Steelwork Design Guide to BS 5950-1: 2000

Volume 1
Section Properties
Member Capacities

7th Edition

The Steel Construction Institute

and

The British Constructional Steelwork Association Limited

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Guide to BS 5950-1: 2000

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Section Properties

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FOREWORD

Seventh Edition

The first edition of this Design Guide was published in 1985; it was revised in 1987 (2nd Edition), in 1992 (3rd Edition), in 1996 (4th Edition), in 1997 (5th edition) and in 2001 (6^{th} edition). It is a basic working tool for users of BS 5950-1 *Structural use of steelwork in building – Code of practice for design – Rolled and welded sections*, which was first published in 1985, revised in 1990 and in 2000.

This edition is a reprint of the 6^{th} edition, in which the section range has been updated. The tables are dual titled with the BS designation as well as the Advance (open sections), Celsius® (hot finished hollow sections) or Hybox® (cold formed hollow sections) branding.

The following structural sections are covered in this publication:

- Universal beams, Universal columns, joists, bearing piles, parallel flange channels, and structural tees cut from Universal beams and Universal columns to BS 4-1
- ASB (asymmetric beams) *Slimdek®* beam produced by Corus (see Corus brochure, *Advance™ sections, 09/2006)*)
- Equal and Unequal angles to BS EN 10056-1
- Hot-finished structural hollow sections to BS EN 10210-2
- Cold-formed structural hollow sections to BS EN 10219-2

Section ranges listed are intended to be in line with sections that are readily available at the time of printing. Some sections, which are readily available but not listed in the above standards, are also included; these sections are designated by "+" in the tables.

The editorial work for the 7th Edition was carried out by Miss E Nunez Moreno, based on the 6^{th} edition produced by Mr A Way and Mr A S Malik, with technical assistance from Mr C M King of the SCI and Mr J C Taylor, formerly of the SCI. The 6^{th} edition was coordinated by Mr D G Brown, of the SCI, and Mr P J Williams, formerly of the BCSA.

This publication has been jointly funded by SCI and BCSA. Acknowledgement is also given to Corus for its support for the work leading to the preparation of the initial formulae on which the capacity tables are based.

The major differences between the 7th Edition (2007) and the 6th Edition (2001) are:

- Branding for Advance, Celsius® and Hybox® sections has now been included.
- Capacity tables for Hot Finished and Cold Formed Hollow Sections are only given for S 355 steel grade and not for S 275.
- Hot Finished Elliptical Hollow Sections have now been included in the section dimensions and property tables.
- Castellated universal beams and universal columns have been omitted.
- Product standards have been updated as shown in the table below.

	6th Edition	7th Edition
Open sections	BS 4-1: 1993	BS 4-1: 2005
Hot Finished Hollow Sections	BS EN 10210-2: 1997	BS EN 10210-2: 2006
Cold Formed Hollow Sections	BS EN 10219-2: 1997	BS EN 10219-2: 2006

Previous Editions

1st Edition, 1985
Based on the following codes:
BS 5950: Part 1: 1985, BS 4360: 1979 including Amd 1 and Amd 2.
UB, UC, joists, bearing piles and channels to BS4: Part 1: 1980 including Amd 1 and 2.
Structural hollow sections to BS 4848: Part 2: 1975.
Angles to BS 4848: Part 4: 1972 including Amd 1 and Amd 2.
Section properties calculated from imperial dimensions and then converted to metric.

2nd Edition, 1987
An update of 1st Edition including changes due to:
BS 4360: 1986.
UB, joists to BS4: Part 1: 1980 Amd 3.
Angles to BS 4848: Part 4: 1972 Amd 3.
Addendum No.1 to 2nd Edition, 1990.
Section properties and member capacities for a new range of additional hollow sections.

3rd Edition, 1992
Combination of 2nd Edition and Addendum No.1. plus changes due to:
BS 5950: Part 1: 1990 Including Amd 1. BS 4360: 1990 and BS EN 10025: 1990.
UB, UC, joists, bearing piles and channels to BS4: Part 1: 1980 including Amd 1 to 5. Structural hollow sections to BS 4848: Part 2: 1991.
Angles to BS 4848: Part 4 including Amd 1 to 3.
In addition to minor corrections, the major alterations in the 3rd Edition were as follows:
Section properties based on true metric dimensions instead of metric equivalent of inch dimensions.
Member capacity tables for struts made of angles and channels revised due to code changes.
Combined axial load and bending tables presented in a more user-friendly format with separate checks for local capacity and overall buckling.

3rd Edition, (Reprinted) 1993 (Reprinted with minor corrections)
Supplement A to 3rd Edition, 1994
Section properties and member capacities for UB, UC, bearing piles, joists and associated tees and castellated sections using the dimensions from BS 4: Part 1: 1993. Additional information such as formulae for reduced plastic modulus, warping constant (H) and torsion constant (J).

3rd Edition (Reprinted) 1994 Reprinted with reference to Supplement A.

4th Edition, 1996
Combination of 3rd edition (reprint) 1994 and Supplement A, 1994.
Minor changes in Explanatory notes and Table headings for clarity and consistency.
Summary of sections covered:
UB's, UC's, joist and bearing piles to BS 4: Part 1: 1993 (as per Supplement A).
Channels to BS 4: Part 1: 1993 (as per 3rd Edition).
Structural hollow sections to BS 4848: Part 2: 1991 (as per 3rd Edition).
Angles to BS 4848: Part 4 1972 (as per 3rd Edition).

5th Edition, 1997
Combination of 4th edition, 1996 (except for structural hollow sections to BS 4848: Part 2) and new tables for structural hollow sections to EN 10210-2
Minor changes for clarity and consistency in Explanatory Notes and under the tables for structural hollow sections.
Summary of sections covered:
UB's, UC's, joist and bearing piles to BS 4: Part 1: 1993 (as per 4th Edition)
Channels to BS 4: Part 1: 1993 (as per 4th Edition).
Angles to BS 4848: Part 4: 1972 (as per 4th Edition).
Structural hollow sections to EN 10210: Part 2: 1997.

6th Edition, 2001
Complete revision of the 5th edition, 1997, updating the information with the standard BS 5950-1: 2000. The formats of the tables were changed to suit the amendments to that standard.
Summary of sections covered:
UB's, UC's, joist, bearing piles and parallel flange channels and structural tees cut from universal beams and universal columns to BS 4: Part 1: 1993.
Castellated Universal beams and columns.
ASB (asymmetric beams) *Slimdek*® beam produced by Corus.
Angles to BS 4848: Part 4: 1972 (as per 4th Edition).
Hot Finished Structural hollow sections to EN 10210: Part 2: 1997 and Cold Formed Structural Hollow Sections to EN 10219-2: 1997.

CONTENTS

A EXPLANATORY NOTES

1 GENERAL

This publication is a design guide to BS 5950: Part 1 now denoted as BS 5950-1: 2000[1]. The symbols used are generally the same as those in BS 5950-1: 2000.

1.1 Material, section dimensions and tolerances

The structural sections referred to in this design guide are of weldable structural steels conforming to the relevant British Standards given in the table below:

Table – Structural steel products

Product	Technical delivery requirements		Dimensions	Tolerances
	Non alloy steels	**Fine grain steels**		
Universal beams, Universal columns, and Universal bearing piles	BS EN 10025[2]	BS EN 10113-1[3]	BS 4-1[4]	BS EN 10034[5]
Joists			BS 4-1[4]	BS 4-1[4] BS EN 10024[6]
Parallel Flange Channels			BS 4-1[4]	BS EN 10279[7]
Angles			BS EN 10056-1[8]	BS EN 10056-2[8]
Structural tees cut from universal beams and universal columns			BS 4-1[4]	—
ASB (asymmetric beams) *Slimdek*® beam	Generally BS EN 10025[2], but see note b)		See note a)	Generally BS EN 10034[5], but also see note b)
Hot Finished Hollow Sections	BS EN 10210-1[9]		BS EN 10210-2[9]	BS EN 10210-2[9]
Cold Formed Hollow Sections	BS EN 10219-1[10]		BS EN 10219-2[10]	BS EN 10219-2[10]
Notes: For full details of the British Standards, see the reference list at the end of the Explanatory Notes. a) See Corus publication[11]. b) For further details consult Corus.				

1.2 Dimensional units

The dimensions of sections are given in millimetres (mm).

1.3 Property units

Generally, the centimetre (cm) is used for the calculated properties but for surface areas and for the warping constant (H), the metre (m) and the decimetre (dm) respectively are used.

Note: 1 dm = 0.1 m = 100 mm

$1\ \text{dm}^6 = 1 \times 10^{-6}\ \text{m}^6 = 1 \times 10^{12}\ \text{mm}^6$

1.4 Mass and force units

The units used are the kilogram (kg), the Newton (N) and the metre per second (m/s^2) so that $1\ N = 1\ kg \times 1\ m/s^2$. For convenience, a standard value of the acceleration due to gravity has been generally accepted as $9.80665\ m/s^2$. Thus, the force exerted by 1 kg under the action of gravity is 9.80665 N and the force exerted by 1 tonne (1000 kg) is 9.80665 kilonewtons (kN).

2 DIMENSIONS OF SECTIONS

2.1 Masses

The masses per metre have been calculated assuming that the density of steel is $7850\ kg/m^3$.

In all cases, including compound sections, the tabulated masses are for the steel section alone and no allowance has been made for connecting material or fittings.

2.2 Ratios for local buckling

The ratios of the flange outstand to thickness (b/T) and the web depth to thickness (d/t) are given for I, H and channel sections. The ratios of the outside diameter to thickness (D/t) are given for circular hollow sections. The ratios d/t and b/t are also given for square and rectangular hollow sections. All the ratios for local buckling have been calculated using the dimensional notation given in Figure 5 of BS 5950-1: 2000 and are for use when element and section class are being checked to the limits given in Tables 11 and 12 of BS 5950-1: 2000.

2.3 Dimensions for detailing

The dimensions C, N and n have the meanings given in the figures at the heads of the tables and have been calculated according to the formulae below. The formulae for N and C make allowance for rolling tolerances, whereas the formulae for n make no such allowance.

2.3.1 Universal beams, universal columns and bearing piles

$N = (B - t)/2 + 10$ mm (rounded to the nearest 2 mm above)

$n = (D - d)/2$ (rounded to the nearest 2 mm above)

$C = t/2 + 2$ mm (rounded to the nearest mm)

2.3.2 Joists

$N = (B - t)/2 + 6$ mm (rounded to the nearest 2 mm above)

$n = (D - d)/2$ (rounded to the nearest 2 mm above)

$C = t/2 + 2$ mm (rounded to the nearest mm)

Note: Flanges of BS 4-1 joists have an 8° taper.

2.3.3 Parallel flange channels

$N = (B - t) + 6$ mm (rounded up to the nearest 2 mm above)

$n = (D - d)/2$ (taken to the next higher multiple of 2 mm)

$C = t + 2$ mm (rounded up to the nearest mm)

3 SECTION PROPERTIES

3.1 General

All section properties have been accurately calculated and rounded to three significant figures. They have been calculated from the metric dimensions given in the appropriate standards (see Section 1.2). For angles, BS EN 10056-1 assumes that the toe radius equals half the root radius.

3.2 Sections other than hollow sections

3.2.1 Second moment of area (*I*)

The second moment of area of the section, often referred to as moment of inertia, has been calculated taking into account all tapers, radii and fillets of the sections.

