BS 5950-1:2000

Incorporating Corrigendum No. 1

Structural use of steelwork in building —

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

ICS: 91.080.10

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DETR (Construction Directorate)
DETR (Highways Agency)
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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This part of BS 5950 supersedes BS 5950-1:1990, which is withdrawn.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.

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.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself 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 213 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
	or	Diameter of section
	or	Diameter of hole
d		Depth of web
	or	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
	or	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
v	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 °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.

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 at the 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 the notional horizontal forces from **2.4.2.4**.

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*

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".

Except where plastic analysis is used, provided that λ_{cr} is not less than 4.0 the secondary forces and moments should be allowed for as follows:

a) if the resistance to horizontal forces is provided by moment-resisting joints or by cantilever columns, either by using sway mode in-plane effective lengths for the columns and designing the beams to remain elastic under the factored loads, or alternatively by using the method specified in b);

b) by multiplying the sway effects (see **2.4.2.8**) 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:

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

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:

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

If λ_{cr} is less than 4.0 second-order elastic analysis should be used.

If plastic analysis is used, reference should be made to **5.5** for portal frames or **5.7** for multi-storey frames.

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 °C for internal steelwork and –15 °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 $T_{27J} \leq T_{min} + 20$ °C:

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

— if $T_{27J} > T_{min} + 20$ °C:

$$t_1 \leq 50(1.2)^N \left(\frac{35 + T_{min} - T_{27J}}{15}\right) \left[\frac{355}{Y_{nom}}\right]^{1.4}$$

in which:

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

where

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

T_{27J} is the test temperature (in °C) for which a minimum Charpy impact value C_v of 27 J is specified in the product standard, or the equivalent value given in Table 7;

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	**Components in tension due to factored loads**		**Components not subject to applied tension**
	Stress ≥ 0.3Y_{nom}	**Stress < 0.3Y_{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**	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.

Table 4 — Thickness t_1 for plates, flats and rolled sections[ab]

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:					
S 275	25	0	0	0	0
S 275 JR	30	0	0	0	0
S 275 J0	65	54	30	0	0
S 275 J2	94	78	65	54	30
S 355	16	0	0	0	0
S 355 JR	21	0	0	0	0
S 355 J0	46	38	21	0	0
S 355 J2	66	55	46	38	21
S 355 K2	79	66	55	46	38
BS EN 10113:					
S 275 M *or* S 275 N	113	94	78	65	54
S 275 ML *or* S 275 NL	162	135	113	94	78
S 355 M *or* S 355 N	79	66	55	46	38
S 355 ML *or* S 355 NL	114	95	79	66	55
S 460 M *or* S 460 N	55	46	38	32	26
S 460 ML *or* S 460 NL	79	66	55	46	38
BS EN 10137:					
S 460 Q	46	38	32	26	15
S 460 QL	66	55	46	38	21
S 460 QL1	95	79	66	55	46
BS EN 10155:					
S 355 J0W *or* S 355 J0WP	46	38	21	0	0
S 355 J2W *or* S 355 J2WP	66	55	46	38	21
S 355 K2W	79	66	55	46	38

[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.

Table 5 — Thickness t_1 for structural hollow sections

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	30	0	0
S 275 J2H	94	78	65	54	30
S 275 NH	113	94	78	65	54
S 275 NLH	162	135	113	94	78
S 355 J0H	46	38	21	0	0
S 355 J2H	66	55	46	38	21
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	30	0	0
S 275 J2H	94	78	65	54	30
S 275 MH *or* S 275 NH	113	94	78	65	54
S 275 MLH *or* S 275 NLH	162	135	113	94	78
S 355 J0H	46	38	21	0	0
S 355 J2H	66	55	46	38	21
S 355 MH *or* S 355 NH	79	66	55	46	38
S 355 MLH *or* S 355 NLH	114	95	79	66	55
S 460 MH *or* S 460 NH	55	46	38	32	26
S 460 MLH *or* S 460 NLH	79	66	55	46	38
BS 7668:					
S 345 J0WH *or* S 345 J0WPH	48	40	22	0	0
S 345 GWH	62	52	43	36	10

Table 6 — Maximum thickness t_2[a] (mm)

Product standard	Steel grade or quality	Sections	Plates and flats	Hollow sections
BS EN 10025	S 275 *or* S 355	100	150	—
BS EN 10113-2	S 275 *or* S 355	150	150	—
	S 460	100	100	—
BS EN 10113-3	S 275, S 355 *or* S 460	150	63	—
BS EN 10137-2	S 460	—	150	—
BS EN 10155	J0WP *or* J2WP	40	16	—
	J0W, J2W *or* K2W	100	100	—
BS EN 10210-1	All	—	—	65
BS EN 10219-1	All	—	—	40
BS 7668	J0WPH	—	—	12
	J0WH *or* GWH	—	—	40

[a] Maximum thickness at which the full Charpy impact value given 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 10113	BS EN 10137	BS EN 10155	BS EN 10210	BS EN 10219	BS 7668
JR	+20 °C	—	—	—	+20 °C	+20 °C	—
J0	0 °C	—	—	0 °C	0 °C	0 °C	0 °C
J2	–20 °C	—	—	–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[b]	—	—	—	—
QL	—	—	–40 °C[b]	—	—	—	—
QL1	—	—	–60 °C[b]	—	—	—	—
G	—	—	—	—	—	—	–15 °C

[a] Equivalent test temperature for 27 J. Product standard specifies 40 J at –20 °C.
[b] Equivalent test temperature for 27 J. Product standard specifies 30 J at the same temperature.

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.

2.4.5.2 *Tying of buildings*

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/m^2 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.

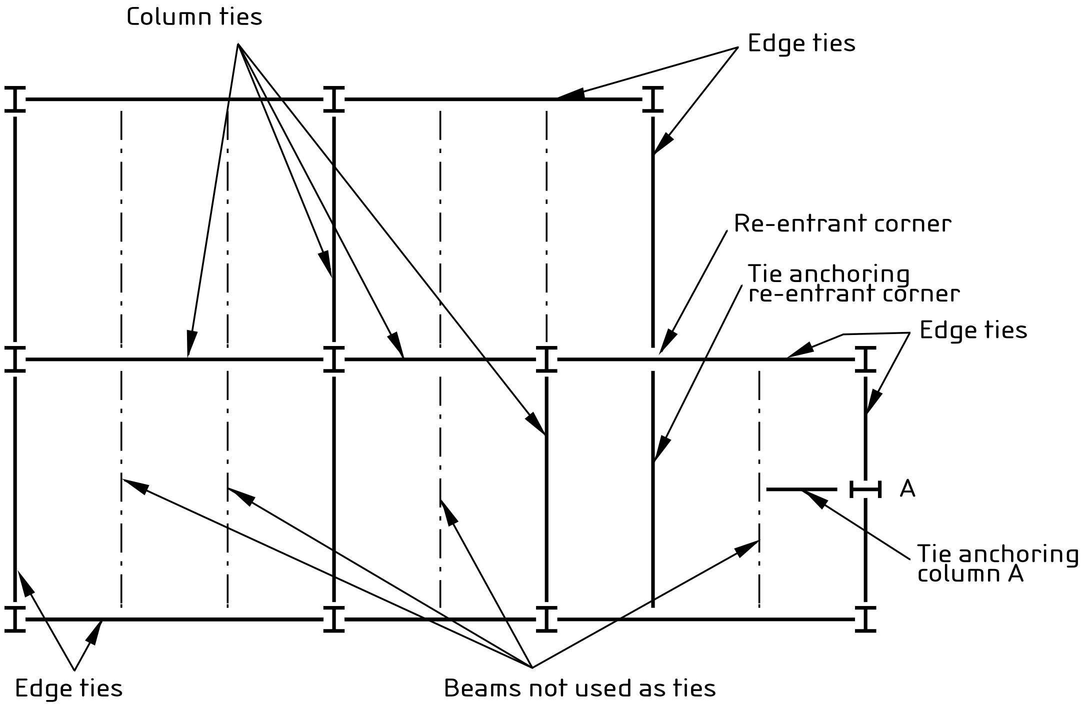


Figure 1 — Example of tying the columns of a building

2.4.5.3 ***Avoidance of disproportionate collapse***

Where regulations stipulate that certain buildings should be specially designed to avoid disproportionate collapse, 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.

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: $0.5(1.4g_k + 1.6q_k)s_tL$ but not less than 75 kN;

— for edge ties: $0.25(1.4g_k + 1.6q_k)s_tL$ 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.

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, or the larger end reaction 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 factored vertical dead and imposed load reaction 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 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. 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 m^2 (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.

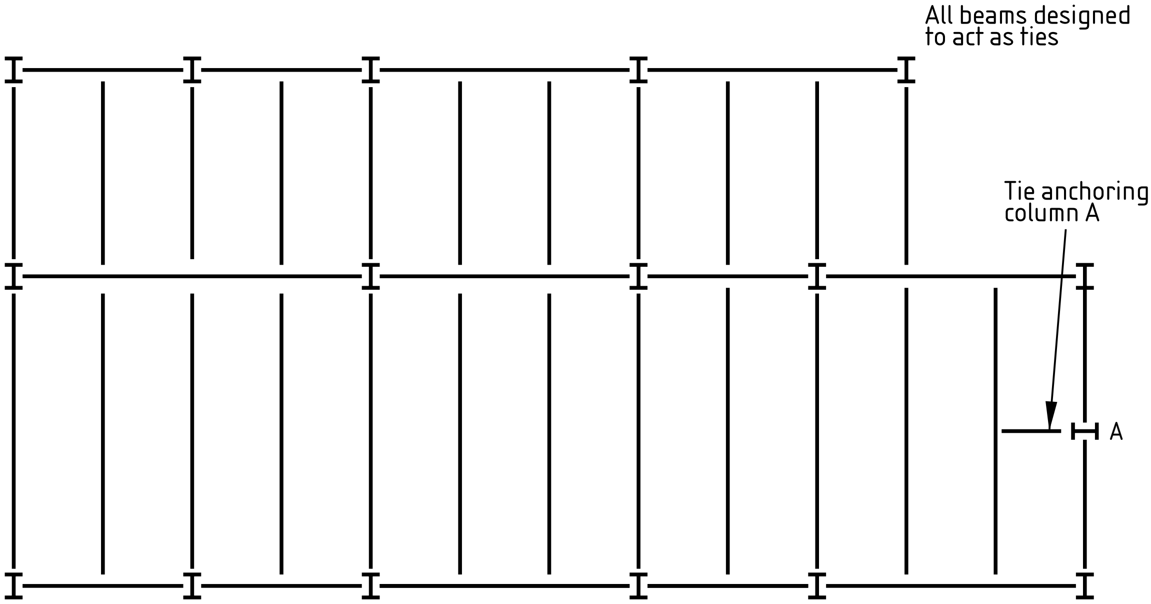


Figure 2 — Example of general tying of a building

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

a) *Vertical deflection of beams due to imposed load*	
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**
b) *Horizontal deflection of columns due to imposed load and wind load*	
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
c) *Crane girders*	
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/mm²
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 N/mm^2
— Shear modulus: $G = E/[2(1 + \nu)]$
— Poisson's ratio: ν = 0.30
— Coefficient of linear thermal expansion
(in the ambient temperature range): $\alpha = 12 \times 10^{-6}$ per °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 HSFG 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**;

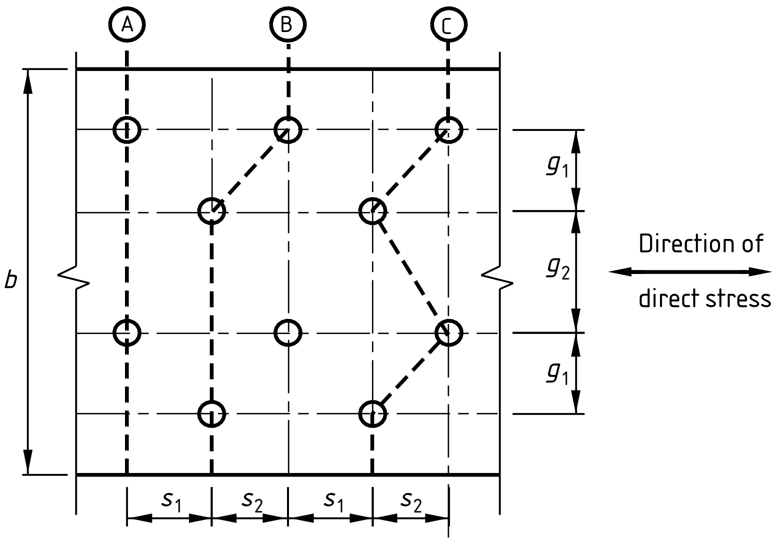
b) the sum of the sectional areas of a chain of holes lying on any diagonal or zig-zag line extending progressively across the member or element, see Figure 3, less an allowance of $0.25s^2t/g$ for each gauge space g that it traverses diagonally, where:

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.



on line A:

$A_n = t[b - 2D]$

on line B:

$A_n = t[b - 3D + 0.25 s_2^2/g_1]$

on line C:

$A_n = t[b - 4D + 0.5 s_2^2/g_1 + 0.25 s_2^2/g_2]$

where D is the hole diameter.

Figure 3 — Staggered holes

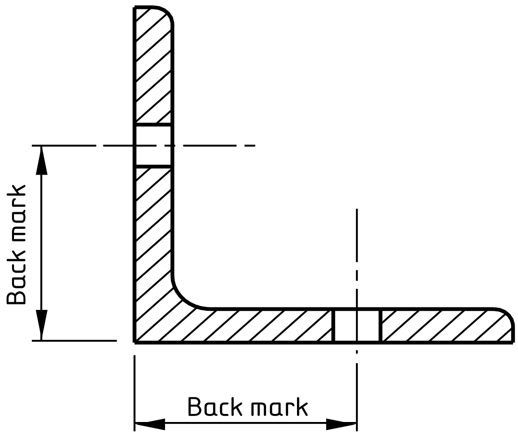


Figure 4 — Angle with holes in both legs

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;

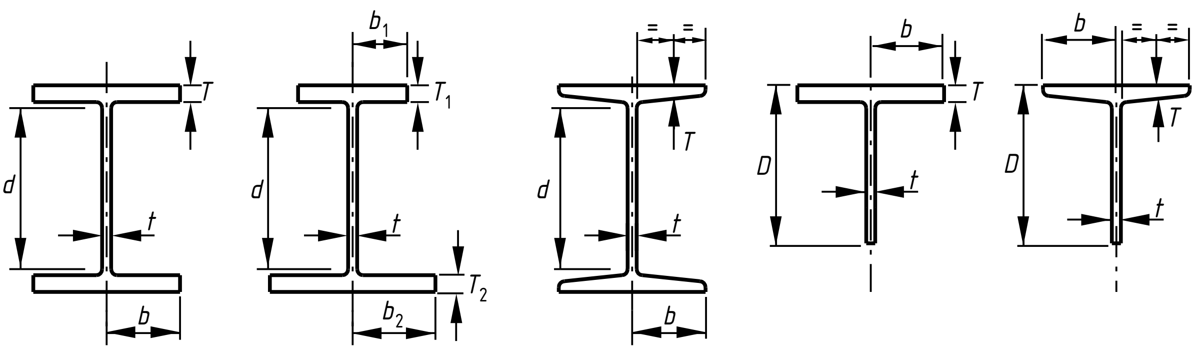
b) *internal elements* attached to other elements on both longitudinal edges and including:

— *webs* comprising internal elements perpendicular to the axis of bending;

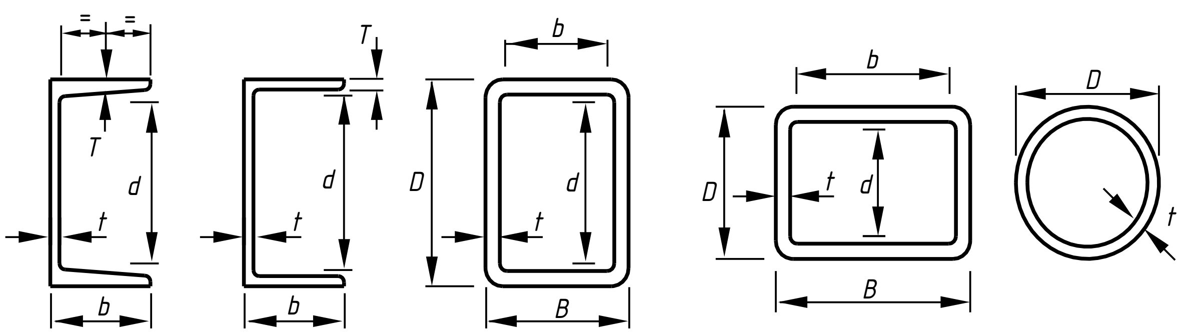
— *flanges* comprising internal elements parallel to the axis of bending.

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.



Rolled I- or H-sections T-sections

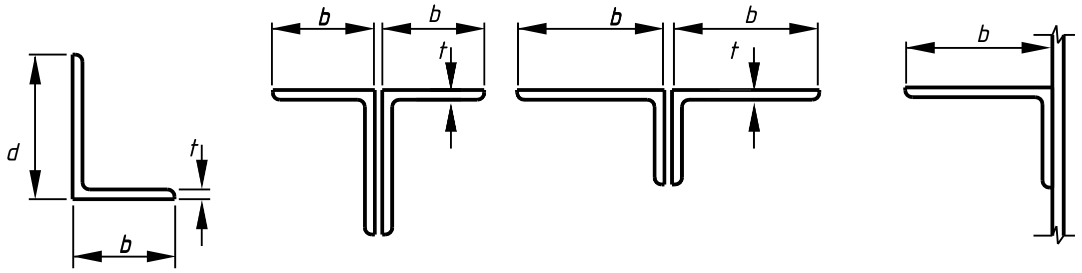


Rolled channels RHS[a] CHS

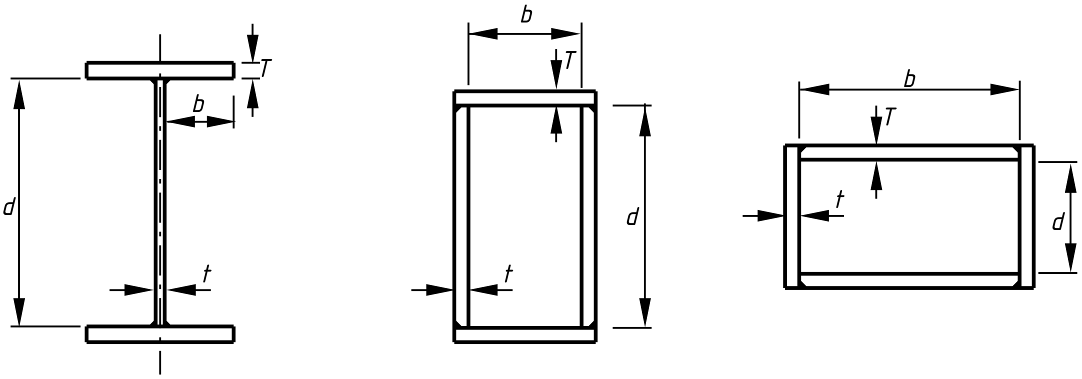
[a] For an RHS or box section, B and b are flange dimensions and D and d are web dimensions. The distinction between webs and flanges depends upon whether the member is bent about its major axis or its minor axis, see **3.5.1**.

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

Figure 5 — Dimensions of compression elements (*continued overleaf*)



Single angle[b] Double angles[b] Outstand[b]



Welded I-section Welded box sections[a]

[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 capacit*y. 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.

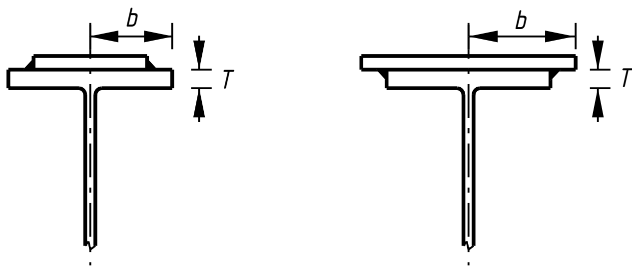
3.5.3 Flanges of compound I- or H-sections

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

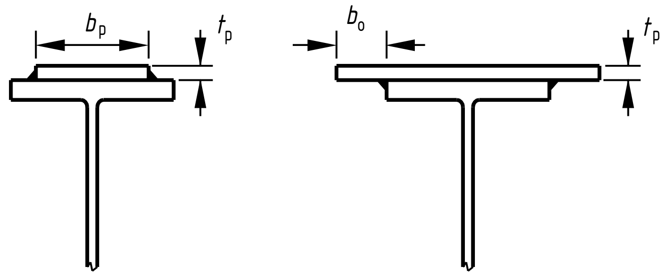
a) the ratio of the outstand b of the compound flange, see Figure 6a), to the thickness T of the original flange should be classified under "*outstand element of compression flange–rolled section*", see Table 11;

b) 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;

c) 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.



a) Outstand of compound flange



b) Internal width of plate c) Outstand of plate

Figure 6 — Dimensions of compound flanges

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	$\dfrac{80\varepsilon}{1 + r_1}$ but $\geq 40\varepsilon$	$\dfrac{100\varepsilon}{1 + r_1}$	$\dfrac{120\varepsilon}{1 + 2r_2}$ but $\geq 40\varepsilon$
		If r_1 is positive:	d/t		$\dfrac{100\varepsilon}{1 + 1.5r_1}$ 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 d/t	9ε 9ε	10ε 10ε	15ε 15ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b + d)/t$	Not applicable		15ε 15ε 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	$40\varepsilon^2$	$50\varepsilon^2$	$140\varepsilon^2$
	Axial compression		D/t	Not applicable		$80\varepsilon^2$
HF RHS	Flange	Compression due to bending	b/t	28ε but $\leq 80\varepsilon - d/t$	32ε but $\leq 62\varepsilon - 0.5d/t$	40ε
		Axial compression	b/t	Not applicable		
	Web	Neutral axis at mid-depth	d/t	64ε	80ε	120ε
		Generally[cd]	d/t	$\dfrac{64\varepsilon}{1+0.6r_1}$ but $\geq 40\varepsilon$	$\dfrac{80\varepsilon}{1+r_1}$ but $\geq 40\varepsilon$	$\dfrac{120\varepsilon}{1+2r_2}$ but $\geq 40\varepsilon$
		Axial compression[d]	d/t	Not applicable		
CF RHS	Flange	Compression due to bending	b/t	26ε but $\leq 72\varepsilon - d/t$	28ε but $\leq 54\varepsilon - 0.5d/t$	35ε
		Axial compression[d]	b/t	Not applicable		
		Neutral axis at mid-depth	d/t	56ε	70ε	105ε
		Generally[cd]	d/t	$\dfrac{56\varepsilon}{1+0.6r_1}$ but $\geq 35\varepsilon$	$\dfrac{70\varepsilon}{1+r_1}$ but $\geq 35\varepsilon$	$\dfrac{105\varepsilon}{1+2r_2}$ but $\geq 35\varepsilon$
		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 $\varepsilon = (275/p_y)^{0.5}$.

[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.

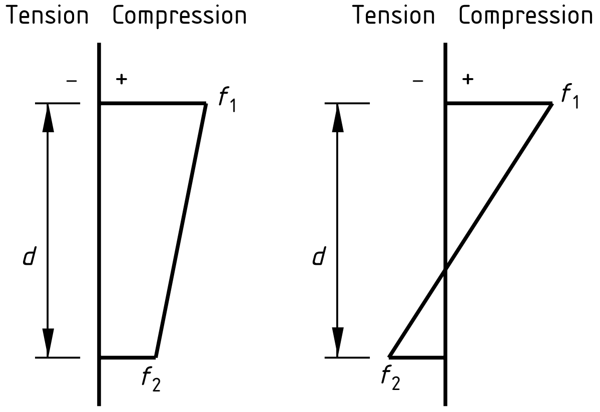


Figure 7 — Stress ratio for a semi-compact web

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 $S_{x,eff}$ and $S_{y,eff}$ about the major and minor axes may be obtained from:

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

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

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:

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

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.

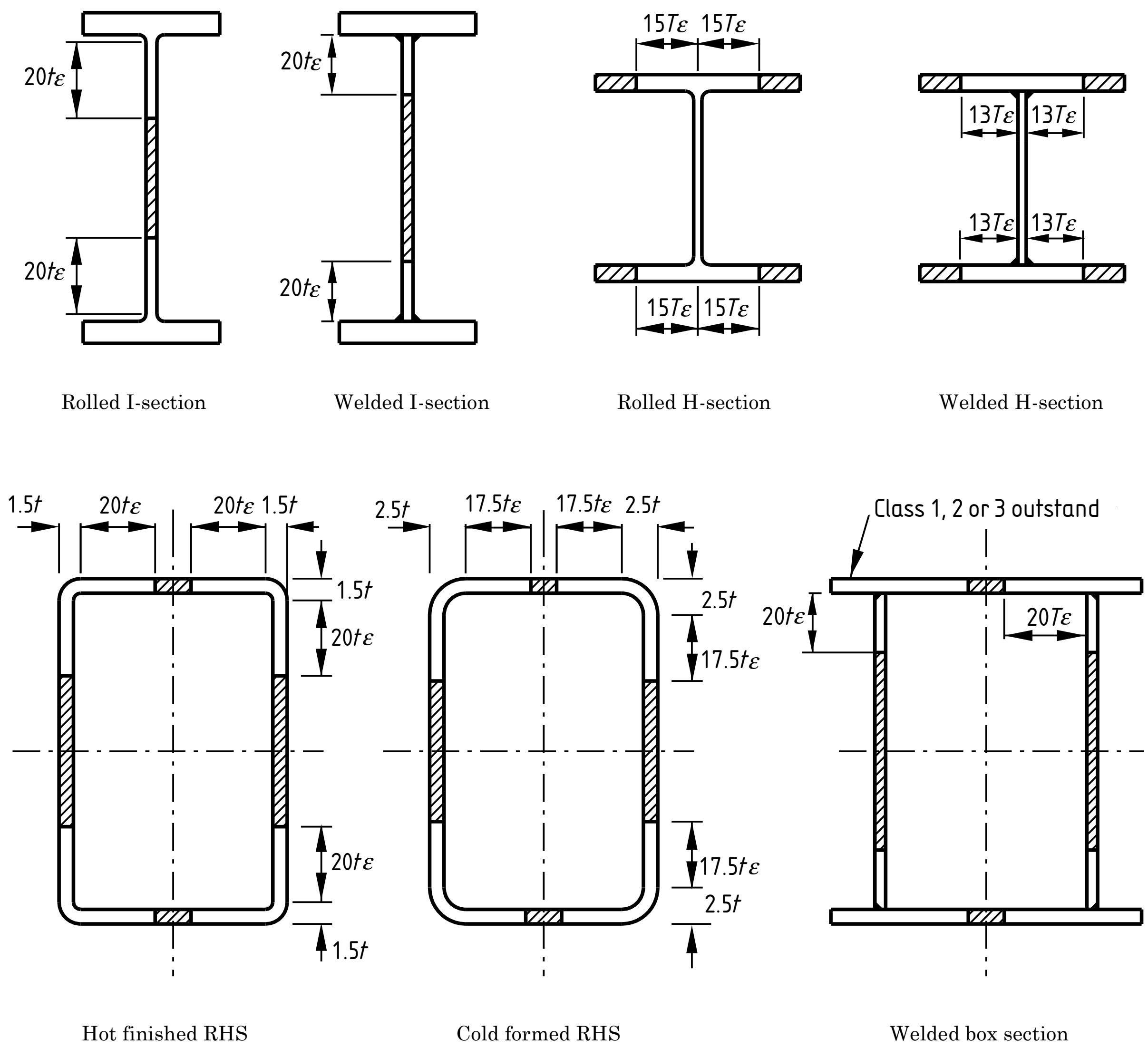
3.6.2 Doubly symmetric cross-sections

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**.

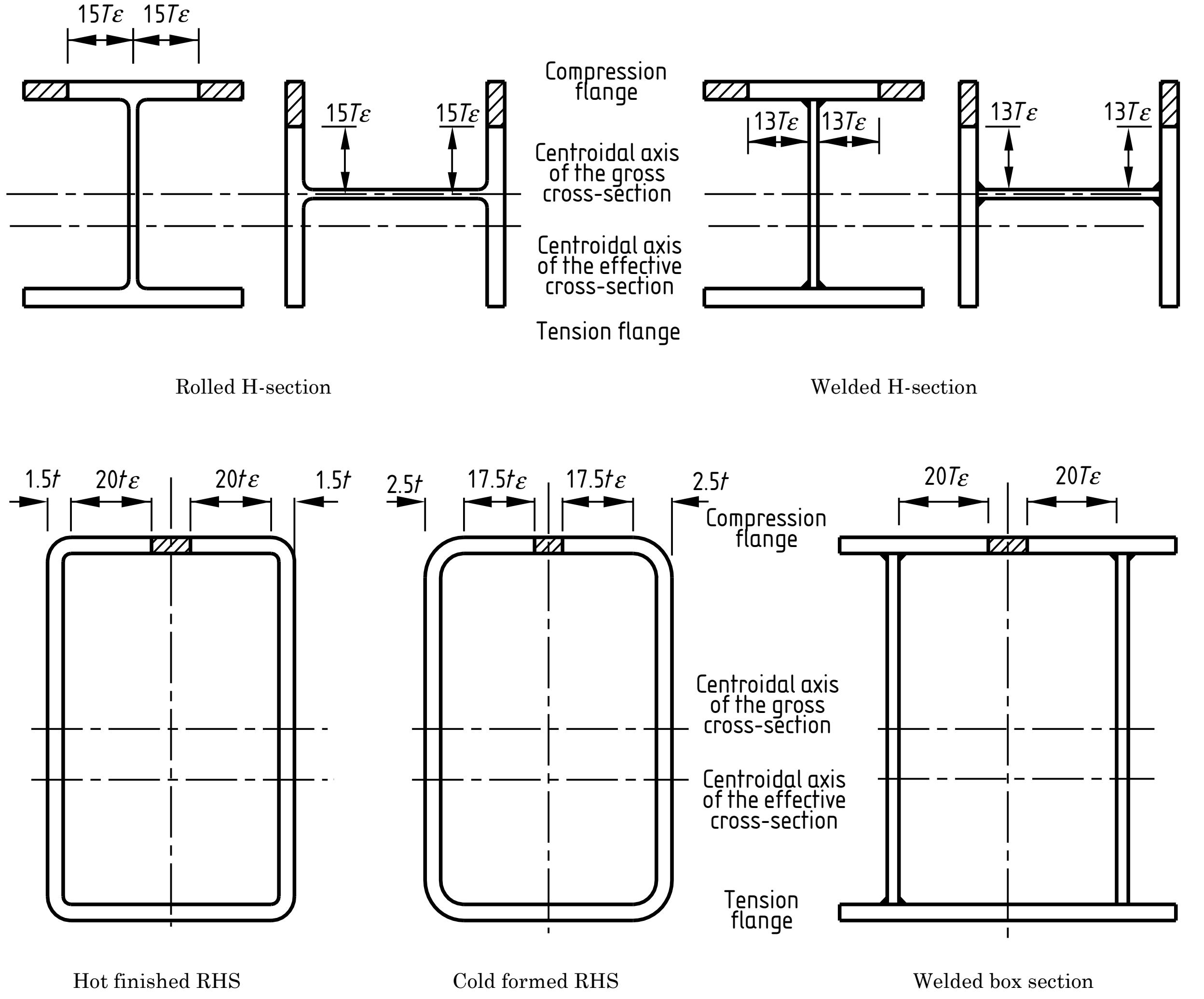
3.6.2.2 *Effective area*

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.



a) Effective cross-sections under pure compression, for determining A_{eff}

Figure 8 — Doubly symmetric slender cross-sections



b) Effective cross-sections, webs fully effective under pure moment, for determining Z_{eff}

Figure 8 — Doubly symmetric slender cross-sections (*continued*)

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 $0.4b_{eff}$ adjacent to the compression flange and $0.6b_{eff}$ adjacent to the elastic neutral axis.

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

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

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.

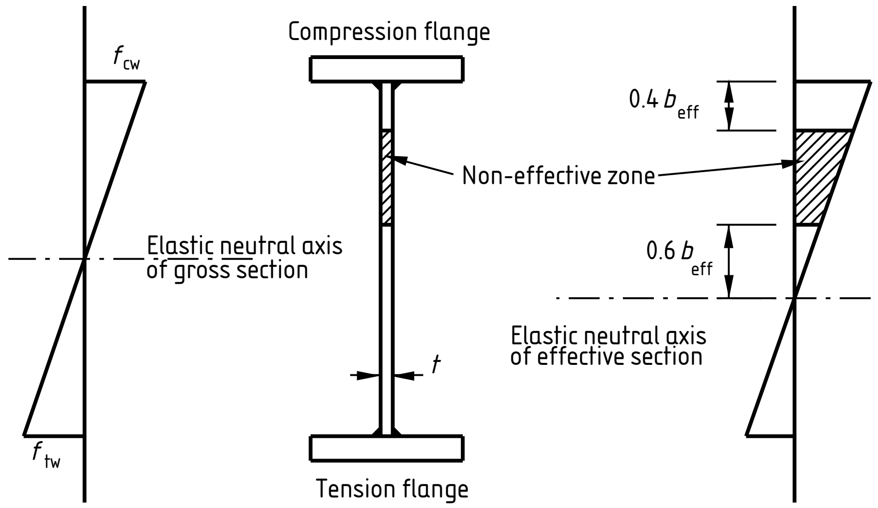


Figure 9 — Effective width for class 4 slender web under pure bending

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:

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

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

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:

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

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:

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

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

Section 4. Design of structural members

4.1 General

4.1.1 Application

This Section 4 applies to the design of simple members and of members that comprise parts of frames.

4.1.2 Class of cross-section

Reference should be made to **3.5** for the classification of cross-sections.

4.1.3 Design strength

The design strength p_y should be obtained from **3.1.1**.

4.2 Members subject to bending

4.2.1 General

4.2.1.1 ***General conditions***

All members subject to bending should meet the following conditions.

a) At critical points the combination of maximum moment and co-existent shear, and the combination of maximum shear and co-existent moment should be checked.

b) The deflection criteria given in **2.5.2** should be taken into account.

c) Unless the member is fully restrained against lateral-torsional buckling as indicated in **4.2.2**, its resistance to lateral-torsional buckling should be checked in accordance with **4.3**.

d) For class 4 slender sections, local buckling should be taken into account as given in **3.6**.

e) When loads or reactions are applied through the flange to the web the conditions of **4.5** for bearing and buckling should be met.

4.2.1.2 ***Span of beams***

The span of a beam should be taken between the effective points of support.

4.2.1.3 ***Length of cantilevers***

The length of a cantilever should be taken as the distance from the effective point of the support to the tip of the cantilever.

4.2.2 Full lateral restraint

If a beam has full lateral restraint to its compression flange, its resistance to lateral-torsional buckling may be assumed to be adequate, provided that it also has nominal torsional restraint at its supports. Nominal torsional restraint at member supports (as distinct from full torsional restraint at member supports, see **4.3.3**) may be supplied by web cleats, partial depth end plates, fin plates or continuity with the next span.

Full lateral restraint may be assumed to exist if the frictional or positive connection of a floor (or other) construction to the compression flange of the member is capable of resisting a lateral force of not less than 2.5 % of the maximum force in the compression flange of the member. This lateral force should be considered as distributed uniformly along the flange, provided that the dead load of the floor and the imposed load it supports together constitute the dominant loading on the member. It should be ensured that the floor (or other) construction is capable of resisting this lateral force.

4.2.3 Shear capacity

The shear force F_v should not be greater than the shear capacity P_v given by:

$$P_v = 0.6p_yA_v$$

in which A_v is the shear area, taken as follows:

a) rolled I, H and channel sections, load parallel to web:	tD
b) welded I-sections, load parallel to web:	td
c) rectangular hollow sections, load parallel to webs:	$AD/(D + B)$
d) welded box sections, load parallel to webs:	$2td$
e) rolled T-sections, load parallel to web:	tD
f) welded T-sections, load parallel to web:	$t(D - T)$
g) circular hollow sections:	$0.6A$
h) solid bars and plates:	$0.9A$
i) any other case:	$0.9A_o$

where

A is the area of the cross-section;

A_o is the area of that rectilinear element of the cross-section which has the largest dimension in the direction parallel to the shear force;

B is the overall breadth;

D is the overall depth;

d is the depth of the web;

t is the web thickness.

In CHS and RHS sections the shear area should be assumed to be located adjacent to the neutral axis.

For the effect of bolt holes on shear capacity, reference should be made to **6.2.3**.

If the ratio d/t exceeds 70ε for a rolled section, or 62ε for a welded section, the web should be checked for shear buckling in accordance with **4.4.5**.

4.2.4 Elastic shear stress

In cross-sections with webs that vary in thickness the distribution of shear stresses should be calculated from first principles assuming linear elastic behaviour. In this case the peak value of the shear stress distribution should not exceed $0.7p_y$. For cross-sections with openings significantly larger than those normally required for bolts, reference should be made to **4.15**.

4.2.5 Moment capacity

4.2.5.1 *General*

The moment capacity M_c should be determined from **4.2.5.2**, **4.2.5.3** and **4.2.5.4** allowing for the effects of co-existing shear. The effects of bolt holes should be allowed for as detailed in **4.2.5.5**.

To avoid irreversible deformation under serviceability loads, the value of M_c should be limited to $1.5p_yZ$ generally and to $1.2p_yZ$ in the case of a simply supported beam or a cantilever.

4.2.5.2 ***Low shear***

Provided that the shear force F_v does not exceed 60 % of the shear capacity P_v:

— for class 1 plastic or class 2 compact cross-sections:

$M_c = p_y S$

— for class 3 semi-compact sections:

$M_c = p_y Z$ or alternatively $M_c = p_y S_{eff}$

— for class 4 slender cross-sections:

$M_c = p_y Z_{eff}$

where

S is the plastic modulus;

S_{eff} is the effective plastic modulus, see **3.5.6**;

Z is the section modulus;

Z_{eff} is the effective section modulus, see **3.6.2**.

4.2.5.3 ***High shear***

Where $F_v > 0.6P_v$:

— for class 1 plastic or class 2 compact cross-sections:

$M_c = p_y(S - \rho S_v)$

— for class 3 semi-compact cross-sections:

$M_c = p_y(Z - \rho S_v/1.5)$ or alternatively $M_c = p_y(S_{eff} - \rho S_v)$

— for class 4 slender cross-sections:

$M_c = p_y(Z_{eff} - \rho S_v/1.5)$

in which S_v is obtained from the following:

— for sections with unequal flanges:

$S_v = S - S_f$

in which S_f is the plastic modulus of the effective section excluding the shear area A_v defined in **4.2.3**;

— otherwise:

S_v is the plastic modulus of the shear area A_v defined in **4.2.3**;

and ρ is given by:

$\rho = [2(F_v/P_v) - 1]^2$

NOTE The reduction factor ρ starts when F_v exceeds $0.5P_v$ but the resulting reduction in moment capacity is negligible unless F_v exceeds $0.6P_v$.

Alternatively, for class 3 semi-compact cross-sections reference may be made to **H.3**, or for class 4 slender cross-sections reference may be made to **3.6** and **H.3**.

If the ratio d/t exceeds 70ε for a rolled section, or 62ε for a welded section, the moment capacity should be determined allowing for shear buckling in accordance with **4.4.4**.

4.2.5.4 *Notched ends*

For notched ends of I, H or channel section members the moment capacity M_c should be taken as follows.

a) *Low shear:* where $F_v \leq 0.75P_v$:

— for singly notched ends:

$M_c = p_y Z$

— for doubly notched ends:

$M_c = p_y t d^2/6$

b) *High shear:* where $F_v > 0.75P_v$:

— for singly notched ends:

$$M_c = 1.5 p_y Z \sqrt{1-(F_v/P_v)^2}$$

— for doubly notched ends:

$$M_c = (p_y t d^2/4)\sqrt{1-(F_v/P_v)^2}$$

where

d is the residual depth of a doubly notched end;

Z is the relevant section modulus of the residual tee at a singly notched end.

4.2.5.5 *Bolt holes*

No allowance need be made for bolt holes in a compression flange (or leg). No allowance need be made for bolt holes in a tension flange (or leg) if, for the tension element:

$a_{t.net} \geq a_t/K_e$

where

a_t is the area of the tension element;

$a_{t.net}$ is the net area of the tension element after deducting bolt holes;

K_e is the factor for effective net area given in **3.4.3**.

No allowance need be made for bolt holes in the tension zone of a web unless there are also bolt holes in the tension flange at the same location. Furthermore, no allowance need be made for bolt holes in a web if the condition given above is satisfied when both a_t and $a_{t.net}$ are based upon the complete tension zone, comprising the tension flange plus the tension zone of the web.

If $a_{t.net}$ is less than a_t/K_e then an effective net area of $K_e a_{t.net}$ may be used.

4.3 Lateral-torsional buckling

4.3.1 General

Unless a beam or cantilever has full lateral restraint to its compression flange as described in **4.2.2**, then in addition to satisfying **4.2** its resistance to lateral-torsional buckling should also be checked.

Generally the resistance of a member to lateral-torsional buckling should be checked as detailed in **4.3.2**, **4.3.3**, **4.3.4**, **4.3.5**, **4.3.6**, **4.3.7** and **4.3.8**. However, for members that satisfy the conditions given in **G.1**, advantage may be taken of the methods for members with one flange restrained given in **G.2**.

4.3.2 Intermediate lateral restraints

4.3.2.1 *General*

If a member that is subject to bending needs intermediate lateral restraints within its length in order to develop the required buckling resistance moment, these restraints should have sufficient stiffness and strength to inhibit lateral movement of the compression flange relative to the supports.

Intermediate lateral restraints should generally be connected to the member as close as practicable to the compression flange and in any case closer to the level of the shear centre of the compression flange than to the level of the shear centre of the member. However, if an intermediate torsional restraint, see **4.3.3**, is also provided at the same cross-section, an intermediate lateral restraint may be connected at any level.

4.3.2.2 *Restraint forces*

4.3.2.2.1 Where intermediate lateral restraint is required at intervals within the length of a beam or cantilever, the intermediate lateral restraints should be capable of resisting a total force of not less than 2.5 % of the maximum value of the factored force in the compression flange within the relevant span, divided between the intermediate lateral restraints in proportion to their spacing.

The intermediate lateral restraints should either be connected to an appropriate system of bracing capable of transferring the restraint forces to the effective points of support of the member, or else connected to an independent robust part of the structure capable of fulfilling a similar function. Where two or more parallel members require intermediate lateral restraint, it is not adequate merely to connect the members together such that they become mutually dependent.

4.3.2.2.2 Where three or more intermediate lateral restraints are provided, each intermediate lateral restraint should be capable of resisting a force of not less than 1 % of the maximum value of the factored force in the compression flange within the relevant span.

The bracing system should be capable of resisting each of the following alternatives:

a) the 1 % restraint force considered as acting at only one point at a time;

b) the 2.5 % restraint force from **4.3.2.2.1**, divided between the intermediate lateral restraints in proportion to their spacing.

4.3.2.2.3 Bracing systems that supply intermediate lateral restraint to more than one member should be designed to resist the sum of the lateral restraint forces from each member that they restrain, determined in accordance with **4.3.2.2.1** and **4.3.2.2.2**, reduced by the factor k_r obtained from:

$$k_r = (0.2 + 1/N_r)^{0.5}$$

in which N_r is the number of parallel members restrained.

4.3.2.2.4 Purlins adequately restrained by sheeting need not normally be checked for forces caused by restraining rafters of roof trusses or portal frames that carry predominantly roof loads, provided that either:

a) there is bracing of adequate stiffness in the plane of the rafters; or

b) the roof sheeting is capable of acting as a stressed-skin diaphragm, see BS 5950-9.

4.3.3 Torsional restraints

A member may be taken as torsionally restrained (against rotation about its longitudinal axis) at any point in its length where both flanges are held in position relative to each other in the lateral direction, by external means not involving the lateral stiffness or resistance of the flanges themselves.

Full torsional restraint at member supports (as distinct from nominal torsional restraint at member supports, see **4.2.2**), should generally be provided in the form of lateral restraint to both flanges, or by similar means to intermediate torsional restraints. Alternatively, full torsional restraint at member supports may be provided by bearing stiffeners as recommended in **4.5.7**.

Intermediate torsional restraints within the length of the member may be provided by means of suitable diaphragms between two similar members, or else by equivalent panels of triangulated bracing.

Each torsional restraint should be capable of resisting a couple comprising two equal and opposite forces, acting at a lever arm equal to the depth between the centroids of the flanges, each equal to the larger of:

a) 1 % of the maximum value of the factored force in the compression flange within the relevant span;

b) 2.5 % of the maximum value of the factored force in the compression flange within the relevant span, divided between the intermediate lateral restraints in proportion to their spacing.

4.3.4 Destablizing load

The destabilizing loading condition should be taken where a load is applied to the top flange of a beam or a cantilever, and both the load and the flange are free to deflect laterally (and possibly rotationally also) relative to the centroid of the cross-section. Otherwise the normal loading condition should be assumed.

4.3.5 Effective length for lateral-torsional buckling

4.3.5.1 ***Simple beams without intermediate lateral restraints***

The effective length L_E for lateral-torsional buckling of a simple beam with restraints at the ends only, should be obtained from Table 13, taking the segment length L_{LT} as equal to the span L of the beam. If the restraint conditions at each end differ, the mean value of L_E should be taken.

The conditions of restraint against rotation of flanges on plan at member supports should be assessed taking into account the stiffness of the connections as well as the stiffness of the supporting members or other construction supplying restraint at the supports.

4.3.5.2 ***Simple beams with intermediate lateral restraints***

The effective length L_E for lateral-torsional buckling of a simple beam with intermediate lateral restraints should be taken as $1.0L_{LT}$ for normal loading conditions or $1.2L_{LT}$ for the destabilizing loading condition (see **4.3.4**), where L_{LT} is the length of the relevant segment between adjacent lateral restraints.

For the segment between a support and the adjacent intermediate lateral restraint, account should be taken of the restraint conditions at the support. The effective length L_E should be taken as the mean of the value given above and the value given by Table 13 for the restraint conditions at the support, taking L_{LT} as the length of the segment between the support and the lateral restraint in both cases.

4.3.5.3 ***Beams with double curvature bending***

In the case of continuous beams or other members subject to double curvature bending, consideration should be given to the regions subject to sagging moments and hogging moments as follows.

a) For a beam with intermediate lateral restraints to each flange, the segment length L_{LT} and the effective length L_E for lateral-torsional buckling should be determined as for a simple beam as given in **4.3.5.2** in hogging moment regions as well as in sagging moment regions. The lateral restraints to the compression flange of each region should extend up to or beyond the points of contraflexure.

b) For a beam with intermediate lateral restraints to the compression flange in the sagging moment region only, for the sagging moment region the segment length L_{LT} and effective length L_E for lateral-torsional buckling should be determined as for a simple beam as given in **4.3.5.2**. The lateral buckling resistance of the beam to the moments in the hogging moment regions should be determined using **G.2**.

c) For a beam directly supporting a concrete or composite floor or roof slab that provides full lateral restraint to the top flange, see **4.2.2**, the lateral buckling resistance of the beam to the moments in the hogging moment regions should be determined using **G.2**.

d) For a beam directly supporting a concrete or composite floor or roof slab that provides both lateral and torsional restraint to the top flange, an allowance may be made for this torsional restraint by assuming virtual lateral restraints to the bottom flange at the points of contraflexure when determining the segment length L_{LT}. In the absence of better information, torsional restraint of the top flange may be assumed if the depth of the beam is less than 550 mm and the slab is either:

— a composite slab with profiled steel sheeting, see BS 5950-4, designed to act compositely with the steel beam, see BS 5950-3.1; or

— a solid in situ concrete slab with a depth of not less than 25 % of the beam depth, designed to be continuous over the beam.

These virtual restraints should not be assumed if another form of allowance is made for the torsional restraint of the top flange by the slab. Lateral restraint of the bottom flange should not be assumed at a point of contraflexure under other restraint conditions, unless a lateral restraint is actually provided at that point.

Table 13 — Effective length L_E for beams without intermediate restraint

Conditions of restraint at supports		Loading condition: Normal	Loading condition: Destabilizing
Compression flange laterally restrained. Nominal torsional restraint against rotation about longitudinal axis, as given in **4.2.2**.	Both flanges fully restrained against rotation on plan.	$0.7L_{LT}$	$0.85L_{LT}$
	Compression flange fully restrained against rotation on plan.	$0.75L_{LT}$	$0.9L_{LT}$
	Both flanges partially restrained against rotation on plan.	$0.8L_{LT}$	$0.95L_{LT}$
	Compression flange partially restrained against rotation on plan.	$0.85L_{LT}$	$1.0L_{LT}$
	Both flanges free to rotate on plan.	$1.0L_{LT}$	$1.2L_{LT}$
Compression flange laterally unrestrained. Both flanges free to rotate on plan.	Partial torsional restraint against rotation about longitudinal axis provided by connection of bottom flange to supports.	$1.0L_{LT} + 2D$	$1.2L_{LT} + 2D$
	Partial torsional restraint against rotation about longitudinal axis provided only by pressure of bottom flange onto supports.	$1.2L_{LT} + 2D$	$1.4L_{LT} + 2D$
D is the overall depth of the beam.			

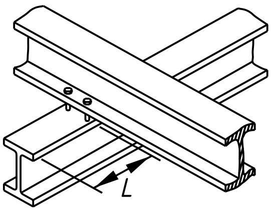
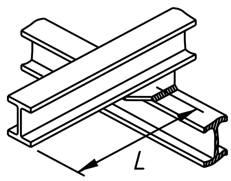
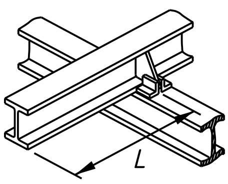
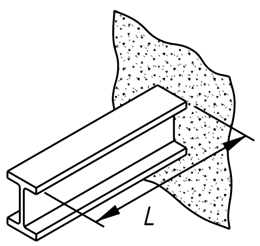
4.3.5.4 ***Cantilevers without intermediate restraints***

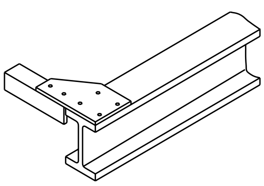
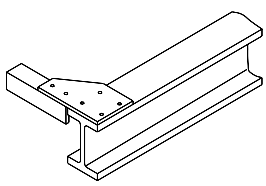
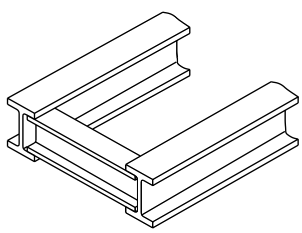
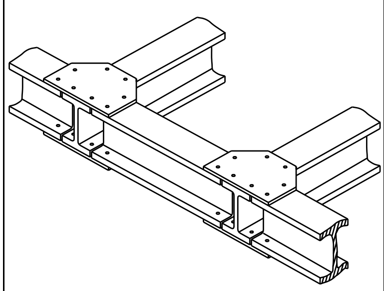
The effective length L_E for lateral-torsional buckling of a cantilever with no intermediate lateral restraint should be obtained from Table 14, taking L as the length of the cantilever. If a bending moment is applied at its tip, the effective length L_E from Table 14 should be increased by the greater of 30 % or $0.3L$.

4.3.5.5 ***Cantilevers with intermediate restraints***

Provided that the end restraint conditions correspond with cases c)4) or d)4) in Table 14, the effective length L_E for lateral-torsional buckling of a cantilever with intermediate lateral restraints to its compression flange should be taken as $1.0L$ for normal loading conditions, taking L as the length of the relevant segment between adjacent lateral restraints. However, for the destabilizing loading condition (see **4.3.4**) L_E should be obtained from Table 14, taking L as the length of the cantilever, unless the top flange also has intermediate lateral restraints.

Table 14 — Effective length L_E for cantilevers without intermediate restraint

Restraint conditions		Loading conditions	
At support	**At tip**	**Normal**	**Destabilizing**
a) Continuous, with lateral restraint to top flange 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
b) Continuous, with partial torsional restraint 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
c) Continuous, with lateral and torsional restraint 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
d) Restrained laterally, torsionally and against rotation on plan 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	0.8L 0.7L 0.6L 0.5L	1.4L 1.4L 0.6L 0.5L

Tip restraint conditions			
1) Free  (not braced on plan)	2) Lateral restraint to top flange  (braced on plan in at least one bay)	3) Torsional restraint  (not braced on plan)	4) Lateral and torsional restraint  (braced on plan in at least one bay)

4.3.6 Resistance to lateral-torsional buckling

4.3.6.1 *General*

Resistance to lateral-torsional buckling need not be checked separately (and the buckling resistance moment M_b may be taken as equal to the relevant moment capacity M_c) in the following cases:

— bending about the minor axis;

— CHS, square RHS or circular or square solid bars;

— RHS, unless L_E/r_y exceeds the limiting value given in Table 15 for the relevant value of D/B;

— I-, H-, channel or box sections, if λ_{LT} does not exceed λ_{L0}, see **4.3.6.5**.

Otherwise, for members subject to bending about their major axis, reference should be made as follows:

— for I-, H-, channel or box section members with equal flanges and a uniform cross-section throughout the length of the relevant segment L between adjacent lateral restraints, see **4.3.6.2**;

— for I-sections or box sections with unequal flanges but with a uniform cross-section throughout the length of the relevant segment L between adjacent lateral restraints, see **4.3.6.3**;

— for I-, H-, channel or box section members with a cross-section that varies within the length of the relevant segment L between adjacent lateral restraints, see **B.2.5**;

— for hot rolled angles, see **4.3.8**;

— for plates, flats or solid rectangular bars, see **B.2.7**;

— for T-sections see, **B.2.8**.

Table 15 — Limiting value of L_E/r_y for RHS

Ratio D/B	Limiting value of L_E/r_y	Ratio D/B	Limiting value of L_E/r_y	Ratio D/B	Limiting value of L_E/r_y
1.25	$770 \times (275/p_y)$	1.5	$515 \times (275/p_y)$	2.0	$340 \times (275/p_y)$
1.33	$670 \times (275/p_y)$	1.67	$435 \times (275/p_y)$	2.5	$275 \times (275/p_y)$
1.4	$580 \times (275/p_y)$	1.75	$410 \times (275/p_y)$	3.0	$225 \times (275/p_y)$
1.44	$550 \times (275/p_y)$	1.8	$395 \times (275/p_y)$	4.0	$170 \times (275/p_y)$

Key:

B is the width of the section;

D is the depth of the section;

L_E is the effective length for lateral-torsional buckling from **4.3.5**;

p_y is the design strength;

r_y is the radius of gyration of the section about its minor axis.

4.3.6.2 *I-, H-, channel and box sections with equal flanges*

In each segment of length L between adjacent lateral restraints, members of I-, H-, channel or box sections with equal flanges should satisfy:

$$M_x \leq M_b/m_{LT} \quad \text{and} \quad M_x \leq M_{cx}$$

where

M_b is the buckling resistance moment, see **4.3.6.4**;

M_{cx} is the major axis moment capacity of the cross-section, see **4.2.5**;

M_x is the maximum major axis moment in the segment;

m_{LT} is the equivalent uniform moment factor for lateral-torsional buckling, see **4.3.6.6**.

4.3.6.3 ***I-sections and box sections with unequal flanges***

In each segment of length L_{LT} between adjacent lateral restraints, members of I or box cross-section with unequal flanges should satisfy:

— for a segment of length L_{LT} subject to single curvature bending, the criteria given in **4.3.6.2** for sections with equal flanges;

— for a segment of length L_{LT} subject to double-curvature bending, the criteria given in **B.2.4.2**.

4.3.6.4 ***Buckling resistance moment***

For lateral-torsional buckling, the buckling resistance moment M_b should be obtained as follows:

a) for rolled I-, H- or channel sections with equal flanges, either using the general method given in c), or alternatively using the simple method given in **4.3.7**;

b) for single angles, as given in **4.3.8**;

c) generally, except as given in a) or b), M_b should be determined from the following:

— for class 1 plastic or class 2 compact cross-sections:

$M_b = p_b S_x$

— for class 3 semi-compact cross-sections:

$M_b = p_b Z_x$; or alternatively

$M_b = p_b S_{x,eff}$

— for class 4 slender cross-sections:

$M_b = p_b Z_{x,eff}$

where

p_b is the bending strength from **4.3.6.5**;

S_x is the plastic modulus about the major axis;

$S_{x,eff}$ is the effective plastic modulus about the major axis, see **3.5.6**;

Z_x is the section modulus about the major axis;

$Z_{x,eff}$ is the effective section modulus about the major axis, see **3.6.2**.

4.3.6.5 ***Bending strength*** p_b

If the equivalent slenderness λ_{LT} from **4.3.6.7** is not more than the limiting slenderness λ_{L0} for the relevant design strength p_y given at the foot of Table 16 and Table 17, then p_b should be taken as equal to p_y and no allowance need be made for lateral-torsional buckling.

Otherwise the bending strength p_b for the relevant values of λ_{LT} and p_y should be obtained from Table 16 for rolled sections or Table 17 for welded sections, or from the formula given in **B.2.1**.

4.3.6.6 ***Equivalent uniform moment factor*** m_{LT}

For the normal loading condition, the equivalent uniform moment factor for lateral-torsional buckling m_{LT} should be obtained from Table 18 for the pattern of major axis moments over the segment length L_{LT}.

For the destabilizing loading condition m_{LT} should be taken as 1.0.

