

Structural use of steelwork in building —

Part 1: Code of practice for design — Rolled and welded sections

ICS 91.080.10

Committees responsible for this British Standard

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British Constructional Steelwork Association

Building Research Establishment Ltd

Cold Rolled Sections Association

Confederation of British Metalforming

DETR (Construction Directorate)

DETR (Highways Agency)

Health and Safety Executive

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Steel Construction Institute

UK Steel Association

Welding Institute

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Foreword

This part of BS 5950 supersedes BS 5950-1:1990, which is withdrawn. A period of three months is being allowed for users to convert to the new standard. This edition introduces technical changes based on a review of the standard, but it does not constitute a full revision.

The start and finish of text introduced or altered by Amendment No. 1 is indicated in the text by tags **A1** **A1**.

This new edition has been prepared following the issue of a number of new related standards adopting European or international standards for materials and processes, plus revisions to standards for loading. It also reflects the transfer of cold formed structural hollow sections from BS 5950-5 to BS 5950-1.

Clauses updated technically include those for sway stability, avoidance of disproportionate collapse, resistance to brittle fracture, local buckling, lateral-torsional buckling, shear resistance, stiffeners, members subject to combined axial force and bending moment, joints, connections and testing. In all cases the reason for changing the recommendations on a topic is structural safety, but where possible some adjustments based on improved knowledge have also been made to the recommendations on these topics to offset potential reductions in economy.

Some of the text has been edited to reduce the risk of misapplication. In addition some topics omitted until now have been added from BS 449, including separators and diaphragms and eccentric loads on beams.

BS 5950 is a standard combining codes of practice covering the design, construction and fire protection of steel structures and specifications for materials, workmanship and erection. It comprises the following parts:

- *Part 1: Code of practice for design — Rolled and welded sections;*
- *Part 2: Specification for materials, fabrication and erection — Rolled and welded sections;*
- *Part 3: Design in composite construction — Section 3.1: Code of practice for design of simple and continuous composite beams;*
- *Part 4: Code of practice for design of composite slabs with profiled steel sheeting;*
- *Part 5: Code of practice for design of cold formed thin gauge sections;*
- *Part 6: Code of practice for design of light gauge profiled steel sheeting;*
- *Part 7: Specification for materials, fabrication and erection — Cold formed sections and sheeting;*
- *Part 8: Code of practice for fire resistant design;*
- *Part 9: Code of practice for stressed skin design.*

Part 1 gives recommendations for the design of simple and continuous steel structures, using rolled and welded sections. Its provisions apply to the majority of such structures, although it is recognized that cases will arise when other proven methods of design may be more appropriate.

This part does not apply to other steel structures for which appropriate British Standards exist.

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people and that construction and supervision will be carried out by capable and experienced organizations.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading. For materials and workmanship reference should be made to BS 5950-2. For erection, reference should be made to BS 5950-2 and BS 5531.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 215 and a back cover.

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Section 1. General

1.1 Scope

This part of BS 5950 gives recommendations for the design of structural steelwork using hot rolled steel sections, flats, plates, hot finished structural hollow sections and cold formed structural hollow sections, in buildings and allied structures not specifically covered by other standards.

NOTE 1 These recommendations assume that the standards of materials and construction are as specified in BS 5950-2.

NOTE 2 Design using cold formed structural hollow sections conforming to BS EN 10219 is covered by this part of BS 5950. Design using other forms of cold formed sections is covered in BS 5950-5.

NOTE 3 Design for seismic resistance is not covered in BS 5950.

NOTE 4 The publications referred to in this standard are listed on page 213.

Detailed recommendations for practical direct application of “second order” methods of global analysis (based on the final deformed geometry of the frame), including allowances for geometrical imperfections and residual stresses, strain hardening, the relationship between member stability and frame stability and appropriate failure criteria, are beyond the scope of this document. However, such use is not precluded provided that appropriate allowances are made for these considerations (see 5.1.1).

The test procedures of 7.1.2 are intended only for steel structures within the scope of this part of BS 5950. Other cases are covered in Section 3.1 or Parts 4, 5, 6 and 9 of BS 5950 as appropriate.

1.2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this British Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 2573-1, *Rules for the design of cranes — Part 1: Specification for classification, stress calculations and design criteria for structures.*

BS 2853, *Specification for the design and testing of steel overhead runway beams.*

BS 3100, *Specification for steel castings for general engineering purposes.*

BS 4395-1, *Specification for high strength friction grip bolts and associated nuts and washers for structural engineering — Part 1: General grade.*

BS 4395-2, *Specification for high strength friction grip bolts and associated nuts and washers for structural engineering — Part 2: Higher grade bolts and nuts and general grade washers.*

BS 4449, *Specification for carbon steel bars for the reinforcement of concrete.*

BS 4482, *Specification for cold reduced steel wire for the reinforcement of concrete.*

BS 4483, *Steel fabric for the reinforcement of concrete.*

BS 4604-1, *Specification for the use of high strength friction grip bolts in structural steelwork — Metric series — Part 1: General grade.*

BS 4604-2, *Specification for the use of high strength friction grip bolts in structural steelwork — Metric series — Part 2: Higher grade (parallel shank).*

BS 5400-3, *Steel, concrete and composite bridges — Part 3: Code of practice for the design of steel bridges.*

BS 5950-2, *Structural use of steelwork in building — Part 2: Specification for materials, fabrication and erection — Rolled and welded sections.*

BS 5950-3, *Structural use of steelwork in building — Part 3: Design in composite construction — Section 3.1: Code of practice for design of simple and continuous composite beams.*

BS 5950-4, *Structural use of steelwork in building — Part 4: Code of practice for design of composite slabs with profiled steel sheeting.*

BS 5950-5, *Structural use of steelwork in building — Part 5: Code of practice for design of cold formed thin gauge sections.*

BS 5950-6, *Structural use of steelwork in building — Part 6: Code of practice for design of light gauge profiled steel sheeting.*

BS 5950-9, *Structural use of steelwork in building — Part 9: Code of practice for stressed skin design.*

- BS 6399-1, *Loading for buildings — Part 1: Code of practice for dead and imposed loads.*
- BS 6399-2, *Loading for buildings — Part 2: Code of practice for wind loads.*
- BS 6399-3, *Loading for buildings — Part 3: Code of practice for imposed roof loads.*
- BS 7419, *Specification for holding down bolts.*
- BS 7608, *Code of practice for fatigue design and assessment of steel structures.*
- BS 7644-1, *Direct tension indicators — Part 1: Specification for compressible washers.*
- BS 7644-2, *Direct tension indicators — Part 2: Specification for nut face and bolt face washers.*
- BS 7668, *Specification for weldable structural steels — Hot finished structural hollow sections in weather resistant steels.*
- BS 8002, *Code of practice for earth retaining structures.*
- BS 8004, *Code of practice for foundations.*
- BS 8110-1, *Structural use of concrete — Part 1: Code of practice for design and construction.*
- BS 8110-2, *Structural use of concrete — Part 2: Code of practice for special circumstances.*
- BS EN 10002-1, *Tensile testing of metallic materials — Part 1: Method of test at ambient temperature.*
- BS EN 10025, *Hot rolled products of non-alloy structural steels — Technical delivery conditions.*
- BS EN 10113-2, *Hot-rolled products in weldable fine grain structural steels — Part 2: Delivery conditions for normalized/normalized rolled steels.*
- BS EN 10113-3, *Hot-rolled products in weldable fine grain structural steels — Part 3: Delivery conditions for thermomechanical rolled steels.*
- BS EN 10137-2, *Plates and wide flats made of high yield strength structural steels in the quenched and tempered or precipitation hardened conditions — Part 2: Delivery conditions for quenched and tempered steels.*
- BS EN 10155, *Structural steels with improved atmospheric corrosion resistance — Technical delivery conditions.*
- BS EN 10210-1, *Hot finished structural hollow sections of non-alloy and fine grain structural steels — Part 1: Technical delivery requirements.*
- BS EN 10219-1, *Cold formed welded structural hollow sections of non-alloy and fine grain steels — Part 1: Technical delivery requirements.*
- BS EN 10250-2, *Open die steel forgings for general engineering purposes — Part 2: Non-alloy quality and special steels.*
- BS EN 22553, *Welded, brazed and soldered joints — Symbolic representation on drawings.*
- CP2, *Earth retaining structures.* Civil Engineering Code of Practice No. 2. London: The Institution of Structural Engineers, 1951.
- CP3:Ch V:Part 2, *Code of basic data for the design of buildings — Chapter V: Loading — Part 2: Wind loads.* London: BSI, 1972.

NOTE Publications to which informative reference is made for information or guidance are listed in the Bibliography.

1.3 Terms and definitions

For the purposes of this part of BS 5950, the following terms and definitions apply.

1.3.1

beam

a member predominantly subject to bending

1.3.2

brittle fracture

brittle failure of steel at low temperature

1.3.3**buckling resistance**

limit of force or moment that a member can withstand without buckling

1.3.4**built-up**

constructed by interconnecting more than one rolled section to form a single member

1.3.5**cantilever**

a beam that is fixed at one end and free to deflect at the other

1.3.6**capacity**

limit of force or moment that can be resisted without failure due to yielding or rupture

1.3.7**column**

a vertical member carrying axial force and possibly moments

1.3.8**compact cross-section**

a cross-section that can develop its plastic moment capacity, but in which local buckling prevents rotation at constant moment

1.3.9**compound section**

sections, or plates and sections, interconnected to form a single member

1.3.10**connection**

location where a member is fixed to a supporting member or other support, including the bolts, welds and other material used to transfer loads

1.3.11**dead load**

a load of constant magnitude and position that acts permanently, including self-weight

1.3.12**design strength**

the notional yield strength of the material used in design, obtained by applying partial factors to the specified minimum yield strength and tensile strength of the material

1.3.13**dynamic load**

part of an imposed load resulting from motion

1.3.14**edge distance**

distance from the centre of a bolt hole to the nearest edge of an element, measured perpendicular to the direction in which the bolt bears

1.3.15**effective length**

for a beam. Length between adjacent restraints against lateral-torsional buckling, multiplied by a factor that allows for the effect of the actual restraint conditions compared to a simple beam with torsional end restraint

for a compression member. Length between adjacent lateral restraints against buckling about a given axis, multiplied by a factor that allows for the effect of the actual restraint conditions compared to pinned ends

1.3.16**elastic analysis**

structural analysis that assumes no redistribution of moments in a continuous member or frame due to plastic hinge rotation

1.3.17**empirical method**

simplified method of design justified by experience or by tests

1.3.18**end distance**

distance from the centre of a bolt hole to the edge of an element, measured parallel to the direction in which the bolt bears

1.3.19**factored load**

specified load multiplied by the relevant partial factor

1.3.20**fatigue**

damage to a structural member caused by repeated application of stresses that are insufficient to cause failure by a single application

1.3.21**foundation**

part of a structure that distributes load directly to the ground

1.3.22**friction grip connection**

a bolted connection that relies on friction to transmit shear between components

1.3.23**H-section**

section with a central web and two flanges, that has an overall depth not greater than 1.2 times its overall width

1.3.24**hybrid section**

I-section with a web of a lower strength grade than the flanges

1.3.25**I-section**

section with a central web and two flanges, that has an overall depth greater than 1.2 times its overall width

1.3.26**imposed load**

load on a structure or member, other than wind load, produced by the external environment or the intended occupancy or use

1.3.27**instability**

inability to carry further load due to vanishing stiffness

1.3.28**joint**

element of a structure that connects members together and enables forces and moments to be transmitted between them

1.3.29**lateral restraint**

for a beam. Restraint that prevents lateral movement of the compression flange

for a compression member. Restraint that prevents lateral movement of the member in a given plane

1.3.30**longitudinal**

along the length of the member

1.3.31**notched end**

connected end of a member with one or both flanges cut away locally for clearance

1.3.32**pattern loading**

loads arranged to give the most severe effect on a particular element

1.3.33**pitch**

distance between centres of bolts lying in the direction of force transfer

1.3.34**plastic analysis**

structural analysis that allows for redistribution of moments in a continuous member or frame due to plastic hinge rotation

1.3.35**plastic cross-section**

a cross-section that can develop a plastic hinge with sufficient rotation capacity to allow redistribution of bending moments within a continuous member or frame

1.3.36**plastic load factor**

the ratio by which each of the factored loads would have to be increased to produce a plastic hinge mechanism

1.3.37**plastic moment**

moment capacity allowing for redistribution of stress within a cross-section

1.3.38**portal frame**

a single storey frame with rigid moment-resisting joints

1.3.39**preloaded bolt**

bolt tightened to a specified initial tension

1.3.40**rotation capacity**

the angle through which a joint can rotate without failing

1.3.41**rotational stiffness**

the moment required to produce unit rotation in a joint

1.3.42**segment**

a portion of the length of a member, between adjacent points that are laterally restrained

1.3.43**semi-compact cross-section**

a cross-section that can develop its elastic capacity in compression or bending, but in which local buckling prevents development of its plastic moment capacity

1.3.44**slender cross-section**

a cross-section in which local buckling prevents development of its elastic capacity in compression and/or bending

1.3.45**slenderness**

the effective length divided by the radius of gyration

1.3.46**slip resistance**

limit of shear that can be applied before slip occurs in a friction grip connection

1.3.47**stability**

resistance to failure by buckling or loss of static equilibrium

1.3.48**strength**

resistance to failure by yielding or buckling

1.3.49**strut**

member carrying predominantly axial compressive force

1.3.50**sub-frame**

part of a larger frame

1.3.51**torsional restraint**

restraint that prevents rotation of a member about its longitudinal axis

1.3.52**transverse**

direction perpendicular to the stronger of the rectangular axes of the member

1.3.53**welded section**

cross-section fabricated from plates by welding

1.4 Major symbols

A	Area
A_e	Effective net area
A_{eff}	Effective cross-sectional area
A_g	Gross cross-sectional area
A_n	Net area
A_s	Shear area of a bolt
A_t	Tensile stress area of a bolt
A_v	Shear area of a member
a	Spacing of transverse stiffeners
or	Effective throat size of weld
B	Width
b	Outstand

