Structural use of steelwork in building —

Part 5. Code of practice for design of cold formed thin gauge sections

ICS 91.080.10

Committees responsible for this British Standard

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British Constructional Steelwork Association Ltd.
British Industrial Fasteners' Federation
British Iron and Steel Producers' Association
Cold Rolled Sections' Association
Department of the Environment (Building Research Establishment)
Department of the Environment (Property and Buildings Directorate)
Department of the Environment (Specialist Services)
Department of Transport (Highways Agency)
Health and Safety Executive
Institution of Civil Engineers
Institution of Structural Engineers
Royal Institute of British Architects
Steel Construction Institute
The Welding Institute

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Section 4. Local buckling

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Foreword

This new edition of this part of BS 5950 has been prepared under the direction of Technical Committee B/525, Building and Civil Engineering Structures. It replaces BS 5950: Part 5:1987 which is withdrawn. BS 5950 is a document combining codes of practice to cover the design, construction and fire protection of steel structures and specifications for materials, workmanship and erection.

This part of BS 5950 gives recommendations for the design of cold formed steel sections in simple and continuous construction and its provisions apply to the majority of structures, although it is recognized that cases will arise when other proven methods of design may be more appropriate. It is intended to be compatible with BS 5950-1 and BS 5950-6, and at the same time to be as self contained as possible.

BS 5950 comprises the following parts:

Part 1, *Code of practice for design in simple and continuous construction: hot rolled sections.*

Part 2, *Specification for materials, fabrication and erection: hot rolled 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 and workmanship: cold formed sections and sheeting.*

Part 8, *Code of practice for fire protection of structural steelwork.*

Part 9, *Code of practice for stressed skin design.*

This edition introduces technical changes but it does not reflect a full review or revision of the standard.

The changes include:

a realignment of this standard with BS 5950-1 and clarification of the design recommendations in section **2** for the structural integrity of cold formed steel framing;

a revision to the recommendations in section **3** taking account of recently published European Standards for basic steel products and publication of a corrected version of Figure 1;

presentation of the modification factors for use with Tables 5 and 6 in a format consistent with the other parts of BS 5950;

new non dimensional expressions for local buckling stress, lateral buckling resistance and critical bending moment in sections **4**, **5** and **6;**

clarification of the recommendations for limiting stress in elements under stress gradient in section **5**;

introduction of design recommendations for back-to-back channels forming compound I sections in sections **5**, **6** and **8**;

the addition of validity limits to the recommendations in section **7** for determining the tensile capacity of simple tension members;

modification of section **8** to clarify certain general limiting parameters and taking account of European Standards for welding electrodes;

replacement of the term “plug welds” by the term “arc spot welds” and redrafting of the recommendations for their design using ultimate strength values rather than yield strength values;

redrafting of section **10** to clarify the evaluation of test results;

deletion of annex E and guidance on standard deviation inserted into section **10**;

modification of annexes A to D clarifying use of symbols and clarification of the method of calculating the factors k, α and C_w.

This part of BS 5950 is primarily equation-orientated, so that the rules can be easily programmed on desk-top computers which are now familiar in design offices. However, to assist the designer obtain simple and rapid analyses, it is possible to use the various tables and graphs provided instead of calculation by means of the equations in many circumstances.

This part of BS 5950 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 are carried out by capable and experienced organizations.

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 viii, pages 1 to 62, an inside back cover and a back cover.

Section 1. General

1.1 Introduction

1.1.1 Aims of economical structural design

The aim of structural design is to provide, with due regard to economy, a structure capable of fulfilling its intended function and sustaining the design loads for its intended life. The design should facilitate fabrication, erection and future maintenance.

The structure should behave as a single three-dimensional entity. The layout of its constituent parts, such as foundations, steelwork, connections 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 this it is necessary to define clearly the basic structural anatomy by which the loads are transmitted to the foundations. Any features of the structure which have a critical influence on its overall stability can then be identified and taken account of in its design.

Each part of the structure should be sufficiently robust and insensitive to the effects of minor incidental loads applied during service to ensure that the safety of other parts is not prejudiced. (See **2.3.5**)

Whilst the ultimate strength recommendations within 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 material and fabrication.

1.1.2 Overall stability

The designer 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.

1.1.3 Accuracy of calculation

For the purpose of checking conformity with the recommendations included in this standard, the final value, (whether observed or calculated), which expresses 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 the value given in this standard.

1.2 Scope

This part of BS 5950 gives recommendations for the design of structural steelwork in buildings and allied structures using cold formed sections. It is primarily intended for steel sections of thickness up to 8 mm. Requirements for materials and construction are given in BS 5950-7.

Sections may be either open or closed and should be made up of flat elements bounded either by free edges or by bends with included angles not exceeding 135° and internal radii not exceeding $5t$ where t is the material thickness.

Closed sections may be made either:

i) by joining together two previously formed open sections by continuous welding;

ii) from a single flat strip, by forming the corners to make a box, and continuously welding the longitudinal joint.

Welded cold formed hollow sections conforming to BS EN 10219 are not covered by this part of BS 5950.

NOTE Cold formed products conforming to BS EN 10219 are the subject of amendments to BS 5950-1 and -2 which are in preparation.

1.3 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this part 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 1140, *Specification for resistance spot welding of uncoated and coated low carbon steel.*

BS 1449-1-1, *Steel plate, sheet and strip — Carbon and carbon-manganese plate sheet and strip.*

BS 1449-1-1.5, *Steel plate, sheet and strip — Specification for cold rolled wide material based on specified minimum strength.*

BS 1449-1-1.8, *Steel plate, sheet and strip — Specification for hot rolled narrow strip based on formability.*

BS 1449-1-1.11, *Steel plate, sheet and strip — Specification for cold rolled narrow strip based on specified minimum strength.*

BS 5135, *Specification for arc welding of carbon and carbon manganese steels.*

BS 5493, *Code of practice for protective coating of iron and steel structures against corrosion*[1].

BS 5502-22, *Buildings and structures for agriculture — Code of practice for design, construction and loading.*

BS 5950-1, *Structural use of steelwork in building — Code of practice for design in simple and continuous construction: hot rolled sections.*

BS 5950-7, *Structural use of steelwork in building — Specification for materials and workmanship: cold formed sections and sheeting.*

BS 6399-1, *Loading for buildings — Code of practice for dead and imposed loads.*

BS 6399-2, *Loading for buildings — Code of practice for wind loads.*

BS 6399-3, *Loading for buildings — Code of practice for imposed roof loads.*

BS 8004, *Code of practice for foundations.*

[1] Will be replaced by BS ISO 12944-1 to -8 and BS EN 14713 which are in preparation.

PD 6484, *Commentary on corrosion at bimetallic contacts and its alleviation.*

BS EN 876, *Destructive tests on welds in metallic materials. Longitudinal tensile test on weld metal in fusion welded joints.*

BS EN 10002-1, *Tensile testing of metallic materials — Method of test at ambient temperature.*

BS EN 10021, *General technical delivery requirements for steel and iron products.*

BS EN 10025, *Hot rolled products of non-alloy structural steels. Technical delivery conditions.*

BS EN 10111, *Continuously hot-rolled low carbon steel sheet and strip for cold forming. Technical delivery conditions.*

BS EN 10147, *Specification for continuously hot-dip zinc coated structural steel sheet and strip — Technical delivery conditions.*

BS EN 10149-2, *Specification for hot rolled flat products made of high yield strength steels for cold forming — Delivery conditions for thermomechanically rolled steels.*

BS EN 10149-3, *Specification for hot rolled flat products made of high yield strength steels for cold forming — Delivery conditions for normalized and normalized rolled steels.*

BS EN 10204, *Metallic products — Types of inspection documents.*

BS EN 20898-1, *Mechanical properties of fasteners — Bolts, screws and studs.*

CP3 *Code of basic data for the design of buildings: Chapter V: Part 2: Wind loads.*

1.4 Terms and definitions

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

1.4.1

capacity

limit of force or moment that can be expected to be carried at a cross-section without causing failure due to yielding, rupture or local buckling

1.4.2

effective length

length between points of effective restraint of a member multiplied by a factor to take account of end conditions and loads

1.4.3

effective width

flat width of an element that can be considered effectively to resist compression

1.4.4

element

distinct portion of the cross-section of a member

NOTE Types of elements are defined in **1.4.5** to **1.4.8**.

1.4.5

stiffened element

a flat element adequately supported at both longitudinal edges

1.4.6

unstiffened element

a flat element adequately supported at only one longitudinal edge

1.4.7

edge stiffened element

a flat element supported at one longitudinal edge by a web and at the other longitudinal edge by a lip or other edge stiffener

1.4.8

multiple stiffened element

an element adequately supported at both longitudinal edges and having intermediate stiffeners

1.4.9

lateral buckling

buckling of a beam accompanied by a combination of lateral displacement and twisting

NOTE This is also known as lateral-torsional buckling.

1.4.10

buckling resistance

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

1.4.11

local buckling

buckling of the elements of a section characterized by the formation of waves or ripples along the member

NOTE It is treated separately from overall buckling resistance and modifies the capacity of cross-sections.

1.4.12

flexural buckling

buckling of a column due to flexure

1.4.13

torsional buckling

buckling of a column by twisting

1.4.14

torsional flexural buckling

buckling of a column by combined flexure and twisting

1.4.15

limit state

condition beyond which a structure would cease to be fit for its intended use

1.4.16

strength

resistance to failure; specifically, limiting value for stress

1.5 Symbols

For the purposes of this part of BS 5950, the following symbols apply:

A		Area
	or	Gross area of a cross-section
A_e		Effective net area of a section
A_{eff}		Effective area
A_n		Net area of a section
A_{st}		Area of an intermediate stiffener
A_t		Tensile stress area of a bolt
a		Effective throat size of a fillet weld
a_1		Net sectional area of connected elements
a_2		Gross sectional area of unconnected elements
B		Overall width of an element
B_f		Half the overall flange width of an element
b		Flat width of an element
b_{eff}		Effective width of a compression element
b_{er}		Reduced effective width of a sub-element
b_{eu}		Effective width of an unstiffened compression element
C_b		Coefficient defining the variation of moments on a beam
C_T		Constant depending on the geometry of a T-section
C_W		Warping constant of a section
c		Distance from the end of a beam to the load or the reaction as shown in Tables 7 and 8
D		Overall web depth
D_c		Depth of the compression zone in a web
D_e		Equivalent depth of an intermediately stiffened web
D_w		Equivalent depth of a stiffened web
D_1		Distance between the centre line of an intermediate web stiffener and the compression flange
d		Diameter of a bolt
	or	Diameter of a spot weld
	or	Flat width of an element as shown in Tables C.1 and D.1
	or	As otherwise defined in a clause
d_e		Distance from the centre of a bolt to the end of an element
d_p		Peripheral diameter of an arc spot weld or elongated arc spot weld
d_r		Recommended tip diameter of an electrode
d_s		Interface diameter of an arc spot weld or elongated arc spot weld
d_w		Visible diameter of an arc spot weld or width of elongated plug weld
E		Modulus of elasticity of steel
e		Distance between a load and a reaction as shown in Tables 7 and 8 or the shear centre position as shown in Table D.1
e_s		Distance between the geometric neutral axis and the effective neutral axis of a section
F_c		Applied axial compressive load
F_s		Shear force (bolts)
F_t		Applied tensile load
F_v		Shear force
F_w		Concentrated load on a web
f_a		Average stress in a flange
f_c		Compressive stress on the effective element
f_w		Applied compressive stress
G		Shear modulus of steel
g		Gauge, i.e. distance measured at right angles to the direction of stress in a member, centre-to-centre of holes in consecutive lines
h		Vertical distance between two rows of connections in channel sections
	or	As defined in annex B
I		Second moment of area of a cross-section about its critical axis
I_{min}		Minimum required second moment of area of a stiffener
I_s		Second moment of area of a multiple stiffened element
I_x, I_y		Second moment of area of a cross-section about the x and y axes respectively
J		St Venant torsion constant of a section
K		Buckling coefficient of an element
L		Length of a member between support points
L_E		Effective length of a member
L_w		Length of a weld
M		Applied moment on a beam
M_b		Buckling resistance moment
M_c		Moment capacity of a cross-section (as determined from **5.2.2**)
M'_c		Design moment capacity of a section utilizing plastic bending capacity (see **5.2.3**)
M_{cr}		Critical bending moment causing local buckling in a beam
M_{cx}		Moment capacity in bending about the x axis in the absence of F_c and M_y
M_{cy}		Moment capacity in bending about the y axis in the absence of F_c and M_x
M_E		Elastic lateral buckling moment of a beam
M_p		Plastic moment capacity of a section
M_x, M_y		Moment about x and y axes respectively
M_Y		Yield moment of a section
N		Number of 90° bends in a section
	or	Length of bearing as shown in Tables 7 and 8
	or	Number of tests
P_{bs}		Bearing capacity of a bolt
P_c		Buckling resistance under axial load
P_{cs}		Short strut capacity

P_E	Elastic flexural buckling load (Euler load) for a column
P_{Ex}, P_{Ey}	Elastic flexural buckling load (Euler load) for a column about x and y axes respectively
P_{fs}	Shear capacity of a fastener
P_{ft}	Tensile capacity of a fastener
P_s	Shear capacity of a connection
P_T	Torsional buckling load of a column
P_{TF}	Torsional flexural buckling load of a column
P_t	Tensile capacity of a member or connection
P_v	Shear capacity or shear buckling resistance of a member
P_w	Concentrated load resistance of a single web
p_c	Compressive strength
p_{cr}	Local buckling stress of an element
p_0	Limiting compressive stress in a flat web
p_s	Shear strength of a bolt
p_v	Shear yield strength
p_y	Design strength of steel
p_w	Design strength of weld
Q	Factor defining the effective cross-sectional area of a section
q_{cr}	Shear buckling strength of a web
$R_{d,i}$	Resistance predicted by the design expression for the specific parameters
R_{eH}	Upper yield strength of steel (as defined by BS EN 10002-1)
R_{eL}	Lower yield strength of steel (as defined by BS EN 10002-1)
R_m	Tensile strength of steel (as defined by BS EN 10002-1)
$R_{p\ 0.2}$	0.2 % proof stress (as defined by BS EN 10002-1)
$R_{t\ 0.5}$	Stress at 0.5 % total elongation (as defined by BS EN 10002-1)
r	Inside bend radius
or	Radius of gyration
r_{cy}	Radius of gyration of a channel about its centroidal axis parallel to the web
r_I	Radius of gyration of a compound I-section
r_o	Polar radius of gyration of a section about the shear centre
r_x, r_y	Radii of gyration of a section about the x and y axes respectively
S	Plastic modulus of a section
S_o	Original cross-sectional area of the parallel length in a tensile test specimen (as defined in BS EN 10002-1)
s	Distance between the centres of bolts normal to the line of applied force or, where there is only a single line of bolts, the width of the sheet
or	Leg length of a fillet weld
or	Standard deviation
s_p	Staggered pitch, i.e. the distance, measured parallel to the direction of stress in a member, centre-to-centre of holes in consecutive lines
t	Net material thickness
or	As otherwise defined in a clause
t_s	Equivalent thickness of a flat element to replace a multiple stiffened element for calculation purposes
t_1, t_2	Thickness of thinner and thicker materials connected by spot welding or as defined in annex B
U_e	Nominal ultimate tensile strength of the electrode
U_f	Minimum tensile strength of fastener
U_s	Nominal ultimate tensile strength of steel (See **3.3.2**)
U_{ss}	Nominal ultimate tensile strength of the steel in the supporting members
u	Deflection of a flange towards the neutral axis due to flange curling
W	Total distributed load on a purlin
W_d	Weight of cladding acting on a sheeting rail
W_w	Wind load acting on a sheeting rail
w	Flat width of a sub-element
or	Intensity of load on a beam
w_s	Equivalent width of a flat element to replace a multiple stiffened element for calculation purposes
x_o	Distance from the shear centre to the centroid of a section measured along the x axis of symmetry
Y_f	Minimum yield strength of a fastener
Y_s	Nominal yield strength of steel (See **3.3.2**)
Y_{sa}	Average yield strength of a cold formed section
Y_{sac}	Modified average yield strength in the presence of local buckling
y	Distance of a flange from the neutral axis
Z_c	Compression modulus of a section in bending
α	Coefficient of linear thermal expansion
or	Effective length multiplier for torsional flexural buckling
β	Ratio of end moments in a beam
or	Constant defined in **6.3.2**
γ_f	Overall load factor
γ_l	Variability of loading factor
γ_m	Material strength factor
γ_p	Structural performance factor
Δ	Beam deflection
Δ_c	Beam deflection at moment M_c
Δ_{cr}	Beam deflection at the point of local buckling
η	Perry coefficient
θ	Angle between the web of a beam and the bearing surface
ν	Poisson ratio

Section 2. Limit state design

2.1 General principles and design methods

2.1.1 General

Structures should be designed following consideration of the limit states at which the proposed design becomes unfit for its intended use, by applying appropriate factors for the ultimate limit state and the serviceability limit state.

All relevant limit states should be considered, but usually it is appropriate to design on the basis of strength and stability at ultimate loading and then to check that the deflection is not excessive under serviceability loading. Examples of limit states relevant to steel structures are given in Table 1.

The overall factor in any design takes account of variability in the following:

— material strength: (γ_m);

— loading: (γ_l);

— structural performance: (γ_p).

In this part of BS 5950 the material factor γ_m is incorporated in the recommended design strengths (see **3.3.2**). 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 Table 2.

NOTE A detailed breakdown of γ factors is given in BS 5950-1.

2.1.2 Methods of design

2.1.2.1 *General*

The design of any structure or its parts may be carried out by one of the methods given in **2.1.2.2** to **2.1.2.7**.

In all cases, the details of members and connections should be capable of realizing the assumptions made in design without adversely affecting any other parts of the structure.

2.1.2.2 *Simple design*

The connections between members are 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 connections may result in some non-elastic deformation of the materials, other than the fasteners.

Sway stability should be maintained in accordance with the recommendations given in **2.3.2.3**.

2.1.2.3 *Rigid design*

The connections are assumed to be capable of developing the strength and/or stiffness required by an analysis assuming full continuity. Such analysis may be made using either elastic or plastic methods.

2.1.2.4 *Semi-rigid design*

Some degree of connection stiffness is assumed, but insufficient to develop full continuity as follows.

a) The moment and rotation capacity of the joints should be based on experimental evidence, which may permit some limited plasticity providing the ultimate tensile capacity of the fastener is not the failure criterion. On this basis, the design should satisfy the strength, stability and stiffness requirements of all parts of the structure when partial continuity at the joints is to be taken into account in assessing moments and forces in the members.

b) As an alternative, in simple beam and column structures an allowance may be made for the inter-restraint of the connections between a beam and a column by an end restraint moment not exceeding 10 % of the free moment applied to the beam, assuming this to be simply supported, provided that the following apply.

1) The beams and columns are designed by the general rules applicable to simple design.

2) The frame is provided with lateral support or braced against side-sway in both directions.

3) The beams are designed for the maximum net moment which includes an allowance for the restraint moment at one or both ends.

Table 1 — Limit states relevant to steel structures

Ultimate limit state		Serviceability limit state	
1	Strength (including general yielding, rupture, buckling and transformation into a mechanism)	6	Deflection
2	Stability against overturning and sway	7	Vibration (e.g. wind induced oscillation)
3	Excessive local deformation	8	Repairable damage due to fatigue
4	Fracture due to fatigue	9	Durability
5	Brittle fracture		

4) Each column is designed to resist the algebraic sum of the restraint moments from the beams at the same level on each side of the column, in addition to moments due to eccentricity of connections.

5) The assumed end restraint moment need not, however, be taken as 10 % of the free moment for all beams, provided that the same restraint moment is used in the design of both the column and beam at each connection.

6) The beam-to-column connections are designed to transmit the appropriate restraint moment, in addition to the end reactions assuming the beams are simply supported.

7) The welds and fasteners are designed for the actual moment capacity of the connection not the assumed moment.

2.1.2.5 *Composite design*

Composite design takes into account the enhanced load capacity and serviceability when steelwork is suitably interconnected to other materials, e.g. concrete, timber and building boards, in order to ensure composite behaviour of the member or structure.

NOTE Recommendations for composite design utilizing steel and concrete are given in BS 5950-3-3.1.

2.1.2.6 *Stressed skin design*

The strengthening and stiffening effect of steel cladding and decking may be taken into account in the structural design.

NOTE Recommendations for stressed skin design are given in BS 5950-9.

2.1.2.7 *Testing*

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 and stiffness may be confirmed by loading tests in accordance with section **10**.

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. In particular, the frequency of vibration resulting from any fluctuating loads compared to the natural frequency of the structure should be checked. Consideration should also be given to connections to ensure that their effectiveness is not reduced.

Loading conditions during erection should receive particular attention. Settlement of supports may need to be taken into account.

2.2.2 Dead, imposed and wind loading

Determination of dead, imposed and wind loads should be made in accordance with BS 6399-1, -2 or -3 as appropriate, and CP3: Chapter V: Part 2.