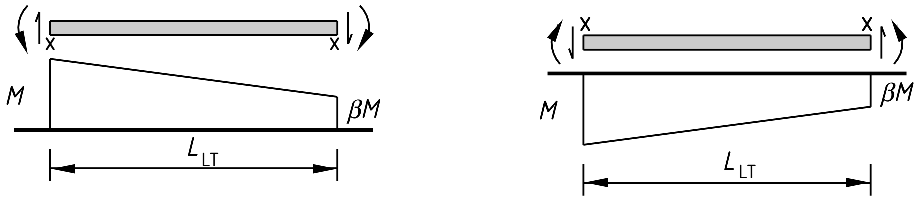
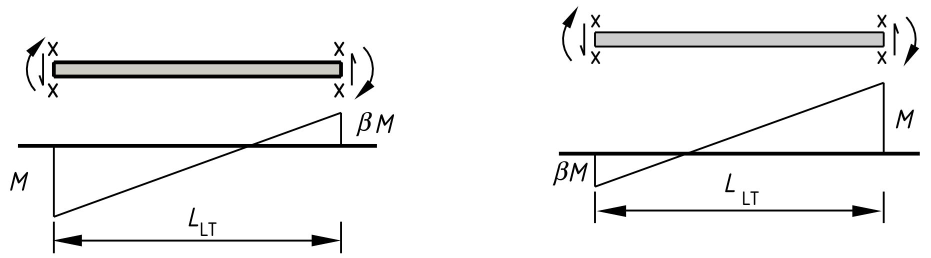
Table 16 — Bending strength p_b (N/mm²) for rolled sections

λ_{LT}	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
25	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
30	235	245	255	265	275	315	325	335	345	355	395	403	421	429	446
35	235	245	255	265	273	307	316	324	332	341	378	386	402	410	426
40	229	238	246	254	262	294	302	309	317	325	359	367	382	389	404
45	219	227	235	242	250	280	287	294	302	309	340	347	361	367	381
50	210	217	224	231	238	265	272	279	285	292	320	326	338	344	356
55	199	206	213	219	226	251	257	263	268	274	299	305	315	320	330
60	189	195	201	207	213	236	241	246	251	257	278	283	292	296	304
65	179	185	190	196	201	221	225	230	234	239	257	261	269	272	279
70	169	174	179	184	188	206	210	214	218	222	237	241	247	250	256
75	159	164	168	172	176	192	195	199	202	205	219	221	226	229	234
80	150	154	158	161	165	178	181	184	187	190	201	203	208	210	214
85	140	144	147	151	154	165	168	170	173	175	185	187	190	192	195
90	132	135	138	141	144	153	156	158	160	162	170	172	175	176	179
95	124	126	129	131	134	143	144	146	148	150	157	158	161	162	164
100	116	118	121	123	125	132	134	136	137	139	145	146	148	149	151
105	109	111	113	115	117	123	125	126	128	129	134	135	137	138	140
110	102	104	106	107	109	115	116	117	119	120	124	125	127	128	129
115	96	97	99	101	102	107	108	109	110	111	115	116	118	118	120
120	90	91	93	94	96	100	101	102	103	104	107	108	109	110	111
125	85	86	87	89	90	94	95	96	96	97	100	101	102	103	104
130	80	81	82	83	84	88	89	90	90	91	94	94	95	96	97
135	75	76	77	78	79	83	83	84	85	85	88	88	89	90	90
140	71	72	73	74	75	78	78	79	80	80	82	83	84	84	85
145	67	68	69	70	71	73	74	74	75	75	77	78	79	79	80
150	64	64	65	66	67	69	70	70	71	71	73	73	74	74	75
155	60	61	62	62	63	65	66	66	67	67	69	69	70	70	71
160	57	58	59	59	60	62	62	63	63	63	65	65	66	66	67
165	54	55	56	56	57	59	59	59	60	60	61	62	62	62	63
170	52	52	53	53	54	56	56	56	57	57	58	58	59	59	60
175	49	50	50	51	51	53	53	53	54	54	55	55	56	56	56
180	47	47	48	48	49	50	51	51	51	51	52	53	53	53	54
185	45	45	46	46	46	48	48	48	49	49	50	50	50	51	51
190	43	43	44	44	44	46	46	46	46	47	48	48	48	48	48
195	41	41	42	42	42	43	44	44	44	44	45	45	46	46	46
200	39	39	40	40	40	42	42	42	42	42	43	43	44	44	44
210	36	36	37	37	37	38	38	38	39	39	39	40	40	40	40
220	33	33	34	34	34	35	35	35	35	36	36	36	37	37	37
230	31	31	31	31	31	32	32	33	33	33	33	33	34	34	34
240	28	29	29	29	29	30	30	30	30	30	31	31	31	31	31
250	26	27	27	27	27	28	28	28	28	28	29	29	29	29	29
λ_{L0}	37.1	36.3	35.6	35.0	34.3	32.1	31.6	31.1	30.6	30.2	28.4	28.1	27.4	27.1	26.5

Table 17 — Bending strength p_b (N/mm²) for welded sections

λ_{LT}	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
25	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
30	235	245	255	265	275	315	325	335	345	355	390	397	412	419	434
35	235	245	255	265	272	300	307	314	321	328	358	365	378	385	398
40	224	231	237	244	250	276	282	288	295	301	328	334	346	352	364
45	206	212	218	224	230	253	259	265	270	276	300	306	316	321	332
50	190	196	201	207	212	233	238	243	248	253	275	279	288	293	302
55	175	180	185	190	195	214	219	223	227	232	251	255	263	269	281
60	162	167	171	176	180	197	201	205	209	212	237	242	253	258	269
65	150	154	158	162	166	183	188	194	199	204	227	232	242	247	256
70	139	142	146	150	155	177	182	187	192	196	217	222	230	234	242
75	130	135	140	145	151	170	175	179	184	188	207	210	218	221	228
80	126	131	136	141	146	163	168	172	176	179	196	199	205	208	214
85	122	127	131	136	140	156	160	164	167	171	185	187	190	192	195
90	118	123	127	131	135	149	152	156	159	162	170	172	175	176	179
95	114	118	122	125	129	142	144	146	148	150	157	158	161	162	164
100	110	113	117	120	123	132	134	136	137	139	145	146	148	149	151
105	106	109	112	115	117	123	125	126	128	129	134	135	137	138	140
110	101	104	106	107	109	115	116	117	119	120	124	125	127	128	129
115	96	97	99	101	102	107	108	109	110	111	115	116	118	118	120
120	90	91	93	94	96	100	101	102	103	104	107	108	109	110	111
125	85	86	87	89	90	94	95	96	96	97	100	101	102	103	104
130	80	81	82	83	84	88	89	90	90	91	94	94	95	96	97
135	75	76	77	78	79	83	83	84	85	85	88	88	89	90	90
140	71	72	73	74	75	78	78	79	80	80	82	83	84	84	85
145	67	68	69	70	71	73	74	74	75	75	77	78	79	79	80
150	64	64	65	66	67	69	70	70	71	71	73	73	74	74	75
155	60	61	62	62	63	65	66	66	67	67	69	69	70	70	71
160	57	58	59	59	60	62	62	63	63	63	65	65	66	66	67
165	54	55	56	56	57	59	59	59	60	60	61	62	62	62	63
170	52	52	53	53	54	56	56	56	57	57	58	58	59	59	60
175	49	50	50	51	51	53	53	53	54	54	55	55	56	56	56
180	47	47	48	48	49	50	51	51	51	51	52	53	53	53	54
185	45	45	46	46	46	48	48	48	49	49	50	50	50	51	51
190	43	43	44	44	44	46	46	46	46	47	48	48	48	48	48
195	41	41	42	42	42	43	44	44	44	44	45	45	46	46	46
200	39	39	40	40	40	42	42	42	42	42	43	43	44	44	44
210	36	36	37	37	37	38	38	38	39	39	39	40	40	40	40
220	33	33	34	34	34	35	35	35	35	36	36	36	37	37	37
230	31	31	31	31	31	32	32	33	33	33	33	33	34	34	34
240	28	29	29	29	29	30	30	30	30	30	31	31	31	31	31
250	26	27	27	27	27	28	28	28	28	28	29	29	29	29	29
λ_{L0}	37.1	36.3	35.6	35.0	34.3	32.1	31.6	31.1	30.6	30.2	28.4	28.1	27.4	27.1	26.5

Table 18 — Equivalent uniform moment factor m_{LT} for lateral-torsional buckling
(*continued overleaf*)

Segments with end moments only (values of m_{LT} from the formula for the general case)	β	m_{LT}
β positive  X Lateral restraint	1.0	1.00
	0.9	0.96
	0.8	0.92
	0.7	0.88
	0.6	0.84
	0.5	0.80
	0.4	0.76
	0.3	0.72
	0.2	0.68
	0.1	0.64
β negative 	0.0	0.60
	– 0.1	0.56
	– 0.2	0.52
	– 0.3	0.48
	– 0.4	0.46
	– 0.5	0.44
	– 0.6	0.44
	– 0.7	0.44
	– 0.8	0.44
	– 0.9	0.44
	– 1.0	0.44

Specific cases (no intermediate lateral restraint)

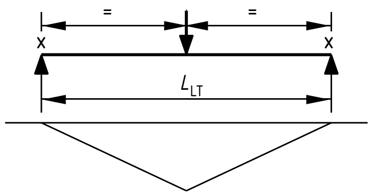
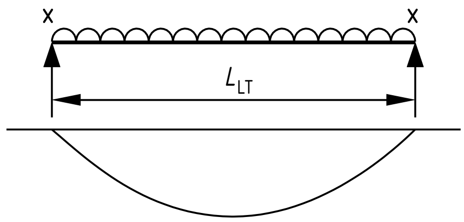
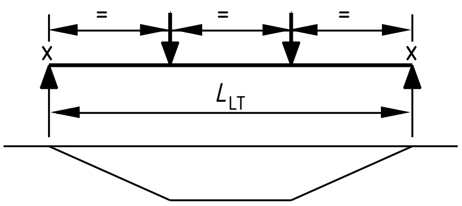
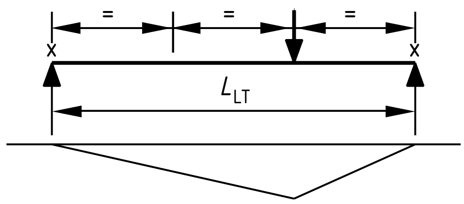
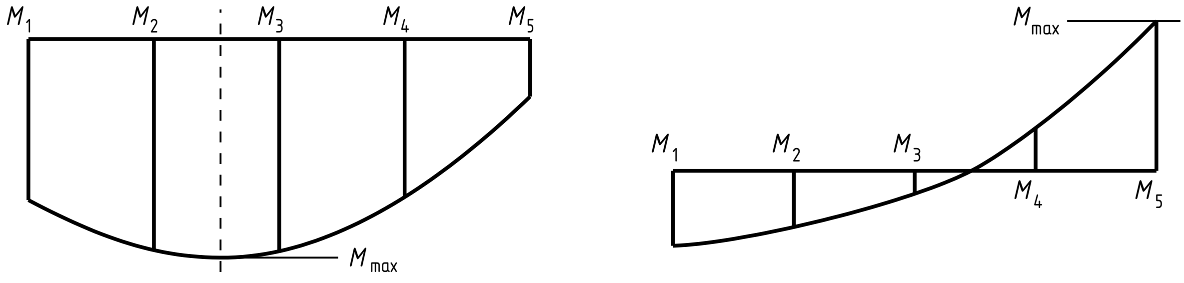
 $m_{LT} = 0.850$	 $m_{LT} = 0.925$
 $m_{LT} = 0.925$	 $m_{LT} = 0.744$

Table 18 — Equivalent uniform moment factor m_{LT} for lateral-torsional buckling (*continued*)

General case (segments between intermediate lateral restraints)

For beams: $m_{LT} = 0.2 + \frac{0.15M_2 + 0.5M_3 + 0.15M_4}{M_{max}}$ but $m_{LT} \geq 0.44$
All moments are taken as positive. The moments M_2 and M_4 are the values at the quarter points, the moment M_3 is the value at mid-length and M_{max} is the maximum moment in the segment.
For cantilevers without intermediate lateral restraint: $m_{LT} = 1.00$.

4.3.6.7 *Equivalent slenderness* λ_{LT}

The equivalent slenderness λ_{LT} should be obtained as follows.

a) *For an I- or H-section member* the equivalent slenderness λ_{LT} should be obtained using:

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$

in which:

$$\lambda = L_E/r_y$$

where

- L_E is the effective length for lateral-torsional buckling, from **4.3.5**;
- r_y is the radius of gyration about the minor axis;
- u is the buckling parameter, see **4.3.6.8**;
- β_W is the ratio defined in **4.3.6.9**.

The slenderness factor v may be obtained from Table 19, depending on the value of the ratio λ/x, where x is the torsional index, see **4.3.6.8**, and the flange ratio η. For sections with equal flanges the flange ratio η should be taken as 0.5. Otherwise η should be obtained from:

$$\eta = \frac{I_{yc}}{I_{yc} + I_{yt}}$$

where

- I_{yc} is the second moment of area of the compression flange about the minor axis of the section;
- I_{yt} is the second moment of area of the tension flange about the minor axis of the section.

Alternatively, v may be determined from the following:

— *for an I-, H- or channel section with equal flanges:*

$$v = \frac{1}{[1 + 0.05(\lambda/x)^2]^{0.25}}$$

— *for an I- or H-section with unequal flanges:*

$$v = \frac{1}{\left[\left(4\eta(1-\eta) + 0.05(\lambda/x)^2 + \psi^2\right)^{0.5} + \psi\right]^{0.5}}$$

in which ψ is the monosymmetry index.

The monosymmetry index ψ may be obtained from **B.2.4.1**. Alternatively, for values of η satisfying $0.1 \leq \eta \leq 0.9$ the monosymmetry index ψ may be approximated as follows:

— for a plain I- or H-section:

$$\psi = k_\eta(2\eta - 1)$$

— for a lipped I-section of depth D with compression flange lips of depth D_L, (see Figure 10):

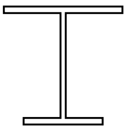
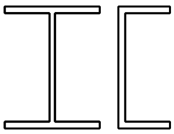
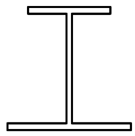
$$\psi = k_\eta(2\eta - 1)(1 + 0.5D_L/D)$$

in which $k_\eta = 0.8$ when $\eta > 0.5$ and $k_\eta = 1.0$ when $\eta < 0.5$.

b) *For a channel section member* the method for an I- or H-section specified in a) may be used if the details of the supports, end restraints, intermediate restraints and connections of the channel to other members that apply load to it are such that the lines of action of the loads and support reactions can be taken as passing through its shear centre, even though this is located outside the back of its cross-section.

c) *For a box section member* (including an RHS with a value of L_E/r_y that exceeds the value given in Table 15) the equivalent slenderness λ_{LT} should be obtained using **B.2.6**.

Table 19 — Slenderness factor *v* for sections with two plain flanges

λ/x	Unequal flanges, larger flange in compression  Compression Tension				Equal flanges 	Unequal flanges, smaller flange in compression Compression Tension 			
	η				η	η			
	0.9	**0.8**	**0.7**	**0.6**	**0.5**	**0.4**	**0.3**	**0.2**	**0.1**
0.5	0.81	0.84	0.88	0.93	1.00	1.11	1.28	1.57	2.20
1.0	0.80	0.83	0.87	0.92	0.99	1.10	1.27	1.53	2.11
1.5	0.80	0.82	0.86	0.91	0.97	1.08	1.24	1.48	1.98
2.0	0.78	0.81	0.85	0.89	0.96	1.06	1.20	1.42	1.84
2.5	0.77	0.80	0.83	0.88	0.93	1.03	1.16	1.35	1.70
3.0	0.76	0.78	0.82	0.86	0.91	1.00	1.12	1.29	1.57
3.5	0.74	0.77	0.80	0.84	0.89	0.97	1.07	1.22	1.46
4.0	0.73	0.75	0.78	0.82	0.86	0.94	1.03	1.16	1.36
4.5	0.71	0.73	0.76	0.80	0.84	0.91	0.99	1.11	1.27
5.0	0.70	0.72	0.75	0.78	0.82	0.88	0.95	1.05	1.20
5.5	0.68	0.70	0.73	0.76	0.79	0.85	0.92	1.01	1.13
6.0	0.67	0.69	0.71	0.74	0.77	0.82	0.89	0.97	1.07
6.5	0.65	0.67	0.70	0.72	0.75	0.80	0.86	0.93	1.02
7.0	0.64	0.66	0.68	0.70	0.73	0.78	0.83	0.89	0.97
7.5	0.63	0.65	0.67	0.69	0.72	0.76	0.80	0.86	0.93
8.0	0.62	0.63	0.65	0.67	0.70	0.74	0.78	0.83	0.89
8.5	0.60	0.62	0.64	0.66	0.68	0.72	0.76	0.80	0.86
9.0	0.59	0.61	0.63	0.64	0.67	0.70	0.74	0.78	0.83
9.5	0.58	0.60	0.61	0.63	0.65	0.68	0.72	0.76	0.80
10.0	0.57	0.59	0.60	0.62	0.64	0.67	0.70	0.74	0.78
11.0	0.55	0.57	0.58	0.60	0.61	0.64	0.67	0.70	0.73
12.0	0.54	0.55	0.56	0.58	0.59	0.61	0.64	0.66	0.70
13.0	0.52	0.53	0.54	0.56	0.57	0.59	0.61	0.64	0.66
14.0	0.51	0.52	0.53	0.54	0.55	0.57	0.59	0.61	0.63
15.0	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59	0.61
16.0	0.48	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59
17.0	0.47	0.48	0.49	0.49	0.50	0.52	0.53	0.55	0.57
18.0	0.46	0.47	0.47	0.48	0.49	0.50	0.52	0.53	0.55
19.0	0.45	0.46	0.46	0.47	0.48	0.49	0.50	0.52	0.53
20.0	0.44	0.45	0.45	0.46	0.47	0.48	0.49	0.50	0.51

NOTE For T-sections see **B.2.8**.

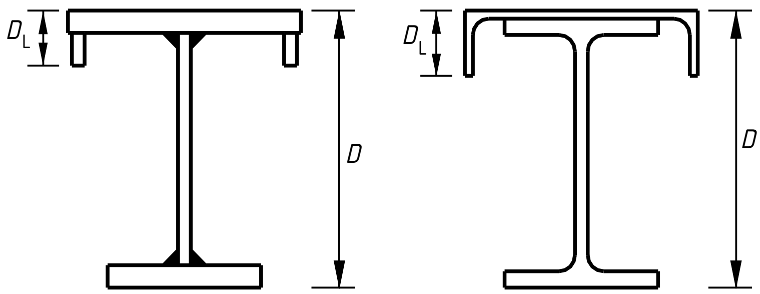


Figure 10 — Examples of lipped I-sections with compression flange lips

4.3.6.8 ***Buckling parameter** u **and torsional index** x*

The buckling parameter u and torsional index x should be obtained from either:

— the formulae given in **B.2.3** (or tables for rolled sections based on these formulae);

— the following pairs of conservative approximations:

— *for rolled I-, H- or channel sections with equal flanges:*

$x = D/T$ used with $u = 0.9$

— *for welded three-plate girders with equal flanges:*

$x = D/T$ used with $u = 1.0$

where

D is the depth of the section;

T is the flange thickness.

4.3.6.9 ***Ratio** β_W*

The ratio β_W should be taken as follows:

— *for class 1 plastic or class 2 compact cross-sections:*

$\beta_W = 1.0$;

— *for class 3 semi-compact cross-sections:*

— if $M_b = p_b Z_x$ in **4.3.6.4**: $\beta_W = Z_x/S_x$;

— if $M_b = p_b S_{x,eff}$ in **4.3.6.4**: $\beta_W = S_{x,eff}/S_x$;

— *for class 4 slender cross-sections:*

$\beta_W = Z_{x,eff}/S_x$.

4.3.7 Equal flanged rolled sections

As a simple (but more conservative) alternative to **4.3.6.5**, **4.3.6.6**, **4.3.6.7**, **4.3.6.8** and **4.3.6.9**, the buckling resistance moment M_b of a plain rolled I, H or channel section with equal flanges may be determined using the bending strength p_b obtained from Table 20 for the relevant values of $(\beta_W)^{0.5}L_E/r_y$ and D/T as follows:

— *for a class 1 plastic or class 2 compact cross-section:*

$M_b = p_b S_x$

— *for a class 3 semi-compact cross-section:*

$M_b = p_b Z_x$

where

- D is the depth of the section;
- L_E is the effective length from **4.3.5**;
- r_y is the radius of gyration of the section about its y-y axis;
- S_x is the plastic modulus about the major axis;
- T is the flange thickness;
- Z_x is the section modulus about the major axis;
- β_W is the ratio specified in **4.3.6.9**.

4.3.8 Buckling resistance moment for single angles

4.3.8.1 *General*

The design of unrestrained single angle members to resist bending should take account of the fact that the rectangular axes of the cross-section (x-x and y-y) are not the principal axes, either by using the basic method given in **4.3.8.2** or the simplified method given in **4.3.8.3**.

4.3.8.2 *Basic method*

For this method the applied moments should be resolved into moments about the principal axes u-u and v-v. The buckling resistance moment M_b for bending about the u-u axis should be based on the value of λ_{LT} obtained from **B.2.9**. The effects of biaxial bending should then be combined in accordance with **4.9**.

4.3.8.3 *Simplified method*

Alternatively to **4.3.8.2**, for equal angles the buckling resistance moment of a single angle with $b/t \leq 15\varepsilon$ subject to bending about the x-x axis, may be determined as follows:

— heel of angle in compression:

$M_b = 0.8 p_y Z_x$

— heel of angle in tension:

$$M_b = p_y Z_x \left(\frac{1\,350\varepsilon - L_E/r_v}{1\,625\varepsilon} \right) \quad \text{but} \quad M_b \leq 0.8 p_y Z_x$$

where

- L_E is the effective length from **4.3.5**, based on the length L_v between restraints against buckling about the v-v axis;
- r_v is the radius of gyration about the v-v axis;
- Z_x is the smaller section modulus about the x-x axis.

If the member is bent with the heel of the angle in tension anywhere within the length L_v between restraints against buckling about the v-v axis, the relevant value of M_b should be applied throughout that segment.

For unequal angles the basic method given in **4.3.8.2** should be used.

Table 20 — Bending strength p_b (N/mm²) for rolled sections with equal flanges

1) Grade S 275 steel, thickness ≤ 16 mm — design strength p_y = 275 N/mm²

$(\beta_W)^{0.5}L_E/r_y$	D/T									
	5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
30	275	275	275	275	275	275	275	275	275	275
35	275	275	275	275	275	275	275	275	275	275
40	275	275	275	275	274	273	272	272	272	272
45	275	275	269	266	264	263	263	262	262	262
50	275	269	261	257	255	253	253	252	252	251
55	275	263	254	248	246	244	243	242	241	241
60	275	258	246	240	236	234	233	232	231	230
65	275	252	239	232	227	224	223	221	221	220
70	274	247	232	223	218	215	213	211	210	209
75	271	242	225	215	209	206	203	201	200	199
80	268	237	219	208	201	196	193	191	190	189
85	265	233	213	200	193	188	184	182	180	179
90	262	228	207	193	185	179	175	173	171	169
95	260	224	201	186	177	171	167	164	162	160
100	257	219	195	180	170	164	159	156	153	152
105	254	215	190	174	163	156	151	148	146	144
110	252	211	185	168	157	150	144	141	138	136
115	250	207	180	162	151	143	138	134	131	129
120	247	204	175	157	145	137	132	128	125	123
125	245	200	171	152	140	132	126	122	119	116
130	242	196	167	147	135	126	120	116	113	111
135	240	193	162	143	130	121	115	111	108	106
140	238	190	159	139	126	117	111	106	103	101
145	236	186	155	135	122	113	106	102	99	96
150	233	183	151	131	118	109	102	98	95	92
155	231	180	148	127	114	105	99	94	91	88
160	229	177	144	124	111	101	95	90	87	84
165	227	174	141	121	107	98	92	87	84	81
170	225	171	138	118	104	95	89	84	81	78
175	223	169	135	115	101	92	86	81	78	75
180	221	166	133	112	99	89	83	78	75	72
185	219	163	130	109	96	87	80	76	72	70
190	217	161	127	107	93	84	78	73	70	67
195	215	158	125	104	91	82	76	71	68	65
200	213	156	122	102	89	80	74	69	65	63
210	209	151	118	98	85	76	70	65	62	59
220	206	147	114	94	81	72	66	62	58	55
230	202	143	110	90	78	69	63	58	55	52
240	199	139	106	87	74	66	60	56	52	50
250	195	135	103	84	72	63	57	53	50	47

Table 20 — Bending strength p_b (N/mm²) for rolled sections with equal flanges (*continued*)

2) Grade S 275 steel, thickness > 16 mm ≤ 40 mm — design strength p_y = 265 N/mm²										
$(\beta_w)^{0.5}L_E/r_y$	*D/T*									
	5	10	15	20	25	30	35	40	45	50
30	265	265	265	265	265	265	265	265	265	265
35	265	265	265	265	265	265	265	265	265	265
40	265	265	265	265	265	264	264	264	263	263
45	265	265	261	258	256	255	254	254	254	254
50	265	261	253	249	247	246	245	244	244	244
55	265	255	246	241	238	236	235	235	234	234
60	265	250	239	233	229	227	226	225	224	224
65	265	245	232	225	221	218	216	215	214	214
70	265	240	225	217	212	209	207	205	204	204
75	263	235	219	210	204	200	198	196	195	194
80	260	230	213	202	196	191	189	187	185	184
85	257	226	207	195	188	183	180	178	176	175
90	254	222	201	188	180	175	171	169	167	166
95	252	217	196	182	173	167	163	160	158	157
100	249	213	190	176	166	160	156	153	150	149
105	247	209	185	170	160	153	148	145	143	141
110	244	206	180	164	154	147	142	138	136	134
115	242	202	176	159	148	140	135	132	129	127
120	240	198	171	154	142	135	129	125	123	121
125	237	195	167	149	137	129	124	120	117	115
130	235	191	163	144	132	124	119	114	111	109
135	233	188	159	140	128	119	114	109	106	104
140	231	185	155	136	124	115	109	105	102	99
145	229	182	152	132	120	111	105	101	97	95
150	227	179	148	129	116	107	101	97	93	91
155	225	176	145	125	112	103	97	93	89	87
160	223	173	142	122	109	100	94	89	86	83
165	221	170	139	119	106	97	91	86	83	80
170	219	167	136	116	103	94	88	83	80	77
175	217	165	133	113	100	91	85	80	77	74
180	215	162	130	110	97	88	82	77	74	71
185	213	160	128	108	95	86	79	75	71	69
190	211	157	125	105	92	83	77	73	69	66
195	209	155	123	103	90	81	75	70	67	64
200	207	153	120	101	88	79	73	68	65	62
210	204	148	116	96	84	75	69	64	61	58
220	200	144	112	93	80	71	65	61	58	55
230	197	140	108	89	77	68	62	58	54	52
240	194	136	104	86	74	65	59	55	52	49
250	190	132	101	83	71	63	57	52	49	47

Table 20 — Bending strength p_b (N/mm²) for rolled sections with equal flanges (*continued*)

3) Grade S 355 steel, thickness ≤ 16 mm — design strength p_y = 355 N/mm²										
$(\beta_W)^{0.5}L_E/r_y$	*D/T*									
	5	10	15	20	25	30	35	40	45	50
30	355	355	355	355	355	355	355	355	355	355
35	355	355	355	354	353	353	352	352	352	352
40	355	352	346	342	341	340	339	339	339	338
45	355	344	335	330	328	327	326	325	325	325
50	355	335	324	318	315	313	312	311	311	311
55	354	327	314	306	302	300	298	297	297	296
60	350	319	303	294	289	286	284	283	282	281
65	346	312	293	283	276	273	270	268	267	266
70	341	305	283	271	264	259	256	254	252	251
75	337	298	274	260	251	246	242	240	238	236
80	333	291	265	249	239	233	229	226	224	222
85	329	284	256	238	228	221	216	213	210	209
90	326	278	247	228	217	209	204	200	198	196
95	322	271	239	219	206	198	193	189	186	184
100	318	265	231	210	197	188	182	178	175	173
105	315	260	224	202	188	178	172	168	165	162
110	311	254	217	194	179	170	163	159	155	153
115	308	248	210	186	171	162	155	150	147	144
120	305	243	204	180	164	154	147	142	139	136
125	301	238	198	173	157	147	140	135	131	129
130	298	233	192	167	151	141	133	128	125	122
135	295	228	187	161	145	135	127	122	118	116
140	292	223	181	156	140	129	122	117	113	110
145	288	219	176	151	135	124	117	111	108	105
150	285	214	172	146	130	119	112	107	103	100
155	282	210	167	142	126	115	107	102	98	95
160	279	206	163	138	121	111	103	98	94	91
165	276	202	159	134	118	107	100	94	90	87
170	273	198	155	130	114	103	96	91	87	84
175	270	195	152	126	111	100	93	87	83	80
180	268	191	148	123	107	97	89	84	80	77
185	265	188	145	120	104	94	87	81	77	74
190	262	184	142	117	101	91	84	79	75	72
195	259	181	139	114	99	88	81	76	72	69
200	257	178	136	111	96	86	79	74	70	67
210	251	172	130	106	91	81	74	69	65	62
220	246	166	125	102	87	77	70	65	62	59
230	241	161	121	98	83	74	67	62	58	55
240	236	156	116	94	80	70	64	59	55	52
250	231	151	112	90	77	67	61	56	52	50

Table 20 — Bending strength p_b (N/mm²) for rolled sections with equal flanges (*continued*)

4) Grade S 355 steel, thickness > 16 mm ≤ 40 mm — design strength p_y = 345 N/mm²

$(\beta_W)^{0.5}L_E/r_y$	*D/T*									
	5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
30	345	345	345	345	345	345	345	345	345	345
35	345	345	345	345	345	344	344	343	343	343
40	345	344	337	334	332	332	331	331	330	330
45	345	335	327	322	320	319	318	318	317	317
50	345	327	316	311	308	306	305	304	304	303
55	345	319	306	299	295	293	292	291	290	289
60	341	312	296	288	283	280	278	277	276	275
65	337	305	287	277	271	267	264	263	261	261
70	333	298	277	265	258	254	251	249	247	246
75	329	291	268	255	246	241	238	235	233	232
80	325	284	259	244	235	229	225	222	220	218
85	322	278	251	234	224	217	212	209	207	205
90	318	272	243	224	213	206	201	197	195	193
95	314	266	235	215	203	195	190	186	183	181
100	311	260	227	207	194	185	179	175	172	170
105	308	254	220	199	185	176	170	166	163	160
110	304	249	213	191	177	167	161	157	153	151
115	301	244	207	184	169	160	153	148	145	142
120	298	238	201	177	162	152	145	141	137	135
125	294	233	195	171	155	145	138	134	130	127
130	291	229	189	165	149	139	132	127	123	121
135	288	224	184	159	144	133	126	121	117	115
140	285	219	179	154	138	128	121	116	112	109
145	282	215	174	149	133	123	116	110	107	104
150	279	211	170	145	129	118	111	106	102	99
155	276	207	165	140	124	114	107	101	97	94
160	273	203	161	136	120	110	102	97	93	90
165	270	199	157	132	117	106	99	93	90	87
170	268	195	153	129	113	102	95	90	86	83
175	265	192	150	125	110	99	92	87	83	80
180	262	188	147	122	106	96	89	84	80	77
185	259	185	143	119	103	93	86	81	77	74
190	257	182	140	116	101	90	83	78	74	71
195	254	178	137	113	98	88	81	76	72	69
200	251	175	134	110	95	85	78	73	69	66
210	246	170	129	105	91	81	74	69	65	62
220	241	164	124	101	87	77	70	65	61	58
230	237	159	119	97	83	73	66	62	58	55
240	232	154	115	93	79	70	63	58	55	52
250	227	149	111	90	76	67	60	56	52	49

4.4 Plate girders

4.4.1 General

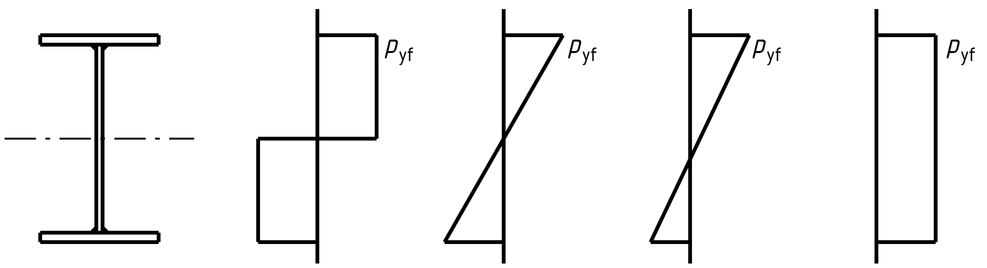
For the design of plate girders, the additional provisions given in **4.4.2**, **4.4.3**, **4.4.4**, **4.4.5** and **4.4.6** should be satisfied, together with the relevant provisions given in **4.2** and **4.3**.

4.4.2 Design strength

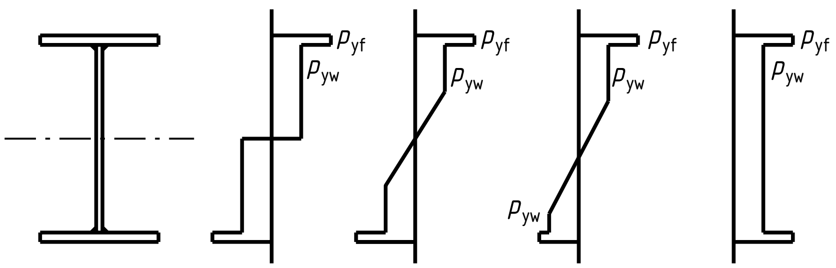
The design strength of the flanges p_{yf} and the design strength of the web p_{yw} should both be determined from **3.1.1**. If $p_{yw} > p_{yf}$ then the design strength of the flanges p_{yf} should always be used when considering moments or axial force, see Figure 11a), but the design strength of the web p_{yw} may optionally be used when considering shear or transverse forces applied to the web.

For a hybrid plate girder (with a web of a lower strength grade than the flanges), the design strength p_{yw} of the web should always be used when considering shear or transverse forces applied to the web, but both design strengths may be taken into account when considering moments or axial force, see Figure 11b).

NOTE The classification of the web should be based on p_{yf}, see Table 11.



a) Web design strength p_{yw} > flange design strength p_{yf}



b) Web design strength p_{yw} < flange design strength p_{yf}

Figure 11 — Cross-sections comprising elements with differing design strengths

4.4.3 Dimensions of webs and flanges

4.4.3.1 *General*

Reference should be made to **3.5** for the classification of webs and compression flanges.

The web thickness should satisfy both **4.4.3.2** and **4.4.3.3**.

4.4.3.2 *Minimum web thickness for serviceability*

To avoid serviceability problems:

a) for webs without intermediate stiffeners: $t \geq d/250$;

b) for webs with transverse stiffeners only:

— where stiffener spacing $a > d$: $t \geq d/250$;

— where stiffener spacing $a \leq d$: $t \geq (d/250)(a/d)^{0.5}$;

c) for webs with longitudinal stiffeners, reference should be made to BS 5400-3.

4.4.3.3 ***Minimum web thickness to avoid compression flange buckling***

To avoid the compression flange buckling into the web:

a) for webs without intermediate stiffeners: $t \geq (d/250)(p_{yf}/345)$;

b) for webs with intermediate transverse stiffeners:

— where stiffener spacing $a > 1.5d$: $t \geq (d/250)(p_{yf}/345)$;

— where stiffener spacing $a \leq 1.5d$: $t \geq (d/250)(p_{yf}/455)^{0.5}$;

where

p_{yf} is the design strength of the compression flange.

4.4.4 Moment capacity

4.4.4.1 ***Web not susceptible to shear buckling***

If the web depth-to-thickness ratio $d/t \leq 62\varepsilon$ it should be assumed not to be susceptible to shear buckling and the moment capacity of the cross-section should be determined using **4.2.5**.

4.4.4.2 ***Web susceptible to shear buckling***

If the web depth-to-thickness ratio $d/t > 70\varepsilon$ for a rolled section, or 62ε for a welded section, it should be assumed to be susceptible to shear buckling. The moment capacity of the cross-section should be determined taking account of the interaction of shear and moment, see Figure 12, using the following methods:

a) *low shear:*

Provided that the applied shear $F_v \leq 0.6V_w$, where V_w is the simple shear buckling resistance from **4.4.5.2**, the moment capacity should be determined from **4.2.5**.

NOTE The reduction factor ρ starts when F_v exceeds $0.5V_w$ but the resulting reduction in moment capacity is negligible unless F_v exceeds $0.6V_w$.

b) *high shear — "flanges only" method:*

If the applied shear $F_v > 0.6V_w$, but the web is designed for shear only, see **4.4.5**, provided that the flanges are not class 4 slender, a conservative value M_f for the moment capacity may be obtained by assuming that the moment is resisted by the flanges alone, with each flange subject to a uniform stress not exceeding p_{yf} .

c) *high shear — general method:*

If the applied shear $F_v > 0.6V_w$, provided that the applied moment does not exceed the "low-shear" moment capacity given in a), the web should be designed using **H.3** for the applied shear combined with any additional moment beyond the "flanges-only" moment capacity M_f given by b).

4.4.4.3 ***Effects of axial force***

If the member is also subject to an axial force, reference should also be made to **4.8**. The value of M_f in **4.4.4.2**b) should be obtained by assuming that the moment and the axial force are both resisted by the flanges alone, with each flange subject to a uniform stress not exceeding p_{yf} .

4.4.5 Shear buckling resistance

4.4.5.1 ***General***

The shear buckling resistance should be checked if the ratio d/t of the web exceeds 70ε for a rolled section or 62ε for a welded section.

For webs required to carry bending moment and/or axial force in addition to shear, reference should also be made to **4.4.4**.

Webs without intermediate stiffeners should be designed using the simplified method given in **4.4.5.2**.

Webs with intermediate transverse stiffeners should be designed by means of either:

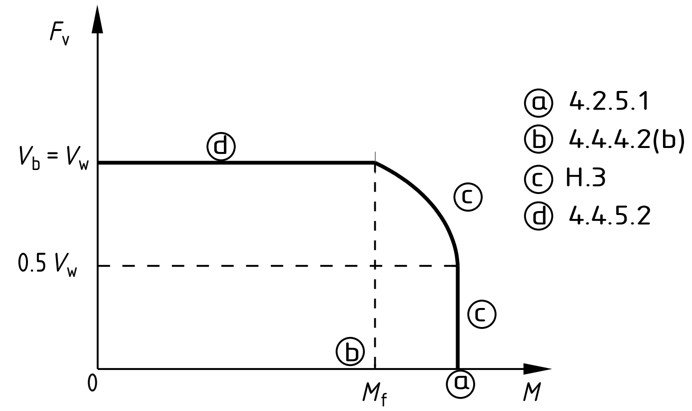
a) the simplified method given in **4.4.5.2**;

b) the more exact method given in **4.4.5.3**;

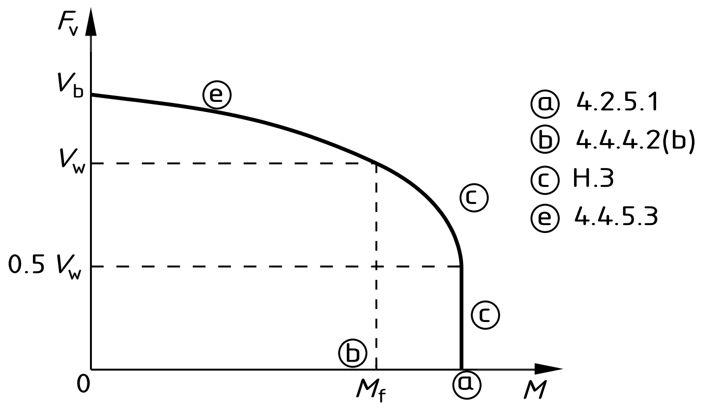
c) reference to BS 5400-3.

In addition, the conditions given in **4.4.5.4** and **4.4.5.5** should be satisfied as appropriate.

Webs with longitudinal stiffeners should be designed by making reference to BS 5400-3.



a) using simplified method for V_b



b) using more exact method for V_b

Figure 12 — Interaction between shear and moment

4.4.5.2 *Simplified method*

The shear buckling resistance V_b of a web with or without intermediate transverse stiffeners may be taken as the simple shear buckling resistance V_w given by:

$$V_b = V_w = dtq_w$$

where

- d is the depth of the web;
- q_w is the shear buckling strength of the web;
- t is the web thickness.

The shear buckling strength q_w should be obtained from **H.1** or from Table 21 depending on the values of d/t and a/d where a is the stiffener spacing. For webs without intermediate stiffeners a/d should be taken as infinity.

4.4.5.3 *More exact method*

Alternatively, the shear buckling resistance V_b of a web panel between two transverse stiffeners may be determined as follows:

— if the flanges of the panel are fully stressed ($f_f = p_{yf}$):

$$V_b = V_w = dtq_w$$

— if the flanges are not fully stressed ($f_f < p_{yf}$):

$$V_b = V_w + V_f \quad \text{but} \quad V_b \le P_v$$

in which V_f is the flange-dependent shear buckling resistance, given by:

$$V_f = \frac{P_v(d/a)[1-(f_f/p_{yf})^2]}{1+0.15(M_{pw}/M_{pf})}$$

where

- f_f is the mean longitudinal stress in the smaller flange due to moment and/or axial force;
- M_{pf} is the plastic moment capacity of the smaller flange, about its own equal area axis perpendicular to the plane of the web, determined using p_{yf};
- M_{pw} is the plastic moment capacity of the web, about its own equal area axis perpendicular to the plane of the web, determined using p_{yw};
- P_v is the shear capacity from **4.2.3**;
- p_{yf} is the design strength of the flange;
- p_{yw} is the design strength of the web.

4.4.5.4 *End anchorage*

End anchorage need not be provided if either of the following conditions apply:

a) the shear capacity, not the shear buckling resistance, is the governing design criterion, indicated by:

$$V_w = P_v$$

b) sufficient shear buckling resistance is available without forming a tension field, indicated by:

$$F_v \le V_{cr}$$

in which V_{cr} is the critical shear buckling resistance from **H.2** or given by the following:

— if $V_w = P_v$ $\quad V_{cr} = P_v$

— if $P_v > V_w > 0.72P_v$ $\quad V_{cr} = (9V_w - 2P_v)/7$

— if $V_w \le 0.72P_v$ $\quad V_{cr} = (V_w/0.9)^2/P_v$

where

- F_v is the maximum shear force;
- V_w is the simple shear buckling resistance from **4.4.5.2**.

In all other cases anchorage should be provided for a longitudinal anchor force H_q representing the longitudinal component of the tension field, as detailed in **H.4**, at:

— the ends of webs without intermediate stiffeners;

— the end panels of webs with intermediate transverse stiffeners.

4.4.5.5 *Panels with openings*

For the design of panels with an opening with any dimension greater than 10 % of the minimum panel dimension, reference should be made to **4.15**. Such panels should not be used as anchor panels and the adjacent panels should be designed as end panels.

Table 21 — Shear buckling strength q_w (N/mm²) of a web

1) Grade S 275 steel, thickness ≤ 16 mm — design strength p_y = 275 N/mm²

d/t	**Stiffener spacing ratio a/d**														
	0.4	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.5**	**3.0**	**∞**
55	165	165	165	165	165	165	165	165	165	165	165	165	165	165	165
60	165	165	165	165	165	165	165	165	165	165	165	165	165	165	165
65	165	165	165	165	165	165	165	165	165	165	165	165	165	165	161
70	165	165	165	165	165	165	165	165	165	165	164	162	160	158	155
75	165	165	165	165	165	165	165	165	163	160	158	156	154	152	148
80	165	165	165	165	165	165	165	162	157	154	152	150	147	146	142
85	165	165	165	165	165	165	162	156	152	149	146	144	141	139	135
90	165	165	165	165	165	163	157	151	146	143	140	138	135	133	128
95	165	165	165	165	165	158	152	146	141	137	134	132	129	127	122
100	165	165	165	165	160	154	147	140	135	131	128	126	123	120	116
105	165	165	165	164	156	149	142	135	129	125	122	120	117	115	110
110	165	165	165	160	152	144	137	130	124	120	117	115	111	109	105
115	165	165	165	156	147	139	132	124	118	114	112	110	106	105	101
120	165	165	163	152	143	135	128	119	113	110	107	105	102	100	96
125	165	165	159	148	139	130	123	114	109	105	103	101	98	96	92
130	165	165	156	145	134	125	118	110	105	101	99	97	94	93	89
135	165	165	152	141	130	121	113	106	101	97	95	93	91	89	86
140	165	162	149	137	126	116	109	102	97	94	92	90	87	86	83
145	165	159	145	133	121	112	105	98	94	91	88	87	84	83	80
150	165	156	142	129	117	109	102	95	91	88	86	84	82	80	77
155	165	153	138	125	113	105	99	92	88	85	83	81	79	78	75
160	165	150	135	121	110	102	96	89	85	82	80	79	76	75	72
165	165	147	131	117	107	99	93	86	82	80	78	76	74	73	70
170	162	144	128	114	103	96	90	84	80	77	75	74	72	71	68
175	160	141	124	110	100	93	87	81	78	75	73	72	70	69	66
180	157	138	121	107	98	90	85	79	76	73	71	70	68	67	64
185	155	135	117	104	95	88	83	77	73	71	69	68	66	65	62
190	152	132	114	102	93	86	80	75	72	69	68	66	64	63	61
195	150	129	111	99	90	84	78	73	70	67	66	65	63	62	59
200	147	126	109	97	88	81	76	71	68	66	64	63	61	60	58
205	145	123	106	94	86	79	75	70	66	64	63	61	60	59	56
210	142	120	103	92	84	78	73	68	65	63	61	60	58	57	55
215	140	117	101	90	82	76	71	66	63	61	60	59	57	56	54
220	137	115	99	88	80	74	70	65	62	60	58	57	56	55	53
225	135	112	97	86	78	72	68	63	60	58	57	56	54	53	51
230	132	110	94	84	76	71	66	62	59	57	56	55	53	52	50
235	130	107	92	82	75	69	65	61	58	56	55	54	52	51	49
240	128	105	90	80	73	68	64	59	57	55	53	52	51	50	48
245	125	103	89	79	72	66	62	58	55	54	52	51	50	49	47
250	123	101	87	77	70	65	61	57	54	53	51	50	49	48	46

Table 21 — Shear buckling strength q_w (N/mm²) of a web (*continued*)

2) Grade S 275 steel, thickness > 16 mm ≤ 40 mm — design strength p_y = 265 N/mm²

d/*t*	Stiffener spacing ratio *a*/*d*													
	0.4	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.5**	**3.0**	**∞**
55	159	159	159	159	159	159	159	159	159	159	159	159	159	159	159
60	159	159	159	159	159	159	159	159	159	159	159	159	159	159	159
65	159	159	159	159	159	159	159	159	159	159	159	159	159	159	157
70	159	159	159	159	159	159	159	159	159	159	159	158	156	154	151
75	159	159	159	159	159	159	159	159	159	156	154	152	150	148	145
80	159	159	159	159	159	159	159	157	153	150	148	147	144	142	138
85	159	159	159	159	159	159	158	152	148	145	143	141	138	136	132
90	159	159	159	159	159	158	153	147	143	139	137	135	132	130	126
95	159	159	159	159	159	154	149	142	137	134	131	129	126	124	120
100	159	159	159	159	156	150	144	137	132	128	126	124	120	118	113
105	159	159	159	159	152	145	139	132	127	123	120	118	114	112	108
110	159	159	159	156	148	141	134	127	121	117	114	112	109	107	103
115	159	159	159	152	144	136	130	122	116	112	110	108	104	103	99
120	159	159	158	149	140	132	125	117	111	108	105	103	100	98	95
125	159	159	155	145	136	127	120	112	107	103	101	99	96	94	91
130	159	159	152	141	132	123	116	108	103	99	97	95	92	91	87
135	159	159	148	137	127	119	111	104	99	96	93	92	89	87	84
140	159	158	145	134	123	114	107	100	95	92	90	88	86	84	81
145	159	155	142	130	119	110	104	97	92	89	87	85	83	81	78
150	159	152	139	126	115	107	100	93	89	86	84	82	80	79	76
155	159	149	135	122	111	103	97	90	86	83	81	80	77	76	73
160	159	147	132	119	108	100	94	87	83	81	79	77	75	74	71
165	159	144	129	115	105	97	91	85	81	78	76	75	73	72	69
170	158	141	125	112	102	94	88	82	78	76	74	73	71	69	67
175	156	138	122	108	99	91	86	80	76	74	72	71	69	67	65
180	153	135	119	105	96	89	83	78	74	72	70	69	67	66	63
185	151	132	115	102	93	86	81	76	72	70	68	67	65	64	61
190	149	129	112	100	91	84	79	74	70	68	66	65	63	62	60
195	146	127	109	97	89	82	77	72	68	66	65	63	62	61	58
200	144	124	107	95	86	80	75	70	67	65	63	62	60	59	57
205	141	121	104	92	84	78	73	68	65	63	61	60	59	58	55
210	139	118	102	90	82	76	71	67	64	61	60	59	57	56	54
215	137	115	99	88	80	74	70	65	62	60	59	58	56	55	53
220	134	112	97	86	78	73	68	64	61	59	57	56	55	54	52
225	132	110	95	84	77	71	67	62	59	57	56	55	53	52	50
230	130	108	93	82	75	70	65	61	58	56	55	54	52	51	49
235	127	105	91	81	73	68	64	60	57	55	54	53	51	50	48
240	125	103	89	79	72	67	63	58	56	54	52	52	50	49	47
245	123	101	87	77	70	65	61	57	54	53	51	50	49	48	46
250	120	99	85	76	69	64	60	56	53	52	50	49	48	47	45

Table 21 — Shear buckling strength q_w (N/mm²) of a web (*continued*)

3) Grade S 355 steel, thickness ≤ 16 mm — design strength p_y = 355 N/mm²															
d/*t*	**Stiffener spacing ratio *a*/*d***														
	0.4	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.5**	**3.0**	**∞**
55	213	213	213	213	213	213	213	213	213	213	213	213	213	213	213
60	213	213	213	213	213	213	213	213	213	213	213	213	209	208	203
65	213	213	213	213	213	213	213	213	212	208	206	204	200	198	193
70	213	213	213	213	213	213	213	209	204	200	197	195	191	189	184
75	213	213	213	213	213	213	209	201	196	191	188	186	182	180	174
80	213	213	213	213	213	209	202	194	187	183	180	177	173	170	164
85	213	213	213	213	211	202	195	186	179	174	171	168	164	161	155
90	213	213	213	213	205	195	187	178	171	166	162	159	155	152	146
95	213	213	213	209	198	189	180	170	163	157	153	151	146	144	138
100	213	213	213	203	192	182	173	162	155	149	146	143	139	137	131
105	213	213	211	197	185	175	165	154	147	142	139	136	132	130	125
110	213	213	206	192	179	168	158	147	140	136	133	130	126	124	119
115	213	213	201	186	173	161	151	141	134	130	127	124	121	119	114
120	213	213	195	180	166	154	145	135	129	124	121	119	116	114	109
125	213	208	190	174	160	148	139	130	124	119	117	115	111	109	105
130	213	204	185	169	154	142	134	125	119	115	112	110	107	105	101
135	213	199	180	163	148	137	129	120	114	111	108	106	103	101	97
140	213	195	175	157	143	132	124	116	110	107	104	102	99	98	94
145	213	190	170	151	138	128	120	112	107	103	101	99	96	94	91
150	209	186	165	146	133	123	116	108	103	100	97	95	93	91	88
155	206	181	159	142	129	119	112	105	100	96	94	92	90	88	85
160	202	177	154	137	125	116	109	101	97	93	91	89	87	85	82
165	198	173	150	133	121	112	105	98	94	91	88	87	84	83	80
170	195	168	145	129	117	109	102	95	91	88	86	84	82	80	77
175	191	164	141	125	114	106	99	93	88	85	83	82	79	78	75
180	187	159	137	122	111	103	97	90	86	83	81	80	77	76	73
185	184	155	133	119	108	100	94	88	83	81	79	77	75	74	71
190	180	151	130	115	105	97	91	85	81	79	77	75	73	72	69
195	176	147	127	113	102	95	89	83	79	77	75	73	71	70	67
200	173	143	123	110	100	93	87	81	77	75	73	72	70	68	66
205	169	140	120	107	97	90	85	79	75	73	71	70	68	67	64
210	165	136	117	104	95	88	83	77	74	71	69	68	66	65	63
215	162	133	115	102	93	86	81	75	72	69	68	67	65	64	61
220	158	130	112	100	91	84	79	74	70	68	66	65	63	62	60
225	155	127	110	98	89	82	77	72	69	66	65	64	62	61	58
230	151	124	107	95	87	80	76	70	67	65	63	62	60	59	57
235	148	122	105	93	85	79	74	69	66	64	62	61	59	58	56
240	145	119	103	91	83	77	72	67	64	62	61	60	58	57	55
245	142	117	101	90	82	76	71	66	63	61	59	58	57	56	54
250	139	115	99	88	80	74	70	65	62	60	58	57	56	55	53

Table 21 — Shear buckling strength q_w (N/mm²) of a web *(continued)*

4) Grade S 355 steel, thickness > 16 mm ≤ 40 mm — design strength p_y = 345 N/mm²															
d/*t*	**Stiffener spacing ratio** *a*/*d*														
	0.4	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.5**	**3.0**	**∞**
55	207	207	207	207	207	207	207	207	207	207	207	207	207	207	207
60	207	207	207	207	207	207	207	207	207	207	207	207	205	203	199
65	207	207	207	207	207	207	207	207	207	204	201	200	196	194	190
70	207	207	207	207	207	207	207	205	200	196	193	191	187	185	180
75	207	207	207	207	207	207	205	197	192	188	185	183	179	177	171
80	207	207	207	207	207	205	198	190	184	180	176	174	170	168	162
85	207	207	207	207	206	198	191	182	176	172	168	165	161	159	153
90	207	207	207	207	200	192	184	175	168	163	160	157	152	150	144
95	207	207	207	205	194	185	177	167	160	155	151	149	144	142	136
100	207	207	207	199	188	178	170	160	152	147	144	141	137	135	129
105	207	207	206	194	182	172	163	152	145	140	137	134	131	128	123
110	207	207	202	188	176	165	156	145	138	134	131	128	125	123	118
115	207	207	197	182	170	159	149	139	132	128	125	123	119	117	113
120	207	207	192	177	164	152	143	133	127	123	120	118	114	112	108
125	207	204	187	171	158	146	137	128	122	118	115	113	110	108	104
130	207	200	182	166	152	140	132	123	117	113	111	109	105	104	100
135	207	195	177	160	146	135	127	118	113	109	106	105	102	100	96
140	207	191	172	155	141	130	122	114	109	105	103	101	98	96	92
145	207	187	167	149	136	126	118	110	105	102	99	97	95	93	89
150	205	183	162	144	131	122	114	106	102	98	96	94	91	90	86
155	201	178	157	140	127	118	111	103	98	95	93	91	88	87	84
160	198	174	152	135	123	114	107	100	95	92	90	88	86	84	81
165	194	170	147	131	119	111	104	97	92	89	87	86	83	82	78
170	191	165	143	127	116	107	101	94	90	87	85	83	81	79	76
175	187	161	139	124	113	104	98	91	87	84	82	81	78	77	74
180	184	157	135	120	109	101	95	89	85	82	80	78	76	75	72
185	180	153	131	117	106	99	93	86	82	80	78	76	74	73	70
190	177	149	128	114	104	96	90	84	80	77	76	74	72	71	68
195	173	145	125	111	101	94	88	82	78	76	74	72	70	69	66
200	170	141	122	108	98	91	86	80	76	74	72	71	69	67	65
205	166	138	119	106	96	89	84	78	74	72	70	69	67	66	63
210	163	134	116	103	94	87	82	76	73	70	68	67	65	64	62
215	159	131	113	101	92	85	80	74	71	68	67	66	64	63	60
220	156	128	111	98	90	83	78	73	69	67	65	64	62	61	59
225	152	125	108	96	88	81	76	71	68	65	64	63	61	60	58
230	149	123	106	94	86	79	74	69	66	64	62	61	60	59	56
235	146	120	103	92	84	78	73	68	65	63	61	60	58	57	55
240	143	118	101	90	82	76	71	67	63	61	60	59	57	56	54
245	140	115	99	88	80	74	70	65	62	60	59	58	56	55	53
250	137	113	97	87	79	73	69	64	61	59	57	56	55	54	52

4.4.6 Design of intermediate transverse web stiffeners

4.4.6.1 *General*

Intermediate transverse stiffeners may be provided on either one or both sides of the web.

4.4.6.2 *Spacing*

Where intermediate transverse web stiffeners are provided, their spacing should conform to **4.4.3**.

4.4.6.3 *Outstand of stiffeners*

The outstand of the stiffeners should conform to **4.5.1.2**.

4.4.6.4 *Minimum stiffness*

Intermediate transverse web stiffeners not subject to external loads or moments should have a second moment of area I_s about the centreline of the web not less than I_s given by:

for $a/d \geq \sqrt{2}$: $$I_s = 0.75 d t_{min}^3$$

for $a/d < \sqrt{2}$: $$I_s = 1.5(d/a)^2 d t_{min}^3$$

where

a is the actual stiffener spacing;

d is the depth of the web;

t_{min} is the minimum required web thickness for the actual stiffener spacing a.

4.4.6.5 *Additional stiffness for external loading*

If an intermediate transverse web stiffener is subject to externally applied forces, the value of I_s given in **4.4.6.4** should be increased by adding I_{ext} as follows:

a) for transverse forces effectively applied in line with the web:

$I_{ext} = 0$ (i.e. no increase in I_s)

b) for transverse forces applied eccentric to the web:

$$I_{ext} = F_x e_x D^2/Et$$

c) for lateral forces, deemed to be applied at the level of the compression flange of the girder:

$$I_{ext} = 2F_h D^3/Et$$

where

D is the overall depth of the section;

E is the modulus of elasticity;

e_x is the eccentricity of the transverse force from the centreline of the web;

F_h is the external lateral force;

F_x is the external transverse force;

t is the actual web thickness.

4.4.6.6 *Buckling resistance*

Intermediate transverse web stiffeners not subject to external forces or moments should meet the condition:

$$F_q \leq P_q$$

in which F_q is the larger value, considering the two web panels each side of the stiffener, of the compressive axial force given by:

$$F_q = V - V_{cr}$$

where

P_q is the buckling resistance of the intermediate web stiffener, from **4.5.5**;

V is the shear in a web panel adjacent to the stiffener;

V_{cr} is the critical shear buckling resistance [see **4.4.5.4**b)] of the same web panel.

Intermediate transverse web stiffeners subject to external forces or moments should meet the conditions for load carrying web stiffeners given in **4.5.3.3**. In addition, they should also satisfy the following:

— if $F_q > F_x$:

$$\frac{F_q - F_x}{P_q} + \frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \le 1$$

— if $F_q \le F_x$:

$$\frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \le 1$$

in which

$M_s = F_x e_x + F_h D$

where

F_h is the external lateral force, if any, see **4.4.6.5**;

F_x is the external transverse force;

M_{ys} is the moment capacity of the stiffener based on its section modulus;

P_x is the buckling resistance of a load carrying stiffener, see **4.5.3.3**.

4.4.6.7 *Connection to web of intermediate stiffeners*

Intermediate transverse web stiffeners that are not subject to external forces or moments should be connected to the web to withstand a shear between each component and the web (in kN per millimetre run) of not less than:

$t^2/(5b_s)$

where

b_s is the outstand of the stiffener (in mm);

t is the web thickness (in mm).