D	Depth of section
<i>or</i>	Diameter of section
<i>or</i>	Diameter of hole
d	Depth of web
<i>or</i>	Nominal diameter of bolt
E	Modulus of elasticity of steel
e	Edge or end distance
F_c	Compressive axial force
F_s	Shear force in a bolt
F_t	Tensile axial force
F_v	Shear force in a member
f_c	Compressive stress due to axial force
f_v	Shear stress
H	Warping constant of section
h	Storey height
I_x	Second moment of area about the major axis
I_y	Second moment of area about the minor axis
J	Torsion constant of section
L	Length
<i>or</i>	Span
L_E	Effective length
M	Moment
M_b	Buckling resistance moment (lateral-torsional buckling)
M_c	Moment capacity
M_r	Reduced moment capacity in the presence of an axial force
m	Equivalent uniform moment factor
P_{bb}	Bearing capacity of a bolt
P_{bg}	Friction grip bearing capacity
P_{bs}	Bearing capacity of connected parts
P_c	Compression resistance
P_s	Shear capacity of a bolt
P_{sL}	Slip resistance provided by a preloaded bolt
P_t	Tension capacity of a member or bolt
P_v	Shear capacity of a member
p_b	Bending strength (lateral-torsional buckling)
p_{bb}	Bearing strength of a bolt
p_{bs}	Bearing strength of connected parts
p_c	Compressive strength
p_s	Shear strength of a bolt
p_t	Tension strength of a bolt
p_w	Design strength of a fillet weld
p_y	Design strength of steel
q_w	Shear buckling strength of a web
r_x	Radius of gyration about the major axis

r_y	Radius of gyration about the minor axis
S_{eff}	Effective plastic modulus
S_x	Plastic modulus about the major axis
S_y	Plastic modulus about the minor axis
s	Leg length of a fillet weld
T	Thickness of a flange
t	Thickness
or	Thickness of a web
t_p	Thickness of a connected part
u	Buckling parameter of a cross-section
V_b	Shear buckling resistance of a web
V_{cr}	Critical shear buckling resistance of a web
ν	Slenderness factor for a beam
x	Torsional index of a cross-section
Z_{eff}	Effective section modulus
Z_x	Section modulus about the major axis (minimum value unless otherwise stated)
Z_y	Section modulus about the minor axis (minimum value unless otherwise stated)
γ_f	Overall load factor
ε	Constant $(275/p_y)^{0.5}$
λ	Slenderness, i.e. the effective length divided by the radius of gyration
λ_{cr}	Elastic critical load factor
λ_{L0}	Limiting equivalent slenderness (lateral-torsional buckling)
λ_{LT}	Equivalent slenderness (lateral-torsional buckling)
λ_0	Limiting slenderness (axial compression)

1.5 Other materials

Where other structural materials are used in association with structural steelwork, they should conform to the appropriate British Standard.

1.6 Design documents

The design documents should contain sufficient information to enable the design to be detailed and the structure fabricated and erected.

The design documents should state the assumed behaviour of the structure, the design assumptions and whether any loads or reactions quoted are factored or unfactored.

Where weld symbols are used on drawings they should be in accordance with BS EN 22553, which should be referenced on the drawings concerned.

1.7 Reference to BS 5400-3

In BS 5400-3 the nominal values of material strengths and the method of applying partial safety factors are different, see Annex A. These differences should be taken into account when referring to BS 5400-3.

Section 2. Limit states design

2.1 General principles and design methods

2.1.1 General principles

2.1.1.1 *Aims of structural design*

The aim of structural design should be to provide, with due regard to economy, a structure capable of fulfilling its intended function and sustaining the specified loads for its intended life. The design should facilitate safe fabrication, transport, handling and erection. It should also take account of the needs of future maintenance, final demolition, recycling and reuse of materials.

The structure should be designed to behave as a one three-dimensional entity. The layout of its constituent parts, such as foundations, steelwork, joints and other structural components should constitute a robust and stable structure under normal loading to ensure that, in the event of misuse or accident, damage will not be disproportionate to the cause.

To achieve these aims the basic anatomy of the structure by which the loads are transmitted to the foundations should be clearly defined. Any features of the structure that have a critical influence on its overall stability should be identified and taken account of in the design.

Each part of the structure should be sufficiently robust and insensitive to the effects of minor incidental loads applied during service that the safety of other parts is not prejudiced. Reference should be made to 2.4.5.

Whilst the ultimate limit state capacities and resistances given in this standard are to be regarded as limiting values, the purpose in design should be to reach these limits in as many parts of the structure as possible, to adopt a layout such that maximum structural efficiency is attained and to rationalize the steel member sizes and details in order to obtain the optimum combination of materials and workmanship, consistent with the overall requirements of the structure.

2.1.1.2 *Overall stability*

The designer who is responsible for the overall stability of the structure should be clearly identified. This designer should ensure the compatibility of the structural design and detailing between all those structural parts and components that are required for overall stability, even if some or all of the structural design and detailing of those structural parts and components is carried out by another designer.

2.1.1.3 *Accuracy of calculation*

For the purpose of deciding whether a particular recommendation is satisfied, the final value, observed or calculated, expressing the result of a test or analysis should be rounded off. The number of significant places retained in the rounded off value should be the same as in the relevant value recommended in this standard.

2.1.2 Methods of design

2.1.2.1 *General*

Structures should be designed using the methods given in 2.1.2.2, 2.1.2.3, 2.1.2.4 and 2.1.2.5.

In each case the details of the joints should be such as to fulfil the assumptions made in the relevant design method, without adversely affecting any other part of the structure.

2.1.2.2 *Simple design*

The joints should be assumed not to develop moments adversely affecting either the members or the structure as a whole.

The distribution of forces may be determined assuming that members intersecting at a joint are pin connected. The necessary flexibility in the connections may result in some non-elastic deformation of the materials, other than the bolts.

The structure should be laterally restrained, both in-plane and out-of-plane, to provide sway stability, see 2.4.2.5, and resist horizontal forces, see 2.4.2.3.

2.1.2.3 Continuous design

Either elastic or plastic analysis may be used.

For elastic analysis the joints should have sufficient rotational stiffness to justify analysis based on full continuity. The joints should also be capable of resisting the moments and forces resulting from the analysis.

For plastic analysis the joints should have sufficient moment capacity to justify analysis assuming plastic hinges in the members. The joints should also have sufficient rotational stiffness for in-plane stability.

2.1.2.4 Semi-continuous design

This method may be used where the joints have some degree of strength and stiffness, but insufficient to develop full continuity. Either elastic or plastic analysis may be used.

The moment capacity, rotational stiffness and rotation capacity of the joints should be based on experimental evidence. This may permit some limited plasticity, provided that the capacity of the bolts or welds is not the failure criterion. On this basis, the design should satisfy the strength, stiffness and in-plane stability requirements of all parts of the structure when partial continuity at the joints is taken into account in determining the moments and forces in the members.

NOTE Details of design procedures of this type are given in references [1] and [2], see Bibliography.

2.1.2.5 Experimental verification

Where design of a structure or element by calculation in accordance with any of the preceding methods is not practicable, or is inappropriate, the strength, stability, stiffness and deformation capacity may be confirmed by appropriate loading tests in accordance with Section 7.

2.1.3 Limit states concept

Structures should be designed by considering the limit states beyond which they would become unfit for their intended use. Appropriate partial factors should be applied to provide adequate degrees of reliability for ultimate limit states and serviceability limit states. Ultimate limit states concern the safety of the whole or part of the structure. Serviceability limit states correspond to limits beyond which specified service criteria are no longer met.

Examples of limit states relevant to steel structures are given in Table 1. In design, the limit states relevant to that structure or part should be considered.

The overall factor in any design has to cover variability of:

- material strength: γ_m
- loading: γ_ℓ
- structural performance: γ_p

In this code the material factor γ_m is incorporated in the recommended design strengths. For structural steel the material factor is taken as 1.0 applied to the yield strength Y_s or 1.2 applied to the tensile strength U_s . Different values are used for bolts and welds.

The values assigned for γ_ℓ and γ_p depend on the type of load and the load combination. Their product is the factor γ_f by which the specified loads are to be multiplied in checking the strength and stability of a structure, see 2.4. A detailed breakdown of γ factors is given in Annex A.

Table 1 — Limit states

Ultimate limit states (ULS)	Serviceability limit states (SLS)
Strength (including general yielding, rupture, buckling and forming a mechanism), see 2.4.1.	Deflection, see 2.5.2.
Stability against overturning and sway stability, see 2.4.2.	Vibration, see 2.5.3.
Fracture due to fatigue, see 2.4.3.	Wind induced oscillation, see 2.5.3.
Brittle fracture, see 2.4.4.	Durability, see 2.5.4.

2.2 Loading

2.2.1 General

All relevant loads should be considered separately and in such realistic combinations as to comprise the most critical effects on the elements and the structure as a whole. The magnitude and frequency of fluctuating loads should also be considered.

Loading conditions during erection should receive particular attention. Settlement of supports should be taken into account where necessary.

2.2.2 Dead, imposed and wind loading

The dead and imposed loads should be determined from BS 6399-1 and BS 6399-3; wind loads should be determined from BS 6399-2 or CP3:Ch V:Part 2.

NOTE In countries other than the UK, loads can be determined in accordance with this clause, or in accordance with local or national provisions as appropriate.

2.2.3 Loads from overhead travelling cranes

For overhead travelling cranes, the vertical and horizontal dynamic loads and impact effects should be determined in accordance with BS 2573-1. The values for cranes of loading class Q3 and Q4 as defined in BS 2573-1 should be established in consultation with the crane manufacturer.

2.2.4 Earth and ground-water loading

The earth and ground-water loading to which the partial factor γ_f of 1.2 given in Table 2 applies, should be taken as the worst credible earth and ground-water loads obtained in accordance with BS 8002. Where other earth and ground-water loads are used, such as nominal loads determined in accordance with CP2, the value of the partial factor γ_f should be taken as 1.4.

When applying γ_f to earth and ground-water loads, no distinction should be made between adverse and beneficial loads. Moreover, the same value of γ_f should be applied in any load combination.

2.3 Temperature change

Where, in the design and erection of a structure, it is necessary to take account of changes in temperature, it may be assumed that in the UK the average temperature of internal steelwork varies from -5°C to $+35^\circ\text{C}$. The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in other conditions, and in locations abroad subjected to different temperature ranges.

2.4 Ultimate limit states

2.4.1 Limit state of strength

2.4.1.1 General

In checking the strength of a structure, or of any part of it, the specified loads should be multiplied by the relevant partial factors γ_f given in Table 2. The factored loads should be applied in the most unfavourable realistic combination for the part or effect under consideration.

The load carrying capacity of each member and connection, as determined by the relevant provisions of this standard, should be such that the factored loads would not cause failure.

In each load combination, a γ_f factor of 1.0 should be applied to dead load that counteracts the effects of other loads, including dead loads restraining sliding, overturning or uplift.

2.4.1.2 Buildings without cranes

In the design of buildings not subject to loads from cranes, the following principal combinations of loads should be taken into account:

- Load combination 1: Dead load and imposed load (gravity loads);
- Load combination 2: Dead load and wind load;
- Load combination 3: Dead load, imposed load and wind load.

Table 2 — Partial factors for loads γ_f

Type of load and load combination	Factor γ_f
Dead load, except as follows.	1.4
Dead load acting together with wind load and imposed load combined.	1.2
Dead load acting together with crane loads and imposed load combined.	1.2
Dead load acting together with crane loads and wind load combined.	1.2
Dead load whenever it counteracts the effects of other loads.	1.0
Dead load when restraining sliding, overturning or uplift.	1.0
Imposed load.	1.6
Imposed load acting together with wind load.	1.2
Wind load.	1.4
Wind load acting together with imposed load.	1.2
Storage tanks, including contents.	1.4
Storage tanks, empty, when restraining sliding, overturning or uplift.	1.0
Earth and ground-water load, worst credible values, see 2.2.4.	1.2
Earth and ground-water load, nominal values, see 2.2.4.	1.4
Exceptional snow load (due to local drifting on roofs, see 7.4 in BS 6399-3:1988).	1.05
Forces due to temperature change.	1.2
Vertical crane loads.	1.6
Vertical crane loads acting together with horizontal crane loads.	1.4 ^a
Horizontal crane loads (surge, see 2.2.3, or crabbing, see 4.11.2).	1.6
Horizontal crane loads acting together with vertical crane loads.	1.4
Vertical crane loads acting together with imposed load.	1.4 ^a
Horizontal crane loads acting together with imposed load.	1.2
Imposed load acting together with vertical crane loads.	1.4
Imposed load acting together with horizontal crane loads.	1.2
Crane loads acting together with wind load.	1.2 ^a
Wind load acting together with crane loads.	1.2
^a Use $\gamma_f = 1.0$ for vertical crane loads that counteract the effects of other loads. A1 NOTE 1 The imposed loads are the imposed floor loads and the imposed roof loads. NOTE 2 The crane loads are the self-weight of the crane, the lifted load and the allowances for dynamic effects. A1	

2.4.1.3 Overhead travelling cranes

The γ_f factors given in Table 2 for vertical loads from overhead travelling cranes should be applied to the dynamic vertical wheel loads, i.e. the static vertical wheel loads increased by the appropriate allowance for dynamic effects, see 2.2.3.

Where a structure or member is subject to loads from two or more cranes, the crane loads should be taken as the maximum vertical and horizontal loads acting simultaneously where this is reasonably possible.

For overhead travelling cranes inside buildings, in the design of gantry girders and their supports the following principal combinations of loads should be taken into account:

- Crane combination 1: Dead load, imposed load and vertical crane loads;
- Crane combination 2: Dead load, imposed load and horizontal crane loads;
- Crane combination 3: Dead load, imposed load, vertical crane loads and horizontal crane loads.

Further load combinations should also be considered in the case of members that support overhead travelling cranes and are also subject to wind loads

2.4.1.4 Outdoor cranes

The wind loads on outdoor overhead travelling cranes should be obtained from:

- a) BS 2573-1, for cranes under working conditions;
- b) BS 6399-2, for cranes that are not in operation.

2.4.2 Stability limit states

2.4.2.1 General

Static equilibrium, resistance to horizontal forces and sway stiffness should be checked.

In checking the stability of a structure, or of any part of it, the loads should be increased by the relevant γ_f factors given in Table 2. The factored loads should be applied in the most unfavourable realistic combination for the part or effect under consideration.

2.4.2.2 Static equilibrium

The factored loads, considered separately and in combination, should not cause the structure, or any part of it (including the foundations), to slide, overturn or lift off its seating. The combination of dead, imposed and wind loads should be such as to have the most severe effect on the stability limit state under consideration, see 2.2.1.

Account should be taken of variations in dead load probable during construction or other temporary conditions.

2.4.2.3 Resistance to horizontal forces

To provide a practical level of robustness against the effects of incidental loading, all structures, including portions between expansion joints, should have adequate resistance to horizontal forces. In load combination 1 (see 2.4.1.2) the notional horizontal forces given in 2.4.2.4 should be applied. In load combinations 2 and 3 the horizontal component of the factored wind load should not be taken as less than 1.0 % of the factored dead load applied horizontally.