Loads on agricultural buildings should be calculated in accordance with BS 5502-22.

NOTE It is intended that BS 6399-2 should eventually replace CP3: Chapter V: Part 2. This may require a change to the design rules for the application of wind loads to structures. For structures designed in accordance with this edition of BS 5950-5, wind loads may continue to be determined in accordance with CP3: Chapter V: Part 2, until such time as it is withdrawn. In such cases, for the design of purlins and sheeting rails, local wind pressure and suction need not be considered.

2.2.3 Accidental loading

Determination of accidental loading should be made in accordance with BS 6399-1 where appropriate.

When considering the continued stability of a structure after it has sustained accidental damage, the loads considered should be those likely to occur before repairs can be completed.

2.2.4 Temperature effects

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 mean temperature of the 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 special conditions, and in locations abroad subject to different temperature ranges.

2.3 Ultimate limit states

2.3.1 Limit states of strength

2.3.1.1 *General*

In checking the strength and stability of the structure the loads should be multiplied by the relevant γ_f factors given in Table 2. The factored loads should be applied in the most unfavourable realistic combination for the component or structure under consideration.

The load capacity of each member and its connections, as determined by the relevant provisions of this part of BS 5950, should be such that the factored loads would not cause failure.

2.3.1.2 *Overhead cranes*

If overhead cranes are provided, detailed designs should be made in accordance with BS 5950-1.

2.3.2 Stability limit state

2.3.2.1 *General*

In considering the overall stability of any structure or part, the loads should be increased by the relevant γ_f factors given in Table 2.

The designer should consider overall frame stability which embraces stability against overturning and sway stability.

2.3.2.2 ***Stability against overturning***

The factored loads should not cause the structure or any part of the structure (including the foundations) to overturn or lift off its seating. The combination of wind, imposed and dead loads should be such as to have the most severe effect on overall stability (see **2.2.1**).

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

Table 2 — Load factors and combinations

Loading	Factor, γ_f
Dead load	1.4
Dead load restraining uplift or overturning	1.0
Dead load acting with wind and imposed loads combined	1.2
Imposed load	1.6
Imposed load acting with wind load	1.2
Wind load	1.4
Wind load acting with imposed load	1.2
Forces due to temperature effects	1.2

2.3.2.3 ***Sway stability***

All structures, including portions between expansion joints, should have adequate strength against sway.

To ensure this, in addition to designing for applied horizontal loads, a separate check should be carried out for notional horizontal forces.

These notional forces may arise from practical imperfections such as lack of verticality and should be taken as the greater of:

1 % of factored dead load from that level, applied horizontally;

0.50 % of factored load (dead plus vertical imposed) from that level, applied horizontally.

These notional 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 loads taken as the sum of:

1.4 × dead load; plus

1.6 × vertical imposed load.

The notional force should not:

a) be applied when considering overturning;

b) be combined with the applied horizontal loads;

c) be combined with temperature effects;

d) be taken to contribute to net reactions on the foundations.

Sway stability may be provided for example by braced frames, joint rigidity or by utilizing staircases, lift cores and shear walls. Whatever system is used, reversal of loading 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 of sway resistance. Where such sway stability is provided by construction other than the steel framework, the steelwork designer should clearly state the need for such construction and the forces acting upon it.

2.3.2.4 Foundation design

Foundations should be designed in accordance with BS 8004 to accommodate all the forces and moments imposed on them. Attention should be given to the method of connecting the steel superstructure to the foundations and the anchorage of any holding-down bolts. 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.

2.3.3 Fatigue

Fatigue need not be considered unless a structure or element is subject to numerous significant fluctuations of load excluding those arising from wind. However, account should be taken of wind-induced oscillations where these occur. When designing for fatigue a γ_f factor of 1.0 should be used.

2.3.4 Brittle fracture

At temperatures below −15 °C consideration should be given to the possibility of brittle fracture in welded tension areas and in the vicinity of punched holes.

2.3.5 Structural integrity

2.3.5.1 ***Recommendations for all structures***

All structures should follow the principles given in **1.1** and **2.1**. The additional recommendations given in **2.3.5.2** and **2.3.5.3** apply to buildings.

2.3.5.2 ***Recommendations for all buildings***

Every building frame should be effectively tied together at each principal floor and roof level. All columns should be anchored in two directions, approximately at right angles, at each principal floor or roof which they support. This anchorage may be provided by either beams or tie members.

Members provided for other purposes may be utilized as ties. When members are checked as ties, other loading may be ignored. Beams designed to carry the floor or roof loading will generally be suitable provided that their end connections are capable of resisting tension.

Where a building is provided with expansion joints, each section between expansion joints should be treated as a separate building for the purpose of this subclause.

2.3.5.3 ***Additional recommendations for certain buildings***

When it is stipulated by appropriate regulations that buildings should be designed to localize accidental damage, reference should be made to BS 5950-1 for additional recommendations.

In construction where vertical loads are resisted by an assembly of closely spaced elements (e.g. cold formed steel framing), the tying members should be distributed to ensure that the entire assembly is effectively tied. In such cases the forces for anchoring the vertical elements at the periphery should be based on the spacing of the elements or taken as 1 % of the factored vertical load in the element without applying the minimum value of 75 kN or 40 kN to the individual elements, provided that each tying member and its connections are designed to resist the appropriate loading.

NOTE Further guidance on methods of reducing the sensitivity of buildings to disproportionate collapse in the event of an accident is given in Approved Document A to the Building Regulations [1].

2.4 Serviceability limit states

2.4.1 Serviceability loads

Generally, the serviceability loads should be taken as the unfactored imposed loads. When considering dead load plus imposed load plus wind load, only 80 % of the imposed load and wind load need be considered.

2.4.2 Deflection

The deflection under serviceability loads of a building or its members should not impair the strength or efficiency of the structure or its components or cause damage to the finishings.

When checking the deflections the most adverse realistic combination and arrangement of unfactored loads should be assumed, and the structure may be assumed to be elastic.

Table 3 gives recommended deflection limits for certain structural members. Circumstances may arise where greater or lesser values would be more appropriate. Other members may also require a deflection limit to be established, e.g. sway bracing.

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

2.5 Durability

In order to ensure the durability of the structure under conditions relevant to both its intended use and intended life the following factors should be considered at the design stage:

a) the environment;

b) the degree of exposure;

c) the shape of the members and the structural detailing;

d) the protective measures if any;

e) whether maintenance is possible.

Reference should be made to BS 5493 when determining suitable treatment.

Where different materials are connected together, such as in composite construction, the effects on the durability of the materials should be taken into consideration. Reference should be made to PD 6484 for guidance on preventing corrosion of bimetallic contacts.

Table 3 — Deflection limits

a) *Deflection of beams due to unfactored imposed loads*	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
All other beams	Span/200
Purlins and sheeting rails	See **2.4.2**
b) *Deflection of columns other than portal frames due to unfactored imposed and wind loads*	
Tops of columns in single-storey buildings	Height/300
In each storey of a building with more than one storey	Height of storey under consideration/300
NOTE 1 On low-pitched and flat roofs the possibility of ponding needs consideration.	
NOTE 2 The designer of a framed structure, e.g. portal or multi-storey, should ensure that the stability is not impaired by the interaction between deflections and axial loads.	

Section 3. Properties of materials and section properties

3.1 Range of thicknesses

The provisions of this part of BS 5950 apply primarily to steel sections with a thickness of not more than 8 mm although the use of thicker material is not precluded.

3.2 Design thickness

The design thickness of the material should be taken as the nominal base metal thickness exclusive of coatings.

3.3 Properties of materials

3.3.1 General

This part of BS 5950 covers the design of structures made from the grades of steel conforming to BS 1449 (See Note 1), BS EN 10025, BS EN 10111, BS EN 10147 or BS EN 10149 that are listed in Table 4. Other steels may be used, subject to approval of the engineer, provided due allowance is made for variation in properties, including ductility.

NOTE 1 BS 1449-1:1983 was re-issued as BS 1449-1-1.1 to BS 1449-1-1.15:1991. Each section of the standard is in the process of harmonization, and will be issued as a new European Standard as the work is completed.

NOTE 2 Requirements for materials are given in BS 5950-7.

Table 4 — Yield, ultimate and design strengths

Type of steel	British Standard	Grade	Nominal yield strength[a] Y_s N/mm^2	Nominal ultimate tensile strength[a] U_s N/mm^2	Design strength p_y N/mm^2
Hot rolled steel sheet of structural quality	BS EN 10025	S 235 S 275 S 355	235 275 355	360 430 510	235 275 355
Continuous hot dip zinc coated carbon steel sheet of structural quality	BS EN 10147	S 220 G S 250 G S 280 G S 320 G S 350G	220 250 280 320 350	300 330 360 390 420	220 250 280 320 350
Hot rolled steel sheet based on formability	BS 1449-1-1.8	HS 3 or HS 4	(170)[b]	(280)[b]	140
Hot rolled low carbon steel sheet for cold forming	BS EN 10111	DD 11 or DD 12	(170)[b]	—	140
Hot rolled high yield strength steel for cold forming Thermomechanically rolled steels	BS EN 10149-2	S 315 MC S 355 MC S 420 MC	315 355 420	390 430 480	315 355 400 [c]
Hot rolled high yield strength steel for cold forming Normalized and normalized rolled steels	BS EN 10149-3	S 260 NC S 315 NC S 355 NC S 420 NC	260 315 355 420	370 430 470 530	260 315 355 420
Cold rolled steel sheet based on minimum strength	BS 1449-1-1.5 (CR) or BS 1449-1-1.11 (CS)	34/20 37/23 43/25 50/35 40/30 43/35 40F30 43F35	200 230 250 350 300 350 300 350	340 370 430 500 400 430 400 430	200 230 250 350 300 350 300 350

[a] Nominal yield and ultimate tensile strengths are given for information only. For details see the appropriate product standard.

[b] Figures in brackets are given for guidance only.

[c] Design strength limited to $0.84U_s$.

3.3.2 Strength of steel

The design strength, p_y, should be taken as Y_s but not greater than $0.84U_s$ where:

Y_s is the nominal yield strength (i.e. the higher yield strength, R_{eH}, or in the case of material with no clearly defined yield, either the 0.2 % proof stress, $R_{p\,0.2}$, or the stress at 0.5 % total elongation, $R_{t\,0.5}$, as specified in the relevant material standard);

U_s is the nominal ultimate tensile strength (i.e. the minimum tensile strength, R_m, as specified in the relevant material standard);

and R_{eH}, $R_{p\,0.2}$, $R_{t\,0.5}$ and R_m are as defined in BS EN 10002-1.

For steels conforming to the standards listed in Table 4, the values of R_{eH}, $R_{p\,0.2}$, $R_{t\,0.5}$ and R_m should normally be taken as specified in the relevant product standard for the steel sheet or strip and used for the formed sections. For information, the resulting values of Y_s and U_s are also given in Table 4 together with appropriate design strength p_y for the relevant grade.

NOTE Formability grades have no guaranteed minimum strength, but can be expected to achieve a nominal yield strength of at least 140 N/mm^2.

Alternatively, for steels conforming to an appropriate British Standard and supplied with specific inspection and testing to BS EN 10021, the values of R_{eH}, $R_{p\,0.2}$, $R_{t\,0.5}$ and R_m may be based on the values declared in an inspection certificate in accordance with BS EN 10204.

Reference should be made to BS 5950-7 for recommendations concerning the testing regime required to determine the characteristic properties of any steel not certified as conforming to an appropriate British Standard.

The design strength, p_y, may be increased due to cold forming as given in **3.4**.

3.3.3 Other properties of steel

The following values for the elastic properties should be used.

Modulus of elasticity	E = 205 kN/mm^2
Shear modulus	G = 79 kN/mm^2
Poisson ratio	ν = 0.30
Coefficient of linear thermal expansion	$\alpha = 12 \times 10^{-6}$ per °C

3.4 Effects of cold forming

The increase in yield strength due to cold forming may be taken into account throughout this part of BS 5950 by replacing the material yield strength, Y_s, by Y_{sa}, the average yield strength of the cold formed section. Y_{sa} may be determined by tests in accordance with section **10**, or calculated as follows:

$$Y_{sa} = Y_s + \frac{5Nt^2}{A}(U_s - Y_s)$$

where

N is the number of full 90° bends in the section with an internal radius < $5t$ (fractions of 90° bends should be counted as fractions of N);

t is the net thickness of the material in millimetres (mm);

U_s is the minimum ultimate tensile strength in newtons per square millimetre (N/mm^2);

A is the gross area of the cross-section in square millimetres (mm^2).

The value of Y_{sa} used in calculations should not exceed 1.25 Y_s or U_s.

The full effect of cold working on the yield strength may be used for calculating the tensile strength of elements. For elements of flat width, b, and thickness, t, under compression the value of Y_{sa} should be modified as follows to provide the appropriate compression yield strength, Y_{sac}.

For stiffened elements:

for $b/t \leq 24\left(\frac{280}{Y_s}\right)^{1/2}$

$Y_{sac} = Y_{sa}$

for $b/t \geq 48\left(\frac{280}{Y_s}\right)^{1/2}$

$Y_{sac} = Y_s$

For unstiffened elements:

for $b/t \leq 8\left(\frac{280}{Y_s}\right)^{1/2}$

$Y_{sac} = Y_{sa}$

for $b/t \geq 16\left(\frac{280}{Y_s}\right)^{1/2}$

$Y_{sac} = Y_s$

For intermediate values of b/t the value of Y_{sac} may be obtained by linear interpolation.

The design strength, p_y, may be taken as Y_{sa} or Y_{sac} as appropriate.

The increase in yield strength due to cold working should not be utilized for members which undergo welding, annealing, galvanizing or any other heat treatment after forming which may produce softening.

3.5 Calculation of section properties

3.5.1 Method of calculation

Section properties should be calculated according to normal good practice, taking due account of the sensitivity of the properties of the overall cross-section to any approximations used and their influence on the predicted resistance of the member. In the calculation of section properties for material up to 3.2 mm thickness it should usually be sufficient to assume that the material is concentrated at the mid-line of the material thickness, and the actual round corners are replaced by intersections of the flat elements.

NOTE Section properties for a range of generic profiles are given in BS 2994.

3.5.2 Cross-section properties

When calculating cross-section properties, holes for fasteners need not be deducted but allowance should be made for large openings or arrays of small holes. Material acting solely as battens or splices should not be included.

3.5.3 Net section properties for members in bending or compression

The net section properties of members with regular or irregular arrays of holes, other than holes required for fastening and filled with bolts, may be determined by analytical methods or by testing in accordance with **10.3** and **10.4** for members in bending or compression respectively.

3.5.4 Section properties for members in tension

3.5.4.1 ***Net area***

The net area, A_n, of a section should be taken as its gross area less deductions for all holes and openings.

3.5.4.2 ***Hole diameter***

When deducting for holes for fasteners, the nominal hole diameter should be used.

3.5.4.3 ***Countersunk holes***

For countersunk holes, the area to be deducted should be the gross cross-sectional area of the hole.

3.5.4.4 ***Non-staggered holes***

The area to be deducted from the gross sectional area should be the maximum sum of the sectional areas of the holes in any cross-section at right angles to the direction of stress in the member.

3.5.4.5 ***Staggered holes***

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

a) the deduction for non-staggered holes;

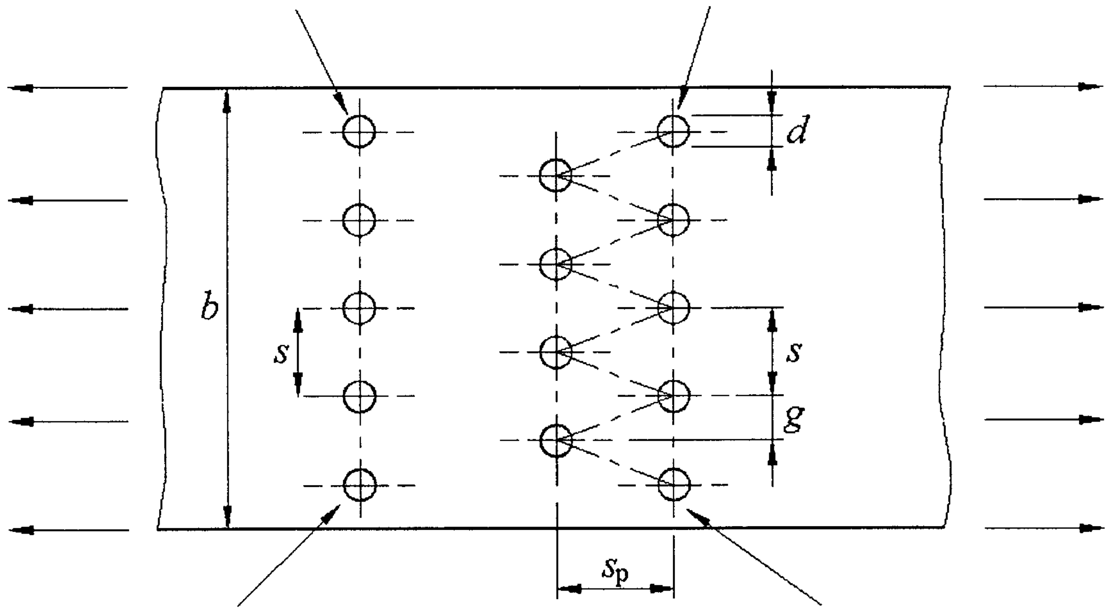
b) the sum of the sectional areas of all holes in any zigzag line extending progressively across the member or part of the member, less $s_p^2t/4g$ for each gauge space in the chain of holes

where

s_p is the staggered pitch, i.e. the distance, measured parallel to the direction of stress in the member centre-to-centre of holes in consecutive lines (see Figure 1);

t is the thickness of the holed material;

g is the gauge, i.e. the distance measured at right angles to the direction of stress in the member, centre-to-centre of holes in consecutive lines (see Figure 1).



Net area after deduction in **3.5.4.5**a) = $bt - 5dt$

Net area after deduction in **3.5.4.5**b) = $bt - \left(9dt - \frac{8s_p^2t}{4g}\right)$

Figure 1 — Nomenclature for staggered holes with example

Section 4. Local buckling

4.1 General

The effects of local buckling should be taken into account in determination of the design strength and stiffness of cold formed members. This may be accomplished using effective cross-sectional properties which are calculated on the basis of the widths of individual elements.

In the calculation of section properties the effective positions of compression elements covered by this section should be located as follows.

a) In the case of elements which are adequately supported on both longitudinal edges, i.e. stiffened elements, the effective width of the element should be taken as composed of two equal portions, one adjacent to each edge.

b) In the case of elements which have only one adequately supported longitudinal edge. i.e. unstiffened elements, the effective width should be taken as located adjacent to the supported edge.

4.2 Maximum width to thickness ratios

The maximum ratios of element flat width, b, to thickness, t, which are covered by the design procedures given in this part of BS 5950 are as follows, for compression elements.

a) Stiffened elements having one longitudinal edge connected to a flange or web element, the other stiffened by:	
simple lip (see Figure 2)	60
any other type of stiffener conforming to **4.6**	90
b) Stiffened elements with both longitudinal edges connected to other stiffened elements	500
c) Unstiffened compression elements	60

NOTE Unstiffened compression elements that have width to thickness ratios exceeding approximately 30 and stiffened compression elements that have width to thickness ratios exceeding approximately 250 are likely to develop noticeable deformations at the full working load, without affecting the ability of the member to carry this load.

4.3 Basic effective width

The ratio of effective width, b_{eff}, to full flat width, b, of an element under compression may be determined from the following:

for $f_c/p_{cr} \leq 0.123$

$$\frac{b_{eff}}{b} = 1$$

for $f_c/p_{cr} > 0.123$

$$\frac{b_{eff}}{b} = [1 + 14\{(f_c/p_{cr})^{1/2} - 0.35\}^4]^{-0.2}$$

where

f_c is the compressive stress on the effective element;

p_{cr} is the local buckling stress of the element given by:

$$p_{cr} = 0.904EK\left(\frac{t}{b}\right)^2$$

where

K is the local buckling coefficient which depends on element type, section geometry and is detailed for various cases in annex B;

t is the material thickness.

4.4 Effective widths of plates with both edges supported (stiffened elements)

4.4.1 Elements under uniform compression

The effective width of a stiffened element under uniform compression should be determined in accordance with **4.3** using the appropriate K factor.

K may be taken as 4 for any stiffened element. In certain cases, detailed in annex B, higher values of K may be used.

For elements made of steel with a yield strength, Y_s, of 280 N/mm^2 and having $K = 4$, the effective widths determined in accordance with **4.3** with f_c = 280 N/mm^2 are listed in Table 5.

For elements in which the compressive stress, f_c is other than 280 N/mm^2, or having K values other than 4, the ratio b_{eff}/b may be obtained from Table 5 using a modified width to thickness ratio, b/t. The values of the modified b/t may be found by multiplying the actual b/t by $\sqrt{(f_c/280)(4/K)}$ where f_c is the actual compressive stress on the element, which may be taken as p_y or, in the case of compression flanges of beams, as p_0, where p_0 is the limiting compressive stress determined in accordance with **5.2.2.2** or **5.2.2.3**.