If the stiffeners are subject to external forces or moments, the resulting shear between the web and the stiffener should be added to the above value.

Intermediate transverse web stiffeners that are not subject to external forces or moments should extend to the compression flange, but need not be connected to it. Intermediate transverse web stiffeners that are not subject to external forces or moments may terminate clear of the tension flange. In such cases the welds connecting the stiffener to the web should terminate not more than $4t$ clear of the tension flange.

4.5 Web bearing capacity, buckling resistance and stiffener design

4.5.1 General

4.5.1.1 *Web stiffeners*

Web stiffeners should be provided where needed at locations where unstiffened webs are subject to local loads or reactions, as follows:

a) *bearing stiffeners*, to prevent crushing of the web due to forces applied through a flange, see **4.5.2**;

b) *load carrying stiffeners*, to resist web buckling due to concentrated loading, see **4.5.3**;

c) *tension stiffeners*, to transmit tensile forces applied via a flange into the web, see **4.5.4**;

d) *intermediate transverse web stiffeners*, to resist web buckling due to shear, see **4.5.5**;

e) *diagonal stiffeners*, to provide local reinforcement of a web in shear, see **4.5.6**;

f) *torsion stiffeners*, to provide torsional restraint at supports, see **4.5.7**.

If the same stiffeners have more than one function, they should meet the requirements for each function.

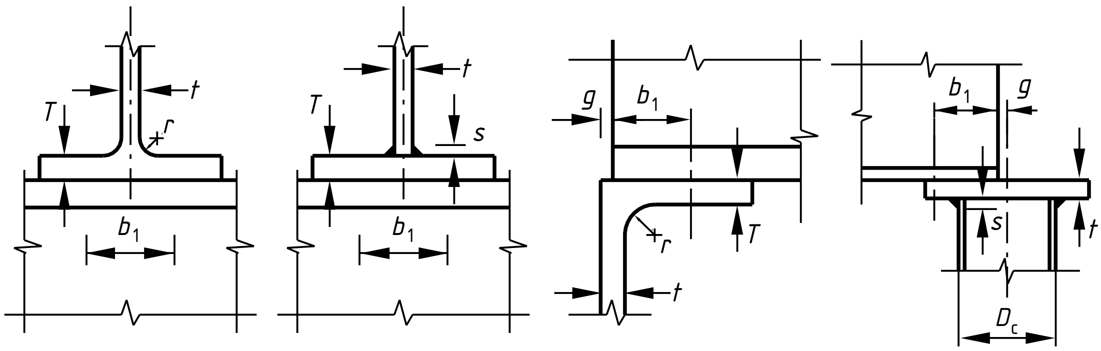
4.5.1.2 *Maximum outstand of web stiffeners*

Unless the outer edge of a web stiffener is itself continuously stiffened, its outstand from the face of the web should not exceed $19\varepsilon t_s$.

If the outstand of a stiffener is between $13\varepsilon t_s$ and $19\varepsilon t_s$ then its design should be based on an effective cross-section with an outstand of $13\varepsilon t_s$.

4.5.1.3 *Stiff bearing length*

The stiff bearing length b_1 should be taken as the length of support that cannot deform appreciably in bending. To determine b_1 the dispersion of load through a steel bearing should be taken as indicated in Figure 13. Dispersion at 45° through packs may be included provided that they are firmly fixed in place.



$b_1 = t + 1.6r + 2T$ $b_1 = t + 1.6s + 2T$ $b_1 = t + T + 0.8r - g$ $b_1 = 0.5D_c + t + 0.8s - g$

Figure 13 — Stiff bearing length

4.5.1.4 *Eccentricity*

Where a load or reaction is applied eccentric from the centreline of the web, or where the centroid of the stiffener does not lie on the centreline of the web, the resulting eccentricity of loading should be allowed for in design.

4.5.1.5 *Hollow sections*

Where concentrated loads are applied to hollow sections consideration should be given to local stresses and deformations and the section reinforced or stiffened as necessary.

NOTE Details of a design procedure for resistance to loads or reactions applied to webs of hollow sections through a flange are given in reference [5], see Bibliography.

4.5.2 Bearing capacity of web

4.5.2.1 *Unstiffened web*

Bearing stiffeners should be provided where the local compressive force F_x applied through a flange by loads or reactions exceeds the bearing capacity P_{bw} of the unstiffened web at the web-to-flange connection, given by:

$$P_{bw} = (b_1 + nk)tp_{yw}$$

in which:

— except at the end of a member:

$n = 5$

— at the end of a member:

$n = 2 + 0.6b_e/k$ but $n \leq 5$

and k is obtained as follows:

— for a rolled I- or H-section: $k = T + r$

— for a welded I- or H-section: $k = T$

where

b_1 is the stiff bearing length, see **4.5.1.3**;

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

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

r is the root radius;

T is the flange thickness:

t is the web thickness.

4.5.2.2 *Stiffened web*

Bearing stiffeners should be designed for the applied force F_x minus the bearing capacity P_{bw} of the unstiffened web. The capacity P_s of the stiffener should be obtained from:

$$P_s = A_{s.net}p_y$$

in which $A_{s.net}$ is the net cross-sectional area of the stiffener, allowing for cope holes for welding.

If the web and the stiffener have different design strengths, the smaller value should be used to calculate both the web capacity P_{bw} and the stiffener capacity P_s.

4.5.3 Buckling resistance

4.5.3.1 *Unstiffened web*

Load carrying web stiffeners should be provided where the local compressive force F_x applied through a flange by a load or reaction exceeds the buckling resistance of the web.

If the flange through which the load or reaction is applied is effectively restrained against both:

a) rotation relative to the web;

b) lateral movement relative to the other flange;

then provided that the distance a_e from the load or reaction to the nearer end of the member is at least $0.7d$, the buckling resistance of the unstiffened web should be taken as P_x given by:

$$P_x = \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$

where

d is the depth of the web;

P_{bw} is the bearing capacity of the unstiffened web at the web-to-flange connection, from **4.5.2.1**

and b_1, k, n and t are as defined in **4.5.2.1**.

If the distance a_e from the load or reaction to the nearer end of the member is less than $0.7d$, the buckling resistance P_x of the web should be taken as:

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$

Where a) or b) is not met, the buckling resistance of the web should be reduced to P_{xr} given by:

$$P_{xr} = \frac{0.7d}{L_E} P_x$$

in which L_E is the effective length of the web, acting as a compression member or a part of a compression member, determined in accordance with **4.7.2** for the appropriate conditions of end restraint.

4.5.3.2 *Loads applied between stiffeners*

Load carrying web stiffeners should be added where the local compressive stress f_{ed} on the compression edge of a web, due to loads or reactions applied through a flange between the web stiffeners already provided, exceeds the compressive strength for edge loading p_{ed}.

For this check, the stress f_{ed} on the compression edge of a web panel of depth d between two transverse stiffeners of spacing a should be calculated as follows:

a) individual point loads and distributed loads shorter than the smaller panel dimension a or d should be divided by the smaller panel dimension;

b) for a series of similar point loads, equally spaced, divide the largest load by the spacing, or by the smaller panel dimension if this is less;

c) add the intensity (force/unit length) of any other distributed loads;

d) divide the sum of a), b) and c) by the web thickness t.

The compressive strength for edge loading p_{ed} should be calculated as follows:

— if the compression flange is restrained against rotation relative to the web:

$$p_{ed} = \left[2.75 + \frac{2}{(a/d)^2}\right]\frac{E}{(d/t)^2}$$

— if the compression flange is not restrained against rotation relative to the web:

$$p_{ed} = \left[1.0 + \frac{2}{(a/d)^2}\right]\frac{E}{(d/t)^2}$$

4.5.3.3 *Buckling resistance of load carrying stiffeners*

The external load or reaction F_x on a load carrying stiffener should not exceed the buckling resistance P_x of the stiffener, given by:

$$P_x = A_s\, p_c$$

The effective area A_s of the load carrying stiffener should be taken as that of a cruciform cross-section made up from the effective area of the stiffeners themselves (see **4.5.1.2**) together with an effective width of web on each side of the centreline of the stiffeners limited to 15 times the web thickness t.

The compressive strength p_c should be determined from **4.7.5** using strut curve c) (see Table 24) and the radius of gyration of the complete cruciform area A_s of the stiffener about its axis parallel to the web.

The design strength p_y should be taken as the lower value for the web or the stiffeners. The reduction of 20 N/mm² referred to in **4.7.5** should not be applied unless the stiffeners themselves are welded sections.

Provided that the flange through which the load or reaction is applied is effectively restrained against lateral movement relative to the other flange, the effective length L_E should be taken as follows:

a) flange restrained against rotation in the plane of the stiffener by other structural elements:

L_E = 0.7 times the length L of the stiffener clear between flanges;

b) flange not so restrained:

L_E = 1.0 times the length L of the stiffener clear between flanges.

If the load or reaction is applied to the flange by a compression member, then unless effective lateral restraint is provided at that point, the stiffener should be designed as part of the compression member applying the load, and the connection should be checked for the effects of strut action, see **C.3**.

If the stiffener also acts as an intermediate transverse stiffener to resist shear buckling, it should be checked for the effect of combined loads in accordance with **4.4.6.6**.

Load carrying stiffeners should also be checked as bearing stiffeners, see **4.5.2.2**.

4.5.4 Tension stiffeners

Tension stiffeners should be provided where the applied load or reaction exceeds either:

a) the tension capacity of the unstiffened web at its connection to the flange;

b) the tension capacity of the unstiffened flange, see **6.3.4** and **6.7.5**.

The tension capacity P_{tw} of an unstiffened web at the web-to-flange connection should be obtained by dispersion through the flange to the web-to-flange connection at a slope of 1 in 2.5 to the flange.

In case a), a tension stiffener required to strengthen an unstiffened web should be designed to carry that portion of the applied load or reaction that exceeds the tension capacity P_{tw} of the unstiffened web. If the web and the stiffener have different design strengths, the smaller value should be used for both.

In case b), a tension stiffener required to strengthen an unstiffened flange, the proportion of the applied load or reaction assumed to be carried by the stiffener should be consistent with the design of the flange.

4.5.5 Intermediate transverse web stiffeners

The buckling resistance P_q of an intermediate transverse web stiffener should be determined as for the buckling resistance P_x of a load carrying stiffener, see **4.5.3.3**, except that:

— the effective length L_E should be taken as 0.7 times its length L clear between flanges;

— stiffeners required only to resist shear buckling need not be checked as bearing stiffeners to **4.5.2.2**.

4.5.6 Diagonal stiffeners

Diagonal stiffeners should be designed to carry that portion of the total applied shear that exceeds the shear capacity P_v of the member, see **4.2.3**. If the web and the stiffener have different design strengths, the smaller value should be used for both.

4.5.7 Torsion stiffeners

Stiffeners that are required to provide torsional restraint at member supports, see **4.3.3**, should have a second moment of area I_s about the centreline of the web that satisfies the criterion:

$$I_s \geq 0.34\alpha_s D^3 T_c$$

in which the coefficient α_s is given by the following:

— if $\lambda \leq 50$:	α_s	=	0.006
— if $50 < \lambda \leq 100$:	α_s	=	$0.3/\lambda$
— if $\lambda > 100$:	α_s	=	$30/\lambda^2$

where

λ is the slenderness L_E/r_y of the member;

D is the overall depth of the member at the support;

L_E is the effective length of the member in the span under consideration;

r_y is the radius of gyration about the minor axis;

T_c is the maximum thickness of the compression flange in the span under consideration.

4.5.8 Connection of stiffeners to webs

Web stiffeners that contribute to resisting loads or reactions applied through a flange should be connected to the web by welds, fitted bolts or preloaded bolts designed to be non-slip under factored loads, see **6.4.2**. This connection should be designed to transmit a force equal to the lesser of:

a) the larger of the forces applied at either end if they act in opposite directions, or the sum of these forces if both act in the same direction;

b) the capacity of the stiffener, see **4.5.2.2**.

4.5.9 Connection of web stiffeners to flanges

4.5.9.1 *Stiffeners in compression*

Web stiffeners required to resist compression should either be fitted against the loaded flange or connected to it by continuous welds, fitted bolts or preloaded bolts designed to be non-slip under factored loads, see **6.4.2**.

The stiffener should be fitted against, or connected to, both flanges where any of the following apply:

a) a load is applied directly over a support;

b) the stiffener forms the end stiffener of a stiffened web;

c) the stiffener acts as a torsion stiffener.

4.5.9.2 *Stiffeners in tension*

Web stiffeners required to resist tension should be connected to the flange transmitting the load or reaction by continuous welds, fitted bolts or preloaded bolts designed to be non-slip under factored loads, see **6.4.2**. This connection should be designed to resist the lesser of the applied load or reaction or the capacity of the stiffener, see **4.5.2.2**.

4.5.10 Length of web stiffeners

Bearing stiffeners or tension stiffeners that do not also have other functions, see **4.5.1.1**, may be curtailed where the capacity P_{us} of the unstiffened web beyond the end of the stiffener is not less than the proportion of the applied load or reaction carried by the stiffener. The capacity P_{us} of the unstiffened web at this point should be obtained from:

$$P_{us} = (b_1 + w)tp_{yw}$$

where

b_1 is the stiff bearing length, see **4.5.1.3**;

w is the length obtained by dispersion at 45° to the level at which the stiffener terminates.

The length of a stiffener that does not extend right across the web should also be such that the local shear stress in the web due to the force transmitted by the stiffener does not exceed $0.6p_{yw}$.

4.6 Tension members

4.6.1 Tension capacity

The tension capacity P_t of a member should generally be obtained from:

$$P_t = p_y A_e$$

in which A_e is the sum of the effective net areas a_e of all the elements of the cross-section, determined from **3.4.3**, but not more than 1.2 times the total net area A_n.

4.6.2 Members with eccentric connections

If members are connected eccentric to their axes, the resulting moments should generally be allowed for in accordance with **4.8.2**. However, angles, channels or T-sections with eccentric end connections may be treated as axially loaded by using the reduced tension capacity given in **4.6.3**.

4.6.3 Simple tension members

4.6.3.1 *Single angle, channel or T-section members*

For a simple tie, designed as axially loaded, consisting of a single angle connected through one leg only, a single channel connected only through the web or a T-section connected only through the flange, the tension capacity should be obtained as follows:

— for bolted connections: $P_t = p_y(A_e - 0.5a_2)$

— for welded connections: $P_t = p_y(A_g - 0.3a_2)$

in which:

$$a_2 = A_g - a_1$$

where

A_g is the gross cross-sectional area, see **3.4.1**;

a_1 is the gross area of the connected element, taken as the product of its thickness and the overall leg width for an angle, the overall depth for a channel or the flange width for a T-section.

4.6.3.2 ***Double angle, channel or T-section members***

For a simple tie, designed as axially loaded, consisting of two angles connected through one leg only, two channels connected only through the web or two T-sections connected only through the flange, the tension capacity should be obtained as follows:

a) if the tie is connected to both sides of a gusset or section and the components are interconnected by bolts or welds and held apart and longitudinally parallel by battens or solid packing pieces in at least two locations within their length, the tension capacity per component should be obtained from:

— for bolted connections: $P_t = p_y(A_e - 0.25a_2)$

— for welded connections: $P_t = p_y(A_g - 0.15a_2)$

b) if the components are both connected to the same side of a gusset or member, or not interconnected as given in a), the tension capacity per component should be taken as given in **4.6.3.1**.

In case a) the outermost interconnection should be within a distance from each end of ten times the smaller leg length for angle components, or ten times the smaller overall dimension for channels or T-sections.

4.6.3.3 ***Other simple ties***

A simple tie consisting of a single angle connected through both legs by lug angles or otherwise, a single channel connected by both flanges or a T-section connected only through the stem (or both the flange and the stem), should be designed as axially loaded. The tension capacity should be based on the effective net area from **3.4.3**.

4.6.3.4 ***Continuous ties***

The internal bays of continuous ties should be designed as axially loaded. The tension capacity should be based on the effective net area from **3.4.3**.

4.6.4 Laced or battened ties

For laced or battened ties, the lacing or battening systems should be designed to resist the greater of:

a) the axial forces, moments and shear forces induced by eccentric loads, applied moments or transverse forces, including self-weight and wind resistance;

b) the axial forces, moments and shear forces induced by a transverse shear on the complete member at any point in its length equal to 1 % of the axial force in the member, taken as shared equally between all transverse lacing or battening systems in parallel planes.

4.7 Compression members

4.7.1 General

4.7.1.1 ***Segment length***

The segment length L of a compression member in any plane should be taken as the length between the points at which it is restrained against buckling in that plane.

4.7.1.2 ***Restraints***

A restraint should have sufficient strength and stiffness to inhibit movement of the restrained point in position or direction as appropriate. Positional restraints should be connected to an appropriate shear diaphragm or system of triangulated bracing.

Positional restraints to compression members forming the flanges of lattice girders should satisfy the recommendations for lateral restraint of beams specified in **4.3.2**. All other positional restraints to compression members should be capable of resisting a force of not less than 1.0 % of the axial force in the member and transferring it to the adjacent points of positional restraint.

Bracing systems that supply positional restraint to more than one member should be designed to resist the sum of the restraint forces from each member that they restrain, reduced by the factor k_r obtained from:

$$k_r = (0.2 + 1/N_r)^{0.5}$$

in which N_r is the number of parallel members restrained.

4.7.2 Slenderness

The slenderness λ of a compression member should generally be taken as its effective length L_E divided by its radius of gyration r about the relevant axis, except as given in **4.7.9**, **4.7.10** or **4.7.13**.

In the case of a single-angle strut with lateral restraints to its two legs alternately, the slenderness for buckling about every axis should be increased by 20 %.

4.7.3 Effective lengths

Except for angles, channels or T-sections designed in accordance with **4.7.10** the effective length L_E of a compression member should be determined from the segment length L centre-to-centre of restraints or intersections with restraining members in the relevant plane as follows.

a) Generally, in accordance with Table 22, depending on the conditions of restraint in the relevant plane, members carrying more than 90 % of their reduced plastic moment capacity M_r in the presence of axial force (see **I.2**) being taken as incapable of providing directional restraint.

b) For continuous columns in multistorey buildings of simple design, in accordance with Table 22, depending on the conditions of restraint in the relevant plane, directional restraint being based on connection stiffness as well as member stiffness.

c) For compression members in trusses, lattice girders or bracing systems, in accordance with Table 22, depending on the conditions of restraint in the relevant plane.

d) For columns in single storey buildings of simple design, see **D.1**.

e) For columns supporting internal platform floors of simple design, see **D.2**.

f) For columns forming part of a continuous structure, see Annex E.

Table 22 — Nominal effective length L_E for a compression member[a]

a) non-sway mode			
Restraint (in the plane under consideration) by other parts of the structure			L_E
Effectively held in position at both ends	Effectively restrained in direction at both ends		0.7L
	Partially restrained in direction at both ends		0.85L
	Restrained in direction at one end		0.85L
	Not restrained in direction at either end		1.0L
b) sway mode			
One end	Other end		L_E
Effectively held in position and restrained in direction	Not held in position	Effectively restrained in direction	1.2L
		Partially restrained in direction	1.5L
		Not restrained in direction	2.0L
[a] Excluding angle, channel or T-section struts designed in accordance with **4.7.10**.			

4.7.4 Compression resistance

The compression resistance P_c of a member should be obtained from the following:

a) for class 1 plastic, class 2 compact or class 3 semi-compact cross-sections:

$$P_c = A_g p_c$$

b) for class 4 slender cross-sections:

$$P_c = A_{eff} p_{cs}$$

where

A_{eff} is the effective cross-sectional area from **3.6**;

A_g is the gross cross-sectional area, see **3.4.1**;

p_c is the compressive strength, see **4.7.5**;

p_{cs} is the value of p_c from **4.7.5** for a reduced slenderness of $\lambda(A_{eff}/A_g)^{0.5}$ in which λ is based on the radius of gyration r of the gross cross-section.

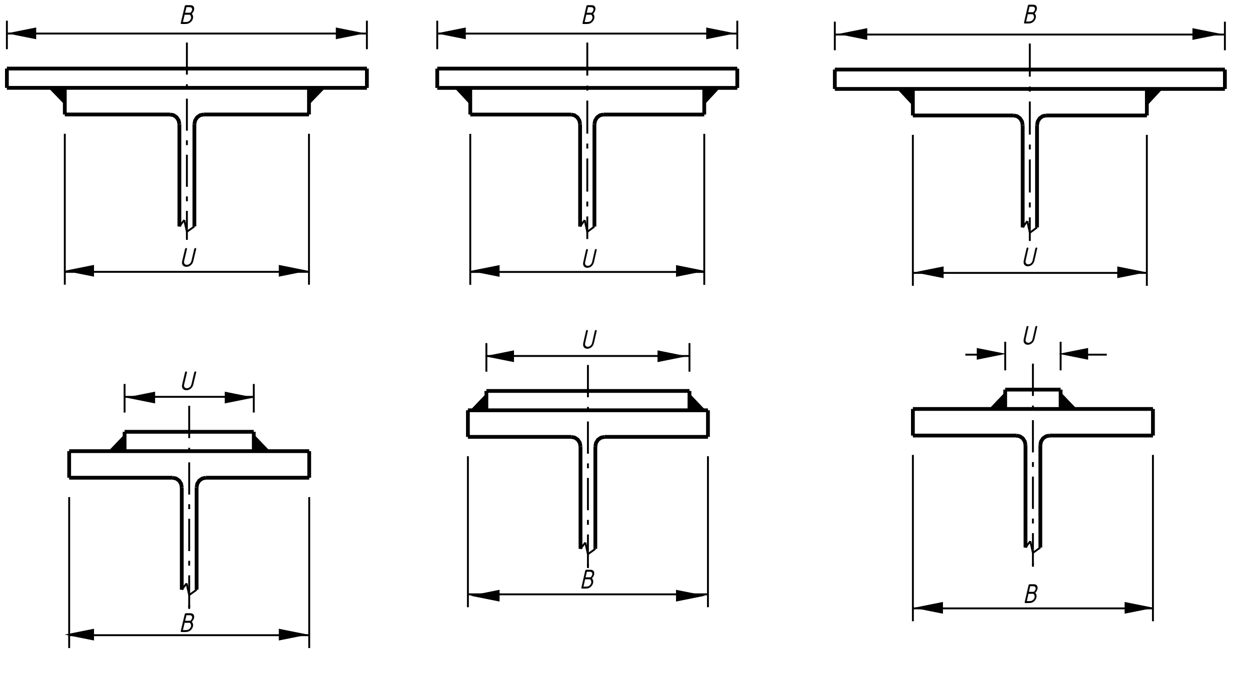
4.7.5 Compressive strength

The compressive strength p_c should be based on the appropriate strut curve for buckling about the relevant axis from Table 23 and Figure 14, depending on the type of cross-section and the maximum thickness.

The value of p_c for the appropriate strut curve should be obtained from Table 24, depending on the design strength p_y and the slenderness λ for buckling about the relevant axis, or from the formula given in **C.1**.

For welded I, H or box sections p_c should be obtained from Table 24 using a p_y value 20 N/mm^2 below that obtained from **3.1.1**, or by using this reduced value of p_y in the formula given in **C.1**.

NOTE This reduced p_y value applies only when using Table 24 or the formula given in **C.1**.



a) $0.25 < U/B < 0.8$ b) $U/B \geq 0.8$ c) $U/B \leq 0.25$

Figure 14 — Rolled I- or H-section with welded flange plates

Table 23 — Allocation of strut curve

Type of section	Maximum thickness (see note 1)	Axis of buckling	
		x-x	y-y
Hot-finished structural hollow section		a)	a)
Cold-formed structural hollow section		c)	c)
Rolled I-section	≤ 40 mm	a)	b)
	> 40 mm	b)	c)
Rolled H-section	≤ 40 mm	b)	c)
	> 40 mm	c)	d)
Welded I or H-section (see note 2 and **4.7.5**)	≤ 40 mm	b)	c)
	> 40 mm	b)	d)
Rolled I-section with welded flange cover plates with 0.25 < *U*/*B* < 0.8 as shown in Figure 14a)	≤ 40 mm	a)	b)
	> 40 mm	b)	c)
Rolled H-section with welded flange cover plates with 0.25 < *U*/*B* < 0.8 as shown in Figure 14a)	≤ 40 mm	b)	c)
	> 40 mm	c)	d)
Rolled I or H-section with welded flange cover plates with *U*/*B* ≥ 0.8 as shown in Figure 14b)	≤ 40 mm	b)	a)
	> 40 mm	c)	b)
Rolled I or H-section with welded flange cover plates with *U*/*B* ≤ 0.25 as shown in Figure 14c)	≤ 40 mm	b)	c)
	> 40 mm	b)	d)
Welded box section (see note 3 and **4.7.5**)	≤ 40 mm	b)	b)
	> 40 mm	c)	c)
Round, square or flat bar	≤ 40 mm	b)	b)
	> 40 mm	c)	c)
Rolled angle, channel or T-section Two rolled sections laced, battened or back-to-back Compound rolled sections		Any axis: c)	

NOTE 1 For thicknesses between 40 mm and 50 mm the value of p_c may be taken as the average of the values for thicknesses up to 40 mm and over 40 mm for the relevant value of p_y.

NOTE 2 For welded I or H-sections with their flanges thermally cut by machine without subsequent edge grinding or machining, for buckling about the y-y axis, strut curve b) may be used for flanges up to 40 mm thick and strut curve c) for flanges over 40 mm thick.

NOTE 3 The category "welded box section" includes any box section fabricated from plates or rolled sections, provided that all of the longitudinal welds are near the corners of the cross-section. Box sections with longitudinal stiffeners are NOT included in this category.

Table 24 — Compressive strength p_c (N/mm²)

1) Values of p_c in N/mm² with λ < 110 for strut curve a															
λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	**245**	**255**	**265**	**275**	**315**	**325**	**335**	**345**	**355**	**400**	**410**	**430**	**440**	**460**
15	235	245	255	265	275	315	325	335	345	355	399	409	429	439	458
20	234	244	254	264	273	312	322	332	342	351	395	405	424	434	453
25	232	241	251	261	270	309	318	328	338	347	390	400	419	429	448
30	229	239	248	258	267	305	315	324	333	343	385	395	414	423	442
35	226	236	245	254	264	301	310	320	329	338	380	389	407	416	434
40	223	233	242	251	260	296	305	315	324	333	373	382	399	408	426
42	222	231	240	249	258	294	303	312	321	330	370	378	396	404	422
44	221	230	239	248	257	292	301	310	319	327	366	375	392	400	417
46	219	228	237	246	255	290	299	307	316	325	363	371	388	396	413
48	218	227	236	244	253	288	296	305	313	322	359	367	383	391	407
50	216	225	234	242	251	285	293	302	310	318	355	363	378	386	401
52	215	223	232	241	249	282	291	299	307	315	350	358	373	380	395
54	213	222	230	238	247	279	287	295	303	311	345	353	367	374	388
56	211	220	228	236	244	276	284	292	300	307	340	347	361	368	381
58	210	218	226	234	242	273	281	288	295	303	334	341	354	360	372
60	208	216	224	232	239	269	277	284	291	298	328	334	346	352	364
62	206	214	221	229	236	266	273	280	286	293	321	327	338	344	354
64	204	211	219	226	234	262	268	275	281	288	314	320	330	335	344
66	201	209	216	223	230	257	264	270	276	282	307	312	321	326	334
68	199	206	213	220	227	253	259	265	270	276	299	303	312	316	324
70	196	203	210	217	224	248	254	259	265	270	291	295	303	306	313
72	194	201	207	214	220	243	248	253	258	263	282	286	293	296	302
74	191	198	204	210	216	238	243	247	252	256	274	277	283	286	292
76	188	194	200	206	212	232	237	241	245	249	265	268	274	276	281
78	185	191	197	202	208	227	231	235	239	242	257	259	264	267	271
80	182	188	193	198	203	221	225	229	232	235	248	251	255	257	261
82	179	184	189	194	199	215	219	222	225	228	240	242	246	248	251
84	176	181	185	190	194	209	213	216	219	221	232	234	237	239	242
86	172	177	181	186	190	204	207	209	212	214	224	225	229	230	233
88	169	173	177	181	185	198	200	203	205	208	216	218	220	222	224
90	165	169	173	177	180	192	195	197	199	201	209	210	213	214	216
92	162	166	169	173	176	186	189	191	193	194	201	203	205	206	208
94	158	162	165	168	171	181	183	185	187	188	194	196	198	199	200
96	154	158	161	164	166	175	177	179	181	182	188	189	191	192	193
98	151	154	157	159	162	170	172	173	175	176	181	182	184	185	186
100	147	150	153	155	157	165	167	168	169	171	175	176	178	178	180
102	144	146	149	151	153	160	161	163	164	165	169	170	172	172	174
104	140	142	145	147	149	155	156	158	159	160	164	165	166	166	168
106	136	139	141	143	145	150	152	153	154	155	158	159	160	161	162
108	133	135	137	139	141	146	147	148	149	150	153	154	155	156	157

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

2) Values of p_c (N/mm²) with $\lambda \geq 110$ for strut curve a															
λ	Steel grade and design strength p_y (in N/mm²)														
	S 275					S 355					S 460				
	235	**245**	**255**	**265**	**275**	**315**	**325**	**335**	**345**	**355**	**400**	**410**	**430**	**440**	**460**
110	130	132	133	135	137	142	143	144	144	145	148	149	150	150	151
112	126	128	130	131	133	137	138	139	140	141	144	144	145	146	146
114	123	125	126	128	129	133	134	135	136	136	139	140	141	141	142
116	120	121	123	124	125	129	130	131	132	132	135	135	136	137	137
118	117	118	120	121	122	126	126	127	128	128	131	131	132	132	133
120	114	115	116	118	119	122	123	123	124	125	127	127	128	128	129
122	111	112	113	114	115	119	119	120	120	121	123	123	124	124	125
124	108	109	110	111	112	115	116	116	117	117	119	120	120	121	121
126	105	106	107	108	109	112	113	113	114	114	116	116	117	117	118
128	103	104	105	105	106	109	109	110	110	111	112	113	113	114	114
130	100	101	102	103	103	106	106	107	107	108	109	110	110	110	111
135	94	95	95	96	97	99	99	100	100	101	102	102	103	103	103
140	88	89	90	90	91	93	93	93	94	94	95	95	96	96	96
145	83	84	84	85	85	87	87	87	88	88	89	89	90	90	90
150	78	79	79	80	80	82	82	82	82	83	83	84	84	84	84
155	74	74	75	75	75	77	77	77	77	78	78	79	79	79	79
160	70	70	70	71	71	72	72	73	73	73	74	74	74	74	75
165	66	66	67	67	67	68	68	69	69	69	70	70	70	70	70
170	62	63	63	63	64	64	65	65	65	65	66	66	66	66	66
175	59	59	60	60	60	61	61	61	61	62	62	62	62	63	63
180	56	56	57	57	57	58	58	58	58	58	59	59	59	59	59
185	53	54	54	54	54	55	55	55	55	55	56	56	56	56	56
190	51	51	51	51	52	52	52	52	53	53	53	53	53	53	53
195	48	49	49	49	49	50	50	50	50	50	50	51	51	51	51
200	46	46	46	47	47	47	47	47	48	48	48	48	48	48	48
210	42	42	42	43	43	43	43	43	43	43	44	44	44	44	44
220	39	39	39	39	39	39	39	40	40	40	40	40	40	40	40
230	35	36	36	36	36	36	36	36	36	36	37	37	37	37	37
240	33	33	33	33	33	33	33	33	33	33	34	34	34	34	34
250	30	30	30	30	30	31	31	31	31	31	31	31	31	31	31
260	28	28	28	28	28	28	29	29	29	29	29	29	29	29	29
270	26	26	26	26	26	26	27	27	27	27	27	27	27	27	27
280	24	24	24	24	24	25	25	25	25	25	25	25	25	25	25
290	23	23	23	23	23	23	23	23	23	23	23	23	23	23	23
300	21	21	21	21	21	22	22	22	22	22	22	22	22	22	22
310	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
320	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19
330	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18
340	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17
350	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

3) Values of p_c (N/mm²) with λ < 110 for strut curve b

λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
15	235	245	255	265	275	315	325	335	345	355	399	409	428	438	457
20	234	243	253	263	272	310	320	330	339	349	391	401	420	429	448
25	229	239	248	258	267	304	314	323	332	342	384	393	411	421	439
30	225	234	243	253	262	298	307	316	325	335	375	384	402	411	429
35	220	229	238	247	256	291	300	309	318	327	366	374	392	400	417
40	216	224	233	241	250	284	293	301	310	318	355	364	380	388	404
42	213	222	231	239	248	281	289	298	306	314	351	359	375	383	399
44	211	220	228	237	245	278	286	294	302	310	346	354	369	377	392
46	209	218	226	234	242	275	283	291	298	306	341	349	364	371	386
48	207	215	223	231	239	271	279	287	294	302	336	343	358	365	379
50	205	213	221	229	237	267	275	283	290	298	330	337	351	358	372
52	203	210	218	226	234	264	271	278	286	293	324	331	344	351	364
54	200	208	215	223	230	260	267	274	281	288	318	325	337	344	356
56	198	205	213	220	227	256	263	269	276	283	312	318	330	336	347
58	195	202	210	217	224	252	258	265	271	278	305	311	322	328	339
60	193	200	207	214	221	247	254	260	266	272	298	304	314	320	330
62	190	197	204	210	217	243	249	255	261	266	291	296	306	311	320
64	187	194	200	207	213	238	244	249	255	261	284	289	298	302	311
66	184	191	197	203	210	233	239	244	249	255	276	281	289	294	301
68	181	188	194	200	206	228	233	239	244	249	269	273	281	285	292
70	178	185	190	196	202	223	228	233	238	242	261	265	272	276	282
72	175	181	187	193	198	218	223	227	232	236	254	257	264	267	273
74	172	178	183	189	194	213	217	222	226	230	246	249	255	258	264
76	169	175	180	185	190	208	212	216	220	223	238	241	247	250	255
78	166	171	176	181	186	203	206	210	214	217	231	234	239	241	246
80	163	168	172	177	181	197	201	204	208	211	224	226	231	233	237
82	160	164	169	173	177	192	196	199	202	205	217	219	223	225	229
84	156	161	165	169	173	187	190	193	196	199	210	212	216	218	221
86	153	157	161	165	169	182	185	188	190	193	203	205	208	210	213
88	150	154	158	161	165	177	180	182	185	187	196	198	201	203	206
90	146	150	154	157	161	172	175	177	179	181	190	192	195	196	199
92	143	147	150	153	156	167	170	172	174	176	184	185	188	189	192
94	140	143	147	150	152	162	165	167	169	171	178	179	182	183	185
96	137	140	143	146	148	158	160	162	164	165	172	173	176	177	179
98	134	137	139	142	145	153	155	157	159	160	167	168	170	171	173
100	130	133	136	138	141	149	151	152	154	155	161	162	164	165	167
102	127	130	132	135	137	145	146	148	149	151	156	157	159	160	162
104	124	127	129	131	133	141	142	144	145	146	151	152	154	155	156
106	121	124	126	128	130	137	138	139	141	142	147	148	149	150	151
108	118	121	123	125	126	133	134	135	137	138	142	143	144	145	147

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

4) Values of p_c (N/mm²) with $\lambda \geq 110$ for strut curve b															
λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	115	118	120	121	123	129	130	131	133	134	138	139	140	141	142
112	113	115	117	118	120	125	127	128	129	130	134	134	136	136	138
114	110	112	114	115	117	122	123	124	125	126	130	130	132	132	133
116	107	109	111	112	114	119	120	121	122	122	126	126	128	128	129
118	105	106	108	109	111	115	116	117	118	119	122	123	124	124	125
120	102	104	105	107	108	112	113	114	115	116	119	119	120	121	122
122	100	101	103	104	105	109	110	111	112	112	115	116	117	117	118
124	97	99	100	101	102	106	107	108	109	109	112	112	113	114	115
126	95	96	98	99	100	103	104	105	106	106	109	109	110	111	111
128	93	94	95	96	97	101	101	102	103	103	106	106	107	107	108
130	90	92	93	94	95	98	99	99	100	101	103	103	104	105	105
135	85	86	87	88	89	92	93	93	94	94	96	97	97	98	98
140	80	81	82	83	84	86	87	87	88	88	90	90	91	91	92
145	76	77	78	78	79	81	82	82	83	83	84	85	85	86	86
150	72	72	73	74	74	76	77	77	78	78	79	80	80	80	81
155	68	69	69	70	70	72	72	73	73	73	75	75	75	76	76
160	64	65	65	66	66	68	68	69	69	69	70	71	71	71	72
165	61	62	62	62	63	64	65	65	65	65	66	67	67	67	68
170	58	58	59	59	60	61	61	61	62	62	63	63	63	64	64
175	55	55	56	56	57	58	58	58	59	59	60	60	60	60	60
180	52	53	53	53	54	55	55	55	56	56	56	57	57	57	57
185	50	50	51	51	51	52	52	53	53	53	54	54	54	54	54
190	48	48	48	48	49	50	50	50	50	50	51	51	51	51	52
195	45	46	46	46	46	47	47	48	48	48	49	49	49	49	49
200	43	44	44	44	44	45	45	45	46	46	46	46	47	47	47
210	40	40	40	40	41	41	41	41	42	42	42	42	42	43	43
220	36	37	37	37	37	38	38	38	38	38	39	39	39	39	39
230	34	34	34	34	34	35	35	35	35	35	35	36	36	36	36
240	31	31	31	31	32	32	32	32	32	32	33	33	33	33	33
250	29	29	29	29	29	30	30	30	30	30	30	30	30	30	30
260	27	27	27	27	27	27	28	28	28	28	28	28	28	28	28
270	25	25	25	25	25	26	26	26	26	26	26	26	26	26	26
280	23	23	23	23	24	24	24	24	24	24	24	24	24	24	24
290	22	22	22	22	22	22	22	22	22	22	23	23	23	23	23
300	20	20	21	21	21	21	21	21	21	21	21	21	21	21	21
310	19	19	19	19	19	20	20	20	20	20	20	20	20	20	20
320	18	18	18	18	18	18	18	19	19	19	19	19	19	19	19
330	17	17	17	17	17	17	17	17	17	18	18	18	18	18	18
340	16	16	16	16	16	16	16	16	17	17	17	17	17	17	17
350	15	15	15	15	15	16	16	16	16	16	16	16	16	16	16

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

5) Values of p_c (N/mm²) with λ < 110 for strut curve c															
λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	**245**	**255**	**265**	**275**	**315**	**325**	**335**	**345**	**355**	**400**	**410**	**430**	**440**	**460**
15	235	245	255	265	275	315	325	335	345	355	398	408	427	436	455
20	233	242	252	261	271	308	317	326	336	345	387	396	414	424	442
25	226	235	245	254	263	299	308	317	326	335	375	384	402	410	428
30	220	228	237	246	255	289	298	307	315	324	363	371	388	396	413
35	213	221	230	238	247	280	288	296	305	313	349	357	374	382	397
40	206	214	222	230	238	270	278	285	293	301	335	343	358	365	380
42	203	211	219	227	235	266	273	281	288	296	329	337	351	358	373
44	200	208	216	224	231	261	269	276	284	291	323	330	344	351	365
46	197	205	213	220	228	257	264	271	279	286	317	324	337	344	357
48	195	202	209	217	224	253	260	267	274	280	311	317	330	337	349
50	192	199	206	213	220	248	255	262	268	275	304	310	323	329	341
52	189	196	203	210	217	244	250	257	263	270	297	303	315	321	333
54	186	193	199	206	213	239	245	252	258	264	291	296	308	313	324
56	183	189	196	202	209	234	240	246	252	258	284	289	300	305	315
58	179	186	192	199	205	229	235	241	247	252	277	282	292	297	306
60	176	183	189	195	201	225	230	236	241	247	270	274	284	289	298
62	173	179	185	191	197	220	225	230	236	241	262	267	276	280	289
64	170	176	182	188	193	215	220	225	230	235	255	260	268	272	280
66	167	173	178	184	189	210	215	220	224	229	248	252	260	264	271
68	164	169	175	180	185	205	210	214	219	223	241	245	252	256	262
70	161	166	171	176	181	200	204	209	213	217	234	238	244	248	254
72	157	163	168	172	177	195	199	203	207	211	227	231	237	240	246
74	154	159	164	169	173	190	194	198	202	205	220	223	229	232	238
76	151	156	160	165	169	185	189	193	196	200	214	217	222	225	230
78	148	152	157	161	165	180	184	187	191	194	207	210	215	217	222
80	145	149	153	157	161	176	179	182	185	188	201	203	208	210	215
82	142	146	150	154	157	171	174	177	180	183	195	197	201	203	207
84	139	142	146	150	154	167	169	172	175	178	189	191	195	197	201
86	135	139	143	146	150	162	165	168	170	173	183	185	189	190	194
88	132	136	139	143	146	158	160	163	165	168	177	179	183	184	187
90	129	133	136	139	142	153	156	158	161	163	172	173	177	178	181
92	126	130	133	136	139	149	152	154	156	158	166	168	171	173	175
94	124	127	130	133	135	145	147	149	151	153	161	163	166	167	170
96	121	124	127	129	132	141	143	145	147	149	156	158	160	162	164
98	118	121	123	126	129	137	139	141	143	145	151	153	155	157	159
100	115	118	120	123	125	134	135	137	139	140	147	148	151	152	154
102	113	115	118	120	122	130	132	133	135	136	143	144	146	147	149
104	110	112	115	117	119	126	128	130	131	133	138	139	142	142	144
106	107	110	112	114	116	123	125	126	127	129	134	135	137	138	140
108	105	107	109	111	113	120	121	123	124	125	130	131	133	134	136

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

6) Values of p_c (N/mm²) with $\lambda \geq 110$ for strut curve c															
λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	102	104	106	108	110	116	118	119	120	122	126	127	129	130	132
112	100	102	104	106	107	113	115	116	117	118	123	124	125	126	128
114	98	100	101	103	105	110	112	113	114	115	119	120	122	123	124
116	95	97	99	101	102	108	109	110	111	112	116	117	118	119	120
118	93	95	97	98	100	105	106	107	108	109	113	114	115	116	117
120	91	93	94	96	97	102	103	104	105	106	110	110	112	112	113
122	89	90	92	93	95	99	100	101	102	103	107	107	109	109	110
124	87	88	90	91	92	97	98	99	100	100	104	104	106	106	107
126	85	86	88	89	90	94	95	96	97	98	101	102	103	103	104
128	83	84	86	87	88	92	93	94	95	95	98	99	100	100	101
130	81	82	84	85	86	90	91	91	92	93	96	96	97	98	99
135	77	78	79	80	81	84	85	86	87	87	90	90	91	92	92
140	72	74	75	76	76	79	80	81	81	82	84	85	85	86	87
145	69	70	71	71	72	75	76	76	77	77	79	80	80	81	81
150	65	66	67	68	68	71	71	72	72	73	75	75	76	76	76
155	62	63	63	64	65	67	67	68	68	69	70	71	71	72	72
160	59	59	60	61	61	63	64	64	65	65	66	67	67	67	68
165	56	56	57	58	58	60	60	61	61	61	63	63	64	64	64
170	53	54	54	55	55	57	57	58	58	58	60	60	60	60	61
175	51	51	52	52	53	54	54	55	55	55	56	57	57	57	58
180	48	49	49	50	50	51	52	52	52	53	54	54	54	54	55
185	46	46	47	47	48	49	49	50	50	50	51	51	52	52	52
190	44	44	45	45	45	47	47	47	47	48	49	49	49	49	49
195	42	42	43	43	43	45	45	45	45	45	46	46	47	47	47
200	40	41	41	41	42	43	43	43	43	43	44	44	45	45	45
210	37	37	38	38	38	39	39	39	40	40	40	40	41	41	41
220	34	34	35	35	35	36	36	36	36	36	37	37	37	37	38
230	31	32	32	32	32	33	33	33	33	34	34	34	34	34	35
240	29	29	30	30	30	30	31	31	31	31	31	31	32	32	32
250	27	27	27	28	28	28	28	28	29	29	29	29	29	29	29
260	25	25	26	26	26	26	26	26	27	27	27	27	27	27	27
270	23	24	24	24	24	24	25	25	25	25	25	25	25	25	25
280	22	22	22	22	22	23	23	23	23	23	23	24	24	24	24
290	21	21	21	21	21	21	21	22	22	22	22	22	22	22	22
300	19	19	20	20	20	20	20	20	20	20	21	21	21	21	21
310	18	18	18	19	19	19	19	19	19	19	19	19	19	19	20
320	17	17	17	17	18	18	18	18	18	18	18	18	18	18	18
330	16	16	16	16	17	17	17	17	17	17	17	17	17	17	17
340	15	15	15	16	16	16	16	16	16	16	16	16	16	16	16
350	15	15	15	15	15	15	15	15	15	15	15	15	15	15	15

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

7) Values of p_c (N/mm²) with λ < 110 for strut curve d															
λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	**245**	**255**	**265**	**275**	**315**	**325**	**335**	**345**	**355**	**400**	**410**	**430**	**440**	**460**
15	235	245	255	265	275	315	325	335	345	355	397	407	425	435	453
20	232	241	250	259	269	305	314	323	332	341	381	390	408	417	434
25	223	231	240	249	257	292	301	309	318	326	365	373	390	398	415
30	213	222	230	238	247	279	287	296	304	312	348	356	372	380	396
35	204	212	220	228	236	267	274	282	290	297	331	339	353	361	375
40	195	203	210	218	225	254	261	268	275	283	314	321	334	341	355
42	192	199	206	214	221	249	256	263	270	277	307	314	327	333	346
44	188	195	202	209	216	244	251	257	264	271	300	306	319	325	337
46	185	192	199	205	212	239	245	252	258	265	293	299	311	317	329
48	181	188	195	201	208	234	240	246	252	259	286	291	303	309	320
50	178	184	191	197	204	228	235	241	247	253	278	284	295	301	311
52	174	181	187	193	199	223	229	235	241	246	271	277	287	292	303
54	171	177	183	189	195	218	224	229	235	240	264	269	279	284	294
56	167	173	179	185	191	213	219	224	229	234	257	262	271	276	285
58	164	170	175	181	187	208	213	218	224	229	250	255	264	268	277
60	161	166	172	177	182	203	208	213	218	223	243	247	256	260	268
62	157	163	168	173	178	198	203	208	212	217	236	240	248	252	260
64	154	159	164	169	174	193	198	202	207	211	229	233	241	245	252
66	150	156	160	165	170	188	193	197	201	205	223	226	234	237	244
68	147	152	157	162	166	184	188	192	196	200	216	220	226	230	236
70	144	149	153	158	162	179	183	187	190	194	210	213	219	222	228
72	141	145	150	154	158	174	178	182	185	189	203	207	213	215	221
74	138	142	146	150	154	170	173	177	180	183	197	200	206	209	214
76	135	139	143	147	151	165	169	172	175	178	191	194	199	202	207
78	132	136	139	143	147	161	164	167	170	173	186	188	193	195	200
80	129	132	136	140	143	156	160	163	165	168	180	182	187	189	194
82	126	129	133	136	140	152	155	158	161	163	175	177	181	183	187
84	123	126	130	133	136	148	151	154	156	159	169	171	176	177	181
86	120	123	127	130	133	144	147	149	152	154	164	166	170	172	175
88	117	120	123	127	129	140	143	145	148	150	159	161	165	167	170
90	114	118	121	123	126	137	139	141	144	146	154	156	160	161	164
92	112	115	118	120	123	133	135	137	139	142	150	152	155	156	159
94	109	112	115	117	120	129	132	134	136	138	145	147	150	152	154
96	107	109	112	115	117	126	128	130	132	134	141	143	146	147	150
98	104	107	109	112	114	123	125	126	128	130	137	138	141	143	145
100	102	104	107	109	111	119	121	123	125	126	133	134	137	138	141
102	99	102	104	106	108	116	118	120	121	123	129	131	133	134	136
104	97	99	102	104	106	113	115	116	118	120	126	127	129	130	132
106	95	97	99	101	103	110	112	113	115	116	122	123	125	126	128
108	93	95	97	99	101	107	109	110	112	113	119	120	122	123	125

Table 24 — Compressive strength p_c (N/mm²) (*continued*)

8) Values of p_c (N/mm²) with $\lambda \geq 110$ for strut curve d

λ	Steel grade and design strength p_y (N/mm²)														
	S 275					S 355					S 460				
	235	245	255	265	275	315	325	335	345	355	400	410	430	440	460
110	91	93	95	96	98	105	106	108	109	110	115	116	118	119	121
112	88	90	92	94	96	102	103	105	106	107	112	113	115	116	118
114	86	88	90	92	94	99	101	102	103	104	109	110	112	113	114
116	85	86	88	90	91	97	98	99	101	102	106	107	109	110	111
118	83	84	86	88	89	95	96	97	98	99	103	104	106	107	108
120	81	82	84	86	87	92	93	94	95	96	101	101	103	104	105
122	79	81	82	84	85	90	91	92	93	94	98	99	100	101	102
124	77	79	80	82	83	88	89	90	91	92	95	96	98	98	99
126	76	77	78	80	81	86	87	88	89	89	93	94	95	96	97
128	74	75	77	78	79	84	85	85	86	87	91	91	93	93	94
130	72	74	75	76	77	82	83	83	84	85	88	89	90	91	92
135	68	70	71	72	73	77	78	79	79	80	83	84	85	85	86
140	65	66	67	68	69	73	73	74	75	75	78	79	80	80	81
145	62	63	64	65	65	69	69	70	71	71	74	74	75	75	76
150	59	60	60	61	62	65	66	66	67	67	69	70	71	71	72
155	56	57	57	58	59	62	62	63	63	64	66	66	67	67	68
160	53	54	55	55	56	58	59	59	60	60	62	62	63	63	64
165	50	51	52	53	53	55	56	56	57	57	59	59	60	60	61
170	48	49	49	50	51	53	53	54	54	54	56	56	57	57	57
175	46	47	47	48	48	50	51	51	51	52	53	53	54	54	55
180	44	45	45	46	46	48	48	49	49	49	50	51	51	51	52
185	42	43	43	44	44	46	46	46	47	47	48	48	49	49	49
190	40	41	41	42	42	44	44	44	44	45	46	46	46	47	47
195	38	39	39	40	40	42	42	42	42	43	44	44	44	45	45
200	37	37	38	38	39	40	40	40	41	41	42	42	42	43	43
210	34	34	35	35	35	37	37	37	37	37	38	38	39	39	39
220	31	32	32	32	33	34	34	34	34	34	35	35	36	36	36
230	29	29	30	30	30	31	31	31	32	32	32	33	33	33	33
240	27	27	28	28	28	29	29	29	29	29	30	30	30	30	31
250	25	25	26	26	26	27	27	27	27	27	28	28	28	28	28
260	24	24	24	24	24	25	25	25	25	25	26	26	26	26	26
270	22	22	22	23	23	23	23	23	24	24	24	24	24	24	25
280	21	21	21	21	21	22	22	22	22	22	23	23	23	23	23
290	19	20	20	20	20	20	21	21	21	21	21	21	21	21	21
300	18	18	19	19	19	19	19	19	19	20	20	20	20	20	20
310	17	17	17	18	18	18	18	18	18	18	19	19	19	19	19
320	16	16	16	17	17	17	17	17	17	17	18	18	18	18	18
330	15	15	16	16	16	16	16	16	16	16	17	17	17	17	17
340	15	15	15	15	15	15	15	15	15	15	16	16	16	16	16
350	14	14	14	14	14	14	14	15	15	15	15	15	15	15	15

4.7.6 Eccentric connections

Moments due to eccentricity of connections should be allowed for in accordance with **4.8** except as follows.

a) *Columns in simple structures.* These should be designed in accordance with **4.7.7**.

b) *Laced, battened struts and batten-starred angle struts.* These may be treated as single integral members and designed as axially loaded struts in accordance with **4.7.8**, **4.7.9** or **4.7.11** respectively.

c) *Angles, channels and T-sections.* The effect of eccentric end connections may be neglected if these members are designed in accordance with **4.7.10**.

d) *Continuous structures.* These should be in accordance with Section 5.

4.7.7 Columns in simple structures

In structures of simple design, see **2.1.2.2**, it is not necessary to consider the effect on columns of pattern loading. For the purpose of column design, all the beams supported by a column at any one level should be assumed to be fully loaded.

The nominal moments applied to the column by simple beams or other simply-supported members should be calculated from the eccentricity of their reactions, taken as follows.

1) For a beam supported on the cap plate, the reaction should be taken as acting at the face of the column, or edge of packing if used, towards the span of the beam.

2) For a roof truss supported on the cap plate, the eccentricity may be neglected provided that simple connections are used that do not develop significant moments adversely affecting the structure.

3) In all other cases the reaction should be taken as acting 100 mm from the face of the steel column, or at the centre of the length of stiff bearing, whichever gives the greater eccentricity.

In multi-storey columns that are effectively continuous at their splices, the net moment applied at any one level should be divided between the column lengths above and below that level in proportion to the stiffness coefficient I/L of each length, except that when the ratio of the stiffness coefficients does not exceed 1.5 the moment may optionally be divided equally.

All equivalent uniform moment factors m should be taken as 1.0. The nominal moments applied to the column should be assumed to have no effect at the levels above and below the level at which they are applied. When only these nominal moments are applied, the column should satisfy the relationship:

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{p_y Z_y} \le 1$$

where

F_c is the compressive force due to axial force;

M_x is the nominal moment about the major axis;

M_y is the nominal moment about the minor axis;

M_{bs} is the buckling resistance moment for simple columns;

P_c is the compression resistance from **4.7.4**;

p_y is the design strength;

Z_y is the section modulus about the minor axis.

For circular or square hollow sections, and for rectangular hollow sections within the limiting value of L_E/r_y given in Table 15, the buckling resistance moment for simple columns M_{bs} should be taken as equal to the moment capacity M_c of the cross-section, see **4.2.5**.

For all other doubly symmetric cross-sections M_{bs} should be taken as the value of M_b determined as described in **4.3.6.4** but (except for rectangular hollow sections) using the equivalent slenderness λ_{LT} of the column given by:

$$\lambda_{LT} = 0.5L/r_y$$

where

L is the distance between levels at which the column is laterally restrained in both directions;

r_y is the radius of gyration about the minor axis.

4.7.8 Laced struts

A laced strut consisting of two or more main components may be designed as a single integral member, provided that the following conditions are met.

a) The main components should be effectively restrained against buckling by a lacing system of flats or sections.

b) The lacing should comprise an effectively triangulated system on each face and as far as practicable the lacing should not vary throughout the length of the member.

c) Except for the panels referred to in f), double intersection lacing systems and single intersection lacing systems mutually opposed in direction on opposite sides of two main components should not be combined with members or diaphragms perpendicular to the longitudinal axis of the strut unless all forces resulting from the deformation of the strut members are calculated and allowed for in the design.

d) Single lacing systems mutually opposed in direction on opposite sides of two main components should not be used unless the resulting torsional effects are allowed for.

e) All lacings, whether in double or single intersection systems, should be inclined at an angle between 40º and 70º to the axis of the member.

f) Tie panels should be provided at the ends of the lacing systems, at points where the lacing is interrupted, and at connections with other members. Tie panels may take the form of battens conforming to **4.7.9**; alternatively, cross braced panels of equivalent rigidity may be used. In either case the tie panels should be designed to carry the loads for which the lacing system is designed.

g) The slenderness λ_c of the main components (based on their minimum radius of gyration) between consecutive points where the lacing is attached should not exceed 50. If the overall slenderness of the member is less than $1.4\lambda_c$ the design should be based on a slenderness of $1.4\lambda_c$.

h) The effective length of a lacing should be taken as the distance between the inner end welds or bolts for single intersection lacing and as 0.7 times this distance for double intersection lacing connected by welds or bolts at the intersection. The slenderness of a lacing should not exceed 180.

i) The lacings and their connections should be designed to carry the forces induced by a transverse shear at any point in the length of the member equal to 2.5 % of the axial force in the member, divided equally amongst all transverse lacing systems in parallel planes. For members carrying moments due to eccentricity of loading, applied end moments or lateral loading, the lacing should be proportioned to resist the shear due to bending in addition to 2.5 % of the axial force.

4.7.9 Battened struts

A battened strut consisting of two or more main components may be designed as a single integral member, provided that the following conditions are met.

a) The main components should be effectively restrained against buckling by a system of battens consisting of plates or sections, so connected to the main components as to form with them an effectively rigid-jointed frame.

b) Battens should be positioned opposite each other in each plane at the ends of the member and at points where it is laterally restrained. Intermediate battens should be positioned opposite each other and be spaced and proportioned uniformly throughout the length of a member.

c) The slenderness λ_c of a main component (based on its minimum radius of gyration) between end welds or end bolts of adjacent battens should not exceed 50. The slenderness λ_b of the battened strut about the axis perpendicular to the plane of the battens should be calculated from:

$$\lambda_b = (\lambda_m^2 + \lambda_c^2)^{0.5}$$

where

λ_m is the ratio L_E/r of the whole member about that axis.

d) If λ_b is less than $1.4\lambda_c$ the design should be based on $\lambda_b = 1.4\lambda_c$.

e) The thickness of plate battens should be not less than 1/50 of the minimum distances between welds or bolts. The slenderness of sections used as battens should not exceed 180. The width of an end batten along the axis of the main components should be not less than the distance between centroids of the main members and not less than half this distance for intermediate battens. Further, the width of any batten should be not less than twice the width of the narrower main component.

f) The battens and the connections between them and the main components should be designed to carry the forces and moments induced by a transverse shear at any point in the length of a member equal to 2.5 % of the axial force in the member. For members carrying moments due to eccentricity of loading, applied end moments or lateral loads, the battens should be proportioned to resist the shear due to bending in addition to 2.5 % of the axial force.

NOTE For battened angle members see **4.7.11** or **4.7.12** as appropriate.

4.7.10 Angle, channel or T-section struts

4.7.10.1 *General*

Struts composed of angles, channels or T-sections may be treated as axially loaded, neglecting the eccentricity of normal end connections, provided that the criteria given in **4.7.10.2**, **4.7.10.3**, **4.7.10.4** and **4.7.10.5** are satisfied.

Alternatively, in the internal segments of continuous struts, such as those forming the legs of towers or the flanges of lattice girders, the effective length may be determined from **4.7.3** and Table 22.

The segment length L should be taken as the distance between the intersection of centroidal axes or the intersections of the setting out lines of the bolts, and r is the radius of gyration about the relevant axis. The axes should be taken as defined in Table 25.

Intermediate restraints may be allowed for in determining the segment lengths L for buckling about each axis, provided they lie at an angle of not more than 45° to the plane of buckling considered.

In the case of a single-angle strut with lateral restraints to its two legs alternately, the slenderness for buckling about every axis should be increased by 20 %.

