 578$ [$F_y < 51$ ksi (352 MPa or 3590 kg/cm²)]
Z-Sections 	$h_o/t < 183$ $b_o/t < 71$ $10 < D/t < 16$ $2.5 < h_o/b_o < 4.1$ $0.15 < D/b_o < 0.34$ 36 deg < θ < 90 deg $E/F_y > 440$ [$F_y < 67$ ksi (462 MPa or 4710 kg/cm²)]
Hats (Decks) with stiffened flange in compression 	$h_o/t < 97$ $b_o/t < 467$ $0 < d_s/t < 26$ (depth of stiffener) $0.14 < h_o/b_o < 0.87$ $0.88 < b_o/b_t < 5.4$ $0 < n \le 4$ (number of compression flange stiffeners) $E/F_y > 492$ [$F_y < 60$ ksi (414 MPa or 4220 kg/cm²)]
Trapezoids (Decks) with stiffened flange in compression 	$h_o/t < 203$ $b_o/t < 231$ $0.42 < (h_o/\sin\theta)/b_o < 1.91$ $1.10 < b_o/b_t < 3.38$ $0 < n_c \le 2$ (number of compression flange stiffeners) $0 < n_w \le 2$ (number of web stiffener/folds) $0 < n_t \le 2$ (number of tension flange stiffeners) 52 deg < θ < 84 deg (angle between web and horizontal plane) $E/F_y > 310$ [$F_y < 95$ ksi (655 MPa or 6680 kg/cm²)]

(1) $r/t < 10$, where r is the centerline bend radius.
See section 1.1.1.1 for definitions of other variables given in Table 1.1.1-2.

1.1.2 Elastic Buckling

Analysis is required for determination of the elastic buckling loads and or moments used in this Appendix. For columns this includes the local, distortional and overall buckling loads: $P_{cr\ell}$, P_{crd}, and P_{cre} of Section 1.2.1. For beams this includes the local, distortional and overall buckling moments: $M_{cr\ell}$, M_{crd}, and M_{cre} of Section 1.2.2. For a given column or beam all three modes may not exist. In this case, the non-existent mode shall be ignored in the calculations of Sections 1.2.1 and 1.2.2. The commentary to this Appendix provides guidance on appropriate analysis procedures for elastic buckling determination.

1.1.3 Serviceability Determination

The bending deflection at any moment (M) due to nominal loads, shall be permitted to be determined by reducing the gross moment of inertia, I_g, to an effective moment of inertia for deflection, as given in Eq. 1.1.3-1:

$$I_{eff} = I_g(M_d/M) \le I_g \qquad (Eq.\ 1.1.3\text{-}1)$$

where

M_d = Nominal strength M_n defined in Section 1.2.2, but with M_y replaced by M in all formulas of Section 1.2.2.

M = Moment due to nominal loads [specified moments] on member to be considered ($M \le M_y$)

1.2 MEMBERS

1.2.1 Column Design

The nominal axial strength, P_n, is the minimum of P_{ne}, $P_{n\ell}$ and P_{nd} as given below. For columns meeting the geometric and material criteria of Section 1.1.1.1, Ω_c and ϕ_c are as follows:

USA and Mexico		Canada
Ω_c (ASD)	ϕ_c (LRFD)	ϕ_c (LSD)
1.80	0.85	0.80

For all other columns, Ω and ϕ of Section A1.1(b) apply.

1.2.1.1 Flexural, Torsional, or Torsional-Flexural Buckling

The nominal axial strength, P_{ne}, for flexural, torsional, or torsional-flexural buckling is

for $\lambda_c \le 1.5$

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}1)$$

for $\lambda_c > 1.5$

$$P_{ne} = \left(\frac{0.877}{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}2)$$

where $\lambda_c = \sqrt{P_y/P_{cre}}$ $\qquad (Eq.\ 1.2.1\text{-}3)$

$P_y = A_g F_y$ (*Eq.* 1.2.1-4)

P_{cre} = Minimum of the critical elastic column buckling load in flexural, torsional, or torsional-flexural buckling determined in accordance with Section 1.1.2

1.2.1.2 Local Buckling

The nominal axial strength, $P_{n\ell}$, for local buckling is

for $\lambda_\ell \le 0.776$

$$P_{n\ell} = P_{ne} \quad (Eq.\ 1.2.1\text{-}5)$$

for $\lambda_\ell > 0.776$

$$P_{n\ell} = \left[1 - 0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right]\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \quad (Eq.\ 1.2.1\text{-}6)$$

where $\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}}$ (*Eq.* 1.2.1-7)

$P_{cr\ell}$ = Critical elastic local column buckling load determined in accordance with Section 1.1.2

P_{ne} is defined in Section 1.2.1.1.

1.2.1.3 Distortional Buckling

The nominal axial strength, P_{nd}, for distortional buckling is

for $\lambda_d \le 0.561$

$$P_{nd} = P_y \quad (Eq.\ 1.2.1\text{-}8)$$

for $\lambda_d > 0.561$

$$P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \quad (Eq.\ 1.2.1\text{-}9)$$

where $\lambda_d = \sqrt{P_y/P_{crd}}$ (*Eq.* 1.2.1-10)

P_{crd} = Critical elastic distortional column buckling load determined in accordance with Section 1.1.2

P_y is given in Eq. 1.2.1-4.

1.2.2 Beam Design

The nominal flexural strength, M_n, is the minimum of M_{ne}, $M_{n\ell}$ and M_{nd} as given below. For beams meeting the geometric and material criteria of Section 1.1.1.2, Ω_b and ϕ_b are as follows:

USA and Mexico		Canada
Ω_b (ASD)	ϕ_b (LRFD)	ϕ_b (LSD)
1.67	0.90	0.85

For all other beams, Ω and ϕ of Section A1.1(b) apply.

1.2.2.1 Lateral-Torsional Buckling

The nominal flexural strength, M_{ne}, for lateral-torsional buckling is

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

where

M_y = S_fF_y, where S_f is the gross section modulus referenced to the extreme fiber in first yield (*Eq*. 1.2.2-4)

M_{cre} = Critical elastic lateral-torsional buckling moment determined in accordance with Section 1.1.2

1.2.2.2 Local Buckling

The nominal flexural strength, $M_{n\ell}$, for local buckling is

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq*. 1.2.2-7)

$M_{cr\ell}$ = Critical elastic local buckling moment determined in accordance with Section 1.1.2

M_{ne} is defined in Section 1.2.2.1.

1.2.2.3 Distortional Buckling

The nominal flexural strength, M_{nd}, for distortional buckling is

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq*. 1.2.2-10)

M_{crd} = Critical elastic distortional buckling moment determined in accordance with Section 1.1.2.

M_y is given in Eq. 1.2.2-4.