The effective width may be obtained from the product of the ratio b_{eff}/b given in Table 5 and the actual element width.

4.4.2 Elements under stress gradient

The effective width of a compression element in which the stress varies linearly from f_{c1}, at one edge to f_{c2} at the other edge with $f_{c1} > f_{c2} > 0$ may be determined in accordance with **4.3** with f_{cm} substituted for f_c, where f_{cm} is the mean value of the compressive stress on the effective element.

In the case of elements in which the stress varies from compression to tension, the design procedure given in section **5** should be used in obtaining element properties.

Table 5 — Effective widths for stiffened elements

b/t	b_{eff}/b	b/t	b_{eff}/b	b/t	b_{eff}/b	b/t	b_{eff}/b
20	1.000	60	0.673	100	0.405	300	0.151
21	1.000	61	0.662	105	0.387	305	0.149
22	1.000	62	0.652	110	0.370	310	0.147
23	1.000	63	0.641	115	0.355	315	0.145
24	0.999	64	0.631	120	0.341	320	0.143
25	0.999	65	0.621	125	0.328	325	0.141
26	0.998	66	0.612	130	0.316	330	0.139
27	0.997	67	0.603	135	0.305	335	0.138
28	0.996	68	0.594	140	0.295	340	0.136
29	0.994	69	0.585	145	0.286	345	0.134
30	0.992	70	0.577	150	0.277	350	0.133
31	0.989	71	0.569	155	0.269	355	0.131
32	0.985	72	0.561	160	0.262	360	0.130
33	0.981	73	0.553	165	0.254	365	0.128
34	0.976	74	0.545	170	0.248	370	0.127
35	0.969	75	0.538	175	0.241	375	0.125
36	0.962	76	0.531	180	0.235	380	0.124
37	0.955	77	0.524	185	0.230	385	0.122
38	0.946	78	0.517	190	0.224	390	0.121
39	0.936	79	0.511	195	0.219	395	0.120
40	0.926	80	0.504	200	0.215	400	0.119
41	0.915	81	0.498	205	0.210	405	0.117
42	0.903	82	0.492	210	0.206	410	0.116
43	0.891	83	0.486	215	0.201	415	0.115
44	0.878	84	0.480	220	0.197	420	0.114
45	0.865	85	0.475	225	0.194	425	0.113
46	0.852	86	0.469	230	0.190	430	0.112
47	0.838	87	0.464	235	0.186	435	0.111
48	0.824	88	0.459	240	0.183	440	0.109
49	0.811	89	0.454	245	0.180	445	0.108
50	0.797	90	0.449	250	0.177	450	0.107
51	0.784	91	0.444	255	0.174	455	0.106
52	0.771	92	0.439	260	0.171	460	0.106
53	0.757	93	0.435	265	0.168	465	0.105
54	0.745	94	0.430	270	0.165	470	0.104
55	0.732	95	0.426	275	0.163	475	0.103
56	0.720	96	0.421	280	0.160	480	0.102
57	0.708	97	0.417	285	0.158	485	0.101
58	0.696	98	0.413	290	0.156	490	0.100
59	0.684	99	0.409	295	0.153	495	0.099
60	0.673	100	0.405	300	0.151	500	0.098

NOTE These effective widths are based on the limit state of strength for steel with Y_S = 280 N/mm^2 and having a buckling coefficient K = 4. For steels with other values of Y_S or sections having $K \neq 4$ see **4.4.1**.

4.5 Effective widths of plates with one edge supported (unstiffened elements)

4.5.1 Elements under uniform compression

The effective width, b_{eu}, of an unstiffened element under uniform compression may be obtained from the following:

$$b_{eu} = 0.89b_{eff} + 0.11b$$

where

- b_{eff} is determined in accordance with **4.3** (the value of K may be taken as 0.425 for any unstiffened element, but higher values may be used for the cases given in annex B);
- b is the full flat width.

For elements of steel with a yield strength, Y_s, of 280 N/mm^2 and having $K = 0.425$, the effective widths determined in accordance with **4.3** and modified in this way with $f_c = 280$ N/mm^2 are listed in Table 6. For elements of steel with Y_s other than 280 N/mm^2 or K values other than 0.425, the ratio b_{eu}/b may be obtained from Table 6 using a modified width to thickness ratio, b/t. The value of the modified b/t may be found by multiplying the actual b/t by $\sqrt{(f_c/280)(0.425/K)}$ where f_c is the actual compressive stress on the element, which may be taken as p_y or, in the case of compression flanges of beams as p_0, where p_0 is the limiting compressive stress determined in accordance with **5.2.2.2** or **5.2.2.3**.

The effective width may be obtained from the product of the ratio b_{eu}/b given in Table 6 and the actual element width.

4.5.2 Elements under combined bending and axial load

The effective width of an unstiffened element subjected to combined bending and axial load may be obtained as follows.

a) If the loading is such as to cause compression of the free edge the effective width may be determined in accordance with **4.5.1** with f_c replaced by the stress at the free edge, f_{cf} and the value of K taken as:

$$K = \frac{1.7}{3 + R}$$

where

- R is the ratio of the stress at the supported edge, f_{cs}, to f_{cf}, computed on the basis that the element is fully effective and with compressive stresses being taken as positive.

Increased values of K for specific cases are given in annex B.

b) If the loading is such as to cause tension of the free edge the element should be treated as a stiffened element, except that the limitations on maximum width to thickness ratios for unstiffened elements given in **4.2** should be observed.

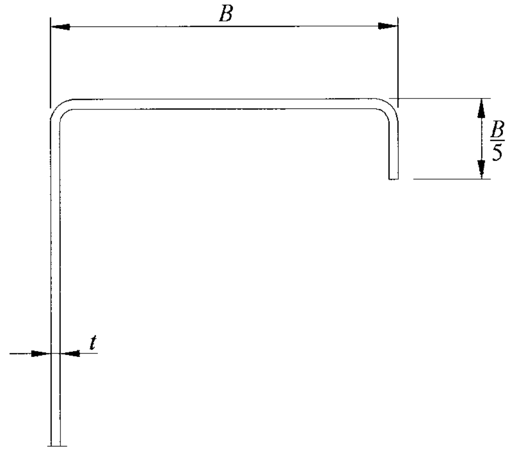


Figure 2 — Simple lip edge stiffener

4.6 Edge stiffeners

In order that a flat compression element may be considered a stiffened element it should be supported along one longitudinal edge by the web, and along the other by a web, lip or other edge stiffener which has adequate bending rigidity to maintain straightness of this edge under load.

Irrespective of its shape, the minimum allowable second moment of area of an edge stiffener, I_{min}, about an axis through the middle surface of the element to be stiffened is:

$$I_{min} = \frac{tB^3}{375}$$

where

- t is the material thickness;
- B is the overall width of the element to be stiffened.

Where the stiffener consists of a simple lip bent at right angles to the stiffened element an overall width of lip equal to one-fifth of the overall element width, B, as indicated in Figure 2, may be taken as satisfying this condition.

Where a beam compression element is stiffened by a simple lip, the lip should not be splayed by more than 20° from the perpendicular.

Table 6 — Effective widths for unstiffened elements

b/t	b_{eu}/b	b/t	b_{eu}/b	b/t	b_{eu}/b
1	1.000	21	0.668	41	0.400
2	1.000	22	0.643	42	0.394
3	1.000	23	0.619	43	0.388
4	1.000	24	0.598	44	0.382
5	1.000	25	0.578	45	0.376
6	1.000	26	0.560	46	0.371
7	1.000	27	0.544	47	0.366
8	0.999	28	0.528	48	0.361
9	0.997	29	0.514	49	0.356
10	0.991	30	0.501	50	0.352
11	0.980	31	0.489	51	0.348
12	0.961	32	0.477	52	0.343
13	0.935	33	0.466	53	0.339
14	0.903	34	0.456	54	0.336
15	0.868	35	0.447	55	0.332
16	0.831	36	0.438	56	0.328
17	0.794	37	0.429	57	0.325
18	0.759	38	0.422	58	0.322
19	0.726	39	0.414	59	0.319
20	0.696	40	0.407	60	0.315

NOTE These effective widths are based on the limit state of strength for steel with $Y_S = 280\ \text{N/mm}^2$ and having a buckling coefficient $K = 0.425$. For steels with other values of Y_S or sections having $K \neq 0.425$ see **4.5.1.**

4.7 Intermediate stiffeners

4.7.1 Minimum stiffener rigidity

In order that a flat compression element may be considered a multiple-stiffened element, it should be stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress, with these stiffeners having a minimum second moment of area, I_{min}, about an axis through the middle surface of the stiffened element given by:

$$I_{min} = 0.2t^4 \left(\frac{w}{t}\right)^2 \left(\frac{Y_s}{280}\right)$$

where

t is the material thickness;

w is the flat width of the sub-element between stiffeners (where sub-elements on either side of an intermediate stiffener are unequal the larger value of w should be used);

Y_s is the minimum yield strength.

4.7.2 Reduced sub-element properties

Where the width to thickness ratio, w/t, of a flat sub-element of a multiple-stiffened compression element is less than 60, the effective width should be determined in accordance with **4.3**. Where w/t exceeds 60, the effective width of the sub-element should be reduced to b_{er} in accordance with the following:

$$\frac{b_{er}}{t} = \frac{b_{eff}}{t} - 0.1\left(\frac{w}{t} - 60\right)$$

where

b_{eff} is the effective width of the sub-element determined in accordance with **4.3**.

For computing the effective properties of a member having compression sub-elements subject to these reductions in effective width, the area of stiffeners, A_{st}, should be considered to be reduced to an effective area, A_{eff}, as follows.

For $w/t \leq 60$

$$A_{eff} = A_{st}$$

For $60 < w/t < 90$

$$A_{eff} = A_{st}\left\{3 - 2\frac{b_{er}}{w} - \frac{1}{30}\left(1 - \frac{b_{er}}{w}\right)\frac{w}{t}\right\}$$

For $w/t \geq 90$

$$A_{eff} = A_{st}\frac{b_{er}}{w}$$

A_{st} and A_{eff} refer to the area of the stiffener alone, exclusive of any portion of adjacent elements and w is as defined in **4.7.1**.

The centroid of the stiffener should be considered to be located at the centroid of the full area of the stiffener, and the second moment of area of the stiffener about its own centroidal axis should be taken as that of the full section of the stiffener.

4.7.3 Limitations in the case of multiple-intermediate stiffeners

Where the spacing of intermediate stiffeners is such that the width to thickness ratio, w/t, of the sub-element is larger than 30, only two intermediate stiffeners (those nearest each web) should be considered effective.

Where the intermediate stiffeners are spaced so closely that the width to thickness ratio of the sub-element is less than 30 then all stiffeners may be considered to be effective.

For the purposes of calculating the effective width of the complete multiple-stiffened element this element should be considered so replaced by an element without intermediate stiffeners whose width, w_s, is the whole width between two webs and whose equivalent thickness, t_s, is determined as follows:

$$t_s = \left(\frac{12I_s}{w_s}\right)^{0.33}$$

where

I_s is the second moment of area of the full area of the multiple stiffened element, including the intermediate stiffeners. about its own neutral axis.

Section 5. Design of members subject to bending

5.1 General

This section is concerned with structural components which are subjected to loads acting normally to the longitudinal axis of the components. Primarily, these loads give rise to bending actions which result in deformation in the line of the loading. However, it is possible for secondary factors, such as instability and torsion, to occur which will cause other types of deformation with rotation of the component cross-section about its longitudinal axis.

5.2 Laterally stable beams

5.2.1 General

This clause is concerned with beams which are laterally stable, either because they are restrained by adequate bracing or because they satisfy the conditions of **5.6**.

5.2.2 Determination of moment capacity

5.2.2.1 *General*

In the case of sections which have stiffened webs or bending elements, the moment capacity should be determined on the basis of a limiting compressive stress in the webs, p_0, determined in accordance with **5.2.2.2** and **5.2.2.3**. This stress is used in evaluation of the effective widths of compression elements, and hence the reduced section properties, and in the determination of the moment capacity, M_c.

In determination of the moment capacity, no allowance should be made for redistribution of compressive stresses, except for sections covered by **5.2.3**.

In cases where tensile stresses reach the minimum yield strength, Y_s, before the compressive stresses reach p_0, plastic redistribution of tensile stresses may be taken into account in analysis.

In the case of sections which have unstiffened webs or bending elements, the same limiting stress approach should be used if bending causes the free edges to be subject to tension. If bending causes compression of the free edges then the moment capacity should be evaluated using the effective width of these elements as given in **5.2.2.5**.

5.2.2.2 *Limiting stress for stiffened webs or bending elements under stress gradient*

The compressive stress, p_0, in a stiffened element which results from bending in its plane, should not exceed the lesser of the following values:

$$p_0 = \left\{1.13 - 0.0019\frac{D_w}{t}\left(\frac{Y_s}{280}\right)^{1/2}\right\}p_y$$

or

$$p_0 = p_y$$

where

D_w is the section depth or twice the depth of the compression zone, D_c, whichever is the greater in millimetres (mm);

D_c is the depth of the compression zone of the web, taken as the distance from the neutral axis of the gross cross-section to the compression element in millimetres (mm)

Y_s is the material yield strength in newtons per square millimetre (N/mm^2);

t is the web thickness in millimetres (mm);

p_y is the design strength in newtons per square millimetre (N/mm^2).

5.2.2.3 *Intermediately stiffened element under stress gradient*

Where a web element has an intermediate stiffener which satisfies the conditions of **4.7.1**, then the limiting compressive stress, p_0, may be taken as the lesser of the following values:

$$p_0 = \left\{1.13 - 0.0019\frac{D_e}{t}\left(\frac{Y_s}{280}\right)^{1/2}\right\}p_y$$

or

$$p_0 = p_y$$

where

D_e is the equivalent depth of the compression zone of the web, taken as the larger of the values given by:

$$D_e = D_w - D_1$$

or

$$D_e = 0.75D_w + 0.25D_1$$

where

D_1 is the distance between the centre line of the intermediate stiffener and the compression flange in millimetres (mm). (Where a web has a number of intermediate stiffeners, the value of D_1 should be assessed on the basis of the stiffener nearest the compression flange, with all other stiffeners disregarded);

D_w, Y_s, p_y and t are as defined in **5.2.2.2**.

5.2.2.4 ***Effective width of elements under uniform compression***

The effective widths of elements under uniform compression should be determined in accordance with section **4**. Values of K for particular components are given in annex B.

5.2.2.5 ***Effective width of unstiffened elements under stress gradient***

The effective width of an unstiffened element subject to bending or combined bending and axial load should be determined in accordance with **4.5.2**. K factors for plain channel section elements are given in annex B.

5.2.2.6 ***Elements under uniform tension***

The effective area should be taken as the whole area of the element minus any allowance for holes.

5.2.2.7 ***Lips***

In the calculation of the section modulus the area of all inward lips should be included, but outward lips should be treated as follows:

a) where an outward lip adjoins a compression flange and has a flat width not greater than $10t(280/Y_s)^{1/2}$ its whole area should be included;

b) where an outward lip adjoins a compression flange and has a flat width exceeding $10t(280/Y_s)^{1/2}$ it should not be included;

c) where an outward lip adjoins a tension flange it should be included;

d) for a lip under uniform compression see **4.5.1**.

where

t is the compression element thickness;

Y_s is the yield strength.

5.2.3 Utilization of plastic bending capacity

5.2.3.1 ***General***

For plastic cross-sections, classified in **5.2.3.2** and **5.2.3.3**, there is a degree of post-compressive yield capacity which may be utilized in determining the moment capacity, providing that:

a) the member is not subject to eccentric loading causing significant twisting and is laterally stable;

b) the effects of cold forming are not included in determining the material yield stress;

c) the depth to thickness ratio of that portion of the web subject to compressive stresses is less than $30\,(280/Y_s)^{1/2}$;

d) the maximum applied shear force is less than $0.35DtY_s$;

e) the angle between any web and the plane of applied loading does not exceed 20°;

f) the ratio of ultimate strength to yield strength of the material is not less than 1.08 and the total elongation at failure in a tensile test is not less than 12 % over an 80 mm gauge length, or 15 % over a gauge length of $5.65\sqrt{S_o}$

where

Y_s is the yield strength;

D is the overall web depth;

t is the compression element thickness;

S_o is the original cross-sectional area of the parallel length in a tensile test specimen (as defined in BS EN 10002-1.)

5.2.3.2 ***Sections with stiffened compression elements***

Maximum moments are as follows:

a) for $\dfrac{b}{t} \leq 25\left(\dfrac{280}{Y_s}\right)^{1/2}$ (plastic cross-sections)

$M'_c = M_p$

b) for $\dfrac{b}{t} \geq 40\left(\dfrac{280}{Y_s}\right)^{1/2}$

$M'_c = M_c$

c) for $25\left(\dfrac{280}{Y_s}\right)^{1/2} \leq \dfrac{b}{t} \leq 40\left(\dfrac{280}{Y_s}\right)^{1/2}$, M'_c may be obtained by linear interpolation between a) and b),

i.e. $M'_c = M_c + \dfrac{40(280/Y_s)^{1/2} - b/t}{15(280/Y_s)^{1/2}}\,(M_p - M_c)$

where

b is the flat width of the compression element;

t is the compression element thickness;

Y_s is the yield strength;

M'_c is the maximum design moment capacity;

M_p is the fully plastic moment for the full section equal to Y_sS where S is the plastic modulus of the section;

M_c is the moment capacity of the section determined in accordance with **5.2.2**.

5.2.3.3 ***Sections with unstiffened compression elements***

Maximum moments are as follows:

a) for $\dfrac{b}{t} \leq 8\left(\dfrac{280}{Y_s}\right)^{1/2}$ (plastic cross-sections)

$M'_c = M_p$,

b) for $\dfrac{b}{t} \geq 13\left(\dfrac{280}{Y_s}\right)^{1/2}$

$M'_c = M_c$

c) for $8\left(\dfrac{280}{Y_s}\right)^{1/2} \leq \dfrac{b}{t} \leq 13\left(\dfrac{280}{Y_s}\right)^{1/2}$, M'_c may be obtained by linear interpolation between a) and b),

i.e. $M'_c = M_c + \dfrac{13(280/Y_s)^{1/2} - b/t}{5(280/Y_s)^{1/2}}\,(M_p - M_c)$

where the symbols are as defined in **5.2.3.2**.

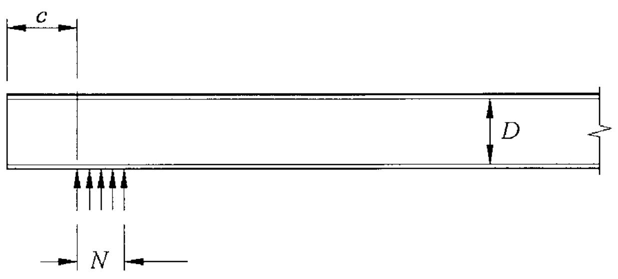
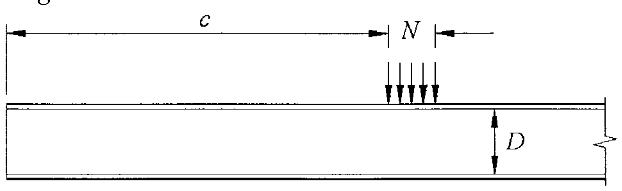
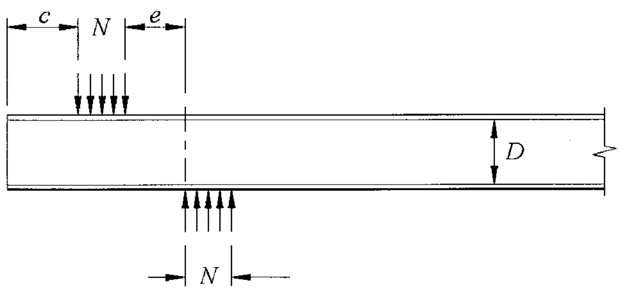
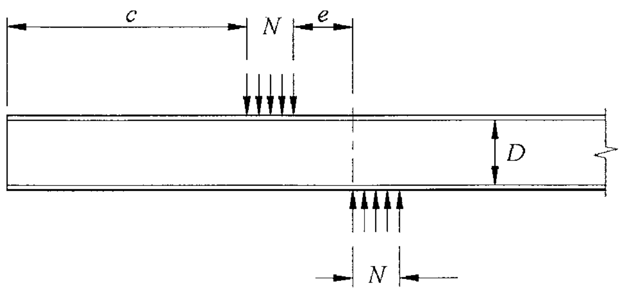
5.2.3.4 ***Utilization of plastic design principles***

The use of plastic limit analysis, with redistribution of moments following the attainment of full plastic moment capacity is permissible for plastic cross-sections which can sustain the fully plastic moment for the full section, M_p. For other sections plastic redistribution of moments should not be used in analysis but advantage may be taken of the increased moment capacity.

