The nominal shear capacity (V_v) shall be calculated from Equations 3.3.4(1) to 3.3.4(3), as appropriate.

For $d_1/t_w \leq \sqrt{Ek_v/f_y}$: $V_v = 0.64 f_y d_1 t_w$ . . . 3.3.4(1)

For $\sqrt{Ek_v/f_y} < d_1/t_w \leq 1.415\sqrt{Ek_v/f_y}$: $V_v = 0.64 t_w^2 \sqrt{Ek_v f_y}$ . . . 3.3.4(2)

For $d_1/t_w > 1.415\sqrt{Ek_v/f_y}$: $V_v = \dfrac{0.905 E k_v t_w^3}{d_1}$ . . . 3.3.4(3)

where

d_1 = depth of the flat portion of the web measured along the plane of the web

t_w = thickness of web

k_v = shear buckling coefficient determined as follows:

(i) For unstiffened webs: $k_v = 5.34$

(ii) For beam webs with transverse stiffeners complying with Clause 2.7:

For $a/d_1 \leq 1.0$: $k_v = 4.00 + [5.34/(a/d_1)^2]$ . . . 3.3.4(4)

For $a/d_1 > 1.0$: $k_v = 5.34 + [4.00/(a/d_1)^2]$ . . . 3.3.4(5)

a = shear panel length for unstiffened web element; *or* distance between transverse stiffeners for stiffened web elements.

For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

3.3.5 Combined bending and shear For beams with unstiffened webs, the design bending moment (M^*) and the design shear force (V^*) shall satisfy—

$$\left(\frac{M^*}{\phi_b M_s}\right)^2 + \left(\frac{V^*}{\phi_v V_v}\right)^2 \leq 1.0$$

For beams with transverse web stiffeners, the design bending moment (M^*) shall satisfy—

$$M^* \leq \phi_b M_b$$

The design shear force (V^*) shall satisfy—

$$V^* \leq \phi_v V_v$$

If $\dfrac{M^*}{\phi_b M_s} > 0.5$; and

$$\frac{V^*}{\phi_v V_v} > 0.7$$

then M^* and V^* shall satisfy—

$$0.6\left(\frac{M^*}{\phi_b M_s}\right) + \left(\frac{V^*}{\phi_v V_v}\right) \leq 1.3$$

where

M_s = nominal section moment capacity about the centroidal axes determined in accordance with Clause 3.3.2

V_v = nominal shear capacity when shear alone exists determined in accordance with Clause 3.3.4

M_b = nominal member moment capacity when bending alone exists determined in accordance with Clause 3.3.3.

3.3.6 Bearing This Clause applies to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member and in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid failure of unstiffened flat webs of flexural members having a flat width ratio (d_1/t_w) less than or equal to 200, the design concentrated loads and reactions (R_b^*) shall satisfy—

$$R_b^* \leq \phi_w R_b$$

where

ϕ_w = capacity [strength reduction] factor for bearing (see Table 1.6)

R_b = nominal capacity for concentrated load or reaction for one solid web connecting top and bottom flanges.

The values of R_b for stiffened and unstiffened flanges, and for the appropriate type and position of loads, are given in Tables 3.3.6(1) and 3.3.6(2). Webs of flexural members for which d_1/t_w is greater than 200 shall be provided with means of transmitting concentrated loads and reactions directly into the webs.

The equations in Tables 3.3.6(1) and 3.3.6(2) apply, if—

(a) $l_b/t_w \leq 210$ and $l_b/d_1 \leq 3.5$;

(b) $r_i/t_w \leq 6$ for beams; and

(c) $r_i/t_w \leq 7$ for decking and cladding;

where

l_b = actual length of bearing. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of l_b shall be taken

t_w = thickness of web

d_1 = depth of the flat portion of the web measured along the plane of the web

r_i = inside bend radius.