Resistance to horizontal forces should be provided using one or more of the following systems:

- triangulated bracing;
- moment-resisting joints;
- cantilever columns;
- shear walls;
- specially designed staircase enclosures, lift cores or similar construction.

Whatever system of resisting horizontal forces is used, reversal of load direction should be accommodated. The cladding, floors and roof should have adequate strength and be so secured to the structural framework as to transmit all horizontal forces to the points at which such resistance is provided.

Where resistance to horizontal forces is provided by construction other than the steel frame, the steelwork design should clearly indicate the need for such construction and state the forces acting on it, see 1.6.

As the specified loads from overhead travelling cranes already include significant horizontal loads, it is not necessary to include vertical crane loads when calculating the minimum wind load.

2.4.2.4 *Notional horizontal forces*

To allow for the effects of practical imperfections such as lack of verticality, all structures should be capable of resisting notional horizontal forces, taken as a minimum of 0.5 % of the factored vertical dead and imposed loads applied at the same level.

NOTE For certain structures, such as internal platform floors or spectator grandstands, larger minimum horizontal forces are given in the relevant design documentation.

The notional horizontal forces should be assumed to act in any one direction at a time and should be applied at each roof and floor level or their equivalent. They should be taken as acting simultaneously with the factored vertical dead and imposed loads (load combination 1, see 2.4.1.2).

As the specified loads from overhead travelling cranes already include significant horizontal loads, the vertical crane loads need not be included when calculating notional horizontal forces.

The notional horizontal forces applied in load combination 1 should not:

- a) be applied when considering overturning;
- b) be applied when considering pattern loads;
- c) be combined with applied horizontal loads;
- d) be combined with temperature effects;
- e) be taken to contribute to the net reactions A_1 of the structure as a whole on its A_1 foundations.

NOTE These conditions do not apply to the minimum wind load (1.0 % of dead load) in 2.4.2.3.

2.4.2.5 *Sway stiffness*

All structures (including portions between expansion joints) should have sufficient sway stiffness, so that the vertical loads acting with the lateral displacements of the structure do not induce excessive secondary forces or moments in the members or connections. Where such “second order” (“P-Δ”) effects are significant, they should be allowed for in the design of those parts of the structure that contribute to its resistance to horizontal forces, see 2.4.2.6.

Sway stiffness should be provided by the system of resisting horizontal forces, see 2.4.2.3. Whatever system is used, sufficient stiffness should be provided to limit sway deformation in any horizontal direction and also to limit twisting of the structure on plan.

Where moment resisting joints are used to provide sway stiffness, unless they provide full continuity of member stiffness, their flexibility should be taken into account in the analysis.

In the case of clad structures, the stiffening effect of masonry infill wall panels or diaphragms of profiled steel sheeting may be explicitly taken into account by using the method of partial sway bracing given in Annex E.

2.4.2.6 *“Non-sway” frames*

A structure or structural frame may be classed as “non-sway” if its sway deformation is sufficiently small for the resulting secondary forces and moments to be negligible. For clad structures, provided that the stiffening effect of masonry infill wall panels or diaphragms of profiled steel sheeting is not explicitly taken into account (see 2.4.2.5), this may be assumed to be satisfied if the sway mode elastic critical load factor λ_{cr} of the frame, under vertical load only, satisfies:

$$\lambda_{cr} \geq 10$$

In all other cases the structure or frame should be classed as “sway-sensitive”, see 2.4.2.7.

Except for single-storey frames with moment-resisting joints, or other frames in which sloping members have moment-resisting connections, λ_{cr} should be taken as the smallest value, considering every storey, determined from:

$$\lambda_{cr} = \frac{h}{200\delta}$$

where

- h is the storey height;
- δ is the notional horizontal deflection of the top of the storey relative to the bottom of the storey, due to $\overline{A1}$ horizontal forces equal to 0.5 % of the factored vertical dead, imposed and crane loads applied to the frame at each storey level $\overline{A1}$.

For single-storey frames with rigid moment-resisting joints, reference should be made to 5.5.

Other frames in which sloping members have moment-resisting connections may either be designed by obtaining λ_{cr} by second-order elastic analysis, or treated like portal frames, see 5.5.

2.4.2.7 “Sway-sensitive” frames

$\overline{A1}$ 2.4.2.7.1 General $\overline{A1}$

All structures that are not classed as “non-sway” (including those in which the stiffening effect of masonry infill wall panels or diaphragms of profiled steel sheeting is explicitly taken into account, see 2.4.2.5), should be classed as “sway-sensitive” $\overline{A1}$ and the secondary forces and moments should be allowed for.

NOTE Either elastic or plastic analysis may be used. $\overline{A1}$

$\overline{A1}$ 2.4.2.7.2 Elastic analysis

Provided that λ_{cr} is not less than 4.0, the secondary forces and moments should be allowed for by using one of the following methods:

a) *Effective length method*: This method applies to cases where the resistance to horizontal forces is provided by moment-resisting joints or cantilever columns. Sway mode in-plane effective lengths should be used for the columns, see 4.7.3 for simple structures or E.2 for continuous structures. The beams should be designed to remain elastic under the factored loads.

b) *Amplified sway method*: The sway effects (see 2.4.2.8) should be multiplied by the amplification factor k_{amp} determined from the following:

1) for clad structures, provided that the stiffening effect of masonry infill wall panels or diaphragms of profiled steel sheeting (see 2.4.2.5) is not explicitly taken into account:

$$k_{amp} = \frac{\lambda_{cr}}{1.15\lambda_{cr} - 1.5} \text{ but } k_{amp} \geq 1.0$$

2) for unclad frames, or for clad structures in which the stiffening effect of masonry infill wall panels or diaphragms of profiled steel sheeting (see 2.4.2.5) is explicitly taken into account:

$$k_{amp} = \frac{\lambda_{cr}}{\lambda_{cr} - 1}$$

c) *Analytical method*. A rigorous form of second order elastic analysis should be used.

If λ_{cr} is less than 4.0, method c) should always be used.

2.4.2.7.3 Plastic analysis

If plastic analysis is used, reference should be made to 5.5 for portal frames or 5.7 for multi-storey frames. The secondary forces and moments should be allowed for by using second order elastic-plastic analysis. Simple plastic theory should not be used for second order analysis. $\overline{A1}$

2.4.2.8 Sway effects

In the case of a symmetrical frame, with symmetrical vertical loads, the sway effects should be taken as comprising the forces and moments in the frame due to the horizontal loads.

In any other case, the forces and moments at the ends of each member may conservatively be treated as sway effects. Otherwise, the sway effects may be found by using one of the following alternatives.

- a) Deducting the non-sway effects.
 - 1) Analyse the frame under the actual restraint conditions.
 - 2) Add horizontal restraints at each floor or roof level to prevent sway, then analyse the frame again.
 - 3) Obtain the sway effects by deducting the second set of forces and moments from the first set.
- b) Direct calculation.
 - 1) Analyse the frame with horizontal restraints added at each floor or roof level to prevent sway.
 - 2) Reverse the directions of the horizontal reactions produced at the added horizontal restraints.
 - 3) Apply them as loads to the otherwise unloaded frame under the actual restraint conditions.
 - 4) Adopt the forces and moments from the second analysis as the sway effects.

2.4.2.9 Foundation design

The design of foundations should be in accordance with BS 8004 and should accommodate all the forces imposed on them. Attention should be given to the method of connecting the steel superstructure to the foundations and to the anchoring of holding-down bolts as recommended in 6.6.

Where it is necessary to quote the foundation reactions, it should be clearly stated whether the forces and moments result from factored or unfactored loads. Where they result from factored loads, the relevant γ_f factors for each load in each combination should be stated.

2.4.3 Fatigue

Fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to normal fluctuations in wind loading need not be considered. However, where aerodynamic instability can occur, account should be taken of wind induced oscillations.

Structural members that support heavy vibrating machinery or plant should be checked for fatigue resistance. In the design of crane supporting structures, only those members that support cranes of utilization classes U4 to U9 as defined in BS 2573 need be checked.

When designing for fatigue a γ_f factor of 1.0 should be used. Resistance to fatigue should be determined by reference to BS 7608.

Where fatigue is critical, all design details should be precisely defined and the required quality of workmanship should be clearly specified.

NOTE BS 5950-2 does not fully cover workmanship for cases where fatigue is critical, but reference can be made to ISO 10721-2.

2.4.4 Brittle fracture

Brittle fracture should be avoided by using a steel quality with adequate notch toughness, taking account of:

- the minimum service temperature;
- the thickness;
- the steel grade;
- the type of detail;
- the stress level;
- the strain level or strain rate.

In addition, the welding electrodes or other welding consumables should have a specified Charpy impact value equivalent to, or better than, that specified for the parent metal, see 6.8.5 and 6.9.1.

In the UK the minimum service temperature $T_{\min}$ in the steel should normally be taken as $-5\text{ }^{\circ}\text{C}$ for internal steelwork and $-15\text{ }^{\circ}\text{C}$ for external steelwork. For cold stores, locations exposed to exceptionally low temperatures or structures to be constructed in other countries, $T_{\min}$ should be taken as the minimum temperature expected to occur in the steel within the intended design life of the structure.

The steel quality selected for each component should be such that the thickness t of each element satisfies:

$$t \leq Kt_1$$

where

- K is a factor that depends on the type of detail, the general stress level, the stress concentration effects and the strain conditions, see Table 3;
- t_1 is the limiting thickness at the appropriate minimum service temperature $T_{\min}$ for a given steel grade and quality, when the factor $K = 1$, from Table 4 or Table 5.

In addition, the maximum thickness of the component should not exceed the maximum thickness t_2 at which the full Charpy impact value applies to the selected steel quality for that product type and steel grade, according to the relevant product standard, see Table 6.

For rolled sections t and t_1 should be related to the same element of the cross-section as the factor K , but t_2 should be related to the thickest element of the cross-section.

Alternatively, the value of t_1 may be determined from the following:

— if $T_{\min} \geq T_{27J} - 20\text{ °C}$:

$$t_1 = 50(1.2)^N \left[\frac{355}{Y_{\text{nom}}} \right]^{1.4}$$

— if $T_{27J} - 20\text{ °C} > T_{\min} \geq T_{27J} - 35\text{ °C}$:

$$t_1 = 110(1.8)^N \left[\frac{355}{Y_{\text{nom}}} \right]^{1.4}$$

— if $T_{\min} < T_{27J} - 35\text{ °C}$:

$$t_1 = 0$$

in which:

$$N = \left(\frac{T_{\min} - T_{27J}}{10} \right) \langle A_1 \rangle$$

where

$T_{\min}$ is the minimum service temperature (in °C) expected to occur in the steel within the intended design life of the part;

$\langle A_1 \rangle T_{27J}$ is the test temperature or equivalent test temperature (in °C) for a minimum Charpy impact value C_v of 27 J as specified in the relevant product standard, see Table 7; $\langle A_1 \rangle$

Y_{nom} is the nominal yield strength (in N/mm²) [the specified minimum yield strength for thickness ≤ 16 mm (or 12 mm for BS 7668), as in the steel grade designation].

Table 3 — Factor K for type of detail, stress level and strain conditions

Type of detail or location	$\langle A_1 \rangle$ Detail $\langle A_1 \rangle$ in tension due to factored loads		Components not subject to applied tension
	Stress $\geq 0.3Y_{\text{nom}}$	Stress $< 0.3Y_{\text{nom}}$	
Plain steel	2	3	4
Drilled holes or reamed holes	1.5	2	3
Flame cut edges	1	1.5	2
Punched holes (un-reamed)	1	1.5	2
Welded, generally	1	1.5	2
Welded across ends of cover plates	0.5	0.75	1
Welded connections to unstiffened flanges, see 6.7.5 $\langle A_1 \rangle$, and tubular nodal joints $\langle A_1 \rangle$	0.5	0.75	1

NOTE 1 Where parts are required to withstand significant plastic deformation at the minimum service temperature (such as crash barriers or crane stops) K should be halved.

NOTE 2 Baseplates attached to columns by nominal welds only, for the purposes of location in use and security in transit, should be classified as plain steel.

NOTE 3 Welded attachments not exceeding 150 mm in length should not be classified as cover plates.

$\langle A_1 \rangle$ NOTE 4 Where abrupt changes in cross-sections coincide with the detail, (other than those covered by descriptions above), e.g. service openings, notched cut-outs, etc., the general stress levels should take into account the additional stress concentration effect.

NOTE 5 The stress considered is the stress excluding residual stresses and stresses from structural imperfections.

Table 4 — Thickness t_1 for plates, flats and rolled sections^{ab}

A1

Product standard, steel grade and quality	Maximum thickness t_1 (mm) when $K = 1$ according to minimum service temperature				
	Normal temperatures		Lower temperatures		
	Internal	External			
	–5 °C	–15 °C	–25 °C	–35 °C	–45 °C
BS EN 10025-2:					
S 275 JR	36	20	—	—	—
S 275 J0	65	54	36	20	—
S 275 J2	94	78	65	54	36
S 355 JR	25	14	—	—	—
S 355 J0	46	38	25	14	—
S 355 J2	66	55	46	38	25
S 355 K2	79	66	55	46	38
S 450 J0	33	27	18	10	—
BS EN 10025-3:					
S 275 N	113	94	78	65	54
S 275 NL	162	135	113	94	78
S 355 N	79	66	55	46	38
S 355 NL	114	95	79	66	55
S 460 N	55	46	38	32	26
S 460 NL	79	66	55	46	38
BS EN 10025-4:					
S 275 M	113	94	78	65	54
S 275 ML	162	135	113	94	78
S 355 M	79	66	55	46	38
S 355 ML	114	95	79	66	55
S 460 M	55	46	38	32	26
S 460 ML	79	66	55	46	38
BS EN 10025-5:					
S 355 J0W or S 355 J0WP	46	38	25	14	—
S 355 J2W or S 355 J2WP	66	55	46	38	25
S 355 K2W	79	66	55	46	38
BS EN 10025-6:					
S 460 Q	46	38	32	26	18
S 460 QL	66	55	46	38	32
S 460 QL1	95	79	66	55	46
^a The values in this table do not apply if the thickness of the part exceeds the relevant limiting thickness for validity of the standard Charpy impact value for that product form, see Table 6. ^b The inclusion of a thickness in this table does not necessarily imply that steel of that thickness can be supplied to that grade in all product forms.					