5.3 Web crushing

The resistance to local crushing of the webs of beams at support points or points of concentrated load should be evaluated using the equations given in Table 7 and Table 8. For built-up I-beams, or similar sections, the distance between the connector and beam flange should be kept as small as practicable.

Table 7 — Shapes having single thickness webs

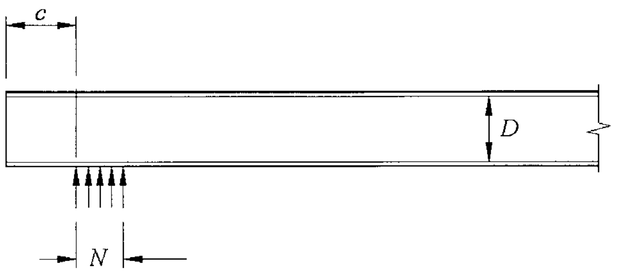
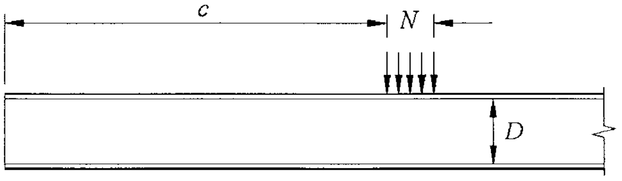
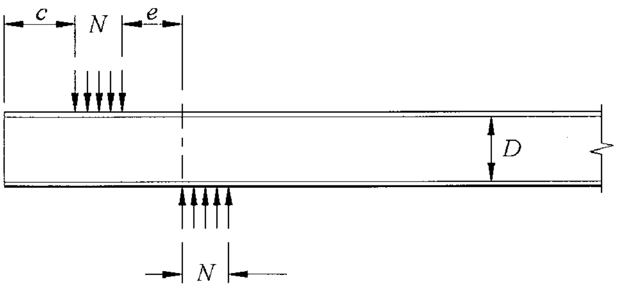
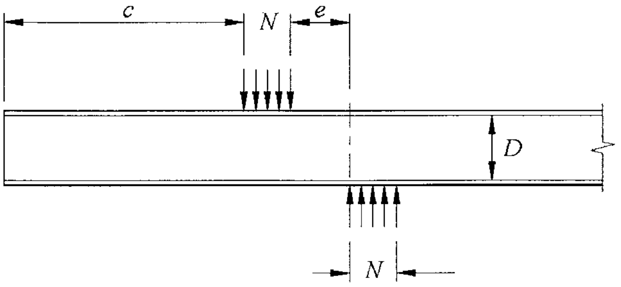
Type and position of loadings	Total web resistance, P_w
Single load or reaction  $c < 1.5D$. Load or reaction near or at free end	Stiffened flanges $P_w = t^2 k C_3 C_4 C_{12} \{2\,060 - 3.8(D/t)\} \times \{1 + 0.01(N/t)\}$ Unstiffened flanges[a] $P_w = t^2 k C_3 C_4 C_{12} \{1\,350 - 1.73(D/t)\} \times \{1 + 0.01(N/t)\}$
Single load or reaction  $c > 1.5D$. Load or reaction far from free end	Stiffened and unstiffened flanges[b] $P_w = t^2 k C_1 C_2 C_{12} \{3\,350 - 4.6(D/t)\} \times \{1 + 0.007(N/t)\}$
Two opposite loads or reactions $e < 1.5D$  $c \leq 1.5D$. Loads or reactions near or at free end	Stiffened and unstiffened flanges $P_w = t^2 k C_3 C_4 C_{12} \{1\,520 - 3.57(D/t)\} \times \{1 + 0.01(N/t)\}$
Two opposite loads or reactions $e < 1.5D$  $c > 1.5D$. Loads or reactions far from free end	Stiffened and unstiffened flanges $P_w = t^2 k C_1 C_2 C_{12} \{4\,800 - 14(D/t)\} \times \{1 + 0.0013(N/t)\}$

[a] When $N/t > 60$, the factor $\{1 + 0.01(N/t)\}$ may be increased to $\{0.71 + 0.015(N/t)\}$

[b] When $N/t > 60$, the factor $\{1 + 0.007(N/t)\}$ may be increased to $\{0.75 + 0.011(N/t)\}$

NOTE In this table P_w represents the total load or reaction for one solid web connecting top and bottom flanges. For beams with two or more such adjacent webs P_w should be determined for each individual web and the results added to obtain the total crushing load.

Table 8 — I-beams and beams with restraint against web rotation

Type and position of loadings	Total web resistance, P_w
Single load or reaction  $c \leq 1.5D$. Load or reaction near or at free end	Stiffened and unstiffened flanges $P_w = t^2 C_7 p_y \{8.8 + 1.1(N/t)^{1/2}\}$
Single load or reaction  $c > 1.5D$. Load or reaction far from free end	Stiffened and unstiffened flanges $P_w = t^2 C_5 C_6 p_y \{13.2 + 2.87(N/t)^{1/2}\}$
Two opposite loads or reactions $e < 1.5D$  $c \leq 1.5D$. Loads or reactions near or at free end	Stiffened and unstiffened flanges $P_w = t^2 C_{10} C_{11} p_y \{8.8 + 1.1(N/t)^{1/2}\}$
Two opposite loads or reactions $e < 1.5D$  $c > 1.5D$. Loads or reactions far from free end	Stiffened and unstiffened flanges $P_w = t^2 C_8 C_9 p_y \{13.2 + 2.87(N/t)^{1/2}\}$

NOTE In this table P_w represents the total load or reaction for one solid web connecting top and bottom flanges. For beams with two or more such adjacent webs P_w should be determined for each individual web and the results added to obtain the total crushing load.

The equations in Table 7 and Table 8 apply to the following.

Beams with: $D/t \leq 200$
$r/t \leq 6$

In these relationships and the equations in Table 7 and Table 8:

D is the overall web depth in millimetres (mm);
t is the web thickness in millimetres (mm);
r is the inside bend radius in millimetres (mm);
N is the actual length of bearing in millimetres (mm); for the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N should be taken;
P_w is the concentrated load resistance of a single web in newtons (N);
c is the distance from the end of the beam to the load or the reaction in millimetres (mm);
C is a constant with the following values:

$C_1 = (1.22 - 0.22k)$
$C_2 = (1.06 - 0.06r/t) \leq 1.0$
$C_3 = (1.33 - 0.33k)$
$C_4 = (1.15 - 0.15r/t) \leq 1.0$ but not less than 0.50
$C_5 = (1.49 - 0.53k) \geq 0.6$
$C_6 = (0.88 + 0.12m)$
$C_7 = 1 + D/t / 750$ when $D/t < 150$;
$C_7 = 1.20$ when $D/t > 150$
$C_8 = 1/k$, when $D/t < 66.5$;
$C_8 = (1.10 - D/t / 665)/k$ when $D/t > 66.5$
$C_9 = (0.82 + 0.15m)$
$C_{10} = (0.98 - D/t / 865)/k$
$C_{11} = (0.64 + 0.31m)$
$C_{12} = 0.7 + 0.3\ (\theta/90)^2$

where

$k = p_y/228$ where p_y is the design strength in newtons per square millimetre(N/mm^2);
$m = t/1.9$;
θ is the angle in degrees between plane of web and plane of bearing surface, where $45° \leq \theta \leq 90°$.

5.4 Shear in webs

5.4.1 General

Separate calculations should be made for maximum and average shear stresses.

5.4.2 Maximum shear stress

The maximum shear stress, calculated on the basis of an accepted method of elastic analysis, should not be greater than $0.7p_y$, where p_y is the design strength.

5.4.3 Average shear stress

The average shear stress should not exceed the lesser of the shear yield strength, p_v or the shear buckling strength, q_{cr}, obtained as follows:

$$p_v = 0.6p_y$$

$$q_{cr} = \left(\frac{1\,000t}{D}\right)^2 \text{ N/mm}^2$$

where

p_y is the design strength in newtons per square millimetre (N/mm^2);
t is the web thickness in millimetres (mm);
D is the web depth in millimetres (mm);
S_o is the original cross-sectional area of the parallel length in a tensile test specimen (as defined in BS EN 10002-1.). In the case of intermediately stiffened webs, where the stiffener rigidity conforms to **4.7.1**, D may be taken as the flat width of the largest sub-element.

5.5 Combined effects

5.5.1 Combined bending and web crushing

Flat webs of sections subject to a combination of bending and concentrated load or reaction should be designed to satisfy the following relationships at the limit state:

a) sections having single-thickness webs:

$$1.2\left(\frac{F_w}{P_w}\right) + \left(\frac{M}{M_c}\right) \leq 1.5$$

$$\frac{F_w}{P_w} \leq 1$$

$$\frac{M}{M_c} \leq 1$$

b) I-beams made from two channels connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web:

$$1.1\left(\frac{F_w}{P_w}\right) + \left(\frac{M}{M_c}\right) \leq 1.5$$

$$\frac{F_w}{P_w} \leq 1$$

$$\frac{M}{M_c} \leq 1$$

where

F_w is the concentrated web load or reaction;

P_w is the concentrated load resistance determined in accordance with **5.3**;

M is the applied bending moment at the point of application of F_w;

M_c is the moment capacity determined in accordance with **5.2.2**.

5.5.2 Combined bending and shear

For beam webs subjected to both bending and shear stresses the member should be designed to satisfy the following relationship:

$$\left(\frac{F_v}{P_v}\right)^2 + \left(\frac{M}{M_c}\right)^2 \leq 1$$

where

F_v is the shear force;

P_v is the shear capacity or shear buckling resistance determined in accordance with **5.4.3** and is equal to $p_v Dt$ or $q_{cr} Dt$ whichever is the lesser;

M is the value of the bending moment acting at the same section as F_v;

M_c is the moment capacity determined in accordance with **5.2.2**.

5.6 Lateral buckling

5.6.1 General

Lateral buckling, also known as lateral-torsional buckling, will not occur if the beam is adequately restrained against lateral movement and twisting. Restraints may be deemed to provide adequate strength if they are capable of resisting a lateral force of 3 % of the maximum force in the compression flange or chord, divided equally between the points of restraint, subject to a minimum force of 1 % per restraint.

Where several members share a common restraint the total force may be taken as the sum of the largest three.

A member composed of two sections in contact or separated back-to-back by a distance not greater than that required for an end gusset connection, may be designed as a single integral member with an effective slenderness as defined in **5.6.3**, provided that the main components are of a similar cross-section with their corresponding rectangular axes aligned and provided that they are interconnected with structural fasteners or by metal-arc welding. The spacing and strength of fasteners should be as recommended in **8.6.2**.

5.6.2 Buckling resistance moment

5.6.2.1 *Determination of* M_b

The buckling resistance moment, M_b, may be calculated as follows:

$$M_b = \frac{M_E M_Y}{\phi_B + \sqrt{\phi_B^2 - M_E M_Y}} \leq M_c$$

where

$$\phi_B = \frac{M_Y + (1 + \eta)M_E}{2}$$

M_c is the moment capacity of the section determined in accordance with **5.2.2**;

M_Y is the yield moment of the section, that is, the product of the design strength, p_y, and the elastic modulus of the gross cross-section with respect to the compression flange, Z_c;

M_E is the elastic lateral buckling resistance moment determined in accordance with **5.6.2.2**;

η is the Perry coefficient, such that:

when $L_E/r_y < 40C_b$

$\eta = 0$

when $L_E/r_y > 40C_b$

$$\eta = 0.002\left(\frac{L_E}{r_y} - 40C_b\right)$$

where

L_E is the effective length determined in accordance with **5.6.3**;

r_y is the radius of gyration of the section about the y axis;

C_b is a coefficient which may be conservatively assumed to be unity, or can be calculated using

$$C_b = 1.75 - 1.05\beta + 0.3\beta^2 \leq 2.3$$

where

β is the ratio of the smaller end moment to the larger end moment M in the unbraced length of a beam. β is taken as positive in the case of single curvature bending and negative in the case of double curvature bending as shown in Figure 3. When the bending moment at any point within the span is greater than M, C_b should be taken as unity.

When this value of M_b exceeds M_c, the ultimate moment should be taken as M_c.

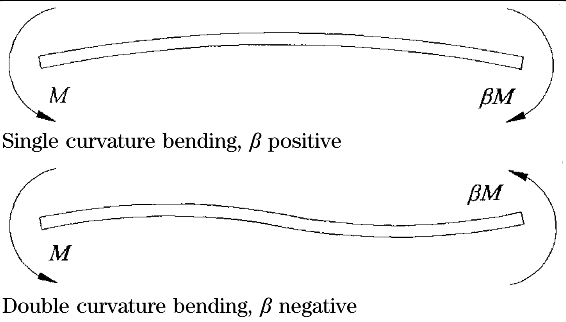


Figure 3 — Single and double curvature bending

5.6.2.2 *Determination of* M_E

The elastic lateral buckling resistance moment, M_E, for sections loaded effectively through the shear centre should be determined as follows:

a) for equal flange I-section and symmetrical channel section beams bent in the plane of the web (in this expression, for simplicity, the term within the braces, {}, may conservatively be taken as 1):

$$M_E = \frac{\pi^2 AED}{2(L_E/r_y)^2} C_b \left\{1 + \frac{1}{20}\left(\frac{L_E}{r_y}\frac{t}{D}\right)^2\right\}^{1/2}$$

NOTE If the channel section is torsionally restrained at the load and support points, it may be considered to be loaded through the shear centre for the purposes of this subclause.

b) for Z-section beams bent in the plane of the web (in this expression, for simplicity, the term within the braces. {}, may conservatively be taken as 1):

$$M_E = \frac{\pi^2 AED}{4(L_E/r_y)^2} C_b \left\{1 + \frac{1}{20}\left(\frac{L_E}{r_y}\frac{t}{D}\right)^2\right\}^{1/2}$$

c) for T-section beams bent in the plane of symmetry such that the flanges are in compression:

$$M_E = \frac{\pi^2 AED}{2(L_E/r_y)^2} C_b C_T \left[\left\{1 + \frac{1}{20}\left(\frac{L_E t}{C_T r_y D}\right)^2\right\}^{1/2} + 1\right]$$

d) for T-section beams bent in the plane of symmetry such that the flanges are in tension:

$$M_E = \frac{\pi^2 AED}{2(L_E/r_y)^2} C_b C_T \left[\left\{1 + \frac{1}{20}\left(\frac{L_E t}{C_T r_y D}\right)^2\right\}^{1/2} - 1\right]$$

where

A is the cross-sectional area of the beam;

E is the modulus of elasticity;

D is the overall web depth

C_T is a constant given by:

$$C_T = \frac{1 + 1.5B/D - 0.25(B/D)^3}{1 + 2B/D}$$

where

B is the total width of the flanges of a T-section;

t is the material thickness;

C_b, L_E and r_y are as defined in **5.6.2.1.**

If a negative value of C_T is obtained the section may be regarded as having adequate lateral restraint.

5.6.3 Effective lengths

When considering lateral buckling the effective length, L_E, of a member should be taken as follows.

a) Where a beam is restrained at the ends only, the effective length should be taken as follows (see Figure 4):

1) for beams not restrained against rotation in the θ_1, θ_2 or θ_3 directions, $L_E = 1.1L$;

2) for beams restrained against torsional rotation θ_1, only, $L_E = 0.9L$;

3) for beams restrained against torsional rotation θ_1, and rotation about the minor axis θ_2, $L_E = 0.8L$;

4) for beams completely restrained against rotation in any direction, $L_E = 0.7L$;

where L is the span between supports.

b) Where a beam is restrained at intervals by substantial connections to other steel members and is part of a fully framed structure, L_E should be taken as 0.8 times the distance between restraints. Where the beam is restrained at intervals by less substantial connections, L_E should be taken as 0.9 times the distance between restraints.

c) Where the length considered is the length between a support and a restraint, the factor L_E/L should be taken as the mean of the values obtained from a) and b).

d) In the case of compound sections composed of two channels back-to-back designed as a single integral member and connected in accordance with **8.6**, the effective slenderness of the compound beam (L_E/r_y) should be calculated as follows:

$$\frac{L_E}{r_y} = \sqrt{\left(\frac{L_E}{r_I}\right)^2 + \left(\frac{s}{r_{cy}}\right)^2} \text{ but not less than } 1.4\, s/r_{cy}.$$

where

L_E is the effective length of the compound member;

r_y is the radius of gyration of the compound section about the axis parallel to the webs allowing for the two elements acting as a single integral member;

r_I is the radius of gyration of the compound section about the axis parallel to the webs, based on normal geometric properties;

s is the longitudinal spacing between adjacent fasteners or welds connecting the two sections together;

r_{cy} is the minimum radius of gyration of one channel section.

The local slenderness of an individual channel, s/r_{cy} should not exceed 50. The strength and the maximum spacing of interconnections should be as recommended in **8.6.2.**

e) For conditions not covered in a) to d), reference should be made to BS 5950-1.

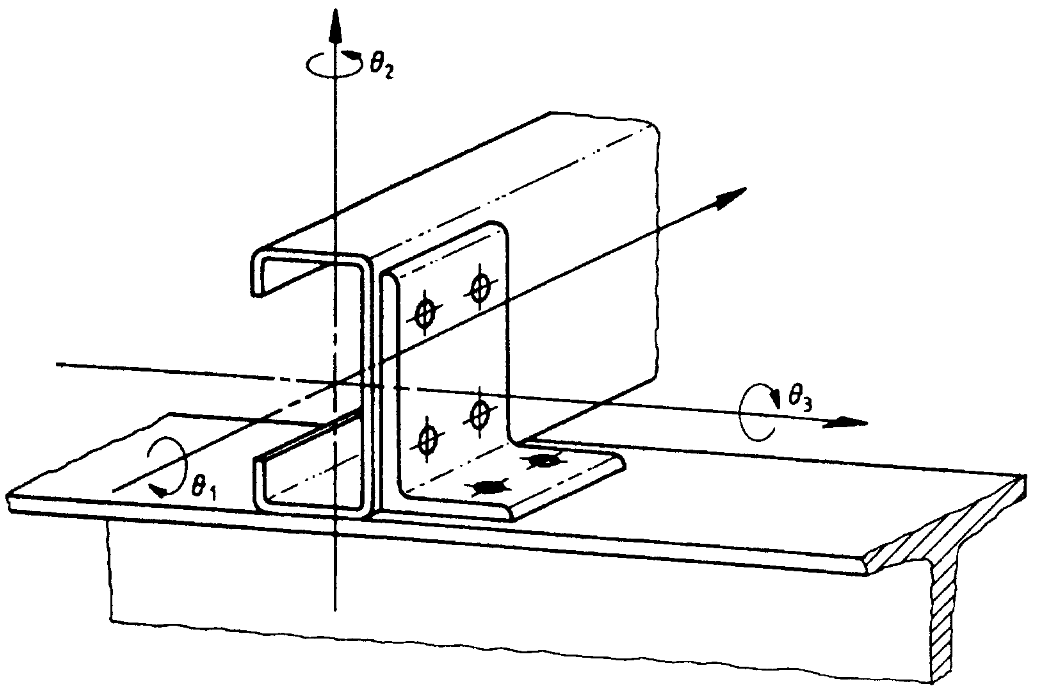


Figure 4 — Restraint condition, for lateral buckling

5.6.4 Destabilizing loads

Destabilizing load conditions exist when a load is applied to a beam and both the load and the beam are free to deflect laterally (and possibly rotationally also) relative to the centroid of the beam. In such cases, the effective lengths given in **5.6.3** should be increased by 20 %.

5.7 Deflections

The recommended deflection limitations for beams are given in **2.4.2**.

The deflection, in the plane of loading, of a laterally stable beam or one which is adequately restrained against twisting, and which does not utilize the plastic capacity, may be calculated from a) or b), whichever is applicable:

a) for M or $M_c \leq M_{cr}$, the full cross-section should be used in evaluating the second moment of area and the deflection calculated using simple beam theory;

b) for $M_{cr} < M \leq M_c$, either M or Δ is determined from a specified value of the other quantity using the equation:

$$\frac{M - M_{cr}}{M_c - M_{cr}} = \frac{\Delta - \Delta_{cr}}{\Delta_c - \Delta_{cr}}$$

where

M is the bending moment for a given loading system;

Δ is the deflection for the given loading system;

M_c is the moment capacity determined in accordance with **5.2.2**;

Δ_c is the deflection corresponding to M_c calculated using the reduced cross-section;

M_{cr} is the critical bending moment given by:

$$M_{cr} = 0.59EK(t/b)^2 Z_c$$

where

K is the buckling coefficient of the compression flange; values of K for different sections and conditions are given in annex B;

t is the thickness of the compression flange;

Z_c is the elastic modulus of the gross cross-section with respect to the compression flange;

b is the flat width of the compression flange;

Δ_{cr} is the deflection of the beam corresponding to M_{cr} calculated using the full cross-section.