For a Z-section having its flange bolted to the section's end support members, the equation for stiffened or partially stiffened flanges and $c < 1.5d_1$ in Table 3.3.6(1) may be multiplied by 1.3 if—

(i) $d_1/t \leq 150$;

(ii) $r_i/t \leq 4$;

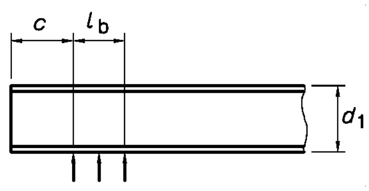
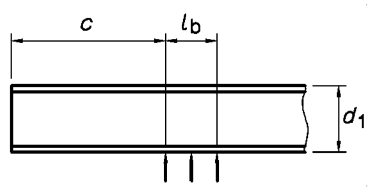
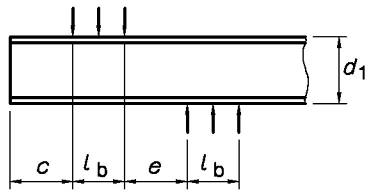
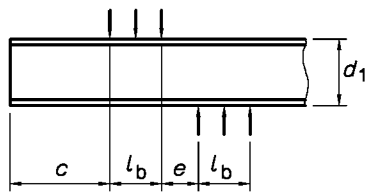
(iii) $t \geq 1.5$ mm; and

(iv) the thickness of the support member ≥ 5 mm.

For two or more webs, R_b shall be calculated for each individual web and the results added to obtain the nominal concentrated load or reaction for the multiple web. Where two webs of a beam are inclined in opposite directions, the R_b equations may be applied to such webs only if they are restrained against spreading.

For built-up I-sections, or similar sections, the distance between the web connector and beam flange shall be kept as small as practicable.

TABLE 3.3.6(1)
SHAPES HAVING SINGLE WEBS

R_b	Type and position of loads
Stiffened or partially stiffened flanges $R_b = 10.0t^2 f_y C_3 C_4 C_\theta \left[1 - 0.00183\left(\frac{d_1}{t_w}\right)\right]\left[1 + 0.01\left(\frac{l_b}{t_w}\right)\right]$	Single load or reaction— $c < 1.5d_1$ 
Unstiffened flanges $R_b = 6.55t^2 f_y C_3 C_4 C_\theta \left[1 - 0.00128\left(\frac{d_1}{t_w}\right)\right]\left[1 + 0.01\left(\frac{l_b}{t_w}\right)\right]$ (see Note 1)	
Stiffened, partially stiffened or unstiffened flanges $R_b = 16.3t^2 f_y C_1 C_2 C_\theta \left[1 - 0.00138\left(\frac{d_1}{t_w}\right)\right]\left[1 + 0.007\left(\frac{l_b}{t_w}\right)\right]$ (see Note 2)	Single load or reaction— $c > 1.5d_1$ 
Stiffened, partially stiffened or unstiffened flanges $R_b = 7.38t^2 f_y C_3 C_4 C_\theta \left[1 - 0.00235\left(\frac{d_1}{t_w}\right)\right]\left[1 + 0.01\left(\frac{l_b}{t_w}\right)\right]$	Two opposite loads or reactions— $c < 1.5d_1$ $e < 1.5d_1$ 
Stiffened, partially stiffened or unstiffened flanges $R_b = 23.3t^2 f_y C_1 C_2 C_\theta \left[1 - 0.00292\left(\frac{d_1}{t_w}\right)\right]\left[1 + 0.0013\left(\frac{l_b}{t_w}\right)\right]$	Two opposite loads or reactions— $c > 1.5d_1$ $e < 1.5d_1$ 

NOTES:

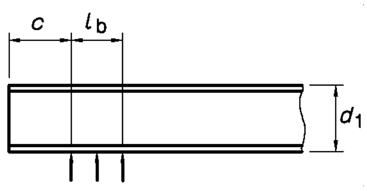
1 If $l_b/t_w > 60$, the factor $[1 + 0.01(l_b/t_w)]$ may be increased to $[0.71 + 0.015(l_b/t_w)]$.

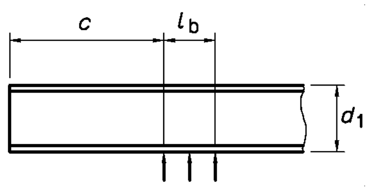
2 If $l_b/t_w > 60$, the factor $[1 + 0.007(l_b/t_w)]$ may be increased to $[0.75 + 0.011(l_b/t_w)]$.