A1

Table 5 — Thickness t_1 for structural hollow sections

A1

Product standard, steel grade and quality	Maximum thickness t_1 (mm) when $K = 1$ according to minimum service temperature				
	Normal temperatures		Lower temperatures		
	Internal	External			
	–5 °C	–15 °C	–25 °C	–35 °C	–45 °C
BS EN 10210:					
S 275 J0H	65	54	36	20	—
S 275 J2H	94	78	65	54	36
S 275 NH	113	94	78	65	54
S 275 NLH	162	135	113	94	78
S 355 J0H	46	38	25	14	—
S 355 J2H	66	55	46	38	25
S 355 K2H <i>or</i> S 355 NH	79	66	55	46	38
S 355 NLH	114	95	79	66	55
S 460 NH	55	46	38	32	26
S 460 NLH	79	66	55	46	38
BS EN 10219:					
S 275 J0H	65	54	36	20	—
S 275 J2H	94	78	65	54	36
S 275 MH <i>or</i> S 275 NH	113	94	78	65	54
S 275 MLH <i>or</i> S 275 NLH	162	135	113	94	78
S 355 J0H	46	38	25	14	—
S 355 J2H	66	55	46	38	25
S 355 K2H <i>or</i> S 355 MH <i>or</i> S 355 NH	79	66	55	46	38
S 355 MLH <i>or</i> S 355 NLH	114	95	79	66	55
S 460 MH <i>or</i> S 460 NH	55	46	38	32	26
S 460 MLH <i>or</i> S 460 NLH	79	66	55	46	38
BS 7668:					
S 345 J0WH <i>or</i> S 345 J0WPH	48	40	26	15	—
S 345 GWH	62	52	43	36	20

A1

Table 6 — Maximum thickness t_2^a (mm)

A1

Product standard	Steel grade	Steel quality	Sections	Plates and flats	Hollow sections
BS EN 10025-2	S 275 or S 355	JR or J0	100	250	—
		J2	100	400	—
		K2	100	150	—
	S 450	J0	100	—	—
BS EN 10025-3	S 275 or S 355	All	250	250	—
	S 460	All	200	200	—
BS EN 10025-4	S 275, S 355 or S 460	All	150	120	—
BS EN 10025-5	All	J0WP or J2WP	40	12	—
		J0W, J2W or K2W	150	150	—
BS EN 10025-6	S 460	All	—	150	—
BS EN 10210-1	All	J0, J2 or K2	—	—	120
		NH or NLH	—	—	65
BS EN 10219-1	All	All	—	—	40
BS 7668	All	J0WPH	—	—	12
		J0WH or GWH	—	—	40

^a Maximum thickness at which the full Charpy impact value specified in the product standard applies.

Table 7 — Charpy test temperature (or equivalent test temperature) T_{27J}

Steel quality	Product standard			
	BS EN 10025	BS EN 10210-1	BS EN 10219-1	BS 7668
JR	+20 °C	+20 °C	+20 °C	—
J0	0 °C	0 °C	0 °C	0 °C
J2	−20 °C	−20 °C	−20 °C	—
K2	−30 °C ^a	—	−30 °C ^a	—
M	−30 °C ^a	—	−30 °C ^a	—
ML	−50 °C	—	−50 °C	—
N	−30 °C ^a	−30 °C ^a	−30 °C ^a	—
NL	−50 °C	−50 °C	−50 °C	—
Q	−20 °C ^a	—	—	—
QL	−40 °C ^a	—	—	—
QL1	−60 °C ^a	—	—	—
G	—	—	—	−15 °C

^a Equivalent test temperature for 27 J, see product standard.

A1

2.4.5 Structural integrity

2.4.5.1 General

The design of all structures should follow the principles given in 2.1.1.1. In addition, to reduce the risk of localized damage spreading, buildings should satisfy the further recommendations given in 2.4.5.2, 2.4.5.3 and 2.4.5.4. For the purposes of 2.4.5.2, 2.4.5.3 and 2.4.5.4 it may be assumed that substantial permanent deformation of members and their connections is acceptable.

[A1] Where regulations require that certain buildings should be constructed so that in event of an accident the building will not suffer collapse to an extent disproportionate to the cause, steel framed buildings designed as recommended in this standard (including the recommendations of 2.1.1.1) may be assumed to meet this requirement provided that:

- a) buildings of Class 1 and Class 2A are designed to conform to 2.4.5.2;
- b) buildings of Class 2B are designed to conform to 2.4.5.2 and 2.4.5.3;
- c) buildings of Class 3 are designed to conform to 2.4.5.2 and 2.4.5.3 in addition to resisting the design conditions that can reasonably be foreseen as possible during the life of the buildings, identified by a systematic risk analysis of normal and abnormal hazards such that any collapse is not disproportionate to the cause.

where

- Class 1 includes houses not exceeding 4 storeys; agricultural buildings; buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height.
- Class 2A includes 5 storey single occupancy houses; hotels not exceeding 4 storeys; flats, apartments and other residential buildings not exceeding 4 storeys; offices not exceeding 4 storeys; industrial buildings not exceeding 3 storeys; retailing premises not exceeding 3 storeys of less than 2 000 m² floor area in each storey; single storey educational buildings; all buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2 000 m² floor area at each storey.
- Class 2B includes hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys; educational buildings greater than 1 storey but not exceeding 15 storeys; retailing premises greater than 3 storeys but not exceeding 15 storeys; hospitals not exceeding 3 storeys; offices greater than 4 storeys but not exceeding 15 storeys; all buildings to which members of the public are admitted which contain floor areas exceeding 2 000 m² but less than 5 000 m² at each storey; car parking not exceeding 6 storeys.
- Class 3 includes all buildings defined above as Class 2A and 2B that exceed the limits on area or number of storeys; grandstands accommodating more than 5 000 spectators; buildings containing hazardous substances or processes.

NOTE For steel beams supported by other materials, reference should be made to BS 5628 for masonry, BS 5268 for timber, BS 8110 for concrete and BS 5950-5 for cold-formed steel. **[A1]**

2.4.5.2 **[A1]** Minimum requirements **[A1]**

All buildings should be effectively tied together at each principal floor level. Each column should be effectively held in position by means of horizontal ties in two directions, approximately at right angles, at each principal floor level supported by that column. Horizontal ties should similarly be provided at roof level, except where the steelwork only supports cladding that weighs not more than 0.7 kN/m² and that carries only imposed roof loads and wind loads.

Continuous lines of ties should be arranged as close as practicable to the edges of the floor or roof and to each column line, see Figure 1. At re-entrant corners the tie members nearest to the edge should be anchored into the steel framework as indicated in Figure 1.

All horizontal ties and their end connections should be of a standard of robustness commensurate with the structure of which they form a part. The horizontal ties may be:

- steel members, including those also used for other purposes;
- steel bar reinforcement that is anchored to the steel frame and embedded in concrete;
- steel mesh reinforcement in a composite slab with profiled steel sheeting, see BS 5950-4, designed to act compositely with steel beams, see BS 5950-3.1, the profiled steel sheets being directly connected to the beams by the shear connectors.

All horizontal ties, and all other horizontal members, should be capable of resisting a factored tensile load, which should not be considered as additive to other loads, of not less than 75 kN.

Each portion of a building between expansion joints should be treated as a separate building.

A1 Where precast concrete or other heavy floor or roof units are used, the bearing details should conform to BS 8110. **A1**

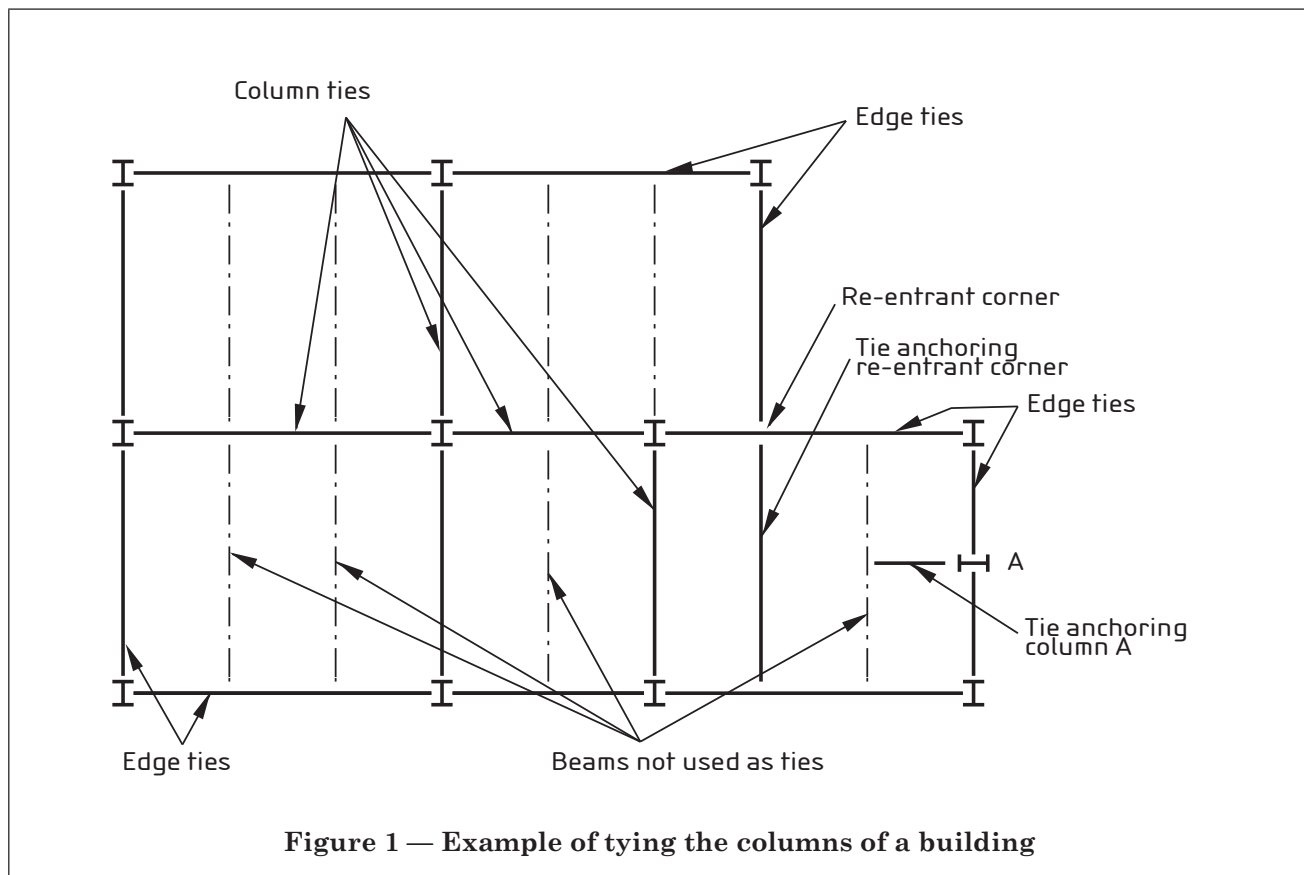


Figure 1 — Example of tying the columns of a building

2.4.5.3 **A1** Limiting the effects of accidental removal of supports **A1**

A1 Where regulations require certain buildings to be specially designed to limit the effect of accidental removal of supports, steel-framed buildings designed as recommended in this standard (including the recommendations of 2.1.1.1 and 2.4.5.2) may be assumed to meet this requirement provided that the following five conditions a) to e) are met. **A1**

- a) *General tying.* Horizontal ties generally similar to those described in 2.4.5.2 should be arranged in continuous lines wherever practicable, distributed throughout each floor and roof level in two directions approximately at right angles, see Figure 2.

Steel members acting as horizontal ties, and their end connections, should be capable of resisting the following factored tensile loads, which need not be considered as additive to other loads:

- for internal ties: $\langle A_t \rangle 0.5(1.4g_k + 1.6q_k)s_tLn \langle A_t \rangle$ but not less than 75 kN;
- for edge ties: $\langle A_t \rangle 0.25(1.4g_k + 1.6q_k)s_tLn \langle A_t \rangle$ but not less than 75 kN.

where

g_k is the specified dead load per unit area of the floor or roof;

L is the span;

q_k is the specified imposed floor or roof load per unit area;

s_t is the mean transverse spacing of the ties adjacent to that being checked.

$\langle A_t \rangle n$ is a factor related to the number of storeys in the structure as follows:

Number of storeys:	Value of factor, n :
5 or more	1.0
4	0.75
3	0.5
2	0.25
1	0

$\langle A_t \rangle$

This may be assumed to be satisfied if, in the absence of other loading, the member and its end connections are capable of resisting a tensile force equal to its end reaction under factored loads $\langle A_t \rangle$ multiplied by $n \langle A_t \rangle$, or the larger end reaction $\langle A_t \rangle$ multiplied by $n \langle A_t \rangle$ if they are unequal, but not less than 75 kN.

Horizontal ties that consist of steel reinforcement should be designed as recommended in BS 8110.

b) *Tying of edge columns.* The horizontal ties anchoring the columns nearest to the edges of a floor or roof should be capable of resisting a factored tensile load, acting perpendicular to the edge, equal to the greater of the load specified in a) or 1 % of the maximum factored vertical dead and imposed load in the column adjacent to that level.

c) *Continuity of columns.* Unless the steel frame is fully continuous in at least one direction, all columns should be carried through at each beam-to-column connection. All column splices should be capable of resisting a tensile force equal to the largest $\langle A_t \rangle$ total factored vertical dead and imposed load $\langle A_t \rangle$ applied to the column at a single floor level located between that column splice and the next column splice down.