5.8 Flange curling

For flexural members with stiffened elements as flanges where the width to thickness ratio, B/t, is greater than 250, substantial flange curling, or movement of the flange towards the neutral axis, may occur. Evaluation of the amount of curling may be carried out using the following:

$$u = 2\frac{f_a^2 B_f^4}{E^2 t^2 y}$$

where

u is the deflection of the centre of the flange towards the neutral axis;

f_a is the average stress in the flange;

B_f is half the overall flange width for a stiffened element;

E is the modulus of elasticity;

t is the flange thickness;

y is the distance of the flange from the neutral axis.

This equation applies to both compression and tension flanges with or without stiffeners. If the stress in the flange has been calculated on the basis of an effective width, b_{eff}, then f_a should be obtained by multiplying the stress on the effective width by the ratio of the effective flange area to the gross flange area.

If the amount of curling, u, is greater than 5 % of the depth of the cross-section then steps should be taken either to reduce this to 5 % of the depth or to take the effects of the curling into account in evaluation of the load bearing capacity.

5.9 Effects of torsion

5.9.1 General

Where possible for open sections, the effects of torsion should be avoided either by the provision of restraints designed to resist twisting or by ensuring that lateral loads are applied through the shear centre.

5.9.2 Direct stresses due to combined bending and torsion

For beams subjected to combined bending and torsion the maximum stress due to both effects combined, determined on the basis of the full unreduced cross-section and the unfactored loads, should not exceed the design strength, p_y.

5.9.3 Angle of twist

The angle of twist of a beam which is subject to torsion should not be so great as to change significantly the shape of the cross-section or its capability to resist bending.

Section 6. Members in compression

6.1 General

6.1.1 Introduction

In the analysis of members in compression, due account should be taken of the effects of local buckling on the behaviour of such members. These effects may be taken into account by considering the member to have an effectively reduced cross-sectional area in resisting compression.

6.1.2 Effective cross-sectional area

The effective cross-sectional area of a compression member may be calculated by summing the effective areas of the individual elements obtained following calculations made in accordance with section **4**. The relative cross-sectional area can be defined by a factor Q, such that:

$$Q = \frac{\text{Effective cross-sectional area}}{\text{Gross cross-sectional area}} = \frac{A_{\text{eff}}}{A}$$

In evaluating the effective cross-sectional area, the effective widths for each element should be determined in accordance with **4.3**, with f_c replaced by the design strength, p_y. The minimum values of the local buckling coefficient, K, to be used in determination of the effective width of an element may be taken as:

for a stiffened element, $K = 4$;

for an unstiffened element, $K = 0.425$.

6.1.3 Use of enhanced *K* values

Where it can be shown that higher K factors are applicable to individual elements of a section, such higher factors may be used in the evaluation of the effective width, b_{eff}, for these elements.

Enhanced values of K which may be used for some sections are given in annex B.

6.2 Flexural buckling

6.2.1 Effective lengths

The effective length of a member in compression should be established in accordance with Table 9 or on the basis of good engineering practice.

6.2.2 Maximum slenderness

The slenderness ratio should be taken as the effective length, L_E, divided by the radius of gyration about the relevant axis, r, except as given in **6.2.5** for back-to-back members.

The maximum values of the slenderness ratio L_E/r should not exceed the following:

for members resisting loads other than wind loads:	180;
for members resisting self weight and wind loads only:	250;
for any member acting normally as a tie but subject to reversal of stress resulting from the action of wind:	350.

6.2.3 Ultimate loads

For sections symmetrical about both principal axes or closed cross-sections which are not subject to torsional flexural buckling, or are braced against twisting, the buckling resistance under axial load, P_c, may be obtained from the following:

$$P_c = \frac{P_E P_{cs}}{\phi + \sqrt{\phi^2 - P_E P_{cs}}}$$

where

$$\phi = \frac{P_{cs} + (1 + \eta)P_E}{2}$$

P_{cs} is the short strut capacity and is equal to $A_{\text{eff}} p_y$

where

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

p_y is the design strength;

P_E is the minimum elastic flexural buckling load and is equal to:

$$\frac{\pi^2 EI}{L_E^2}$$

where

E is the modulus of elasticity;

I is the second moment of area of the cross-section about the critical axis;

L_E is the effective length of the member about the critical axis;

η is the Perry coefficient, such that:

for $L_E/r \leq 20$, $\eta = 0$

for $L_E/r > 20$, $\eta = 0.002(L_E/r - 20)$

where

r is the radius of gyration of the gross cross-section corresponding to P_E.

Alternatively the value of P_c may be obtained using the values in Table 10 by multiplying the strength obtained in the table by the full cross-sectional area of the member.

Resistance to torsional flexural buckling may also be determined using these expressions modified in accordance with **6.3.2**.

6.2.4 Singly symmetrical sections

For sections symmetrical about a single axis and which are not subject to torsional flexural buckling, or which are braced against twisting, the effects of movement of the effective neutral axis should be taken into account in evaluation of the maximum load.

The movement of the effective neutral axis may be calculated by determining the neutral axis position of the gross cross-section and that of the effective cross-section. In evaluation of the neutral axis position of the effective cross-section the effective portions

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	$1.0L$
Effectively held in position at both ends and restrained in direction at one end	$0.85L$
Effectively held in position and partially restrained in direction at both ends	$0.85L$
Effectively held in position and restrained in direction at both ends	$0.7L$
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	$1.2L$
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but not held in	$1.5L$
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	$2.0L$

should be positioned as detailed in **4.1** and shown in Figure 5.

The buckling resistance, P'_c, may then be evaluated from:

$$P'_c = \frac{M_c P_c}{(M_c + P_c e_s)}$$

where

M_c is the moment capacity determined in accordance with **5.2.2**, having due regard to the direction of moment application as indicated in Figure 5;

P_c is the buckling resistance under axial load determined in accordance with **6.2.3**;

e_s is the distance between the geometric neutral axis of the gross cross-section and that of the effective cross-section as indicated in Figure 5.

6.2.5 Compound sections composed of channels back-to-back

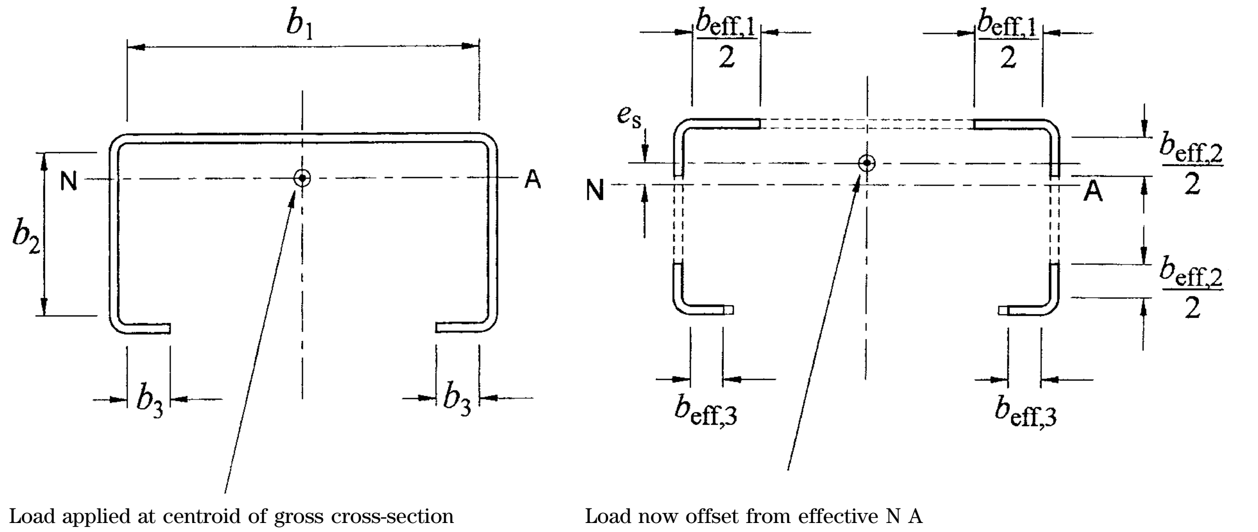
A compound section composed of two sections in contact or separated back-to-back by a distance not greater than that required for an end gusset connection, may be designed as a single integral member subject to the following conditions:

a) The channels should be of a similar cross-section with their corresponding rectangular axes aligned.

b) The main components should be interconnected with structural fasteners or welds in accordance with **8.6**.

c) The effective slenderness of the compound section (L_E/r_y), about the axis parallel to the webs of the channels should be calculated as follows:

$$\frac{L_E}{r_y} = \sqrt{\left(\frac{L_E}{r_I}\right)^2 + \left(\frac{s}{r_{cy}}\right)^2} \text{ but not less than } 1.4s/r_{cy}.$$



a) Gross cross-section

b) Effective cross-section

Figure 5 — Compression of singly symmetrical section

where

L_E is the effective length of the compound member;

r_y is the radius of gyration of the compound section about the axis parallel to the webs allowing for the two elements acting as a single integral member;

r_I is the radius of gyration of the complete compound section about the axis parallel to the webs, based on normal geometric properties;

s is the longitudinal spacing between adjacent fasteners or welds connecting the two sections together;

r_{cy} is the minimum radius of gyration of one channel section.

The local slenderness of an individual channel, s/r_{cy} should not exceed 50.

d) The strength and the maximum spacing of interconnections should be as recommended in **8.6.2.**

6.3 Torsional flexural buckling

6.3.1 General

The design procedure given in **6.3.2** applies only to struts which are braced in both the x and y directions at the ends of the strut or at points of support.

6.3.2 Sections with at least one axis of symmetry (x axis)

For members which have at least one axis of symmetry, taken as the x axis, and which are subject to torsional flexural buckling, design according to **6.2**, and the values given in Table 10 may be used provided that a factored slenderness ratio, $\alpha L_E/r$, is used in place of the actual slenderness ratio. Values of α for a number of cross-sectional shapes are given in annex C. For other cross-sections values of α may be determined as follows:

for $P_E \le P_{TF}$ $\quad \alpha = 1$

for $P_E > P_{TF}$ $\quad \alpha = \left(\frac{P_E}{P_{TF}}\right)^{1/2}$

where

P_E is the elastic flexural buckling load for a column given by:

$$\frac{\pi^2 EI}{L_E^2}$$

where

E is the modulus of elasticity;

I is the second moment of area about the y axis;

L_E is the effective length corresponding to the minimum radius of gyration;

P_{TF} is the torsional flexural buckling load of a column given by:

$$\frac{1}{2\beta}\left[(P_{Ex} + P_T) - \{(P_{Ex} + P_T)^2 - 4\beta P_{Ex} P_T\}^{1/2}\right]$$

where

P_{Ex} is the elastic flexural buckling load for a column about the x axis given by:

$$P_{Ex} = \frac{\pi^2 EI_x}{L_E^2}$$

P_T is the torsional buckling load of a column given by:

$$P_T = \frac{1}{r_o^2}\left(GJ + 2\frac{\pi^2 EC_w}{L_E^2}\right)$$

β is a constant given by:

$$\beta = 1 - \left(\frac{x_o}{r_o}\right)^2$$

In these equations:

r_o is the polar radius of gyration about the shear centre given by:

$$r_o = (r_x^2 + r_y^2 + x_o^2)^{1/2}$$

r_x, r_y are the radii of gyration about the x and y axes;

G is the shear modulus;

x_o is the distance from the shear centre to the centroid measured along the x axis;

J is the St Venant torsion constant which may be taken as the summation of $bt^3/3$ for all elements, where b is the element flat width and t is the thickness;

I_x is the second moment of area about the x axis;

C_w is the warping constant for the cross-section.

Approximate equations for C_w and x_o for some commonly used sections are given in annex D.

6.3.3 Non-symmetrical sections

For non-symmetrical cross-sections the maximum load should be determined either by analysis or by testing in accordance with section **10**.

Table 10 — Compressive strength, p_c

L_E/r	Compressive strength, p_c N/mm^2										
	$Q\left(\frac{Y_s}{280}\right)$										
	0.25	**0.30**	**0.35**	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**
0	70.0	84.0	98.0	112.0	126.0	140.0	154.0	168.0	182.0	196.0	210.0
10	70.0	84.0	98.0	112.0	126.0	140.0	154.0	168.0	182.0	196.0	210.0
20	70.0	84.0	98.0	112.0	126.0	140.0	154.0	168.0	182.0	196.0	210.0
30	68.6	82.3	96.0	110.0	123.0	137.0	151.0	165.0	178.0	192.0	206.0
40	67.2	80.6	93.9	107.0	121.0	134.0	147.0	161.0	174.0	187.0	201.0
50	65.7	78.8	91.8	105.0	118.0	131.0	144.0	156.0	169.0	182.0	195.0
60	64.2	76.9	89.5	102.0	115.0	127.0	139.0	151.0	164.0	176.0	188.0
70	62.6	74.9	87.0	99.0	111.0	123.0	134.0	146.0	157.0	168.0	179.0
80	60.9	72.7	84.2	95.6	107.0	118.0	128.0	138.0	148.0	158.0	167.0
90	59.1	70.3	81.2	91.7	102.0	112.0	121.0	130.0	139.0	146.0	154.0
100	57.2	67.7	77.8	87.4	96.5	105.0	113.0	120.0	127.0	133.0	139.0
110	55.2	64.9	74.1	82.6	90.5	97.7	104.0	110.0	115.0	120.0	124.0
120	53.0	61.9	70.1	77.5	84.1	90.0	95.1	99.6	104.0	107.0	110.0
130	50.7	58.7	65.8	72.1	77.6	82.2	86.2	89.6	92.5	95.0	97.1
140	48.3	55.4	61.5	66.7	71.1	74.8	77.9	80.5	82.6	84.5	86.0
150	45.8	52.0	57.2	61.5	65.0	67.9	70.3	72.3	73.9	75.3	76.5
160	43.3	48.6	53.0	56.5	59.3	61.6	63.5	65.1	66.3	67.4	68.4
170	40.7	45.4	49.0	51.9	54.2	56.0	57.5	58.7	59.7	60.6	61.3
180	38.3	42.2	45.3	47.6	49.5	51.0	52.2	53.2	54.0	54.7	55.3
190	35.9	39.3	41.8	43.8	45.3	46.6	47.5	48.3	49.0	49.6	50.1
200	33.7	36.6	38.7	40.3	41.6	42.6	43.4	44.1	44.7	45.1	45.5
210	31.6	34.0	35.8	37.2	38.3	39.1	39.8	40.4	40.8	41.2	41.6
220	29.6	31.7	33.2	34.4	35.3	36.0	36.6	37.1	37.5	37.8	38.1
230	27.7	29.5	30.9	31.9	32.6	33.2	33.7	34.1	34.5	34.8	35.0
240	26.0	27.6	28.7	29.6	30.2	30.8	31.2	31.6	31.8	32.1	32.3
250	24.4	25.8	26.8	27.5	28.1	28.6	28.9	29.2	29.5	29.7	29.9

Table 10 — Compressive strength, p_c *(continued)*

L_E/r	Compressive strength, p_c N/mm^2										
	$Q\left(\frac{Y_s}{280}\right)$										
	0.80	**0.85**	**0.90**	**0.95**	**1.00**	**1.05**	**1.10**	**1.15**	**1.20**	**1.25**	**1.30**
0	224.0	238.0	252.0	266.0	280.0	294.0	308.0	322.0	336.0	350.0	364.0
10	224.0	238.0	252.0	266.0	280.0	294.0	308.0	322.0	336.0	350.0	364.0
20	224.0	238.0	252.0	266.0	280.0	294.0	308.0	322.0	336.0	350.0	364.0
30	219.0	233.0	247.0	260.0	274.0	287.0	301.0	315.0	328.0	342.0	356.0
40	214.0	227.0	240.0	253.0	267.0	280.0	293.0	306.0	319.0	332.0	345.0
50	207.0	220.0	232.0	245.0	257.0	270.0	282.0	294.0	306.0	319.0	331.0
60	199.0	211.0	223.0	234.0	245.0	256.0	267.0	278.0	289.0	299.0	309.0
70	189.0	199.0	210.0	219.0	229.0	238.0	247.0	255.0	263.0	271.0	279.0
80	176.0	185.0	193.0	200.0	208.0	214.0	221.0	226.0	232.0	237.0	241.0
90	161.0	167.0	173.0	179.0	183.0	188.0	192.0	196.0	199.0	202.0	205.0
100	144.0	149.0	153.0	156.0	160.0	162.0	165.0	167.0	169.0	171.0	173.0
110	128.0	131.0	133.0	136.0	138.0	140.0	142.0	143.0	145.0	146.0	147.0
120	112.0	115.0	116.0	118.0	120.0	121.0	122.0	123.0	124.0	125.0	126.0
130	98.9	101.0	102.0	103.0	104.0	105.0	106.0	107.0	107.0	108.0	109.0
140	87.4	88.5	89.6	90.5	91.3	92.0	92.6	93.1	93.7	94.1	94.5
150	77.5	78.4	79.2	79.9	80.5	81.0	81.5	81.9	82.3	82.7	83.0
160	69.2	69.8	70.4	71.0	71.5	71.9	72.3	72.6	72.9	73.2	73.4
170	62.0	62.5	63.0	63.4	63.8	64.1	64.5	64.7	65.0	65.2	65.4
180	55.8	56.3	56.7	57.0	57.3	57.6	57.8	58.1	58.3	58.4	58.6
190	50.5	50.9	51.2	51.5	51.7	52.0	52.2	52.3	52.5	52.7	52.8
200	45.9	46.2	46.5	46.7	46.9	47.1	47.3	47.4	47.6	47.7	47.8
210	41.9	42.1	42.4	42.6	42.7	42.9	43.0	43.2	43.3	43.4	43.5
220	38.3	38.6	38.8	38.9	39.1	39.2	39.4	39.5	39.6	39.7	39.8
230	35.2	35.4	35.6	35.8	35.9	36.0	36.1	36.2	36.3	36.4	36.5
240	32.5	32.7	32.8	32.9	33.1	33.2	33.3	33.3	33.4	33.5	33.6
250	30.1	30.2	30.3	30.4	30.5	30.6	30.7	30.8	30.9	30.9	31.0

Table 10 — Compressive strength, p_c *(continued)*

L_E/r	Compressive strength, p_c N/mm^2										
	$Q\left(\frac{Y_s}{280}\right)$										
	1.35	**1.40**	**1.45**	**1.50**	**1.55**	**1.60**	**1.65**	**1.70**	**1.75**	**1.80**	**1.85**
0	378.0	392.0	406.0	420.0	434.0	448.0	462.0	476.0	490.0	504.0	518.0
10	378.0	392.0	406.0	420.0	434.0	448.0	462.0	476.0	490.0	504.0	518.0
20	378.0	392.0	406.0	420.0	434.0	448.0	462.0	476.0	490.0	504.0	518.0
30	369.0	383.0	396.0	410.0	424.0	437.0	451.0	464.0	478.0	491.0	505.0
40	358.0	371.0	384.0	397.0	410.0	423.0	435.0	448.0	461.0	474.0	486.0
50	342.0	354.0	366.0	378.0	389.0	400.0	412.0	423.0	434.0	445.0	456.0
60	319.0	329.0	338.0	347.0	356.0	365.0	373.0	381.0	389.0	396.0	404.0
70	286.0	292.0	298.0	304.0	310.0	315.0	320.0	325.0	329.0	333.0	336.0
80	246.0	250.0	253.0	257.0	260.0	263.0	265.0	268.0	270.0	272.0	274.0
90	207.0	210.0	212.0	214.0	215.0	217.0	218.0	220.0	221.0	222.0	223.0
100	175.0	176.0	177.0	178.0	180.0	181.0	181.0	182.0	183.0	184.0	184.0
110	148.0	149.0	150.0	150.0	151.0	152.0	152.0	153.0	154.0	154.0	154.0
120	126.0	127.0	128.0	128.0	129.0	129.0	130.0	130.0	130.0	131.0	131.0
130	109.0	110.0	110.0	110.0	111.0	111.0	111.0	112.0	112.0	112.0	112.0
140	94.9	95.3	95.6	95.9	96.2	96.4	96.7	96.9	97.1	97.3	97.5
150	83.3	83.6	83.8	84.1	84.3	84.5	84.7	84.9	85.0	85.2	85.3
160	73.7	73.9	74.1	74.3	74.5	74.6	74.8	74.9	75.0	75.2	75.3
170	65.6	65.8	65.9	66.1	66.2	66.4	66.5	66.6	66.7	66.8	66.9
180	58.8	58.9	59.0	59.2	59.3	59.4	59.5	59.6	59.7	59.8	59.8
190	52.9	53.1	53.2	53.3	53.4	53.5	53.5	53.6	53.7	53.8	53.8
200	47.9	48.0	48.1	48.2	48.3	48.4	48.4	48.5	48.6	48.6	48.7
210	43.6	43.7	43.8	43.8	43.9	44.0	44.0	44.1	44.2	44.2	44.3
220	39.8	39.9	40.0	40.0	40.1	40.2	40.2	40.3	40.3	40.3	40.4
230	36.5	36.6	36.7	36.7	36.8	36.8	36.9	36.9	36.9	37.0	37.0
240	33.6	33.7	33.7	33.8	33.8	33.9	33.9	33.9	34.0	34.0	34.0
250	31.0	31.1	31.1	31.2	31.2	31.3	31.3	31.3	31.4	31.4	31.4

6.4 Combined bending and compression

6.4.1 General

Compression members which are also subjected to bending should be checked for local capacity at the points of greatest bending moment and axial load (usually at the ends). These members should also be checked for overall buckling.