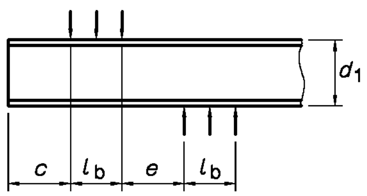
TABLE 3.3.6(2)

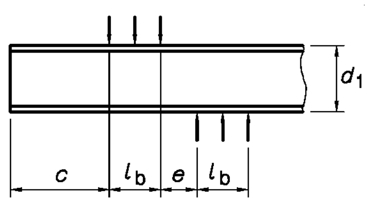
BACK-TO-BACK CHANNEL BEAMS AND BEAMS WITH RESTRAINT AGAINST WEB ROTATION

<table>
<tr><th>R_b</th><th>Type and position of loads</th></tr>
<tr><td>Stiffened, partially stiffened and unstiffened flanges

$$R_b = 10.0t^2 f_y C_7 \left[1 + 0.126\sqrt{\left(\frac{l_b}{t_w}\right)}\right]$$</td><td>Single load or reaction—
$c < 1.5d_1$

</td></tr>
<tr><td>Stiffened, partially stiffened and unstiffened flanges

$$R_b = 15.0t^2 f_y C_5 C_6 \left[1 + 0.217\sqrt{\left(\frac{l_b}{t_w}\right)}\right]$$</td><td>Single load or reaction—
$c > 1.5d_1$

</td></tr>
<tr><td>Stiffened, partially stiffened and unstiffened flanges

$$R_b = 10.0t^2 f_y C_{10} C_{11} \left[1 + 0.126\sqrt{\left(\frac{l_b}{t_w}\right)}\right]$$</td><td>Two opposite loads or reactions—
$c < 1.5d_1$
$e < 1.5d_1$

</td></tr>
<tr><td>Stiffened, partially stiffened and unstiffened flanges

$$R_b = 15.0t^2 f_y C_8 C_9 \left[1 + 0.217\sqrt{\left(\frac{l_b}{t_w}\right)}\right]$$</td><td>Two opposite loads or reactions—
$c > 1.5d_1$
$e < 1.5d_1$

</td></tr>
</table>

NOTE TO TABLES 3.3.6(1) AND 3.3.6(2): The following applies to the equations given in Tables 3.3.6(1) and 3.3.6.2:

$C_1 = (1.22 - 0.22k)$

$C_2 = \left(1.06 - \dfrac{0.06r_i}{t_w}\right) \leq 1.0$

$C_3 = (1.33 - 0.33k)$

$C_4 = \left(1.15 - \dfrac{0.15r_i}{t}\right) \leq 1.0$ but not less than 0.50

$C_5 = (1.49 - 0.53k) \geq 0.6$

$C_6 = (0.88 + 0.12m)$

$C_7 = \left[1 + \dfrac{d_1/t_w}{750}\right]$ if $d_1/t_w \leq 150$

$= 1.20$ if $d_1/t_w > 150$

$C_8 = \dfrac{1}{k}$ if $d_1/t_w \leq 66.5$

$= \left[1.10 - \dfrac{d_1/t_w}{665}\right]/k$ if $d_1/t_w > 66.5$

$C_9 = (0.82 + 0.15m)$

$C_{10} = \left[0.98 - \dfrac{d_1/t_w}{865}\right]/k$

$C_{11} = (0.64 + 0.31m)$

$C_\theta = 0.7 + 0.3(\theta/90)^2$

where

C_1 to C_{11}, and C_θ = coefficient used to determine R_b

k = non-dimensional yield stress

$= \dfrac{f_y}{228}$

m = non-dimensional thickness

$= \dfrac{t}{1.905}$

θ = angle between the plane of the web and the plane of the bearing surface. θ shall be within the following limits:

$90° \geq \theta \geq 45°$.

3.3.7 Combined bending and bearing Unstiffened flat webs of shapes subjected to a combination of bending and reaction or concentrated load shall be designed as follows:

(a) *Shapes having single unstiffened webs* Shapes having single unstiffened webs shall satisfy—

$$1.07\left(\frac{R^*}{\phi_w R_b}\right) + \left(\frac{M^*}{\phi_b M_s}\right) \leq 1.42$$

At the interior supports of continuous spans, the above interaction is not applicable to deck or beams with two or more single webs, where the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 250 mm.