3.2.2 Radius of gyration (*r*)

The radius of gyration is a parameter used in buckling calculation and is derived as follows:

$$r = [I / A]^{1/2}$$

where:

A is the cross-sectional area.

3.2.3 Elastic modulus (*Z*)

The elastic modulus is used to calculate the elastic moment capacity based on the design strength of the section or the stress at the extreme fibre of the section from a known moment. It is derived as follows:

$$Z = I / y$$

where:

y is the distance to the extreme fibre of the section from the elastic neutral axis.

The elastic moduli of the tee are calculated at the outer face of the flange and toe of the tee formed at the net section.

For parallel flange channels, the elastic modulus about the minor (y-y) axis is given at the toe of the section, i.e.:

$y \quad = \quad B - c_y$

where:

B is the width of the section

c_y is the distance from the back of the web to the centroidal axis.

For angles, the elastic moduli about both axes are given at the toes of the section, i.e:

$y_x \quad = \quad A - c_x$

$y_y \quad = \quad B - c_y$

where:

A is the leg length perpendicular to x-x axis

B is the leg length perpendicular to y-y axis

c_x is the distance from the back of the angle to the centre of gravity, referred to as the x-x axis

c_y is the distance from the back of the angle to the centre of gravity, referred to as the y-y axis.

3.2.4 Buckling parameter (*u*) and torsional index (*x*)

The buckling parameter and torsional index used in buckling calculations are derived as follows:

(a) For bi-symmetric flanged sections and flanged sections symmetrical about the minor axis only:

$u \quad = \quad [(4\, S_x^{\,2}\, \gamma \,/\, (A^2\, h^2)]^{1/4}$

$x \quad = \quad 0.566\, h\, [A/J]^{1/2}$

(b) For flanged sections symmetric about the major axis only:

$u \quad = \quad [(I_y\, S_x^{\,2}\, \gamma \,/\, (A^2\, H)]^{1/4}$

$x \quad = \quad 1.132\, [(A\, H) \,/\, (I_y\, J)]^{1/2}$

where:

S_x is the plastic modulus about the major axis

$\gamma \quad = \quad [1 - I_y/I_x]$

I_x is the second moment of area about the major axis

I_y is the second moment of area about the minor axis

A is the cross-sectional area

h is the distance between shear centres of flanges (for T sections, h is the distance between the shear centre of the flange and the toe of the web)

H is the warping constant

J is the torsion constant.

3.2.5 Warping constant (*H*) and torsion constant (*J*)

(a) I and H sections

The warping constant and torsion constant for I and H sections are calculated using the formulae given in the SCI publication P057 *Design of members subject to combined bending and torsion*[12].

(b) Tee sections

For Tee sections cut from UB and UC sections, the warping constant (*H*) and torsion constant (*J*) have been derived as given below.

$$H = \frac{1}{144} T^3 B^3 + \frac{1}{36}\left(d - \frac{T}{2}\right)^3 t^3$$

$$J = \frac{1}{3} BT^3 + \frac{1}{3}(d - T)\, t^3 + \alpha_1 D_1^{\,4} - 0.21\, T^4 - 0.105\, t^4$$

where:

$$\alpha_1 = -0.042 + 0.2204 \frac{t}{T} + 0.1355 \frac{r}{T} - 0.0865 \frac{t\,r}{T^2} - 0.0725 \frac{t^2}{T^2}$$

$$D_1 = \frac{(T + r)^2 + (r + 0.25\, t)\, t}{2r + T}$$

Note: These formulae do not apply to tee sections cut from joists which have tapered flanges. For such sections, details are given in SCI Publication 057[12].

(c) Parallel flange channels

For parallel flange channels, the warping constant (*H*) and torsion constant (*J*) are calculated as follows:

$$H = \frac{h^2}{4}\left[I_y - A\left(c_y - \frac{t}{2}\right)^2 \left(\frac{h^2 A}{4 I_x} - 1\right)\right]$$

$$J = \frac{2}{3} BT^3 + \frac{1}{3}(D - 2T)\, t^3 + 2\alpha_3 D_3^{\,4} - 0.42\, T^4$$

where:

c_y = is the distance from the back of the web to the Centroidal axis

$$\alpha_3 = -0.0908 + 0.2621 \frac{t}{T} + 0.1231 \frac{r}{T} - 0.0752 \frac{t\,r}{T^2} - 0.0945 \left(\frac{t}{T}\right)^2$$

$$D_3 = 2\left[(3r + t + T) - \sqrt{2\,(2r + t)(2r + T)} \right]$$

Note: The formula for the torsion constant (*J*) is applicable to parallel flange channels only and does not apply to tapered flange channels.

(d) Angles

For angles, the torsion constant (*J*) is calculated as follows:

$$J = \frac{1}{3}bt^3 + \frac{1}{3}(d-t)t^3 + \alpha_3 D_3^{\,4} - 0.21\,t^4$$

where:

$$\alpha_3 = 0.0768 + 0.0479\frac{r}{t}$$

$$D_3 = 2\left[\left(3r+2t\right) - \sqrt{2\left(2r+t\right)^2}\;\right]$$

(e) ASB sections

For ASB (asymmetric beams) *Slimdek*® beam, the warping constant (H) and torsion constant (J) are as given in Corus brochure, *Structural sections* [11].

3.2.6 Plastic modulus (S)

The full plastic moduli about both principal axes are tabulated for all sections except angle sections. For angle sections, BS 5950-1: 2000 requires design using the elastic modulus.

The reduced plastic moduli under axial load are tabulated for both principal axes for all sections except asymmetric beams and angle sections. For angle sections, BS 5950-1: 2000 requires design using the elastic modulus.

When a section is loaded to full plasticity by a combination of bending and axial compression about the major axis, the plastic neutral axis shifts and may be located either in the web or the tension flange (or in the taper part of the flange for a joist) depending on the relative values of bending and axial compression. Formulae giving the reduced plastic modulus under combined loading have to be used, which use a parameter n as follows:

$$n = \frac{F}{A\,p_y}$$ (This is shown in the member capacity tables as F/P_z)

where:

F is the factored axial load

A is the cross-sectional area

p_y is the design strength of the steel.

For each section, there is a "change" value of n. Formulae for reduced plastic modulus and the "change" value are given below.

(a) Universal beams, universal columns and bearing piles

If the value of n calculated is less than the change value, the plastic neutral axis is in the web and the formula for lower values of "n" must be used. If n is greater than the change value, the plastic neutral axis lies in the tension flange and the formula for higher values of n must be used. The same principles apply when the sections are loaded axially and bent about the minor axis, lower and higher values of n indicating that the plastic neutral axis lies inside or outside the web respectively.

Major axis bending:

Reduced plastic modulus: Change value:

$S_{rx} = K_1 - K_2 n^2$ for $n < \dfrac{(D-2T)t}{A}$

$S_{rx} \quad = \quad K_3\,(1 - n)\,(K_4 + n) \qquad \text{for} \quad n \geq \dfrac{(D - 2T)\,t}{A}$

where:

$K_1 = S_x \qquad K_2 = \dfrac{A^2}{4t}$

$K_3 = \dfrac{A^2}{4B} \qquad K_4 = \dfrac{2DB}{A} - 1$

Minor axis bending:

Reduced plastic modulus: Change value:

$S_{ry} \quad = \quad K_1 - K_2\,n^2 \qquad \text{for} \quad n < \dfrac{t\,D}{A}$

$S_{ry} \quad = \quad K_3\,(1 - n)\,(K_4 + n) \qquad \text{for} \quad n \geq \dfrac{t\,D}{A}$

where:

$K_1 = S_y \qquad K_2 = \dfrac{A^2}{4D}$

$K_3 = \dfrac{A^2}{8T} \qquad K_4 = \dfrac{4BT}{A} - 1$

(b) Joists

Major axis bending:

If the value of n calculated is less than the lower change value (n_1), the plastic neutral axis is in the web and the formula for lower values of n must be used. If n is greater than the higher change value (n_2), the plastic neutral axis lies in the part of the tension flange that is not tapered and the formula for higher values of n must be used. If the value of n calculated lies between the lower change value (n_1) and the higher change value (n_2), the plastic neutral axis lies in the tapered part of the flange and then a linear interpolation between the two formulae is used to calculate the reduced plastic modulus.

Reduced plastic modulus Change value

$S_{rx} = S_{rx1} = K_1 - K_2\,n^2 \qquad \text{for} \;\; n \leq n_1 = \left\{\dfrac{D}{A} - \dfrac{2}{A}\left(T + \dfrac{B-t}{4}\tan(\theta)\right)\right\}t$

$S_{rx} = S_{rx2} = K_3\,(1 - n)\,(K_4 + n) \qquad \text{for} \;\; n \geq n_2 = 1 - \dfrac{2B}{A}\left(T - \dfrac{B-t}{4}\tan(\theta)\right)$

$S_{rx} = S_{rx1} + (S_{rx2} - S_{rx1})\left(\dfrac{n - n_1}{n_2 - n_1}\right) \qquad \text{for } n_1 < n < n_2$

where:

$K_1 = S_x \qquad K_2 = \dfrac{A^2}{4t}$

$$K_3 = \frac{A^2}{4B} \qquad K_4 = \frac{2DB}{A} - 1$$

$$\theta = 8^0 \text{ (flange taper)}$$

Minor axis bending:

The same principles apply when the sections are loaded axially and bent about the minor axis, lower and higher values of n indicating that the plastic neutral axis lies inside or outside the web respectively.

Reduced plastic modulus — Change value

$$S_{ry} = K_1 - K_2 n^2 \qquad \text{for } n < \frac{tD}{A}$$

$$S_{ry} = K_3 (1 - n)(K_4 + n) \qquad \text{for } n \geq \frac{tD}{A}$$

where:

$$K_1 = S_y \qquad K_2 = \frac{A^2}{4D}$$

$$K_3 = 0.87 \frac{A^2}{8T} \qquad K_4 = \frac{4BT}{A} - 1$$

(c) Parallel flange channels

Major axis bending:

If the value of n calculated is less than the change value, the plastic neutral axis is in the web and the formula for lower values of n must be used. If n is greater than the change value, the plastic neutral axis lies in the flange and the formula for higher values of n must be used.

Reduced plastic modulus — Change value

$$S_{rx} = K_1 - K_2 n^2 \qquad \text{for } n < \frac{(D - 2T)\,t}{A}$$

$$S_{rx} = K_3 (1 - n)(K_4 + n) \qquad \text{for } n \geq \frac{(D - 2T)\,t}{A}$$

where:

$$K_1 = S_x \qquad K_2 = \frac{A^2}{4t}$$

$$K_3 = \frac{A^2}{4B} \qquad K_4 = \frac{2DB}{A} - 1$$

Minor axis bending:

In calculating the reduced plastic modulus of a channel for axial force combined with bending about the minor axis, the axial force is considered as acting at the centroidal axis of the cross-section whereas it is considered to be resisted at the plastic neutral axis. The value of the reduced plastic modulus takes account of the resulting moment due to eccentricity relative to the net centroidal axis.

The reduced plastic modulus of a parallel flange channels bending about the minor axis depends on whether the stresses induced by the axial force and applied moment are the same or of opposite kind towards the back of the channel. Where the stresses are of the same kind, an initial increase in axial force may cause a small initial rise of the "reduced" plastic modulus, due to the eccentricity of the axial force

For each section there is again a change value of n. For minor axis bending the position of the plastic neutral axis when there is no axial load may be either in the web or the flanges. When the value of n is less than the change value, the formula for lower values of n must be used. If n is greater than the change value, the formula for higher values of n must be used.