The checks detailed in **6.4.2** and **6.4.3** apply to members which have at least one axis of symmetry and which are not subject to torsional or torsional flexural buckling.

6.4.2 Local capacity check

The member should satisfy the following relationship:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

- F_c is the applied axial load;
- P_{cs} is the short strut capacity defined in **6.2.3**;
- M_x is the applied bending moment about the x axis;
- M_{cx} is the moment capacity in bending about the x axis in the absence of F_c and M_y, see **5.2.2** and **5.6**;
- M_y is the applied bending moment about the y axis;
- M_{cy} is the moment capacity in bending about the y axis in the absence of F_c and M_x, see **5.2.2** and **5.6**.

6.4.3 Overall buckling check

For beams not subject to lateral buckling, the following relationship should be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{C_{bx}\,M_{cx}\left(1 - \frac{F_c}{P_{Ex}}\right)} + \frac{M_y}{C_{by}\,M_{cy}\left(1 - \frac{F_c}{P_{Ey}}\right)} \leq 1$$

For beams subject to lateral buckling the following relationship should be satisfied:

$$\frac{F_c}{P_c} + \frac{M_x}{M_b} + \frac{M_y}{C_{by}\,M_{cy}\left(1 - \frac{F_c}{P_{Ey}}\right)} \leq 1$$

where

- P_c is the axial buckling resistance in the absence of moments, see **6.2.3**;
- P_{Ex} is the flexural buckling load in compression for bending about the x axis;
- P_{Ey} is the flexural buckling load in compression for bending about the y axis;
- C_{bx}, C_{by} are C_b factors as defined in **5.6.2.1** with regard to moment variation about the x and y axes respectively;
- M_b is the lateral buckling resistance moment about the x (major) axis as defined in **5.6.2**;

F_c, M_x, M_{cx}, M_y and M_{cy} are as defined in **6.4.2**.

The magnitudes of moments M_x and M_y, should take into account any moment induced by the change in neutral axis position of the effective cross-section caused by the axial load. In determination of C_{bx} and C_{by} the effects of change in the neutral axis position on the moment variation may be neglected.

Section 7. Members in tension

7.1 General

A member may be considered a simple tension member only if it is connected in such a way as to eliminate any resulting moments due to connection eccentricity. Where members are connected eccentrically to the axis of the member it is essential that the resulting moment is taken into account in accordance with **7.3** except for angles, channels and T-sections designed in accordance with **7.2** which may be treated as simple tension members.

7.2 Tensile capacity

7.2.1 General

The tensile capacity, P_t, of a member should be determined from:

$$P_t = A_e p_y$$

where

A_e is the effective net area of the section determined in accordance with **7.2.2** and **7.2.3** where appropriate, otherwise taken as equal to the net area, A_n, determined in accordance with **3.5.4**;

p_y is the design strength.

These rules may only be used when the width to thickness ratios of the unconnected elements are less than 20.

For width to thickness ratios greater than 20, the eccentricity of the force should be taken into account and the member designed in accordance with **7.3**.

7.2.2 Single angles, plain channels and T-sections

For single angle ties connected through one leg only, single plain channel sections connected only through the web and T-sections connected only through the flange, the effective area, A_e, should be taken as:

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

where

a_1 is the net sectional area of the connected leg;

a_2 is the gross sectional area of the unconnected leg or legs.

For double angle ties connected to one side of a gusset or section, the angles may be designed individually in this way.

7.2.3 Double angles, plain channels and T-sections

If the two components are parallel back-to-back and:

a) in contact or separated by solid packing pieces by a distance not exceeding the aggregate thickness of the parts;

b) connected by bolts or welding such that the slenderness of the individual components does not exceed 80;

then the effective area, A_e, may be taken as:

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

where

a_1 is the net sectional area of the connected parts;

a_2 is the gross sectional area of the unconnected parts.

7.3 Combined bending and tension

Members subject to both axial tension and bending stresses should be proportioned such that the following relationships are satisfied at the ultimate limit state:

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

and

$$\frac{M_x}{M_{cx}} \leq 1$$

and

$$\frac{M_y}{M_{cy}} \leq 1$$

where

F_t is the applied tensile load

P_t is the tensile capacity determined in accordance with **7.2.1**;

M_x, M_y, M_{cx} and M_{cy} are as defined in **6.4.2**.

NOTE For small values of F_t an overall lateral buckling check should be carried out.

Section 8. Connections

8.1 General recommendations

8.1.1 General

Connections should be designed using a realistic assumption of the distribution of internal forces, taking account of relative stiffnesses. This distribution should correspond with direct load paths through the elements of connections. It is essential that equilibrium with the external applied factored loads is maintained.

Ease of fabrication and erection should be considered in the design of joints and splices. Attention should be paid to clearances necessary for tightening of fasteners, welding procedures, subsequent inspection, surface treatment and maintenance.

The ductility of steel assists the distribution of forces generated within a joint. Therefore residual stresses and stresses due to tightening of fasteners and normal accuracy of fit-up need not usually be calculated.

8.1.2 Intersections

Usually, members meeting at a joint should be arranged with their centroidal axes meeting at a point. Where there is eccentricity at intersections, the members and connections should be designed to accommodate the resulting moments. In the case of bolted framing of angles and tees the setting-out lines of the bolts may be adopted instead of the centroidal axis.

8.1.3 Joints in simple construction

Joints between members in simple construction should be capable of transmitting the forces calculated in design and should be capable of accepting the resulting rotation (see **2.1.2.2**). They should not develop significant moments which adversely affect members of the structure.

8.1.4 Joints in rigid construction

Joints between members in rigid construction should be capable of transmitting the forces and moments calculated from the design method. For elastic design the rigidity of the joint should not be less than that of the members. For plastic design the moment capacity of a joint at a plastic hinge location should not be less than that of the member. In addition the joint should possess sufficient plastic rotation capacity (see **2.1.2.3**).

8.1.5 Joints in semi-rigid construction

Joints between members in semi-rigid construction should provide a predictable degree of interaction between members, as described in **2.1.2.4**. They should be capable of transmitting the restraint moments in addition to the other forces and moments at the joints. It is important that the connection is neither too rigid nor too flexible to fulfil accurately the assumptions made in design. If the design strength of the connection is less than that of the connected members, it should be demonstrated that the deformation capacity of the connection is sufficient for full redistribution of load up to the relevant limit state to take place.

8.1.6 Strength of individual fasteners

The strength of individual fasteners may be calculated in accordance with **8.1.7** or determined by testing in accordance with section **10**.

8.1.7 Forces in individual fasteners

Except for welds, the shear forces on individual fasteners in a connection may be assumed to be equal provided that the material is less than or equal to 4 mm thick. Otherwise the shear forces on individual fasteners should be calculated by elastic analysis.

8.1.8 Joints subject to vibration and/or load reversal

Where a connection is subject to impact or vibration, pretensioned friction grip fasteners, locking devices or welding should be used.

Where a connection is subject to reversal of stress (unless due solely to wind) or where for some special reason slipping of bolts is unacceptable, then pretensioned friction grip fasteners, fitted bolts or welding should be used.

8.1.9 Splices

8.1.9.1 *General*

Splices should be designed to hold the connected members in place and wherever practicable the members should be arranged so that the centroidal axis of the splice coincides with the centroidal axis of the members joined. If eccentricity is present then the resulting forces should be catered for.

8.1.9.2 *Splices in compression members*

Where the members are not prepared for tight contact in bearing, the splice should be designed to transmit all the moments and forces to which the member at that point is subjected. Where the members are prepared for full contact in bearing the splice should provide continuity of stiffness about both axes and resist any tension where bending is present.

The splice should be as near as possible to the ends of the member or points of inflexion. Where this is not achieved account should be taken of the moment induced by strut action.

8.1.9.3 *Splices in tension members*

The splice covers should be designed to transmit all the moments and forces to which the member at that point is subjected.

8.1.9.4 *Splices in beams*

Beam splices should be designed to transmit all the forces and moments in the member at that point and have adequate stiffness.

8.2 Bolted connections

8.2.1 General

The recommendations given in **8.2.2** to **8.2.9** are applicable to bolts in nominally 2 mm oversize clearance holes.

8.2.2 Bolt pitch and edge distances

8.2.2.1 ***Minimum pitch***

For material less than or equal to 4 mm thick, the distance between the centres of adjacent bolts in the line of stress should not be less than $3d$, where d is the diameter of the bolt. For material greater than 4 mm thick, the minimum pitch should not be less than $2.5d$.

8.2.2.2 ***Minimum edge and end distances***

The distance between the centre of a bolt and any edge of the connected member should not be less than $1.5d$.

8.2.3 Effective diameter and areas of bolts

Since threads can occur in the shear plane, the tensile stress area of the bolt, A_t, for resisting both shear and tension should be taken as the tensile stress area as specified in BS EN 20898-1. For bolts where the tensile stress area is not defined, A_t should be taken as the area at the bottom of the threads.

Where it can be shown that the threads do not occur in the shear plane, the shank area, A, may be used in the calculation of shear capacity.

In the calculation of thread length, allowance should be made for tolerance and thread run off.

8.2.4 Shear capacity of bolt

The shear capacity, P_s, of a bolt should be taken as:

$$P_s = p_s A_n$$

where

p_s is the shear strength obtained from Table 11;

A_n is A_t or A as appropriate as defined in **8.2.3**.

Table 11 — Strength of bolts in clearance holes

	Bolt property class		Other grades of fasteners
	4.6	8.8	
Shear strength, p_s N/mm^2	160	375	$0.48U_f$ but $\leq 0.69Y_f$
Tensile strength, p_t N/mm^2	195	450	$0.58U_f$ but $\leq 0.83Y_f$
Y_f is the specified minimum yield strength of the fastener U_f is the specified minimum tensile strength of the fastener			

8.2.5 Bearing capacity

8.2.5.1 ***General***

The effective capacity of a bolt in bearing should be taken as the least bearing capacity of the connected material.

8.2.5.2 ***Bearing capacity of connected elements***

The bearing capacity, P_{bs}, for each bolt in the line of force, when washers are used under both the bolt head and the nut, should be taken as:

for $t \leq 1$ mm

$$P_{bs} = 2.1dtp_y$$

for 1 mm $< t \leq 3$ mm

a) for $\dfrac{d_e}{d} \leq 3$,

$$P_{bs} = \left\{2.1 + \left(0.3\frac{d_e}{d} - 0.45\right)(t-1)\right\}dtp_y$$

b) for $\dfrac{d_e}{d} > 3$, $P_{bs} = (1.65 + 0.45t)dtp_y$

for 3 mm $< t \leq 8$ mm

a) for $\dfrac{d_e}{d} \leq 3$, $P_{bs} = \left(1.2 + 0.6\dfrac{d_e}{d}\right)dtp_y$

b) for $\dfrac{d_e}{d} > 3$, $P_{bs} = 3.0dtp_y$

where

t is the minimum thickness of the connected material in millimetres (mm);

d is the nominal diameter in millimetres (mm);

p_y is the design strength in newtons per square millimetre (N/mm^2);

d_e is the distance from the centre of a bolt to the end of the connected element in the direction of the bolt force in millimetres (mm).

These values should be reduced by 25 % when only a single washer or no washers are used.

All of these values apply to plain or metallic coated steel. They do not necessarily apply to other coatings for which suitable values should be determined by testing.

8.2.6 Tensile stress on net section

The tensile stress on the net area of section in a bolted connection should not exceed either p_y or $(0.1 + 3d/s)p_y$

where:

p_y is the design strength in newtons per square millimetre (N/mm^2);

d is the diameter of the bolt in millimetres (mm);

s is the distance between centres of bolts normal to the line of force (see Figure 1) or, where there is only a single line of bolts, the width of sheet in millimetres (mm).

8.2.7 Bolts subject to tension

8.2.7.1 ***Tension capacity***

The tension capacity, P_t, of a bolt should be obtained from:

$$P_t = p_t A_t$$

where

p_t is the tension strength obtained from Table 11;

A_t is the tensile stress area.

8.2.7.2 ***Prying***

In connections subject to tension prying action need not be taken into account provided the stresses given in Table 11 are used.

8.2.8 Combined shear and tension

When bolts are subject to both shear and tension the following relationship should be satisfied in addition to the recommendations in **8.2.3** to **8.2.7**:

$$\frac{F_s}{P_s} + \frac{F_t}{P_t} \leq 1.4$$

where

F_s is the applied shear;

F_t is the applied tension;

P_s is the shear capacity determined in accordance with **8.2.4**;

P_t is the tension capacity determined in accordance with **8.2.7.1**.

8.2.9 Moment capacity of bolt groups

The moment capacity of a group of bolts in shear, where the thickness of the thinner material is less than or equal to 4 mm, may be determined in accordance with **8.1.7** by assuming that each bolt carries its ultimate capacity P, where P is the lesser of P_s and P_{bs} determined in accordance with **8.2.4** and **8.2.5**.

8.3 Friction grip bolts

High strength friction grip bolts conforming to BS 4395-1 may be used for bolted connections, but any additional advantages over **8.2** should be proved by testing in accordance with section **10**.

8.4 Weld detail and design

8.4.1 General

The details of all welded connections should conform to BS 5135.

8.4.2 Details of fillet welds

8.4.2.1 ***End returns***

Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the leg length of the weld unless access or the configuration renders this impracticable. This detail is particularly important for fillet welds on the tension side of parts carrying a bending load.

8.4.2.2 ***Lap joints***

In lap joints the minimum lap should not be less than $4t$ where t is the thickness of the thinner part joined. Single fillet welds should only be used where the parts are restrained to prevent opening of the joint.

8.4.2.3 ***End connections***

Where the end of an element is connected only by longitudinal fillet welds the length, L_w, of each weld should be not less than the transverse spacing, T_s (see Figure 6).

8.4.2.4 ***Single fillet welds***

A single fillet weld should not be subject to a bending moment about the longitudinal axis of the weld.

8.4.2.5 ***Intermittent fillet welds***

Intermittent fillet welds should not be used for members subject to fatigue or where capillary action could lead to the formation of rust pockets. The longitudinal spacing along any one edge of the element between effective lengths of weld, as given in **8.4.3.2**, should not exceed 300 mm or $16t$ for compression elements or $24t$ for tension elements, where t is the thickness of the thinner part joined.

End runs of fillet welds should extend to the end of the part connected.

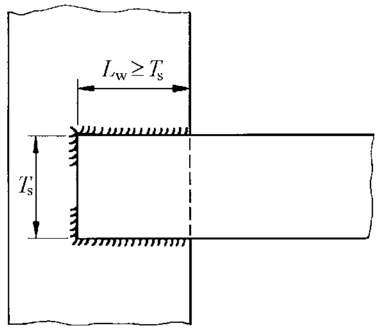


Figure 6 — End connection

8.4.3 Design of fillet welds

8.4.3.1 ***Design strength***

The design strength, p_w, of a fillet weld should be taken as the lesser of:

$$p_w = 0.5U_e$$

$$p_w = 0.5U_s$$

where

U_e is the nominal ultimate tensile strength of the electrode, i.e. the minimum tensile strength given by the product standard for the relevant electrode;

U_s is the nominal ultimate tensile strength of the steel (See **3.3.2**).

For information values of the tensile strength of electrodes for various welding processes are given in Table 12.

Table 12 — Tensile properties of all-weld metal

Electrode type and product standard	Strength designation symbol	Minimum yield strength N/mm²	Minimum tensile strength U_e N/mm²
Wire electrodes for gas shielded metal arc welding (BS EN 440), covered electrodes for manual metal arc welding (BS EN 499), wire electrodes for submerged arc welding (BS EN 756), tubular electrodes (BS EN 758).	35	380	440
	38	355	470
	42	420	500
	46	460	530
	50	500	560

NOTE For yield strength the lower yield (R_{eL}) is used when yielding occurs, otherwise the 0.2 % proof strength ($R_{p0.2}$) is used

Where the fillet welds are symmetrically disposed as shown in Figure 7 the strength of the weld may be taken as equal to the design strength of the parent metal provided that:

a) the weld is made with a suitable electrode (or other welding consumable) which will produce all-weld tensile specimens as specified in BS EN 876 having both a minimum yield strength and a minimum tensile strength not less than those specified for the parent metal;

b) the sum of the throat thicknesses is greater than or equal to the connected plate thickness;

c) the weld is subject to direct compression or tension.

8.4.3.2 ***Effective length***

The effective length of a run of fillet weld should be taken as the overall length less one leg length, *s*, for each end which does not continue round a corner.

The effective length should not be less than 4*s*.

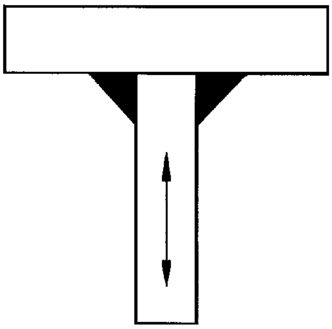


Figure 7 — Symmetrical fillet welds

8.4.3.3 ***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 lying within the weld cross-sections and joining the outermost extremities of the fusion faces. It should not, however, be taken as greater than 0.7 times the effective leg lengths.

8.4.3.4 ***Angle of intersection of members connected by fillet welds***

Where the fusion faces form an angle of greater than 120° or less than 30° the adequacy of the joint should be demonstrated by test.

8.4.3.5 ***Design rules for fillet welds***

The vector sum of the design stresses due to all forces and moments transmitted by the weld should not exceed the design strength, p_w. The design stress in a fillet weld should be calculated on a thickness equal to the effective throat size, *a*.

For a fillet weld with unequal size legs, a deep penetration fillet weld or a partial penetration butt weld with a superimposed fillet weld, the shear and tension stress on the fusion line should not exceed 0.7 p_y and 1.0 p_y respectively.

8.4.4 Partial penetration butt welds

Partial penetration butt welds should not be used for intermittent welds or for welds subject to fatigue.

8.4.5 Design of butt welds

The design strength, p_y, of a full penetration butt weld should be taken as equal to the yield strength, Y_s, of the parent metal, provided that the weld is made with a suitable electrode (or other welding consumable) which will provide all weld tensile specimens as specified in BS EN 876 having both a minimum yield strength and a minimum tensile strength not less than those specified for the parent metal.

8.4.6 Single flare V welds

8.4.6.1 ***General***

Single flare V welds, as shown in Figure 8, should not be designed to transmit any force other than shear.

8.4.6.2 ***Design***

The shear capacity, P_s in newtons (N), may be taken as

for $L_w/t < 25$, $$P_s = 0.6\left(1 - \frac{L_w}{100t}\right)L_w t p_y$$

for $L_w/t \geq 25$, $$P_s = 0.45\, L_w t p_y$$

where

L_w is the length of the weld in millimetres (mm);

t is the thickness of the thinner part joined in millimetres (mm);

p_y is the design strength of the steel in newtons per square millimetre (N/mm²).

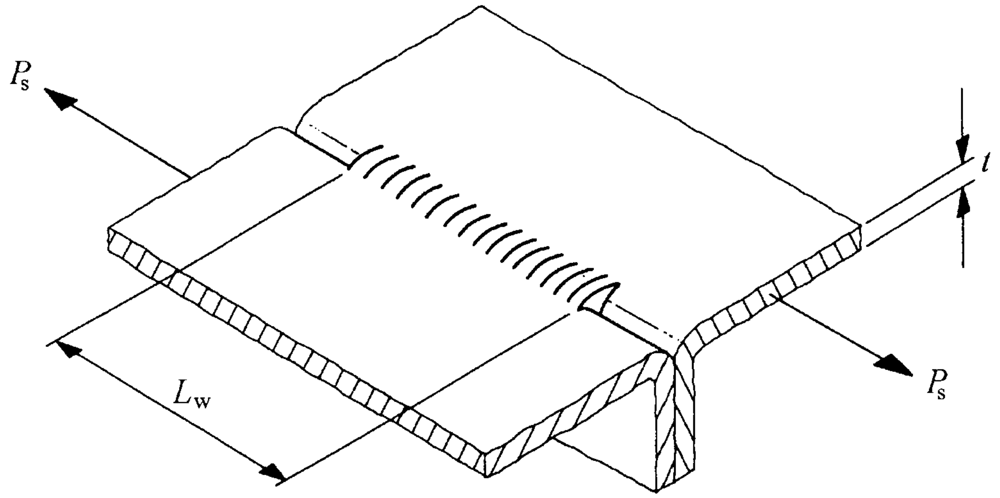


Figure 8 — V weld

8.4.7 Arc spot welds

8.4.7.1 *General*

Arc spot welds, as shown in Figure 9, should not be designed to transmit any forces other than shear.