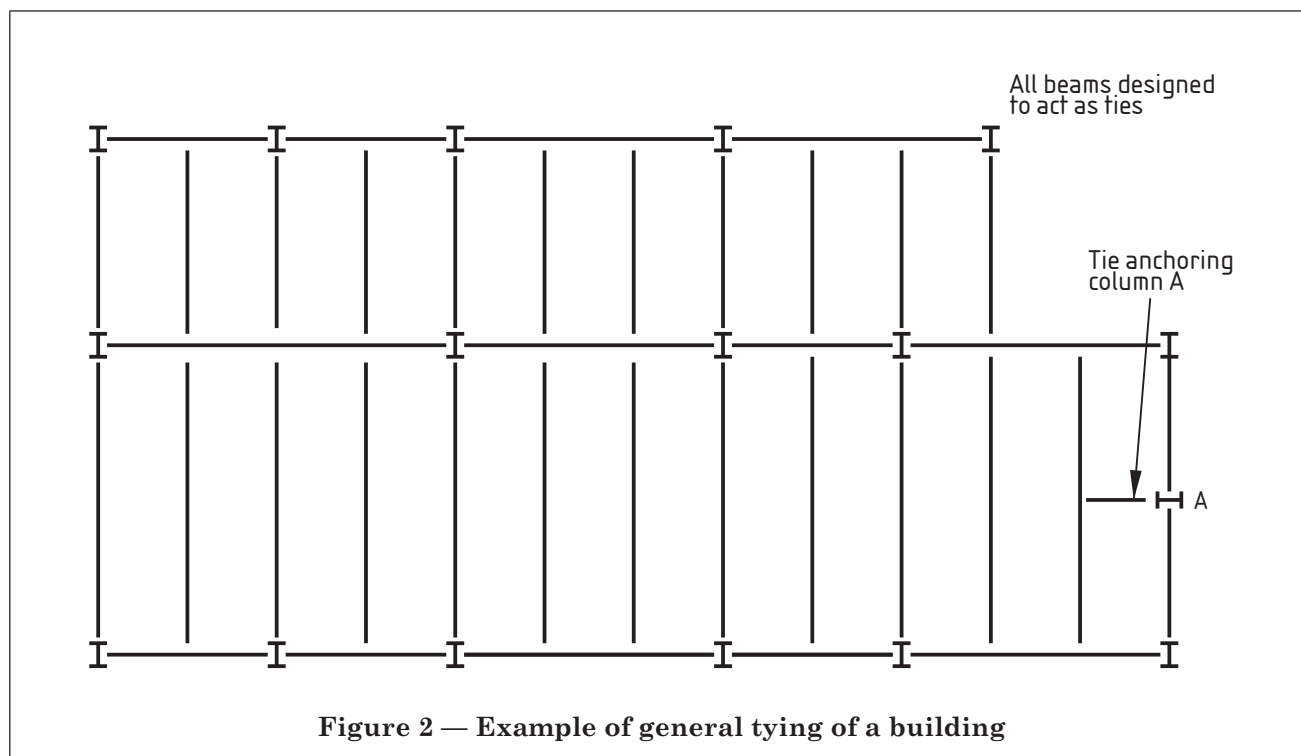
d) *Resistance to horizontal forces.* Braced bays or other systems for resisting horizontal forces as recommended in 2.4.2.3 should be distributed throughout the building such that, in each of two directions approximately at right angles, no substantial portion of the building is connected at only one point to a system for resisting horizontal forces.

e) *Heavy floor units.* Where precast concrete or other heavy floor $\langle A_t \rangle$, stair $\langle A_t \rangle$ or roof units are used they should be effectively anchored in the direction of their span, either to each other over a support, or directly to their supports as recommended in BS 8110.

If any of the first three conditions a) to c) are not met, the building should be checked, in each storey in turn, to ensure that disproportionate collapse would not be precipitated by the notional removal, one at a time, of each column $\langle A_t \rangle$ and each beam supporting one or more columns $\langle A_t \rangle$. If condition d) is not met, a check should be made in each storey in turn to ensure that disproportionate collapse would not be precipitated by the notional removal, one at a time, of each element of the systems providing resistance to horizontal forces.

The portion of the building at risk of collapse should not exceed 15 % of the floor or roof area or 70 m² (whichever is less) at the relevant level and at one immediately adjoining floor or roof level, either above or below it. If the notional removal of a column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area, that column or element should be designed as a key element, as recommended in 2.4.5.4.

In these checks for notional removal of members, only a third of the ordinary wind load and a third of the ordinary imposed load need be allowed for, together with the dead load, except that in the case of buildings used predominantly for storage, or where the imposed load is of a permanent nature, the full imposed load should be used. A partial factor γ_f of 1.05 should be applied, except that when considering overturning the dead load supplying the restoring moment should be multiplied by a partial factor γ_f of 0.9.



2.4.5.4 Key elements

In a multi-storey building that is required by regulations to be designed to avoid disproportionate collapse, a member that is recommended in 2.4.5.3 to be designed as a key element should be designed for the accidental loading specified in BS 6399-1.

Any other steel member or other structural component that provides lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same accidental loading.

The accidental loading should be applied to the member from all horizontal and vertical directions, in one direction at a time, together with the reactions from other building components attached to the member that are subject to the same accidental loading, but limited to the maximum reactions that could reasonably be transmitted, considering the breaking resistances of such components and their connections.

In this check the effects of ordinary loads should also be considered, to the same extent and with the same partial factor γ_f as recommended in 2.4.5.3.

2.5 Serviceability limit states

2.5.1 Serviceability loads

Generally the serviceability loads should be taken as the unfactored specified values. However, exceptional snow load (due to local drifting on roofs, see 7.4 in BS 6399-3:1988) should not be included in the imposed load when checking serviceability.

In the case of combined imposed load and wind load, only 80 % of the full specified values need be considered when checking serviceability. In the case of combined horizontal crane loads and wind load, only the greater effect need be considered when checking serviceability.

2.5.2 Deflection

The deflections of a building or part under serviceability loads should not impair the strength or efficiency of the structure or its components, nor cause damage to the finishings.

When checking for deflections the most adverse realistic combination and arrangement of serviceability loads should be assumed, and the structure may be assumed to behave elastically.

Table 8 gives suggested limits for the calculated deflections of certain structural members. Circumstances may arise where greater or lesser values would be more appropriate. Other members may also need deflection limits.

On low pitched and flat roofs the possibility of ponding should be investigated.

For deflection limits for runway beams reference should be made to BS 2853.

2.5.3 Vibration and oscillation

Vibration and oscillation of building structures should be limited to avoid discomfort to users and damage to contents. Reference to specialist literature should be made as appropriate.

NOTE Guidance on floor vibration is given in reference [3], see Bibliography.

2.5.4 Durability

In order to ensure the durability of the structure under conditions relevant both to its intended use and to its intended life, the following factors should be taken into account in design:

- a) the environment of the structure and the degree of exposure;
- b) the shape of the members and the structural detailing;
- c) the protective measures, if any;
- d) whether inspection and maintenance are possible.

As an alternative to the use of protective coatings, weather resistant steels to BS EN 10155 may be used.

Table 8 — Suggested limits for calculated deflections

<i>a) Vertical deflection of beams due to imposed load</i>	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
Other beams (except purlins and sheeting rails)	Span/200
Purlins and sheeting rails	See 4.12.2
<i>b) Horizontal deflection of columns due to imposed load and wind load</i>	
Tops of columns in single-storey buildings, except portal frames	Height/300
Columns in portal frame buildings, not supporting crane runways	To suit cladding
Columns supporting crane runways	To suit crane runway
In each storey of a building with more than one storey	Height of that storey/300
<i>c) Crane girders</i>	
Vertical deflection due to static vertical wheel loads from overhead travelling cranes	Span/600
Horizontal deflection (calculated on the top flange properties alone) due to horizontal crane loads	Span/500

Section 3. Properties of materials and section properties

3.1 Structural steel

3.1.1 Design strength

This standard covers the design of structures fabricated from structural steels conforming to the grades and product standards specified in BS 5950-2. If other steels are used, due allowance should be made for variations in properties, including ductility and weldability.

The design strength p_y should be taken as $1.0Y_s$ but not greater than $U_s/1.2$ where Y_s and U_s are respectively the minimum yield strength R_{eH} and the minimum tensile strength R_m specified in the relevant product standard. For the more commonly used grades and thicknesses of steel from the product standards specified in BS 5950-2 the value of p_y may be obtained from Table 9. Alternatively, the values of R_{eH} and R_m may be obtained from the relevant product standard.

NOTE Additional requirements apply where plastic analysis is used, see 5.2.3.

Table 9 — Design strength p_y

Steel grade	Thickness ^a less than or equal to mm	Design strength p_y N/mm ²
S 275	16	275
	40	265
	63	255
	80	245
	100	235
	150	225
S 355	16	355
	40	345
	63	335
	80	325
	100	315
	150	295
S 460	16	460
	40	440
	63	430
	80	410
	100	400

^a For rolled sections, use the specified thickness of the thickest element of the cross-section.

3.1.2 Notch toughness

The notch toughness of the steel, as quantified by the Charpy impact properties, should conform to that for the appropriate quality of steel for avoiding brittle fracture, see 2.4.4.

3.1.3 Other properties

For the elastic properties of steel, the following values should be used.

- Modulus of elasticity: $E = 205\,000 \text{ N/mm}^2$
- Shear modulus: $G = E/[2(1 + \nu)]$
- Poisson's ratio: $\nu = 0.30$
- Coefficient of linear thermal expansion
(in the ambient temperature range): $\alpha = 12 \times 10^{-6} \text{ per } ^\circ\text{C}$

3.2 Bolts and welds

3.2.1 Bolts, nuts and washers

Assemblies of bolts, nuts and washers should correspond to one of the matching combinations specified in BS 5950-2. Holding-down bolt assemblies should conform to BS 7419.

3.2.2 Friction grip fasteners

Friction grip fasteners should generally be preloaded HSBG bolts, with associated nuts and washers, conforming to BS 4395-1 or BS 4395-2. Direct tension indicators conforming to BS 7644 may be used.

Other types of friction grip fasteners may also be used provided that they can be reliably tightened to at least the minimum shank tensions specified in BS 4604.

3.2.3 Welding consumables

All welding consumables, including covered electrodes, wires, filler rods, flux and shielding gases, should conform to the relevant standard specified in BS 5950-2.

The yield strength Y_e , tensile strength U_e and minimum elongation of a weld should be taken as equal to respectively the minimum yield strength R_{eL} or $R_{p0.2}$ (depending on the relevant product standard), tensile strength R_m and minimum percentage elongation on a five diameter gauge length according to the appropriate product standard, all as listed for standard classes 35, 42 and 50 in Table 10.

Table 10 — Strength and elongation of welds

Class	Yield strength Y_e (N/mm ²)	Tensile strength U_e (N/mm ²)	Minimum elongation (%)
35	355	440	22
42	420	500	20
50	500	560	18

3.3 Steel castings and forgings

Steel castings and forgings may be used for components in bearings, junctions and other similar parts. Castings should conform to BS 3100 and forgings should conform to BS EN 10250-2. Unless better information is available, design strengths corresponding to structural steel grade S 275 may be adopted.

NOTE Guidance on steel castings is given in reference [4], see Bibliography.

3.4 Section properties

3.4.1 Gross cross-section

Gross cross-section properties should be determined from the specified shape and nominal dimensions of the member or element. Holes for bolts should not be deducted, but due allowance should be made for larger openings. Material used solely in splices or as battens should not be included.

3.4.2 Net area

The net area of a cross-section or an element of a cross-section should be taken as its gross area, less the deductions for bolt holes given in 3.4.4.

3.4.3 Effective net area

The effective net area a_e of each element of a cross-section with bolt holes should be determined from:

$$a_e = K_e a_n \quad \text{but} \quad a_e \leq a_g$$

in which the effective net area coefficient K_e is given by:

- for grade S 275: $K_e = 1.2$
- for grade S 355: $K_e = 1.1$
- for grade S 460: $K_e = 1.0$
- for other steel grades: $K_e = (U_s/1.2)/p_y$

where

- a_g is the gross area of the element;
- a_n is the net area of the element;
- p_y is the design strength;
- U_s is the specified minimum tensile strength.

3.4.4 Deductions for bolt holes

3.4.4.1 Hole area

In deducting for bolt holes (including countersunk holes), the sectional area of the hole in the plane of its own axis should be deducted, not that of the bolt.

3.4.4.2 Holes not staggered

Provided that the bolt holes are not staggered, the area to be deducted should be the sum of the sectional areas of the bolt holes in a cross-section perpendicular to the member axis or direction of direct stress.

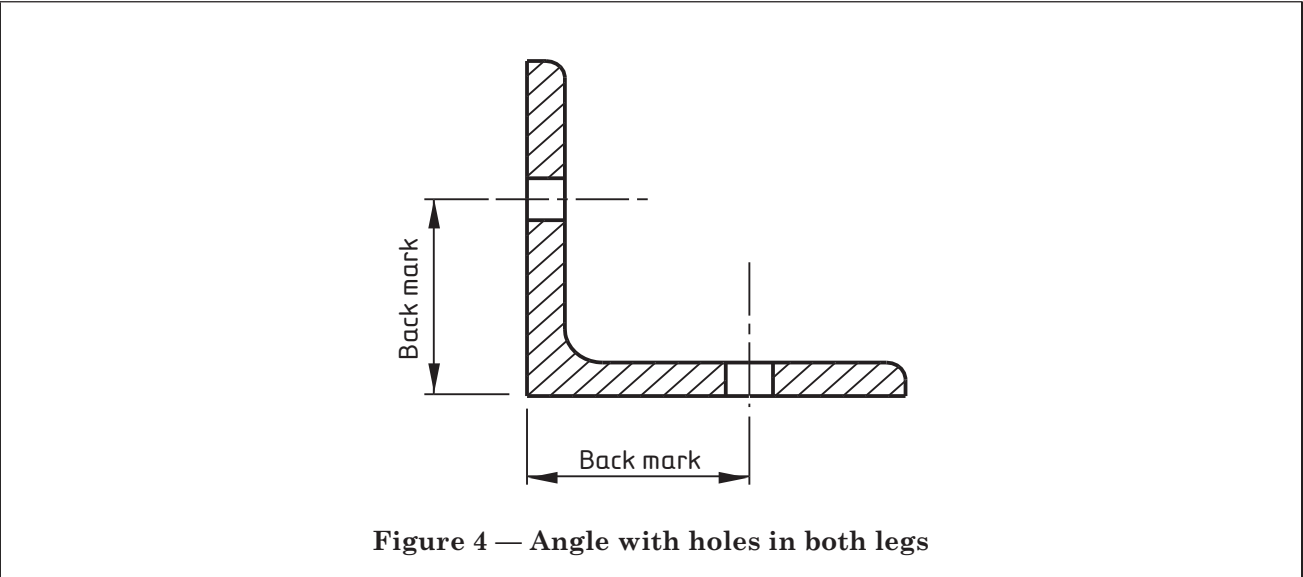
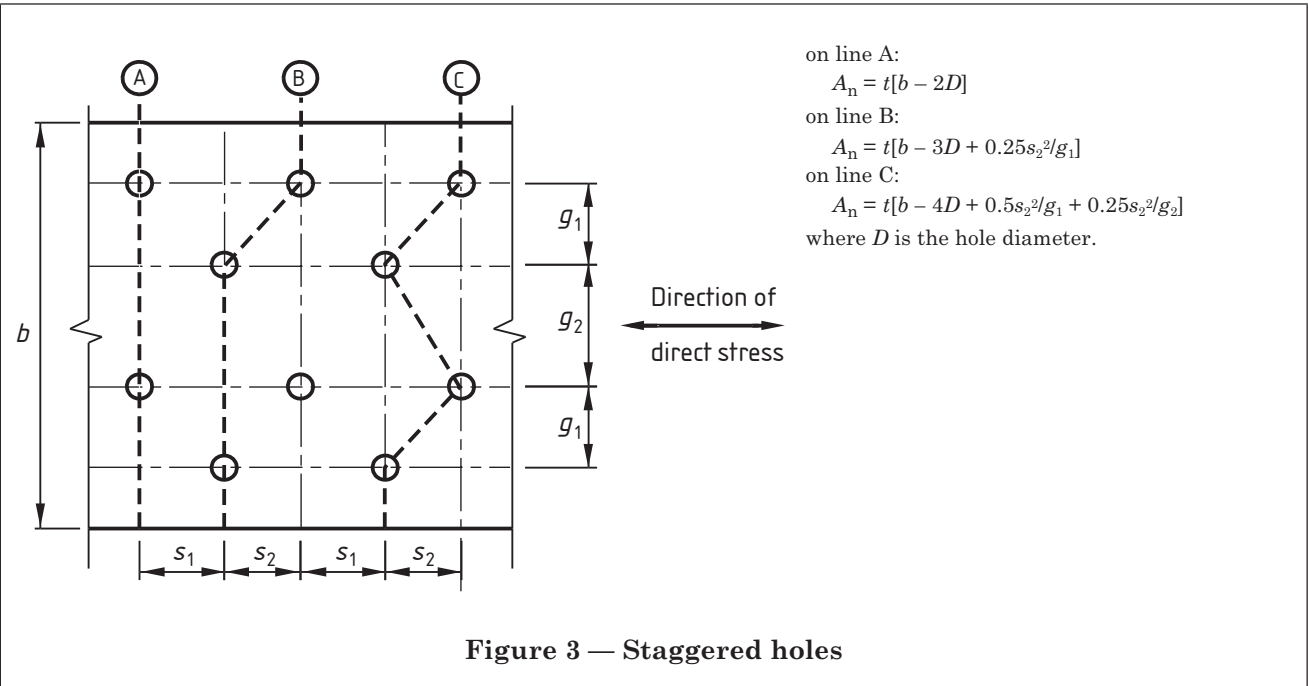
3.4.4.3 Staggered holes