The formulae concerned are complex and are therefore not quoted here.

3.2.7 Equivalent slenderness coefficient (ϕ_a) and monosymmetry index (ψ_a)

The equivalent slenderness coefficient (ϕ_a) is tabulated for both equal and unequal angles. Two values of the equivalent slenderness coefficient are given for each unequal angle. The larger value is based on the major axis elastic modulus (Z_u) to the toe of the short leg and the lower value is based on the major axis elastic modulus to the toe of the long leg.

The equivalent slenderness coefficient (ϕ_a) is calculated as follows:

$$\phi_a = \left[\frac{Z_u{}^2 \gamma_a}{AJ}\right]^{0.5}$$

Definitions of all the individual terms are given in BS 5950-1[1], Clause B.2.9.

The monosymmetry index (ψ_a) is only applicable for unequal angles and is calculated as follows:

$$\psi_a = \left[2v_0 - \frac{\int v_i\left(u_i{}^2 + v_i{}^2\right)dA}{I_u}\right]\frac{1}{t}$$

Definitions of all the individual terms are given in BS 5950-1[1], Clause B.2.9.

3.3 Hollow sections

Section properties are given for both hot-finished and cold-formed hollow sections. The ranges of hot-finished and cold-formed sections covered are different. The section ranges listed are in line with sections that are readily available from the major section manufacturers. For the same overall dimensions and wall thickness, the section properties for hot-finished and cold-formed sections are different because the corner radii are different.

3.3.1 Common properties

For comment on second moment of area, radius of gyration and elastic modulus, see Section 3.2.1, 3.2.2 and 3.2.3.

For hot-finished square and rectangular hollow sections, the sectional properties have been calculated, using corner radii of 1.5t externally and 1.0t internally, as specified by BS EN 10210-2[9].

For cold-formed square and rectangular hollow sections, the sectional properties have been calculated, using the external corner radii of $2t$ if $t \le 6$ mm, 2.5t if 6 mm $<$ t $\le$ 10 mm and $3t$ if $t > 10$ mm as specified by BS EN 10219-2[10]. The internal corner radii used is $1.0t$ if $t \le 6$ mm, $1.5t$ if 6 mm $<$ t $\le$ 10 mm and $2t$ if $t > 10$ mm, as specified by BS EN 10219-2[10].

3.3.2 Torsion constant (*J*)

For circular hollow sections:

$$J = 2I$$

For square and rectangular hollow sections:

$$J = \frac{4A_h{}^2 t}{h} + \frac{t^3 h}{3}$$

For elliptical hollow sections:

$$J = \frac{4A_m{}^2 t}{U} + \frac{Ut^3}{3}$$

where:

I is the second moment of area

t is the thickness of section

h is the mean perimeter $= 2\,[(B - t) + (D - t)] - 2\,R_c\,(4 - \pi)$

A_h is the area enclosed by mean perimeter of a square or rectangular hollow section and $= (B - t)\,(D - t) - R_c^2\,(4 - \pi)$

B is the breadth of section

D is the depth of section

R_c is the average of internal and external corner radii.

A_m is the area enclosed by the mean perimeter of an elliptical hollow sections and

$$= \frac{\pi(H-t)(B-t)}{4}$$

$$U = \frac{\pi}{2}(H + B - 2t)\left(1 + 0.25\left(\frac{H-B}{H+B-2t}\right)^2\right)$$

3.3.3 Torsion modulus constant (*C*)

For circular hollow sections

$$C = 2Z$$

For square and rectangular hollow sections

$$C = J \Big/ \left(t + \frac{2A_h}{h}\right)$$

For elliptical hollow sections

$$C = \frac{J}{t + \left(\frac{2A_m}{U}\right)}$$

where:

Z is the elastic modulus and J, t, A_h, h, A_m and U are as defined in Section 3.3.2.

3.3.4 Plastic modulus of hollow sections (S)

The full plastic modulus (S) is given in the tables. When a member is subject to a combination of bending and axial load the plastic neutral axis shifts. Formulae giving the reduced plastic modulus under combined loading have to be used, which use the parameter n as defined below.

$$n = \frac{F}{A\ p_y} \qquad \text{(This is shown in the member capacity tables as } F/P_z\text{)}$$

where:

F is the factored axial load

A is the cross-sectional area

p_y is the design strength of the steel.

For square and rectangular hollow sections there is a "change" value of n. Formulae for reduced plastic modulus and "change" value are given below.

(a) Circular hollow sections

$$S_r = S \cos\left(\frac{n\pi}{2}\right)$$

(b) Square and rectangular hollow sections

If the value of n calculated is less than the change value, the plastic neutral axis is in the webs and the formula for lower values of n must be used. If n is greater than the change value, the plastic neutral axis lies in the flange and the formula for higher values of n must be used.

Major axis bending:

Reduced plastic modulus — Change value

$$S_{rx} = S_x - \frac{A^2 n^2}{8t} \qquad \text{for} \quad n \le \frac{2t\,(D-2t)}{A}$$

$$S_{rx} = \frac{A^2}{4(B-t)}(1-n)\left[\frac{2D(B-t)}{A} + n - 1\right] \qquad \text{for} \quad n > \frac{2t\,(D-2t)}{A}$$

Minor axis bending:

Reduced plastic modulus — Change value

$$S_{ry} = S_y - \frac{A^2 n^2}{8t} \qquad \text{for} \quad n \le \frac{2t\,(B-2t)}{A}$$

$$S_{ry} = \frac{A^2}{4(D-t)}(1-n)\left[\frac{2B(D-t)}{A} + n - 1\right] \qquad \text{for} \quad n > \frac{2t\,(B-2t)}{A}$$

where:

S, S_x, S_y	are the full plastic moduli about the relevant axes
A	is the gross cross-sectional area
D, B and t	are as defined in Section 3.3.2.

4 EFFECTIVE SECTION PROPERTIES

4.1 General

In BS 5950-1: 2000 effective section properties are required for design of members with class 3 semi-compact and class 4 slender cross-sections. Effective section properties are given for sections subject to compression, bending and also combined axial compression and bending. Effective section properties depend on the grade of steel used; effective properties for grades S275 and S355 are given next to each other or on facing pages.

4.2 Effective section properties of members subject to compression (except angles)

The compression resistance of class 4 slender sections is calculated using the effective cross-sectional area A_{eff} instead of the gross area A, according to BS 5950-1[1], Clause 4.7.4(b). The section classification depends on the width/thickness ratios of the webs and the flanges. The tables list sections that can be class 4 slender and the character W or F indicates whether the section is slender due to the web or the flange. The effective area of the section is given for all slender sections and is calculated as follows:

For UB, UC and Joists with the web controlling the classification

$$A_{eff} = A - t(d - 40t\varepsilon)$$

For circular hollow sections

$$A_{eff} = A\,[\,(80/(D/t))\,(275/p_y)\,]^{0.5}$$

For hot-finished square and rectangular hollow sections with web controlling

$$A_{eff} = A - 2t(\,(D\text{-}3t) - 40t\varepsilon)$$

For hot-finished square and rectangular hollow sections with web and flange controlling

$$A_{eff} = A - 2t(\,(D\text{-}3t) + (B\text{-}3t) - 80t\varepsilon)$$

For cold-formed square and rectangular hollow sections with web controlling

$$A_{eff} = A - 2t(\,(D\text{-}5t) - 35t\varepsilon)$$

For cold-formed square and rectangular hollow sections with web and flange controlling

$$A_{eff} = A - 2t(\,(D\text{-}5t) + (B\text{-}5t) - 70t\varepsilon)$$

where:

ε	$= (275/p_y)^{0.5}$
A	is the gross cross-sectional area
d	is the depth between fillets

p_y is the design strength

D, B and t are as defined in Section 3.3.2.

The ratio of effective area to gross area (A_{ef}/A) is also given in the table to provide a guide as to how much of the section is effective.

4.3 Effective section properties of members subject to compression (angles)

The cross-section classification is given along with the design strength and the reduced design strength. The reduced design strength is used for angles instead of the effective area because otherwise the effect of the additional moments induced by the shift of the centroid of the effective section would need to be considered (BS 5950-1, Clause 3.6.3).

Only members which are class 4 slender in S275 or S355 are listed. The reduced design strength for class 4 slender sections is calculated as follows:

$$p_{yr} = \text{Minimum}\ \{\ p_y\,(15\varepsilon\,/(d/t))^2\ ,\quad p_y\,(15\varepsilon\,/(b/t))^2\ ,\quad p_y(24\varepsilon\,/((b+d)/t))^2\ \}$$

where:

$\varepsilon = (275/p_y)^{0.5}$

d is the length of the long leg

b is the length of the short leg

p_y is the design strength

t is the leg thickness.

4.4 Effective section properties of members subject to pure bending

The section classification depends on the width/thickness ratios of the webs and the flanges. The tables give sections that are class 3 semi-compact or class 4 slender in S275 or S355 and the character W or F indicates whether the web or the flange controls the section classification.

For class 3 semi-compact sections the effective plastic modulus S_{eff} is given, which is calculated as in BS 5950-1, Clauses 3.5.6.2, 3.5.6.3 and 3.5.6.4.

For class 4 slender sections the effective elastic modulus Z_{eff} and the effective second moment of area I_{eff} are given and are calculated as follows:

For hot-finished square and rectangular hollow sections with flange controlling

$$I_{xeff} = I_x - \frac{(B-40t\varepsilon-3t)t^3}{12} - (B-40t\varepsilon-3t)t\left(\frac{D}{2}-\frac{t}{2}\right)^2 - A_{eff}\left(y_{xeff}-\frac{D}{2}\right)^2$$

$$Z_{xeff} = I_{xeff}\ /\ y_{xeff}$$

where:

$$y_{xeff} = \frac{AD-(B-40t\varepsilon-3t)t^2}{2(A-(B-40t\varepsilon-3t)t)}$$

$A_{eff} = A - (B - 40t\varepsilon - 3t)t$

I_x is the second moment of area bout the x axis

I_y is the second moment of area bout the y axis

A is the gross cross-sectional area and D, B, t, ε are as defined in Section 4.3.

For cold-formed square and rectangular hollow sections with flange controlling

$$I_{xeff} = I_x - \frac{(B - 35t\varepsilon - 5t)t^3}{12} - (B - 35t\varepsilon - 5t)\,t\left(\frac{D}{2} - \frac{t}{2}\right)^2 - A_{eff}\left(y_{xeff} - \frac{D}{2}\right)^2$$

$$Z_{xeff} = I_{xeff} / y_{xeff}$$

where:

$$y_{xeff} = \frac{AD - (B - 35t\varepsilon - 5t)t^2}{2(A - (B - 35t\varepsilon - 5t)t)}$$

$$A_{eff} = A - (B - 35t\varepsilon - 5t)t$$

I_x is the second moment of area bout the x axis

I_y is the second moment of area bout the y axis

A is the gross cross-sectional area and D, B, t, ε are as defined in Section 4.3.

Other section types do not become class 4 slender when subject to bending only, for the range of sections covered by this publication.

Also given in the tables is β_w, which is defined as:

S_x/S_x	For class 1 plastic and class 2 compact sections
S_{xeff}/S_x	For class 3 semi-compact sections
Z_{xeff}/S_x	For class 4 slender sections

4.5 Effective section properties of members subject to axial compression and bending

The cross-section classification is dependent on the level of axial load. Therefore, the classification and effective plastic modulus is given for a range of axial loads, expressed as a proportion of the squash load, P_z. The tables give sections which can be class 3 semi-compact or class 4 slender in S275 or S355 and the character W or F indicates whether the section classification is controlled by the web or the flange.