4.7.10.2 *Single angles*

For a single angle connected by one leg to a gusset, or directly to another member, at each end:

a) by two or more bolts in standard clearance holes in line along the angle, or by an equivalent welded connection, the slenderness λ should be taken as the greatest of:

1) $0.85L_v/r_v$ but $\geq 0.7L_v/r_v + 15$

2) $1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$

3) $0.85L_b/r_b$ but $\geq 0.7L_b/r_b + 30$

b) by two bolts in line along the angle, one in a standard clearance hole and one in a kidney-shaped slot, the slenderness λ should be taken as the greatest of:

1) $1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$

2) $1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$

3) $1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$

c) by a single bolt, the compression resistance should be taken as 80 % of the compression resistance of an axially loaded member and the slenderness λ should be taken as the greatest of:

1) $1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$

2) $1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$

3) $1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$

4.7.10.3 ***Double angles***

For double angles interconnected back-to-back as recommended in **4.7.13** or battened as recommended in **4.7.12** and connected by one leg of each angle to a gusset, or directly to another member, at each end:

a) to one side of a gusset or member by two or more bolts in line along each angle or by an equivalent weld, the slenderness λ should be taken as the greater of:

1) $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

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

b) to one side of a gusset or member by one bolt in each angle, the slenderness λ should be taken as the greater of:

1) $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

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

c) to both sides of a gusset or member by two or more bolts in standard clearance holes, in line along the angles, the slenderness λ should be taken as the greater of:

1) $0.85L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

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

d) to both sides of a gusset or member by two bolts in line along each angle, one in a standard clearance hole and one in a kidney-shaped slot, the slenderness λ should be taken as the greater of:

1) $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

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

e) to both sides of a gusset or member by a single bolt through each angle, the compression resistance should be taken as 80 % of the compression resistance of an axially loaded member and the slenderness λ should be taken as the greater of:

1) $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

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

where in a) to e) $\lambda_c = L_v/r_v$ in which L_v is measured between interconnecting bolts for back-to-back struts or between end welds or end bolts of adjacent battens for battened angle struts.

4.7.10.4 ***Single channels***

For a single channel connected only by its web to a gusset, or directly to another member at each end:

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

1) $0.85L_x/r_x$

2) $1.0L_y/r_y$ but $\geq 0.7L_y/r_y + 30$

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

1) $1.0L_x/r_x$

2) $1.0L_y/r_y$ but $\geq 0.7L_y/r_y + 30$

Table 25 — Angle, channel and T-section struts

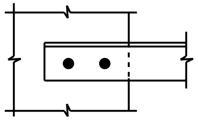
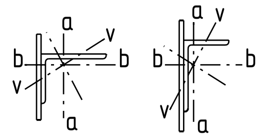
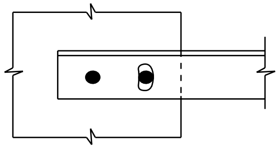
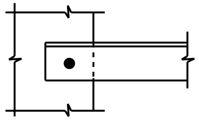
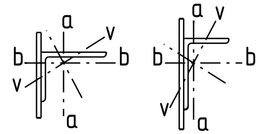
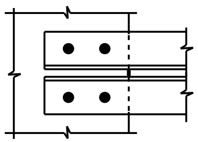
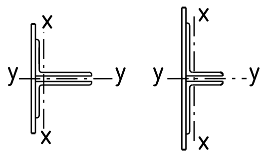
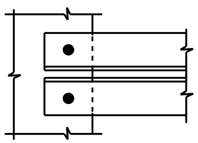
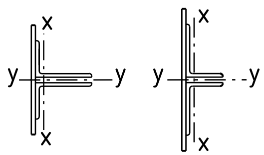
Clause	Connection	Sections and axes	Slenderness ratios (see notes 1 and 2)
4.7.10.2a)			v-v axis: $0.85L_v/r_v$ but $\geq 0.7L_v/r_v + 15$ a-a axis: $1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$ b-b axis: $0.85L_b/r_b$ but $\geq 0.7L_b/r_b + 30$
4.7.10.2b) **4.7.10.2**c)	 (kidney-shaped slot)  (see note 3)		v-v axis: $1.0L_v/r_v$ but $\geq 0.7L_v/r_v + 15$ a-a axis: $1.0L_a/r_a$ but $\geq 0.7L_a/r_a + 30$ b-b axis: $1.0L_b/r_b$ but $\geq 0.7L_b/r_b + 30$ (see note 3)
4.7.10.3a)	 (see note 4)		x-x axis: $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$ y-y axis: $[(0.85L_y/r_y)^2 + \lambda_c{}^2]^{0.5}$ but $\geq 1.4\lambda_c$ (see note 5)
4.7.10.3b)	 (see note 4)		x-x axis: $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$ y-y axis: $[(L_y/r_y)^2 + \lambda_c{}^2]^{0.5}$ but $\geq 1.4\lambda_c$ (see note 5)

Table 25 — Angle, channel and T-section struts (*continued*)

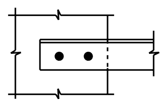
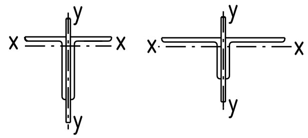
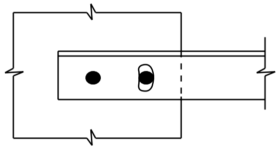
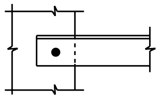
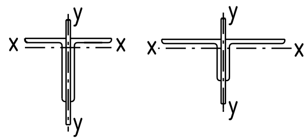
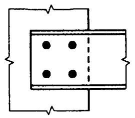
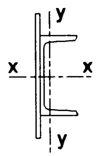
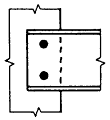
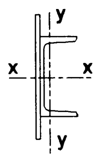
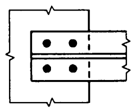
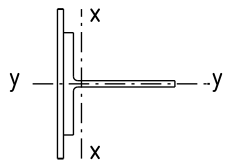
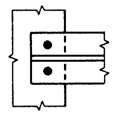
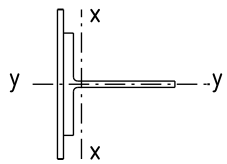
Clause	Connection	Sections and axes	Slenderness ratios (see notes 1 and 2)
4.7.10.3c)	 (see note 4)		x-x axis: $0.85L_x/r_x$ but $\geq 0.7L_x/r_x + 30$ y-y axis: $[(L_y/r_y)^2 + \lambda_c{}^2]^{0.5}$ but $\geq 1.4\lambda_c$ (see note 5)
4.7.10.3d) **4.7.10.3**e)	 (kidney-shaped slot)  (see notes 3 and 4)		x-x axis: $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$ y-y axis: $[(L_y/r_y)^2 + \lambda_c{}^2]^{0.5}$ but $\geq 1.4\lambda_c$ (see notes 3 and 5)
4.7.10.4a)			x-x axis: $0.85L_x/r_x$ y-y axis: $1.0L_y/r_y$ but $\geq 0.7L_y/r_y + 30$
4.7.10.4b)			x-x axis $1.0L_x/r_x$ y-y axis $1.0L_y/r_y$ but $\geq 0.7L_y/r_y + 30$

Table 25 — Angle, channel and T-section struts (*continued*)

Clause	Connection	Sections and axes	Slenderness ratios (see notes 1 and 2)
4.7.10.5a)			x-x axis: $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$ y-y axis: $0.85L_y/r_y$
4.7.10.5b)			x-x axis: $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$ y-y axis $1.0L_y/r_y$

NOTE 1 The length L is taken between the intersections of the centroidal axes or the intersections of the setting out lines of the bolts, irrespective of whether the strut is connected to a gusset or directly to another member.

NOTE 2 Intermediate restraints reduce the value of L for buckling about the relevant axes. For single angle members, L_v is taken between lateral restraints, perpendicular to either a-a or b-b.

NOTE 3 For single or double angles connected by one bolt, the compression resistance is also reduced to 80 % of that for an axially loaded member, see **4.7.10.2**b) or **4.7.10.3**d).

NOTE 4 Double angles are either battened (see **4.7.12**) or interconnected back-to-back (see **4.7.13**). Battens or interconnecting bolts are also needed at the ends of members.

NOTE 5 $\lambda_c = L_v/r_v$ with L_v measured between interconnecting bolts for back-to-back struts, or between end welds or end bolts of adjacent battens for battened angle struts.

4.7.10.5 *Single T-sections*

For a single T-section connected only by its flange to a gusset, or directly to another member at each end:

a) by two or more rows of bolts arranged symmetrically across the flange, or by an equivalent welded connection, the slenderness λ should be taken as the greater of:

1) $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

2) $0.85L_y/r_y$

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

1) $1.0L_x/r_x$ but $\geq 0.7L_x/r_x + 30$

2) $1.0L_y/r_y$

4.7.11 Starred angle struts

A battened angle member of cruciform cross-section may be designed as a single integral compression member provided that it meets the conditions given in **4.7.9** with the following modifications.

a) The battens should be connected to the backs of angles parallel to both the rectangular axes of the member. They should alternate in each plane and the effective length of a main component should be taken as the spacing centre-to-centre of the battens in the same plane.

b) The transverse shear of not less than 2.5 % of the axial force should be taken as acting perpendicular to the minor axis of the member. The battens in each plane should be designed for the components of this shear resolved perpendicular to the rectangular axes plus any transverse shear due to the weight or wind resistance of the member.

4.7.12 Battened parallel angle struts

A battened parallel angle member composed of two similar angles arranged symmetrically with their corresponding rectangular axes aligned may be designed as a single integral compression member providing that in all other respects it meets the conditions given in **4.7.9**.

The eccentricity of end connections should be allowed for as given in **4.7.10.3**.

4.7.13 Back-to-back struts

4.7.13.1 *Components separated*

A member composed of two angles, channels or T-sections, separated back-to-back by a distance not exceeding that required for the end gusset connection, may be designed as a single integral compression member provided that the following conditions are satisfied.

a) The main components should be of similar cross-section and their corresponding rectangular axes should be aligned.

b) The main components should be interconnected by bolts. Where the components are connected together by welding the member should be designed as a battened strut as given in **4.7.9**.

c) The member should not be subjected to transverse loads perpendicular to the connected surfaces other than the weight or the wind resistance of the member.

d) The slenderness λ of the compound strut about the axis parallel to the connected surfaces should be calculated from **4.7.9**c) for battened struts.

e) The main components should be connected at intervals so that the member is divided into at least three bays of approximately equal length. At the ends of the member the main components should be interconnected by not less than two bolts along each line along the length of the member.

f) The interconnecting bolts should be designed to transmit the longitudinal shear between the main components induced by a transverse shear Q at any point in the member; Q should be taken as not less than 2.5 % of the factored axial compression in the member plus any load due to self weight or wind resistance of the member. In no case should the bolts be less than 16 mm in diameter. The longitudinal shear per interconnection should be taken as $0.25Q\lambda_c$ in which λ_c is the slenderness of the main component centre-to-centre of interconnections.

g) At all interconnections the bolts should pass through solid steel packings, washers or gussets. In struts at least two bolts should be provided in line across the width of all members that are sufficiently wide to accommodate them.

4.7.13.2 *Components in contact*

A member composed of two angles, channels or T-sections in contact back-to-back or separated by continuous steel packing may be designed as a single integral compression member provided that the following conditions are met.

a) The main components should be similar sections arranged symmetrically with their corresponding rectangular axes aligned.

b) Interconnection should be as follows.

1) When interconnection is by means of bolts, at least two bolts should be used in line across the width of the member, provided that it is sufficiently wide. The spacing of the bolts should not exceed 300 mm or 32t where t is the thickness of the thinner part joined.

2) When interconnection is by means of welds, both pairs of edges of the main components should be welded. The spacing centre-to-centre of interconnections should be taken as the spacing centre-to-centre of consecutive effective lengths of weld on the same edge. The space between consecutive welds on the same edge should not exceed 300 mm or 16t where t is the minimum thickness of the parts joined.

c) The member should not be subject to transverse load perpendicular to the connected surfaces other than the weight or wind resistance of the member.

d) The slenderness λ of the compound strut about the axis parallel to the connected surfaces should be calculated from **4.7.9**c).

e) The main components should be interconnected at intervals so that the member is divided into at least three bays of approximately equal length. At the ends of the member the main components should be interconnected by not less than two bolts in each line along the length of the member, or by equivalent welds.

f) The interconnecting welds or bolts should be designed to transmit the longitudinal shear between the components as given in **4.7.13.1**f).

g) In members exposed to the weather or other corrosive influences the components should be connected by continuous welds, or bolts as specified in **6.2.2.5**.

4.8 Members with combined moment and axial force

4.8.1 General

Members subject to combined moment and axial tension should satisfy **4.8.2** and members subject to combined moment and axial compression should satisfy **4.8.3**.

In determining which interaction expressions apply, the classification of the cross-section should generally be based on the combined moment and axial force and this classification should be used in obtaining the moment capacity and buckling resistance moment from **4.2** and **4.3** for use in the interaction expressions.

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

For class 4 slender cross-sections the effective section properties should be determined as detailed in **3.6**.

Provided that the shear force F_v does not exceed 60 % of the shear capacity P_v (see **4.2.3**) nor 60 % of the simple shear buckling resistance V_w where relevant (see **4.4.5**), the cross-section capacity of a member of I, H, channel or RHS section may be assumed to be unaffected by shear. However, where F_v exceeds $0.6P_v$ or $0.6V_w$ the resistance of the web to the combined effects of axial force, moment and shear should be checked using **H.3**. If necessary, stresses due to axial force or moment may be shed from the web to the flanges using the method for plate girders given in **4.4.4**.

NOTE The reduction factor ρ starts when F_v exceeds $0.5P_v$ or $0.5V_w$ but the resulting reduction in moment capacity is negligible unless F_v exceeds $0.6P_v$ or $0.6V_w$.

The buckling resistance of the member may be assumed to be unaffected by shear.

For members with asymmetric cross-sections reference may optionally be made to **I.3**.

Moments in angle, channel or T-section members due to eccentricity of connections should be treated as recommended in **4.6.3** for tension members or **4.7.10** for compression members.

4.8.2 Tension members with moments

4.8.2.1 *General*

The cross-section capacity of tension members with moments should be checked using **4.8.2.2** or **4.8.2.3** at those locations where the moments and axial force are largest.

Tension members with moments should also be checked for resistance to lateral-torsional buckling in accordance with **4.3** under moment alone.

4.8.2.2 *Simplified method*

Generally the following relationship should be satisfied:

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

F_t is the axial tension at the critical location;

M_{cx} is the moment capacity about the major axis from **4.2.5**;

M_{cy} is the moment capacity about the minor axis from **4.2.5**;

M_x is the moment about the major axis at the critical location;

M_y is the moment about the minor axis at the critical location;

P_t is the tension capacity from **4.6.1**.

In the case of cross-sections that are not doubly-symmetric, reference may optionally be made to **I.3**.

4.8.2.3 *More exact method*

Alternatively, a member of class 1 plastic or class 2 compact cross-section, subject to a moment about only one axis should satisfy the relevant criterion as follows:

— major axis moment only:

$$M_x \leq M_{rx}$$

— minor axis moment only:

$$M_y \leq M_{ry}$$

where

M_{rx} is the major axis reduced plastic moment capacity in the presence of axial force, see **I.2**;

M_{ry} is the minor axis reduced plastic moment capacity in the presence of axial force, see **I.2**.

In addition, provided that the cross-section is also doubly-symmetric, a member subject to moments about both axes may be checked using:

$$\left(\frac{M_x}{M_{rx}}\right)^{z_1} + \left(\frac{M_y}{M_{ry}}\right)^{z_2} \leq 1$$

where

z_1 is a constant taken as follows:

- 2.0 for I- and H-sections with equal flanges;
- 2.0 for solid or hollow circular sections;
- 5/3 for solid or hollow rectangular sections;
- 1.0 for all other cases;

z_2 is a constant taken as follows:

- 1.0 for I- and H-sections;
- 2.0 for solid or hollow circular sections;
- 5/3 for solid or hollow rectangular sections;
- 1.0 for all other cases.

4.8.3 Compression members with moments

4.8.3.1 *General*

The cross-sectional capacity of compression members with moments should be checked at those locations where the moments and axial force are largest, using **4.8.3.2**.

The buckling resistance of the member as a whole should also be checked, using either the simplified approach given in **4.8.3.3.1** or the more exact approach for doubly-symmetric sections given in **4.8.3.3.2** or **4.8.3.3.3**. As a further alternative, the buckling resistance of a member of doubly-symmetric class 1 plastic or class 2 compact cross-section may be verified using the method for stocky members given in **I.1**.

For the application of **4.8.3.3** to a single angle section see **I.4**.

4.8.3.2 *Cross-section capacity*

The cross-section capacity may be checked as follows.

a) Generally, except for class 4 slender cross-sections:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

In the case of cross-sections that are not doubly-symmetric, reference may optionally be made to **I.3**.

b) Alternatively, for class 1 plastic or class 2 compact cross-sections, **4.8.2.3** may be applied.

c) For class 4 slender cross-sections:

$$\frac{F_c}{A_{eff} p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

A_{eff} is the effective cross-sectional area from **3.6**;

A_g is the gross cross-sectional area;

F_c is the axial compression at the critical location;

and the other symbols are as detailed in **4.8.2.2**.

4.8.3.3 *Member buckling resistance*

4.8.3.3.1 *Simplified method*

The buckling resistance of a member may be verified by checking that the following relationships are both satisfied:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

where

F_c is the axial compression;

M_b is the buckling resistance moment, generally from **4.3**, but from **I.4** for single angle members;

M_{LT} is the maximum major axis moment in the segment length L governing M_b;

M_x is the maximum major axis moment in the segment length L_x governing P_{cx};

M_y is the maximum minor axis moment in the segment length L_y governing P_{cy};

P_c is the smaller of P_{cx} and P_{cy};

P_{cx} is the compression resistance from **4.7.4**, considering buckling about the major axis only;

P_{cy} is the compression resistance from **4.7.4**, considering buckling about the minor axis only;

Z_x is the section modulus about the major axis;

Z_y is the section modulus about the minor axis.

For a class 4 slender cross-section the effective section modulus Z_{eff} should be used, see **3.6**.

The equivalent uniform moment factors m_{LT}, m_x and m_y should be obtained from **4.8.3.3.4**.

For cross-sections that are not doubly-symmetric, reference may optionally be made to **I.3**.

4.8.3.3.2 *More exact method for I- or H-sections with equal flanges*

The buckling resistance of a member of I- or H-section with equal flanges may be verified by checking the following:

a) for members with moments about the major axis only:

— for major axis in-plane buckling:

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) \le 1$$

— for out-of-plane buckling:

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \le 1$$

b) for members with moments about the minor axis only:

— for minor axis in-plane buckling:

$$\frac{F_c}{P_{cy}} + \frac{m_y M_y}{M_{cy}}\left(1 + \frac{F_c}{P_{cy}}\right) \le 1$$

— for out-of-plane buckling:

$$\frac{F_c}{P_{cx}} + 0.5\frac{m_{yx} M_y}{M_{cy}} \le 1$$

c) for members with moments about both axes:

— for major axis buckling:

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) + 0.5\frac{m_{yx} M_y}{M_{cy}} \le 1$$

— for lateral-torsional buckling:

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{M_{cy}}\left(1 + \frac{F_c}{P_{cy}}\right) \le 1$$

— for interactive buckling:

$$\frac{m_x M_x(1 + 0.5(F_c/P_{cx}))}{M_{cx}(1 - F_c/P_{cx})} + \frac{m_y M_y(1 + F_c/P_{cy})}{M_{cy}(1 - F_c/P_{cy})} \le 1$$

where

M_{cx} is the major axis moment capacity from **4.2.5**;

M_{cy} is the minor axis moment capacity from **4.2.5**.

The equivalent uniform moment factors m_{LT}, m_x, m_y and m_{yx} should be obtained from **4.8.3.3.4** and the other symbols are as defined in **4.8.3.3.1**.

4.8.3.3.3 *More exact method for CHS, RHS or box sections with equal flanges*

The buckling resistance of a member of CHS, RHS or box section with equal flanges may be verified by checking the following:

a) for members with moments about the major axis only:

— for major axis in-plane buckling:

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) \le 1$$

— for out-of-plane buckling:

— provided that no lateral-torsional buckling check is needed (see **4.3.6.1**):

$$\frac{F_c}{P_{cy}} + 0.5\frac{m_{LT} M_{LT}}{M_{cx}} \le 1$$

— if a lateral-torsional buckling check is needed (see **4.3.6.1**):

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \le 1$$

b) for members with moments about the minor axis only:

— for minor axis in-plane buckling:

$$\frac{F_c}{P_{cy}} + \frac{m_y M_y}{M_{cy}}\left(1 + 0.5\frac{F_c}{P_{cy}}\right) \le 1$$

— for out-of-plane buckling:

$$\frac{F_c}{P_{cx}} + 0.5\frac{m_{yx} M_y}{M_{cy}} \le 1$$

c) for members with moments about both axes:

— for major axis buckling:

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) + 0.5\frac{m_{yx} M_y}{M_{cy}} \le 1$$

— for minor axis buckling, provided that no lateral-torsional buckling check is needed (see **4.3.6.1**):

$$\frac{F_c}{P_{cy}} + 0.5\frac{m_{LT} M_{LT}}{M_{cx}} + \frac{m_y M_y}{M_{cy}}\left(1 + 0.5\frac{F_c}{P_{cy}}\right) \le 1$$

— for minor axis buckling, if a lateral-torsional buckling check is needed (see **4.3.6.1**):

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{M_{cy}}\left(1 + 0.5\frac{F_c}{P_{cy}}\right) \le 1$$

— for interactive buckling:

$$\frac{m_x M_x(1 + 0.5(F_c/P_{cx}))}{M_{cx}(1 - F_c/P_{cx})} + \frac{m_y M_y(1 + 0.5(F_c/P_{cy}))}{M_{cy}(1 - F_c/P_{cy})} \le 1$$

4.8.3.3.4 *Equivalent uniform moment factors*

The equivalent uniform moment factors for use in **4.8.3.3** should be based upon the pattern of moments over the relevant segment length and obtained as follows:

— the factor m_{LT} for lateral-torsional buckling:
from Table 18 for the pattern of major axis moments over the segment length L_{LT} governing M_b;

— the factor m_x for major axis flexural buckling:
from Table 26 for the pattern of major axis moments over the segment length L_x governing P_{cx};

— the factor m_y for minor axis flexural buckling:
from Table 26 for the pattern of minor axis moments over the segment length L_y governing P_{cy};

— the factor m_{yx} for lateral flexural buckling:
from Table 26 for the pattern of minor axis moments over the segment length L_x governing P_{cx}.

where

L_{LT} is the segment length between restraints against lateral-torsional buckling, see **4.3**;

L_x is the segment length between restraints against flexural buckling about the major axis;

L_y is the segment length between restraints against flexural buckling about the minor axis.

For cantilever columns and for members in sway-sensitive frames, see **2.4.2.7**, the following modifications should be made, depending upon the method used to allow for the effects of sway:

— if sway mode in-plane effective lengths are used, the value of m_x, m_y or m_{yx} for moments in that plane should not be taken as less than 0.85;

— if amplified sway moments are used, see **5.6.4**, the factor m_x, m_y or m_{yx} for moments in that plane should not be applied to the amplified sway moment, thus terms of the form mM should be replaced by terms of the form $k_{amp}M_s + mM_n$ in which M_s is the sway moment, M_n is the non-sway moment and k_{amp} is the amplification factor from **2.4.2.7** for moments in that plane.

4.9 Members with biaxial moments

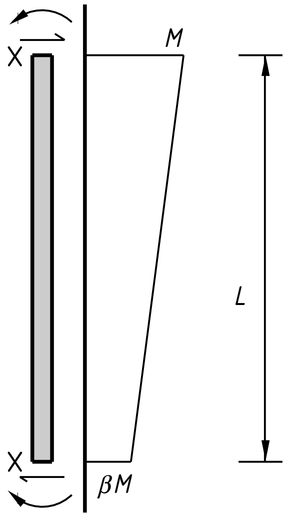
Members subject to moments about both axes in the absence of tensile or compressive axial force should be designed in accordance with **4.8.3**, taking the value of F_c as zero.

For the application of **4.9** to a single angle section see **I.4**.

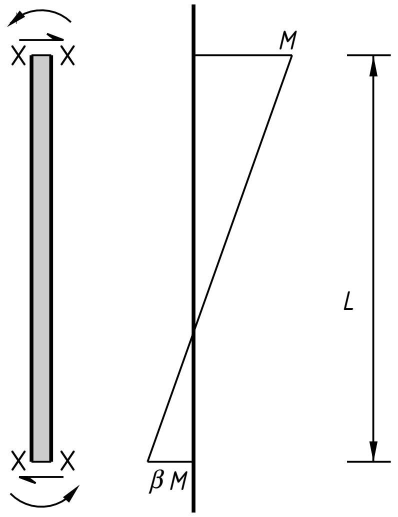
Table 26 — Equivalent uniform moment factor m for flexural buckling

Segments with end moments only (values of m from the formula for the general case)

β positive



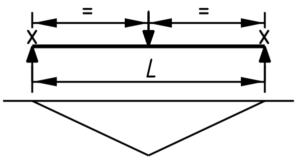
β negative



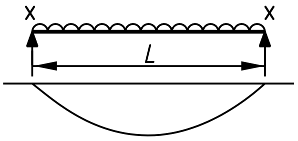
β	m
1.0	1.00
0.9	0.96
0.8	0.92
0.7	0.88
0.6	0.84
0.5	0.80
0.4	0.76
0.3	0.72
0.2	0.68
0.1	0.64
0.0	0.60
– 0.1	0.58
– 0.2	0.56
– 0.3	0.54
– 0.4	0.52
– 0.5	0.50
– 0.6	0.48
– 0.7	0.46
– 0.8	0.44
– 0.9	0.42
– 1.0	0.40

Segments between intermediate lateral restraints

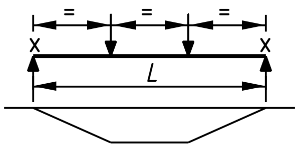
Specific cases



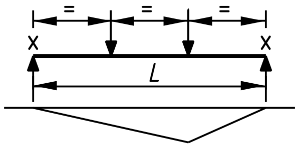
$m = 0.90$



$m = 0.95$

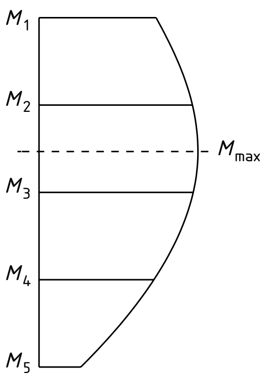


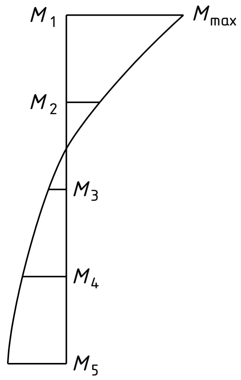
$m = 0.95$



$m = 0.80$

General case





$$m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{max}} \text{ but } m \geq \frac{0.8M_{24}}{M_{max}}$$

The moments M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length.

If M_2, M_3 and M_4 all lie on the same side of the axis, their values are all taken as positive. If they lie both sides of the axis, the side leading to the larger value of m is taken as the positive side.

The values of M_{max} and M_{24} are always taken as positive. M_{max} is the maximum moment in the segment and M_{24} is the maximum moment in the central half of the segment.

4.10 Members in lattice frames and trusses

In the design of lattice frames and trusses, unless fatigue is a design consideration, it may be assumed that:

a) the in-plane lengths of chord members (rafters or bottom chords) should be taken as the distance between connections to internal members, and the out-of-plane lengths as the distance between purlins or longitudinal ties, provided that such ties are properly connected to an adequate restraint system;

b) for the purpose of calculating the effective length of members, the fixity of the connections and the rigidity of adjacent members may be taken into account;

c) for the purpose of calculating the forces in the members, the connections may be taken as pinned;

d) if the exact locations of the purlins are not fixed relative to the points where the rafter is connected to the internal members, the bending moment in the rafter may be taken as $wL^2/6$, in which L is the length of the rafter between such points and w is the total load per unit length applied perpendicular to the rafter.

If the sheeting spans directly from truss to truss without using purlins, the stability of the rafter should be investigated and the sheeting should be adequately fixed. This method of providing restraint to the rafter should not be used unless the loading is mainly roof loading.

4.11 Gantry girders

4.11.1 General

Gantry girders resisting loads from overhead travelling cranes, see **2.2.3** and **2.4.1**, should satisfy the conditions given in **4.11.2**, **4.11.3**, **4.11.4** and **4.11.5**, in addition to those given in **4.2**, **4.3**, **4.4**, **4.5** and **4.9**.

4.11.2 Crabbing of trolley

Gantry girders intended to carry cranes of loading class Q1 and Q2 as defined in BS 2573-1 need not be designed for the effects of crabbing action.

Gantry girders intended to carry cranes of class Q3 and Q4 as defined in BS 2573-1 should be designed for the following couple due to crabbing action. This couple need not be combined with the horizontal loads obtained from **2.2.3**. The couple is due to the crabbing action of two wheels or bogies comprising two equal and opposite forces F_R acting transverse to the rail, one at each end of the wheelbase:

$$F_R = \frac{L_c W_w}{40 a_w} \quad \text{but} \quad F_R \geq \frac{W_w}{20}$$

where

a_w is the distance between the centres of the two end wheels or between the pivots of the bogies, except that if horizontal guide rollers are used a_w is the wheelbase of the guide rollers;

L_c is the span of the crane;

W_w is the largest load (including dynamic effects) on a wheel or bogie pivot.

4.11.3 Lateral-torsional buckling

Due to the interaction between crane wheels and crane rails, crane loads need not be treated as destabilizing, see **4.3.4**, provided that the rails are not mounted on resilient pads. In either case, the equivalent uniform moment factor m_{LT} in **4.3.6.2** should be taken as 1.0.

4.11.4 Local compression under wheels

The local compressive stress in the web due to a crane wheel load may be obtained by distributing it over a length x_R given by:

$$x_R = 2(H_R + T) \quad \text{but} \quad x_R \leq s_w$$

where

H_R is the rail height;

s_w is the minimum distance between centres of adjacent wheels;

T is the flange thickness.

Alternatively, where the properties of the rail are known:

$$x_R = K_R \left(\frac{I_f + I_R}{t}\right)^{1/3} \quad \text{but} \quad x_R \leq s_w$$

where

I_f is the second moment of area of the flange about its horizontal centroidal axis;

I_R is the second moment of area of the crane rail about its horizontal centroidal axis;

K_R is a constant;

t is the web thickness.

The constant K_R should be taken as follows:

a) when the crane rail is mounted directly on the beam flange: $K_R = 3.25$;

b) where a suitable resilient pad not less than 5 mm thick is interposed between the crane rail and the beam flange: $K_R = 4.0$.

The stress obtained by dispersing the wheel load over the length x_R should not be greater than p_{yw}.

4.11.5 Welded girders

Web to top flange welds should be continuous and should preferably be full penetration butt welds. They should be checked for the local effects of crane wheel loads by assuming that these are transmitted to the web by the welds alone, over a length x_R determined as in **4.11.4**, in addition to all other effects.

4.12 Purlins and side rails

4.12.1 General

Purlins and side rails may be designed on the assumption that the cladding provides lateral restraint to an angle section, or to the face against which it is connected in the case of members with other cross-sections, provided that the type of cladding and its fixings are such that it is capable of acting in this manner.

4.12.2 Deflections

The deflections of purlins and side rails should be limited to suit the characteristics of particular cladding.

4.12.3 Wind loading

Wind loading should be determined in accordance with BS 6399-2 or CP3:Ch V:Part 2. Where justified by sufficient general or particular evidence, the effects of load sharing with adjacent purlins and side rails, end fixity and end anchorage under wind loading, may be taken into account in determining the member capacity.

4.12.4 Empirical design of purlins and side rails

4.12.4.1 *General*

As an alternative to other methods, purlins and side rails comprising structural hollow sections or hot rolled angles may be designed using the empirical method given in **4.12.4.2** to **4.12.4.4**.

Such empirically designed purlins and side rails may be used to provide restraint to the members that directly support them, without needing to be checked for restraint forces

Purlins and side rails comprising cold formed sections should be designed in accordance with BS 5950-5.

4.12.4.2 ***Conditions***

For the empirical design method the following conditions should be met.

a) The members should be of steel to a minimum of grade S 275.

b) Unfactored loads should be used for empirical design.

c) The span of the members should not exceed 6.5 m centre-to-centre of main supports.

d) If the members generally span only one bay, each end should be connected by at least two bolts.

e) If the members are generally continuous over two or more bays, with staggered joints in adjacent lines of members, single bay members should have at least one end connected by two or more bolts.

4.12.4.3 ***Purlins***

Purlins satisfying **4.12.4.2** may be designed using the following empirical rules.

a) The slope of the roof should not exceed 30° from the horizontal.

b) The loading on the purlin should be substantially uniformly distributed. Not more than 10 % of the total roof load on the member should be due to other types of load.

c) The section modulus Z of a purlin about its axis parallel to the plane of the cladding should be not less than the larger of the two values Z_p and Z_q given in Table 27.

d) The member dimensions D perpendicular to the plane of the cladding, and (if applicable) B parallel to the plane of the cladding, should be not less than the respective values given in Table 27.

Table 27 — Empirical values for purlins

Purlin section	Z_p (cm³)	Z_q (cm³)		D (mm)	B (mm)
		Wind load from BS 6399-2	Wind load from CP3:Ch V:Part 2		
Angle	$W_pL/1\,800$	$W_qL/2\,250$	$W_qL/1\,800$	$L/45$	$L/60$
CHS	$W_pL/2\,000$	$W_qL/2\,500$	$W_qL/2\,000$	$L/65$	—
RHS	$W_pL/1\,800$	$W_qL/2\,250$	$W_qL/1\,800$	$L/70$	$L/150$

NOTE 1 W_p and W_q are the total unfactored loads (in kN) on one span of the purlin, acting perpendicular to the plane of the cladding, due to (dead plus imposed) and (wind minus dead) loading respectively.

NOTE 2 L is the span of the purlin (in mm) centre-to-centre of main vertical supports. However, if properly supported sag rods are used, L may be taken as the sag rod spacing in determining B only.

4.12.4.4 ***Side rails***

Side rails satisfying **4.12.4.2** may be designed using the following empirical rules.

a) The slope of the cladding should not exceed 15° from the vertical.

b) Side rails should not generally be subjected to loads other than wind load and the self-weight of the cladding. Not more than 10 % of the total load on the member about the axis under consideration should be due to loading from other sources or due to loads that are not uniformly distributed.

c) The elastic section moduli Z_1 and Z_2 of the side rail about its axes parallel to and perpendicular to the plane of the cladding respectively should be not less than the values given in Table 28.

d) The member dimensions D perpendicular to the plane of the cladding and B parallel to the plane of the cladding should be not less than the respective values given in Table 28, except that if Z_1 is larger than the tabulated minimum value, the tabulated minimum value of D may be reduced in the same proportion. However, in no case should D be less than the tabulated minimum value for B.

Table 28 — Empirical values for side rails

Side rail section	Z_1 (cm³)		Z_2 (cm³)	D (mm)	B (mm)
	Wind load from BS 6399-2	Wind load from CP3:Ch V:Part 2			
Angle	$W_1L/2\,250$	$W_1L/1\,800$	$W_2L/1\,200$	$L/45$	$L/60$
CHS	$W_1L/2\,500$	$W_1L/2\,000$	$W_2L/1\,350$	$L/65$	—
RHS	$W_1L/2\,250$	$W_1L/1\,800$	$W_2L/1\,200$	$L/70$	$L/100$

NOTE 1 W_1 and W_2 are the total unfactored loads (in kN) on one span of the side rail, acting perpendicular to the plane of the cladding and parallel to the plane of the cladding respectively.

NOTE 2 L is the span of the side rail (in mm), taken as follows:

a) for Z_1 and D: the span centre-to-centre of vertical supports;

b) for Z_2 and B: the span centre-to-centre of vertical supports, except that where properly supported sag rods are used L may be taken as the sag rod spacing.

4.13 Column bases

4.13.1 General

Column bases should be of sufficient size, stiffness and strength to transmit the axial force, bending moments and shear forces in columns to their foundations or other supports without exceeding the load carrying capacity of these supports. Holding-down bolts should be provided where necessary.

The nominal bearing pressure between a baseplate and a support may be determined on the basis of a uniform distribution of pressure. For concrete foundations the bearing strength may be taken as $0.6f_{cu}$ where f_{cu} is the characteristic cube strength of the concrete base or the bedding material, whichever is less.

Baseplates may be designed either by the effective area method given in **4.13.2** or by other rational means.

4.13.2 Effective area method

4.13.2.1 *Effective area*

If the size of a baseplate is larger than required to limit the nominal bearing pressure to $0.6f_{cu}$, see **4.13.1**, a portion of its area should be taken as ineffective, see Figure 15.

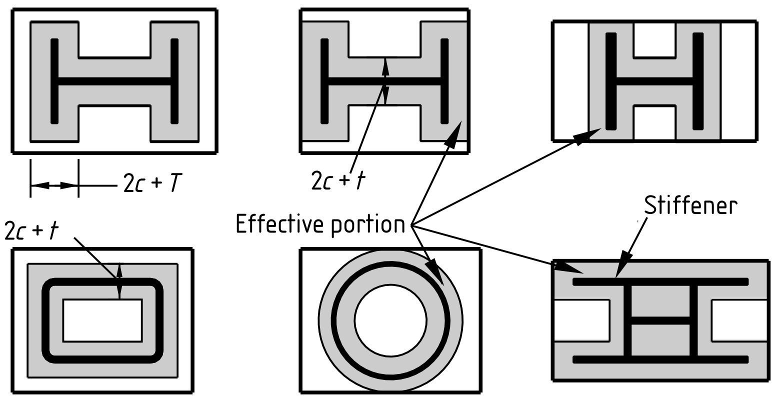


Figure 15 — Effective area of a baseplate

4.13.2.2 ***Axial forces***

For axial forces applied concentrically to the baseplate, the thickness of the baseplate should be not less than t_p given by:

$$t_p = c[3w/p_{yp}]^{0.5}$$

where

- c is the largest perpendicular distance from the edge of the effective portion of the baseplate to the face of the column cross-section, see Figure 15;
- p_{yp} is the design strength of the baseplate;
- T is the flange thickness (or maximum thickness) of the column;
- w is the pressure under the baseplate, based on an assumed uniform distribution of pressure throughout the effective portion.

If the baseplate is not concentric with the column, the moments in the baseplate due to axial load in the column should not exceed $p_{yp}Z_p$, where Z_p is the section modulus of the baseplate.

If stiffeners are used to transmit forces from the column to the baseplate, the projection c may be measured from the faces of the stiffeners, provided that they are designed for the resulting forces, see **4.13.2.5**.

4.13.2.3 ***Applied moments***

If moments are applied to the baseplate by the column, the moments in the baseplate should be calculated assuming a uniform pressure $w \leq 0.6f_{cu}$ under the effective portion of the compression zone and should not exceed $p_{yp}S_p$, where S_p is the plastic modulus of the baseplate.

In addition, the thickness of the baseplate should be not less than that required for axial load, see **4.13.2.2**.

4.13.2.4 ***Holding-down bolts***

Holding-down bolts should be checked for tension due to moments applied to the base by the column, using the tension capacity P_t given in **6.6**.

To avoid underestimating the tension in the holding-down bolts by overestimating the lever arm, the effective centre of compression under the baseplate should not be assumed to be located under an outstand of the baseplate, unless the moment in that outstand is limited to $p_{yp}Z_p$.

4.13.2.5 ***Stiffeners***

In a stiffened base, the moment in a stiffener due to the bearing pressure on the effective area used in the design of the baseplate should not exceed $p_{ys}Z_s$, where p_{ys} is the design strength of the stiffener and Z_s is its section modulus.

When the effective area of the baseplate is less than its gross area, the connections of the stiffeners should also be checked separately for the effects of a linear distribution of bearing pressure on the gross area as well as for the effects of the distribution used in the design of the baseplate and the stiffeners.

4.13.3 Connection of baseplates

Provided that the contact areas on the baseplate and the end of the column (including, in stiffened bases, the contact areas of the stiffeners) are in tight bearing contact, compression may be transmitted to the baseplate in direct bearing. Welds or bolts should be provided to transmit any shear or tension developed at the connection due to all realistic combinations of factored loads, see **2.2.1**.

Where the contact surfaces are not suitable to transmit compression in direct bearing, welds or bolts should be provided to transmit all forces and moments.

4.14 Cased sections

4.14.1 General

As an alternative to other methods, a section encased in concrete may be designed by the empirical methods given in **4.14.2**, **4.14.3** and **4.14.4** as appropriate, provided that it meets the following conditions.

a) The steel section should be either a single rolled or fabricated I or H-section with equal flanges, or a pair of rolled channels in contact back-to-back or separated back-to-back by not less than 20 mm nor more than half their depth. Double channel sections should satisfy the criteria given in **4.7.13.2** if in contact, otherwise they should be laced or battened to satisfy the criteria of **4.7.8** or **4.7.9** respectively.

b) The overall dimensions of the steel section should not exceed 1 000 mm × 500 mm, the dimension of 1 000 mm being measured parallel to the web or webs.

c) Primary structural connections to the member should preferably be made directly to the steel section. In such cases the eccentricity given in **4.7.6**a) should be taken from the face of the steel section.

d) The steel section should be unpainted and free from oil, grease, dirt and loose rust or millscale.

e) The steel section should be solidly encased in ordinary dense structural concrete with a 28 day cube strength of at least 25 N/mm^2.

f) There should be a minimum rectangle of solid casing (which may be chamfered at the corners) that gives a cover to the outer face and edges of the steel member of not less than 50 mm.

g) The concrete casing should extend the full length of the member and its connections. The concrete should be thoroughly compacted, especially below cleats, cap plates and beam soffits. There should be sufficient clearance at all points so that the concrete can be efficiently worked around the steel elements.

h) The casing should be reinforced using steel fabric complying with BS 4483, reference D 98. Alternatively, steel reinforcement or wire of not less than 5 mm diameter or their equivalent, complying with BS 4449 or BS 4482 may be used at a maximum spacing of 200 mm to form a cage of closed links and longitudinal bars. The reinforcement should be arranged to pass through the centre of the concrete cover to the flanges. The minimum lap of the reinforcement, and the details of the links, should conform to BS 8110.

i) The effective length L_E of the cased section should be limited to $40b_c$, $100b_c^2/d_c$ or $250r$ whichever is least, where:

b_c is the minimum width of solid casing within the depth of the steel section;

d_c is the minimum depth of solid casing within the width of the steel section;

r is the minimum radius of gyration of the steel section alone.

4.14.2 Cased columns

Cased columns that conform to the conditions given in **4.14.1** may be designed on the following basis.

a) The radius of gyration r_y of the member about its axis in the plane of its web or webs should be taken as $0.2b_c$ but not more than $0.2(B + 150)$ mm and not less than that of the steel section alone, where b_c is the minimum width of solid casing within the depth of the steel section and B is the overall width of the steel flange or flanges.

b) The radius of gyration r_x of the member about its axis parallel to the planes of the flanges should be taken as that of the steel section alone.

c) The compression resistance P_c of the cased section should be determined from:

$$P_c = (A_g + 0.45A_c f_{cu}/p_y)p_c \quad \text{but} \quad P_c \leq P_{cs}$$

in which P_{cs} is the short strut capacity of the cased section, given by:

$$P_{cs} = (A_g + 0.25A_c f_{cu}/p_y)p_y$$

where

A_c is the gross sectional area of the concrete, but neglecting any casing in excess of 75 mm from the overall dimensions of the steel section and neglecting any applied finish;

A_g is the gross sectional area of the steel member;

f_{cu} is the characteristic 28 day cube strength of the concrete, but $f_{cu} \leq 40$ N/mm²;

p_c is the compressive strength of the steel section, determined as given in **4.7.5** using r_y and r_x as defined in a) and b) and taking $p_y \leq 355$ N/mm²;

p_y is the design strength of the steel, but $p_y \leq 355$ N/mm².

4.14.3 Cased members subject to bending

Cased beams that satisfy the conditions given in **4.14.1** should be designed as for an uncased section (see **4.2** and **4.3**) except that the radius of gyration r_y may be taken as in **4.14.2**. All other properties should be taken as for the uncased section. The buckling resistance moment M_b should not exceed the moment capacity M_c of the uncased section nor 1.5 times the value of M_b for the uncased section.

In the calculation of deflections, the effective second moment of area of the cased section I_{cs} may be taken as that of the steel section plus the transformed net area of the concrete, i.e.:

$$I_{cs} = I_s + (I_c - I_s)E_c/E$$

where

E is the modulus of elasticity of steel;

E_c is the modulus of elasticity for the relevant grade of concrete, see BS 8110;

I_c is the second moment of area of the gross concrete cross-section;

I_s is the second moment of area of the steel member.

4.14.4 Cased members subject to axial force and moment

A cased section conforming to the conditions given in **4.14.1** and subject to combined axial compression and bending moment should satisfy the following relationships.

a) Cross-section capacity at locations of the largest bending moments and axial force:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

F_c is the compressive axial force at the critical location;

M_{cx} is the major axis moment capacity of the steel section, see **4.2.5**;

M_{cy} is the minor axis moment capacity of the steel section, see **4.2.5**;

M_x is the moment about the major axis at the critical location;

M_y is the moment about the minor axis at the critical location;

P_{cs} is the short strut capacity from **4.14.2**c).

b) Member buckling resistance:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_x}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

where

F_c is the maximum compressive axial force;

M_b is the buckling resistance moment from **4.3** using section properties as given in **4.14.3**;

M_x is the maximum moment about the major axis;

M_y is the maximum moment about the minor axis;

m_{LT} is the equivalent uniform moment factor for lateral-torsional buckling, see **4.8.3.3.4**;

m_x is the equivalent uniform moment factor for major axis buckling, see **4.8.3.3.4**;

m_y is the equivalent uniform moment factor for minor axis buckling, see **4.8.3.3.4**;

P_c is the compression resistance from **4.14.2**c), considering buckling about both axes;

P_{cy} is the compression resistance from **4.14.2**c), considering buckling about the minor axis only.

4.15 Web openings

4.15.1 General

Except as stipulated in **3.4** for bolt holes, the effects of openings should be taken into account in design. Appropriate reinforcement should be provided at all openings where the applied forces and moments exceed the capacity of the net cross-section, or the applied shear exceeds its shear capacity.

In addition to complying with **4.2**, **4.3**, **4.4**, **4.5**, **4.6**, **4.7**, **4.8** and **4.9** as relevant, members with web openings should comply with **4.15.2**, **4.15.3**, **4.15.4** and **4.15.5** as appropriate.

4.15.2 Isolated circular openings

4.15.2.1 *Unreinforced openings*

Isolated unreinforced circular openings may be located in the web of a beam without considering net section properties provided that.

a) The member has a class 1 plastic or class 2 compact cross-section.

b) The cross-section has an axis of symmetry in the plane of bending.

c) The openings are located within the middle third of the depth of the cross-section.

d) The openings are located within the middle half of the span of the member.

e) The spacing centre-to-centre of adjacent openings measured parallel to the axis of the member is not less than 2.5 times the diameter of the larger opening.

f) The distance from the centreline of each opening to the nearest point load is not less than the depth of the member.

g) The load on the member is substantially uniformly distributed.

h) The shear due to point loads does not exceed 10 % of the shear capacity of the cross-section.

i) The maximum shear in the member does not exceed 50 % of the shear capacity of the cross-section.

If the dimensions, openings or loading do not satisfy a) to i) the member should be designed using **4.15.3**.

4.15.2.2 *Reinforced openings*

Web reinforcement may be provided adjacent to openings to compensate for the material removed. It should be carried past the opening for such a distance that the local shear stress due to force transfer between the reinforcement and the web does not exceed $0.6p_y$.

Members with isolated reinforced openings should be designed using **4.15.3**.

4.15.3 Members with isolated openings

4.15.3.1 *General*

Members with isolated openings should satisfy **4.15.3.2**, **4.15.3.3**, **4.15.3.4**, **4.15.3.5** and **4.15.3.6**.

4.15.3.2 *Local buckling*

All compression elements of the cross-section, including those adjacent to web openings, should be checked for local buckling in accordance with **3.6**.

4.15.3.3 ***Shear***

The shear stress on the net shear area at openings should not exceed $0.6p_y$. In addition, the secondary Vierendeel moments due to shear forces at openings should be determined.

If the ratio d/t of the web exceeds 70ε for a rolled section, or 62ε for a welded section, allowance should be made for the influence of the web opening on the shear buckling resistance of the web.

NOTE Details of a design procedure allowing for this effect are given in reference [6], see Bibliography.

4.15.3.4 ***Moment capacity***

The moment capacity of the cross-section should be determined from the net section properties, allowing for the effects of secondary Vierendeel moments due to shear at web openings.

NOTE Details of a design procedure allowing for this effect are given in reference [6], see Bibliography.

4.15.3.5 ***Point loads***

Load bearing stiffeners should be provided where point loads are applied closer to an opening than the overall depth of the member. If a point load is applied within the length of an opening, the additional secondary moments should be taken into account in the design.

4.15.3.6 ***Deflection***

The additional deflection due to openings should be added to the primary deflection.

4.15.4 Members with multiple openings

4.15.4.1 ***General***

Members with multiple openings should satisfy **4.15.4.2**, **4.15.4.3**, **4.15.4.4**, **4.15.4.5**, **4.15.4.6**, **4.15.4.7** and **4.15.4.8**.

4.15.4.2 ***Local buckling***

All compression elements of the cross-section, including those adjacent to web openings, should be checked for local buckling in accordance with **3.6**.

4.15.4.3 ***Shear***

The shear stress based on the net shear area at openings in the cross-section should not exceed $0.6p_y$. The shear stress across a web post between two openings, based on the shear area of the web post at its narrowest point, should not exceed $0.7p_y$.

4.15.4.4 ***Moment capacity***

The moment capacity of the cross-section should be determined from the net section properties, allowing for the effects of secondary Vierendeel moments due to shear at web openings.

4.15.4.5 ***Buckling resistance moment***

Beams with incomplete lateral restraint should be designed in accordance with **4.3** using the section properties of the net cross-section and an equivalent slenderness λ_{LT} calculated using the section properties applicable at the centreline of an opening.

4.15.4.6 ***Deflection***

The additional deflection due to openings should be added to the primary deflection.

4.15.4.7 ***Resistance to concentrated loads***

At points of concentrated load or reaction, the resistance of the web should be checked using **4.5**. Where necessary, openings should be filled and stiffeners provided.

4.15.4.8 ***Web posts***

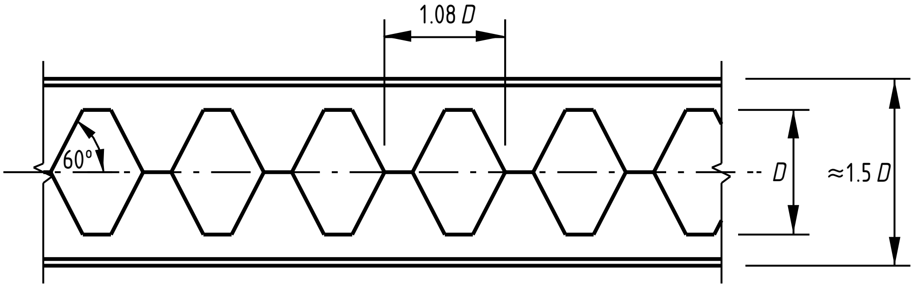
The stability of the web posts between the openings and at the ends of the member should be verified. Where necessary stiffeners should be provided.

NOTE A design method for checking the stability of web posts is given in reference [7], see Bibliography.

4.15.5 Castellated beams

In the case of castellated beams with the standard proportions shown in Figure 16, fabricated from rolled I or H-sections or from channels, it may be assumed that the web posts are stable provided that the ratio d/t for the web of the expanded cross-section does not exceed 70ε.

This should not be assumed to apply to members with other types of multiple opening, or to castellated beams with openings of different shapes or proportions.



D is the serial size of the original section

Figure 16 — Proportions of standard castellated members

4.16 Separators and diaphragms

Separators or diaphragms may be used to inter-connect I-section or channel beams, placed side by side, to enable them to act together in resisting applied forces, or to limit their relative deflections, as follows.

a) *Separators*. Separators, consisting of spacers and through bolts, may be used to transfer lateral forces between the beams, or to maintain their horizontal spacing in service.

b) *Diaphragms*. Diaphragms should be used where it is required to transfer vertical forces between the beams or to maintain their relative levels in service.

If it is required to increase the resistance to lateral forces or the lateral stiffness, the individual beams should be battened or laced to form a built-up member, or joined to form a compound member. Similar measures should be used if it is required to increase the total resistance to lateral-torsional buckling.

4.17 Eccentric loads on beams

4.17.1 General

Where a beam is eccentrically loaded in such a way that the torsional resistance of the beam is necessary to maintain equilibrium, the beam and its connections should be designed for the resulting torsion.

NOTE Guidance on design for torsion is given in reference [8], see Bibliography.

4.17.2 Distributed loads

Where a beam is loaded eccentrically by a wall, a leaf of a wall or any other distributed load, the beam and its connections should be designed for torsion, unless torsional deformation of the beam is restricted by suitable connection to a floor slab or another part of the structure capable of resisting the resulting moment.

4.17.3 Point loads

Where a beam is eccentrically loaded by point loads from other structural members that also restrict its torsional deformation, the beam need not be designed for torsion if the resulting moments can be resisted by the supported members and their connections.

Section 5. Continuous structures

5.1 General

5.1.1 Application

Section 5 applies to structures or elements of structures that are physically continuous over supports, or in which full continuity is provided by moment-resisting joints, see **6.1.5**.

The recommendations for frames with rigid moment-resisting joints apply to "first order" methods of global analysis (based on their initial un-deformed geometry), "second order" effects (stability effects due to deformations under load) being covered by recommendations for member buckling and frame stability.

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.

5.1.2 Pattern loading

5.1.2.1 *Dead load*

Dead load γ_f factors should be varied when considering overturning, uplift or sliding, see **2.4.2.2**. They should not be varied when considering pattern loading of imposed loads or wind load.

5.1.2.2 *Imposed floor load*

For load combination 1 (gravity loads, see **2.4.1.2**), the imposed floor load should be arranged in the most unfavourable but realistic pattern for each element.

5.1.2.3 *Imposed roof load*

For load combination 1 (gravity loads, see **2.4.1.2**), the imposed roof loads should not be patterned except as recommended in respect of partial loading, asymmetric loads and local drifting of snow in **4.5**, in **7.1**, **7.2** and **7.3** and in **7.4** respectively of BS 6399-3:1988.

NOTE In the case of local drifting a reduced value of γ_f applies, see Table 2.

5.1.2.4 *Wind load*

For load combination 2 (dead load and wind load, see **2.4.1.2**), the wind load should not be patterned except for the asymmetric loads recommended in **2.1.3.7** of BS 6399-2:1997.

5.1.2.5 *Wind load combined with imposed load*

For load combination 3 (dead load, imposed load and wind load, see **2.4.1.2**), pattern loading need not be applied.

5.1.3 Base stiffness

5.1.3.1 *General*

In determining the stiffness of a base, account should be taken of the behaviour of the ground, the stiffness of the foundation itself and the characteristics of the steel baseplate or other connection. The stiffness of a base with a pin or a rocker should be taken as zero.

In the absence of detailed knowledge of the stiffness of the base, design may be based on the assumptions detailed in **5.1.3.2**, **5.1.3.3** and **5.1.3.4**.

5.1.3.2 *Nominally rigid base*

If a column is rigidly connected to a suitable foundation, the following recommendations should be adopted.

a) In elastic global analysis the stiffness of the base should be taken as equal to the stiffness of the column for all ultimate limit state calculations. However, in determining deflections under serviceability loads, the base may be treated as rigid.

b) In plastic global analysis any base moment capacity between zero and the plastic moment capacity of the column may be assumed, provided that the foundation is designed to resist a moment equal to this assumed moment capacity, together with the forces obtained from the analysis. In elastic-plastic global analysis the assumed base stiffness should be consistent with the assumed base moment capacity, but should not exceed the stiffness of the column.

5.1.3.3 *Nominally pinned base*

If a column is nominally pin-connected to a foundation that is designed assuming that the base moment is zero, the base should be assumed to be pinned when using elastic global analysis to calculate the other moments and forces in the frame under ultimate limit state loading.

The stiffness of the base may be assumed to be equal to the following proportion of the column stiffness:

a) 10 % when checking frame stability or determining in-plane effective lengths;

b) 20 % when calculating deflections under serviceability loads.

5.1.3.4 *Nominal semi-rigid base*

A nominal base stiffness of up to 20 % of the stiffness of the column may be assumed in elastic global analysis, provided that the foundation is designed for the moments and forces obtained from this analysis.

5.1.4 Independently braced frames

Where sway stability (see **2.4.2.5**) is provided to a frame with moment-resisting joints by an independent system of resistance to horizontal forces (see **2.4.2.3**) it may be treated as "non-sway" (see **2.4.2.6**) if:

a) the stabilizing system has a spring stiffness (horizontal reaction per unit displacement) at least four times larger than the total spring stiffness of all the frames to which it gives horizontal support (i.e. the supporting system reduces the horizontal displacement of the frames by at least 80 %);

b) the stabilizing system is designed to resist all the horizontal loads applied to the frame, including the notional horizontal forces, see **2.4.2.4**.

5.2 Global analysis

5.2.1 Methods

Either elastic or plastic global analysis may be used. Elastic analysis should normally be first order linear elastic. Plastic analysis should normally be first order rigid-plastic or first order elastic-plastic (either linear elastic-plastic or the elasto-plastic "plastic zones" method). The use of second order elastic or second order elastic-plastic methods is not precluded, but no detailed recommendations are given for their use, see **5.1.1**.

5.2.2 Elastic analysis

When elastic global analysis is used for a continuous beam or a moment-resisting frame, the moments in a member, calculated by elastic frame analysis, may be modified by redistributing up to 10 % of the peak calculated moment in that member for the same load combination, provided that:

a) the forces and moments in the frame remain in equilibrium with the applied loads;

b) the members in which moments are reduced have class 1 plastic or class 2 compact cross-sections;

c) moments are not reduced about the minor axis of any column.

5.2.3 Plastic analysis

5.2.3.1 *General*

Plastic global analysis may be used for structures or elements of structures that meet the conditions given in **5.2.3.2**, **5.2.3.3**, **5.2.3.4**, **5.2.3.5**, **5.2.3.6**, **5.2.3.7** and **5.2.3.8**. Members containing plastic hinges in one plane should not also be used to resist moments in another plane under the same load case.