(b) *Back-to-back channel beams and beams with restraint against web rotation* Back-to-back channel beams and beams with restraint against web rotation, such as I-sections made by welding two angles to a channel, shall satisfy—

$$0.82\left(\frac{R^*}{\phi_w R_b}\right) + \left(\frac{M^*}{\phi_b M_s}\right) \leq 1.32$$

If $d_1/t_w \leq 2.33/\sqrt{(f_y/E)}$ and $\lambda \leq 0.673$, the nominal concentrated load or reaction strength may be determined in accordance with Clause 3.3.6.

In Items (a) and (b), the following applies:

R^* = design concentrated load or reaction in the presence of bending moment

R_b = nominal capacity for concentrated load or reaction in the absence of bending moment determined in accordance with Clause 3.3.6

M^* = design bending moment at, or immediately adjacent to, the point of application of the design concentrated load or reaction (R^*)

M_s = nominal section moment capacity about the centroidal axes determined in accordance with Clause 3.3.1, excluding Clause 3.3.3.2

b_f = flat width of the beam flange which contacts the bearing plate

t_w = thickness of the web

λ = slenderness ratio (see Clause 2.2.1.2).

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.1 General This Clause applies to members in which the resultant of all loads acting on the member is an axial load passing through the centroid of the effective section calculated at the critical stress (f_n). The design compressive axial force (N^*) shall satisfy the following:

(a) $N^* \leq \phi_c N_s$

(b) $N^* \leq \phi_c N_c$

where

ϕ_c = capacity [strength reduction] factor for members in compression (see Table 1.6)

N_s = nominal section capacity of the member in compression

$= A_e f_y$ . . . 3.4.1(1)

A_e = effective area at yield stress (f_y)

N_c = nominal member capacity of the member in compression

$= A_e f_n$ . . . 3.4.2(2)

A_e = effective area at the critical stress (f_n). For sections with circular holes, A_e shall be determined in accordance with Clause 2.2.2.2. If the product of the number of holes in the effective length region and the hole diameter divided by the effective length does not exceed 0.015, A_e can be determined ignoring the holes.

f_n = critical stress and shall be determined from Equation 3.4.1(3) or Equation 3.4.1(4) as appropriate.

For $\lambda_c \leq 1.5$: $f_n = (0.658^{\lambda_c^2}) f_y$ . . . 3.4.1(3)

For $\lambda_c > 1.5$: $f_n = (0.877/\lambda_c^2) f_y$ . . . 3.4.1(4) where

λ_c = non-dimensional slenderness used to determine f_n

$$= \sqrt{\frac{f_y}{f_{oc}}} \qquad \ldots 3.4.1(5)$$

f_{oc} = least of the elastic flexural, torsional and torsional-flexural buckling stress determined in accordance with Clauses 3.4.1 to 3.4.4, or a rational elastic buckling analysis.

Angle sections shall be designed for the design axial force (N^*) acting simultaneously with a moment equal to $N^*l/1000$ applied about the minor principal axis causing compression in the tips of the angle legs.

NOTE: The slenderness ratio (l_e/r) of all compression members should not exceed 200, except that during construction only, l_e/r should not exceed 300.

3.4.2 Sections not subject to torsional or flexural-torsional buckling For doubly-symmetric sections, closed cross-sections and any other sections that can be shown not to be subject to torsional or flexural-torsional buckling, the elastic flexural buckling stress (f_{oc}) shall be determined from Equation 3.4.2 as follows:

$$f_{oc} = \frac{\pi^2 E}{(l_e/r)^2} \qquad \ldots 3.4.2$$

where

l_e = effective length of member

r = radius of gyration of the full, unreduced cross-section.

NOTES:

1 In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length (l_e) for compression members which do not depend upon their own bending stiffness for lateral stability of the frame or truss, should be taken as equal to the unbraced length (l), unless analysis shows that a smaller value may be used.

2 In a frame which depends upon its own bending stiffness for lateral stability, the effective length (l_e) of the compression members should be determined by a rational method and should not be less than the actual unbraced length.