The effective plastic modulus is calculated in accordance with BS 5950-1, Clauses 3.5.6.2, 3.5.6.3 and 3.5.6.4. It should be noted that the limits used from Tables 11 and 12 are the "Web – Generally" limits.

5 CAPACITY AND RESISTANCE TABLES

Code Ref.

5.1 General

The values displayed in the member capacity and resistance tables have been rounded to three significant figures.

Capacity and resistance tables are given for strength grades S275 and S355, except for cold-formed hollow sections, where tables are given for grade S355 only.

5.2 Design strength

The member capacity and resistance tables have been based on the following values of design strength p_y. 3.1.1

Table 9

Steel Grade	Flange Thickness less than or equal to (mm)	Design strength p_y (N/mm^2)
S275	16	275
	40	265
	63	255
	80	245
	100	235
	150	225
S355	16	355
	40	345
	63	335
	80	325
	100	315
	150	295

6 COMPRESSION TABLES

6.1 Compression members: UB sections, UC sections, joists, and hollow sections

(a) Compression Resistance, P_c 4.7.4 (a)

(i) For non-slender (Class 1, 2 or 3) cross-sections:

$$P_c = A_g\, p_c$$

where:

A_g is the gross cross-sectional area

p_c is the compressive strength.

The compressive strength p_c is obtained using the following values of Robertson constant, a. 4.7.5 Annex. C.2

Code Ref.

Table 23

Type of Section	Robertson constant *a*	
	Axis of buckling	
	x-x	y-y
Hot-finished Structural Hollow sections	2.0	2.0
Cold-formed Structural Hollow sections	5.5	5.5
UB sections (flange thickness up to 40 mm)	2.0	3.5
UB sections (flange thickness over 40 mm)	3.5	5.5
UC sections (flange thickness up to 40 mm)	3.5	5.5
UC sections (flange thickness over 40 mm)	5.5	8.0
Joists (when D/B > 1.2)	2.0	3.5
Joists (when D/B ≤ 1.2)	3.5	5.5

For I and H sections with a flange thickness between 40 mm and 50 mm, the value of p_c is taken as the average of the values obtained for thickness up to 40 mm and over 40 mm, as noted in Table 23. Table 23 Note 1

(ii) For class 4 slender cross-sections:

$P_c = A_{eff}\, p_{cs}$ for class 4 slender cross-sections 4.7.4 (b)

where:

A_{eff} is the effective cross-sectional area

p_{cs} is the value of p_c for a reduced slenderness $= \lambda(A_{eff} / A_g)^{0.5}$

The section classification of a section is partly dependent on the level of axial load applied. None of the universal columns, joists or parallel flange channels can be slender under axial compression only, but some universal beams and hollow sections can be slender. Sections that can be slender under axial compression are marked thus *.

The sections concerned are:

UB, when $d/t > 40\varepsilon$ Table 11 and Table 12

Hot-finished SHS, RHS, when $d/t > 40\varepsilon$

Cold-formed SHS, RHS, when $d/t > 35\varepsilon$

All CHS, when $D/t > 80\varepsilon^2$

where:

d is the depth of the web

t is the thickness of the web or wall

D outside diameter

$\varepsilon = (275/p_y)^{0.5}$

p_y is the design strength.

Code Ref.

If a cross-section can be slender under axial load, the tabulated compression resistance is only based on the slender cross-section equation (given above) if the value from this equation is greater than the axial load required to make the cross-section slender. Otherwise, the compression resistance of a potentially slender section is given as the smaller of the non-slender compression resistance and the axial load required to make the section slender. Tabulated values based on the equation for slender cross-sections are printed in italic type.

An example is given below:

686 × 254 × 170 UB S275

For this section, $d/t = 42.4 > 40\varepsilon$

Hence, the section can become slender if axial load is sufficiently high.

The axial load at which the section becomes slender is 5410 kN.

This value is calculated by setting d/t for the section equal to the class 3 limit from BS 5950-1[1], Table 11 and then solving for the value of axial load (F_c).

For $L_E = 4$ m, $P_{cx} = p_{cs} A_{eff} = 5660$ kN (slender cross-section)

Hence table shows *5660* kN in italic type because, $p_{cs} A_{eff} >$ value at which cross-section becomes slender, 5410 kN.

For $L_E = 12$ m, $P_{cx} = p_c A = 5400$ kN (non-slender cross-section)

Hence the table shows 5400 kN in normal type because, $p_c A <$ value at which cross-section becomes slender, 5410 kN.

(b) Compression resistances P_{cx} and P_{cy}

The values of compression resistance P_{cx} and P_{cy} for buckling about the two principal axes are based on:

- The effective lengths (L_E) given at the head of the table. 4.7.3
- The slenderness (λ), calculated as follows:

  For UB, UC, joist and hollow sections,

  $\lambda = L_E/r_x$ for x axis buckling 4.7.2

  $\lambda = L_E/r_y$ for y axis buckling

Code Ref.

6.2 Compression members: Single parallel flange channels

(a) Compression Resistance, $P_c = A_g\, p_c$

where:

A_g is the gross cross-section area

p_c is the compressive strength.

Compressive strength P_c has been obtained using the Robertson constant, *a,* of 5.5. Table 23

Note: None of the parallel flange channels are slender under axial compression Annex C.2

(b) The values of compression resistance P_{cx} and P_{cy} for buckling about the two principle axes have been based on:

- The effective lengths (L_E) given at the head of the table.
- The slenderness (λ), calculated as follows:

(i) For a single channel subject to concentric axial load,

$\lambda = L_E/r_x$ for x axis buckling 4.7.2

$\lambda = L_E/r_y$ for y axis buckling

(ii) For a single channel connected only through its web,

- by two or more rows of bolts arranged symmetrically across the web or by equivalent welded connection, the slenderness, λ should be taken as the greater of:

$\lambda_x = 0.85\, L_x/r_x$ and 4.7.10.4 (a) Table 25

$\lambda_y = 1.0\, L_y/r_y$ but $\geq 0.7\, L_y/r_y + 30$

- by two or more bolts arranged symmetrically in a single row across the web or by equivalent welded connection, the slenderness, λ should be taken as the greater of:

$\lambda_x = 1.0\, L_x/r_x$ and 4.7.10.4 (b) Table 25

$\lambda_y = 1.0\, L_y/r_y$ but $\geq 0.7\, L_y/r_y + 30$

where:

L_x and L_y are the lengths between intersections

r_x and r_y are the radii of gyration of the single channel about the x and y axes.

Code Ref.

6.3 Compound compression members: two parallel flange channels

6.3.1 Two parallel flange channels laced

(a) Compression resistance, $P_c = A_g\, p_c$ 4.7.4

where:

A_g is the gross cross-section area of the two channels

p_c is the compressive strength and has been obtained using a Robertson constant of 5.5. Table 23 Annex C.2

Note: None of the parallel flange channels listed are slender under axial compression.

(b) The values of compression resistance, P_{cx} and P_{cy}, for buckling about the two principal axes, are based on:

- The effective lengths (L_E) given at the head of the tables. 4.7.3
- Slenderness (λ) calculated as follows: 4.7.2

$\lambda_x = L_E/r_{xm}$ and $\lambda_y = L_E/r_{ym}$

where:

r_{xm}, r_{ym}, are the radii of gyration of the compound member about the x and y axes.

6.3.2 Two parallel flange channels back to back

(a) Compression resistance, $P_c = A_g\, p_c$ 4.7.4

where:

A_g is the gross cross-section area of the two channels

p_c is the compressive strength and has been obtained using a Robertson constant of 5.5. Table 23 Annex C.2

Note: None of the parallel flange channels listed are slender under axial compression.

(b) The values of compression resistance P_{cx} and P_{cy} for buckling about the two principal axes, are based on:

- The length (L) given at the head of the tables.
- An effective length (L_E) about either axes taken as 1.0 L. If L_E is less than 1.0 L, the values are conservative.
- Slenderness (λ) calculated as follows:

(i) For buckling about the x axis, $\lambda_x = L_E/r_{xm}$ 4.7.13.2

(ii) For buckling about the y axis, $\lambda_b = (\lambda_m^2 + \lambda_c^2)^{0.5}$ but $\geq 1.4\,\lambda_c$ 4.7.9 c)

Code Ref.

where:

λ_m = L_E/r_{ym} for the compound member

λ_c = $L_c/r_y \leq 50$ for a single channel

r_{xm}, r_{ym} are the radii of gyration of the compound member about the x and y axes

r_y is the minimum radius of gyration of a single channel.

L_c is the length L divided by the number of bays. There are a sufficient number of bays so that $\lambda_c \leq 50$. The number of bays is at least three and if there are more than three, the compression resistance is printed in bold type. 4.7.9 (c)

6.4 Compression members: single angles

(a) Compression resistance, $P_c = A_g\, p_c$ 4.7.4

where:

A_g is the gross cross-section area

p_c is the compressive strength and has been obtained using a Robertson constant, a, of 5.5 Table 23 Annex C.2

In the case of a single bolt at each end, the compression resistance should be taken as 80% of that for an axially loaded member with the same slenderness. (Note: no values are given for this case).

Sections which are slender are marked * and their resistances have been calculated using a reduced design strength. 3.6.5

An angle cross-sections is slender if (using code notation):

d/t or $b/t > 15\varepsilon$ or $(d + b) / t > 24\varepsilon$ Table 11

Or using the notation in these tables, the requirements become:

A/t or $B/t > 15\varepsilon$ or $(A + B) / t > 24\varepsilon$

In these circumstances, the design strength is reduced by the least of these factors:

$$\left(\frac{15\varepsilon}{A/t}\right)^2 : \left(\frac{15\varepsilon}{B/t}\right)^2 : \left(\frac{24\varepsilon}{(A+B)/t}\right)^2$$ 3.6.5

where:

$\varepsilon = (275/p_y)^{0.5}$

Code Ref.

(b) The values of compression resistance are based on:

- The length (L) between intersections of centroidal axes or setting out line of the bolts given at the head of the tables.
- The slenderness (λ), calculated as follows:

(i) For two or more bolts in standard clearance holes in line along the angle at each end or an equivalent welded connection, the slenderness should be taken as the greater of: 4.7.10.2 (a) Table 25

$0.85\ L_v/r_v$ but $\geq 0.7\ L_v/r_v + 15$; and

$1.0\ L_a/r_a$ but $\geq 0.7\ L_a/r_a + 30$; and

$0.85\ L_b/r_b$ but $\geq 0.7\ L_b/r_b + 30$.

(iii) For a single bolt at each end, the λ should be taken as the greater of: 4.7.10.2 (c)

$1.0\ L_v/r_v$ but $\geq 0.7\ L_v/r_v + 15$; and Table 25

$1.0\ L_a/r_a$ but $\geq 0.7\ L_a/r_a + 30$; and

$1.0\ L_b/r_b$ but $\geq 0.7\ L_b/r_b + 30$.

where:

r_v is the minimum radius of gyration

r_a is the radius of gyration about the axis parallel to the connected leg

r_b is the radius of gyration about the axis perpendicular to the connected leg.