Where the bolt holes are staggered, the area to be deducted should be the greater of:

- a) the deduction for non-staggered holes given in 3.4.4.2;
- b) the sum of the sectional areas of a chain of holes lying on any diagonal or zig-zag line extending progressively across the member or element, see Figure 3, less an allowance of $0.25s^2t/g$ for each gauge space g that it traverses diagonally, where:

- g is the gauge spacing perpendicular to the member axis or direction of direct stress, between the centres of two consecutive holes in the chain, see Figure 3;
- s is the staggered pitch, i.e. the spacing parallel to the member axis or direction of direct stress, between the centres of the same two holes, see Figure 3;
- t is the thickness of the holed material.

For sections such as angles with holes in both legs, the gauge spacing g should be taken as the sum of the back marks to each hole, less the leg thickness, see Figure 4.



3.5 Classification of cross-sections

3.5.1 General

Cross-sections should be classified to determine whether local buckling influences their capacity, without calculating their local buckling resistance.

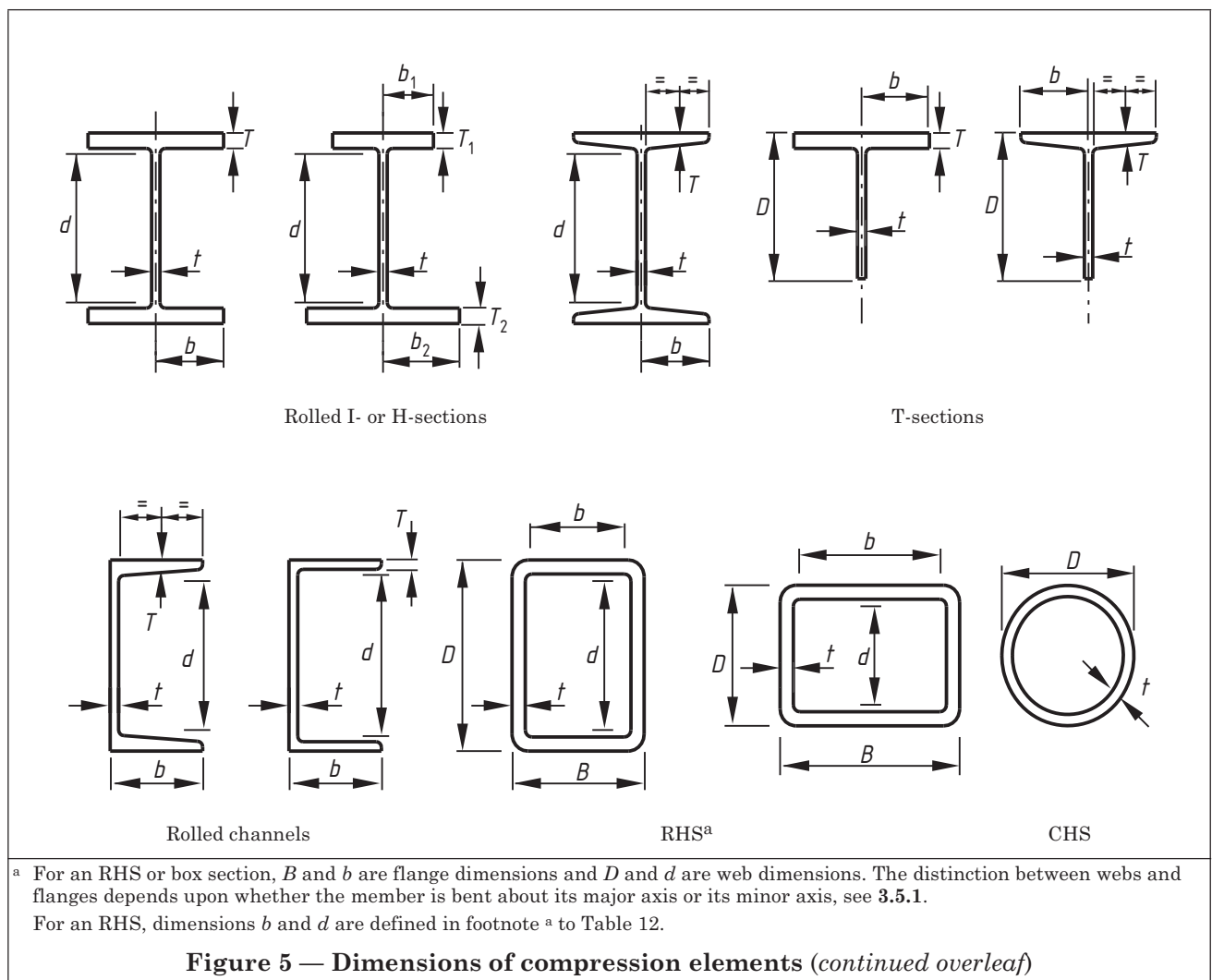
The classification of each element of a cross-section subject to compression (due to a bending moment or an axial force) should be based on its width-to-thickness ratio. The dimensions of these compression elements should be taken as shown in Figure 5. The elements of a cross-section are generally of constant thickness; for elements that taper in thickness the thickness specified in the relevant standard should be used.

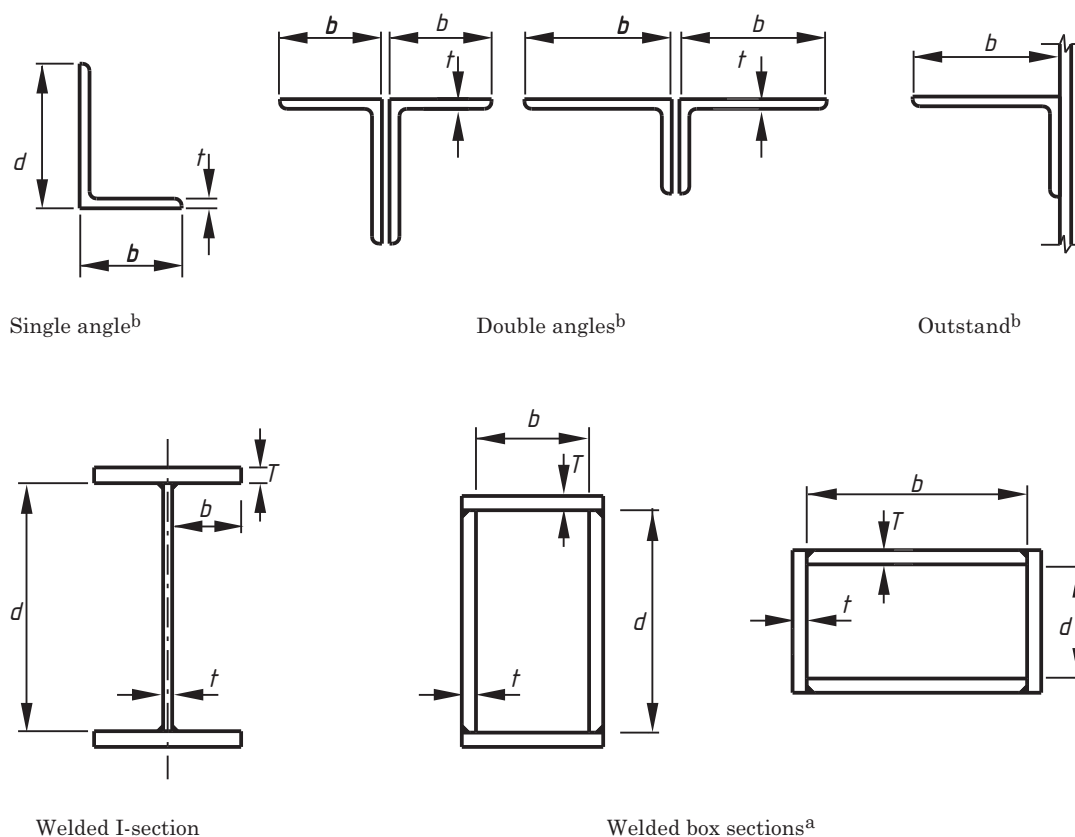
A distinction should be made between the following types of element:

- a) *outstand elements* attached to an adjacent element at one edge only, the other edge being free;
- b) *internal elements* attached to other elements on both longitudinal edges and including:
 - *webs* comprising internal elements perpendicular to the axis of bending;
 - *flanges* comprising internal elements parallel to the axis of bending.

All compression elements should be classified in accordance with 3.5.2. Generally, the complete cross-section should be classified according to the highest (least favourable) class of its compression elements. Alternatively, a cross-section may be classified with its compression flange and its web in different classes.

Circular hollow sections should be classified separately for axial compression and for bending.





^a For an RHS or box section, B and b are flange dimensions and D and d are web dimensions. The distinction between webs and flanges depends upon whether the member is bent about its major axis or its minor axis, see 3.5.1.

For an RHS, dimensions b and d are defined in footnote ^a to Table 12.

^b For an angle, b is the width of the outstand leg and d is the width of the connected leg.

Figure 5 — Dimensions of compression elements (continued)

3.5.2 Classification

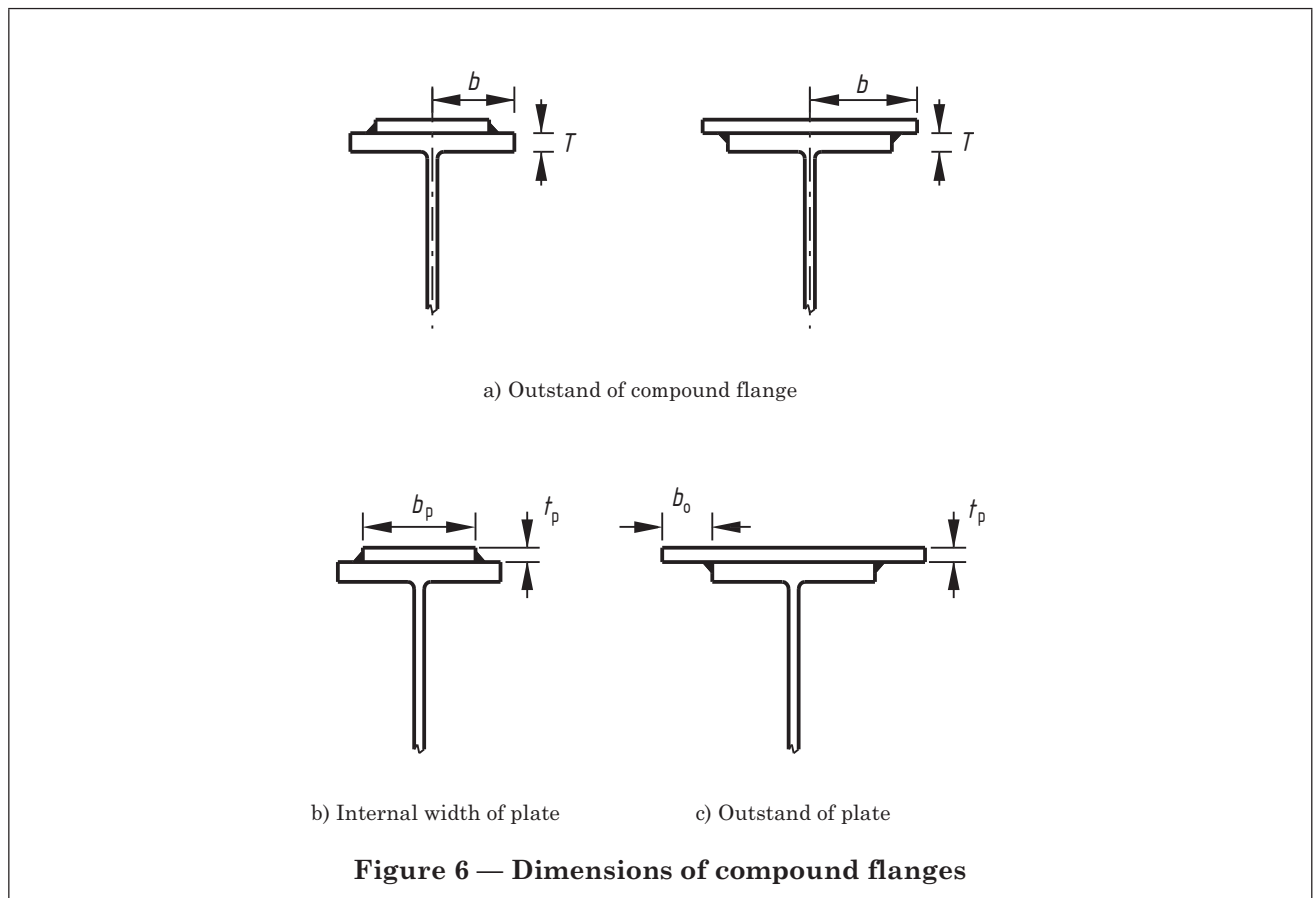
The following classification should be applied.