3.4.3 Doubly- or singly-symmetric sections (see Figures 1.5(a) and (c)) subject to torsional or flexural-torsional buckling For sections subject to torsional or flexural-torsional buckling, f_{oc} shall be taken as the smaller of f_{oc} calculated in accordance with Clause 3.4.2 and f_{oc} calculated from Equation 3.4.3(1) as follows:

$$f_{oc} = \frac{1}{2\beta}\left[(f_{ox} + f_{oz}) - \sqrt{(f_{ox} + f_{oz})^2 - 4\beta f_{ox} f_{oz}}\right] \qquad \ldots 3.4.3(1)$$

Alternatively, a conservative estimate of f_{oc} can be obtained from Equation 3.4.3(2) as follows:

$$f_{oc} = f_{oz} f_{ox}/(f_{oz} + f_{ox}) \qquad \ldots 3.4.3(2)$$

where f_{oz} and f_{ox} are specified in Clause 3.3.3.2(a).

$$\beta = 1 - (x_o/r_{o1})^2 \qquad \ldots 3.4.3(3)$$

For singly-symmetric sections, the x-axis shall be assumed to be the axis of symmetry.

3.4.4 Point-symmetric sections (see Figure 1.5(b)) For point symmetric sections subject to flexural or torsional buckling, f_{oc} shall be taken as the smaller of f_{oc} calculated in accordance with Clause 3.4.2 and f_{oz} specified in Clause 3.3.3.2(a).

3.4.5 Non-symmetric sections (see Figure 1.5(d)) For shapes whose cross-sections do not have any symmetry, either about an axis or about a point, f_{oc} shall be taken as the smallest value which shall satisfy cubic Equation 3.4.5 as follows:

$$f_{oc}^3(r_{o1}^2 - x_o^2 - y_o^2) - f_{oc}^2[r_{o1}^2(f_{ox} + f_{oy} + f_{oz}) - (f_{oy}x_o^2 + f_{ox}y_o^2)]$$
$$+ f_{oc}r_{o1}^2(f_{ox}f_{oy} + f_{oy}f_{oz} + f_{ox}f_{oz}) - (f_{ox}f_{oy}f_{oz}r_{o1}^2) = 0 \qquad \ldots 3.4.5$$

Alternatively, compression members composed of such shapes may be tested in accordance with Clause 6.2.

3.4.6 Singly-symmetric sections (see Figure 1.5(c)) subject to distortional buckling For monosymmetric sections subject to distortional buckling, such as lipped channels with additional rear flanges, the value of N_c in Equation 3.4.1(2) shall be the lesser of the following:

(a) $A_e f_n$ calculated in accordance with Equations 3.4.1(3) and 3.4.1(4).

(b) For $f_{od} > \frac{f_y}{2}$:

$$Af_n = Af_y\left(1 - \frac{f_y}{4f_{od}}\right) \qquad \ldots 3.4.6(1)$$

For $\frac{f_y}{13} \leq f_{od} \leq \frac{f_y}{2}$:

$$Af_n = Af_y\left[0.055\left(\sqrt{\frac{f_y}{f_{od}}} - 3.6\right)^2 + 0.237\right] \qquad \ldots 3.4.6(2)$$

f_{od} may be calculated using either the appropriate equations given in Appendix D or a rational elastic buckling analysis. A is the area of the full cross-section.

3.4.7 Columns with one flange through-fastened to sheeting This Clause applies to channel or Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to sheeting by screw fasteners.

The nominal member capacity (N_c) in axial compression of simple span or continuous channels or Z-sections shall be determined from Equation 3.4.7 as follows:

(a) *For weak axis*—

$$N_c = (0.79s + 0.54)(0.046t + 0.93)(0.1b_f - 0.064d + 22.8)\frac{EA}{29500} \qquad \ldots 3.4.7$$

where

s = fastener distance from the centre-line of the web divided by the flange width for Z-sections; *or*
flange width minus the fastener distance from the centre-line of the web divided by the flange width for channel sections

t = thickness of the channel or Z-section

b_f = flange width of the channel or Z-section

d = depth of the channel or Z-section

A = gross cross-sectional area of the channel or Z-section.