6.5 Compound compression members: two angles

The tables assume that the angles are interconnected back to back, as recommended in Clause 4.7.13 of the code. 4.7.13

(a) Compression resistance, $P_c = A_g\ p_c$

where:

A_g is the gross cross-section area of the two angles

p_c is the compressive strength and has been obtained using a Robertson constant of 5.5. Table 23 Annex C.2

Sections which are slender are marked * and their resistances have been calculated using a reduced design strength. 3.6.5

An angle cross-sections is slender if (using code notation):

d/t or $b/t > 15\varepsilon$ or $(d + b)/t > 24\varepsilon$ Table 11

Code Ref.

Or using the notation in these tables, the requirements become:

A/t or $B/t > 15\varepsilon$ or $(A + B)/t > 24\varepsilon$

In these circumstances, the design strength is reduced by the least of these factors: 3.6.5

$$\left(\frac{15\varepsilon}{A/t}\right)^2 \quad : \quad \left(\frac{15\varepsilon}{B/t}\right)^2 \quad : \quad \left(\frac{24\varepsilon}{(A+B)/t}\right)^2$$

where:

$\varepsilon = (275/p_y)^{0.5}$

(b) The values of compression resistance are based on:

- The length (L) between intersections of centroidal axes or setting out line of the bolts given at the head of the tables.
- The slenderness (λ), calculated as follows:

(i) For double angles connected to one side of a gusset or member by two or more bolts in line along each angle, or by an equivalent weld at each end, the slenderness λ should be taken as the greater of: 4.7.10.3 (a) Table 25

$1.0\ L_x/r_{xm}$ but $\geq 0.7\ L_x/r_{xm} + 30$; and

$[(0.85\ L_y/r_{ym})^2 + \lambda_c^2]^{0.5}$ but $\geq 1.4\ \lambda_c$

(ii) For double angles connected to one side of a gusset or member by one bolt in each angle, the slenderness λ should be taken as the greater of: 4.7.10.3 (b) Table 25

$1.0\ L_x/r_{xm}$ but $\geq 0.7\ L_x/r_{xm} + 30$; and

$[(1.0\ L_y/r_{ym})^2 + \lambda_c^2]^{0.5}$ but $\geq 1.4\ \lambda_c$

(iii) For double angles connected to both sides of a gusset or member by two or more bolts in line along each angle, the slenderness λ should be taken as the greater of: 4.7.10.3 (c) Table 25

$0.85\ L_x/r_{xm}$ but $\geq 0.7\ L_x/r_x + 30$; and

$[(1.0\ L_y/r_{ym})^2 + \lambda_c^2]^{0.5}$ but $\geq 1.4\ \lambda_c$

(iv) For double angles connected to both sides of a gusset or member by a single bolt in each angle, the slenderness λ should be taken as the greater of: 4.7.10.3 (e) Table 25

$1.0\ L_x/r_{xm}$ but $\geq 0.7\ L_x/r_{xm} + 30$; and

$[(1.0\ L_y/r_{ym})^2 + \lambda_c^2]^{0.5}$ but $\geq 1.4\ \lambda_c$

For double angles connected to both sides of a gusset or member by a single bolt in each angle, the compression resistance should be taken as 80% of that for an axially loaded member with the same slenderness. (Note: no values are given for this case).

Code Ref.

Where:

λ_c = L_c/r_v but not greater than 50

r_v is the minimum radius of gyration of a single angle

r_{xm} and r_{ym} are the radii of gyration of the double angles about the x and y axes

L_c is the length L divided by the number of bays. There are a sufficient number of bays so that $\lambda_c \leq 50$. The number of bays is at least three and if there are more than three, the compression resistance is printed in bold type. 4.7.9 (c)

7 TENSION TABLES

7.1 Tension members: Single angles

The value of tension capacity P_t is given generally by equivalent tension area x p_y :

(i) For bolted sections, $P_t = (A_e - 0.5a_2)p_y$ 4.6.3.1

(ii) For welded sections, $P_t = (A_g - 0.3a_2)p_y$

where:

A_e is the effective net area of the angle 4.6.1/3.4.3

A_g is the gross area of the angle

p_y is the design strength of the angle

a_2 is defined below.

Note: A block shear check (BS 5959-1: 2000, clause 6.2.4 and Figure 22) is also required for tension members. However, 'block shear' capacities have not been tabulated, as there are too many variables in the possible bolt arrangements.

The effective net area of the section A_e is given by:

For bolted sections, $A_e = a_{e1} + a_{e2}$ but $\leq 1.2(a_{n1} + a_{n2})$ 4.6.1/3.4.3

where:

$a_{e1} = K_e\, a_{n1}$ but $\leq a_1$ 3.4.3

$a_{e2} = K_e\, a_{n2}$ but $\leq a_2$

$a_{n1} = a_1$ – area of bolt holes in connected leg

$a_{n2} = a_2$

A_g = Gross area of single angle

K_e = 1.2 for grade S275 3.4.3

= 1.1 for grade S355

Code Ref.

a_1 = Gross area of connected leg

= A x t if long leg connected

= B x t if short leg connected

a_2 = $A_g - a_1$. 4.6.3.1

7.2 Compound tension members: Two angles

The values of tension capacity are based on the effective net area of the section, calculated as for single angles in Section 7.1.

The value of tension capacity P_t is given generally by equivalent tension area x p_y :

(i) For bolted sections,

$P_t = 2(A_e - 0.25a_2)p_y$ For a gusset between the angles 4.6.3.2 (a)

$P_t = 2(A_e - 0.5a_2)p_y$ For a gusset on the back of the angles 4.6.3.2 (b)

(ii) For welded sections,

$P_t = 2(A_g - 0.15a_2)p_y$ For a gusset between the angles 4.6.3.2 (a)

$P_t = 2(A_g - 0.3a_2)p_y$ For a gusset on the back of the angles 4.6.3.2 (b)

Symbols as defined in Section 7.1.

Note: A block shear check (BS 5950-1: 2000, Clause 6.2.4 and Figure 22) is also required for tension members. However, 'block shear' capacities have not been tabulated as there are too many variables in the possible bolt arrangements.

8 BENDING TABLES

8.1 Bending: UB sections, UC sections, joists and parallel flange channels

(a) Moment capacity, assuming shear load is low (<60% of the shear capacity):

$M_{cx} = p_y S_x$ but $\leq 1.2\, p_y Z_x$ for class 1 (plastic) and class 2 (compact sections) 4.2.5.2

$M_{cx} = p_y S_{xeff}$ but $\leq 1.2\, p_y Z_x$ for class 3 (semi-compact sections)

If the moment capacity is governed by 1.2 $p_y Z_x$ the values in the tables have been printed in italic type because higher values may be used in some circumstances. This limit is only appropriate for simply supported beams and cantilevers. For other cases, a general limit of 1.5 $p_y Z_x$ should be applied; a full explanation is given in AD195 [13]. 4.2.5.1

(b) Where the shear load is high (>60% of the shear capacity), the values should be checked and reduced, if necessary.

Code Ref.

(c) The moment capacity M_{cx} has been based on the section classification given but it should be noted that this classification applies to members subject to bending about the x-x axis only

Note: None of the universal beams, universal columns, joists or parallel flange channels in grade S275 or S355 are slender under bending only.

(d) Buckling resistance moment, M_b

$M_b = p_b\, S_x \le M_{cx}$ for class 1 (plastic) and class 2 (compact sections) 4.3.6.4 (c)

$M_b = p_b\, S_{xeff} \le M_{cx}$ for class 3 (semi-compact sections)

p_b has been obtained for particular values of $\lambda_{LT} = u\, v\, \lambda\, \beta_w^{0.5}$ 4.3.6.7

where:

$\beta_w = 1.0$ for class 1 (plastic) and class 2 (compact sections) 4.3.6.9

$= S_{xeff} / S_x$ for class 3 (semi-compact sections)

λ is the slenderness $= L_E/r_y$ 4.3.6.7

u is a Buckling parameter (as defined in the code) 4.3.6.8

v is a Slenderness factor (as defined in the code) 4.3.6.7

(e) The buckling resistance moments, M_b, have been based on: Table 19

- The effective length, L_E, given at the head of the tables.
- Section classification as above.

8.2 Bending: Hollow sections

8.2.1 Circular and square hollow sections

(a) Moment capacity assuming shear load is low (< 60% of shear capacity):

$M_c = p_y\, S$ but $\le 1.2\, p_y Z$ for class 1 (plastic) and class 2 (compact sections) 4.2.5.2

$M_c = p_y\, S_{eff}$ but $\le 1.2\, P_y Z$ for class 3 (semi-compact sections)

$M_c = p_y\, Z_{eff}$ for class 4 (slender sections)

If the moment capacity is governed by 1.2 p_y Z the values in the tables have been printed in italic type because higher values may be used in some circumstances. This limit is only appropriate for simply supported beams and cantilevers. For other cases, a general limit of 1.5 p_y Z should be applied, a full explanation is given in AD195 [13]. 4.2.5.1

Code Ref.

(b) Where the shear load is high (>60% of the shear capacity), the values for M_c should be checked and reduced, if necessary. 4.2.5.3

(c) The section classification is given in the tables but it should be noted that this classification applies to members subject to bending only. For square hollow sections all section classifications are possible under pure bending. None of the circular hollow sections are slender under pure bending.

(d) The second moment of area (I) is repeated in the tables as it is required for deflection checks.

(e) Shear capacity $P_v = 0.36\, p_y A$ for circular hollow sections 4.2.3

$P_v = 0.6\, p_y (D/2D) A$ for square hollow sections

8.2.2 Rectangular hollow sections

(a) Moment capacity

The values of M_{cx} and M_{cy} have been calculated as for circular and square hollow sections (Section 8.2.1).

(b) The section classification given in the tables applies to members subject to bending only about the appropriate axes. Sections can be slender for pure bending about the x-x or y-y axis in both S275 and S355 because the d/t ratio of the flange is greater than 40ε for hot-finished sections and 35ε for cold-formed sections. It should be noted that a section may be slender when bending about the y-y axis and not slender when bending about the x-x axis. Table 11

(c) The limiting length, L_c, is the length above which lateral torsional buckling should be checked. 4.3.6.1 Table 15

(d) The second moment of area (I) is repeated in the tables as it is required for deflection checks.

9 WEB BEARING AND BUCKLING TABLES

9.1 UB sections, UC sections and joists: bearing, buckling and shear capacities for unstiffened webs

Values have been calculated as follows:

(a) Bearing

The bearing capacity P_w, of the unstiffened web is given by:

$P_w = (b_1 + n\,k)\, t\, p_{yw}$ (BS 5950-1 notation) 4.5.2.1

$= b_1 C2 + C1$ (Capacity table notation)

Code Ref.

where:

b_1 is the stiff bearing length

n = 5, for continuous over bearing 4.5.2.1

= $2 + 0.6\, b_e / k$, for end bearing

= 2, for end bearing (b_e taken as zero in the tables)

b_e is the distance to the nearer end of the member from the end of the stiff bearing 4.5.2.1

k = ($T + r$) for rolled sections

t is the thickness of the web

T is the thickness of the flange

r is the root radius

p_{yw} is the design strength of the web.

(i) Bearing factor $C1$ is due to the beam alone

Generally, $C1 = n\, k\, t\, p_{yw}$.

$\therefore \quad C1 = 5\,(T + r)\, t\, p_{yw}$ for continuous over bearing

$\therefore \quad C1 = 2\,(T + r)\, t\, p_{yw}$ for end bearing.

(ii) Bearing factor $C2$ is equal to $t\, p_{yw}$ and must be multiplied by b_1 to give the stiff bearing contribution.