- Class 1 plastic: *Cross-sections with plastic hinge rotation capacity.* Elements subject to compression that meet the limits for class 1 given in Table 11 or Table 12 should be classified as class 1 plastic.
- Class 2 compact: *Cross-sections with plastic moment capacity.* Elements subject to compression that meet the limits for class 2 given in Table 11 or Table 12 should be classified as class 2 compact.
- Class 3 semi-compact: *Cross-sections in which the stress at the extreme compression fibre can reach the design strength, but the plastic moment capacity cannot be developed.* Elements subject to compression that meet the limits for class 3 given in Table 11 or Table 12 should be classified as class 3 semi-compact.
- Class 4 slender: *Cross-sections in which it is necessary to make explicit allowance for the effects of local buckling.* Elements subject to compression that do not meet the limits for class 3 semi-compact given in Table 11 or Table 12 should be classified as class 4 slender.

3.5.3 Flanges of compound I- or H-sections

The classification of the compression flange of a compound section, fabricated by welding a flange plate to a rolled $\overline{A_1}$ or welded $\overline{A_2}$ I- or H-section should take account of the width-to-thickness ratios shown in Figure 6 as follows:

- the ratio of the outstand b of the compound flange, see Figure 6a), to the thickness T of the original flange should be classified under “*outstand element of compression flange–rolled section*”, see Table 11;
- the ratio of the internal width b_p of the plate between the lines of welds or bolts connecting it to the original flange, see Figure 6b), to the thickness t_p of the plate should be classified under “*internal element of compression flange*”, see Table 11;
- the ratio of the outstand b_o of the plate beyond the lines of welds or bolts connecting it to the original flange, see Figure 6c), to the thickness t_p of the plate should be classified under “*outstand element of compression flange–welded section*”, see Table 11.



3.5.4 Longitudinally stiffened elements

For the design of compression elements with longitudinal stiffeners, reference should be made to BS 5400-3.

Table 11 — Limiting width-to-thickness ratios for sections other than CHS and RHS

Compression element		Ratio ^a	Limiting value ^b		
			Class 1 plastic	Class 2 compact	Class 3 semi-compact
Outstand element of compression flange	Rolled section	b/T	9ε	10ε	15ε
	Welded section	b/T	8ε	9ε	13ε
Internal element of compression flange	Compression due to bending	b/T	28ε	32ε	40ε
	Axial compression	b/T	Not applicable		
Web of an I-, H- or box section ^c	Neutral axis at mid-depth	d/t	80ε	100ε	120ε
	Generally ^d	If r_1 is negative:	d/t	$\frac{100\varepsilon}{1 + r_1}$	$\frac{120\varepsilon}{1 + 2r_2}$ but $\geq 40\varepsilon$
		If r_1 is positive:	d/t	$\frac{80\varepsilon}{1 + r_1}$ but $\geq 40\varepsilon$	
	Axial compression ^d		d/t	Not applicable	
Web of a channel		d/t	40ε	40ε	40ε
Angle, compression due to bending (Both criteria should be satisfied)		b/t	9ε	10ε	15ε
		d/t	9ε	10ε	15ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)		b/t	Not applicable		15ε
		d/t			15ε
		$(b + d)/t$			24ε
Outstand leg of an angle in contact back-to-back in a double angle member		b/t	9ε	10ε	15ε
Outstand leg of an angle with its back in continuous contact with another component					
Stem of a T-section, rolled or cut from a rolled I- or H-section		D/t	8ε	9ε	18ε

^a Dimensions b , D , d , T and t are defined in Figure 5. For a box section b and T are flange dimensions and d and t are web dimensions, where the distinction between webs and flanges depends upon whether the box section is bent about its major axis or its minor axis, see 3.5.1.

^b The parameter $\varepsilon = (275/p_y)^{0.5}$.

^c For the web of a hybrid section ε should be based on the design strength p_{yf} of the flanges.

^d The stress ratios r_1 and r_2 are defined in 3.5.5.

Table 12 — Limiting width-to-thickness ratios for CHS and RHS

Compression element			Ratio ^a	Limiting value ^b		
				Class 1 plastic	Class 2 compact	Class 3 semi-compact
CHS	Compression due to bending		D/t	$40\epsilon^2$	$50\epsilon^2$	$140\epsilon^2$
	Axial compression		D/t	Not applicable		$80\epsilon^2$
HF RHS	Flange	Compression due to bending	b/t	28ϵ but $\leq 80\epsilon - d/t$	32ϵ but $\leq 62\epsilon - 0.5d/t$	40ϵ
		Axial compression	b/t	Not applicable		
	Web	Neutral axis at mid-depth	d/t	64ϵ	80ϵ	120ϵ
		Generally ^{cd}	d/t	$\frac{64\epsilon}{1 + 0.6r_1}$ but $\geq 40\epsilon$	$\frac{80\epsilon}{1 + r_1}$ but $\geq 40\epsilon$	$\frac{120\epsilon}{1 + 2r_2}$ but $\geq 40\epsilon$
		Axial compression ^d	d/t	Not applicable		
CF RHS	Flange	Compression due to bending	b/t	26ϵ but $\leq 72\epsilon - d/t$	28ϵ but $\leq 54\epsilon - 0.5d/t$	35ϵ
		Axial compression ^d	b/t	Not applicable		
	Web	Neutral axis at mid-depth	d/t	56ϵ	70ϵ	105ϵ
		Generally ^{cd}	d/t	$\frac{56\epsilon}{1 + 0.6r_1}$ but $\geq 35\epsilon$	$\frac{70\epsilon}{1 + r_1}$ but $\geq 35\epsilon$	$\frac{105\epsilon}{1 + 2r_2}$ but $\geq 35\epsilon$
		Axial compression ^d	d/t	Not applicable		

Abbreviations

CF Cold formed;

CHS Circular hollow section — including welded tube;

HF Hot finished;

RHS Rectangular hollow section — including square hollow section.

^a For an RHS, the dimensions b and d should be taken as follows:— for HF RHS to BS EN 10210: $b = B - 3t$; $d = D - 3t$ — for CF RHS to BS EN 10219: $b = B - 5t$; $d = D - 5t$ and B , D and t are defined in Figure 5. For an RHS subject to bending B and b are always flange dimensions and D and d are always web dimensions, but the definition of which sides of the RHS are webs and which are flanges changes according to the axis of bending, see 3.5.1.^b The parameter $\epsilon = (275/p_y)^{0.5}$.^c For RHS subject to moments about both axes see H.3.^d The stress ratios r_1 and r_2 are defined in 3.5.5.

3.5.5 Stress ratios for classification

The stress ratios r_1 and r_2 used in Table 11 and Table 12 should be determined from the following:

a) for I- or H-sections with equal flanges:

$$r_1 = \frac{F_c}{dtp_{yw}} \text{ but } -1 < r_1 \leq 1$$

$$r_2 = \frac{F_c}{A_g p_{yw}}$$

b) for I- or H-sections with unequal flanges:

$$r_1 = \frac{F_c}{dtp_{yw}} + \frac{(B_t T_t - B_c T_c) p_{yf}}{dtp_{yw}} \text{ but } -1 < r_1 \leq 1$$

$$r_2 = \frac{f_1 + f_2}{2p_{yw}}$$

c) for RHS or welded box sections with equal flanges:

$$r_1 = \frac{F_c}{2dtp_{yw}} \text{ but } -1 < r_1 \leq 1$$

$$r_2 = \frac{F_c}{A_g p_{yw}}$$

where

A_g is the gross cross-sectional area;

B_c is the width of the compression flange;

B_t is the width of the tension flange;

d is the web depth;

F_c is the axial compression (negative for tension);

f_1 is the maximum compressive stress in the web, see Figure 7;

f_2 is the minimum compressive stress in the web (negative for tension), see Figure 7;

p_{yf} is the design strength of the flanges;

p_{yw} is the design strength of the web (but $p_{yw} \leq p_{yf}$);

T_c is the thickness of the compression flange;

T_t is the thickness of the tension flange;

t is the web thickness.

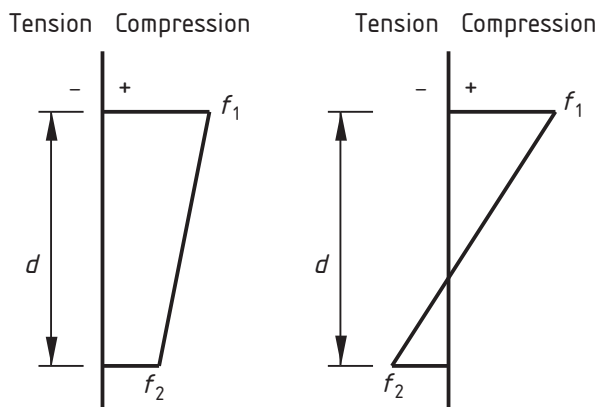


Figure 7 — Stress ratio for a semi-compact web

3.5.6 Effective plastic modulus

3.5.6.1 General

Class 3 semi-compact sections subject to bending should be designed using either the section modulus Z or the effective plastic modulus S_{eff} . For I- or H-sections with equal flanges, RHS and CHS, the effective plastic modulus should be determined from 3.5.6.2, 3.5.6.3 or 3.5.6.4 respectively. For I- or H-sections with unequal flanges subject to bending in the plane of the web, reference should be made to H.3. For other cross-sections S_{eff} should be taken as equal to the section modulus Z .

3.5.6.2 I- or H-sections with equal flanges

For class 3 semi-compact I- or H-sections with equal flanges, the effective plastic moduli $S_{x,\text{eff}}$ and $S_{y,\text{eff}}$ about the major and minor axes may be obtained from:

$$S_{x,\text{eff}} = Z_x + (S_x - Z_x) \left[\frac{\left(\frac{\beta_{3w}}{d/t}\right)^2 - 1}{\left(\frac{\beta_{3w}}{\beta_{2w}}\right)^2 - 1} \right] \quad \text{but} \quad S_{x,\text{eff}} \leq Z_x + (S_x - Z_x) \left[\frac{\frac{\beta_{3f}}{b/T} - 1}{\frac{\beta_{3f}}{\beta_{2f}} - 1} \right]$$

$$S_{y,\text{eff}} = Z_y + (S_y - Z_y) \left[\frac{\frac{\beta_{3f}}{b/T} - 1}{\frac{\beta_{3f}}{\beta_{2f}} - 1} \right] \quad \text{[A1]} \quad \text{but} \quad S_{y,\text{eff}} \leq S_y \quad \text{[A1]}$$

where

- b is the flange outstand, see Figure 5;
- d is the web depth;
- S_x is the plastic modulus about the major axis;
- S_y is the plastic modulus about the minor axis;
- T is the flange thickness;
- t is the web thickness;
- Z_x is the section modulus about the major axis;

- Z_y is the section modulus about the minor axis;
 β_{2f} is the limiting value of b/T from Table 11 for a class 2 compact flange;
 β_{2w} is the limiting value of d/t from Table 11 for a class 2 compact web;
 β_{3f} is the limiting value of b/T from Table 11 for a class 3 semi-compact flange;
 β_{3w} is the limiting value of d/t from Table 11 for a class 3 semi-compact web.

3.5.6.3 Rectangular hollow sections

For class 3 semi-compact RHS the effective plastic moduli $S_{x,eff}$ and $S_{y,eff}$ for major and minor axis bending may both be obtained by considering bending about the respective axis, using the following:

$$S_{eff} = Z + (S - Z) \left[\frac{\frac{\beta_{3w}}{d/t} - 1}{\frac{\beta_{3w}}{\beta_{2w}} - 1} \right] \quad \text{but} \quad S_{eff} \leq Z + (S - Z) \left[\frac{\frac{\beta_{3f}}{b/t} - 1}{\frac{\beta_{3f}}{\beta_{2f}} - 1} \right]$$

where

- β_{2f} is the limiting value of b/t from Table 12 for a class 2 compact flange;
 β_{2w} is the limiting value of d/t from Table 12 for a class 2 compact web;
 β_{3f} is the limiting value of b/t from Table 12 for a class 3 semi-compact flange;
 β_{3w} is the limiting value of d/t from Table 12 for a class 3 semi-compact web;

and the dimensions b , d and t of an RHS are as defined in Table 12.

NOTE For an RHS subject to bending B and b are always flange dimensions and D and d are always web dimensions, but the definition of which sides of the RHS are webs and which are flanges changes according to the axis of bending, see 3.5.1.

3.5.6.4 Circular hollow sections

For class 3 semi-compact CHS of diameter D and thickness t the effective plastic modulus S_{eff} should be obtained from:

$$S_{eff} = Z + 1.485 \left[\left[\left(\frac{140}{D/t} \right) \left(\frac{275}{p_y} \right) \right]^{0.5} - 1 \right] (S - Z)$$

3.6 Slender cross-sections

3.6.1 Effective section properties

The local buckling resistance of class 4 slender cross-sections may be allowed for in design by adopting effective section properties. Due allowance should be made for the possible effects of any shift of the centroid of the effective cross-section compared to that of the gross cross-section, see 3.6.3.

Generally the methods given in 3.6.2, 3.6.3, 3.6.4, 3.6.5 and 3.6.6 should be used, but more exact methods of calculating resistance to local buckling may also be used where appropriate.

In members that are mainly stressed by axial compression, the possible effects of local buckling on serviceability should be taken into account for cross-sections that include internal elements wider than 70ϵ times their thickness for cold formed RHS, or 80ϵ times for other sections.