The in-plane stability of the members of a continuous frame designed using plastic analysis should be established by checking the in-plane stability of the frame itself, see **5.5.4**.

The out-of-plane stability of members designed using plastic analysis should be ensured by providing lateral restraints in accordance with **5.3**.

5.2.3.2 *Type of loading*

Plastic global analysis may be used where the loading is predominantly static. It should not be used where fatigue is a design criterion.

5.2.3.3 ***Grades of steel***

Plastic global analysis may be used for all the grades of steel listed in BS 5950-2. It may also be used for other steel grades that satisfy the following additional criteria.

a) The ultimate tensile strain is at least 20 times the yield strain.

b) The ultimate tensile strength is at least 1.2 times the yield strength.

c) The elongation on a gauge length of $5.65\sqrt{A_o}$ is at least 15 %, where A_o is the original area.

5.2.3.4 ***Fabrication restrictions***

For a length along the member each side of a plastic hinge location equal to the overall depth D at that location, the following restrictions should be applied to the tension flange and noted in the design documents.

a) Holes should either be drilled full size, or punched at least 2 mm undersize and then reamed.

b) All sheared or hand flame cut edges should be finished smooth by grinding, chipping or planing.

5.2.3.5 ***Cross-section restrictions***

All members containing plastic hinge locations should have class 1 plastic cross-sections, at the plastic hinge location. In addition, the cross-section should be symmetrical about its axis perpendicular to the axis of plastic hinge rotation.

Members with cross-sections that vary along their length should also satisfy the following criteria.

a) Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance along the member from the plastic hinge location of at least $2d$, where d is the clear depth of the web at the plastic hinge location.

b) Adjacent to plastic hinge locations, the compression flange should be class 1 plastic for distances each way along the member from the plastic hinge location of not less than the greater of:

— $2d$, where d is as defined in a);

— the distance to the adjacent point at which the moment in the member has fallen to 0.8 times the reduced plastic moment capacity of the cross-section at the point concerned.

c) Elsewhere, the compression flange should be class 1 plastic or class 2 compact and the web should be class 1 plastic, class 2 compact or class 3 semi-compact.

5.2.3.6 ***Bolt holes***

The conditions given in **4.2.5.5** for bolt holes for which no allowance need be made, should be met at plastic hinge locations and up to the adjacent points at which the moment in the member has fallen to 80 % of the reduced plastic moment capacity of the cross-section at the point concerned.

5.2.3.7 ***Stiffeners at plastic hinge locations***

Web stiffeners should be provided where a force that exceeds 10 % of the shear capacity of the cross-section (see **4.2.3**) is applied to the web within a distance $D/2$ of a plastic hinge location, measured along the member, where D is its overall depth at that location. These stiffeners should be designed in accordance with **4.5.2.2** and **4.5.3.3** and should be provided no further than $D/2$ along the member from the hinge location and from the point where the force is applied.

If the stiffeners are flat plates, the outstand to thickness ratio b_s/t_s should not exceed 9. Where sections are used, they should satisfy the condition:

$$(I_{so}/J_s)^{0.5} \le 9$$

where

I_{so} is the second moment of area of the stiffener about the face of the web;

J_s is the torsion constant of the stiffener.

5.2.3.8 ***Haunches***

Haunches should be proportioned to avoid plastic hinges forming within their length.

5.3 Stability out-of-plane for plastic analysis

5.3.1 General

Members and segments containing plastic hinges should satisfy the recommendations given in **5.3.2**, **5.3.3**, **5.3.4** and **5.3.5**, except that these recommendations need not be applied at plastic hinge locations where it can be demonstrated that, under all load combinations, the plastic hinge is "non-rotated", because under that load combination it is the last hinge to form or it is not yet fully formed.

Other than where **5.3.4** applies, the spacing of lateral restraints to a member or segment not containing a plastic hinge should be such that **4.8.3.3** or **I.1** is satisfied for buckling out-of-plane, except that this spacing need not be less than L_m determined from **5.3.3**.

If for any reason the plastic load factor λ_p is more than the required load factor λ_r for the load case under consideration, the resistance of a member or segment to out-of-plane buckling should be checked using moments and forces corresponding to λ_r rather than λ_p. This may be done by either:

— modifying the moments and forces from a plastic analysis by multiplying them by λ_r / λ_p;

— using elastic-plastic analysis to determine the moments and forces at a load factor of λ_r.

5.3.2 Restraints at plastic hinges

Under all ultimate limit state load combinations, both flanges should have lateral restraint at each plastic hinge location, designed to resist a force equal to 2.5 % of the force in the compression flange. Where it is not practicable to provide such restraint directly at the hinge location, it should be provided within a distance $D/2$ along the length of the member, where D is its overall depth at the plastic hinge location.

For three-flange haunches reference should also be made to **5.3.5.1**.

5.3.3 Segment adjacent to a plastic hinge

Except where **5.3.4** applies, the length of a segment adjacent to a plastic hinge location, between points at which the compression flange is laterally restrained, should not exceed L_m calculated as follows.

a) *Conservative method*: L_m may be taken as equal to L_u obtained from:

$$L_u = \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{x}{36}\right)^2 \left(\frac{p_y}{275}\right)^2\right]^{0.5}}$$

where

f_c is the compressive stress (in N/mm²) due to axial force;

p_y is the design strength (in N/mm²);

r_y is the radius of gyration about the minor axis;

x is the torsional index, see **4.3.6.8**.

Where the member has unequal flanges r_y should be taken as the lesser of the values for the compression flange only or the whole section.

Where the cross-section of the member varies within the length between restraints, the minimum value of r_y and the maximum value of x should be used.

b) *Approximate method allowing for moment gradient*: For I-section members of uniform cross-section with equal flanges and $D/B \geq 1.2$, where D is the overall depth and B is the flange width, in steel grade S 275 or S 355, in which f_c does not exceed 80 N/mm²:

$$L_m = \phi L_u$$

in which L_u is given in a) and ϕ is given by the following:

— for $1 \geq \beta \geq \beta_u$: $\phi = 1$

— for $\beta_u > \beta > 0$: $\phi = 1 - (1 - KK_o)(\beta_u - \beta)/\beta_u$

— for $0 \geq \beta > -0.75$: $\phi = K(K_o - 4(1 - K_o)\,\beta/3$

— for $\beta \leq -0.75$: $\phi = K$

where β is the end moment ratio, using the same sign convention as in Table 18.

The limiting value β_u should be determined from the following:

— *for steel grade S 275:* $\beta_u = 0.44 + \frac{x}{270} - \frac{f_c}{200}$

— *for steel grade S 355:* $\beta_u = 0.47 + \frac{x}{270} - \frac{f_c}{250}$

The coefficients K_o and K should be obtained as follows:

$K_o = (180 + x)/300$

— for $20 \le x \le 30$: $K = 2.3 + 0.03x - xf_c/3\,000$

— for $30 \le x \le 50$: $K = 0.8 + 0.08x - (x - 10)f_c/2\,000$

in which f_c and x are as defined in a).

NOTE This approximation is based on the method given in reference [9], see Bibliography.

5.3.4 Member or segment with one flange restrained

The following approach may be used for a member or segment that has a laterally unrestrained compression flange, provided that the other flange has intermediate lateral restraint at intervals such that the following conditions are satisfied:

a) adjacent to plastic hinge locations, the spacing of the intermediate lateral restraints should not exceed the value of L_m determined from **5.3.3**;

b) elsewhere **4.8.3.3** or **I.1** should be satisfied for out-of-plane buckling when checked using an effective length L_E equal to the spacing of the intermediate lateral restraints, except that this spacing need not be less than L_m determined from **5.3.3**.

Where these conditions are satisfied, the spacing of restraints to the compression flange should be such that:

— adjacent to plastic hinge locations the out-of-plane buckling resistance satisfies **G.3.3**;

— elsewhere **G.2** is satisfied for out-of-plane buckling.

As an alternative to satisfying **G.3.3** and **G.2**, the simple method given below may be used, provided that:

— conditions a) and b) given above are satisfied;

— the member is an I-section with $D/B \ge 1.2$, where D is the depth and B is the flange width;

— for haunched segments D_h is not greater than $2D_s$, see Figure 17;

— for haunched segments, the haunch flange is not smaller than the member flange;

— the steel is grade S 275 or grade S 355.

In the simple method, the spacing L_y between restraints to the compression flange should not exceed the limiting spacing L_s given conservatively by the following:

— *for steel grade S 275:*

$$L_s = \frac{620r_y}{K_1[72 - (100/x)^2]^{0.5}}$$

— *for steel grade S 355:*

$$L_s = \frac{645r_y}{K_1[94 - (100/x)^2]^{0.5}}$$

where

r_y is the minor axis radius of gyration of the un-haunched rafter;

x is the torsional index, see **4.3.6.8**, of the un-haunched rafter;

and K_1 has the following values, see Figure 17:

— for an un-haunched segment: $K_1 = 1.00$;

— for a haunch with $D_h/D_s = 1$: $K_1 = 1.25$;

— for a haunch with $D_h/D_s = 2$: $K_1 = 1.40$;

— for a haunch generally: $K_1 = 1 + 0.25(D_h/D_s)^{2/3}$.

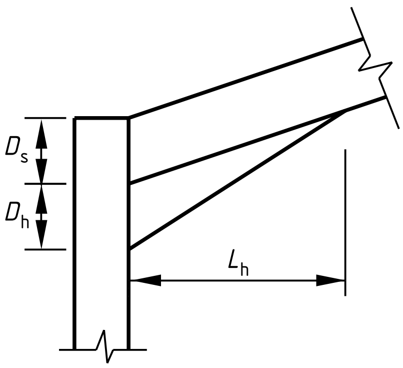


Figure 17 — Dimensions of a haunch

5.3.5 Haunches

5.3.5.1 *Three-flange haunches*

Where a plastic hinge location occurs immediately adjacent to one end of a three-flange haunch (see **G.1.2**), the tapered segment need not be treated as a segment adjacent to a plastic hinge location if both of the following criteria are satisfied:

— both flanges have lateral restraint in accordance with **5.3.2** at the plastic hinge location itself or within a distance $D/2$ along the length of the tapered segment only, not the uniform segment;

— the depth of the haunch is sufficient for the tapered segment to remain elastic throughout its length.

5.3.5.2 *Two-flange haunches*

Where a plastic hinge location occurs immediately adjacent to one end of a two-flange haunch (see **G.1.2**), the tapered segment should satisfy one of the following criteria:

— the moment at the adjacent lateral restraint does not exceed 85 % of the reduced plastic moment capacity, reduced to allow for axial load;

— the length L_y to the adjacent lateral restraint to the compression flange does not exceed 85 % of the limiting length L_m from **5.3.3** or L_s from **5.3.4** or **G.3** for a segment with one flange restrained.

5.4 Continuous beams

5.4.1 Elastic design

If elastic global analysis is used for a continuous beam, the moment capacity of the cross-section and the buckling resistance moment of the beam should be obtained using **4.2** and **4.3** or, in appropriate cases, **G.2**.

5.4.2 Plastic design

Plastic global analysis may be used for a continuous beam, provided that the conditions given in **5.2.3** are satisfied. In addition, the out-of-plane stability of the beam should satisfy **5.3**, taking λ_r as 1.0.

5.5 Portal frames

5.5.1 General

Either elastic or plastic analysis, see **5.2**, may be used for single-storey frames with rigid moment-resisting joints. All load combinations should be covered, including both uniform and non-uniform imposed roof loads. Notional horizontal forces should be applied when checking load combination 1 (gravity loads, see **2.4.1.2**). In addition, the frame should be stabilized against sway out-of-plane, see **2.4.2.5**.

Other frames with sloping members and moment-resisting joints may also be treated like portal frames.

5.5.2 Elastic design

If elastic global analysis is used for a portal frame, the cross-section capacity should be checked using **4.8.1** and **4.8.3.2** and the out-of-plane buckling resistance should be checked using **4.8.3.3** or **I.1**, or in appropriate cases, **G.2**.

For independently braced frames, see **5.1.4**, the in-plane member buckling resistances should also be checked using **4.8.3.3**, with in-plane effective lengths obtained using **E.4.2**.

In all other cases, the in-plane stability of the frame should be verified by checking the cross-section capacity and out-of-plane buckling resistance of the members using amplified moments and forces, taken as the values given by linear elastic analysis multiplied by the required load factor λ_r from **5.5.4**.

5.5.3 Plastic design

Plastic global analysis may be used for a portal frame provided that the conditions given in **5.2.3** are satisfied. Multi-bay frames should also be checked for localized failure mechanisms.

The in-plane stability of the frame should be verified by checking that the plastic load factor λ_p satisfies:

$$\lambda_p \geq \lambda_r$$

where λ_r is the required load factor from **5.5.4** for the relevant load combination.

The out-of-plane stability of the members should be checked as detailed in **5.5.5**.

5.5.4 In-plane stability

5.5.4.1 *General*

The in-plane stability of a portal frame should be checked under each load combination. Except for a tied portal, one of the following should be used:

a) the sway-check method given in **5.5.4.2**, together with the snap-through check given in **5.5.4.3**;

b) the amplified moments method given in **5.5.4.4**;

c) second order analysis, see **5.5.4.5**.

A tied portal should be checked as recommended in **5.5.4.6**.

5.5.4.2 *Sway-check method*

5.5.4.2.1 *General*

The sway-check method may be used to verify the in-plane stability of portal frames in which each bay satisfies the following conditions:

a) the span L does not exceed 5 times the mean height h of the columns;

b) the height h_r of the apex above the tops of the columns does not exceed 0.25 times the span L;

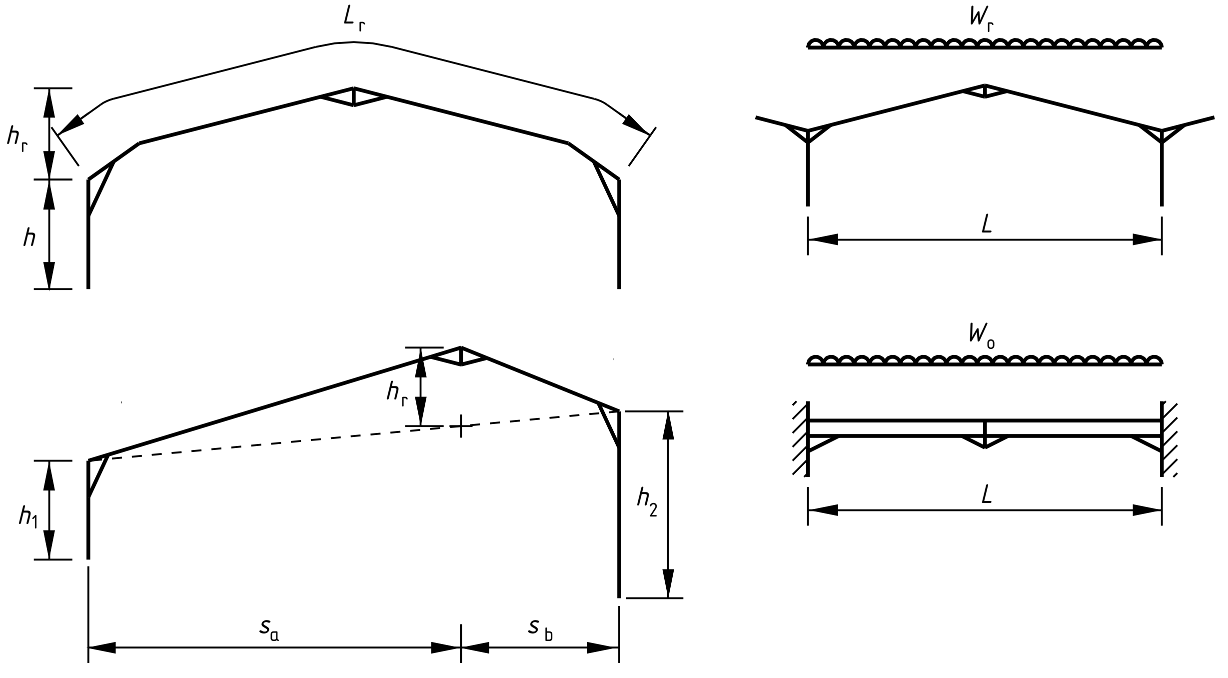
c) if the rafter is asymmetric h_r satisfies the criterion:

$$(h_r/s_a)^2 + (h_r/s_b)^2 \leq 0.5$$

in which s_a and s_b are the horizontal distances from the apex to the columns, see Figure 18a).

Provided that these conditions are met, linear elastic analysis should be used to calculate the notional horizontal deflections at the top of each column due to a set of notional horizontal forces applied in the same direction to each column and equal to 0.5 % of the vertical reaction at the base of the respective column for the relevant load case.

Generally, these notional horizontal forces should be applied at the tops of the respective columns. However, in the case of columns supporting loads from crane gantries, or other significant vertical loads applied within their height, the notional horizontal forces derived from such loads should be applied to the column at the same level as the relevant vertical load.



a) Dimensions b) Arching ratio

Figure 18 — Portal frame definitions

5.5.4.2.2 *Gravity loads*

For gravity loads (load combination 1, see **2.4.1.2**), the notional horizontal deflections should be determined without any allowance for the stiffening effects of cladding. If the notional horizontal deflections δ_i at the tops of the columns do not exceed $h_i/1\,000$, where h_i is the height of that column, then for gravity loads the required load factor λ_r for frame stability should be taken as 1.0.

Provided that the frame is not subject to loads from valley beams or crane gantries or other concentrated loads larger than those from purlins, the $h_i/1\,000$ sway criterion for gravity loads may be assumed to be satisfied if in each bay:

$$\frac{L_b}{D} \le \frac{44L}{\Omega h}\left(\frac{\rho}{4 + \rho L_r/L}\right)\left(\frac{275}{p_{yr}}\right)$$

in which

$$L_b = L - \left(\frac{2D_h}{D_s + D_h}\right)L_h$$

$$\rho = \left(\frac{2I_c}{I_r}\right)\left(\frac{L}{h}\right) \quad \text{for a single bay frame;}$$

$$\rho = \left(\frac{I_c}{I_r}\right)\left(\frac{L}{h}\right) \quad \text{for a multi-bay frame;}$$

and Ω is the arching ratio, given by:

$$\Omega = W_r/W_o$$

where

D is the cross-section depth of the rafter;

D_h is the additional depth of the haunch, see Figure 17;

D_s is the depth of the rafter, allowing for its slope, see Figure 17;

h is the mean column height;

I_c is the in-plane second moment of area of the column (taken as zero if the column is not rigidly connected to the rafter, or if the rafter is supported on a valley beam);

I_r is the in-plane second moment of area of the rafter;

L is the span of the bay;

L_b is the effective span of the bay;

L_h is the length of a haunch, see Figure 17;

L_r is the total developed length of the rafters, see Figure 18a);

p_{yr} is the design strength of the rafters in N/mm^2;

W_o is the value of W_r for plastic failure of the rafters as a fixed-ended beam of span L;

W_r is the total factored vertical load on the rafters of the bay, see Figure 18b).

If the two columns or the two rafters of a bay differ, the mean value of I_c/I_r should be used.

If the haunches at each side of the bay are different, the mean value of L_b should be used.

5.5.4.2.3 *Horizontal loads*

For load combinations that include wind loads or other significant horizontal loads, allowance may be made for the stiffening effects of cladding in calculating the notional horizontal deflections δ_i, see **5.5.4.2.1**.

Provided that the $h_i/1\,000$ sway criterion is satisfied for gravity loads, then for load cases involving horizontal loads the required load factor λ_r for frame stability should be determined using:

$$\lambda_r = \frac{\lambda_{sc}}{\lambda_{sc} - 1}$$

in which λ_{sc} is the smallest value, considering every column, determined from:

$$\lambda_{sc} = \frac{h_i}{200\delta_i}$$

using the notional horizontal deflections δ_i for the relevant load case.

If $\lambda_{sc} < 5.0$ second order analysis should be used.

Provided that the frame is not subject to loads from valley beams or crane gantries or other concentrated loads larger than those from purlins, then λ_{sc} may be approximated using:

$$\lambda_{sc} = \frac{220DL}{\Omega h L_b}\left(\frac{\rho}{4 + \rho L_r/L}\right)\left(\frac{275}{p_{yr}}\right)$$

If the wind loads are such that the axial forces are tensile in all rafters and columns, then the required load factor λ_r should be taken as 1.0.

5.5.4.3 ***Snap-through***

In each internal bay of a single-storey frame with three or more bays the rafter should satisfy the following:

$$\frac{L_b}{D} \leq \frac{22(4 + L/h)}{4(\Omega - 1)}\left(1 + \frac{I_c}{I_r}\right)\left(\frac{275}{p_{yr}}\right)\tan 2\theta$$

If the arching ratio Ω is less than one, no limit need be placed on L_b/D.

For a symmetrical ridged bay θ should be taken as the slope of the rafters. For other roof shapes the value of θ should be determined from:

$$\theta = \tan^{-1}(2h_r/L)$$

5.5.4.4 ***Amplified moments method***

For each load case the in-plane stability of a portal frame may be checked using the lowest elastic critical load factor λ_{cr} for that load case. This should be determined taking account of the effects of all the members on the in-plane elastic stability of the frame as a whole.

NOTE Information on determining λ_{cr} for a portal frame is given in reference [10], see Bibliography.

In this method, the required load factor λ_r for frame stability should be determined from the following:

— if $\lambda_{cr} \geq 10$: $\lambda_r = 1.0$

— if $10 > \lambda_{cr} \geq 4.6$: $\lambda_r = \dfrac{0.9\lambda_{cr}}{\lambda_{cr} - 1}$

If $\lambda_{cr} < 4.6$ the amplified moments method should not be used.

5.5.4.5 ***Second-order analysis***

The in-plane stability of a portal frame may be checked using either elastic or elastic-plastic second order analysis. When these methods are used the required load factor λ_r for frame stability should be taken as 1.0.

NOTE Guidance on an appropriate method is given in reference [10], see Bibliography.

5.5.4.6 ***Tied portals***

The in-plane stability of a tied portal should be checked using elastic or elastic-plastic second order analysis. The required load factor λ_r for frame stability should be taken as 1.0.

NOTE Guidance on an appropriate method is given in reference [10], see Bibliography.

The method used should allow for the increase in the tie force due to the reduction in the lever arm from the apex to the tie, caused by extension of the tie and deformation of the rafter, unless the tie is supported by a hanger designed to avoid reducing this lever arm. To make allowance for the effects of plasticity when elastic-plastic analysis is used, in the absence of a more exact analysis the total reduction of the lever arm may be taken as twice that predicted by linear elastic analysis.

5.5.5 Out-of-plane stability

The out-of-plane stability of all frame members should be ensured under all load cases, not just the critical load case for the plastic resistance of the frame members. Where differential settlement of foundations is a design criterion, this should be taken into account in checking out-of-plane stability.

Lateral restraints should be provided in accordance with **5.3**. The restraints or virtual restraints to the bottom flange of the rafter shown in Figure 19 should extend up to or beyond the point of contraflexure.

If the purlins and their connections to the rafter are capable of providing torsional restraint to the top flange of the rafter, an allowance for this torsional restraint may be made by assuming a virtual lateral restraint to the bottom flange at the point of contraflexure, whether or not the top flange is restrained at this point.

This virtual restraint should not be assumed if another form of allowance is made for the torsional restraint of the top flange by the purlins.

Torsional restraint of the top flange by the purlins may be assumed if the following criteria are all satisfied.

a) The rafter is an I-section with $D/B \geq 1.2$, where D is the depth and B is the flange width.

b) For haunched rafters D_h is not greater than $2D_s$, see Figure 17.

c) Every length of purlin has at least two bolts in each purlin-to-rafter connection.

d) The depth of the purlin section is not less than 0.25 times the depth D of the rafter.

Lateral restraint of the bottom flange should not be assumed at the point of contraflexure under other restraint conditions, unless a lateral restraint is actually provided at that point.

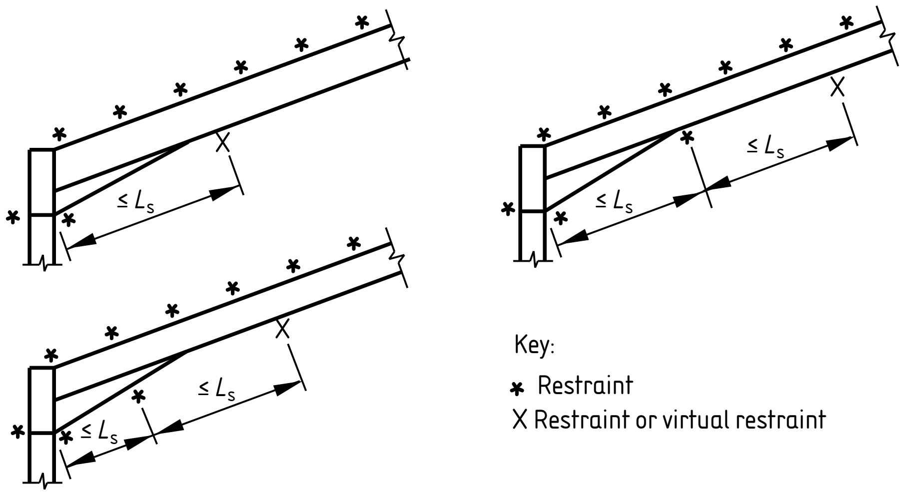


Figure 19 — Haunch restraints

5.6 Elastic design of multi-storey rigid frames

5.6.1 General

If elastic global analysis is used for a multi-storey frame with rigid moment-resisting joints, the cross-section capacity should be checked using **4.8.1** and **4.8.3.2** and the buckling resistance should be checked using **4.8.3.3** or **I.1** or, in appropriate cases, **G.2**.

The load cases to be checked and the in-plane effective lengths to be taken for the columns should be determined from **5.6.2**, **5.6.3** or **5.6.4** as relevant.

5.6.2 Independently braced frames

Independently braced frames, see **5.1.4**, should be designed to resist gravity loads (load combination 1, see **2.4.1.2**). The non-sway mode in-plane effective lengths of the columns should be used, see Annex E.

The maximum moments in the beams and the combinations of axial force and moments in the columns that give the worst cases for cross-section capacity (see **4.8.3.2**) and for member buckling resistance (see **4.8.3.3**), should be determined by applying both full and pattern loading of imposed load.

In order to reduce the number of load cases, suitable sub-frames may be used for pattern loading.

NOTE Information on sub-frames is given in reference [11], see Bibliography.

5.6.3 Non-sway frames

Non-sway frames, see **2.4.2.6**, should be designed to resist gravity loads (load combination 1, see **2.4.1.2**), as for independently braced frames, see **5.6.2**. They should also be checked for combined vertical and horizontal loads (load combinations 2 and 3, see **2.4.1.2**) without pattern loading. The non-sway mode in-plane effective lengths of the columns should be used, see Annex E.

5.6.4 Sway-sensitive frames

Sway-sensitive frames, see **2.4.2.7**, should initially be designed to resist gravity loads (load combination 1, see **2.4.1.2**), as for independently braced frames, see **5.6.2**, without taking account of sway.

Sway-sensitive frames should then be checked in the sway mode by applying the notional horizontal forces, see **2.4.2.4**, together with the full gravity load (load combination 1, see **2.4.1.2**) without any pattern loading. They should also be checked in the sway mode for combined vertical and horizontal loads (load combinations 2 and 3, see **2.4.1.2**) without pattern loading.

Provided that $\lambda_{cr} \geq 4.0$ sway should be allowed for by using one of the following methods.

a) *Effective length method*: In this method, sway mode in-plane effective lengths, see Annex E, should be used for the columns. The beams should be designed to remain elastic under the factored loads.

b) *Amplified sway method*: The sway moments, see **2.4.2.8**, should be multiplied by the amplification factor k_{amp} from **2.4.2.7** and the internal forces adjusted to maintain equilibrium with the applied loads. In this method, non-sway mode in-plane effective lengths, see Annex E, should be used for the columns.

If λ_{cr} is less than 4.0, second order elastic analysis should be used to allow for sway.

5.7 Plastic design of multi-storey rigid frames

5.7.1 General

If plastic global analysis is used for a multi-storey frame with rigid moment-resisting joints, the conditions for plastic analysis given in **5.2.3** should be satisfied. In addition, the frame should be stabilized against sway out-of-plane, see **2.4.2.5**.

Members should be checked for the forces and moments determined as given in **5.7.2** or **5.7.3** as relevant. The cross-section capacity should be determined using **4.8.1** and **4.8.3.2**. Out-of-plane buckling of members containing plastic hinges should be prevented by providing restraints as recommended in **5.3**. Out-of-plane buckling of other members should be checked as detailed in **4.8.3.3** or **I.1** or, in appropriate cases, **G.2**. The resistance of the columns to in-plane buckling should be checked as given in **5.7.2** or **5.7.3**, as appropriate.

5.7.2 Independently braced frames

Independently braced frames, see **5.1.4**, should be designed to resist gravity loads (load combination 1, see **2.4.1.2**). In checking the columns for resistance to in-plane member buckling, the effective length L_E in the plane of the frame should generally be taken as equal to the storey height L. However, if some of the beams in that plane have been designed to remain elastic, the in-plane effective length L_E for the non-sway mode may be determined from Annex E.

Columns should also be checked under the combination of axial force and moments that gives the worst case for member buckling, determined by applying pattern loading to the imposed load. In this check the in-plane effective length for the non-sway mode should be determined from Annex E. An appropriate sub-frame may be used to take account of pattern loading.

NOTE Information on this application of sub-frames is given in reference [12], see Bibliography.

5.7.3 Unbraced frames

5.7.3.1 *General*

Except for frames that satisfy the frame stability check given in **5.7.3.2**, all plastically designed multi-storey frames that are not independently braced against sway should be designed to resist sway mode failure using either elastic analysis (see **5.6**) or second order elastic-plastic analysis (see **5.2.1**).

Multi-storey frames that are not independently braced should also be checked for possible non-sway modes of failure as recommended for independently braced frames in **5.7.2**.

5.7.3.2 *Frame stability check*

The use of this simplified check for frame stability should be limited to frames that also satisfy the following conditions.

a) The bases of the columns should be fixed (but see also **5.1.3**).

b) The plastic hinge mechanism should be a sway mode, with plastic hinges assumed in all the beams and at the base of each column, but no other hinges in the columns.

c) It should be ensured that no localized beam or storey-height plastic hinge mechanisms can form at a lower load factor than the overall frame mechanism.

d) The storey height of the frame should nowhere exceed the mean spacing of its columns in that storey.

e) The lower lengths of the columns should be designed to remain elastic under the theoretical plastic hinge moments at the bases assumed in a).

The elastic critical load factor λ_{cr} should be determined from **F.2**, taking into account the base stiffness, determined as detailed in **5.1.3**.

The plastic load factor λ_p should not be less than the required load factor λ_r for frame stability given by the following:

a) 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 taken into account:

— if $\lambda_{cr} \geq 10$: $\lambda_r = 1.0$

— if $10 > \lambda_{cr} \geq 4.6$: $\lambda_r = \dfrac{0.9\lambda_{cr}}{\lambda_{cr} - 1}$

b) 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 taken into account:

— if $\lambda_{cr} \geq 20$: $\lambda_r = 1.0$

— if $20 > \lambda_{cr} \geq 5.75$: $\lambda_r = \dfrac{0.9\lambda_{cr}}{\lambda_{cr} - 1}$

If λ_{cr} is less than 4.6 for case a) or 5.75 for case b), either elastic analysis or second order elastic-plastic analysis (see **5.2.1**) should be used.

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Section 6. Connections

6.1 General recommendations

6.1.1 General

Joints should be designed on the basis of realistic assumptions of the distribution of internal forces. These assumptions should correspond with direct load paths through the joint, taking account of the relative stiffnesses of the various components of the joint. In all cases, equilibrium should be maintained between the internal forces and the external applied loads.

Where other members are connected to the surface of a web or flange of a member, the ability of the web or flange to transfer the applied forces should be checked.

Ease of fabrication and erection should also be taken into account in the design of connections and splices. Attention should be paid to clearances necessary for tightening bolts (particularly for preloaded bolts), welding procedures, subsequent inspection, surface treatment and maintenance.

Because the ductility of structural steel assists the distribution of forces generated within a joint, residual stresses and stresses due to tightening of bolts and imperfect fit-up need not normally be calculated.

As non-preloaded bolts in clearance holes generally slip before starting to transfer load in shear, they should not be assumed to share load with welds or preloaded bolts and one form of connection should normally be designed to carry the total load. However, preloaded bolts designed to be non-slip under factored loads may be designed to share load with welds, provided that the final tightening is done after welding.

6.1.2 Detailing

The connections between members should be capable of withstanding the forces and moments to which they are subjected, within acceptable deformation limits and without invalidating the design assumptions.

The detailing of connections should take account of possible dimensional variations due to rolling margins and fabrication variations, leading to some degree of lack of fit.

6.1.3 Intersections

Where there is eccentricity at intersections, the members and connections should be designed to accommodate the resulting moments, forces, deflections and rotations. In the case of bolted framing consisting of angles and T-sections, the intersections of the setting-out lines of the bolts may be adopted instead of the intersections of the centroidal axes.

In joints involving structural hollow sections, limited eccentricity between member intersections should be introduced where necessary to suit other features of connection design, see **6.7.3.3**.

6.1.4 Joints in simple design

In simple design, joints between members should be capable of transmitting the calculated forces and should also be capable of accepting the resulting rotation, see **2.1.2.2**. They should not develop significant moments that adversely affect members of the structure.

6.1.5 Joints in continuous design

In continuous design, joints between members should be capable of transmitting the forces and moments calculated in the global analysis. In the case of elastic analysis, the rigidity of the joints should be such that the stiffness of the frame is not less than that assumed in the analysis to an extent that would reduce its load carrying capacity. In the case of plastic analysis, a joint at a plastic hinge location should have a moment capacity at least equal to that of the member, and should also have sufficient plastic rotation capacity.

The fabrication restrictions given in **5.2.3.4** should also be applied where local yield lines are assumed in the design of components of moment-resisting connections. This applies irrespective of whether elastic or plastic global analysis is used for the structure.

6.1.6 Joints in semi-continuous design

In semi-continuous design, joints should provide a predictable degree of interaction between members as described in **2.1.2.4**. They should be capable of transmitting the in-plane restraint moments in addition to the other forces and moments at the joints. It should be ensured that the joints are neither too rigid nor too flexible to fulfil accurately the assumptions made in design.

6.1.7 Connections subject to vibration, load reversal or fatigue

6.1.7.1 *Vibration*

In connections subject to impact or vibration, preloaded bolts, locking devices or welds should be used.

6.1.7.2 *Load reversal*

In connections transferring load in shear, that are subject to load reversal (unless the reversal is due solely to wind) or in which slipping of bolts is unacceptable for some other reason, fitted bolts, preloaded bolts or welding should be used.

6.1.7.3 *Fatigue*

If fatigue is a design criterion, see **2.4.3**, the fabrication restrictions given in **5.2.3.4** should be applied.

6.1.8 Splices

6.1.8.1 *General*

Splices should be designed to hold the connected members in place. Wherever practicable, the members should be arranged so that the centroidal axis of the splice coincides with the centroidal axes of the members joined. If eccentricity occurs, the resulting moments, forces, deflections and rotations should be allowed for.

6.1.8.2 *Splices in compression members*

If the splice is not intended to transmit compression by direct contact of cross-sections in bearing, it should be designed to transmit all the moments and forces in the member at that point.

If the splice is intended to transmit compression by direct contact of cross-sections in bearing, it should be designed to resist any moments in the member at that point and to maintain the intended member stiffness about each axis.

Splices should be as close as practicable to the points of inflexion of their buckled shape. Where this is not achieved their capacity should be increased to take account of the moments induced by strut action, see **C.3**, and of the additional moments due to moment amplification, see **I.5**. Allowance should be made for moments due to strut action about each axis, but only about one axis at a time.

6.1.8.3 *Splices in tension members*

The splice should be designed to transmit all the moments and forces to which the member is subjected at that point.

6.1.8.4 *Splices in beams*

Beam splices should be designed to transmit all the forces and moments in the member at that point and to maintain the required member stiffness about both axes.

Splices in laterally unrestrained beams should be as close as practicable to the points of inflexion of their buckled shape. Where this is not achieved, their capacity should be increased to take account of the additional internal moment, see **B.3**, and of the additional moments due to moment amplification, see **I.5**.

6.1.9 Column web panel zone

In a moment-resisting joint between a beam and a column (or a rafter and a column) the web of the column within the depth of the beam should be treated as a web panel zone, irrespective of whether the column web is stiffened or unstiffened, see Figure 20. In the column web panel zone the local shear force F_{vp} due to moment transfer should be taken into account.

In a welded joint with a single beam, the panel zone shear F_{vp} should be obtained from:

$$F_{vp} = M_{tra}/(D_b - T_b)$$

where

D_b is the beam depth;
M_{tra} is the moment transferred from the beam to the column at the joint;
T_b is the beam flange thickness.

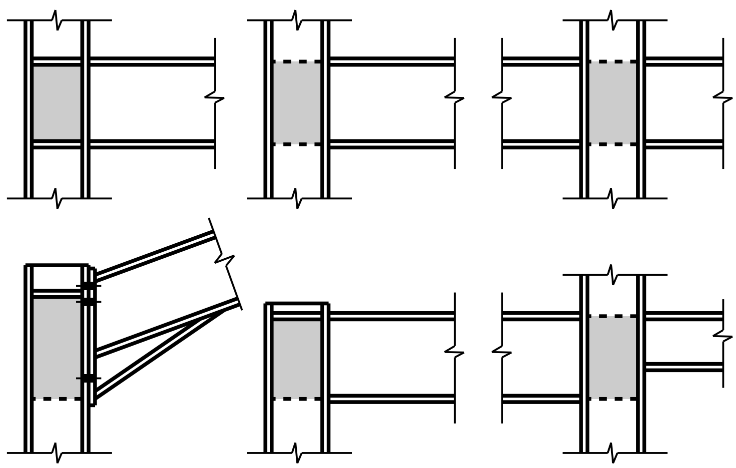


Figure 20 — Column web panel zone

In a bolted end-plate joint with a single beam, the panel zone shear F_{vp} should be obtained from the sum of the bolt forces due to the moment M_{tra} transferred from the beam to the column, taking each bolt force as the tension it transfers from the end-plate to the column.

In a joint with two beams connected on opposite sides, the panel zone shear F_{vp} should be determined taking account of the net effect of the moments in both beams.

At a bolted joint, the shear capacity P_v given in **4.2** may be developed in the column web panel zone. No reduction in moment capacity need be made in the members connected by the joint to allow for the effect of shear within the web panel zone.

At a welded joint, the moment capacities of the members connected by the joint need not be reduced to allow for the shear F_v in the web panel zone, provided that $F_v/P_v \leq 0.8$. The full shear capacity P_v may be developed in the web panel zone, provided that the end moment in each member does not exceed its elastic moment capacity $p_y Z_x$. Otherwise F_v should satisfy $F_v/P_v \leq 1 - \Sigma\mu$ in which $\Sigma\mu$ is the sum of the values of μ for each of the members connected at the joint, and μ is given by:

$$\mu = 0.05\frac{(M - M_{el})}{(M_p - M_{el})} \quad \text{but} \quad 0.05 \geq \mu \geq 0$$

where

M is the end moment in that member;

M_{el} is the elastic moment capacity $p_y Z_x$ of the member;

M_p is the plastic moment capacity $p_y S_x$ of the member.

If the web of the column is unstiffened, the web thickness should not be reduced below that needed within the web panel zone for distances above and below this zone equal to the column depth.

6.2 Connections using bolts

6.2.1 Bolt spacing

6.2.1.1 *Minimum spacing*

For standard clearance holes or holes for fitted bolts the spacing between centres of bolts should not be less than $2.5d$, where d is the nominal diameter of the bolts. For oversize holes or slotted holes the hole spacing should be increased to leave at least the same width of steel clear between the holes as for standard clearance holes.

6.2.1.2 ***Maximum spacing in unstiffened plates***

Where a plate is not stiffened by a web or outstand, the spacing between centres of adjacent bolts in a line lying in the direction of stress should not exceed 14t, where t is the thickness of the thinner element. If it is exposed to corrosive influences, the maximum spacing of bolts in each of two directions at right angles should not exceed the lesser of 16t or 200 mm, where t is the thickness of the thinner outside ply.

6.2.2 Edge and end distances

6.2.2.1 ***General***

For standard clearance holes, and for holes for fitted bolts, the edge distance should be taken as the distance from the centre of the hole to the adjacent edge, measured perpendicular to the direction of stress. The end distance should be taken as the distance from the centre of the hole to the adjacent edge, measured in the direction in which the bolt bears. The end distance should also be sufficient to provide adequate bearing capacity, see **6.3.3.3** and **6.4.4**.

6.2.2.2 ***Slotted holes***

For slotted holes, edge and end distances should be measured from the edge or end of the material to the centre of its end radius or to the centreline of the slot, see Figure 21.

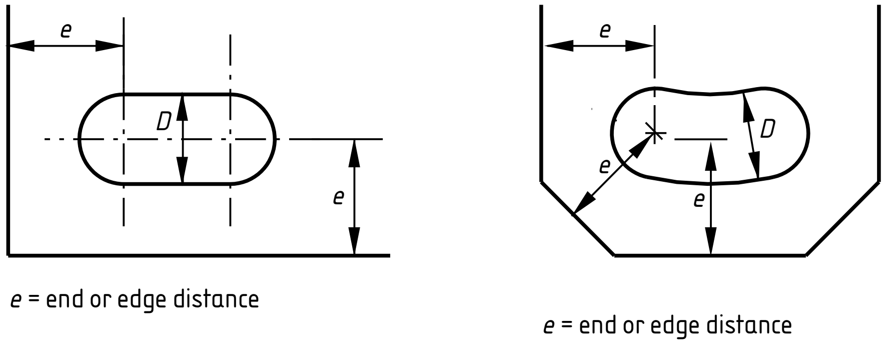


Figure 21 — Minimum edge and end distances

6.2.2.3 ***Oversize holes***

For oversize holes, the edge and end distances should be taken as the distance from the relevant edge, see **6.2.2.1**, less half the nominal diameter of the oversize hole, plus half the diameter D of a standard clearance hole.

6.2.2.4 ***Minimum edge or end distance***

The distance from the centre of a bolt hole to the edge or end of any part should be not less than the value given in Table 29.

6.2.2.5 ***Maximum edge or end distance***

Where parts are connected by bolts, the distance from an unstiffened edge to the nearest line of bolts should not exceed 11$t\varepsilon$. This does not apply to bolts interconnecting the components of back-to-back tension members, see **4.6.3**. Where the parts are exposed to corrosive influences, the edge distance should not exceed 40 mm + 4t.

Table 29 — Minimum edge and end distances of bolts

Quality of cut	Edge and end distances
For a rolled, machine flame cut, sawn or planed edge or end	1.25D
For a sheared or hand flame cut edge or end	1.40D
NOTE D is the diameter of a standard clearance hole for a bolt of the relevant nominal diameter, see Table 33.	

6.2.3 Effect of bolt holes on shear capacity

Bolt holes need not be allowed for in the shear area provided that:

$$A_{v.net} \geq 0.85A_v/K_e$$

where

$A_{v.net}$ is the net shear area after deducting bolt holes;

K_e is the effective net area coefficient from **3.4.3**.

If $A_{v.net}$ is less than $0.85A_v/K_e$ the net shear capacity should be taken as $0.7p_yK_eA_{v.net}$.

6.2.4 Block shear

Block shear failure through a group of bolt holes at a free edge, see Figure 22, (consisting of failure in shear at the row of bolt holes along the shear face of the hole group, accompanied by tensile rupture along the line of bolt holes on the tension face of the hole group, see Figure 22) should be prevented by checking that the reaction F_r does not exceed the block shear capacity P_r determined from:

$$P_r = 0.6p_yt[L_v + K_e(L_t - kD_t)]$$

where

D_t is the hole size for the tension face, generally the hole diameter, but for slotted holes the dimension perpendicular to the direction of load transfer should be used;

k is a coefficient with values as follows:

— for a single line of bolts: $k = 0.5$;

— for two lines of bolts: $k = 2.5$;

L_t is the length of the tension face, see Figure 22;

L_v is the length of the shear face, see Figure 22;

t is the thickness.

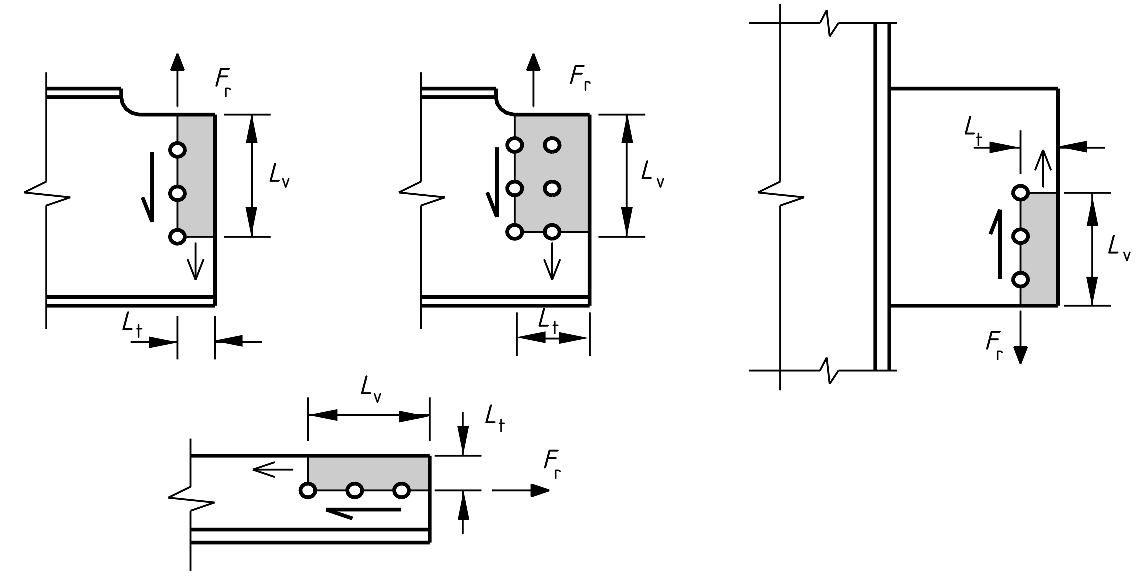


Figure 22 — Block shear — Effective shear area

6.3 Non-preloaded bolts

6.3.1 Effective areas of bolts

Bolts may have short, standard or long thread lengths, or have fully threaded shanks. The shear area A_s of a bolt should normally be taken as the tensile stress area A_t as specified in the appropriate bolt standard. For bolts without a defined tensile stress area, A_t should be taken as the area at the bottom of the threads.

Where it can be shown that threads do not occur in the shear plane, A_s may be taken as the shank area A. In the calculation of thread length, allowance should be made for tolerances and the thread run off.

6.3.2 Shear capacity

6.3.2.1 *General*

Provided that no reduction is required for the effect of packing (see **6.3.2.2**), large grips (see **6.3.2.3**), kidney-shaped slots (see **6.3.2.4**), or long joints (see **6.3.2.5**), the shear capacity P_s of a bolt should be taken as:

$$P_s = p_s A_s$$

where

A_s is the shear area (A or A_t) as defined in **6.3.1**;

p_s is the shear strength obtained from Table 30.

Table 30 — Shear strength of bolts

Bolt grade		Shear strength p_s (N/mm²)
4.6		160
8.8		375
10.9		400
General grade HSFG to BS 4395-1	≤M24	400
	≥M27	350
Higher grade HSFG to BS 4395-2		400
Other grades ($U_b \le$ 1000 N/mm²)		$0.4U_b$
U_b is the specified minimum tensile strength of the bolt.		

6.3.2.2 *Packing*

The total thickness of steel packing t_{pa} at a shear plane should not exceed $4d/3$, where d is the nominal diameter of the bolts. For multiple packs, the number of plies should preferably not exceed four. Where t_{pa} exceeds $d/3$ the shear capacity P_s should be taken as:

$$P_s = p_s A_s \left(\frac{9d}{8d + 3t_{pa}} \right)$$

but not more than given in **6.3.2.3** for large grip lengths or **6.3.2.5.** for long joints.

6.3.2.3 *Large grip lengths*

Where the grip length T_g (i.e. the total thickness of the connected plies) exceeds $5d$, where d is the nominal diameter of the bolts, the shear capacity P_s should be taken as:

$$P_s = p_s A_s \left(\frac{8d}{3d + T_g} \right)$$

but not more than given in **6.3.2.2** for the effect of packing or **6.3.2.5.** for long joints.

6.3.2.4 *Kidney-shaped slots*

Where a connection has two bolts, one in a standard clearance hole and one in a kidney-shaped slot, see **6.3.3.3**, the shear capacity of each bolt should be taken as $0.8P_s$.

6.3.2.5 *Long joints*

Where the lap length L_j of a splice or connection transferring tension or compression with more than two rows of bolts (i.e. the distance between the first and last rows of bolts, measured in the direction of load transfer, see Figure 23) exceeds 500 mm, the shear capacity P_s should be taken as:

$$P_s = p_s A_s \left(\frac{5\ 500 - L_j}{5\ 000} \right)$$

but not more than given in **6.3.2.2** for the effect of packing or **6.3.2.3** for large grip lengths.

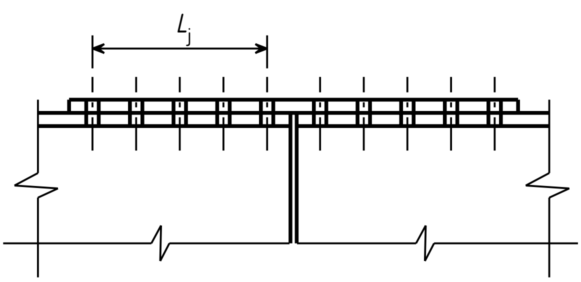


Figure 23 — Lap length of a splice

6.3.3 Bearing capacity

6.3.3.1 *General*

The bearing capacity of a bolt on any connected part should be taken as the lesser of the bearing capacity P_{bb} of the bolt (see **6.3.3.2**) and the bearing capacity P_{bs} of the part (see **6.3.3.3**).

6.3.3.2 *Bearing capacity of bolt*

The bearing capacity of the bolt itself should be taken as:

$$P_{bb} = d t_p p_{bb}$$

where

- d is the nominal diameter of the bolt;
- p_{bb} is the bearing strength of the bolt, obtained from Table 31;
- t_p is the thickness of the connected part, or, if the bolts are countersunk, the thickness of the part minus half the depth of countersinking.

Table 31 — Bearing strength of bolts

Bolt grade		Bearing strength p_{bb} (N/mm²)
4.6		460
8.8		1000
10.9		1300
General grade HSFG to BS 4395-1	≤M24	1000
	≥M27	900
Higher grade HSFG to BS 4395-2		1300
Other grades ($U_b \leq$ 1000 N/mm²)		$0.7(U_b + Y_b)$

NOTE 1 U_b is the specified minimum tensile strength of the bolt.

NOTE 2 Y_b is the specified minimum yield strength of the bolt.

6.3.3.3 *Bearing capacity of connected part*

The bearing capacity P_{bs} of the connected part should be taken as follows:

$$P_{bs} = k_{bs} d t_p p_{bs} \quad \text{but} \quad P_{bs} \leq 0.5 k_{bs} e t_p p_{bs}$$

where

- e is the end distance, as defined in **6.2.2.1**;
- p_{bs} is the bearing strength of the connected part, see Table 32.

Table 32 — Bearing strength p_{bs} of connected parts

Steel grade	S 275	S 355	S 460	Other grades
Bearing strength p_{bs} (N/mm²)	460	550	670	$0.67(U_S + Y_S)$

NOTE 1 U_s is the specified minimum tensile strength of the steel.

NOTE 2 Y_s is the specified minimum yield strength of the steel.

Provided that the sizes of the holes for non-preloaded bolts do not exceed the standard dimensions given in Table 33, the coefficient k_{bs} allowing for the type of hole should be taken as follows:

- — for bolts in standard clearance holes: k_{bs} = 1.0
- — for bolts in oversized holes: k_{bs} = 0.7
- — for bolts in short slotted holes: k_{bs} = 0.7
- — for bolts in long slotted holes: k_{bs} = 0.5
- — for bolts in kidney-shaped slots: k_{bs} = 0.5

Table 33 — Standard dimensions of holes for non-preloaded bolts

Nominal diameter of bolt	Standard clearance hole	Oversize hole[a]	Short slotted hole		Long slotted hole[b]		Kidney-shaped slot	
	Diameter	Diameter	Width	Length	Width	Length	Width	Length
mm	mm	mm	mm	mm	mm	mm	mm	mm
12	13	16	13	16	13	42	13	36
16	18	20	18	22	18	56	18	48
20	22	25	22	26	22	70	22	60
22	24	27	24	28	24	77	24	66
24	26	30	26	32	26	84	26	72
≥ 27	$d + 3$	$d + 8$	$d + 3$	$d + 10$	$d + 3$	$3.5d$	$d + 3$	$3.0d$

NOTE d is the nominal diameter of the bolt (in mm).

[a] Larger diameter holes may be used for holding-down bolts.
[b] Longer slots may be used for expansion joints.

6.3.4 Bolts subject to tension

6.3.4.1 *General*

The tension capacity of a connection using bolts (including 90° countersunk head bolts) should be checked using one of the following methods:

— the simple method given in **6.3.4.2**;

— the more exact method given in **6.3.4.3**.

6.3.4.2 *Simple method*

The simple method may be used if the connection satisfies both of the following:

— the cross-centre spacing of the bolt lines should not exceed 55 % of the flange width or end-plate width, see Figure 24;

— if a connected part is designed assuming double curvature bending, see Figure 25b), its moment capacity per unit width should be taken as $p_y t_p^2/6$, where t_p is the thickness of the connected part.

In the simple method the prying force need not be calculated. The tensile force per bolt F_t transmitted by the connection should not exceed the nominal tension capacity P_{nom} of the bolt, obtained from:

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

where

A_t is the tensile stress area as specified in the appropriate bolt standard. For bolts where the tensile stress area is not defined, A_t should be taken as the area at the bottom of the threads;

p_t is the tension strength of the bolt obtained from Table 34.

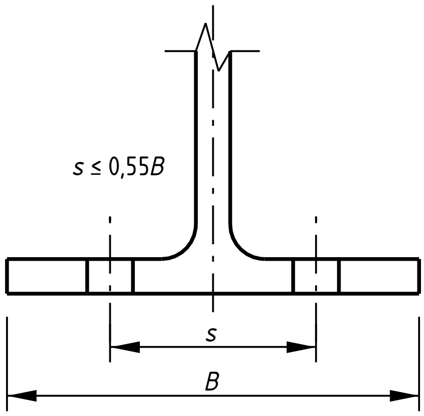


Figure 24 — Maximum cross-centres of bolt lines for the simple method

Table 34 — Tension strength of bolts

Bolt grade		Tension strength p_t (N/mm²)
4.6		240
8.8		560
10.9		700
General grade HSFG to BS 4395-1	≤M24	590
	≥M27	515
Higher grade HSFG to BS 4395-2		700
Other grades ($U_b \leq 1000$ N/mm²)		$0.7U_b$ but $\leq Y_b$
NOTE 1 U_b is the specified minimum tensile strength of the bolt.		
NOTE 2 Y_b is the specified minimum yield strength of the bolt.		

6.3.4.3 *More exact method*

The more exact method may be used for a connection in which both of the connected parts satisfy one or more of the following:

a) the connected part spans between two or more supporting parts;

b) the outstand of the connected part is designed assuming single curvature bending, see Figure 25a);

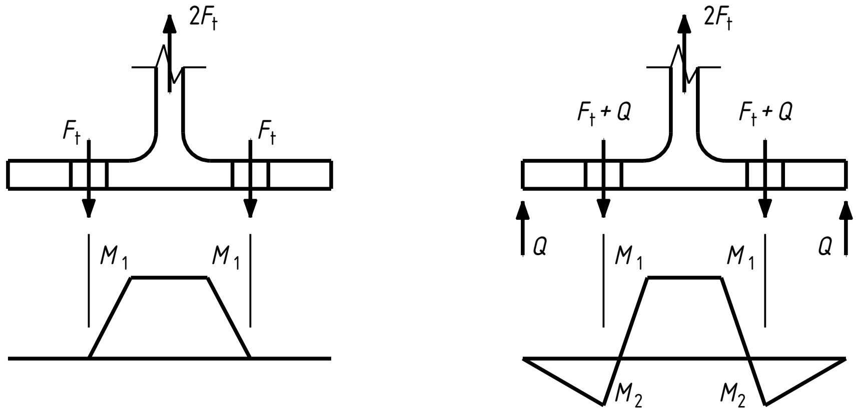
c) the outstand of the connected part is designed assuming double curvature bending, see Figure 25b), and the resulting prying force Q is calculated and included in the total applied tension F_{tot} in the bolt;

d) the connected part spans between two or more supporting parts in one direction, but acts as an outstand in the other direction, and the resulting prying force Q is calculated and included in the total applied tension F_{tot} in the bolt.

NOTE In cases a) and b) no prying force is necessary for equilibrium.

In the more exact method the moment capacity per unit width of the connected part should be taken as $p_y t_p^2/4$ and the total applied tension F_{tot} in the bolt, including the calculated prying force, should not exceed the tension capacity P_t obtained from:

$$P_t = p_t A_t$$



a) Single curvature bending b) Double curvature bending

Figure 25 — Design of outstands

6.3.4.4 *Combined shear and tension*

Bolts that are subject to shear F_s as well as tension should, in addition to the conditions given in **6.3.1** to **6.3.4.3**, satisfy the following:

— for the simple method:

$$\frac{F_s}{P_s} + \frac{F_t}{P_{nom}} \leq 1.4$$

— for the more exact method:

$$\frac{F_s}{P_s} + \frac{F_{tot}}{P_t} \leq 1.4$$

where

P_s is the shear capacity, see **6.3.2**.

6.4 Preloaded bolts

6.4.1 General

Depending on the reason for adopting preloading, a connection using preloaded HSFG bolts should be designed as one of the following:

a) a normal "bearing type" connection;

b) non-slip in service;

c) non-slip under factored loads.

In case a) the preloaded bolts should be designed in the same way as non-preloaded bolts, see **6.3**.

For cases b) and c) the slip resistance should be checked as recommended in **6.4.2** and **6.4.5**.

In case b) the shear capacity and bearing capacity after slipping should also be checked, see **6.4.4**.

NOTE The resistance of a friction grip connection to slip in service is a serviceability criterion, but for ease of use it is presented in a modified form, suitable for checking under factored loads.