(b) Buckling

Generally the buckling resistance, P_x, of the unstiffened web is given by:

4.5.3.1

$$P_x = K \frac{25\,\varepsilon\, t}{\sqrt{(b_1 + n\,k)d}} P_w$$

Combining with (a), this can be re-written as:

$$P_x = K \left[\left(\frac{25\sqrt{275}\,t}{\sqrt{d/t}} \right)^2 P_w \right]^{0.5}$$

(BS 5950-1 notation)

$= K\,(C4\, P_w)^{0.5}$ (Capacity table notation) 4.5.3.1

if $a_e \geq 0.7d$ then $K = 1$

if $a_e < 0.7d$ then $K = \dfrac{a_e}{1.4d} + 0.5$

Code Ref.

where:

d is the depth between fillets

t is the web thickness

P_w is the web bearing capacity from (a) above.

a_e is the distance from the centre of the load or reaction to the nearer end of the member.

(i) Buckling factor $C4$ is the same for end bearing and continuous over bearing,

$$C4 = \left(\frac{25\sqrt{275}\ t}{\sqrt{d/t}} \right)^2$$

(c) Shear

The shear capacity of the section is given by:

$$P_v = 0.6\ p_y\ t\ D$$ 4.2.3

where:

D = Total depth of section.

Note: Since none of the rolled sections have d/t > 70ε, there is no need to check for shear buckling of the web.

9.2 Parallel flange channels: bearing, buckling and shear capacities for unstiffened webs

The nominal cross-section dimensions of parallel flange channels give a square heel, but the tolerances include a small heel radius (see Corus brochure *Structural sections*[11]), based on BS EN 10279:2000[7]. The heel radius can be between zero and 0.3 times the flange thickness.

In most cases the loads and reactions are applied directly to the web (by angle cleats, end plates or fin plates), so this is usually of no significance. However, if a force from a load or reaction is applied to the channel through the flange, the presence of a heel radius may produce eccentricity of this force relative to the web, so reduction factors, K_b for bearing and K_w for buckling to allow for this have been included in the tables.

The use of these reduction factors will provide acceptable design, but if necessary the eccentricity can be eliminated. A simple method is to use a continuously welded flange plate local to the stiff bearing, extending at least to the back of the channel. Depending on details, other methods may be appropriate.

Code Ref.

Values have been calculated as follows:

(a) Bearing capacity, P_w

$$P_w = K_b\,(b_1 + nk)\,t\,p_{yw} \qquad \text{(BS 5950-1 notation)}$$

$$= K_b\,(b_1\,C2 + C1) \qquad \text{(Capacity table notation)}$$

Where

K_b is a reduction factor to allow for heel radius.

All other notation as in Section 9.1.

(b) Buckling resistance, P_x

$$P_x = K_w\ K\left[\left(\frac{25\sqrt{275\,t}}{\sqrt{d/t}}\right)^2 P_w\right]^{0.5} \qquad \text{(BS 5950-1 notation)}$$

$$= K_w\ K\,(C4\ P_W)^{0.5} \qquad \text{(Capacity table notation)}$$

where

K_w is a reduction factor to allow for heel radius.

All other notation as in Section 9.1.

(c) Shear capacity 4.2.3

$P_v = 0.6\,p_y\,t\,D$

9.3 Square and rectangular hollow sections: bearing, buckling and shear capacities for unstiffened webs

Values have been calculated as follows:

(a) Bearing

The bearing capacity, P_W, of the unstiffened web is given by:

$$P_w = (b_1 + nk)2t\,p_{yw} \qquad \text{(BS 5950-1 notation)}$$ 4.5.2.1

$$= b_1C2 + C1 \qquad \text{(Capacity table notation)}$$

where:

- b_1 is the stiff bearing length (see figures below)
- n = 5, for continuous over bearing
  = 2, for end bearing
- k = t for hollow sections
- t is the thickness of the web
- p_{yw} is the design strength of the web.

Code Ref.

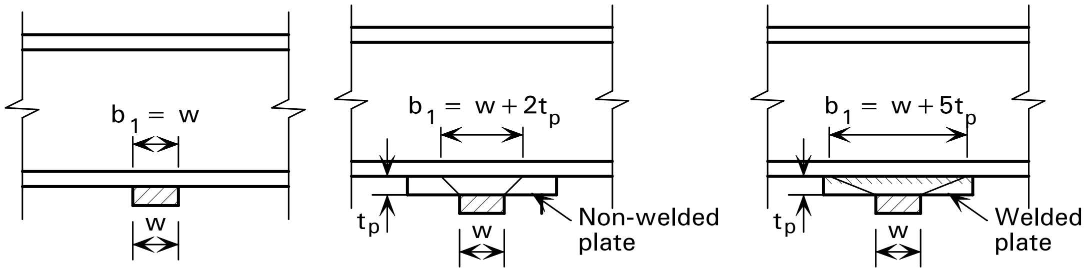


Figure illustrating examples of stiff bearing length, b_1

(i) Bearing factor $C1$ is due to the beam alone, for both webs.

Generally, $C1 = 2\ n\ k\ t\ p_{yw}$

∴ $C1 = 2 \times 5\ t^2\ p_{yw}$ for continuous over bearing

∴ $C1 = 2 \times 2\ t^2\ p_{yw}$ for end bearing

(ii) Bearing factor $C2$ is equal to $2tp_{yw}$ for both webs and must be multiplied by b_1 to give the stiff bearing contribution.

(b) Buckling

The buckling resistance P_x of the two unstiffened webs is given by:

$P_x = (b_1 + n_1)\ 2\ t\ p_c$ (BS 5950-1 notation) But, not given in code. 4.5.3.1

$= b_1 C2 + C1$ (Capacity table notation)

where:

b_1 is the stiff bearing length

n_1 is the length obtained by dispersion at 45° through half the depth of the section

t is the wall thickness

p_c is the compressive strength based on:

- Web slenderness, $\lambda = 1.5\left[\frac{D - 2t}{t}\right]\sqrt{3}$ Table 24
- Strut curve (c). Annex C.2

Unless loads or reactions are applied through welded flange plates, the additional effects of moments in the web due to eccentric loading must be taken into account, which will result in lower buckling values.

Code Ref.

(i) Buckling factor $C1$ is the portion of $(n_1\ t\ p_c)$ due to the beam alone.

$C1 = 4D\ t\ p_c / 2$ for welded flange plates

$C1 = 4P$ for non-welded flange plates

where:

P is the limiting force in each web (derived below).

The factor 4 allows for two webs and dispersion of load in two directions and applies to a member that is continuous over bearing or an end bearing member with a continuously welded sealing plate.

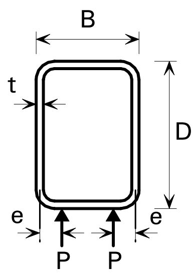
For non-welded flange plates, the equivalent eccentricity of loading from the centreline of the web is given by:

$$e = 0.026B + 0.978t + 0.002D$$

This expression has been derived from research [14] and is also applicable to cold-formed hollow sections [15]

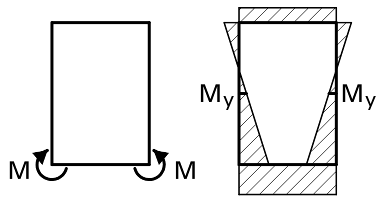
The loading P creates a fixed end moment of

$$M = Pe\left[\frac{B-t-e}{B-t}\right]$$



and thus the moment at the centreline of the web can be found as follows [16]:

$$M_y = \frac{0.5M\,(3+a)}{a^2+4a+3}$$



where:

a $= D/B$

Using the interaction expression $\dfrac{F}{Ap_c} + \dfrac{M_y}{p_y\ Z_y} \leq 1$

The limiting value of P is given by substituting for M_y and rearranging:

$$P = \frac{Dt^2 p_y p_c}{2t\,p_y + 12\left[M_y/P\right] p_c}$$

(ii) Buckling factor $C2$ is the stiff bearing component factor and is equal to $C1/D$ and must be multiplied by $b1$ to give the stiff bearing contribution.

Code Ref.

(c) Shear

The shear resistance capacity of the section is given by: 4.2.3

$P_v \quad = 0.6\, p_y\, A\, D\, /\, (D + B)$

where:

A is the cross-sectional area.

10 AXIAL & BENDING TABLES

10.1 Axial load and bending: UB sections, UC sections, joists, and parallel flange channels

Generally, members subject to axial compression and bending should be checked for cross-section capacity (Clause 4.8.3.2) and member buckling (Clause 4.8.3.3).

Columns in simple construction should be checked in accordance with Clause 4.7.7.

All the relevant parameters required to evaluate the interaction equations given in the above clauses have been presented in tabular form, as follows:

(a) Cross-section capacity check 4.8.3.2

The tables are applicable to members subject to combined tension and bending and also to members subject to combined compression and bending. However, the values in the tables are conservative for tension, as the more onerous compression section classification limits have been used.

Values are given in the tables for:

(i) $P_z = A_g p_y$ (p_y is the design strength)

(ii) F/P_z limits

The compact and semi-compact limits are the maximum values of F/P_z up to which the section is either compact or semi-compact, respectively. The compact limit is given in bold type.

(iii) M_{cx} and M_{cy}

These are the moment capacities (with low shear load) about the major and minor

axes respectively. They have been calculated as in Section 8.1 using S_x, S_{xeff}, and S_y, S_{yeff}, as appropriate.

Note: S_{xeff} and S_{yeff} can change with F/P_Z values.

Code Ref.

When F/P_z exceeds the semi-compact limit, the section is slender, due to the web and the moment capacities tabulated are based on a reduced design strength and the gross section properties, instead of $p_y Z_{eff}$

The symbol $ indicates that the section would be overloaded due to axial load alone i.e. the section is slender and $F > A_{eff}\, p_y$.

(iv) M_{rx} and M_{ry}

These have been determined using the reduced plastic moduli given in the section property tables for the values of F/P_z at the head of the table. Values of M_{rx} and M_{ry} are not valid for semi-compact and slender sections hence, no values are shown when F/P_z exceeds the limit for a compact section (shown as " – " in the tables).

$M_{rx} \quad = p_y\, S_{rx} \;\leq\; M_{cx}$ 4.8.2.3

$M_{ry} \quad = p_y\, S_{ry} \;\leq\; M_{cy}$ Annex I.2.1

(b) Member buckling check 4.8.3.3

The symbol * denotes that the section is slender when fully stressed under axial compression only (due to the web becoming slender). None of the sections listed are slender due to the flanges being slender. Under combined axial compression and bending, the section would be compact or semi-compact up to the given F/P_z limits.

Values are given in the table for:

(i) $P_z \quad = A_g\, p_y$

(ii) $p_y\, Z_x$

This is used in the simplified method for member buckling. 4.8.3.3.1

(iii) $p_y\, Z_y$

This is used for columns in simple construction and in the simplified method for member buckling. 4.7.7 4.8.3.3.1

(iv) F/P_z limit

The limits in normal and bold type are the maximum values up to which the section is either semi-compact or compact, respectively. The tabulated resistances are only valid up to the given F/P_z limit.

(v) P_{cx} and P_{cy}

These are the compression resistances for buckling about the major and minor axes respectively. The adjacent F/P_z limit, ensures that the section is not slender and have been calculated as in Section 6.1.

Code Ref.

(vi) M_b is the buckling resistance moment, used in both the simplified and the more exact method. Values of M_b are given for two F/P_z limits the higher limit ensures the section is semi-compact and the lower limit (in bold) ensures the section is compact, M_b is calculated accordingly as in Section 8.1.