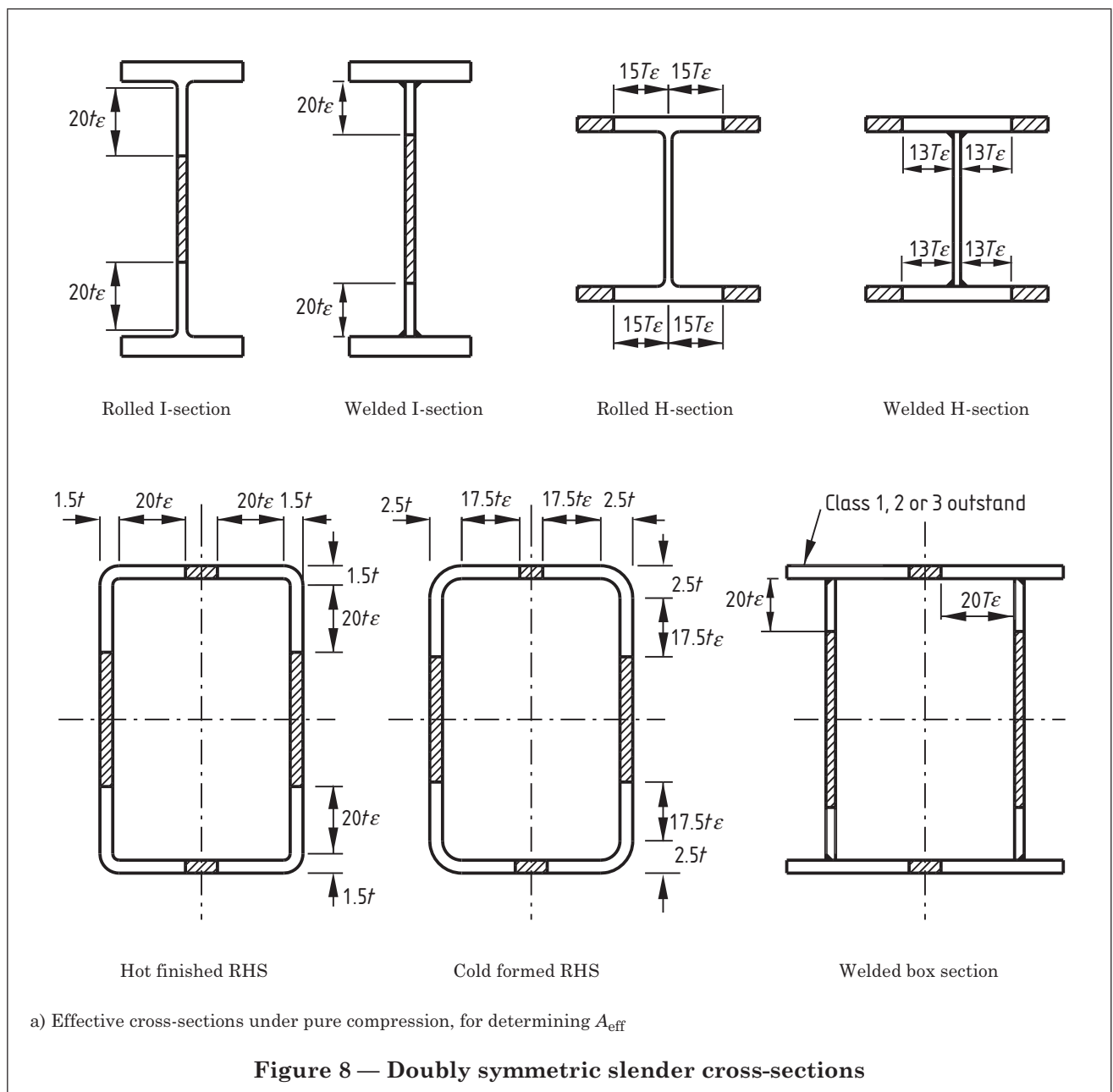
3.6.2 Doubly symmetric cross-sections

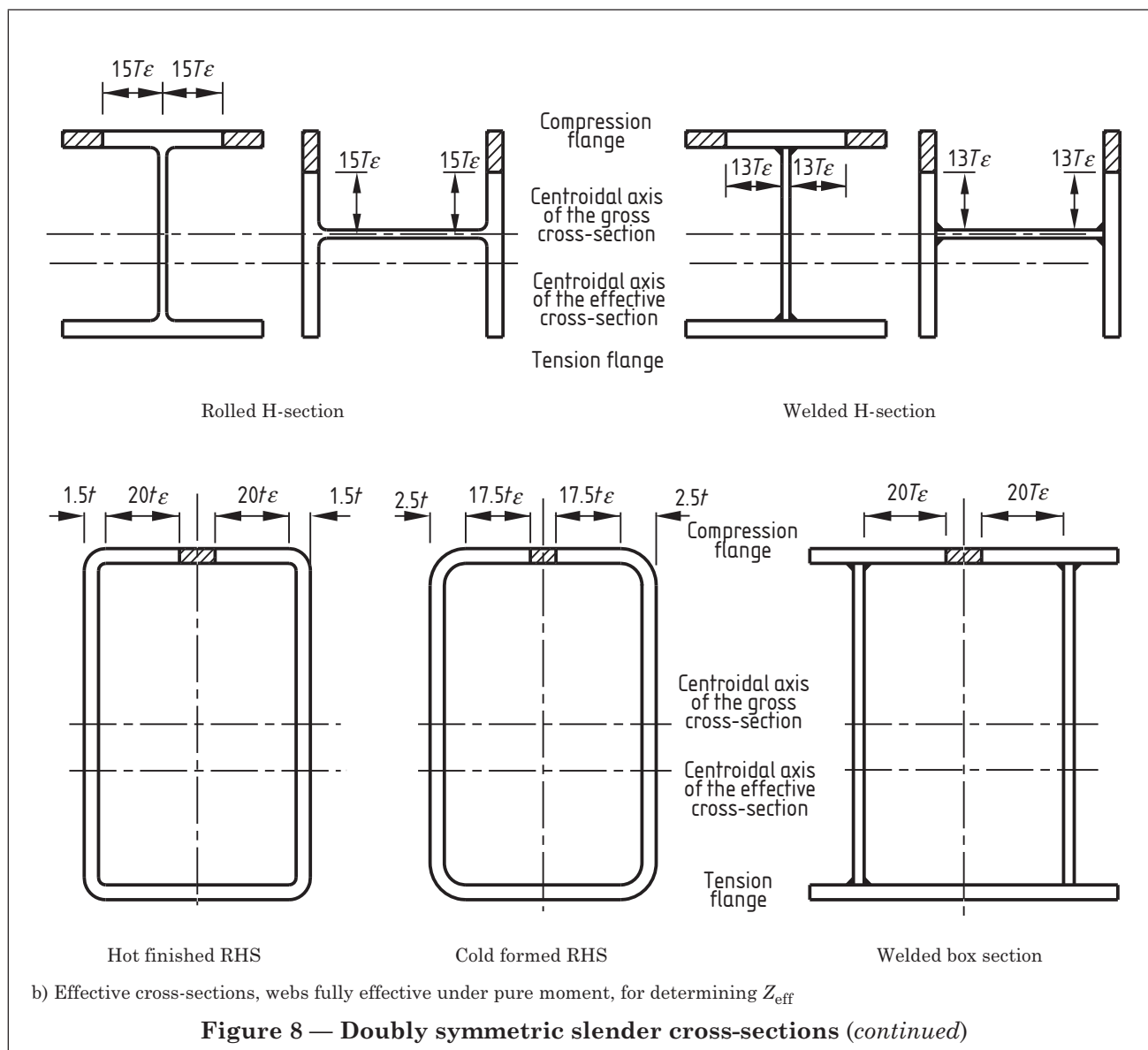
3.6.2.1 General

The methods given in 3.6.2.2, 3.6.2.3 and 3.6.2.4 may be used for doubly symmetric cross-sections that include class 4 slender elements. The effective cross-sectional area A_{eff} and the values of effective section modulus Z_{eff} for bending about the major and minor axes should each be determined from separate effective cross-sections as detailed in 3.6.2.2, 3.6.2.3 and 3.6.2.4.

3.6.2.2 Effective area

The effective cross-sectional area A_{eff} should be determined from the effective cross-section as shown in Figure 8a). The effective width of a class 4 slender web element or internal flange element should be taken as 35ε times its thickness for cold formed RHS, or 40ε times for other sections, comprising two equal portions with a central non-effective zone. The effective width of a class 4 slender outstand element should be taken as equal to the maximum width for class 3 derived from Table 11.





3.6.2.3 Effective modulus when web is fully effective

For cross-sections with webs that are not class 4 slender under pure bending, the effective section modulus Z_{eff} should be determined from an effective cross-section in which the effective width of any class 4 slender element in the compression flange is determined as detailed in 3.6.2.2, see Figure 8b).

If the whole cross-section is fully effective for bending about a given axis then Z_{eff} should be taken as equal to the section modulus Z about that axis.

If the cross-section is not fully effective in resisting bending about the major or minor axis, causing the relevant effective cross-section to be asymmetric about the axis of bending, the smaller of the two values of Z_{eff} for that axis should be used.

3.6.2.4 Effective modulus when web is slender

For cross-sections with webs that are class 4 slender under pure bending, the effective section modulus Z_{eff} should be determined from an effective cross-section obtained by adopting an effective width b_{eff} for the compression zone of the web, arranged as indicated in Figure 9, with $0.4b_{\text{eff}}$ adjacent to the compression flange and $0.6b_{\text{eff}}$ adjacent to the elastic neutral axis.

The effective width b_{eff} of the compression zone under pure bending should be obtained from:

$$b_{\text{eff}} = \frac{120\epsilon t}{\left(1 + \frac{f_{\text{cw}} - f_{\text{tw}}}{p_{\text{yw}}}\right)\left(1 + \frac{f_{\text{tw}}}{f_{\text{cw}}}\right)}$$

where

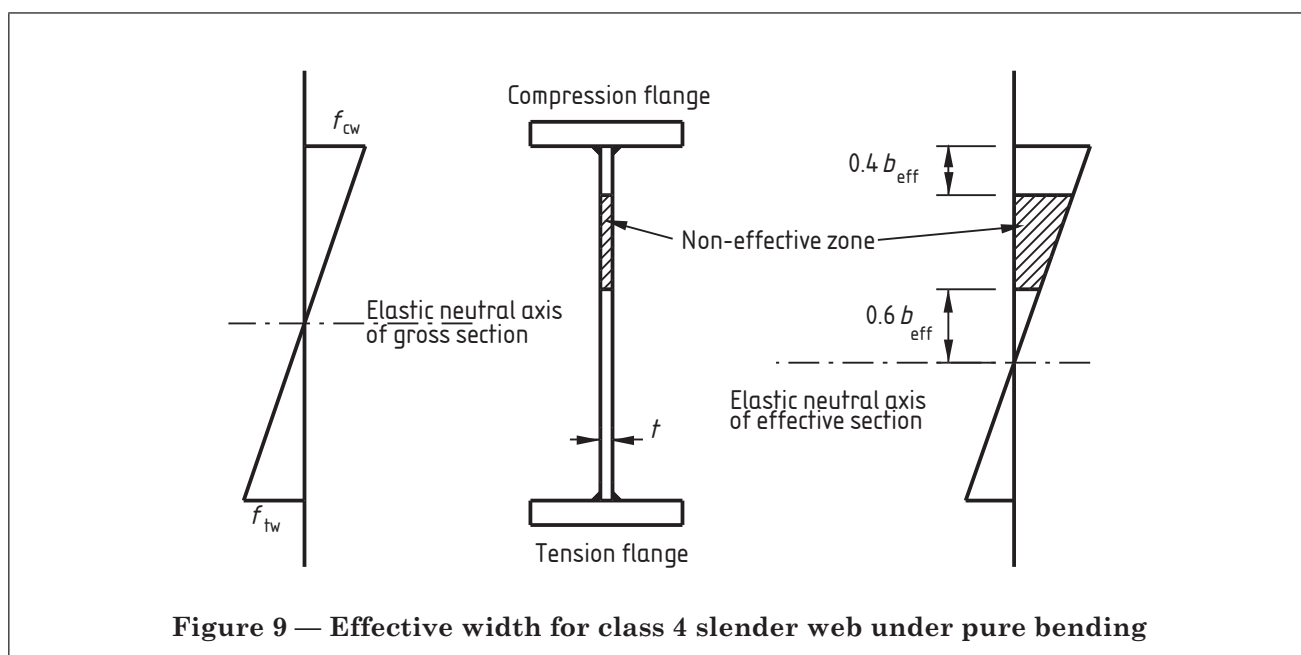
f_{cw} is the maximum compressive stress in the web, see Figure 9;

f_{tw} is the maximum tensile stress in the web, see Figure 9;

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

t is the web thickness.

The values of f_{cw} and f_{tw} used to determine b_{eff} should be based on a cross-section in which the web is taken as fully effective, using the effective width of the compression flange if this is class 4 slender.



3.6.3 Singly symmetric and unsymmetric cross-sections

The effective widths detailed in 3.6.2.2 may also be used for class 4 singly symmetric and unsymmetric cross-sections, provided that account is taken of the additional moments induced in the member due to the shift of the centroid of the effective cross-section compared to that of the gross cross-section.

These additional moments should be obtained by assuming that the axial compressive force F_c acts at the centroid of the gross cross-section, but is resisted by an equal and opposite force acting at the centroid of the effective cross-section that corresponds to the case of a uniform stress equal to the design strength p_y acting throughout its effective cross-sectional area. The additional moments should be taken into account in the checks on cross-section capacity and member buckling resistance given in 4.2, 4.3, 4.4, 4.7, and 4.8, except where a more onerous condition occurs if they are omitted.

3.6.4 Equal-leg angle sections

For class 4 slender hot rolled equal-leg angle sections, the method given in 3.6.3 may be used. Alternatively, the effective cross-sectional area A_{eff} and effective section modulus Z_{eff} about a given axis may conservatively be obtained using:

$$\frac{A_{\text{eff}}}{A} = \frac{12\varepsilon}{b/t}$$

$$\frac{Z_{\text{eff}}}{Z} = \frac{15\varepsilon}{b/t}$$

where

b is the leg length;

t is the thickness.

3.6.5 Alternative method

As an alternative to the methods detailed in 3.6.2, 3.6.3 and 3.6.4, a reduced design strength p_{yr} may be calculated at which the cross-section would be class 3 semi-compact. The reduced design strength p_{yr} should then be used in place of p_y in the checks on section capacity and member buckling resistance given in 4.2, 4.3, 4.4, 4.7 and 4.8. The value of this reduced design strength p_{yr} may be obtained from:

$$p_{\text{yr}} = (\beta_3/\beta)^2 p_y$$

in which β is the value of b/T , b/t , D/t or d/t that exceeds the limiting value β_3 given in Table 11 or Table 12 for a class 3 semi-compact section.

NOTE Unless the class 3 semi-compact limit is exceeded by only a small margin, the use of this alternative method can be rather conservative.

3.6.6 Circular hollow sections

Provided that the overall diameter D does not exceed $240t\varepsilon^2$ the effective cross-sectional area A_{eff} and effective section modulus Z_{eff} of a class 4 slender circular hollow section of thickness t may be determined from:

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

$$\frac{Z_{\text{eff}}}{Z} = \left[\left(\frac{140}{D/t} \right) \left(\frac{275}{p_y} \right) \right]^{0.25}$$