6.4.2 Slip resistance

The slip resistance P_{sL} of a preloaded bolt should be determined as follows:

— for connections designed to be non-slip in service:

$$P_{sL} = 1.1 K_s \mu P_o$$

— for connections designed to be non-slip under factored loads:

$$P_{sL} = 0.9 K_s \mu P_o$$

where

P_o is the minimum shank tension as specified in BS 4604;

μ is the slip factor.

The coefficient K_s allowing for the type of hole should be taken as follows:

— for preloaded bolts in standard clearance holes: $K_s = 1.0$;

— for preloaded bolts in oversized holes or short slotted holes: $K_s = 0.85$;

— for preloaded bolts in long slotted holes, loaded perpendicular to the slot: $K_s = 0.85$;

— for preloaded bolts in long slotted holes, loaded parallel to the slot: $K_s = 0.7$.

Connections with preloaded bolts in long slotted holes, loaded parallel to the slot, should always be designed to be case c) non-slip under factored loads, i.e. using $P_{sL} = 0.9K_s\mu P_o$ not $1.1 K_s\mu P_o$.

If waisted-shank HSFG bolts are used, the connection should always be designed to be case c) non-slip under factored loads, i.e. using $P_{sL} = 0.9K_s\mu P_o$ not $1.1K_s\mu P_o$.

6.4.3 Slip factor

The slip factor μ should be obtained from Table 35. Alternatively, it may be determined from the results of tests as specified in BS 4604.

Table 35 — Slip factors for preloaded bolts

Class	Condition of faying surfaces		Slip factor μ
	Preparation	Treatment	
A	Blasted with shot or grit	Loose rust removed, no pitting	0.5
		Spray metallized with aluminium	
		Spray metallized with a zinc based coating that has been demonstrated to provide a slip factor of at least 0.5	
B	Blasted with shot or grit	Spray metallized with zinc	0.4
C	Wire brushed	Loose rust removed, tight mill scale	0.3
	Flame cleaned		
D	Untreated	Untreated	0.2
	Galvanized		

6.4.4 Capacity after slipping

For friction grip connections designed to be non-slip in service, case b) of **6.4.1**, the slip resistance P_{sL} should not be taken as more than the shear capacity P_s determined from **6.3.2**, nor more than the friction grip bearing capacity P_{bg} given by:

$$P_{bg} = 1.5 d t_p p_{bs} \quad \text{but} \quad P_{bg} \le 0.5 e t_p p_{bs}$$

where p_{bs} is the bearing strength of connected parts from Table 32 and d, e and t_p are as given in **6.3.3**.

NOTE This bearing capacity applies only to preloaded bolts designed to be non-slip in service, case b) of **6.4.1**. For the bearing capacity of preloaded bolts in bearing type connections see **6.3.3**.

6.4.5 Combined shear and tension

Preloaded bolts in friction grip connections that are also subject to externally applied tension should satisfy:

— for connections designed to be non-slip in service ($P_{sL} = 1.1K_s\mu P_o$):

$$\frac{F_s}{P_{sL}} + \frac{F_{tot}}{1.1P_o} \leq 1 \qquad \text{but} \qquad F_{tot} \leq A_t p_t$$

— for connections designed to be non-slip under factored loads ($P_{sL} = 0.9K_s\mu P_o$):

$$\frac{F_s}{P_{sL}} + \frac{F_{tot}}{0.9P_o} \leq 1$$

where

A_t is the tensile stress area, see **6.3.4.2**;

F_s is the applied shear;

F_{tot} is the total applied tension in the bolt, including the calculated prying force, see **6.3.4.3**;

P_o is the specified minimum preload, see BS 4604;

p_t is the tension strength of the bolt given in Table 34.

Preloaded bolts in bearing type connections should be designed in the same way as non-preloaded bolts, see **6.3**.

6.4.6 Holes for preloaded bolts

6.4.6.1 *Sizes of holes*

Sizes of holes for preloaded bolts should not exceed the standard dimensions given in Table 36. Standard clearance holes should be used unless oversize or slotted holes are required.

6.4.6.2 *Oversize and short slotted holes*

Oversize and short slotted holes may be used in all plies of a friction grip connection, provided that standard hardened washers are placed over the holes in the outer plies.

6.4.6.3 *Long slotted holes*

Long slotted holes should not be used in more than one of the connected plies at any individual faying surface of a friction grip connection.

Where long slotted holes are used in an outer ply of a preloaded connection, an external cover plate of sufficient size to completely cover the slot should be provided. The cover plate should be at least 8 mm thick and of structural material. A hardened washer should also be placed under the head or nut, whichever is to be turned.

6.4.6.4 *Spacing and edge distance*

Where oversize or slotted holes are used the hole spacing and edge distance should be checked to ensure that, in addition to satisfying **6.2.1** and **6.2.2**, they provide the required capacity in the connected parts.

Table 36 — Standard dimensions of holes for preloaded bolts

Nominal diameter of bolt	Standard clearance hole	Oversize hole	Short slotted hole		Long slotted hole	
	Diameter	Diameter	Width	Length	Width	Length
mm	mm	mm	mm	mm	mm	mm
16	18	20	18	22	18	40
20	22	25	22	26	22	50
22	24	27	24	28	24	55
24	26	30	26	32	26	60
≥ 27	$d + 3$	$d + 8$	$d + 3$	$d + 10$	$d + 3$	$2.5d$

NOTE d is the nominal diameter of the bolt (in mm).

6.5 Pin connections

6.5.1 Pin connected tension members

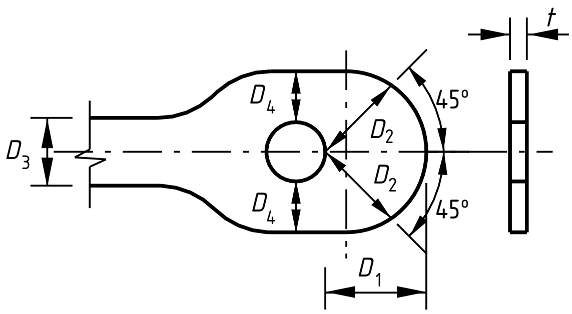
In pin connected tension members and their connecting parts, the thickness of an unstiffened element that contains a hole for a pin should be not less than 25 % of the distance from the edge of the hole to the edge of the element, measured perpendicular to the axis of the member, see Figure 26. Where the connected elements are clamped together by external nuts, this limit on thickness need not be applied to internal plies.

The net cross-sectional area beyond a hole for a pin, in all directions within 45° of the member axis, see Figure 26, should be not less than the net cross-sectional area A_r required for the member. Perpendicular to the member axis, the net cross-sectional area each side of the hole should be not less than $2A_r/3$.

All pins should be provided with locking devices to ensure that the pin cannot come out in service.

6.5.2 Pin plates

Pin plates that are provided to increase the net cross-sectional area of a member, or to increase the bearing capacity of a pin, should be arranged to avoid eccentricity and should be of sufficient size to distribute the load from the pin into the member.



$A_r = F_t/p_y$
$D_1 \geq A_r/t$
$D_2 \geq A_r/t$
$D_3 \geq A_r/t$
$D_4 \geq 0.67A_r/t$
$t \geq 0.25D_4$

Figure 26 — Pin-ended tension members

6.5.3 Design of pins

6.5.3.1 *General*

The capacity of a pin connection should be determined from the shear capacity of the pin, see **6.5.3.2**, and the bearing capacity on each connected part, see **6.5.3.3**, taking due account of the distribution of load between the various parts. The moment in the pin should also be checked, see **6.5.3.4**.

6.5.3.2 *Shear capacity*

The shear capacity of a pin should be taken as follows:

a) if rotation is not required and the pin is not intended to be removable: $0.6p_{yp}A$

b) if rotation is required or if the pin is intended to be removable: $0.5p_{yp}A$

where

A is the cross-sectional area of the pin;

p_{yp} is the design strength of the pin.

6.5.3.3 *Bearing capacity*

The bearing capacity of a pin should be taken as follows:

a) if rotation is not required and the pin is not intended to be removable: $1.5p_ydt$

b) if rotation is required or if the pin is intended to be removable: $0.8p_ydt$

where

d is the diameter of the pin;

p_y is the lower of the design strengths of the pin and the connected part;

t is the thickness of the connected part.

6.5.3.4 *Bending*

The moments in a pin should be calculated on the basis that the connected parts form simple supports. It should generally be assumed that the reactions between the pin and the connected parts are uniformly distributed along the length in contact on each part. Alternatively, if the thickness of one or more connected parts exceeds that needed to provide sufficient bearing capacity according to **6.5.3.3**, the moments may be calculated assuming that the reactions are distributed over reduced contact lengths adjacent to the interfaces, based upon the minimum thicknesses needed to provide sufficient bearing capacity.

The moment capacity of the pin should be taken as follows:

a) if rotation is not required and the pin is not intended to be removable: $1.5p_{yp}Z$

b) if rotation is required or if the pin is intended to be removable: $1.0p_{yp}Z$

where

p_{yp} is the design strength of the pin;

Z is the section modulus of the pin.

6.6 Holding-down bolts

Holding-down bolts should be provided where necessary to resist the effects of the factored loads determined in accordance with **2.4**. They should be designed to resist tension due to uplift forces and tension due to bending moments as appropriate.

Holding-down bolts required to resist tension should be anchored by a washer plate or other load distributing member embedded in the foundation. This plate or member should be designed to span any grout tube or adjustment tube provided for the holding-down bolt. Alternatively, a bend or hook in accordance with the minimum bend radius recommended in BS 8110 may be used.

The tension capacity P_t of a holding-down bolt should be obtained from:

$$P_t = 0.8 p_t A_t$$

where

A_t is the tensile stress area as specified in the appropriate bolt standard. For holding-down bolts where the tensile stress area is not defined, A_t should be taken as the area at the bottom of the threads;

p_t is the tension strength of the bolt obtained from Table 34.

The embedded length, and the arrangement of the load distributing assembly, should be such that the anchorage force can be transmitted to the foundation without exceeding the load capacity of the foundation.

If expanding anchors or resin-grouted anchors are used, it should be demonstrated that the required capacity can reliably be achieved, both by the anchor and by the foundation. Rag bolts or indented foundation bolts that are cement-grouted into pockets cast in a concrete foundation should not be used to resist tension.

It should be demonstrated that there is sufficient capacity available to transmit the horizontal shear force between a column and a foundation by one of the following:

— the frictional resistance at the interface between the base plate and the foundation;

— the shear resistance of the holding-down bolts, allowing for the resistance of the concrete around them;

— the shear resistance of that part of the foundation surrounding the base plate;

— special elements for resisting shear force, such as block or bar shear connectors.

6.7 Welded connections

6.7.1 Through thickness tension

Where a welded connection transmits tension through the thickness of the connected part, the combination of the connection details, the welding procedure and the through-thickness properties of the part should be such as to avoid lamellar tearing.

6.7.2 Details for fillet welds

6.7.2.1 *General*

Where the use of intermittent fillet welds could lead to corrosion, all fillet welds should be made continuous.

6.7.2.2 *End returns*

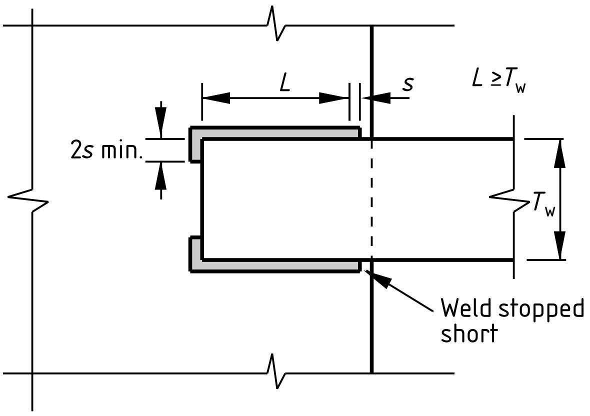
Fillet welds finishing at the ends or sides of parts should be returned continuously around the corners for a distance of at least twice the leg length s of the weld, see Figure 27, unless access or the configuration renders this impracticable. In the case of fillet welds on the tension side of a connection that is subject to significant moments, the connection should be detailed such that end returns are practicable.

6.7.2.3 *Lap joints*

In lap joints, the minimum lap should be not less than $4t$ where t is the thickness of the thinner part joined. Single fillet welds should not be used except where the parts are restrained to prevent opening of the joint.

6.7.2.4 ***End connections***

Where the end of an element is connected only by longitudinal fillet welds, the length L of each weld should be not less than the transverse spacing T_w, see Figure 27.



NOTE See also **6.7.2.1**.

Figure 27 — Welded end connections

6.7.2.5 ***Single fillet welds***

A single fillet weld should not be subject to a bending moment about its longitudinal axis that would open the root of the weld.

6.7.2.6 ***Intermittent fillet welds***

The longitudinal spacing along any one edge of the element between effective lengths of weld, as given in **6.8.2**, should not exceed the lesser of 300 mm or 16t for compression elements or 24t for tension elements, where t is the thickness of the thinner part joined. The spacing of welds in back-to-back tension members and compression members should be as given in **4.6.3** and **4.7.13** respectively.

End runs of intermittent fillet welds should extend to the end of the part they connect.

6.7.3 Details for structural hollow sections (SHS)

6.7.3.1 ***Butt joints***

A weld connecting two structural hollow sections directly end-to-end should be a full penetration butt weld.

6.7.3.2 ***End connections***

A weld connecting the end of an SHS to the surface of another member should be continuous, and may be either:

— a butt weld throughout;

— a fillet weld throughout;

— a fillet weld in one part and a butt weld in another, with a continuous transition from one to the other.

6.7.3.3 ***Joint layout***

Joints at which two or more SHS are connected to a larger member (either an SHS or another type) should be set out so that the connected members either form an overlap joint with sufficient overlap to transfer the forces between the members, or a gap joint with sufficient clearance between the welds connecting each member. Where necessary, eccentricity should be introduced between the intersections of the members in order to achieve this.

NOTE Information on the design of overlap and gap joints for SHS members, including the effects of eccentricity is given in DD ENV 1993-1-1/A1, see Bibliography.

6.7.3.4 *Joint design*

All welded connections of SHS to the surface of another member should be investigated to ensure that the stiffness and strength of the connection are sufficient to transmit the forces in the connected members, without causing weld failure due to significant non-uniform distribution of stresses in the connection, forming local yield-line mechanisms or developing excessive punching shear in the member to which they are connected.

NOTE Information on joint design for SHS is given in DD ENV 1993-1-1/A1, see Bibliography.

6.7.4 Partial penetration butt welds

Intermittent partial penetration butt welds should not be used.

6.7.5 Welded connections to unstiffened flanges

Where a plate (or beam flange) is welded to an unstiffened flange of an I- or H-section column (or other member), see Figure 28, the applied force F_x perpendicular to the flange should not exceed P_x calculated from the following:

— for a rolled I- or H-section column:

$$P_x = [4\sqrt{2}\,T_c^2 + (t_c + 1.6r_c)t_p]p_{yc} \quad \text{but} \quad P_x \le (5T_c + t_c + 1.6r_c)t_p p_{yp}$$

— for a welded I or H-section column:

$$P_x = [4\sqrt{2}\,T_c^2 + (t_c + 1.6s_c)t_p]p_{yc} \quad \text{but} \quad P_x \le (5T_c + t_c + 1.6s_c)t_p p_{yp}$$

where

- p_{yc} is the design strength of the flange of the column;
- p_{yp} is the design strength of the connected plate;
- r_c is the root radius of a rolled I- or H-section column;
- s_c is the leg size of the web-to-flange welds of a welded I- or H- column;
- T_c is the flange thickness of the column;
- t_c is the web thickness of the column;
- t_p is the thickness of the connected plate (or beam flange).

The welds connecting a plate (or beam flange) to an unstiffened flange of an I- or H-section column should be of uniform size throughout, but should be designed to resist the applied force F_x assuming this to be concentrated over an effective breadth b_e of the flange, as shown in Figure 28, given by:

$$b_e = \frac{P_x}{t_p p_{yp}}$$

If b_e is less than $0.5(F_x/P_x)b_p$, where b_p is the overall width of the plate (or beam flange), the flange of the column (or other member) should be stiffened even if F_x does not exceed P_x.

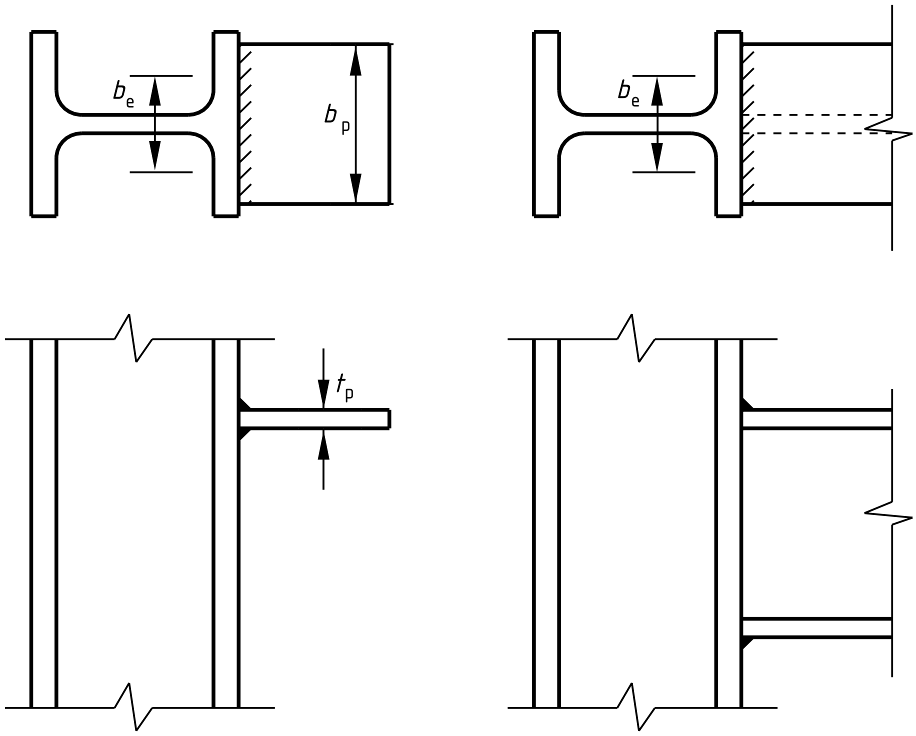


Figure 28 — Welded connection to an unstiffened flange

6.8 Design of fillet welds

6.8.1 Angle of intersection

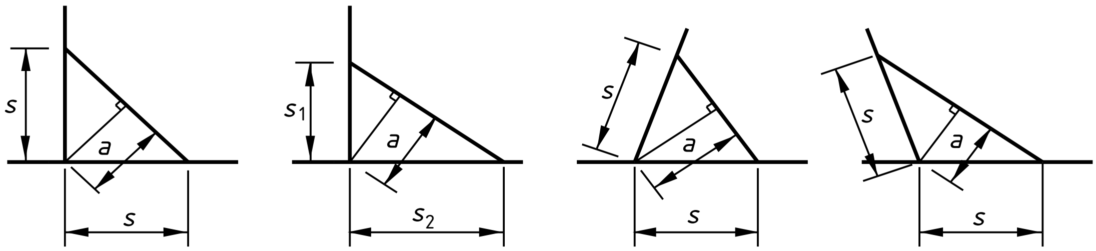
The angle of intersection of members connected by fillet welds should be such that the angle between the fusion faces of a weld is not less than 60° and not more than 120°. Outside these limits the adequacy of the connection should be determined on the basis of tests in accordance with Section 7.

6.8.2 Effective length

The effective length of a fillet weld should be taken as the length over which the fillet is full size. In the absence of better information this may be taken as equal to the overall length, less one leg length s for each end that does not continue around a corner. A fillet weld with an effective length less than $4s$ or less than 40 mm should not be used to carry load.

6.8.3 Throat size

The effective throat size a of a fillet weld should be taken as the perpendicular distance from the root of the weld to a straight line joining the fusion faces that lies just within the cross-section of the weld, see Figure 29.



a) Equal leg fillet b) Unequal leg fillet c) Acute angled fillet d) Obtuse angled fillet

Figure 29 — Effective throat size *a* of a fillet weld

6.8.4 Deep penetration fillet welds

Where deep penetration fillet welds are produced by submerged arc welding or similar methods, with a depth of penetration p to the minimum depth of fusion (see Figure 30) of at least 2 mm, provided that it can be shown that the required penetration can consistently be achieved, the effective throat size a should be measured to the minimum depth of fusion as shown in Figure 30.

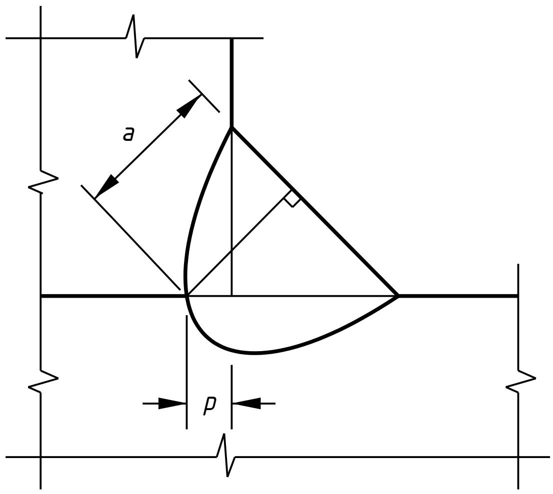


Figure 30 — Deep penetration fillet weld

6.8.5 Design strength

Fillet welds should be made using electrodes or other welding consumables with a specified Charpy impact value equivalent to, or better than, that specified for the parent metal. The design strength p_w of a fillet weld should be determined from Table 37, corresponding to the electrode classification and the steel grade, or the lower grade for connections between different steel grades.

Table 37 — Design strength of fillet welds p_w

Steel grade	Electrode classification (see Table 10)		
	35 N/mm^2	**42** N/mm^2	**50** N/mm^2
S 275	220	(220)[a]	(220)[a]
S 355	(220)[b]	250	(250)[a]
S 460	(220)[b]	(250)[b]	280

For other types of electrode and/or other steel grades:

$p_w = 0.5U_e$ but $p_w \leq 0.55U_s$

where

U_e is the minimum tensile strength of the electrode, as specified in the relevant product standard;

U_s is the specified minimum tensile strength of the parent metal.

[a] Over-matching electrodes.
[b] Under-matching electrodes. Not to be used for partial penetration butt welds.

6.8.6 Design stress

The force per unit length transmitted by a fillet weld at a given point in its length should be determined from the applied forces and moments, using the elastic section properties of the weld or weld group, based on effective throat sizes, see **6.8.3**. The design stress in a fillet weld should be calculated as the force per unit length transmitted by the weld, divided by the effective throat size a.

6.8.7 Capacity of a fillet weld

6.8.7.1 *General*

Provided that the effective throat size a of a fillet weld does not exceed $0.7s$, where s is the length of the smaller leg for a plain fillet weld or the smaller fusion face for any other case, its capacity should be checked using either **6.8.7.2** or **6.8.7.3**. If $a > 0.7s$ its capacity should either be checked taking a as equal to $0.7s$ or alternatively as a butt weld, see **6.9.3**.

6.8.7.2 *Simple method*

The capacity should be taken as sufficient if throughout the length of the weld the vector sum of the design stresses due to all forces and moments transmitted by the weld does not exceed its design strength p_w, see **6.8.5**.

6.8.7.3 *Directional method*

Alternatively to **6.8.7.2**, the forces per unit length transmitted by the weld may be resolved into a longitudinal shear F_L parallel to the axis of the weld, see Figure 31a), and a resultant transverse force F_T perpendicular to this axis, see Figure 31b).

The longitudinal shear capacity P_L per unit length of weld should be taken as:

$$P_L = p_w a$$

The transverse capacity P_T per unit length of weld, in a direction at an angle θ to the weld throat, should be taken as:

$$P_T = KP_L$$

Throughout its length, the weld should satisfy the following relationship:

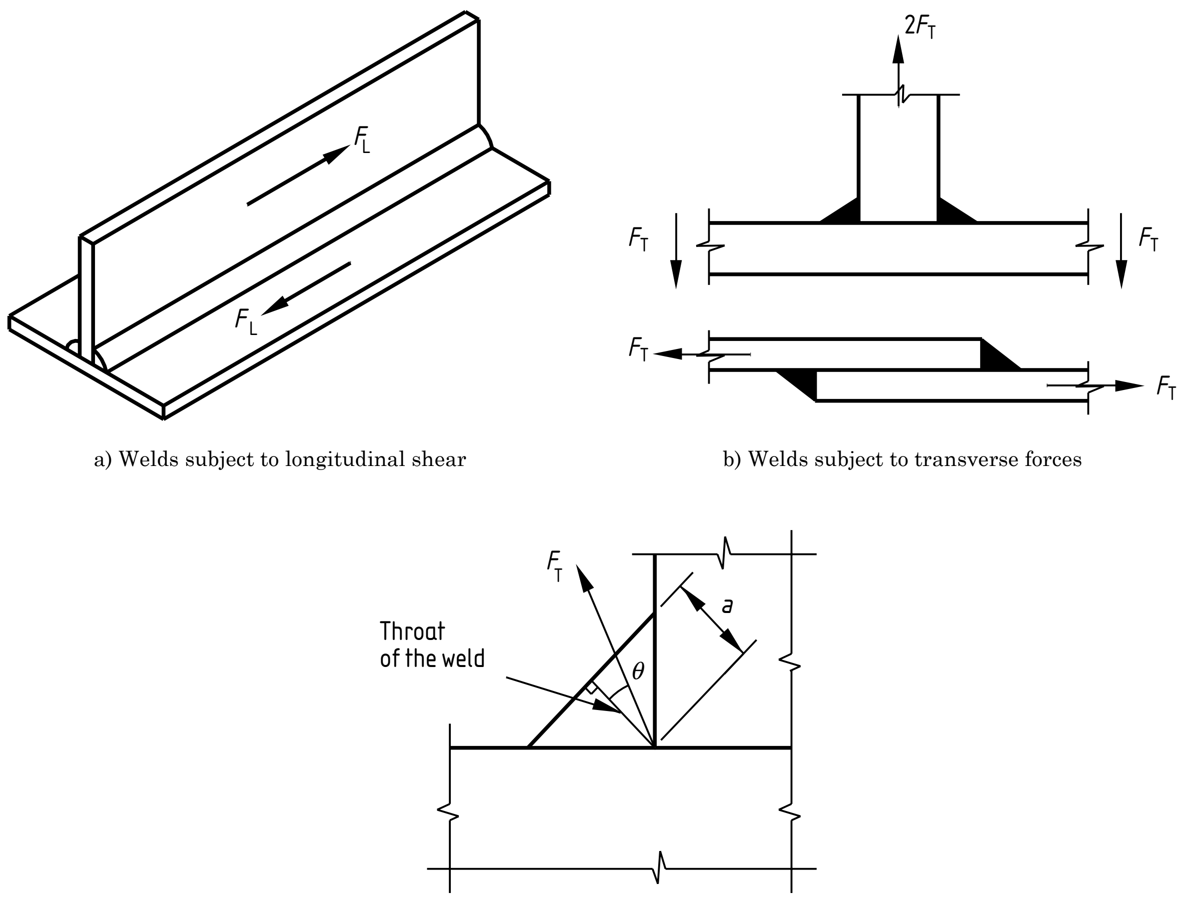
$$(F_L/P_L)^2 + (F_T/P_T)^2 \leq 1$$

The coefficient K should be obtained from:

$$K = 1.25\sqrt{\frac{1.5}{1 + \cos^2\theta}}$$

in which θ is the angle between the force F_T and the throat of the weld, see Figure 31c).

NOTE For a transverse force parallel to one leg of an equal leg fillet weld that connects two elements that are at right angles to each other, $\theta = 45°$ and $K = 1.25$.



a) Welds subject to longitudinal shear

b) Welds subject to transverse forces

c) Resultant transverse force on weld

Figure 31 — Fillet welds — Directional method

6.9 Design of butt welds

6.9.1 Design strength

All full penetration butt welds and partial penetration butt welds should be made using matching electrodes or other welding consumables. A matching electrode should have a specified minimum tensile strength, yield strength, elongation at failure and Charpy impact value each equivalent to, or better than, those specified for the parent metal. Provided that a matching electrode is used, the design strength of a butt weld should be taken as equal to that of the parent metal.

6.9.2 Throat size of partial penetration butt welds

The throat size of a single-sided partial penetration butt weld, see Figure 32a) and Figure 32c), or the size of each throat of a double-sided partial penetration butt weld, see Figure 32b) and Figure 32d), should be taken as equal to the minimum depth of penetration from that side of the weld.

The minimum throat size of a longitudinal partial penetration butt weld should be $2\sqrt{t}$ where t is the thickness (in mm) of the thinner part joined, unless a larger throat size is needed to resist the applied forces.

6.9.3 Capacity of partial penetration butt welds

Single-sided partial penetration butt welds that are asymmetric relative to the parts joined should not be used to resist tension or compression, unless the connection is suitably restrained against rotation. In calculating the stress in the weld, the resulting eccentricity should be taken into account.

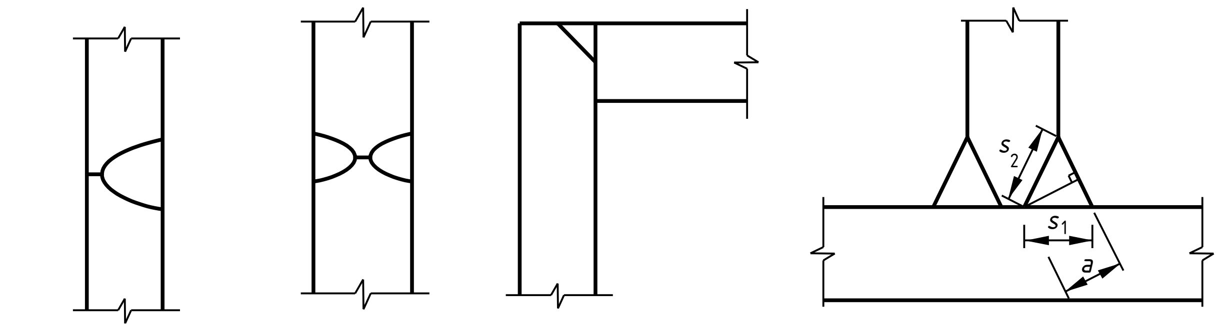
The capacity of a partial penetration butt weld in a butt joint, see Figure 32a) and Figure 32b), or a corner joint, see Figure 32c), should be taken as sufficient if throughout the weld the stress does not exceed the relevant strength of the parent material.

The capacity of a tee-butt joint with a pair of partial penetration butt welds with additional fillets, see Figure 32d), should be determined by treating it:

— as a butt weld, if $a > 0.7s$;

— as a fillet weld, see **6.8.6**, if $a \leq 0.7s$;

in which a is the effective throat size and s is the length of the smaller fusion face, see Figure 32d).



a) Butt joint with single-sided partial penetration butt weld

b) Butt joint with double-sided partial penetration butt weld

c) Corner joint with single-sided partial penetration butt weld

d) Tee-butt joint with a pair of partial penetration butt welds with external fillets

Figure 32 — Partial penetration butt welds

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Section 7. Loading tests

7.1 General

7.1.1 Purpose of testing

Testing is not required for structures and parts of structures designed as recommended in Section 1 to Section 6 of this part of BS 5950.

Experimental verification by loading tests as stipulated in **2.1.2.5** may be undertaken in place of design by calculation, or to provide data needed for design by calculation, if:

a) the design or construction is not entirely in accordance with Section 1 to Section 6 of this part of BS 5950;

b) the capacity of an existing structure or component is in doubt;

c) appropriate analytical or design procedures are not available for designing the particular component or structure by calculation alone;

d) the design load carrying capacity of a component or structure is to be established from a knowledge of its ultimate capacity;

e) it is intended to construct a number of similar structures on the basis of prototype testing.

To qualify for acceptance on the basis of loading tests, structures and components should be of robust and practical construction and reasonably insensitive to incidental loads.

This section does not apply to the testing of scale models or of items subject to fluctuating loads that could cause fatigue to become the design criterion.

7.1.2 Types of loading tests

The following types of loading test may be carried out as appropriate:

a) a proof test for confirming general structural behaviour, see **7.5**;

b) a strength test against the required factored loads, see **7.6**;

c) a failure test to determine the ultimate capacity and mode of failure, see **7.7**.

NOTE These test procedures are intended only for steel structures within the scope of this part of BS 5950. In other cases reference should be made to Section 3.1 or Parts 4, 5, 6 or 9 of BS 5950 as appropriate.

7.1.3 Quality control

If a structure or component has been designed on the basis of the strength tests or failure tests detailed in **7.6** or **7.7**, quality control should be carried out during production in order to confirm consistency.

An appropriate number of samples (not less than two) should be selected at random from each production batch. These samples should be carefully examined to establish whether they are similar in all relevant respects to the prototype tested. Particular attention should be given to the following:

— dimensions of components and connections;

— tolerances and workmanship;

— quality of steel used (checked by reference to mill certificates).

If, from this examination, it is not possible to determine either the variations or the effect of variations compared to the prototype, a proof test as detailed in **7.5** should be carried out. In this test, the deflections should be measured at the same positions as in the initial proof test on the prototype. The maximum measured deflection should not exceed 120 % of the deflection recorded during the proof test on the prototype and the residual deflection should not be more than 105 % of that recorded for the prototype.

7.2 Test conditions

7.2.1 General

The tests should simulate the behaviour of the structure or component in service. The test rig should have sufficient strength and stiffness for the expected loads and should provide sufficient clearance for the expected deflections. It should follow the movements of the test specimen without interruption and should not offer more restraint to deformation of the test specimen than would be available in service.

Each test specimen should be similar in all respects to the structure or component that it represents. It should be free to deflect under load. Unintended eccentricities at points of load application or supports should be avoided. Lateral and torsional restraints should represent the actual conditions expected in service and should be applied with the same eccentricities as in service.

The loading devices should reproduce the magnitude and distribution of the loads and reactions, and simulate the way they are applied in service, without localizing the applied forces at the points of greatest resistance. The supporting devices should reproduce the support conditions to be used in service.

Due attention should be paid to the safety of the test arrangements, particularly in failure tests. Failure of a test specimen should not lead to general instability of the test rig.

7.2.2 Measurements

Load and deflection measurements should be monitored as closely as practicable.

The deflections should be measured at sufficient points where the movement is expected to be high to enable the maximum deflections of the test specimen to be determined. The anticipated magnitudes of the deflections should be estimated in advance, with generous allowances for movement beyond the elastic range.

In some situations it may be desirable to determine the magnitude of stresses in a specimen. This may be demonstrated qualitatively by means of brittle coatings or quantitatively by measurements of strain. Such information should be treated as supplementary to the load-deflection behaviour.

7.2.3 Loading

The rate of load application should be such that the behaviour can be considered to be quasi-static.

The difference between the self-weight of a test specimen and the actual dead load in service should be taken into account in calculating the test loads.

If a load combination includes forces on more than one line of action, each increment of the test loading should be applied proportionately to each of these forces.

7.3 Test procedures

7.3.1 Preliminary loading

Prior to any test, in order to bed down the test specimen onto the test rig, preliminary loading not exceeding the unfactored values of the relevant loads may be applied and then removed.

7.3.2 Load increments

The test loading should be applied in at least five regular increments and the load-deformation behaviour of the test specimen should be recorded. The increments should be based on the expected load-deformation behaviour. Their number should be sufficient to give a full record of the behaviour of the test specimen.

Sufficient time should be allowed after each increment for the test specimen to reach stationary equilibrium. After each increment the test specimen should be carefully examined for signs of rupture, yielding or local or overall buckling. Unloading should be completed in regular decrements with deflection readings taken at each stage and after unloading is complete.

At each increment or decrement of the loading, the deflections or strains should be measured at one or more principal locations on the structure. Readings of deflections or strains should not be taken until the structure has completely stabilized after a load increment.

A running plot should be maintained of the principal deflection against loading. When this indicates significant non-linearity the load increments should be reduced.

7.3.3 Coupon tests

To validate comparisons between loading tests carried out on different specimens or at different times, the properties of the steel used in the test specimens should be established by means of coupon tests.

Generally the coupons should be cut from the same sections or plates as the test specimens. Where appropriate they may be recovered from unyielded areas of the test specimens after the completion of testing.

The yield strength and tensile strength of the steel should be determined by tensile testing in accordance with BS EN 10002-1.

The properties of the specimens used in a particular loading test may be taken as the mean of a set of coupon tests, one for each relevant component tested.

If the material properties are required in advance of load testing (as when determining the test load for a strength test, see **7.6.2**), a single coupon test from each lot of material for the components of an individual test specimen may be used to obtain a weighted mean yield strength for the whole assembly.

7.3.4 Test report

The following information should be included in the test report:

— details of the actual tests carried out;

— the actual dimensional measurements of the test specimen;

— details of the loading method and testing procedure;

— a diagram showing the positions of the loading points and the measuring devices;

— all test results necessary for the test evaluation;

— a record of all other observations from the test.

7.4 Relative strength coefficient

7.4.1 General

In strength tests and failure tests the effect of variations of geometry or material properties of test specimens, compared to their nominal values, should be taken into account by means of relative strength coefficients.

7.4.2 For a strength test

For a strength test, the relative strength coefficient should be applied in determining the test load, see **7.6.2**.

The relative strength coefficient should take into account the actual cross-sectional dimensions of the specimen and the actual yield strength of the steel in the specimen, determined from coupon tests, see **7.1.3**.

When the test is to be carried out on an assembly of structural components, the relative strength coefficient should be based on a weighted mean value of the actual yield strength of each component, in which the weighting is applied to make appropriate allowance for the influence of each part of the test specimen on the expected performance. Provided that the actual cross-sectional dimensions of the components do not exceed their nominal dimensions, the relative strength coefficient R_s may be obtained from:

$$R_s = \frac{\text{Weighted mean yield strength}}{\text{Nominal yield strength}}$$

If the actual cross-sectional dimensions exceed the nominal dimensions, the relative strength coefficient R_s should be obtained by making appropriate adjustments to the weighted mean yield strength, to allow for the influence of each cross-sectional dimension of the test specimen on its expected performance.

In the absence of other information, the relative importance of each component of an assembly to its overall performance may be based on appropriate monitoring during the preliminary proof test stipulated in **7.6.1**.

Alternatively, if reliable information about the expected failure mode is available from other similar tests, the relative strength coefficient R_s may be determined as for a failure test, see **7.4.3**.

7.4.3 For a failure test

For a failure test, the relative strength coefficient should be applied in determining the design capacity from the test results, see **7.7.3**.

If realistic assessments of the capacity can be made using the provisions of Section 1 to Section 6, or by other proven methods of design by calculation that take account of all buckling effects, the relative strength coefficient R_s may be obtained from:

$$R_s = \frac{\text{Capacity assessed using actual yield strength and actual dimensions}}{\text{Capacity assessed using nominal yield strength and nominal dimensions}}$$

Otherwise the relative strength coefficient R_s should be determined according to the observed failure mode, as follows:

a) *for a ductile yielding failure*:

$$R_s = \frac{\text{Mean yield strength}}{\text{Nominal yield strength}} \times R_p$$

in which the mean yield strength relates to the cross-section at which failure is observed;

b) *for a sudden failure due to rupture in tension or shear:*

$$R_s = \frac{\text{Mean ultimate tensile strength}}{\text{Nominal yield strength}} \times R_p$$

in which the mean tensile strength relates to the cross-section at which failure is observed;

c) *for a sudden failure due to buckling:*

$$R_s = \frac{1.2 \times \text{mean yield strength}}{\text{Nominal yield strength}} \times R_p$$

in which the mean yield strength relates to the cross-section at which failure is observed;

d) *for a ductile failure due to overall member buckling:*

$$R_s = \frac{\text{Buckling strength for mean yield strength}}{\text{Buckling strength for nominal yield strength}} \times R_p$$

in which the buckling strength relates to the relevant slenderness λ from the appropriate buckling curve and the mean yield strength relates to the cross-section at which failure is observed; alternatively, R_s may be obtained as in a) if the relevant slenderness or the appropriate buckling curve are in doubt;

e) *for a ductile failure due to local buckling of a flat element:*

$$R_s = \frac{\text{Actual yield strength}}{\text{Nominal yield strength}} \times \frac{\text{Actual thickness}}{\text{Nominal thickness}} \times R_p$$

but $R_s \geq \left[\frac{\text{Actual yield strength}}{\text{Nominal yield strength}}\right]^{0.5} \times \left[\frac{\text{Actual thickness}}{\text{Nominal thickness}}\right]^2 \times R_p$

and $R_s \geq 1$

where

$$R_p = \frac{\text{Actual value of section property}}{\text{Nominal value of section property}} \quad \text{but } R_p \geq 1$$

in which the section property is that relevant to resisting the observed failure mode, and the values relate to the cross-section at which failure is observed.

7.5 Proof test

7.5.1 General

A proof test may be used as a non-destructive test to confirm the general structural behaviour of a structure, structural assembly or component. Any irregularities occurring during the test should be closely scrutinized and the reasons for their occurrence recorded.

It should be recognized that the loading applied in a proof test may cause permanent local distortions. Such effects do not necessarily indicate structural failure, but the relevance of their occurrence to the continued use of the components concerned should be decided before testing.

During a proof test, the loads should be applied in a number of regular increments at regular time intervals and the principal deflections should be measured at each stage. If the deflections show significant non-linearity, the load increments should be reduced. Unloading should be completed in regular decrements, with deflection readings taken at each stage.

On the attainment of the proof test load, it should be maintained at a near constant value to allow repeat measurements for detecting possible creep. The loads and deflections should be measured at regular checking intervals of at least 5 minutes. The loading should be adjusted to remain constant until there is no significant increase in deflection during at least three checking intervals after the attainment of the proof test load.

7.5.2 Proof test load

The test load for a proof test should be taken as equal to the sum of:

1) 1.0 × (actual dead load present during the test);

2) one of the following as appropriate:

   a) 1.25 × (imposed load) plus 1.15 × (remainder of dead load);

   b) 1.15 × (remainder of dead load) plus 1.2 × (wind load);

   c) 1.2 × (wind uplift) minus 1.0 × (remainder of dead load);

   d) 1.15 × (remainder of dead load) plus 1.0 × (imposed load and wind load).

7.5.3 Proof test criteria

The structure or component should demonstrate substantially linear behaviour under the proof test load. On removal of the test load the residual deflection should not exceed 20 % of the maximum deflection recorded during this test. If these criteria are not satisfied the proof test may be repeated once only. Under this repeat application of the proof test loading the structure should demonstrate substantially linear behaviour and the residual deflection should not exceed 10 % of the maximum recorded during the repeat test.

On attainment of the proof test load, the loading should be sustained for at least 15 minutes without any increase in the deflections.

7.6 Strength test

7.6.1 General

A strength test may be used to confirm the calculated load carrying capacity of a structural assembly or component. Prior to a strength test, the test specimen should pass the proof test detailed in **7.5**.

Where several similar items are to be constructed to a common design, and one or more prototypes satisfy all the criteria of this strength test, the others may be accepted without load testing provided that quality control as stipulated in **7.1.3** confirms that they are similar in all relevant respects to the prototypes.

On the attainment of the strength test load, it should be maintained at a near constant value to allow repeat measurements for detecting possible creep. The loads and deflections should be measured at regular checking intervals of at least 5 minutes. The loading should be adjusted to remain constant until there is no significant increase in deflection during at least three checking intervals after the attainment of the strength test load.

7.6.2 Strength test load

The test load for a strength test should be based on the factored load for design by calculation obtained from Section 2 using the appropriate γ_f factors for the relevant combination of dead, imposed and wind loads.

The total test load (including the self-weight of the test specimen) should be determined using:

$$(\text{Strength test load}) = R_s \times (\text{factored load})$$

in which R_s is the relative strength coefficient determined from **7.4.2**.

7.6.3 Strength test criteria

Under the strength test load none of the following events should occur in any part of the test specimen:

— collapse or fracture;

— a crack begins to spread in a vital part of the specimen;

— the displacement becomes grossly excessive.

On removal of the strength test load the residual deflection should not exceed 80 % of the maximum deflection recorded during this test.

7.7 Failure test

7.7.1 General

A failure test may be used to determine the real mode of failure and the ultimate load carrying capacity of a structure or component. Because it is only from a test to failure that this information can be obtained, when the specimen for a strength test is not required for use in service, it may be advantageous to obtain this additional information after completing the strength test.

Even if determining the ultimate load carrying capacity is the prime objective, it is still desirable to carry out a proof test and a strength test first, before starting to determine the failure load. In such cases, an estimate should be made of the anticipated design capacity as a basis for the proof test and strength test loads. It may then be desirable to adjust this estimated value on the basis of the strength test.

During a test to failure, the loading should first be applied in increments up to the strength test load. Subsequent load increments should then be based on an examination of a plot of the principal deflections.

7.7.2 Failure criterion

The ultimate load carrying capacity should be taken as the value of the test load beyond which the structure or component is unable to sustain any further increase in load. At this load, gross permanent distortion is likely to have occurred. In some cases excessive deformation may define the ultimate capacity.

Failure of a test specimen should be considered to have occurred in any of the following events:

— collapse or fracture;

— a crack begins to spread in a vital part of the specimen;

— the displacement becomes grossly excessive.

The test result should be taken as the maximum value of the loading applied to the test specimen either coincident with failure or immediately prior to failure, as appropriate.

7.7.3 Determination of design capacity

The design capacity for an item similar to that tested may be determined from:

$$\text{Design capacity} = K_t \times \left[\frac{\text{mean test result}}{R_s}\right]$$

in which R_s is the relative strength coefficient determined from **7.4.3**.

If the resulting design capacity falls below that obtained from the strength test, the latter should be taken.

For a single test K_t should be taken as 0.8. For a set of two or three related tests, in which all the results are within ±10 % of the mean value, K_t should be taken as 0.9, provided that in the case of two related tests the lower of the two test results is used in place of the mean test result.

For four or more related tests K_t should be determined from:

$$K_t = 1.1 \times \left(1 - \frac{k \times s}{(\text{mean test result})}\right)$$

in which s is the standard deviation of the test results, obtained from:

$$s = \sqrt{\frac{\sum_{i=1}^{n} (v_i)^2 - \frac{1}{n}\left(\sum_{i=1}^{n} v_i\right)^2}{n-1}}$$

where

k is a statistical factor obtained from Table 38 for the appropriate number of tests;

n is the number of tests;

v_i is the result of test i.

Table 38 — Statistical factor *k*

Number of tests n	4	5	6	8	10	20	30	∞
Value of k	2.63	2.33	2.18	2.00	1.92	1.76	1.73	1.64

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Annex A (informative) Safety format in BS 5950-1 and references to BS 5400-3

A.1 Introduction

The safety formats in BS 5950-1 and BS 5400-3 are both partial factor formats related to the version of ISO 2394 current when they were drafted (see Bibliography). However, there are differences in implementation, and these need to be taken into account where BS 5950-1 makes reference to BS 5400-3.

A.2 Design loads

Design (or factored) loads F_d are determined from characteristic (or specified) loads F_k using:

$$F_d = \gamma_{\ell 1} \gamma_{\ell 2} F_k$$

where

$\gamma_{\ell 1}$ is a partial factor to allow for variation of loads from their characteristic values;

$\gamma_{\ell 2}$ is a partial factor to allow for the reduced probability that various loads acting together will simultaneously reach their characteristic values.

A.3 Design load effects

Design load effects S_d are determined from design loads F_d using:

$$S_d = \text{Effects of } \gamma_p F_d$$

where

γ_p is a partial factor to allow for variations of the structural behaviour from that expected.

A.4 Design resistance

Design resistance R_d is determined from characteristic (or specified) material strengths f_k using:

$$R_d = \text{Function of } f_k/\gamma_m$$

where

γ_m is a partial factor to allow for manufacturing tolerances and variations of material strengths from their characteristic values.

A.5 Verification of structural adequacy

For a satisfactory design (in addition to other criteria), at both ultimate and serviceability limit states:

$$R_d \geq S_d$$

i.e. $\text{Function of } (f_k/\gamma_m) \geq \text{Effects of } (\gamma_p \gamma_{\ell 1} \gamma_{\ell 2} F_k)$

A.6 Implementation in BS 5950-1

In BS 5950-1 the resistance R_d is generally determined from tabulated design strengths p_y based on:

$$p_y = Y_s/\gamma_{m1} \quad \text{but} \quad p_y \leq U_s/\gamma_{m2}$$

where

U_s is the specified minimum ultimate tensile strength;

Y_s is the specified minimum yield strength;

γ_{m1} is a partial factor for resistance based on yield strength of material;

γ_{m2} is a partial factor for resistance based on ultimate tensile strength of material.

For ultimate limit states the factored loads F_d are determined using:

$$F_d = \gamma_f F_k$$

in which:

$$\gamma_f = \gamma_\ell \gamma_p \text{ (see \textbf{2.1.3})}$$

and $$\gamma_\ell = \gamma_{\ell 1} \gamma_{\ell 2}$$

Resistance at serviceability limit states is covered indirectly, by modifying ultimate limit state verifications, in the following cases only:

— irreversible deformation due to bending moments, see **4.2.5.1**;

— yielding of net cross-section in tension at holes for bolts, see **4.6.1**;

— slip of preloaded bolts, see **6.4.2** and **6.4.5**.

Serviceability loads are defined in **2.5.1**. The ratio (factored load)/(serviceability load) is generally about 1.5 with a minimum of 1.4. Thus, a partial factor of approximately 1.2 (generally about 1.25, with a minimum of 1.15) for resistance under serviceability loads is achieved by:

— the limit of $1.2p_yZ$ in **4.2.5.1** for simply supported beams and cantilevers (for other members, the deformation is restricted by continuity and the limit becomes $1.5p_yZ$);

— the limiting value of $1.2A_n$ for P_t in **4.6.1**;

— the coefficient 1.1 in **6.4.2** and **6.4.5** for connections designed to be non-slip in service, compared to the coefficient of 0.9 for connections designed to be non-slip under factored loads.

A.7 Implementation in BS 5400-3

In BS 5400-3 the resistance R_d is generally determined from the nominal yield strength σ_y using:

$$R_d = \frac{(\text{function of } \sigma_y \text{ and geometric variables})}{\gamma_m \gamma_{f3}}$$

in which:

$$\gamma_m = \gamma_{m1}\gamma_{m2}$$

and $$\gamma_{f3} = \gamma_p$$

where

γ_{m1} is a partial factor to allow for reduction of strength compared with the characteristic value;

γ_{m2} is a partial factor to allow for other possible causes of weakness, including tolerances.

For ultimate limit states γ_{f3} has a constant value of 1.1, but the value of γ_m varies according to the resistance under consideration.

It should be noted that the definition of σ_y in BS 5400-3 differs from that of p_y in BS 5950-1. As a result their values are not necessarily the same, even though in many cases they are.

The design loads F_d for ultimate limit states are determined using:

$$F_d = \gamma_{fL} F_k$$

in which:

$$\gamma_{fL} = \gamma_{f1}\gamma_{f2}$$

Resistance at serviceability limit states is covered explicitly, but only for a small number of particular cases. The values of γ_{fL} are different from those for ultimate limit states and γ_{f3} is taken as unity. The value of γ_m is generally unity, but is taken as 1.2 for the slip resistance of preloaded bolts.

A.8 Comparison of partial factors

The methods of applying partial factors in BS 5400-3 and BS 5950-1 are compared in Table A.1.

Table A.1 — Comparison of partial factors

Partial factor	$\gamma_{\ell 1}$	$\gamma_{\ell 2}$	γ_{p}	γ_{m1}	γ_{m2}
BS 5400-3	γ_{f1}	γ_{f2}	γ_{f3}	γ_{m1}	γ_{m2}
	$\gamma_{fL} = \gamma_{f1} \times \gamma_{f2}$			$\gamma_{m} = \gamma_{m1} \times \gamma_{m2}$	
BS 5950-1	$\gamma_{\ell 1}$	$\gamma_{\ell 2}$	γ_{p}	γ_{m1}	γ_{m2}
	$\gamma_{\ell} = \gamma_{\ell 1} \times \gamma_{\ell 2}$			$\gamma_{m} = \gamma_{m1}$ or γ_{m2}	
	$\gamma_{f} = \gamma_{\ell} \times \gamma_{p}$			(included in design strength)	

Annex B (normative)
Lateral-torsional buckling of members subject to bending

B.1 Basic case

The basic case for lateral-torsional buckling should be taken as that of a simply supported beam of uniform cross-section with equal flanges, restrained at its supports against lateral deflection and against rotation about its longitudinal axis, but otherwise unrestrained, and subject to a uniform moment about its major axis.

Modifications should be made to the basic case for:

— non-uniform members, using the equivalent slenderness factor n;

— unequal flanges, using the monosymmetry index ψ;

— differing conditions of restraint or support, using the effective length factor L_E/L;

— differing conditions of loading, using the equivalent uniform moment factor m_{LT}.

B.2 Buckling resistance

B.2.1 *Bending strength*

The bending strength p_b for resistance to lateral-torsional buckling should be taken as the smaller root of:

$$(p_E - p_b)(p_y - p_b) = \eta_{LT} p_E p_b$$

from which the value of p_b may be obtained using:

$$p_b = \frac{p_E p_y}{\phi_{LT} + (\phi_{LT}^2 - p_E p_y)^{0.5}}$$

in which:

$$p_E = (\pi^2 E / \lambda_{LT}^2)$$

$$\phi_{LT} = \frac{p_y + (\eta_{LT} + 1) p_E}{2}$$

where

p_y is the design strength;

λ_{LT} is the equivalent slenderness.

B.2.2 *Perry factor and Robertson constant*

The Perry factor η_{LT} should be taken as follows:

a) for rolled sections:

$$\eta_{LT} = a_{LT}(\lambda_{LT} - \lambda_{L0})/1\,000 \quad \text{but} \quad \eta_{LT} \geq 0$$

b) for welded sections:

— if $\lambda_{LT} \leq \lambda_{L0}$: $\eta_{LT} = 0$
— if $\lambda_{L0} < \lambda_{LT} < 2\lambda_{L0}$: $\eta_{LT} = 2a_{LT}(\lambda_{LT} - \lambda_{L0})/1\,000$
— if $2\lambda_{L0} \leq \lambda_{LT} \leq 3\lambda_{L0}$: $\eta_{LT} = 2a_{LT}\lambda_{L0}/1\,000$
— if $\lambda_{LT} > 3\lambda_{L0}$: $\eta_{LT} = a_{LT}(\lambda_{LT} - \lambda_{L0})/1\,000$

The Robertson constant a_{LT} should be taken as 7.0 and the limiting equivalent slenderness λ_{L0} should be taken as $0.4(\pi^2 E/p_y)^{0.5}$.

B.2.3 ***Uniform I, H and channel sections with equal flanges***

For segments of uniform cross-section the equivalent slenderness λ_{LT} should be taken as:

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$

in which:

$$v = \frac{1}{[1 + 0.05(\lambda/x)^2]^{0.25}}$$

where u and x are defined as follows, λ is defined in **4.3.6.7** and β_W is defined in **4.3.6.9**.

The buckling parameter u and x the torsional index are given by:

— for I or H-sections:

$$u = \left(\frac{4S_x{}^2\gamma}{A^2h_s{}^2}\right)^{0.25}$$

$$x = 0.566h_s(A/J)^{0.5}$$

— for channels with equal flanges:

$$u = \left(\frac{I_yS_x{}^2\gamma}{A^2H}\right)^{0.25}$$

$$x = 1.132\left(\frac{AH}{I_yJ}\right)^{0.5}$$

in which:

$$\gamma = (1 - I_y/I_x)$$

where

A	is the cross-sectional area;
H	is the warping constant;
h_s	is the distance between the shear centres of the flanges;
I_x	is the second moment of area about the major axis;
I_y	is the second moment of area about the minor axis;
J	is the torsion constant;
S_x	is the plastic modulus about the major axis.

Alternatively, for welded I and H-sections with equal flanges, the torsional index x may be obtained from:

$$x = (D - T)\left[\frac{2BT + dt}{2BT^3 + dt^3}\right]^{0.5}$$

B.2.4 *Uniform I and H-sections with unequal flanges*

B.2.4.1 *Equivalent slenderness*

For segments of uniform cross-section the equivalent slenderness λ_{LT} should be taken as:

$$\lambda_{LT} = u v \lambda \sqrt{\beta_W}$$

in which:

$$v = \frac{1}{[(4\eta(1-\eta) + 0.05(\lambda/x)^2 + \psi^2)^{0.5} + \psi]^{0.5}}$$

where the torsional index x, the flange ratio η and the monosymmetry index ψ are as defined below, u is as defined for an I or H-section in **B.2.3** and the other parameters are as detailed in **4.3.6.7**.

The torsional index x for an I or H-section is given by:

$$x = 0.566 h_s (A/J)^{0.5}$$

in which:

$$h_s = \left(D - \frac{(T_c + T_t)}{2}\right)$$

where

T_c is the thickness of the compression flange;

T_t is the thickness of the tension flange.

Alternatively, for welded I and H-sections with unequal flanges, the torsional index may be obtained from:

$$x = h_s \left[\frac{B_c T_c + B_t T_t + dt}{B_c T_c^3 + B_t T_t^3 + dt^3}\right]^{0.5}$$

where

B_c is the width of the compression flange;

B_t is the width of the tension flange.

The flange ratio η is given by:

$$\eta = \frac{I_{yc}}{I_{yc} + I_{yt}}$$

where

I_{yc} is the second moment of area of the compression flange about the minor axis of the section;

I_{yt} is the second moment of area of the tension flange about the minor axis of the section.

The monosymmetry index ψ may be evaluated using:

$$\psi = \frac{1}{h_s}\left(\frac{2(I_{yc}h_c - I_{yt}h_t)}{(I_{yc} + I_{yt})} - \frac{(I_{yc}h_c - I_{yt}h_t) + (B_c T_c h_c^3 - B_t T_t h_t^3) + \frac{t}{4}[d_c^4 - d_t^4]}{I_x}\right)$$

in which:

$d_c = h_c - T_c/2;$

$d_t = h_t - T_t/2;$

$I_{yc} = B_c^3 T_c/12;$

$I_{yt} = B_t^3 T_t/12;$

where h_c and h_t are the distances from the centres of the flanges to the centroid of the section.

Alternatively, the monosymmetry index ψ may be approximated as detailed in **4.3.6.7**.

B.2.4.2 *Double-curvature bending*

In segments subject to double-curvature bending, sections with unequal flanges should satisfy both of the following criteria:

$M_{x,1} \leq M_{b,1}$

$M_{x,2} \leq M_{b,2}$

where

$M_{b,1}$ is the lateral-torsional buckling resistance moment for compression in the top flange;

$M_{b,2}$ is the lateral-torsional buckling resistance moment for compression in the bottom flange;

$M_{x,1}$ is the maximum major axis moment producing compression in the top flange;

$M_{x,2}$ is the maximum major axis moment producing compression in the bottom flange.