(vii) M_{bs} is the buckling resistance moment for use with columns in simple construction and has been calculated as for M_b except that p_b has been obtained using a value of $\lambda_{LT} = 0.5\lambda$, where $\lambda = L/r_y$. As for M_b there are two sets of values that apply to the two adjacent F/P_z limits. 4.7.7

10.2 Axial load and bending: hollow sections

Generally, members subject to axial compression and bending should be checked for cross-section capacity and member buckling.

Columns in simple construction should be checked in accordance with Clause 4.7.7.

All the relevant parameters required to evaluate the interaction equations given in the above clauses have been presented in tabular form, as follows:

(a) Cross-section capacity check 4.8.3.2

The symbol * denotes that the section is slender when fully stressed under axial compression only.

Values are given in the tables for:

(i) $P_z = A_g p_y$ (p_y is the design strength)

(ii) F/P_z limits

The compact and semi-compact limits are the maximum values of F/P_z up to which the section is either compact or semi-compact, respectively. The compact limit is given in bold type. There are no F/P_z values for circular hollow sections.

(iii) M_c , M_{cx} and M_{cy}

M_c for circular and square hollow sections, M_{cx} and M_{cy} for rectangular hollow sections. These are the moment capacities (with low shear load) about the major and minor axes respectively. They have been calculated as in Section 8.2 using S_x, S_{xeff}, Z_x and Z_{xeff}, S_y, S_{yeff}, Z_y and Z_{yeff}, as appropriate.

When F/P_z exceeds the semi-compact limit, and therefore the section is slender, if it is the web of the section that is slender the moment capacities tabulated are based on a reduced design strength and the gross section properties.

The symbol $ indicates that the section would be overloaded due to axial load alone i.e. the section is slender and $F > A_{eff}\, p_y$.

Code Ref.

(iv) M_r , M_{rx} and M_{ry}

M_r for circular and square hollow sections, M_{rx} and M_{ry} for rectangular hollow sections. These have been determined using the reduced plastic moduli given in the section property tables for the values of F/P_z at the head of the table. The M_r, M_{rx} and M_{ry} values have been limited to the corresponding values of M_c, M_{cx} and M_{cy}. Values of M_{rx} and M_{ry} are not valid for semi-compact and slender sections therefore, no values are shown when F/P_z exceeds the limit for a compact section, shown as " – " in the tables. 4.8.2.3 Annex I.2.1

$$M_{rx} = p_y S_{rx} \leq M_{cx}$$

$$M_{ry} = p_y S_{ry} \leq M_{cy}$$

(b) Member buckling check 4.8.3.3

The simplified method (Clause 4.8.3.3.1) assumes that out-of-plane, lateral torsional buckling controls. However, in certain instances, it is likely that in-plane buckling, e.g. chord of truss with purlins not at node point, will be more critical. Thus, it is recommended that the more exact method should be used.

The symbol * denotes that the section is slender when fully stressed under axial compression only. Under combined axial compression and bending, the section would be compact or semi-compact up to the given F/P_z limits.

Values are given in the tables for:

(i) $p_y Z$

For circular and square hollow sections. This is used for columns in simple construction and in the simplified method for member buckling. 4.7.7 4.8.3.3.1

(ii) $p_y Z_x$

For rectangular hollow sections. This is used in the simplified method for member buckling. 4.8.3.3.1

(iii) $p_y Z_y$

For rectangular hollow sections. This is used for columns in simple construction and in the simplified method for member buckling. 4.7.7 4.8.3.3.1

(iv) F/P_z limit

For square and rectangular hollow sections. The compact and semi-compact limits are the maximum values of F/P_z up to which the section is either compact or semi-compact, respectively. The compact limit is given in bold type.

(v) P_c , P_{cx} and P_{cy}

P_c for circular and square hollow sections, P_{cx} and P_{cy} for rectangular hollow sections. These are the compression resistances for buckling about the relevant axes. They have been calculated as in Section 6.1. The adjacent F/P_z limit ensures that the section is not slender except where the F/P_z limit is zero. For this case the section is slender and allowance has been made in calculating the compression resistance, the values are displayed in italic type.

Code Ref.

(vi) M_b

M_b is the buckling resistance moment, used in both the simplified and the more exact method. Values of M_b are given for two F/P_z limits. The higher limit ensures the section is semi-compact and the lower limit (in bold) ensures the section is compact, M_b is calculated accordingly. For circular and square hollow sections, M_b equals M_c. For rectangular hollow sections, M_b equals M_c provided that L_E is within the limiting length L_c given in the Bending tables; see Section 8.2.2. 4.3.6.1

(vii) M_{bs} 4.7.7

M_{bs} is the buckling resistance moment for use with columns in simple construction. For hollow sections, M_{bs} equals M_b therefore values are not tabulated separately.

11 BOLTS, WELDS AND FLOOR PLATES

11.1 Bolt capacities

The types of bolts covered are:

- Grades 4.6, 8.8 and 10.9, as specified in BS 4190[17]: ISO metric black hexagon bolts, screws and nuts.
- Non-preloaded and preloaded HSFG bolts as specified in BS 4395[18]: High strength friction grip bolts and associated nuts and washers for structural engineering. Part 1: General grade and Part 2: Higher grade.

  Preloaded HSFG bolts should be tightened to minimum shank tension (P_o) as specified in BS 4604[19]
- Countersunk bolts as specified in BS 4933[20]: ISO metric black cup and countersunk bolts and screws with hexagon nuts

Information on assemblies of matching bolts, nuts and washers is given in BS 5950-2[1]

(a) Non-preloaded bolts, Ordinary (Grades 4.6, 8.8 and 10.9) and HSFG (General and Higher Grade):

(i) The tensile stress area (A_t) is obtained from the above standards:

(ii) The tension capacity of the bolt is given by:

$P_{nom} = 0.8 p_t A_t$ Nominal 6.3.4.2

$P_t = p_t A_t$ Exact 6.3.4.3

where:

p_t is the tension strength of the bolt. Table 34

Code Ref.

(iii) The shear capacity of the bolt is given by:

$$P_s = p_s A_s$$ 6.3.2.1

where:

p_s is the shear strength of the bolt — Table 30

A_s is the shear area of the bolt.

In the tables, A_s has been taken as equal to A_t.

The shear capacity given in the tables must be reduced for large packings, large grip lengths, kidney shaped slots or long joints when applicable. — 6.3.2.2, 6.3.2.3, 6.3.2.4, 6.3.2.5

(iv) The effective bearing capacity given is the lesser of the bearing capacity of the bolt given by:

$$P_{bb} = d\, t_p\, p_{bb}$$ 6.3.3.2

and the bearing capacity of the connected ply given by:

$$P_{bs} = k_{bs}\, d\, t_p\, p_{bs}$$ 6.3.3.3

assuming that the end distance is greater than or equal to twice the bolt diameter to meet the requirement that $P_{bs} \leq 0.5\, k_{bs}\, e\, t_p\, p_{bs}$

where:

d is the nominal diameter of the bolt

t_p is the thickness of the ply.

For countersunk bolts, t_p is taken as the ply thickness minus half the depth of countersinking. Depth of countersinking is taken as half the bolt diameter based on a 90^0 countersink. — 6.3.3.2

p_{bb} is the bearing strength of the bolt — Table 31

p_{bs} is the bearing strength of the ply — Table 32

e is the end distance

k_{bs} is a coefficient to allow for hole type. — 6.3.3.3

Tables assume standard clearance holes, therefore k_{bs} is taken as 1.0. For oversize holes and short slots, k_{bs} = 0.7. For long slots and kidney shaped slots, k_{bs} = 0.5.

Code Ref.

(b) Preloaded HSFG bolts (general grade and higher grade): 6.4

(i) The proof load of the bolt (P_o) is obtained from BS 4604[19]. The same proof load is used for countersunk bolts as for non-countersunk bolts. For this to be acceptable the head dimensions must be as specified in BS 4933[20].

(ii) The tension capacity (P_t) of the bolt is taken as: 6.4.5

$1.1\ P_o$ for non-slip in service

$0.9\ P_o$ for non-slip under factored load

(iii) The slip resistance of the bolt is given by: 6.4.2

$P_{SL} = 1.1\ K_s \mu P_o$ for non-slip in service

$P_{SL} = 0.9\ K_s \mu P_o$ for non-slip under factored load

where:

K_s is taken as 1.0 for fasteners in standard clearance holes 6.4.2

μ is the slip factor. Table 35

(iv) The bearing resistance is only applicable for non-slip in service and is taken as:

$P_{bg} = 1.5\ d\ t_p\ p_{bs}$ 6.4.4

assuming that the end distance is greater than or equal to three times the bolt diameter, to meet the requirement that $P_{bg} \le 0.5\ e\ t_p\ p_{bs}$.

where:

d is the nominal diameter of the bolt

t_p is the thickness of the ply

p_{bs} is the bearing strength of the ply. Table 32

(v) The shear capacity of the bolt is given by: 6.4.1 a)

$P_s = p_s A_s$ 6.3.2.1

where:

p_s is the shear strength of the bolt Table 30

A_s is the shear area of the bolt

In the tables, A_s has been taken as equal to A_t.

Code Ref.

11.2 Welds

Capacities of longitudinal and transverse fillet welds per unit length are tabulated. The weld capacities are given by,

Longitudinal shear capacity, $P_L = p_w a$ 6.8.7.3

Transverse capacity, $P_T = K p_w a$

where:

p_w is the weld design strength Table 37

a is the throat thickness, taken as 0.7 x the leg length

K is the enhancement factor for transverse welds. 6.8.7.3

The plates are assumed to be at 90° and therefore $K = 1.25$. Electrode classifications of E35 and E42 are assumed for steel grade S275 and S355 respectively. Welding consumables are in accordance with BS EN 440[21], BS EN 449[22], BS EN 756[23], BS EN 758[24], or BS EN 1668[25] as appropriate. Table 37

11.3 Floor plates

It is usual to consider floor plates as supported on all four edges, even though two edges may only be supported by stiffeners or joint covers. If the plates are securely bolted or welded to the supporting system, they may be considered as encastre, which increases the load carrying capacity slightly and reduces the deflection considerably.

The thickness given is exclusive of any raised pattern.

The breadth is the smaller dimension and the length the greater, irrespective of the position of the main support members.

The maximum uniformly distributed load on the plate (w) is given by Pounder's formula and the maximum skin stress is limited to the design strength p_y.

For calculating the maximum deflection (d_{max}) at serviceability, the uniformly distributed imposed load (w_{imp}) on plate, is derived as follows:

$$w = \gamma_{dead} w_{dead} + \gamma_{imp} w_{imp}$$

$$w_{imp} = \frac{w - \gamma_{dead} w_{dead}}{\gamma_{imp}}$$

Code Ref.

(a) For plates simply supported on all four edges

This formula assumes that there is no resistance to uplift at plate corners.

$$w = \frac{4 p_y t^2}{3kB^2 \left[1 + \frac{14}{75}(1-k) + \frac{20}{57}(1-k)^2 \right]}$$

$$d_{max} = \frac{m^2 - 1}{m^2} \times \frac{5k\, w_{imp}\, B^4}{32\, E\, t^3} \left[1 + \frac{37}{175}(1-k) + \frac{79}{201}(1-k)^2 \right]$$

Where resistance to uplift at corners is provided, the above formula will be conservative. Higher values may be obtained using the reference below.