Arc spot welds should not be used where the thinnest connected part is more than 4 mm thick or through connected material having a total thickness Σt of more than 4 mm.

Weld washers as shown in Figure 9d) should be used when the thickness of the connected material is less than 0.7 mm. Weld washers should have a thickness of between 1.2 mm and 2.0 mm with a prepunched hole of 10 mm diameter minimum.

Arc spot welds should have a minimum interface diameter d_s of 10 mm.

8.4.7.2 *Design*

The shear capacity P_s should be taken as the smaller of a) or b):

a) for failure at the periphery:

for $\dfrac{d_p}{\Sigma t} \leq 18 \left(\dfrac{420}{U_s}\right)^{1/2}$

$$P_s = 1.2 d_p \Sigma t U_s$$

for $18 \left(\dfrac{420}{U_s}\right)^{1/2} < \dfrac{d_p}{\Sigma t} < 31 \left(\dfrac{420}{U_s}\right)^{1/2}$

$$P_s = 21.65 \left(\frac{420}{U_s}\right)^{1/2} (\Sigma t)^2 U_s$$

for $\dfrac{d_p}{\Sigma t} \geq 31 \left(\dfrac{420}{U_s}\right)^{1/2}$

$$P_s = 0.7 d_p \Sigma t U_s$$

b) for failure at the interface:

$$P_s = \frac{\pi}{4} d_s^2 p_w$$

where:

- d_p is the peripheral diameter of the weld;
- d_s is the interface diameter of the weld;
- p_w is the design strength of the weld;
- Σt is the total thickness of connected material;
- U_s is the nominal ultimate tensile strength of the connected material.

The peripheral diameter d_p of an arc spot weld should be obtained as follows:

for a single connected sheet or part of thickness t:

$$d_p = d_w - t$$

for multiple connected sheets or parts of total thickness Σ:

$$d_p = d_w - 2\Sigma t$$

The interface diameter d_s of an arc spot weld, (see Figure 9), should be obtained from:

$$d_s = 0.7 d_w - 1.5 \Sigma t \text{ but } d_s \leq 0.55 d_w$$

where:

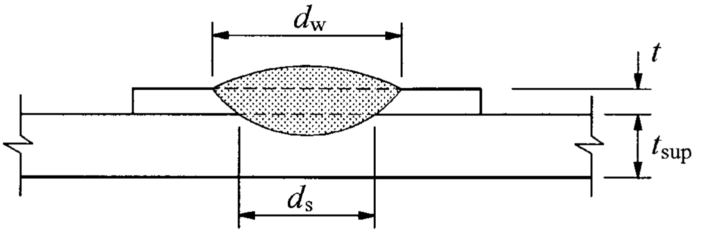
- d_w is the visible diameter of the arc spot weld, (see Figure 9.)

The design strength p_w of the weld should be obtained from:

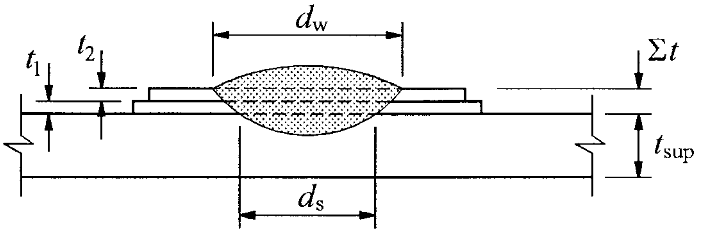
$$p_w = 0.5 U_e \text{ but } p_w \leq 0.5 U_{ss}$$

where:

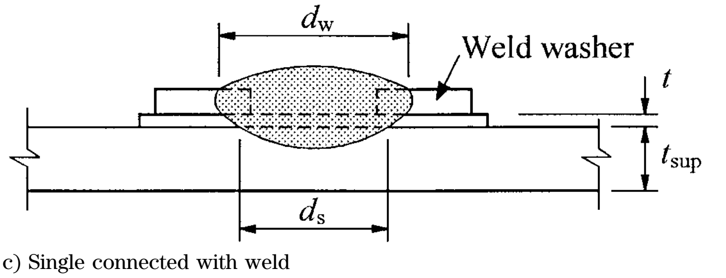
- U_e is the nominal ultimate tensile strength of the electrodes (see Table 12);
- U_{ss} is the nominal ultimate tensile strength of the steel in the supporting member (see **3.3.2**).



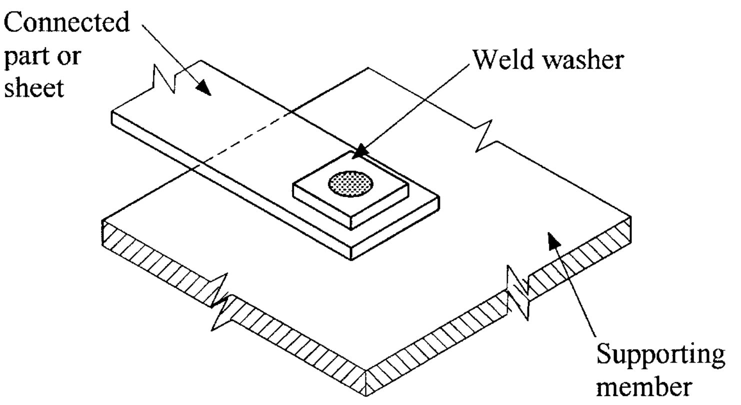
a) Single connected sheet ($\Sigma t = t$)



b) Two connected sheets ($\Sigma t = t_1 + t_2$)



c) Single connected with weld



d) Arc spot weld with weld washer

Figure 9 — Arc spot welds

8.4.7.3 ***End and edge distances***

The minimum distance measured parallel to the direction of force transfer, from the centreline of an arc spot weld to the nearest edge of an adjacent weld or to the end of the connected part towards which the force is directed, should not be less than the value of e_{min} given by the following:

if $U_s/Y_s \geq 1.15$

$$e_{min} = \frac{F_s}{0.7 U_s t}$$

if $U_s/Y_s < 1.15$

$$e_{min} = \frac{F_s}{0.6 U_s t}$$

where:

t is the thickness of the thinnest connected sheet.

In addition, the minimum distance from the centreline of an arc spot weld to the end or edge of the connected sheet should not be less than 1.5 d_w where d_w is the visible diameter of the arc spot weld.

8.4.8 Elongated arc spot welds

8.4.8.1 ***General***

The general recommendations for elongated arc spot welds, as shown in Figure 10, are as given in **8.4.7.1** for circular arc spot welds.

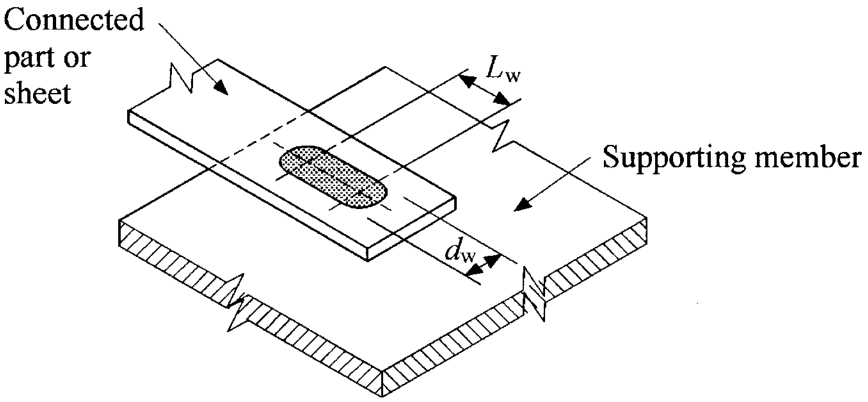


Figure 10 — Elongated arc spot weld

8.4.8.2 ***Design***

The shear capacity P_s should be taken as the smaller of the following:

a) for failure at the periphery:

$$P_s = (0.4\,L_w + 1.33\,d_p)\,\Sigma t U_s$$

b) for failure at the interface:

$$P_s = \left(\frac{\pi}{4}\,d_s^{\,2} + d_s L_w\right) p_w$$

where:

L_w is the length of the weld, see Figure 10;

and d_p, d_s, p_w, Σt and U_s are as defined in **8.4.7.2**.

8.4.8.3 ***End and edge distances***

The minimum end and edge distances for elongated arc spot welds should be as given in **8.4.7.3** for circular arc spot welds.

In addition, the minimum clear distance between the weld and the end of the sheet and between the weld and the edge of the sheet should not be less than $1.0d_w$.

8.5 Resistance spot welds

8.5.1 General

Resistance spot welds where the total thickness welded does not exceed 8 mm should conform to BS 1140. Where the total thickness exceeds 8 mm, the resistance spot welds should be made, inspected and tested to the satisfaction of the engineer.

8.5.2 Details of resistance spot welds

8.5.2.1 ***Minimum pitch***

The distance between the centres of resistance spot welds should be not less than $3d$, where d is the diameter of the resistance spot weld.

8.5.2.2 ***Edge distance***

The distance from the centre of any resistance spot weld to the edge of a plate should not be less than $1.5d$, where d is the diameter of the resistance spot weld.

8.5.2.3 ***Unconventional joint details***

Where calculation is not practicable, a joint or other detail should be designed on an experimental basis as given in section **10**.

8.5.2.4 ***Diameter***

The diameter d of a resistance spot weld should correspond as closely as practicable to the recommended tip diameter of the electrode d_r in millimetres (mm), given by:

$$d_r = 5t^{1/2}$$

where:

t is the thickness of the sheet in contact with the electrode in millimetres (mm).

8.5.3 Design of resistance spot welds

The shear capacity P_s in newtons (N) of a resistance spot weld of diameter, d, in millimetres (mm) in as-rolled and hot dip galvanized material should be taken as follows:

for $t_1 \leq t_2 \leq 2.5t_1$:

$$P_s = 2.7t^{1/2}dp_y$$

where:

p_y is the design strength of the thinner sheet in newtons per square millimetre (N/mm^2);

t is as defined in **8.5.2.4**.

for $t_2 > 2.5t_1$:

$$P_s = 2.7t^{1/2}dp_y \quad \text{but} \quad P_s \leq 0.691d^2p_y \quad \text{and}$$

$$P_s \leq 3.08t_1dp_y$$

where:

t_1 is the thickness of the thinner sheet in millimetres (mm);

t_2 is the thickness of the thicker sheet in millimetres (mm).

8.6 Maximum pitch for connections in sections

8.6.1 Maximum pitch: compression members

The distance between centres, in the line of stress, of fasteners connecting a compression cover plate or sheet to a non-integral stiffener or other element should not exceed any of the following:

a) the spacing required to transmit the shear between the connected parts;

b) $37t(280/Y_s)^{1/2}$ where t is the thickness of the cover plate or sheet in millimetres (mm), and Y_s is the yield strength of the cover plate or sheet in newtons per square millimetre (N/mm^2);

c) three times the flat width of the narrowest unstiffened compression element in that portion of the cover plate or sheet which is adjacent to the connection, or $30t\ (280/Y_s)^{1/2}$ whichever is greater.

NOTE The recommendations of b) do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

8.6.2 Maximum pitch: connection of two channels to form an I-section

For compound sections composed of two channels back-to-back, interconnected by structural fasteners or welds, either the individual members should be designed between points of interconnection in accordance with section **5** and section **6** as appropriate, or the compound section may be designed as a single integral section on the basis of an effective slenderness as defined in **5.6.3** or **6.2.5** provided the longitudinal spacing s of the interconnections does not exceed the following values.

a) For a compression member, designed in accordance with **6.2.5**, at least two fasteners should be provided in line across the width of all members that are sufficiently wide to accommodate them. Moreover, the spacing of interconnections, s, should be such that

i) the member is divided into at least three parts of approximately equal length;

ii) $s \leq 50r_{cy}$

where

s is the longitudinal spacing of interconnections;

r_{cy} is the minimum radius of gyration of one channel.

The interconnecting structural fasteners or welds should be designed to transmit the longitudinal shear, F_s, between the channels induced by a transverse shear, Q, at any point in the compound section. The value of Q should be taken as not less than 2.5 % of the design axial force in the compound plus any load due to self weight or wind resistance. The resulting longitudinal shear force per interconnection, should be taken as:

$$F_s = 0.25Q\left(\frac{s}{r_{cy}}\right)$$

where s/r_{cy} is the local slenderness of an individual channel as given in **5.6.3**d) or **6.2.5**.

b) For a flexural member designed in accordance with **5.6.3**d), at least two structural fasteners or welds should be provided in line across the width of all sections. The tendency of the individual channels to separate by twisting should be resisted by limiting the spacing of interconnections, s, such that:

i) the beam length is divided into at least three parts of approximately equal length;

ii) $s \leq 50r_{cy}$

where

s is the longitudinal spacing of interconnections;

r_{cy} is the minimum radius of gyration of one channel;

iii) the tensile capacity, P_t of the individual interconnections is greater than the induced transverse shear force, F_s, (See Figure 11), that is

$$P_t \geq F_s$$

where

$$F_s = \frac{Fe}{2h}$$

and

e is the distance of the shear centre of the channel from the mid-plane of the web;

h is the vertical distance between the two rows of connections near or at the top and bottom flanges;

F is the local concentrated load or reaction between the points of interconnection under consideration;

or, for distributed load

$$F = ws$$

and

w is the load intensity on the beam acting on a bearing length of $s/2$ each side of the interconnections under consideration.

The required maximum permissible spacing of connections depends upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it should be determined at the point of maximum local load intensity. If, however, this procedure would result in uneconomically close spacing, then either the spacing may be varied along the beam according to the variation of the load intensity, or reinforcing cover plates may be welded to the flanges at the points where concentrated loads occur; the strength in shear of the connections joining such plates to the flanges should then be used for P_t, and h in the equation should be taken as the depth of the beam.

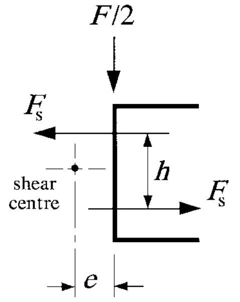


Figure 11 — Connection forces in back-to-back members

8.7 Screws, blind rivets and powder actuated fasteners

Recommendations for screws, blind rivets and powder actuated fasteners are given in annex A.

8.8 Holding-down bolts

Holding-down bolts should be designed to resist the effect of factored loads determined in accordance with **2.3.2.4**. They should provide resistance to tension due to uplift forces and bending moments where appropriate. They should also provide resistance to shear.

The tension capacity of the bolt should be calculated in accordance with **8.2.7**.

Section 9. Simplified rules for commonly used members

9.1 General

This section gives empirical rules for the design of certain commonly used members for which a full theoretical analysis may be impracticable or not justified. The design rules given in this section may be used as an alternative to the analytical methods given in sections **2** to **8** or the testing methods given in section **10**. Members designed by a proven method need not conform to the empirical rules.

The design rules in this section apply to all steels with a yield strength, Y_s, of not less than 250 N/mm^2.

9.2 Z purlins with lips

9.2.1 General

Where Z purlins with lips are designed in accordance with these simplified rules, the following recommendations apply.

a) When designing purlins, unfactored loads should be considered.

b) The imposed load should be determined in accordance with **2.2.2** but should be taken as not less than 0.60 kN/m^2. For agricultural buildings, imposed loads should be in accordance with the recommendations in BS 5502-22.

c) The cladding and fixings should be capable of providing lateral restraint to the purlin and of carrying the component of load in the plane of the roof slope.

d) The purlin should be designed to carry the component of load normal to the roof slope. The purlin may also be allowed to carry a nominal axial load due to wind or restraint forces, provided the axial stress on the full cross-section due to these causes does not exceed 6 N/mm^2.

e) The design rules apply to purlins up to 8 m span in roof slopes up to 22½°. For spans up to 5 m, the purlins may be nominally simply supported with a two bolt connection, but for spans above 5 m the purlins should be continuous or provided with sleeves or splices with a moment capacity equal to that of the member. In multispan conditions, the design rules included in this section may be used provided adjacent spans do not differ by more than 20 %.

f) Where a purlin span exceeds 4.6 m, anti-sag bars should be provided so that the laterally unsupported length does not exceed 3.8 m. In pitched roof buildings, the anti-sag bars should be tied across the apex; in monopitch roofs, the anti-sag bars should be anchored to a rigid apex support or their forces transferred diagonally to the main frames. Erection and wind uplift bracing should be employed according to normal good practice.

g) The purlin lips may be splayed outwards at an angle not exceeding 10°. In the calculation of section properties, the width of the lips should be taken as one-fifth of the flange width.

h) The purlin cleats should provide torsional restraint to the purlin not less effective than that normally given by angle cleats.

9.2.2 Design rules

The properties of a Z purlin should be as follows, (see also Figure 12).

$100t \geq$ overall depth $\geq L/45$

Total width over both flanges $\geq L/60$

Overall width of compression flange/thickness

$$\frac{B}{t} \leq 35$$

Width of lip $\geq B/5$

For simply supported purlins: section modulus

$$\geq \frac{WL}{1\,400}\ \text{cm}^3$$

For continuous or semi-rigidly jointed purlins:

$$\text{section modulus} \geq \frac{WL}{1\,800}\ \text{cm}^3$$

where

L	is the span of the purlin in millimetres (mm) between the centres of supports;
W	is the normal component of the unfactored distributed dead plus imposed load in kilonewtons (kN);
B	is the width of the compression flange in millimetres (mm);
t	is the thickness of the purlin in millimetres (mm).

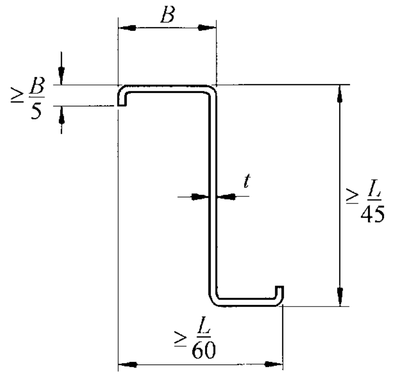


Figure 12 — Z purlins and sheeting rails

9.2.3 Wind uplift

The net allowable wind uplift acting in the direction normal to the roof when the purlins are restrained as given in **9.2.1**f), should be taken as 50 % of the allowable load, W, under dead plus imposed load conditions.

9.3 Z sheeting rails with lips

9.3.1 General

When Z sheeting rails are designed in accordance with these simplified rules, the following recommendations apply.

a) When designing sheeting rails, unfactored loads should be considered.

b) The design rules apply to sheeting rails up to 8 m span. For spans up to 6.5 m, the rails may be nominally supported with a two bolt connection, but for spans above 6.5 m the rails should be continuous or provided with sleeves or splices with a moment capacity equal to that of the member. In multi-span conditions, the design rules included in this section may be used provided adjacent spans do not differ by more than 20 %.

c) The self weight of the sheeting may be carried in one of the following ways:

1) by a dwarf wall or floor slab [see Figure 13a)];

2) by an eaves beam and vertical loading [see Figure 13b)];

3) by diagonal bracing and vertical loading [see Figure 13c)];

4) by the sheeting acting as a diaphragm [see Figure 13d)]; this method may only be used for metal sheeting fixed with self tapping screws or other direct fixings to the rails and with seams fastened at not more than 450 mm centres.

In cases 1) and 4) there is nominally no vertical bending on the rail; in cases 2) and 3) there is vertical bending between the points of support. Nevertheless in all cases considered in **9.3.3.2** the design expressions include an element of vertical support.

In each case, vertical supports are required to restrain the inside flange of the rail and prevent twisting. The cladding and fixings should be capable of providing lateral restraint to the outside flange.

9.3.2 Vertical supports

9.3.2.1 *Spacing and strength*

The vertical supports detailed in **9.3.1**c) should be positioned at midspan for spans up to 4.6 m and at one-third span points for greater spans. They should be designed to carry an axial tensile or compressive force, as appropriate, of 2 kN, or the force due to the self weight of the cladding, if applicable, whichever is the greater. In addition, they should provide adequate lateral and torsional restraint to the rail.

9.3.2.2 *Cleats and details*

The rail cleats and other details should be adequately strong and rigid to transfer the self weight of the cladding from the rail face to the main member. Even if this self weight is supported elsewhere, the cleats and other details should be designed for a minimum vertical force of 2 kN at the rail face.

9.3.3 Design rules

9.3.3.1 *Dimensions*

The dimensions of a Z sheeting rail should be as follows, with reference to Figure 13.

$100t \geq$ overall depth $\geq L/45$

Total width over both flanges $\geq L/60$

Overall width of compression flange/thickness

$$\frac{B}{t} \leq 35$$

Width of lip $\geq B/5$

where

- L is the span of the sheeting rail in millimetres (mm);
- B is the flange width in millimetres (mm);
- t is the thickness of the sheeting rail in millimetres (mm).

The lips may be splayed outwards at an angle not exceeding 10°. In the calculation of section properties the width of the lips should be taken as $B/5$.