B.2.5 ***Tapered or haunched I, H or channel section members***

For a member or segment in which the cross-section varies along its length, the equivalent uniform moment factor m_{LT} should be taken as 1.0. Throughout the length of the segment the major axis moment M_x should not exceed the corresponding value of M_b determined using:

— the properties of the cross-section at that point;

— a constant value of the bending strength p_b based on the properties of the cross-section at the point of maximum moment within that segment, determined using a modified equivalent slenderness λ_{LT} taken as n times the value of λ_{LT} obtained from **B.2.3** or **B.2.4**.

Provided that R_f is not less than 0.2 the equivalent slenderness factor n should be determined from:

$$n = (1.5 - 0.5R_f) \quad \text{but} \quad n \geq 1.0$$

where R_f is the ratio of the flange area at the point of minimum moment within the segment to that at the point of maximum moment. The value of the ratio R_f should be taken as the smaller of the values based either on the ratio of the total area of both flanges or on the ratio of the area of the compression flange only.

B.2.6 ***Box sections (including RHS)***

B.2.6.1 *Equivalent slenderness*

For a closed box section, including an RHS, the equivalent slenderness λ_{LT} should be taken as:

$$\lambda_{LT} = 2.25(\phi_b \lambda \beta_W)^{0.5}$$

in which:

$$\phi_b = \left(\frac{S_x^2 \gamma_b}{AJ}\right)^{0.5}$$

$$\gamma_b = \left(1 - \frac{I_y}{I_x}\right)\left(1 - \frac{J}{2.6 I_x}\right)$$

$$\lambda = L_E/r_y$$

where

A is the cross-sectional area;

I_x is the second moment of area about the major axis;

I_y is the second moment of area about the minor axis;

J is the torsion constant, see **B.2.6.2** and **B.2.6.3**;

L_E is the effective length for lateral-torsional buckling from **4.3.5**;

r_y is the minor axis radius of gyration;

S_x is the plastic modulus about the major axis;

β_W is the ratio defined in **4.3.6.9**.

B.2.6.2 *Torsion constant for a box section*

For a closed box section the torsion constant J may be obtained from the approximate formula:

$$J = 4A_h^2/\Sigma(s_i/t_i)$$

where

A_h is the area enclosed by the mean perimeter;

s_i is the breadth of an individual enclosing element i;

t_i is the thickness of element i.

B.2.6.3 *Torsion constant for an RHS*

For an RHS, as an alternative to the approximate formula given in **B.2.6.2** the following more accurate formula may be used:

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

where

h is the mean perimeter;

t is the thickness of the RHS.

B.2.7 ***Plates and flats***

For an individual plate, flat, or other solid rectangular cross-section subject to a moment about its major axis, the equivalent uniform moment factor m_{LT} should be taken as 1.0 and the buckling resistance moment M_b should be determined as defined in **4.3.6.4** using the equivalent slenderness λ_{LT} given by:

$$\lambda_{LT} = 2.8\left(\frac{\beta_W L_E d}{t^2}\right)^{0.5}$$

where

d is the depth;

L_E is the effective length for lateral-torsional buckling from **4.3.5**;

t is the thickness;

β_W is the ratio defined in **4.3.6.9**.

B.2.8 ***T-sections***

B.2.8.1 *Axes*

For symmetrical T-sections the axis perpendicular to the centreline of the web should always be taken as the x-x axis and the axis on the centreline of the web should always be taken as the y-y axis, irrespective to which is the major axis and which the minor axis.

B.2.8.2 *Equivalent slenderness*

For a T-section the equivalent uniform moment factor m_{LT} should be taken as 1.0 and the buckling resistance M_b should be determined as defined in **4.3.6.4** using the equivalent slenderness λ_{LT} obtained from the following:

a) if $I_{xx} = I_{yy}$: lateral-torsional buckling does not occur and λ_{LT} is zero;

b) if $I_{yy} > I_{xx}$: lateral-torsional buckling occurs about the x-x axis and λ_{LT} is given by:

$$\lambda_{LT} = 2.8\left(\frac{\beta_W L_E B}{T^2}\right)^{0.5}$$

c) if $I_{xx} > I_{yy}$: lateral-torsional buckling occurs about the y-y axis and λ_{LT} is given by:

$$\lambda_{LT} = u v \lambda \sqrt{\beta_W}$$

in which:

$$u = \left(\frac{4 S_x^{\ 2} \gamma}{A^2 (D - T/2)^2}\right)^{0.25}$$

$$v = \frac{1}{[(w + 0.05(\lambda/x)^2 + \psi^2)^{0.5} + \psi]^{0.5}}$$

$$w = \frac{4H}{I_y (D - T/2)^2}$$

$$x = 0.566(D - T/2)(A/J)^{0.5}$$

$$\gamma = (1 - I_y/I_x)$$

$$\lambda = L_E / r_y$$

where

- B is the flange width;
- D is the overall depth of the T-section;
- H is the warping constant, see **B.2.8.3**;
- I_x is the second moment of area about the x-x axis;
- I_y is the second moment of area about the y-y axis;
- L_E is the effective length for lateral-torsional buckling from **4.3.5**;
- r_y is the minor axis radius of gyration;
- S_x is the plastic modulus about the x-x axis;
- T is the flange thickness;
- t is the web or stem thickness;
- β_W is the ratio defined in **4.3.6.9**.

The monosymmetry index ψ should be taken as positive when the flange of the T-section is in compression and negative when the flange is in tension. It may be evaluated using:

$$\psi = \left(2y_o - \frac{y_o B^3 T/12 + BTy_o{}^3 + \frac{t}{4}[(c-T)^4 - (D-c)^4]}{I_x}\right)\frac{1}{(D-T/2)}$$

in which:

$$y_o = c - T/2$$

where c is the distance from the outside of the flange to the centroid of the section.

When the flange is in tension the monosymmetry index ψ may conservatively be taken as –1.0.

B.2.8.3 *Warping constant*

For a T-section the warping constant H should be obtained from:

$$H = \frac{B^3T^3}{144} + \frac{(D-T/2)^3t^3}{36}$$

B.2.9 ***Angle sections***

B.2.9.1 *Axes*

Except when using the approximate method given in **4.3.8**, moments applied to unrestrained angles should be related to their principal axes u-u and v-v, not their geometric axes x-x and y-y.

B.2.9.2 *Equal angles*

For a single equal leg angle, subject to moments about its major axis u-u, the equivalent slenderness λ_{LT} should be taken as:

$$\lambda_{LT} = 2.25(\phi_a\lambda_v)^{0.5}$$

in which:

$$\phi_a = \left(\frac{Z_u{}^2\gamma_a}{AJ}\right)^{0.5}$$

$$\gamma_a = (1 - I_v/I_u)$$

$$\lambda_v = L_v/r_v$$

where

- A is the cross-sectional area;
- I_u is the second moment of area about the major axis;
- I_v is the second moment of area about the minor axis;
- J is the torsion constant;
- L_v is the length between points where the member is restrained in both the x-x and y-y directions;
- r_v is the radius of gyration about the minor axis v-v;
- Z_u is the section modulus about the major axis u-u.

B.2.9.3 *Unequal angles*

For a single unequal leg angle, subject to moments about its major axis u-u, the equivalent slenderness λ_{LT} should be taken as:

$$\lambda_{LT} = 2.25 \nu_a (\phi_a \lambda_v)^{0.5}$$

in which:

$$\nu_a = \frac{1}{\left[\left(1 + \left(\frac{4.5\psi_a}{\lambda_v}\right)^2\right)^{0.5} + \frac{4.5\psi_a}{\lambda_v}\right]^{0.5}}$$

The monosymmetry index ψ_a for an unequal angle should be taken as positive when the short leg is in compression and negative when the long leg is in compression. If the long leg is in compression anywhere within the segment length L_v then ψ_a should be taken as negative. It may be evaluated from:

$$\psi_a = \left[2\nu_o - \frac{\int \nu_i (u_i^2 + \nu_i^2) \mathrm{d}A}{I_u}\right]\frac{1}{t}$$

in which u_i and ν_i are the coordinates of an element of the cross-section and ν_o is the coordinate of the shear centre along the v-v axis, relative to the centroid of the cross-section.

B.3 Internal moments

B.3.1 ***General***

The additional internal "second-order" minor-axis moment (equivalent to the strut action moment in a compression member) in a member subject to external applied major axis moment, should be taken as having a maximum value $M_{y,max}$ midway between points of inflexion of the buckled shape (the points between which the effective length L_E is measured) given by:

$$M_{y,max} = (p_y/p_b - 1)(M_{cy}/M_{cx})m_{LT}M_x$$

where

- M_{cx} is the major axis moment capacity of the cross-section, assuming zero shear, see **4.2.5**;
- M_{cy} is the minor axis moment capacity of the cross-section, assuming zero shear, see **4.2.5**;
- M_x is the maximum major axis moment in the length L of the segment;
- m_{LT} is the equivalent uniform moment factor for lateral-torsional buckling, see **4.3.6.6**;
- p_b is the bending strength for resistance to lateral-torsional buckling, see **4.3.6.5** (or **B.2.1**).

The additional internal minor axis moment M_{ys} at a distance L_z along the member from a point of inflexion should be obtained from:

$$M_{ys} = M_{y,max} \sin(180 L_z / L_E)$$

B.3.2 ***T-sections***

In applying **B.3.1** to a T-section, the subscripts x and y should always be taken as referring to the major axis and the minor axis respectively, even where the opposite subscript is used in **B.2.8.2**b).

B.3.3 ***Angles***

In applying **B.3.1** to an angle, the subscripts x and y should be taken as referring to the major axis u–u and minor axis v–v respectively.

Annex C (normative) Compressive strength

C.1 Strut formula

The compressive strength p_c should be taken as the smaller root of:

$$(p_E - p_c)(p_y - p_c) = \eta p_E p_c$$

from which the value of p_c may be obtained using:

$$p_c = \frac{p_E p_y}{\phi + (\phi^2 - p_E p_y)^{0.5}}$$

in which:

$$\phi = \frac{p_y + (\eta + 1)p_E}{2}$$

$$p_E = (\pi^2 E/\lambda^2)$$

where

p_y is the design strength;

λ is the slenderness, see **4.7.2**.

C.2 Perry factor and Robertson constant

The Perry factor η for flexural buckling under axial force should be taken as:

$$\eta = a(\lambda - \lambda_0)/1\,000 \quad \text{but} \quad \eta \geq 0$$

in which the limiting slenderness λ_0 should be taken as $0.2(\pi^2 E/p_y)^{0.5}$.

The Robertson constant a should be taken as follows:

- for strut curve (a): $a = 2.0$;
- for strut curve (b): $a = 3.5$;
- for strut curve (c): $a = 5.5$;
- for strut curve (d): $a = 8.0$.

C.3 Strut action

The additional internal "second-order" bending moment due to strut action in a member not subject to external applied moment, should be taken as having a maximum value M_{max} midway between points of inflexion of the buckled shape (the points between which the effective length L_E is measured) given by:

$$M_{max} = (p_y/p_c - 1)f_c S$$

where

f_c is the compressive stress due to axial force;

p_c is the compressive strength, see **4.7.5** (or **C.1**);

S is the plastic modulus of the strut for bending about the axis of buckling.

The strut action moment M_s at a distance L_z along the member from a point of inflexion should be obtained from:

$$M_s = M_{max} \sin(180 L_z/L_E)$$

Annex D (normative) Effective lengths of columns in simple structures

D.1 Columns for single storey buildings

D.1.1 *Typical cases*

The effective lengths of columns for single storey buildings of simple design, see **2.1.2.2**, should be determined by reference to the typical cases illustrated in Figure D.1 to Figure D.5, provided that the following conditions apply.

a) In the plane of the diagram the columns act as cantilevers tied together by the roof trusses, but in this plane the tops of the columns are not otherwise held in position or restrained in direction.

b) Perpendicular to the plane of the diagram, the tops of the columns are effectively held in position by members connecting them to a braced bay, or by other suitable means. In the case of Figure D.3 to Figure D.5 the braced bay also holds the columns in position at crane girder level.

c) The bases of the columns are effectively held in position and restrained in direction in both planes.

d) The foundations are capable of providing restraint commensurate with that provided by the base.

D.1.2 *Variations*

Where the conditions differ from those detailed in **D.1.1**, the following modifications should be made to the effective lengths shown in Figure D.1 to Figure D.5.

a) If, in the plane of the diagram, the tops of the columns are effectively held in position by horizontal bracing or other suitable means, the effective lengths in this plane should be obtained from Table 22a).

b) If, in the plane of the diagram, the roof truss or other roof member is connected to the columns by a connection capable of transmitting appreciable moment, the effective length of the stanchion in this plane should be determined in accordance with Annex E.

c) If, perpendicular to the plane of the diagram, one flange only of the stanchion is restrained at intervals by sheeting rails, then for buckling out-of-plane the method given in Annex G should be used.

d) If, perpendicular to the plane of the diagram, the base of the column is not effectively restrained in direction, the effective lengths $0.85L$ or $0.85L_1$ in Figure D.1 to Figure D.5 should be increased to $1.0L$ or $1.0L_1$ respectively.

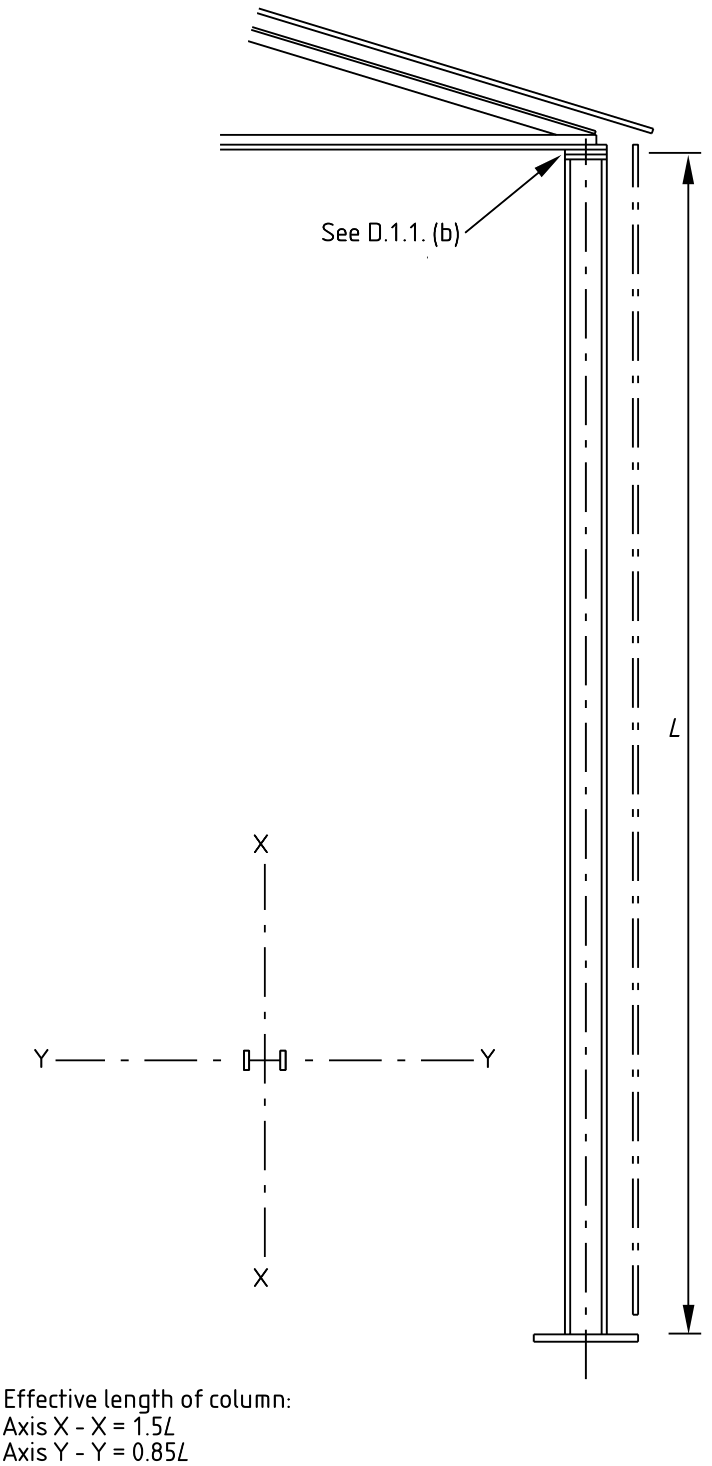


Figure D.1 — Side column without intermediate lateral restraint

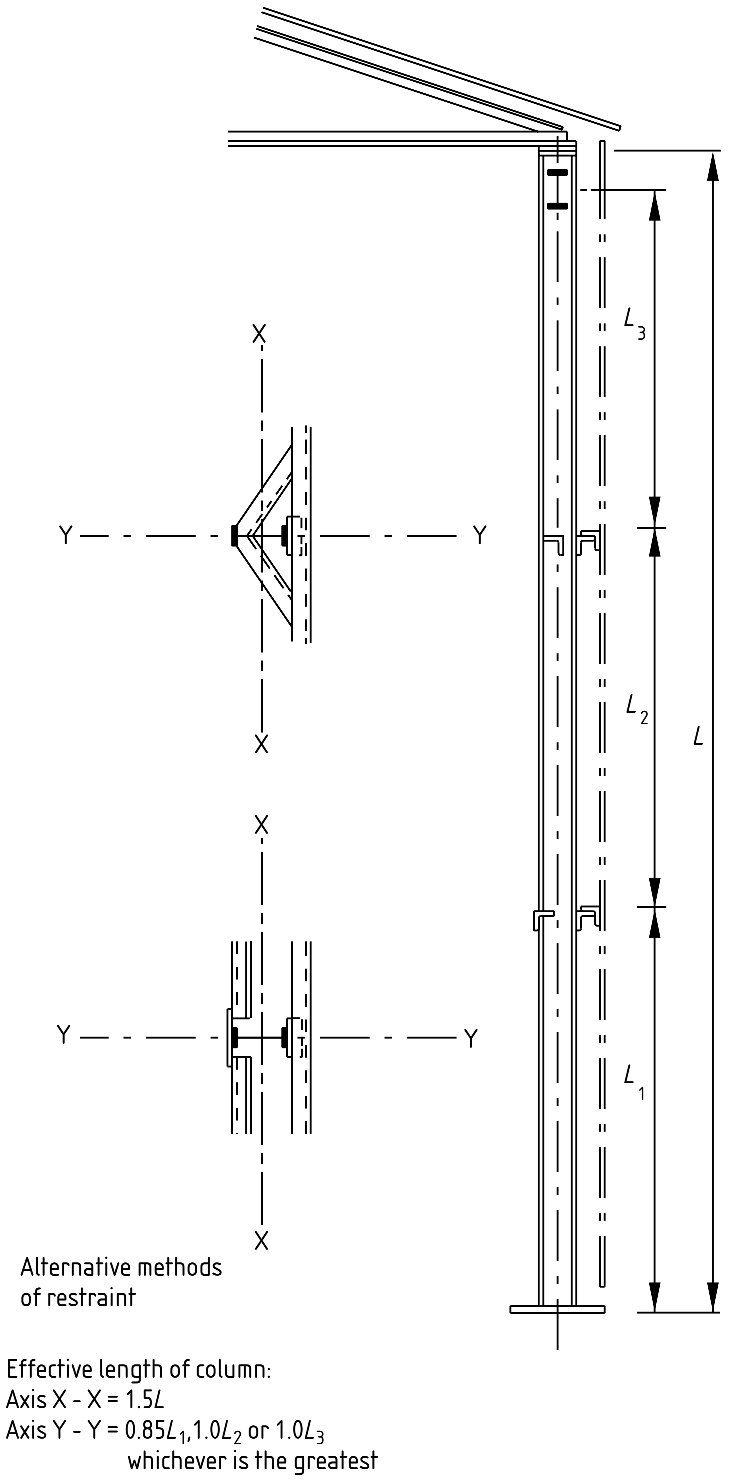
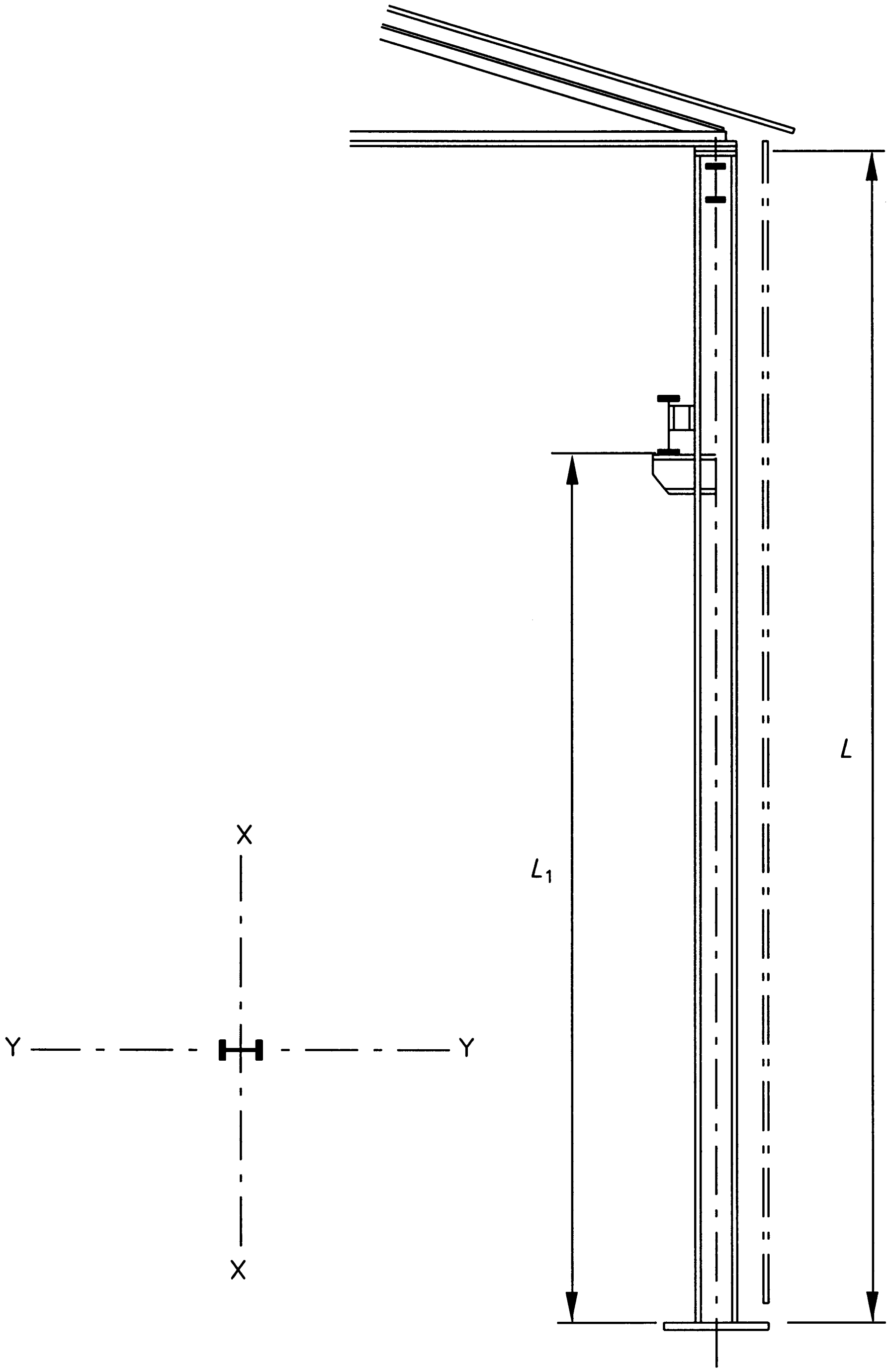


Figure D.2 — Side column with intermediate lateral restraint to both flanges



Effective length of column:
Axis X - X = 1.5L
Axis Y - Y = 0.85L_1

Figure D.3 — Simple side column with crane gantry beams

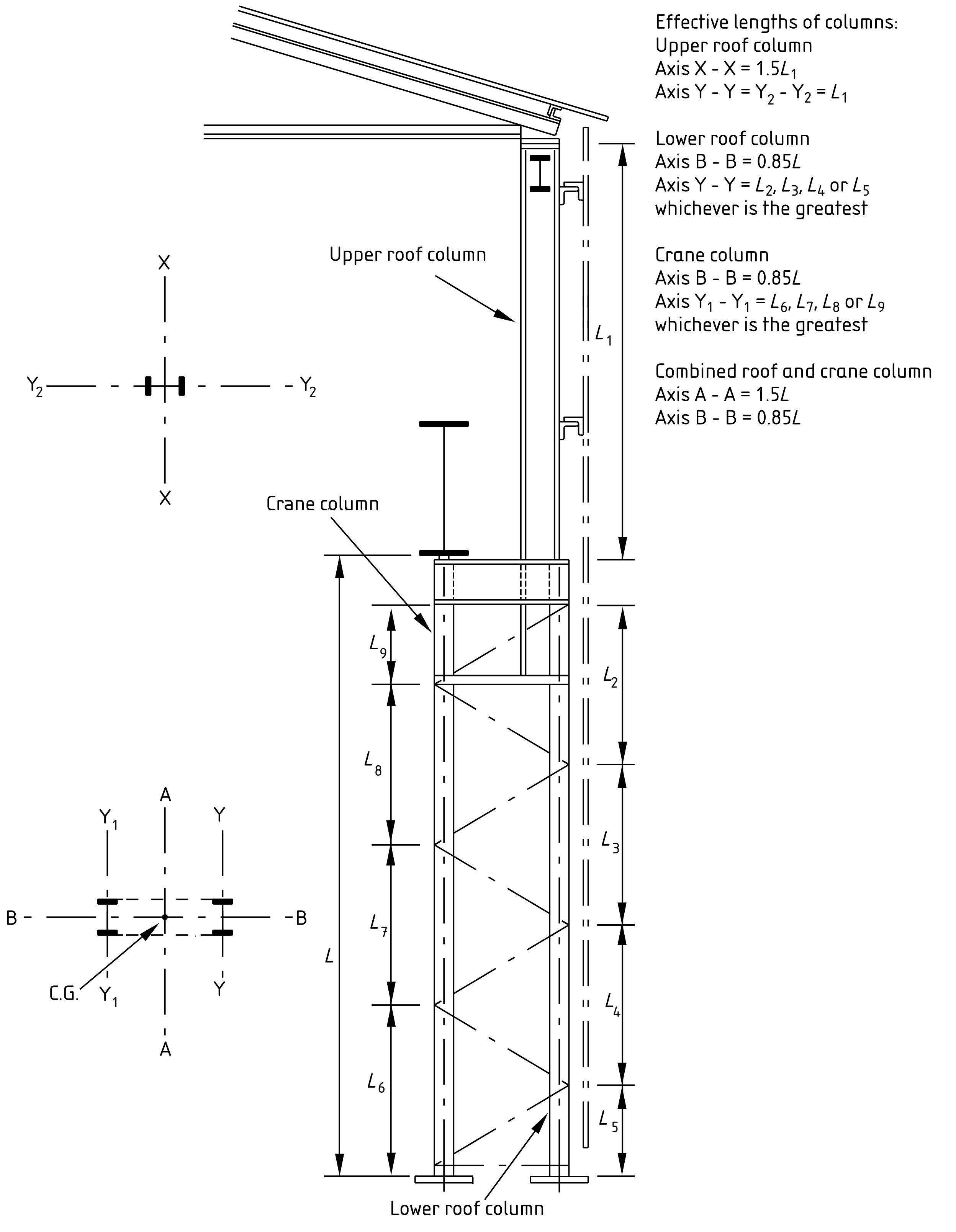


Figure D.4 — Compound side column with crane gantry beams

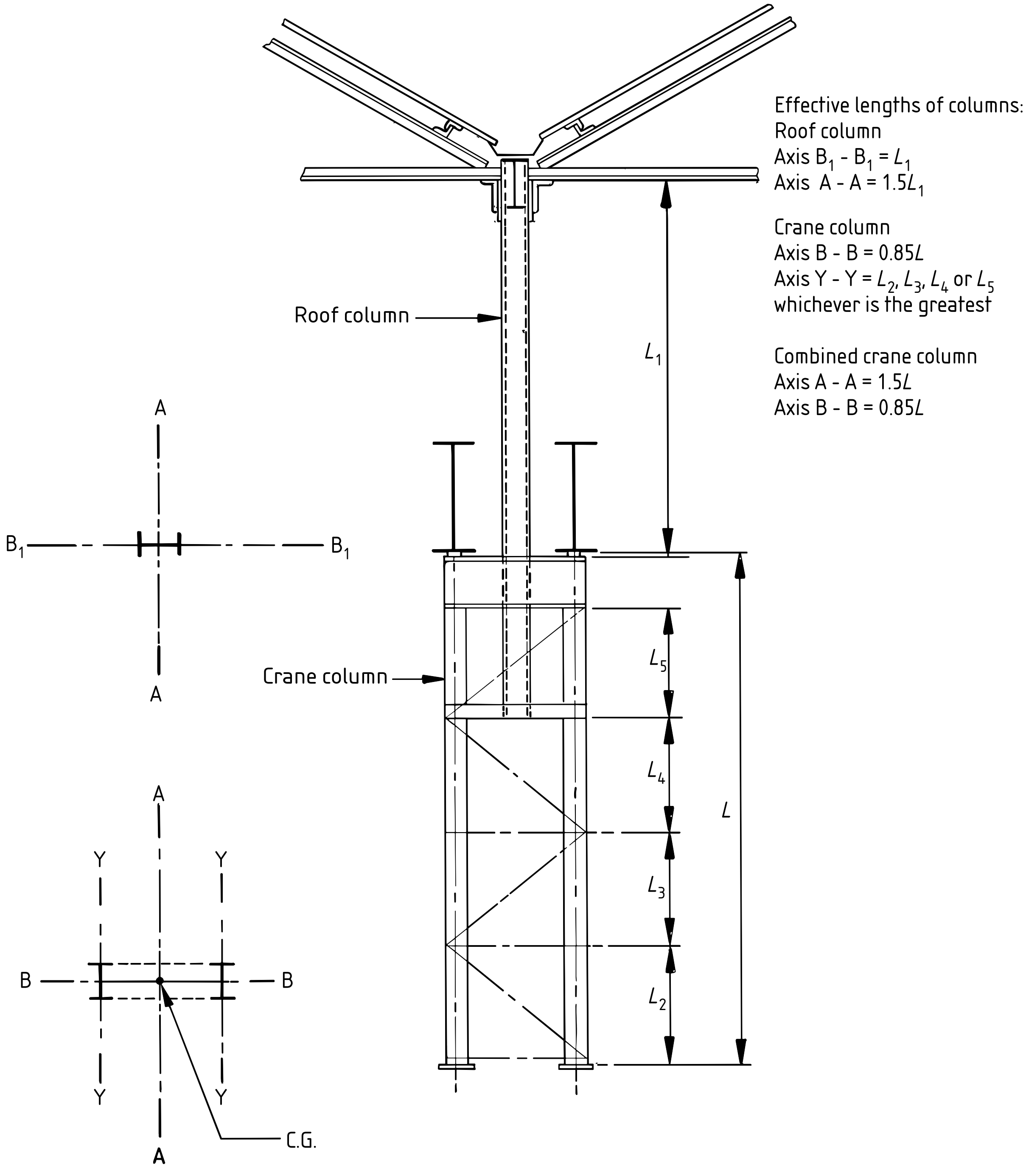


Figure D.5 — Compound valley column with crane gantry beams

D.2 Columns supporting internal platform floors

The effective lengths of columns supporting internal platform floors of simple design, see **2.1.2.2**, should be determined from Table D.1, depending on the conditions of directional restraint at the head and the base of the column in the relevant plane, and on whether the platform is braced against sway in that plane by some appropriate means other than the strength and stiffness of the columns themselves.

For columns that are unbraced in the relevant plane, where at least five columns act together to resist sway, the reduced effective lengths given in case c) of Table D.1 may be used, except for columns supporting storage loads.

In assessing the conditions of fixity, no greater directional restraint should be assumed than can reliably be provided at the head of a column by the cap-plate, the beams and the connection details, or at the base of a column by the baseplate, the base slab and the connection between them.

Table D.1 — Effective lengths of columns for internal platform floors

Case	Directional restraint at base of column	Directional restraint at head of column			
		Effectively restrained	Partially restrained	Nominally pinned	Truly pinned
a) Braced column	Effectively restrained	0.70*L*	0.80*L*	0.85*L*	0.90*L*
	Partially restrained	0.80*L*	0.85*L*	0.90*L*	0.95*L*
	Nominally pinned	0.85*L*	0.90*L*	0.95*L*	1.00*L*
	Truly pinned	0.90*L*	0.95*L*	1.00*L*	Avoid
b) Unbraced column Except as in case c)	Effectively restrained	1.50*L*	2.00*L*	2.50*L*	3.00*L*
	Partially restrained	2.00*L*	2.50*L*	3.00*L*	4.00*L*[a]
	Nominally pinned	2.50*L*	3.00*L*	Avoid	Avoid
	Truly pinned	3.00*L*	Avoid	Avoid	Avoid
c) Unbraced column	Effectively restrained	1.20*L*	1.50*L*	2.00*L*	2.50*L*
Five or more columns tied together	Partially restrained	1.50*L*	2.00*L*	2.50*L*	3.00*L*[a]
	Nominally pinned	2.00*L*	2.50*L*	Avoid	Avoid
No storage loads	Truly pinned	2.50*L*	Avoid	Avoid	Avoid

[a] For buckling about major axis only. To be avoided for buckling about minor axis.

Annex E (normative)
Effective lengths of compression members in continuous structures

E.1 General

The effective length L_E for in-plane buckling of a column or other compression member in a continuous structure with moment-resisting joints, should be determined using the methods given in this annex.

Generally, the effective length ratio L_E/L should be obtained from Figure E.1 for the non-sway mode or Figure E.2 for the sway mode, as appropriate.

Distribution factors for columns in multi-storey buildings may be determined using the limited frame method given in **E.2**. The stiffening effect of infill wall panels may be taken into account as given in **E.3**.

Distribution factors for other compression members should be determined by reference to **E.4**.

In structures in which frames with moment-resisting joints provide sway resistance to simple columns (or other columns that do not contribute to the sway resistance in that plane), the in-plane effective lengths of the columns contributing to the sway resistance should be increased as detailed in **E.5**.

Alternatively, the effective length may be derived from the elastic critical load factor, taking account of the vertical loads supported by the whole structure, see **E.6**.

E.2 Columns in multi-storey buildings

E.2.1 *Limited frame method*

For columns in multi-storey beam-and-column framed buildings with full continuity at moment-resisting joints and concrete or composite floor and roof slabs, the effective length L_E for in-plane buckling of a column-length may be determined on the basis of the limited frame shown in Figure E.3. The distribution factors k_1 and k_2 for the ends of the column-length should be obtained from:

$$k = \frac{\text{Total stiffness of the columns at the joint}}{\text{Total stiffness of all the members at the joint}}$$

If any member shown in Figure E.3 is not present in the actual structure, or is not rigidly connected to the column-length being designed, its stiffness should be taken as zero in determining distribution factors.

If the moment at one end of the column-length being designed exceeds 90 % of its reduced plastic moment capacity M_r in the presence of axial force, the distribution factor k for that end of the column-length should be taken as unity.

E.2.2 *Beam stiffness*

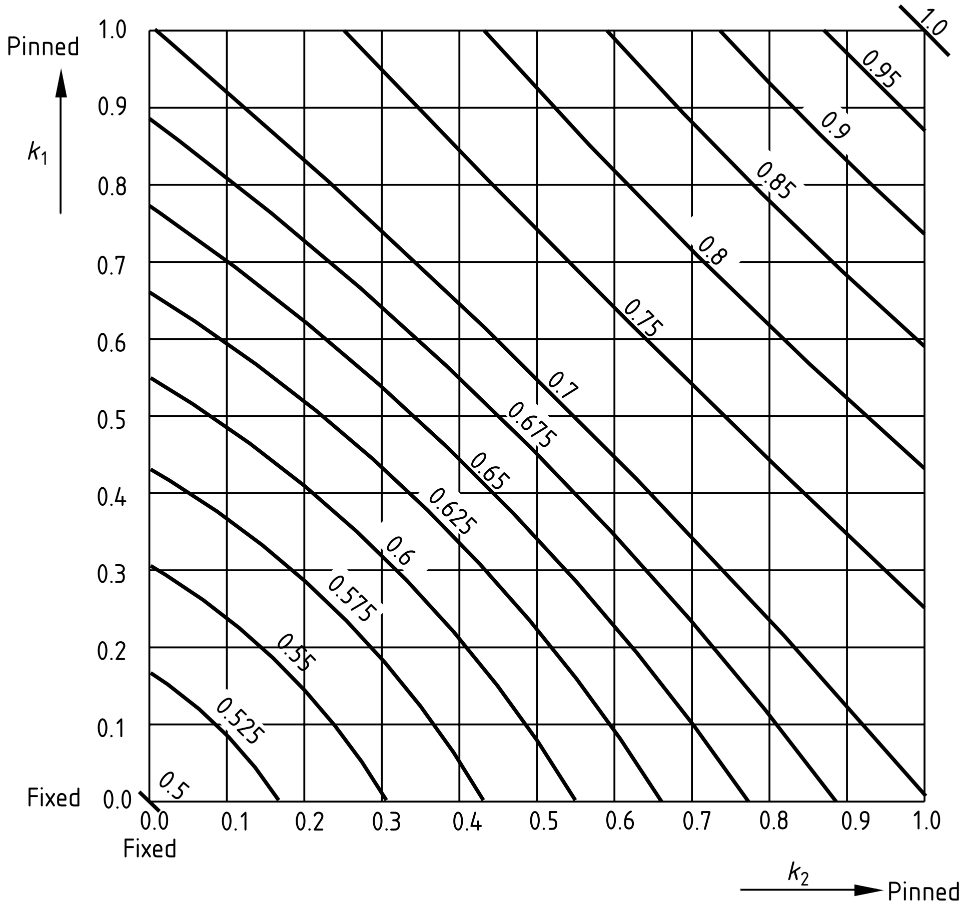
The stiffness coefficient K_b for a beam directly supporting a concrete or composite floor slab should normally be taken as I/L for both the sway mode and the non-sway mode, provided that the beam does not carry axial force, other than that due to sharing wind loads or notional horizontal loads between columns.

The stiffness coefficient K_b for any other beam should be obtained:

— from Table E.1 for other beams in buildings with concrete or composite floor slabs;

— by reference to **E.4.1** for beams in other rectilinear frames.

For beams with axial forces, reference should be made to **E.4.2**. If a beam has semi-rigid connections, its effective stiffness coefficient should be reduced accordingly.



NOTE This figure shows values of L_E/L that satisfy:

$$k_1 = \frac{1 - Ak_2}{A - Bk_2}$$

in which:

$$A = 1 - \frac{1}{4}\left[\frac{\alpha^2}{1 - \alpha\cot\alpha} + \alpha\cot\alpha\right]$$

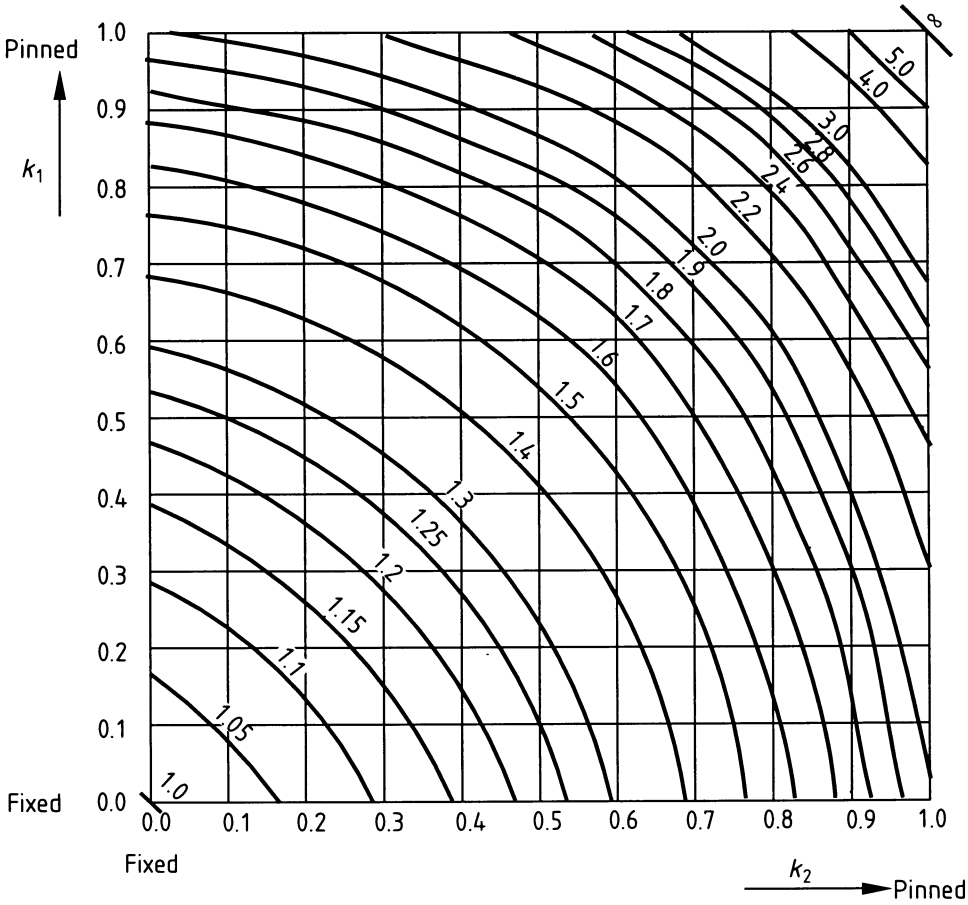
$$B = A^2 - \frac{1}{16}\left[\frac{\alpha^2}{1 - \alpha\cot\alpha} - \alpha\cot\alpha\right]^2$$

$$\alpha = \frac{\pi/2}{L_E/L}[\text{radians}]$$

A conservative value of L_E/L for given values of k_1 and k_2 may be obtained from:

$$L_E/L \approx 0.5 + 0.14(k_1 + k_2) + 0.055(k_1 + k_2)^2$$

Figure E.1 — Effective length ratio L_E/L for the non-sway buckling mode



NOTE This figure shows values of L_E/L that satisfy:

$$k_1 = \frac{1 - Ak_2}{A - Bk_2}$$

in which:

$$A = 1 - 0.25\alpha(\cot\alpha - \tan\alpha)$$

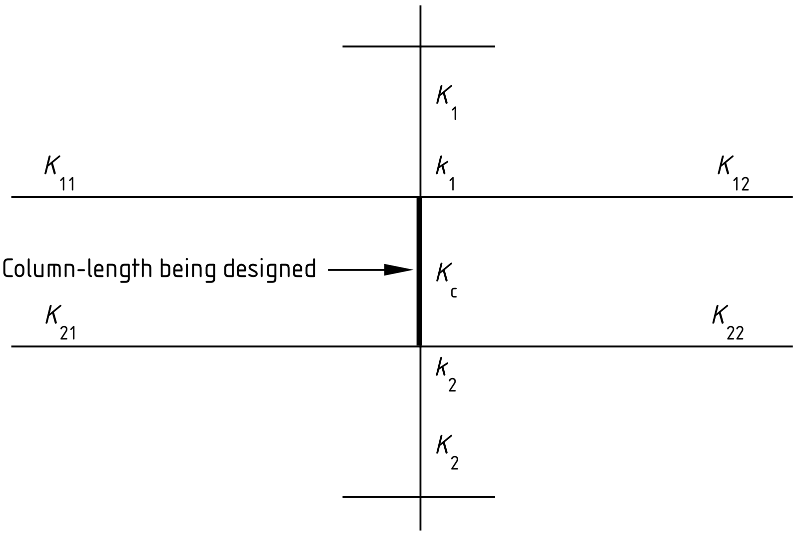
$$B = A^2 - [0.25\alpha(\cot\alpha + \tan\alpha)]^2$$

$$\alpha = \frac{\pi/2}{L_E/L}\text{[radians]}$$

A conservative value of L_E/L for given values of k_1 and k_2 may be obtained from:

$$L_E/L \approx \left[\frac{1 - 0.2(k_1 + k_2) - 0.12k_1k_2}{1 - 0.8(k_1 + k_2) + 0.6k_1k_2}\right]^{0.5}$$

Figure E.2 — Effective length ratio L_E/L for the sway buckling mode



Distribution factors:

$$k_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}}$$

$$k_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}}$$

where

K_1 and K_2 are the values of K_c for the adjacent column-lengths;

K_{11}, K_{12}, K_{21} and K_{22} are the values of K_b for the adjacent beams.

Figure E.3 — Distribution factors for continuous columns

Table E.1 — Stiffness coefficients K_b of beams in buildings with floor slabs

Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete or composite floor or roof slabs	$1.0I/L$	$1.0I/L$
Other beams with direct loads	$0.75I/L$	$1.0I/L$
Beams with end moments only	$0.5I/L$	$1.5I/L$

Wherever a peak moment in a beam exceeds 90 % of its reduced plastic moment capacity M_r in the presence of axial force, it should be treated as pinned at that point. If such a point occurs only at the far end of the beam the value of K_b should be taken as $0.75I/L$ (or the value from Table E.1, if less), otherwise K_b should be taken as zero.

In a structure designed using plastic analysis, a beam should be taken as having a stiffness coefficient K_b of zero unless it has been designed to remain elastic.

E.2.3 ***Base stiffness***

The base stiffness should be determined by reference to **5.1.3**. In determining the distribution factor k at the foot of a column, the base stiffness should be treated as a beam stiffness, not a column stiffness.

E.2.4 ***Column stiffness***

The stiffness coefficient K_c of an adjacent column-length above or below the column-length being designed should normally be taken as I/L. If the far end of an adjacent column-length is not rigidly connected, its stiffness coefficient K_c should be taken as $0.75I/L$.

Where a peak moment in an adjacent column-length exceeds 90 % of its reduced plastic moment capacity M_r in the presence of axial force, it should be treated as pinned at that point. If such a point occurs only at the far end of the adjacent column-length its stiffness coefficient K_c should be taken as $0.75I/L$, otherwise it should be taken as having a stiffness coefficient of zero.

E.3 Partial sway bracing

E.3.1 *General*

The stiffening effect of infill wall panels in buildings with unbraced frames (see **5.1.4**) that do not satisfy the conditions for non-sway frames given in **2.4.2.6**, may be taken into account. Provided that the panels conform to **E.3.2**, the method for columns in multi-storey buildings given in **E.2** may be used in association with Figure E.4 and Figure E.5 and the appropriate value of the relative stiffness k_p.

The value of k_p for the infill wall panels should be obtained from **E.3.3**. For intermediate values of k_p between 0 and 1 or 1 and 2 interpolation may be carried out, using Figure E.2 for $k_p = 0$.

E.3.2 *Wall panels*

To be taken as effective, a wall panel should be located in the plane of the frame and extend the full clear height of the storey. It may be composed of any material capable of resisting a diagonal compressive force.

E.3.3 *Relative stiffness*

The relative stiffness k_p of the effective bracing in any storey should be obtained using:

$$k_p = \frac{h^2 \Sigma S_p}{80 E \Sigma K_c} \quad \text{but } k_p \leq 2$$

where

E is the modulus of elasticity of steel;

h is the storey height;

ΣK_c is the sum of the stiffness coefficients K_c of the columns in that storey of the frame, see **E.2.4**;

ΣS_p is the sum of the spring stiffnesses (horizontal force per unit horizontal deflection) of the panels in that storey of the frame, see **E.3.4**.

E.3.4 *Stiffness of panels*

The spring stiffness S_p of an infill wall panel may be determined from:

$$S_p = \frac{0.6(h/b)tE_p}{[1 + (h/b)^2]^2}$$

where

b is the width of the panel;

E_p is the modulus of elasticity for the panel material;

h is the storey height;

t is the thickness of the panel.

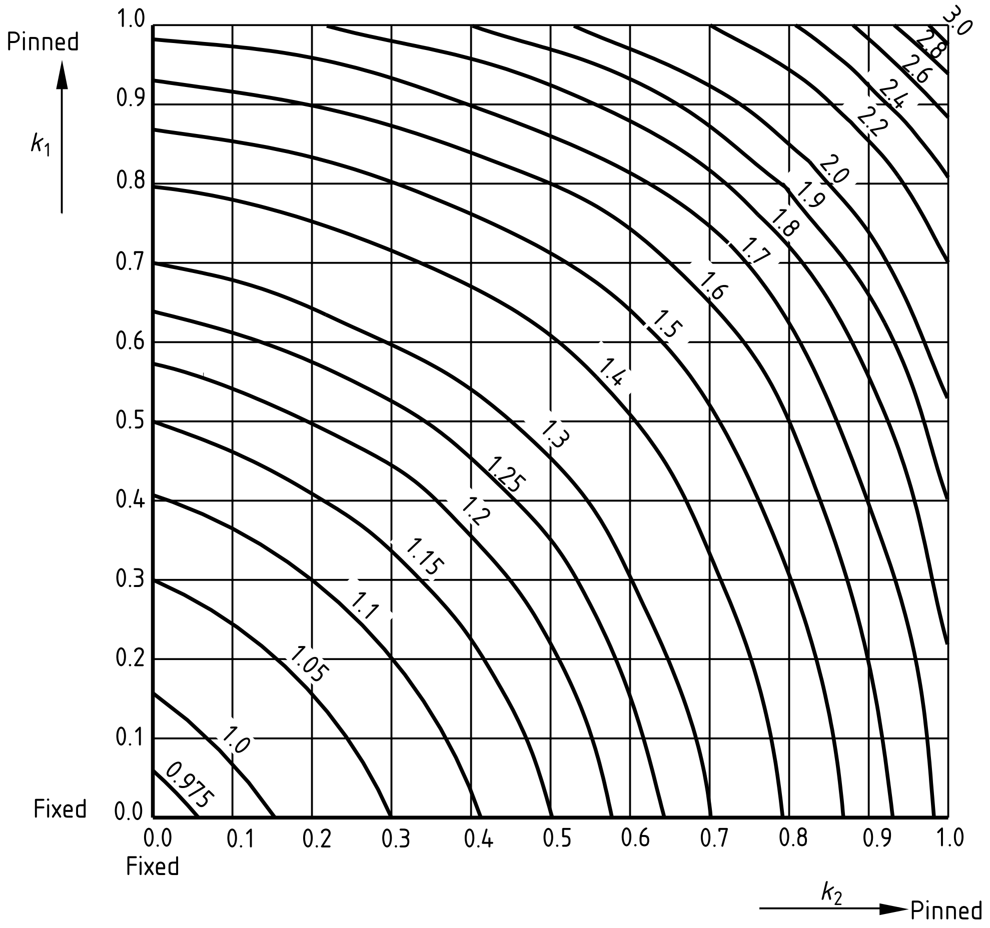


Figure E.4 — Effective length ratio L_E/L with partial sway bracing of relative stiffness k_p = 1

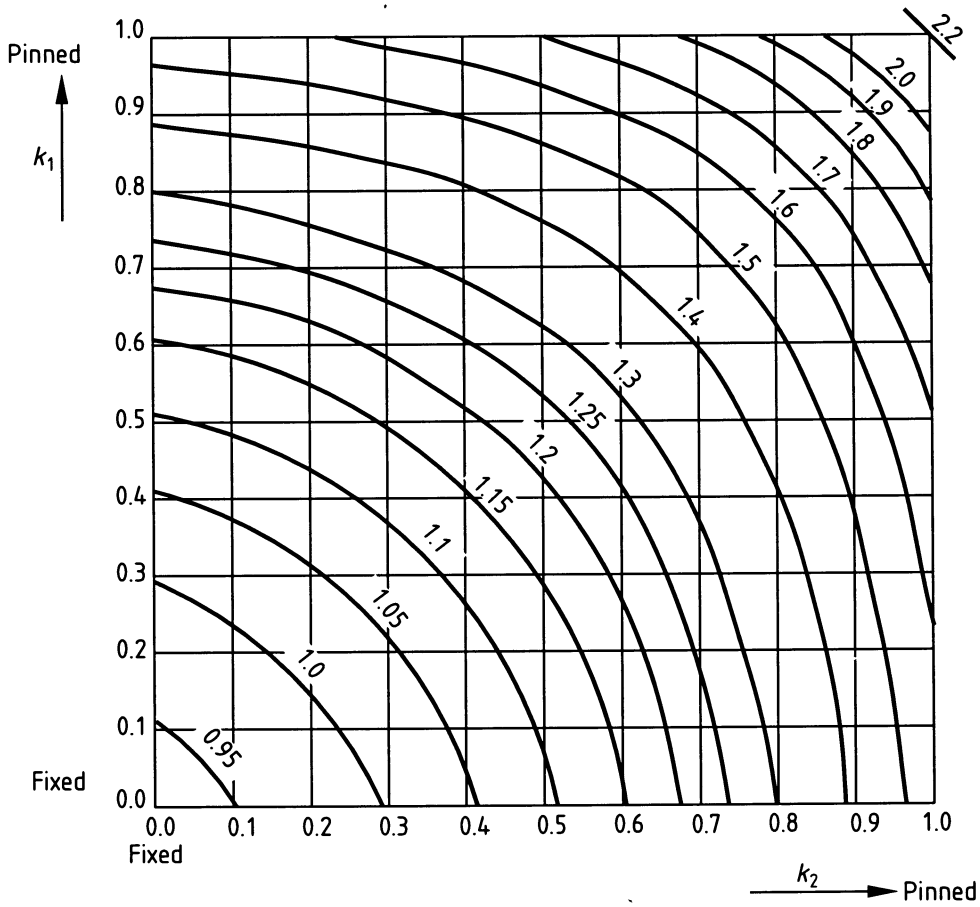


Figure E.5 — Effective length ratio L_E/L with partial sway bracing of relative stiffness $k_p = 2$

E.4 Other compression members

E.4.1 *Other rectilinear frames*

For rectilinear (beam-and-column type) frames with full continuity at moment-resisting joints, in which the beams do not have the benefit of supporting concrete or composite slabs, the in-plane effective lengths of the compression members may be determined using the limited frame method given in **E.2.1** as follows.

Provided that the beams, or other restraining members, are not subject to axial forces, and are designed to remain elastic, their stiffness coefficients should be obtained from Table E.2. The general case given in the table should be used to allow for the effects of loads on beams on their stiffness coefficients. If a beam has semi-rigid connections, its effective stiffness coefficient should be reduced accordingly.

Table E.2 — General stiffness coefficients K_b for beams

Rotational restraint at far end of beam	Beam stiffness coefficient K_b
Fixed at far end	$1.0I/L$
Pinned at far end	$0.75I/L$
Rotation at near end (double curvature)	$1.5I/L$
Rotation equal and opposite to that at near end (single curvature)	$0.5I/L$
General case. Rotation θ_a at near end and θ_b at far end	$(1 + 0.5\theta_b/\theta_a)I/L$

Where a peak moment in a beam exceeds 90 % of its reduced plastic moment capacity M_r in the presence of axial force, it should be treated as pinned at that point. In a structure designed using plastic analysis, a beam should be taken as having a stiffness coefficient K_b of zero, unless it has been designed to remain elastic.

Provided that the frame is regular in layout, unless a different fixity condition occurs at its far end the stiffness coefficients K_b for the beams should be taken as $0.5I/L$ (single curvature) for the non-sway mode and $1.5I/L$ (double curvature) for the sway mode.

E.4.2 *Effect of axial forces in restraining members*

Figure E.1 and Figure E.2 may also be used for compression members in continuous frames with moment-resisting joints in which the restraining members are subject to axial forces, provided that the effect of axial force on the stiffness of the restraining members is taken into account.

For the non-sway mode, the stiffness coefficients may be adjusted using appropriate stability functions. Alternatively, the increased stiffness due to axial tension should be neglected and the effects of axial compression should be allowed for by using the conservative approximations given in Table E.3.

For the sway mode, the in-plane effective length should be obtained from the elastic critical load factor λ_{cr} of the frame, see **E.6**.

Table E.3 — Approximate values of K_b for beams subject to axial compression

Rotational restraint at far end of beam	Beam stiffness coefficient K_b
Fixed at far end	$1.0I/L(1 - 0.4P_c/P_E)$
Pinned at far end	$0.75I/L(1 - 1.0P_c/P_E)$
Rotation as at near end (double curvature)	$1.5I/L(1 - 0.2P_c/P_E)$
Rotation equal and opposite to that at near end (single curvature)	$0.5I/L(1 - 1.0P_c/P_E)$
NOTE $P_E = \pi^2 EI/L^2$.	

E.5 Mixed frames

In moment-resisting or triangulated frames that provide sway resistance to simple columns or to other columns that do not contribute to the sway resistance in that plane, the in-plane effective lengths of the columns in each storey contributing to the sway resistance should be increased by the ratio β_F given by:

$$\beta_F = \sqrt{\frac{\Sigma V_{sr} + \Sigma V_{su}}{\Sigma V_{sr}}}$$

where

ΣV_{sr} is the total vertical load in that storey in the columns that resist sway in that plane;

ΣV_{su} is the total vertical load in that storey in the columns not resisting sway in that plane.

Alternatively, the effective lengths may be derived from the elastic critical load factor, see **E.6**, taking account of the vertical loads supported by the whole structure.

E.6 Using the elastic critical load factor

The non-sway mode and sway mode effective lengths L_E of all the columns in a structure may be based on the relevant non-sway or sway mode elastic critical load factor λ_{cr} of the whole structure in the relevant plane, provided that this is determined taking account of all the vertical loads on the structure, including those supported by simple columns and by frames that do not contribute to the sway resistance in that plane.

The in-plane effective length L_E of each column that contributes to the sway resistance of the structure in the relevant plane should be obtained using:

$$L_E = \sqrt{\frac{\pi^2 EI}{\lambda_{cr} F_c}}$$

where

- E is the modulus of elasticity of steel;
- F_c is the axial compression in the column;
- I is the in-plane second moment of area of the column;
- λ_{cr} is the relevant elastic critical load factor of the whole structure in that plane.

The use of these effective lengths should be strictly limited to checking the resistances of the columns to the vertical loads used to determine λ_{cr} for the structure analysed. Because the validity of this method is limited to predicting the in-plane buckling resistance P_c of the column length with the highest utilization ratio F_c/P_c a complete re-analysis should be carried out if any additional loads or revised member sizes are introduced, even where other members appear from the current analysis to have spare capacity.

The elastic critical load factor λ_{cr} may be obtained by second-order elastic analysis. Alternatively, the sway mode elastic critical load factor may be determined using Annex F.