(b) For plates encastre on all four edges

The plate must be secured to prevent uplift, which would otherwise occur at the plate corners.

$$w = \frac{2 p_y t^2}{k\, B^2 \left[1 + \frac{11}{35}(1-k) + \frac{79}{141}(1-k)^2 \right]}$$

$$d_{max} = \frac{m^2 - 1}{m^2} \times \frac{k\, w_{imp}\, B^4}{32\, E\, t^3} \left[1 + \frac{47}{210}(1-k) + \frac{200}{517}(1-k)^2 \right]$$

where:

L = length of plate (mm) ($L > B$)

B = breadth of plate (mm)

t = thickness of the plate (mm)

$k = \dfrac{L^4}{L^4 + B^4}$

p_y = design strength of plate

E = Young's modulus (205×10^3 N/mm^2)

$1/m$ = Poisson's ratio ($m = 3.0$)

γ_{dead} = load factor for dead load (1.4)

γ_{imp} = load factor for imposed load (1.6)

d_{max} = maximum deflection (mm) at serviceability due to imposed loads only

Code Ref.

w = uniformly distributed load on plate (ultimate) N/mm^2

w_{dead} = uniformly distributed self weight of plate (N/mm^2)

w_{imp} = uniformly distributed imposed load on plate (N/mm^2)

Tables are only given for grade S275 and have been based on plate design strength p_y of 275 N/mm^2.

Further explanation is given in Chapter 30 of *Steel Designers' Manual* [26]

12 SECTION DESIGNATIONS AND STEEL GRADES

12.1 Open Sections

The dimension and member capacity tables given in this publication are dual titled. The tables give the name of the section type, as given in the relevant British Standard (e.g. Universal beams and Universal columns, for I sections to BS4-1 and Equal leg angles and Unequal leg angles for sections to BS EN 10056-1) followed by the Corus designation in their Advance range of sections. An example of this dual titling is given below:

UNIVERSAL BEAMS

Advance UKB

The Advance range of sections encompasses all the UB, UC, Tee and PFC sections in BS 4-1 and most of the angle sections in BS EN 10056-1. The dimensions and properties of the Advance sections are the same as those of the corresponding British Standard sections and the same standards for dimensional tolerance apply. The Advance range also includes additional beam and column sections not in BS 4-1 and angle sections not in BS EN 10056-1; these are designated by '+' in the tables.

The only difference between Advance sections and BS sections is that the Advance sections are always CE Marked. The BS sections do not require CE Marking in the UK.

Tables are also included for Asymmetric Slimflor Beams. These sections are manufactured by Corus; they are part of the Advance range and they are CE Marked. Tables are included for joist sections to BS 4-1.

The table below shows the relationship between the BS 4-1 section designation and the section designation for the Advance sections.

Comparison of section designation systems

BS designation		Corus Advance designation	
Universal Beam	UB*	UK Beam	UKB
Universal Column	UC*	UK Column	UKC
Parallel Flange Channel	PFC*	UK Parallel Flange Channel	UKPFC
Tee		UK Tee	UKT
Equal leg angle Unequal leg angle	L	UK Angle	UKA
* These abbreviations are commonly used but are not a BS designation			

Where capacity tables cover Advance sections, the steel grade is also dual titled. The strength grade designation given in BS EN 10025-2: 2004 is given first, followed by the grade designation for the Advance sections. An example of this dual titling of the steel grade is given below:

S 275 / Advance275

The Advance designation is a simplified designation that encompasses the specification to BS EN 10025 and the additional quality control procedures to ensure CE Marking. It also enables a shorter form of designating the grade when ordering. The table below shows the designations in the two systems.

Steel grade designations

BS Designation	Advance Sections designation
BS EN 10025-2:2004 S275JR	Advance275JR
BS EN 10025-2:2004 S275J0	Advance275J0
BS EN 10025-2:2004 S275J2	Advance275J2
BS EN 10025-2:2004 S355JR	Advance355JR
BS EN 10025-2:2004 S355J0	Advance355J0
BS EN 10025-2:2004 S355J2	Advance355J2
BS EN 10025-2:2004 S355K2	Advance355K2

12.2 Hollow Sections

The dimension and member capacity tables given in this publication are dual titled. The tables give the name of the section type, as given in the relevant British Standard (e.g. Hot finished circular hollow section, for a circular section to BS EN 10210-2 and Cold formed square hollow section, for a square section to BS EN 10219-2), followed by the Corus designation from the Celsius® range of hot finished sections or from the Hybox® range of high strength cold formed sections. An example of this dual titling, for hot finished circular hollow sections, is as follows:

HOT FINISHED CIRCULAR HOLLOW SECTIONS

Celsius® CHS

The tables include circular, square, rectangular and elliptical hollow sections available in the Celsius® and Hybox® ranges and the dimensional and sectional properties are either as tabulated in the Standards or are calculated in accordance with the Standards. The only difference between a section to BS EN 10210-2 or to BS EN 10210-2 and its equivalent Celsius® or Hybox® sections is that the Corus section will always be CE Marked.

The table below shows the relationship between section designations in BS EN 10210: 2006 and BS EN 10219: 2006, and those for Celsius® and Hybox® sections produced by Corus.

Comparison of designation systems for hollow sections

BS EN 10210: 2006	**Corus designation**
Hot finished circular hollow section	Celsius® CHS
Hot finished square hollow section	Celsius® SHS
Hot finished rectangular hollow section	Celsius® RHS
Hot finished elliptical hollow section	Celsius® OHS
BS EN 10219: 2006	
Cold formed circular hollow section	Hybox® CHS
Cold formed square hollow section	Hybox® SHS
Cold formed rectangular hollow section	Hybox® RHS

In the capacity tables the steel grade is also dual titled. The strength grade designation given in BS EN 10210-1 and BS EN 10219-1 is given first, followed by the grade designation for the Celsius® or Hybox® sections. An example of this dual titling of the steel grade is given below:

S 355 / Celsius® 355

In all cases, the mechanical properties of Celsius® or Hybox® hollow sections meet all the requirements given in BS EN 10210-1:2006 or BS EN 10219-1:2006, as appropriate. The table below shows the relationship between the steel grades given in the standards and those for Celsius® and Hybox® sections from Corus.

Comparison of designations for hollow sections

BS Designation	Corus designation
BS EN 10210-1: 2006 S355J2H	Celsius® 355*
BS EN 10219-1: 2006 S355J2H	Hybox® 355

* A limited range of sections is also available in grade S355K2H – consult Corus for availability

13 REFERENCES

1 BRITISH STANDARDS INSTITUTION
BS 5950 Structural use of steelwork in building
BS 5950-1: 2000 Code of Practice for design – Rolled and welded sections
BS 5950-2: 2000 Specification for materials, fabrication and erection: Rolled and welded sections

2 BRITISH STANDARDS INSTITUTION
BS EN 10025: 1993 Hot rolled products of non-alloy structural steels. Technical delivery conditions (including amendment 1995)
Replaced by BS EN 10025-2:2004

3 BRITISH STANDARDS INSTITUTION
BS EN 10113 Hot rolled products in weldable fine grain structural steels
BS EN 10113-1: 1993 General delivery conditions. (Replaces BS 4360:1990)
Replaced by BS EN 10025-3:2004 and BS EN 10025-4:2004

4 BRITISH STANDARDS INSTITUTION
BS4 Structural steel sections
BS4-1: 1993 Specification for hot rolled sections
(Including amendment 2001)
Replaced by BS 4-1:2005

5 BRITISH STANDARDS INSTITUTION
BS EN 10034: 1993 Structural steel I and H sections. Tolerances on shape and dimensions (Replaces BS 4-1: 1980)

6 BRITISH STANDARDS INSTITUTION
BS EN 10024: 1995 Hot rolled taper flange I sections. Tolerances on shape and dimensions

7 BRITISH STANDARDS INSTITUTION
BS EN 10279: 2000 Hot rolled steel channels. Tolerances on shape, dimension and mass (Including amendment 1, amendment 2: 200)

8 BRITISH STANDARDS INSTITUTION
BS EN 10056 Specification for structural steel equal and unequal angels
BS EN 10056-1: 1999 Dimensions (Replaces BS 4848-4: 1972)
BS EN 10056-2: 1999 Tolerances on shape and dimensions (Replaces BS 4848-4: 1972)

9 BRITISH STANDARDS INSTITUTION
BS EN 10210 Hot finished structural hollow sections of non-alloy and fine grain structural steels
BS EN 10210-1: 1994 Technical delivery requirements (Replaces BS 4360: 1990)
Replaced by BS EN 10210-1:2006
BS EN 10210-2: 1997 Tolerances, dimensions and sectional properties. (Replaces BS 4848-2: 1991)
Replaced by BS EN 10210-2:2006

10 BRITISH STANDARDS INSTITUTION
BS EN 10219 Cold formed welded structural sections of non-alloy and fine grain steels
BS EN 10219-1: 1997 Technical delivery requirements
Replaced by BS EN 10219-1:2006
BS EN 10219-2: 1997 Tolerances and sectional properties. (Replaces BS 6363: 1983)
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11 Structural sections to BS 4: Part 1: 1963 and BS EN 10056: 1999
Corus Construction and Industrial Sections, 03/2001

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Brighton University, January 1971].

15 SALTER, P.R.
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The Steel Construction Institute, 1998

16 KLEINLOGEL, A.
Rigid frame formulas (translated from German; page 418)
Frederick Ungar Publishing Co., 1958

17 BRITISH STANDARDS INSTITUTION
BS 4190: 2001 ISO metric black hexagon bolts, screws and nuts - Specification

18 BRITISH STANDARDS INSTITUTION
BS 4395 Specification for high strength friction grip bolts and associated nuts and washers for structural engineering
BS 4395-1: 1969 General grade (including amendments 1, amendments 2: 1997)
BS 4395-2: 1969 Higher grade bolts and nuts and general grade washers (including amendment 1, amendment 2: 1976)
BS 4395 is current (and may be used) although superseded by the six Parts of BS EN 14399

19 BRITISH STANDARDS INSTITUTION
BS 4604 Specification for the use of high strength friction grip bolts in structural steelwork. Metric series
BS 4604-1: 1970 General grade (including amendment 1, amendment 2, and amendment 3: 1982)
BS 4604-2: 1970 – Higher grade (parallel shank) (including amendment 1, amendment 2: 1972)

20 BRITISH STANDARDS INSTITUTION
BS 4933: 1973 Specification for ISO metric black cup and countersunk head bolts and screws with hexagon nuts

21 BRITISH STANDARDS INSTITUTION
BS EN 440: 1995 Welding consumables. Wire electrodes and deposits for gas shielded metal arc welding of non alloy and fine grain steels. Classification

22 BRITISH STANDARDS INSTITUTION
BS EN 499: 1995 Welding consumables. Covered electrodes for manual metal arc welding of non alloy and fine grain. Classification
Replaced by BS EN ISO 2560:2005

23 BRITISH STANDARDS INSTITUTION
BS EN 756: 1996 Welding consumables. Wire electrodes and wire-flux combinations for submerged arc welding of non alloy and fine grain steels. Classification
Replaced by BS EN 756:2004

24 BRITISH STANDARDS INSTITUTION
BS EN 758: 1997 Welding consumables. Tubular cored electrodes for metal arc welding with and without a gas shield of non-alloy and fine grain steels. Classification

25 BRITISH STANDARDS INSTITUTION
BS EN 1668: 1997 Welding consumables. Rods, wires and deposits for tungsten inert gas welding of non alloy and fine grain steels. Classification

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The Steel Construction Institute and Blackwell Science, 1994
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