NOTE: Units of t, b_f and d in Equation 3.4.7 are in millimetres, since Equation 3.4.7 is not non-dimensional.

Equation 3.4.7 shall be limited to roof and wall systems complying with the following:

(i) Channel and Z-sections not exceeding 3.2 mm in thickness.

(ii) Channel and Z-sections with depths of 150 to 300 mm.

(iii) Flanges are edge stiffened compression elements.

(iv) $70 \leq$ depth/thickness ≤ 170.

(v) $2.8 \leq$ depth/flange width < 5.

(vi) $16 \leq$ flat width/thickness of flange < 50.

(vii) Both flanges prevented from moving laterally at the supports.

(viii) Roof or wall panels with fasteners spaced 300 mm on centre or less and having a minimum rotational lateral stiffness of 10.3 kN/mm^2 (fastener at mid-flange width as determined by the relevant AISI test procedure. (See Note.)

(ix) Minimum yield stress of 230 MPa.

(x) Span lengths from 4.5 to 9 m.

(b) *For strong axis*—the equations given in Clauses 3.4.1 and 3.4.2 shall be used.

NOTE: See the test procedure titled 'Rational-lateral stiffness test method for beam-to-panel assemblies' in the AISI Specification.

3.5 COMBINED AXIAL LOAD AND BENDING

3.5.1 Combined axial compressive load and bending The design axial compressive load (N^*), and the design bending moments (M_x^* and M_y^*) about the x- and y-axes of the effective section, respectively, shall satisfy the following:

(a) $$\frac{N^*}{\phi_c N_c} + \frac{C_{mx} M_x^*}{\phi_b M_{bx} \alpha_{nx}} + \frac{C_{my} M_y^*}{\phi_b M_{by} \alpha_{ny}} \leq 1.0$$

(b) $$\frac{N^*}{\phi_c N_s} + \frac{M_x^*}{\phi_b M_{bx}} + \frac{M_y^*}{\phi_b M_{by}} \leq 1.0$$

If $N^*/\phi_c N_c \leq 0.15$, the following interaction may be used in lieu of Items (a) and (b):

$$\frac{N^*}{\phi_c N_c} + \frac{M_x^*}{\phi_b M_{bx}} + \frac{M_y^*}{\phi_b M_{by}} \leq 1.0$$

where

N_c = nominal member capacity of the member in compression determined in accordance with Clause 3.4

C_{mx}, C_{my} = coefficients for unequal end moment whose value shall be taken as follows:

(i) For compression members in frames subject to joint translation (side-sway):

$$C_m = 0.85$$

(ii) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending:

$$C_m = 0.6 - 0.4(M_1/M_2) \qquad \ldots 3.5.1(1)$$

M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive if the member is bent in reverse curvature and negative if it is bent in single curvature.

(iii) For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

(A) For members whose ends are restrained:

$C_m = 0.85$

(B) For members whose ends are unrestrained:

$C_m = 1.0$

M_x^*, M_y^* = design bending moment about the *x*- and *y*-axes of the effective section, respectively, determined for the design axial force alone. For angle sections, M_y^* shall be taken either as the design bending moment, or the required flexural moment plus $N^* l/1000$, whichever results in a lower value of N_c

M_{bx}, M_{by} = nominal member moment capacity about the *x*- and *y*-axes, respectively, determined in accordance with Clause 3.3.3

ϕ_b = capacity [strength reduction] factor for bending

= 0.95 and 0.90 for bending strength (see Table 1.6); *or*
0.90 for laterally unbraced beam (see Table 1.6)

ϕ_c = capacity [strength reduction] factor for members in compression

= 0.85

N_s = nominal section capacity of the member in compression determined in accordance with Clause 3.4, with f_n equal to f_y

α_{nx}, α_{ny} = moment amplification factors

$$= 1 - \left(\frac{N^*}{N_e}\right) \qquad \ldots 3.5.1(2)$$

N_e = elastic buckling load

$$= \pi^2 E I_b/(l_{eb})^2 \qquad \ldots 3.5.1(3)$$

I_b = second moment of area of the full, unreduced cross-section about the bending axis

l_{eb} = effective length in the plane of bending.