Annex F (normative) Frame stability

F.1 General

The method given in this annex may be used to determine the sway mode elastic critical load factor λ_{cr} of a multi-storey frame in which all floor and roof beams are horizontal. Alternatively, second-order elastic analysis may be used, or other recognized methods that can be shown to predict values of λ_{cr} that will provide at least the same structural reliability as the method given here.

This method should not be used for single storey frames.

The elastic critical load factor λ_{cr} of a frame should be taken as the ratio by which each of the factored loads would have to be increased to cause elastic instability of the frame in a sway mode. The possibility of localized (storey-height) sway modes should also be taken into account.

F.2 Deflection method

Linear elastic analysis should be used to determine the notional horizontal deflections of the frame due to notional horizontal forces applied at each floor or roof level equal to 0.5 % of the total factored dead and imposed vertical load applied at that level.

Allowance should be made for the effects of any semi-rigid connections and for the effects of the base stiffness, determined as recommended in **5.1.3** for use in ultimate limit state calculations.

Any columns that are taken as pin-ended in resisting the sway moments in the plane under consideration, see **5.1.4**, should also be treated as pin-ended in this deflection calculation.

The value of the sway mode elastic critical load factor λ_{cr} should be determined using:

$$\lambda_{cr} = \frac{1}{200\phi_{max}}$$

in which ϕ_{max} is the largest value for any storey of the sway index ϕ for each storey, given by:

$$\phi = \frac{\delta_u - \delta_L}{h}$$

where

h is the storey height;

δ_u is the notional horizontal deflection of the top of the storey;

δ_L is the notional horizontal deflection of the bottom of the storey.

F.3 Partial sway bracing

In any storey of height h the stiffening effect of infill wall panels, characterized by the relative stiffness k_p given in **E.3.3** (up to a maximum value of $k_p = 2$), may be allowed for by introducing a diagonal bracing member in that storey, with an area A given by:

$$A = \frac{k_p \Sigma K_c}{h(h/b)}[1 + (h/b)^2]^{3/2}$$

where

b is the width of the braced bay;

$\sum K_c$ is the sum of the stiffness coefficients K_c of the columns in that storey, see **E.2.4**.

Annex G (normative)
Members with one flange laterally restrained

G.1 General

G.1.1 *Application*

This annex may be used for checking the out-of-plane stability of I-section members or segments in which one flange has continuous lateral restraint (or at least one intermediate lateral restraint at appropriate spacing, see **G.1.4**) and the non-restrained flange is in compression for at least part of its length. The segment length L_y should be taken between the points at which the non-restrained flange is laterally restrained.

The member may be uniform, tapered or haunched. The moments and forces applied to haunched or tapered members should be related to the axis of the minimum depth cross-section as a reference axis, see Figure G.1.

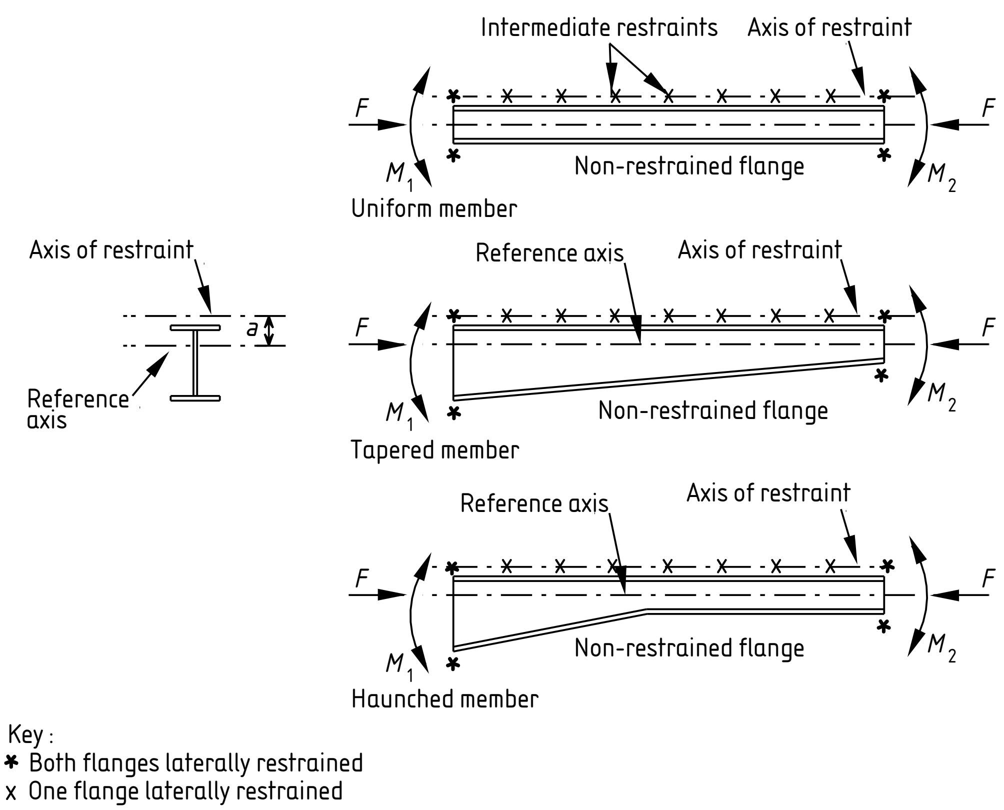


Figure G.1 — Members with one flange restrained

G.1.2 ***Types of haunching***

The recommendations in this annex may be used for haunched or tapered members with two types of haunching, see Figure G.2, as follows:

— two-flange haunch, of varying depth but with uniform flanges of equal size;

— three-flange haunch made by welding a T-section of varying depth to a uniform doubly symmetric I-section. The additional T-section should be obtained from a similar or larger I-section.

G.1.3 ***Section properties***

In haunched or tapered members or segments with one flange laterally restrained, the section properties should be based on cross-sections perpendicular to the reference axis, but with the thickness of the sloping flange measured perpendicular to its outer surface.

G.1.4 ***Procedure***

Generally the overall out-of-plane buckling of members or segments with one flange restrained should be checked using **G.2**. Members or segments that contain plastic hinge locations should be checked using **G.3**.

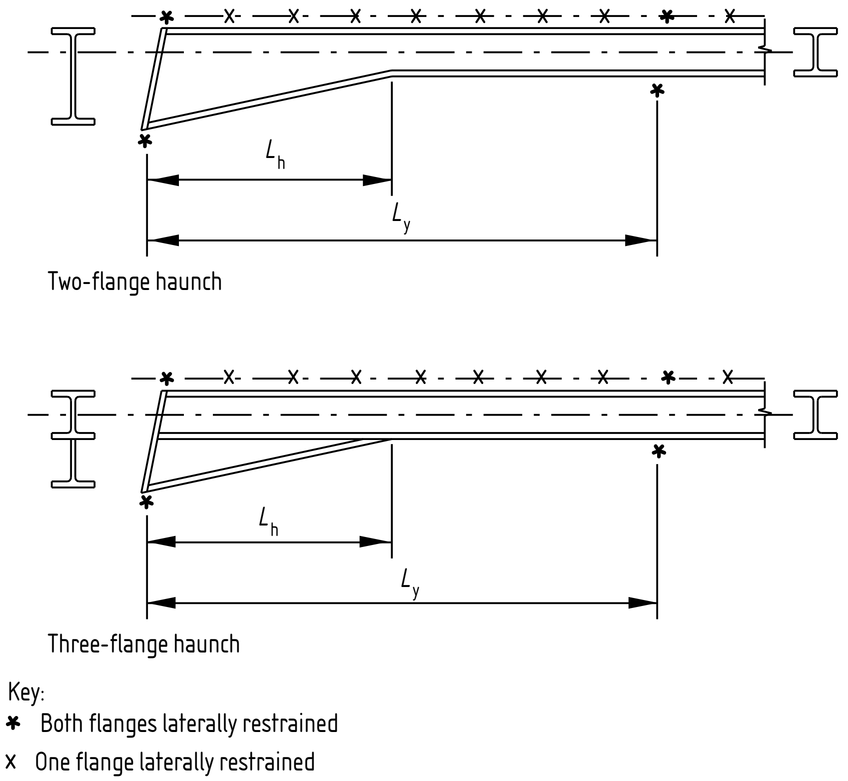


Figure G.2 — Types of haunches

In addition to the checks given in **G.2** and **G.3**:

— the cross-section capacity should be checked in accordance with **4.8.3.2**;

— the resistance to out-of-plane buckling between intermediate lateral restraints to the restrained flange should be checked using an effective length L_E equal to the spacing of these intermediate restraints (generally in accordance with **4.8.3.3** or **I.1**, with M_b and P_{cy} based on L_E, or in accordance with **5.3** in the case of plastic design);

— the in-plane buckling resistance should be checked, generally in accordance with **4.8.3.3** or **I.1**, but in accordance with Section 5 in the case of continuous structures.

G.2 Lateral buckling resistance

G.2.1 *Uniform members*

For a uniform member or segment with one flange laterally restrained and a non-restrained compression flange, in place of the methods given in **4.3**, **4.7** and **4.8**, its resistance to overall out-of-plane buckling about the axis of restraint should be checked by showing that the following criterion is satisfied:

$$\frac{F_c}{P_c} + \frac{m_t M_x}{M_b} \le 1$$

where

F_c is the axial compression;

M_b is the buckling resistance moment from **4.3.6** for an equivalent slenderness λ_{TB}, see **G.2.4**;

M_x is the maximum moment about the major axis;

m_t is the equivalent uniform moment factor for restrained buckling, see **G.4.2**;

P_c is the compression resistance from **4.7.4** for a slenderness λ_{TC}, see **G.2.3**.

G.2.2 ***Tapered or haunched members***

For a tapered or haunched member or segment with one flange laterally restrained and a non-restrained compression flange, its resistance to overall out-of-plane buckling about the axis of restraint should be checked as follows in place of the methods given in **4.3**, **4.7**, **4.8** and **B.2.5**. For this check it should be shown that the following criterion is satisfied at all points in the length of the member or segment at which the non-restrained flange is in compression:

$$M_{xi} \leq M_{bi}(1 - F_c/P_c)$$

where

F_c is the longitudinal compression on the reference axis;

M_{bi} is the buckling resistance moment M_b from **4.3.6** for an equivalent slenderness λ_{TB}, see **G.2.4.2**, based on the appropriate modulus S, S_{eff}, Z or Z_{eff} of the cross-section at the point i considered;

M_{xi} is the moment about the major axis acting at the point i considered;

P_c is the compression resistance from **4.7.4** for a slenderness λ_{TC}, see **G.2.3**, based on the properties of the minimum depth of cross-section within the segment length L_y.

G.2.3 ***Slenderness*** λ_{TC}

The slenderness λ_{TC} used to obtain the compression resistance P_c should be taken as:

$$\lambda_{TC} = y\lambda$$

in which:

$$y = \left[\frac{1 + (2a/h_s)^2}{1 + (2a/h_s)^2 + 0.05(\lambda/x)^2}\right]^{0.5}$$

$$\lambda = L_y/r_y$$

where

a is the distance between the reference axis and the axis of restraint, see Figure G.1;

h_s is the distance between the shear centres of the flanges;

L_y is the length of the segment;

r_y is the radius of gyration for buckling about the minor axis;

x is as defined in **4.3.6.8**.

G.2.4 ***Equivalent slenderness*** λ_{TB}

G.2.4.1 *Uniform members*

For a uniform member or segment, the equivalent slenderness λ_{TB} used to obtain the buckling resistance moment M_b should be obtained from:

$$\lambda_{TB} = n_t u v_t \lambda$$

in which:

$$v_t = \left[\frac{4a/h_s}{1 + (2a/h_s)^2 + 0.05(\lambda/x)^2}\right]^{0.5}$$

where

n_t is the slenderness correction factor, see **G.4.3**;

u is the buckling parameter defined in **4.3.6.8**;

and a, h_s, x and λ are as defined in **G.2.3**.

G.2.4.2 *Haunched and tapered members*

For a haunched or tapered member or segment, the equivalent slenderness λ_{TB} used to obtain the buckling resistance moment M_b should be obtained from:

$$\lambda_{TB} = c n_t \nu_t \lambda$$

in which:

— for a two-flange haunch:

$$\nu_t = \left[\frac{4a/h_s}{1 + (2a/h_s)^2 + 0.05(\lambda/x)^2}\right]^{0.5}$$

— for a three-flange haunch:

$$\nu_t = \left[\frac{4a/h_s}{1 + (2a/h_s)^2 + 0.05(\lambda/x)^2 + 0.02(\lambda/x_h)^2}\right]^{0.5}$$

where

c is the taper factor, see **G.2.5**;

x_h is the value of the torsional index x (see **4.3.6.8**) for the I-section from which the additional T-section forming the haunch is made;

and the other terms are as in **G.2.4.1**, except that h_s, r_y and x apply to the minimum depth of cross-section within the segment length L_y.

G.2.5 *Taper factor*

For an I-section with $D \geq 1.2B$ and $x \geq 20$ the taper factor c should be obtained as follows:

— for tapered members or segments:

$$c = 1 + \frac{3}{x-9}\left(\frac{D_{max}}{D_{min}} - 1\right)^{2/3}$$

— for haunched members or segments:

$$c = 1 + \frac{3}{x-9}(D_h/D_s)^{2/3}(L_h/L_y)^{0.5}$$

where

B is the breadth of the minimum depth cross-section;

D_h is the additional depth of the haunch or taper, see Figure G.3;

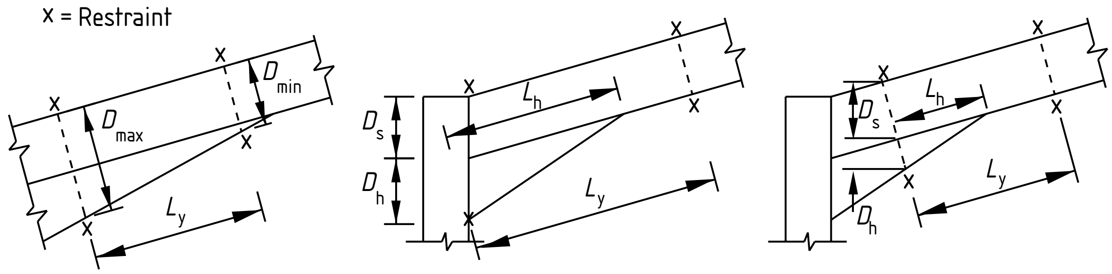
D_{max} is the maximum depth of cross-section within the length L_y, see Figure G.3;

D_{min} is the minimum depth of cross-section within the length L_y, see Figure G.3;

D_s is the vertical depth of the un-haunched section, see Figure G.3;

L_h is the length of haunch within the segment length L_y, see Figure G.3.

x is the torsional index of the minimum depth cross-section, see **4.3.6.8**.



a) Tapered segment b) Haunched segment c) Haunched segment

Figure G.3 — Dimensions defining taper factor

G.3 Lateral restraint adjacent to plastic hinges

G.3.1 *General*

Where a plastic hinge location occurs in a member or segment with one flange laterally restrained and compression in the non-restrained flange, the provision of lateral restraints at, and adjacent to, the plastic hinge location should satisfy the recommendations given in **G.3.2** and **G.3.3** respectively, except that these recommendations need not be applied at plastic hinge locations where it can be demonstrated that, under all ultimate state load combinations, the plastic hinge is "non-rotated", because under that load combination it is the last hinge to form or it is not yet fully formed.

The spacing of lateral restraints to the non-restrained flange of a member or segment not containing a plastic hinge should be such that **G.2** is satisfied for buckling out-of-plane.

In addition, the spacing of intermediate lateral restraints to the restrained flange should be such that both of the following conditions are satisfied:

a) adjacent to plastic hinge locations, the spacing of the intermediate lateral restraints should not exceed the value of L_m determined from **5.3.3**;

b) elsewhere **4.8.3.3** or **I.1** should be satisfied for out-of-plane buckling when checked using an effective length L_E equal to the spacing of the intermediate lateral restraints, except that this spacing need be not less than L_m determined from **5.3.3**.

G.3.2 *Restraints at plastic hinges*

Under all ultimate limit state load combinations, both flanges should have lateral restraint at each plastic hinge location, designed to resist a force equal to 2.5 % of the force in the compression flange. Where it is not practicable to provide such restraint directly at the hinge location, it should be provided within a distance $D/2$ along the length of the member, where D is its overall depth at the plastic hinge location.

G.3.3 *Segment adjacent to a plastic hinge*

G.3.3.1 *Uniform members*

For uniform members or segments under linear moment gradient, the length L_y between points at which the non-restrained flange is laterally restrained should not exceed the limiting spacing L_s given by:

$$L_s = \frac{L_k}{\sqrt{m_t}}\left(\frac{M_{px}}{M_{rx} + aF_c}\right)^{0.5}$$

where

a	is the distance between the reference axis and the axis of restraint, see Figure G.1;
F_c	is the axial compression;
L_k	is the limiting length for a uniform member subject to uniform moment, see **G.3.3.3**;
M_{px}	is the plastic moment capacity about the major axis;
M_{rx}	is the reduced plastic moment capacity about the major axis in the presence of axial force;
m_t	is the equivalent uniform moment factor for restrained buckling, see **G.4.2**.

For uniform members or segments with intermediate loads, the length L_y between points at which the non-restrained flange is laterally restrained should not exceed the limiting spacing L_s given by:

$$L_s = \frac{L_k}{n_t}$$

where

n_t	is the slenderness correction factor, see **G.4.3**.

G.3.3.2 *Haunched or tapered members*

For haunched or tapered members or segments the length L_y between points at which the non-restrained flange is laterally restrained should not exceed the limiting spacing L_s given by:

$$L_s = \frac{L_k}{cn_t}$$

where

c	is the taper factor, see **G.2.5**;
L_k	is the limiting length for a uniform member subject to uniform moment, see **G.3.3.3**.

G.3.3.3 *Limiting length L_k*

The limiting length L_k for a uniform member or segment subject to uniform moment, for an I-section member with $x \geq 20$ and depth $D \geq 1.2$ times its breadth B should be obtained from:

$$L_k = \frac{(5.4 + 600p_y/E)r_y x}{(5.4x^2 p_y/E - 1)^{0.5}}$$

where

r_y	is the minor axis radius of gyration;
x	is the torsional index, see **4.3.6.8**.

For haunched or tapered members or segments the values of r_y and x should be taken as equal to those for the minimum depth of cross-section within the length L_y between points at which the non-restrained flange is laterally restrained.

G.4 Non-uniform moments

G.4.1 *Methods*

In a member or segment with one flange laterally restrained and compression in the non-restrained flange, a non-uniform pattern of major axis moments over the length L_y between points at which the non-restrained flange is laterally restrained may be allowed for through either:

— an equivalent uniform moment factor m_t from **G.4.2**;

— a slenderness correction factor n_t from **G.4.3**.

Only one of these allowances should be included in the same check. Allowing for a non-uniform moment by using values of m_t or n_t less than 1.0 should be treated as two mutually exclusive alternatives.

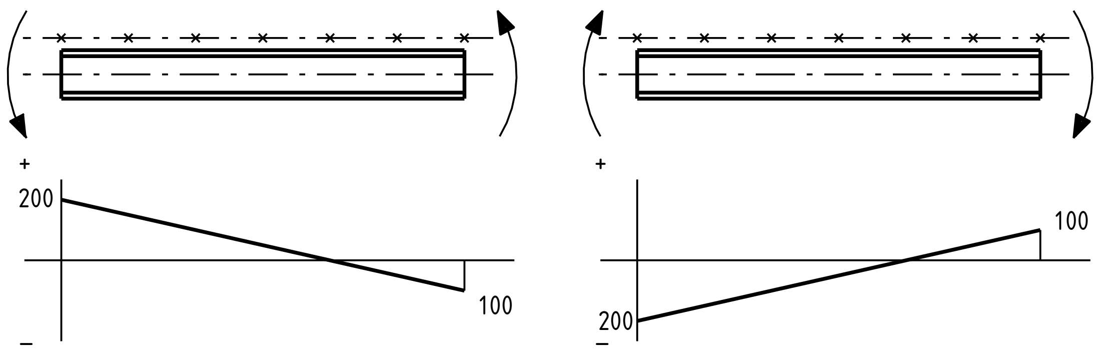
For a uniform member with one flange restrained, if a value of m_t less than 1.0 is used in **G.2.1**, then the value of n_t should be taken as 1.0 in **G.2.4.1**. Alternatively, a value of n_t less than 1.0 may be used in **G.2.4.1** provided that the value of m_t is taken as 1.0 in **G.2.1**.

G.4.2 *Equivalent uniform moment factor* m_t

An equivalent uniform moment factor m_t less than 1.0 may be used to allow for non-uniform moments in a uniform member with one flange restrained and compression in the non-restrained flange.

The values of m_t specified in Table G.1 may be used for a linear moment gradient, depending on the value of the end moment ratio β_t and the value of y, see **G.2.3**. The end moment ratio β_t should be taken as the ratio of the algebraically smaller end moment to the larger end moment. Moments that produce compression in the non-restrained flange should be taken as positive. If this ratio is less than –1.0 the value of β_t should be taken as –1.0, see Figure G.4.

NOTE This definition of β_t is different from the definition of β in Table 18 and Table 26.



$\beta_t = \frac{-100}{+200} = -0.5$

$\beta_t = \frac{-200}{+100}$ but $\beta_t \geq -1.0$

Thus $\beta_t = -1.0$

Figure G.4 — Value of β_t

Deviations of the pattern of major axis moments from a linear moment gradient over the member or segment length L_y should be allowed for by either:

— taking m_t as 1.0 and allowing for the non-uniform moments by treating the member or segment as a haunched or tapered segment with a value of n_t less than 1.0;

— substituting a conservative linear moment gradient, with a numerical value at all points not less than the applied moment and the same numerical value of maximum moment, see Figure G.5.

Table G.1 — Equivalent uniform moment factor m_t

End moment ratio β_t	Parameter y										
	0.0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**
–1.0	1.00	0.76	0.61	0.51	0.44	0.39	0.35	0.31	0.28	0.26	0.24
–0.9	1.00	0.78	0.63	0.52	0.45	0.40	0.36	0.32	0.30	0.28	0.26
–0.8	1.00	0.80	0.64	0.53	0.46	0.41	0.37	0.34	0.32	0.30	0.28
–0.7	1.00	0.81	0.66	0.55	0.47	0.42	0.39	0.36	0.34	0.32	0.30
–0.6	1.00	0.83	0.67	0.56	0.49	0.44	0.40	0.38	0.36	0.34	0.33
–0.5	1.00	0.85	0.69	0.58	0.50	0.46	0.42	0.40	0.38	0.37	0.36
–0.4	1.00	0.86	0.70	0.59	0.52	0.48	0.45	0.43	0.41	0.40	0.39
–0.3	1.00	0.88	0.72	0.61	0.54	0.50	0.47	0.45	0.44	0.43	0.42
–0.2	1.00	0.89	0.74	0.63	0.57	0.53	0.50	0.48	0.47	0.46	0.45
–0.1	1.00	0.90	0.76	0.65	0.59	0.55	0.53	0.51	0.50	0.49	0.49
0.0	1.00	0.92	0.78	0.68	0.62	0.58	0.56	0.55	0.54	0.53	0.52
0.1	1.00	0.93	0.80	0.70	0.65	0.62	0.59	0.58	0.57	0.57	0.56
0.2	1.00	0.94	0.82	0.73	0.68	0.65	0.63	0.62	0.61	0.61	0.60
0.3	1.00	0.95	0.84	0.76	0.71	0.69	0.67	0.66	0.65	0.65	0.65
0.4	1.00	0.96	0.86	0.79	0.75	0.72	0.71	0.70	0.70	0.69	0.69
0.5	1.00	0.97	0.88	0.82	0.78	0.76	0.75	0.75	0.74	0.74	0.74
0.6	1.00	0.98	0.91	0.85	0.82	0.81	0.80	0.79	0.79	0.79	0.79
0.7	1.00	0.98	0.93	0.89	0.87	0.85	0.85	0.84	0.84	0.84	0.84
0.8	1.00	0.99	0.95	0.92	0.91	0.90	0.90	0.89	0.89	0.89	0.89
0.9	1.00	1.00	0.98	0.96	0.95	0.95	0.95	0.95	0.95	0.94	0.94
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

NOTE The values of m_t specified in this table are calculated from:

$$m_t = B_0 + B_1\beta_t + B_2\beta_t^2$$

in which:

$$B_0 = \frac{1 + 10y^2}{1 + 20y^2}, \qquad B_1 = \frac{5y}{\pi + 10y}, \qquad B_2 = \frac{0.5}{1 + \pi y} - \frac{0.5}{1 + 20y^2}$$

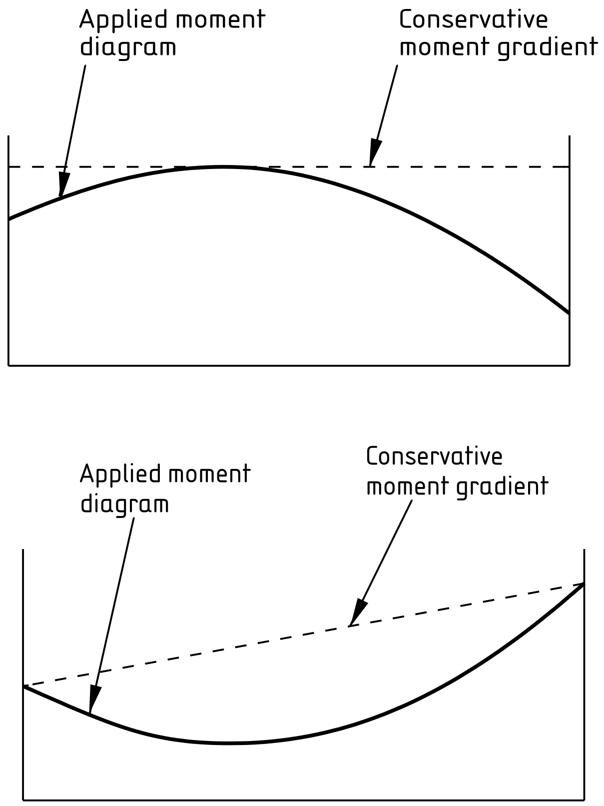


Figure G.5 — Conservative moment gradients

G.4.3 ***Slenderness correction factor n_t***

A slenderness correction factor n_t may be used to allow for non-uniform moments in a member or segment with one flange laterally restrained and compression in the non-restrained flange, provided that the equivalent uniform moment factor m_t is taken as 1.0.

The value of n_t should be determined from:

$$n_t = \left[\frac{1}{12}\{R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2(R_S - R_E)\}\right]^{0.5}$$

in which R_1 to R_5 are the values of R at the ends, quarter points and mid-length, see Figure G.6, and only positive values of R should be included.

In addition, only positive values of $(R_S - R_E)$ should be included, where:

— R_E is the greater of R_1 or R_5;
— R_S is the maximum value of R anywhere in the length L_y.

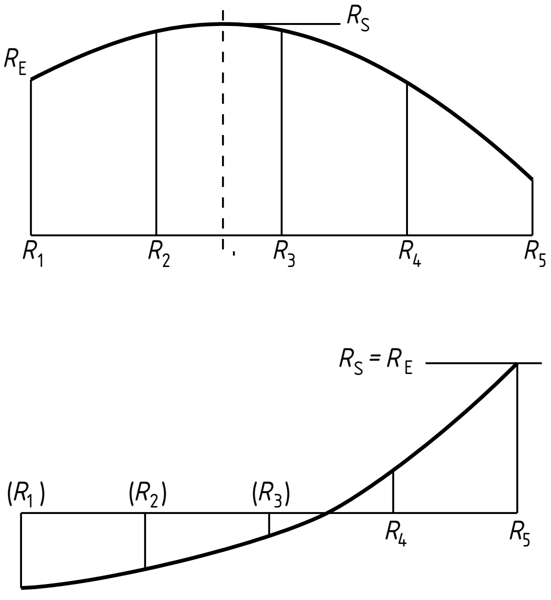


Figure G.6 — Moment ratios

When checking the lateral buckling resistance, see **G.2.4**, the value of R should be obtained from:

$$R = \frac{M_x}{p_y Z_{xc}}$$

where

M_x is the major axis moment, taken as positive when it produces compression in the non-restrained flange;

Z_{xc} is the section modulus about the major axis, for calculating the maximum compressive stress.

When considering the spacing of lateral restraints to the non-restrained flange adjacent to a plastic hinge location, see **G.3.3**, the value of R should be obtained from:

$$R = \frac{M_x + aF_c}{p_y S_x}$$

where

a is the distance between the reference axis and the axis of restraint, see Figure G.1;

F_c is the axial compression;

S_x is the major axis plastic modulus.

Annex H (normative) Web buckling resistance

H.1 Shear buckling strength

The shear buckling strength q_w of the web of an I-section may be determined as follows:

— for a welded I-section:

— if $\lambda_w \leq 0.8$: $q_w = p_v$

— if $0.8 < \lambda_w < 1.25$: $q_w = [(13.48 - 5.6\lambda_w)/9]p_v$

— if $\lambda_w \geq 1.25$: $q_w = 0.9p_v/\lambda_w$

— for a rolled I-section:

— if $\lambda_w \leq 0.9$: $q_w = p_v$

— if $\lambda_w > 0.9$: $q_w = 0.9p_v/\lambda_w$

in which:

$p_v = 0.6p_{yw}$

$\lambda_w = [p_v/q_e]^{0.5}$

— if $a/d \leq 1$: $q_e = \left[0.75 + \frac{1}{(a/d)^2}\right]\left[\frac{1\,000}{d/t}\right]^2$ [in N/mm²]

— if $a/d > 1$: $q_e = \left[1 + \frac{0.75}{(a/d)^2}\right]\left[\frac{1\,000}{d/t}\right]^2$ [in N/mm²]

where

a is the stiffener spacing, taken as infinity for webs without intermediate stiffeners;

d is the depth of the web;

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

t is the web thickness.

H.2 Critical shear buckling resistance

The critical shear buckling resistance V_{cr} of the web of an I-section may be determined using:

$V_{cr} = dtq_{cr}$

in which q_{cr} is the critical shear strength determined as follows:

— for a welded I-section:

— if $\lambda_w \leq 0.8$: $q_{cr} = p_v$

— if $0.8 < \lambda_w < 1.25$: $q_{cr} = (1.64 - 0.8\lambda_w)p_v$

— if $\lambda_w \geq 1.25$: $q_{cr} = p_v/\lambda_w^2$

— for a rolled I-section:

— if $\lambda_w \leq 0.9$: $q_{cr} = p_v$

— if $0.9 < \lambda_w < 1.25$: $q_{cr} = [(8.1/\lambda_w - 2)/7]p_v$

— if $\lambda_w \geq 1.25$: $q_{cr} = p_v/\lambda_w^2$

in which p_v and λ_w are as defined in **H.1**.

H.3 Resistance of a web to combined effects

H.3.1 *General*

A web required to resist moment or axial force combined with shear should be checked using the reduction factor ρ specified in **H.3.2** if the simple shear buckling resistance V_w (see **4.4.5.2**) is less than the shear capacity P_v (see **4.2.3**).

In the case of members with unequal flanges, or subject to applied axial force, the longitudinal effects to be resisted by the web should be arranged in the form of:

— an axial force F_{cw} for compression, or F_{tw} for tension, acting at the mid-depth of the web;

— a bending moment M_w about the mid-depth of the web.

For an RHS or welded box section, account should be taken of the additional internal moments in the member due to "strut-action" (see **C.3**) and moment amplification (see **I.5.1**). In the case of an RHS or welded box section subject to moments about both axes, the maximum axial force in each face should be determined taking account of the moment about the axis parallel to that face.

In all cases, the capacity of the cross-section as a whole should also be checked, see **4.2**, **4.7** and **4.8**.

H.3.2 *Reduction factor for shear buckling*

The reduction factor ρ for shear buckling should be determined as follows:

— if $F_v \le 0.6V_w$: $\rho = 0$

— if $F_v > 0.6V_w$: $\rho = [2(F_v/V_w) - 1]^2$

where

F_v is the shear force.

NOTE The reduction factor ρ starts when F_v exceeds $0.5V_w$ but the resulting reduction in moment capacity is negligible unless F_v exceeds $0.6V_w$.

H.3.3 *Sections other than RHS*

H.3.3.1 *Combined shear, moment and axial compression*

A web subject to combined shear, moment and axial compression should satisfy the following:

— *Class 1 plastic, class 2 compact or class 3 semi-compact web*:

$$\frac{M_w(1-\rho)}{M_{pw}} + \left(\frac{F_{cw}}{P_w}\right)^2 \le (1-\rho)^2$$

$$\left(\frac{M_w}{M_{cw}}\right)^2 + \frac{F_{cw}}{P_{cw}} \le (1-\rho)^2$$

in which:

$$M_{cw} = \left(\frac{K}{d/t}\right)^2 t d^2 p_y$$

$$M_{pw} = 0.25 t d^2 p_y$$

$$P_{cw} = \left(\frac{K}{d/t}\right)^2 t d p_y$$

$$P_w = t d p_y$$

and the coefficient K should be determined from the following:

— for a class 1 plastic web: $K = 40\varepsilon$

— for a class 2 compact web: $K = \frac{100\varepsilon}{3}\left(1 + \frac{d/t}{200\varepsilon}\right)$

— for a class 3 semi-compact web: $K = 50\varepsilon$

— *Class 4 slender web:*

$$\frac{M_w(1-\rho)}{M_{cw}} + \frac{F_{cw}}{P_{cw}} \leq (1-\rho)^2$$

in which:

$$M_{cw} = \left(\frac{120\varepsilon}{d/t}\right)\frac{td^2}{6}p_y$$

$$P_{cw} = \left(\frac{40\varepsilon}{d/t}\right)tdp_y$$

H.3.3.2 *Combined shear, moment and axial tension*

A web subject to combined shear, moment and axial tension should satisfy the following:

— *Class 1 plastic or class 2 compact web*:

$$\frac{M_w(1-\rho)}{M_{pw}} + \left(\frac{F_{tw}}{P_w}\right)^2 \leq (1-\rho)^2$$

$$\left(\frac{M_w}{M_{tw}}\right)^2 - \frac{F_{tw}}{P_{tw}} \leq (1-\rho)^2$$

in which:

$$M_{tw} = \left(\frac{K}{d/t}\right)^2 td^2 p_y$$

$$P_{tw} = \left(\frac{K}{d/t}\right)^2 tdp_y$$

and the coefficient K is determined from the following:

— for a class 1 plastic web: $K = 40\varepsilon$

— for a class 2 compact web: $K = 50\varepsilon$

— *Class 3 semi-compact or class 4 slender web*:

$$\frac{M_w}{M_{tw}} - \frac{F_{tw}}{P_{tw}} \leq (1-\rho)$$

in which:

$$M_{tw} = \left(\frac{120\varepsilon}{d/t}\right)\frac{td^2}{6}p_y$$

$$P_{tw} = \left(\frac{120\varepsilon}{d/t}\right)tdp_y$$

H.3.3.3 *Combined shear, moment and edge loading*

If a load is applied to one edge of the web and resisted by shear in the member, the provisions of **H.3.3.1** should be applied using the maximum shear and the maximum moment occurring within the same web panel between transverse stiffeners, but not further than $2d$ apart, where d is the web depth.

If a compressive load F_x is applied to one edge of the web and resisted at the opposite edge, but there is no axial force in the web (due to applied axial force or due to unequal flanges), the web should satisfy the following:

— *for a class 1 plastic, class 2 compact or class 3 semi-compact web*:

$$\left(\frac{M_w}{M_{cw}}\right)^2 + \frac{F_x}{P_x} \le (1-\rho)^2$$

in which:

$$M_{cw} = \left(\frac{K}{d/t}\right)^2 td^2 p_y$$

where K is as defined in **H.3.3.1**.

— *for a class 4 slender web*:

$$\frac{M_w(1-\rho)}{M_{cw}} + \frac{F_x}{P_x} \le (1-\rho)^2$$

in which:

$$M_{cw} = \left(\frac{120\varepsilon}{d/t}\right)\frac{td^2}{6}p_y$$

where

P_x is the buckling resistance as defined in **4.5.3.1**.

If a load is applied to one edge of the web and resisted at the opposite edge, and the web is also subject to axial force (due to applied axial force or due to unequal flanges), reference should be made to BS 5400-3.

H.3.4 ***RHS sections***

H.3.4.1 *Combined shear, moment and axial compression*

An RHS web subject to combined shear, moment and axial compression should satisfy the following:

— *Class 1 plastic or class 2 compact web*:

$$\frac{M_w(1-\rho)}{M_{pw}} + \left(\frac{F_{cw}}{P_w}\right)^2 \le (1-\rho)^2$$

$$\left(\frac{M_w}{M_{cw}}\right)^2 + \frac{F_{cw}}{P_{cw}} \le (1-\rho)^2$$

in which:

$$M_{cw} = \left(\frac{K}{d/t}\right)^2 td^2 p_y$$

$$P_{cw} = \left(\frac{K}{d/t}\right)^2 tdp_y$$

and M_{pw} and P_w are as defined in **H.3.3.1** but the coefficient K should be determined from the following:

— for a hot finished RHS: $K = 40\varepsilon$
— for a cold formed RHS: $K = 35\varepsilon$

— *Class 3 semi-compact or class 4 slender web:*

$$\frac{M_w(1-\rho)}{M_{cw}} + \frac{F_{cw}}{P_{cw}} \le (1-\rho)^2$$

in which:

$$M_{cw} = \left(\frac{K_b}{d/t}\right)\frac{td^2}{6}p_y$$

$$P_{cw} = \left(\frac{K_c}{d/t}\right)tdp_y$$

and the coefficients K_b and K_c are determined from the following:

— for a hot finished RHS: $K_b = 120\varepsilon$ and $K_c = 40\varepsilon$
— for a cold formed RHS: $K_b = 105\varepsilon$ and $K_c = 35\varepsilon$

H.3.4.2 *Combined shear, moment and axial tension*

An RHS web subject to combined shear, moment and axial tension should satisfy the following:

— *Class 1 plastic or class 2 compact web*:

$$\frac{M_w(1-\rho)}{M_{pw}} + \left(\frac{F_{tw}}{P_w}\right)^2 \le (1-\rho)^2$$

$$\left(\frac{M_w}{M_{tw}}\right)^2 - \frac{F_{tw}}{P_{tw}} \le (1-\rho)^2$$

in which:

$$M_{tw} = \left(\frac{K}{d/t}\right)^2 td^2 p_y$$

$$P_{tw} = \left(\frac{K}{d/t}\right)^2 tdp_y$$

and the coefficient K is determined from the following:

— for a hot finished RHS: $K = 40\varepsilon$
— for a cold formed RHS: $K = 35\varepsilon$

and the other symbols are as in **H.3.3.1**.

— *Class 3 semi-compact or class 4 slender web*:

$$\frac{M_w}{M_{tw}} - \frac{F_{tw}}{P_{tw}} \le (1-\rho)$$

in which:

$$M_{tw} = \left(\frac{K}{d/t}\right)\frac{td^2}{6}p_y$$

$$P_{tw} = \left(\frac{K}{d/t}\right)tdp_y$$

and the coefficient K is determined from the following:

— for a hot finished RHS: $K = 120\varepsilon$
— for a cold formed RHS: $K = 105\varepsilon$

H.4 End anchorage

H.4.1 *General*

Except as provided in **4.4.5.4**, end anchorage should be provided for a longitudinal anchor force H_q representing the longitudinal component of the tension field, see Figure H.1, at:

— the ends of webs without intermediate stiffeners;
— the end panels of webs with intermediate transverse stiffeners.

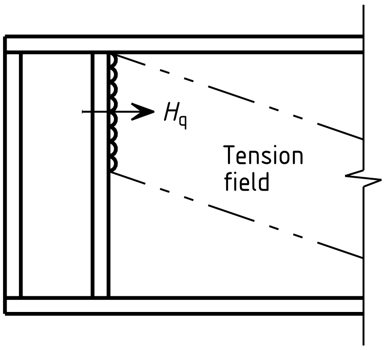


Figure H.1 — Anchor force H_q

The longitudinal anchor force H_q should be obtained from the following:

— if the web is fully loaded in shear ($F_v \geq V_w$):

$$H_q = 0.5dtp_y[1 - V_{cr}/P_v]^{0.5}$$

— if the web is not fully loaded in shear ($F_v < V_w$) then optionally:

$$H_q = 0.5dtp_y\left(\frac{F_v - V_{cr}}{V_w - V_{cr}}\right)[1 - V_{cr}/P_v]^{0.5}$$

where

d is the depth of the web;
F_v is the maximum shear force;
P_v is the shear capacity from **4.2.3**;
t is the web thickness;

V_{cr} is the critical shear buckling resistance from **4.4.5.4** or **H.2**;

V_w is the simple shear buckling resistance from **4.4.5.2**.

End anchorage should be provided by one of the following:

— a single stiffener end post, see **H.4.2**;

— a twin stiffener end post, see **H.4.3**;

— an anchor panel, see **H.4.4**.

H.4.2 ***Single stiffener end post***

A single stiffener end post, see Figure H.2, should be designed to resist the girder reaction plus the in-plane bending moment M_{tf} due to the anchor force H_q. Generally this moment should be taken as $0.15H_qd$, but if the top of the end post is connected to the girder flange by means of full strength welds and both the width and the thickness of the girder flange are not less than those of the end post, then a moment of $0.10H_qd$ may be adopted.

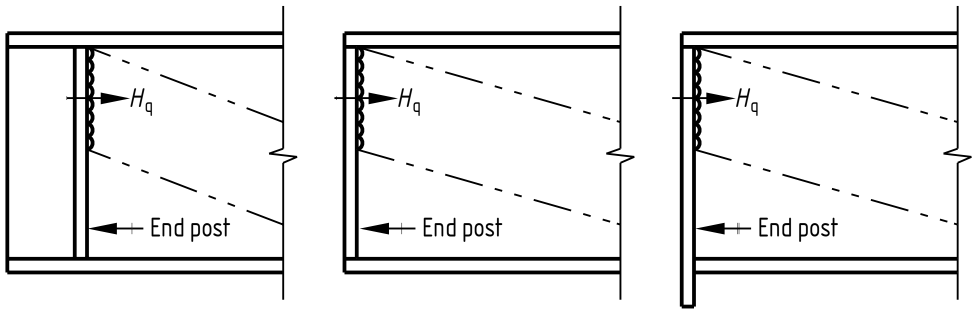


Figure H.2 — Single stiffener end posts

H.4.3 ***Twin stiffener end post***

In a twin stiffener end post, see Figure H.3, the web should satisfy:

$$R_{tf} \le V_{cr.ep}$$

in which the shear force R_{tf} in the end post, due to the anchor force H_q in the end panel is given by:

$$R_{tf} = 0.75H_q$$

where

$V_{cr.ep}$ is the critical shear buckling resistance (see **4.4.5.4**) of the web of the end post, treated as a beam spanning between the flanges of the girder.

The end stiffener should be designed to resist the relevant compressive force F_e due to the support reaction of the girder, plus a compressive force F_{tf} due to the anchor force given by:

$$F_{tf} = 0.15H_q(d/a_e)$$

where

a_e is the spacing centre-to-centre of the two end stiffeners.

The other stiffener forming part of the end post should be designed to resist the relevant compressive force F_s due to the support reaction of the girder, neglecting the tensile force F_{tf} due to the anchor force. However, if $F_{tf} > F_s$ it should also be checked for a tensile force equal to $(F_{tf} - F_s)$.

NOTE Depending on the locations of the stiffeners and the support, either F_e or F_s can be zero.

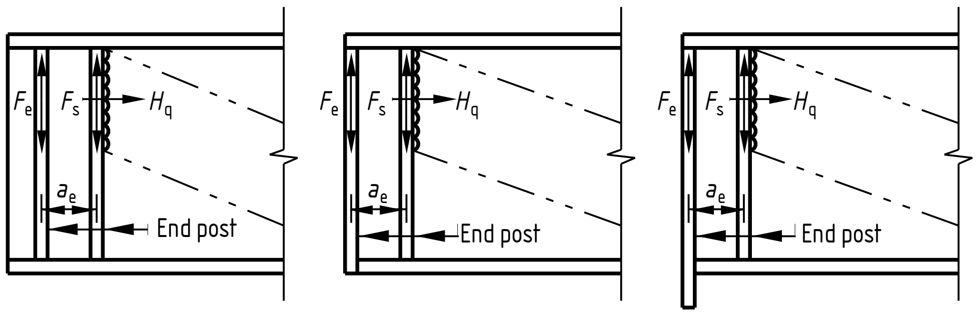


Figure H.3 — Twin stiffener end posts

H.4.4 ***Anchor panel***

An anchor panel, see Figure H.4, should satisfy:

$$F_v \le V_{cr}$$

In addition, the web of an anchor panel should satisfy:

$$R_{tf} \le V_{cr.ep}$$

in which the shear force R_{tf} in the anchor panel, due to the anchor force H_q in the next panel is given by:

$$R_{tf} = 0.75H_q$$

where

$V_{cr.ep}$ is the critical shear buckling resistance V_{cr} (see **4.4.5.4** or **H.2**) of the web of the anchor panel, treated as a beam spanning between the flanges of the girder.

The end stiffener should be designed to resist the relevant compressive force F_s due to the support reaction of the girder, plus a compressive force F_{tf} due to the anchor force given by:

$$F_{tf} = 0.15H_q d/a_e$$

where

a_e is the spacing centre-to-centre of the two stiffeners bounding the anchor panel.

The other stiffener bounding the anchor panel should be designed to resist the relevant compressive force F_s due to the support reaction of the girder, neglecting the tensile force F_{tf} due to the anchor force. However, if $F_{tf} > F_s$ it should also be checked for a tensile force equal to $(F_{tf} - F_s)$.

NOTE Depending on the locations of the stiffeners and the support, either F_e or F_s can be zero.

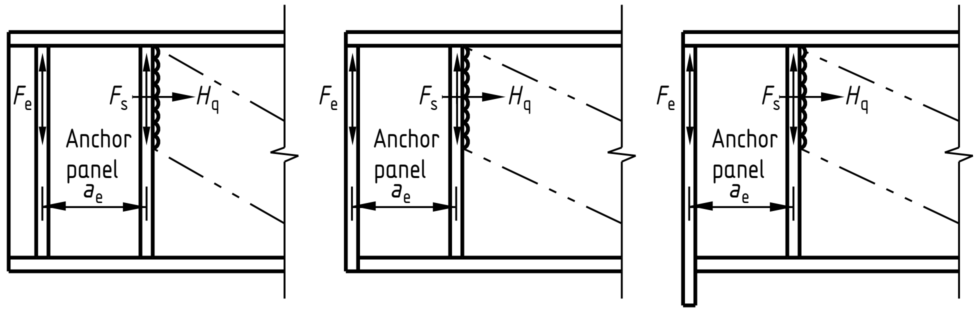


Figure H.4 — Anchor panels

Annex I (normative) Combined axial compression and bending

I.1 Stocky members

As a further alternative to the methods given in **4.8.3.3** the following approach may be used for stocky members of doubly-symmetric class 1 plastic or class 2 compact cross-section.

a) for members with moments about the major axis only:

— for major axis in-plane buckling:

$$m_x M_x \leq M_{ax}$$

— for out-of-plane buckling:

$$m_{LT} M_{LT} \leq M_{ab}$$

b) for members with moments about the minor axis only:

— for minor axis in-plane buckling:

$$m_y M_y \leq M_{ay}$$

— for out-of-plane buckling:

$$m_{yx} M_y \leq 2M_{cy}(1 - F_c / P_{cx})$$

c) for members with moments about both axes:

— for major axis buckling:

$$\frac{m_x M_x}{M_{ax}} + \frac{0.5 m_{yx} M_y}{M_{cy}(1 - F_c/P_{cx})} \leq 1$$

— for lateral-torsional buckling:

$$\frac{m_{LT} M_{LT}}{M_{ab}} + \frac{m_y M_y}{M_{ay}} \leq 1$$

— for interactive buckling:

$$\frac{m_x M_x}{M_{ax}} + \frac{m_y M_y}{M_{ay}} \leq 1$$

M_{ab} is given by the following:

— if $\lambda_r \leq \lambda_{r0}$:

$$M_{ab} = M_{rx} \quad \text{but} \quad M_{ab} \leq M_{xy}$$

— if $\lambda_{r0} < \lambda_r < 85.8\varepsilon$:

$$M_{ab} = M_{ob} + \frac{(85.8\varepsilon - \lambda_r)}{(85.8\varepsilon - \lambda_{r0})}[M_{rx} - M_{ob}] \quad \text{but} \quad M_{ab} \leq M_{xy}$$

— if $\lambda_r \geq 85.8\varepsilon$:

$$M_{ab} = M_{ob}$$

in which:

$$M_{ob} = M_b(1 - F_c/P_{cy})$$

$$\lambda_r = \frac{(r_b\lambda_{LT} + r_c\lambda_y)}{(r_b + r_c)}$$

$$\lambda_{r0} = \frac{17.15\varepsilon(2r_b + r_c)}{(r_b + r_c)}$$

$$r_b = m_{LT}M_{LT}/M_b$$

$$r_c = F_c/P_{cy}$$

M_{ax} is given by the following:

— if $\lambda_x \le 17.15\varepsilon$:

$$M_{ax} = M_{rx}$$

— if $17.15\varepsilon < \lambda_x < 85.8\varepsilon$:

$$M_{ax} = M_{ox} + \frac{(85.8\varepsilon - \lambda_x)}{68.65\varepsilon}[M_{rx} - M_{ox}]$$

— if $\lambda_x \ge 85.8\varepsilon$:

$$M_{ax} = M_{ox}$$

in which:

$$M_{ox} = \frac{M_{cx}(1 - F_c/P_{cx})}{(1 + 0.5F_c/P_{cx})}$$

M_{ay} is given by the following:

— if $\lambda_y \le 17.15\varepsilon$:

$$M_{ay} = M_{ry}$$

— if $17.15\varepsilon < \lambda_y < 85.8\varepsilon$:

$$M_{ay} = M_{oy} + \frac{(85.8\varepsilon - \lambda_y)}{68.65\varepsilon}[M_{ry} - M_{oy}]$$

— if $\lambda_y \ge 85.8\varepsilon$:

$$M_{ay} = M_{oy}$$

in which:

$$M_{oy} = \frac{M_{cy}(1 - F_c/P_{cy})}{(1 + k_y(F_c/P_{cy}))}$$

and the coefficient k_y should be taken as 1.0 for I- or H-sections, but 0.5 for CHS, RHS or box sections.

M_{xy} is given by the following:

— for I- or H-sections:

$$M_{xy} = M_{cx}[1 - F_c/P_{cy}]^{0.5}$$

— for CHS, RHS or box sections:

$$M_{xy} = 2M_{cx}(1 - F_c/P_{cy})$$

where

- M_{rx} is the major axis reduced plastic moment capacity in the presence of axial force, see **I.2**;
- M_{ry} is the minor axis reduced plastic moment capacity in the presence of axial force, see **I.2**;
- λ_x is the major axis slenderness for buckling as a compression member, see **4.7.3**;
- λ_y is the minor axis slenderness for buckling as a compression member, see **4.7.3**;

and the other symbols are as defined in **4.8.3.3**.

I.2 Reduced plastic moment capacity

I.2.1 *I- or H-section with equal flanges*

The reduced plastic moment capacities M_{rx} and M_{ry} of a class 1 plastic or class 2 compact I- or H-section with equal flanges in the presence of an axial force should be obtained using:

$$M_{rx} = p_y S_{rx}$$
$$M_{ry} = p_y S_{ry}$$

in which S_{rx} and S_{ry} are the values of the reduced plastic modulus about the major and minor axes.

The values of S_{rx} and S_{ry} should be based on the value of the axial force ratio n given by:

$$n = \frac{F}{Ap_y}$$

For an I- or H-section with equal parallel flanges, the reduced plastic modulus S_{rx} about the major axis should be obtained from the following:

— if $n \le t(D - 2T)/A$: $S_{rx} = S_x - \left(\frac{A^2}{4t}\right)n^2$

— if $n > t(D - 2T)/A$: $S_{rx} = \left(\frac{A^2}{4B}\right)\left[\left(\frac{2BD}{A} - 1\right) + n\right](1 - n)$

and the reduced plastic modulus S_{ry} about the minor axis should be obtained from the following:

— if $n \le tD/A$: $S_{ry} = S_y - \left(\frac{A^2}{4D}\right)n^2$

— if $n > tD/A$: $S_{ry} = \left(\frac{A^2}{8T}\right)\left[\left(\frac{4BT}{A} - 1\right) + n\right](1 - n)$

where

- A is the cross-section area;
- B is the flange width;
- D is the overall depth;
- F is the axial force (tension or compression);
- 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.

NOTE Tabulated coefficients can be found in published tables.

I.2.2 ***Other cases***

In other cases the reduced plastic modulus may be determined on the basis of the principles of statics.

NOTE Formulae and tabulated coefficients can be found in published tables.

I.3 Asymmetric members

In evaluating any of the linear interaction relationships given in **4.8.2.2**, **4.8.3.2**a) and **4.8.3.3.1**, if the cross-section is not symmetrical about the relevant axis, when the section modulus is used for the moment capacity and resistance moment terms, account may be taken of the sense of the moments.

To do this, the expressions should first be re-arranged to form a summation of stresses, as follows:

— re-arranged from **4.8.2.2**:

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \le p_y$$

— re-arranged from **4.8.3.2**a):

$$\frac{F_c}{A_g} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \le p_y$$

— re-arranged from **4.8.3.3.1**:

$$\frac{F_c}{Ap_c/p_y} + \frac{m_x M_x}{Z_x} + \frac{m_y M_y}{Z_y} \le p_y$$

$$\frac{F_c}{Ap_{cy}/p_y} + \frac{m_{LT} M_{LT}}{Z_x(p_b/p_y)} + \frac{m_y M_y}{Z_y} \le p_y$$

Then, as an alternative to using the lower value of the section modulus in each case, the resulting stresses at the critical points on the cross-section, according to the relevant expression, may be determined for each moment using the appropriate section modulus and added algebraically to the stress resulting from the axial force to determine the peak stress.

I.4 Single angle members

I.4.1 ***General***

The design of single angle members to resist combined axial compression and bending should take account of the fact that the rectangular axes of the cross-section (x-x and y-y) are not the principal axes, either by using the basic method given in **I.4.2** or the simplified method given in **I.4.3**.

I.4.2 ***Basic method***

For this method the applied moments should be resolved into moments about the principal axes u-u and v-v. The buckling resistance moment M_b for bending about the u-u axis should be based on the value of λ_{LT} obtained from **B.2.9**. The method given in **4.8.3.3.1** should then be used, by applying those terms that refer to the major axis to the u-u axis and those that refer to the minor axis to the v-v axis. The method for asymmetric sections given in **I.3** may be used in evaluating the relevant interaction expression.

I.4.3 ***Simplified method***

Alternatively to **I.4.2**, for equal angles the applied moments may be resolved into moments about the x-x and y-y axes. The following modification of the relationship specified in **4.8.3.3.1** should then be satisfied:

$$\frac{F_c}{P_c} + \frac{m_{LTx} M_x}{M_{bx}} + \frac{m_{LTy} M_y}{M_{by}} \le 1$$

in which:

$m_{LTx} \ge 0.6$ and $m_{LTy} \ge 0.6$

where

F_c	is the axial compression;
L_{Ex}	is the length between points restrained against buckling about the x-x axis;
L_{Ey}	is the length between points restrained against buckling about the y-y axis;
M_{bx}	is the buckling resistance moment M_b from **4.3.8.3** using L_{Ey} and Z_x;
M_{by}	is the buckling resistance moment M_b from **4.3.8.3** using L_{Ex} and Z_y;
M_x	is the maximum moment about the x-x axis;
M_y	is the maximum moment about the y-y axis;
m_{LTx}	is the equivalent uniform moment factor m_{LT} obtained from Table 18, based on the pattern of moments about the x-x axis over the length L_{Ey};
m_{LTy}	is the equivalent uniform moment factor m_{LT} obtained from Table 18, based on the pattern of moments about the y-y axis over the length L_{Ex};
P_c	is the compression resistance from **4.7.4** considering buckling about any axis, including v-v;
Z_x	is the section modulus for bending about the x-x axis;
Z_y	is the section modulus for bending about the y-y axis.

I.5 Internal moments

I.5.1 *General*

The internal "second-order" moments in a member subject to combined axial compression and bending should be taken as including those of the following that are relevant:

a) a "strut action" moment produced by resisting flexural buckling due to the axial force, see **C.3**;

b) an additional minor-axis moment produced by resisting lateral-torsional buckling due to major axis moments, see **B.3**;

c) an additional major axis moment due to amplification of the applied major axis moments;

d) an additional minor axis moment due to amplification of the applied minor axis moments.

Item a) should be considered about each axis, but only about one axis at a time.

Items b) and c) should be treated as alternatives, depending on which has the more severe effect.

The additional moments due to amplification of the applied major and minor axis moments should each be taken as having a maximum value midway between points of inflexion of the buckled shape (the points between which the effective length for buckling about the relevant axis is measured) given by:

$$M_{add,x,max} = \frac{m_x M_x}{(p_{Ex}/f_c - 1)} \quad \text{in which} \quad p_{Ex} = \frac{\pi^2 E}{\lambda_x{}^2}$$

$$M_{add,y,max} = \frac{m_y M_y}{(p_{Ey}/f_c - 1)} \quad \text{in which} \quad p_{Ey} = \frac{\pi^2 E}{\lambda_y{}^2}$$

where

f_c	is the compressive stress due to axial force;
M_x	is the maximum moment about the major axis;
M_y	is the maximum moment about the minor axis;
m_x	is the equivalent uniform moment factor for buckling about the major axis from **4.8.3.3.4**;
m_y	is the equivalent uniform moment factor for buckling about the minor axis from **4.8.3.3.4**.

The additional internal moments $M_{add,xs}$ and $M_{add,ys}$ at a distance L_z along the member from a point of inflexion should be obtained from:

$$M_{add,xs} = M_{add,x,max} \sin(180(L_z/L_{Ex}))$$

$$M_{add,ys} = M_{add,y,max} \sin(180(L_z/L_{Ey}))$$

where

L_{Ex} is the effective length for flexural buckling about the major axis;

L_{Ey} is the effective length for flexural buckling about the minor axis.

I.5.2 *T-sections*

In applying **I.5.1** to a T-section, the subscripts x and y should always be taken as referring to the major axis and the minor axis respectively, even where the opposite subscript is used in **B.2.8.2**b).

I.5.3 *Angles*

In applying **I.5.1** to an angle, the subscripts x and y should be taken as referring to the major axis u-u and minor axis v-v respectively.

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