9.3.3.2 *Design expressions*

The design expressions are as given in Table 13, where:

- L is the span of the sheeting rail between centres of supports in millimetres (mm);
- W_w is the unfactored positive wind load on the rail in kilonewtons (kN) (causing tension in the inside, i.e. the unrestrained flange);
- W_d is the unfactored weight of cladding, etc., acting on the rail in kilonewtons (kN) or 2 kN, whichever is the greater (see Note);
- Z_x, Z_y are the section moduli about x and y axes in cubic centimetres (cm^3).

NOTE In the arrangements shown in Figure 13a) and d), the self weight of the cladding is not carried by the rail, hence $W_d = 2$ kN. In the arrangements shown in Figure 13b) and c), the self weight of the cladding is carried by the rail, hence W_d is this self weight or 2 kN whichever is greater.

9.3.3.3 *Wind suction*

For a given section, the expressions given in Table 13 define the unfactored positive wind load W_w which may be applied to the rail. For negative wind (causing compression in the unrestrained flange) the maximum suction load should not exceed $0.5W_w$.

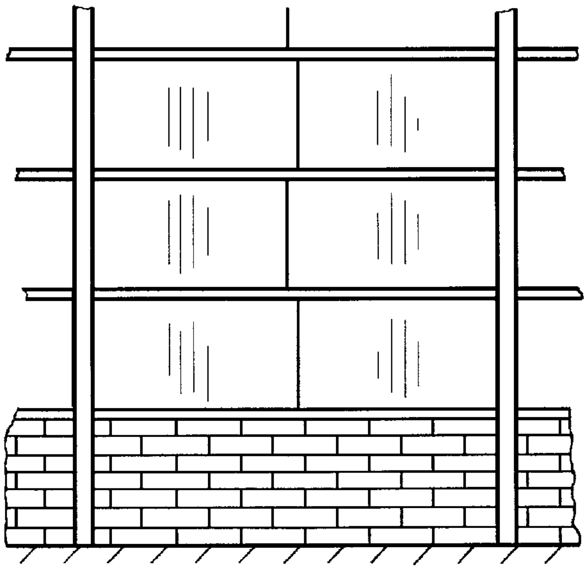
9.4 Lattice joists

9.4.1 General

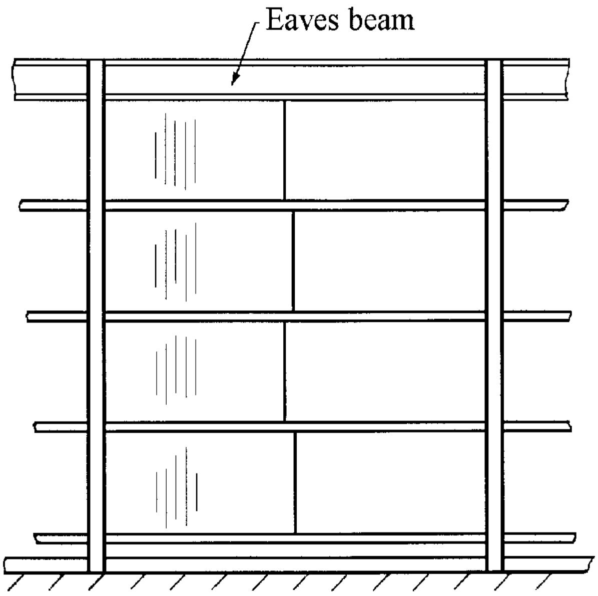
Lattice joists for use in roofs and floors should be designed as normal simply supported trusses but should be subject to the rules and limitations given in **9.4.2** and **9.4.3**. The rules apply to lattice joists which are laterally supported in accordance with **9.4.3** and in which the upper and lower chord members are substantially parallel. It is assumed that secondary bending stresses are insignificant.

Table 13 — Design expressions for Z sheeting rails

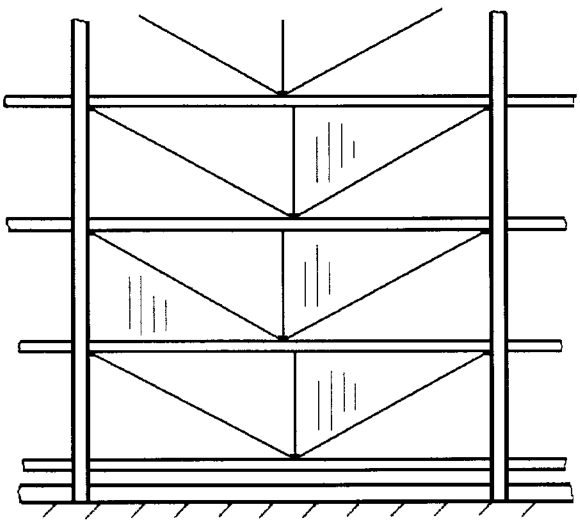
End conditions of rail	Vertical supports	
	One row at midspan for spans up to 4.6 m	Two rows at one-third span for spans 4.6 m to 8.0 m
Simply supported	$\frac{W_w L}{1\,750 Z_x} + \frac{W_d L}{6\,000 Z_y} \leq 1$	$\frac{W_w L}{1\,750 Z_x} + \frac{W_d L}{16\,880 Z_y} \leq 1$
Continuous or semi-rigidly jointed	$\frac{W_w L}{2\,250 Z_x} + \frac{W_d L}{6\,000 Z_y} \leq 1$	$\frac{W_w L}{2\,250 Z_x} + \frac{W_d L}{16\,880 Z_y} \leq 1$



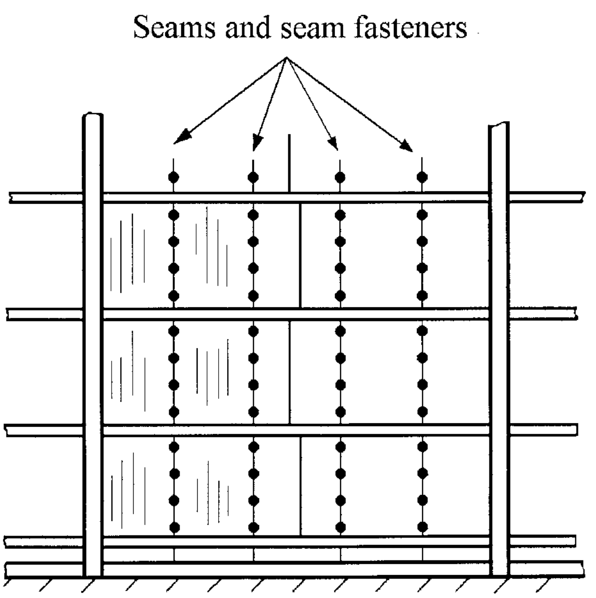
a) Dwarf wall of floor slab and vertical supports



b) Eaves Beam and vertical supports



c) Diagonal bracing and vertical supports



d) Sheeting acting as a diaphragm

NOTE For sag rod positions refer to **9.3.2.1**

Figure 13 — Supports for self weight of sheeting

9.4.2 Design rules and limitations

The design rules and limitations are as follows.

a) In designing lattice joists, factored loads should be considered.

b) For the purpose of calculating the forces in members, the connections may be assumed to be pinned.

c) 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.

d) Where the exact position of a vertical point load on the top or bottom chord is not known, the local bending moment should be taken as $W_p L/6$ where L is the distance between nodes and W_p is the factored point load. Prior to cladding, a value of W_p = 2.25 kN should be used in the design of the top chord.

e) The web members should be designed from considerations of shear under the total applied load. The minimum value of shear taken should be 20 % of the end reaction, or 2.25 kN, whichever is greater.

f) Timber or other materials may be set in the top or bottom chords but should be neglected in the design calculations.

g) If the following ratios of clear span to depth (measured at midspan) are maintained, the deflections of lattice joists need not be calculated.

Roof joists:	span/depth $\leq$ 24
Floor joists: span $\leq$ 10 m,	span/depth $\leq$ 24
Floor joists: span > 10 m,	span/depth $\leq$ 20

These ratios may be exceeded provided the deflection can be shown not to exceed the appropriate limits given in **2.4.2**.

h) For lattice joists used in roofs, upward camber may be employed to offset the deflection due to dead load only, or to provide drainage falls or prevent ponding.

9.4.3 Lateral support

The roof or floor deck, together with the fixings and any intermediate members, should be capable of providing the lateral support necessary to the lattice joist in the completed structure. In the erection stage additional lateral braces may be necessary.

If, in the completed structure, the roof or floor deck does not offer lateral support to the compression flange of the lattice joist, then lateral braces should be provided as follows.

a) Each lateral brace should be capable of resisting 2.5 % of the maximum force in the compression chord of the lattice joist. Where several lattice joists share a common bracing system, the minimum total lateral force may be taken as the sum of the lateral restraint forces required for the three largest joists.

b) All lateral braces should have L/r < 250 where L is the distance between connection points and r is the least radius of gyration.

c) The spacing of the lateral braces along the compression chord of the lattice truss should be such that L/r_y < 120 where L is the spacing and r_y is the radius of gyration of the compression chord about its vertical axis.

Section 10. Loading tests

10.1 General

10.1.1 Purpose of testing

Testing is not required for structures and parts of structures designed in accordance with sections **1** to **9** of this part of BS 5950.

Experimental verification by loading tests as stipulated in **2.1.2.7** 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 sections **1** to **9** 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 to design the particular structure or component by calculation alone, for example, to ensure that appropriate account is taken of the effects of interaction with other components;

d) the design load-carrying capacity of a structural assembly or component is to be established from a knowledge of its performance as determined from testing, for example:

— if it is desired to take account of the actual properties of the cold formed member;

— if realistic data for design cannot otherwise be obtained;

— if it is desired to produce resistance tables based on tests, or on a combination of tests and analysis;

e) if it is intended to construct a number of similar structures on the basis of a prototype.

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.

10.1.2 Types of loading tests

The following types of test may be carried out as appropriate:

1) a component test to establish values of a parameter for use in design, (see **10.5**);

2) a proof test for confirming general structural behaviour, (see **10.6**);

3) a strength test against the required factored loads, (see **10.7**);

4) a failure test to determine the ultimate capacity and mode of failure, (see **10.8**).

These test procedures are intended only for steel structures within the scope of this part of BS 5950.

NOTE Recommendations for the testing of other types of steel structures are given in BS 5950-1, -3-3.1, -4, -6, or -9, as appropriate.

10.1.3 Quality control

If a structure or component has been designed on the basis of strength or failure tests detailed in **10.7** or **10.8**, 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 the manufacturer's mill inspection document).

If, from this examination, it is not possible to determine either the variations or the effect of variations compared to the tested prototype, a proof test as detailed in **10.6** should be carried out. In this test, the deflections should be measured at the same positions as in the initial proof testing 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.

10.2 Test conditions

10.2.1 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.

10.2.2 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.

10.2.3 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.

10.3 Test procedures

10.3.1 Preliminary loading

Prior to any test, preliminary loading (not exceeding the unfactored values of the relevant loads) may be applied in order to bed down the test specimen onto the test rig, and then removed.

10.3.2 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 of the principal deflection against loading should be maintained. When this indicates significant non-linearity the load increments should be reduced.

10.3.3 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 recovered from flat unyielded areas of the test specimens after the completion of the load testing. Where appropriate, they may be cut from the same sections, sheet or strip as the test specimens, provided that adequate traceability exists to demonstrate that the coupons tested relate to the specimens used in the loading tests.

The yield strength and ultimate 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.

Where the material properties are required in advance of load testing, (as when determining the test load in a strength test, see **10.7.2**), single coupon tests from each different lot of material for the components of an individual test specimen may be used to obtain a weighted mean yield strength of the whole assembly.

NOTE For tests to determine the characteristic properties of steel supplied without test certificates see **3.3.2**.

10.3.4 Test report

A test report including the following information should be prepared:

- — 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;
- — if possible, tested samples should be retained. If not possible photographs of the samples after testing should be kept.

10.4 Relative strength coefficient

10.4.1 General

Test results should be appropriately adjusted to allow for variations between the actual measured properties of the test specimens and their nominal values.

The measured basic yield strength of the specimen should not deviate from the nominal yield strength, Y_s, by more than $\pm$25 %. The measured material thickness should not exceed the nominal thickness by more than 12 %.

Adjustments should be made in respect of the actual measured properties for all tests, except where the design expression that uses the test results also uses the measured value of the properties as appropriate.

The effect of variations of the geometry or material properties of the test specimens, compared to their nominal values, should be taken into account by means of a relative strength coefficient.

10.4.2 Predetermining a test load

When the objective of the loading test is to achieve a predetermined level of performance, as in a strength test to **10.7**, the relative strength coefficient should be applied in determining the test load, see **10.7.2**.

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 **10.3.3**.

When a 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 **10.6**.

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 **10.4.2**.

10.4.3 Calibrating the results of a failure test

When the objective of the loading test is to determine the ultimate failure load of a component or assembly, as in a failure test, (see **10.8**), the relative strength coefficient should be applied in determining the design capacity from the test results, see **10.8.3.1**.

If realistic assessments of the capacity can be made using the provisions of sections **1** to **9**, or by other proven methods of design by calculation that take account of all buckling effects, the relative strength coefficient R_s may be determined 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 ultimate 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 L_E/r and the mean yield strength relates to the cross-section at which failure is observed, or alternatively R_s is obtained as in a) if the relevant slenderness is in doubt;

e) *for a ductile failure due to local buckling of a flat element:*

$$R_s = \frac{\text{actual yield strength}}{\text{nominal yield strength}} \times \frac{\text{actual thickness}}{\text{nominal thickness}} \times R_p$$

$$\text{but } R_s \geq \left[\frac{\text{actual yield strength}}{\text{nominal yield strength}}\right]^{0.5} \times \left[\frac{\text{actual thickness}}{\text{nominal thickness}}\right]^{2} \times R_p$$

and $R_s \geq 1$

where:

$$R_p = \frac{\text{actual value of section property}}{\text{nominal value of section property}} \text{ but } R_p \geq 1$$

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.

10.5 Component tests

10.5.1 General

If component tests are carried out to determine the value of a parameter to be used in design, sufficient tests shall be carried out to establish a characteristic value. At least four specimens should be tested from each batch of material and the results adjusted using the relative strength coefficient (see **10.4**) and the modification factor K_t given in **10.8.3.1**.

10.5.2 Full cross-section tension test

The result of a full cross-section tension test may be used to determine the average yield strength, Y_{sa}, of the cross-section. If advantage is to be taken of the enhanced yield stress, obtained from such tests, in assessing the design capacity of components or structures produced from the same material, the recommendations of **10.5.1** should be observed.

The specimen should have a length of at least five times the width of the widest plane element in the cross-section. The load should be applied through end supports that ensure a uniform stress distribution across the cross-section. The failure zone should occur at a distance from the end supports of not less than the width of the widest plane element in the cross-section.

10.5.3 Full cross-section compression tests

10.5.3.1 *Stub column tests*

Stub column tests may be used to determine the short strut capacity, P_{cs}, (see **6.2.3**) of a thin gauge cross-section, taking account of the enhanced yield stress due to cold forming and the effects of local buckling. If the results of such tests are to be used in assessing the design capacity of components or structures produced from the same material, the recommendations of **10.5.1** should be observed.

It is essential that the results of this test are not used to circumvent the recommendations of **6.2.3**.

The specimen should have a length of at least three times the widest plane element. In the case of a cross-section with edge or intermediate stiffeners the length of specimen should be not less than the expected buckling length of the stiffeners. If the length of the specimen is greater than 20 times the least radius of gyration, intermediate supports should be provided at a spacing not exceeding 20 times the least radius of gyration.

The ends of the specimen should be flat and perpendicular to its longitudinal axis.

The loading should be applied through small spherical bearings to the longitudinal axis of the specimen located at the centroid of the calculated effective cross-section. Alternatively, if it is required only to determine the enhanced yield stress due to cold forming, the loading may be applied through parallel platens and end rotation restraints may be used.

10.5.3.2 *Tests on complete compression members*

A test on a complete compression member may be used to determine the member behaviour taking into account the influence of restraints afforded by other components. The test set-up should reflect the in-service conditions as accurately as possible bearing in mind the influence of:

a) restraints offered by base plates and/or the connections to other members;

b) the line of action of the loads;

c) connection flexibility.

10.5.4 Full cross-section bending tests

10.5.4.1 *Pure bending tests on representative lengths of member*

The results of a pure bending test on a representative length of a member may be used to give the base moment of resistance of the member. If the results of such tests are to be used in determining the design bending capacity of a member, the recommendations of **10.5.1** should be observed.

The specimen should normally have a length at least eight times the greatest cross-sectional dimension. The compression flange should be supported at distances equal to or less than 20 times the least radius of gyration of the section. Two point loads should be applied to the specimen so that there is a length of uniform bending at the centre of the specimen of at least 0.2 times the span and no greater than 0.33 times the span. The loads should be applied through the shear centre of the section or the section should be torsionally restrained at the load and support points. At the points of load application, local buckling of the specimen may be restrained as necessary to ensure that failure occurs within the central portion of the span. Deflections should be measured at the ends of the member, at the load positions and at midspan.

10.5.4.2 *Internal support test*

An internal support test may be used to determine the moment rotation relationship of a continuous or jointed member. The test span and loading arrangement should reflect the conditions in service, taking account of the ratio of moment to shear force at the support and allowing for any redistribution of moment. The test should be continued into the post-yield phase until the deflection has reached a value six times the maximum elastic displacement, or until the applied load has reduced to between 10 % and 15 % of its peak value. The post yield rotation for a range of applied test loads may be determined from a consideration of the load deflection plot and a mean value of the moment-rotation characteristic determined for use in design by analysis. A characteristic value of the rotational stiffness may be taken as the mean of at least two tests, provided that each result is adjusted as appropriate using the relative strength coefficient (see **10.4**) and is within ±10 % of the mean value.

10.5.4.3 *Bending tests on complete members*

A bending test on a complete member may be used to determine the influence of lateral restraint from other components or materials on the member under consideration. Appropriate tests may be carried out and their results used in design provided that the test set up accurately reflects the in-service conditions with particular regard to:

a) supports, bearing in mind their influence in restraining warping and torsion;

b) loading conditions;

c) connections.

10.5.5 Testing of connections with fasteners

Tests on connections with fasteners may be carried out to determine their tensile, compressive or bending strength and the corresponding values of slip or rotation.

NOTE Formal procedures for many practical cases are given in ECCS Publications No. 21: *European recommendations for steel construction: the design and testing of connections in steel sheeting and sections.*[2] and ECCS Publication No. 42: *European recommendations for steel construction: mechanical fasteners for use in steel sheeting and sections* [3]

These tests should reflect the in-service conditions as accurately as possible and the results may be used in the analysis of the structure or assembly.

10.6 Proof test

10.6.1 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 min. The loading should be adjusted to remain constant until there is no significant increase in deflection during at least three checking intervals subsequent to the attainment of the proof test load.

10.6.2 Proof test load

The test load for a proof test should be taken as equal to the sum of:

a) 1.0 × (actual dead load present during the test);

b) one of the following as appropriate:

1) 1.25 × (imposed load) plus 1.15 × (remainder of dead load);

2) 1.15 × (remainder of dead load) plus 1.2 × (wind load);

3) 1.2 × (wind uplift) minus 1.0 × (remainder of dead load);

4) 1.15 × (remainder of dead load) plus 1.0 × (imposed load and wind load).

10.6.3 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 this repeat test.

10.7 Strength test

10.7.1 General

A strength test may be used to confirm the calculated load-carrying capacity of a structural assembly or component. Before a strength test is carried out, the test specimen should first pass the proof test detailed in **10.6**.

If a number of similar items are to be constructed using 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 (see **10.2.3**) 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 min. The loading should be adjusted to remain constant until there is no significant increase in deflection during at least three checking intervals subsequent to the attainment of the strength test load.

10.7.2 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:

(Strength test load) = R_s × (factored load)

in which R_s is the relative strength coefficient determined in accordance with **10.4**.

10.7.3 Criteria

On removal of the strength test load the residual deflection should not exceed 80 % of the maximum deflection recorded during this test.

Under the strength test load none of the following events should occur in any part of the test specimen:

— collapse or fracture;

— a crack beginning to spread in a vital part of the specimen;

— the displacement becoming grossly excessive.

10.8 Failure test

10.8.1 General

A failure test may be used to determine the real mode of failure and the ultimate load carrying capacity of a structural assembly or component. Because it is only from a test to failure that this information can be obtained, when a 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 followed by a strength test before proceeding 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 load. 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.

10.8.2 Failure criteria

The ultimate load-carrying capacity should be taken as the value of the test load beyond which the component or assembly 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 beginning to spread in a vital part of the specimen;

— the displacement becoming 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.

10.8.3 Evaluation of test results

10.8.3.1 *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 in accordance with **10.4**, and the mean test result is based on a minimum of four tests.

If the resulting design capacity falls below the mean result obtained from the strength test, the latter should be taken.

For four or more related tests K_t should be determined from:

$$K_t = 1.1 \times \left[1 - \frac{k \times s}{\text{(mean test result)}}\right]; \text{ but not greater than } 1.0$$

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 14 for the appropriate number of tests;

n is the number of tests;

v_i is the result of test i.

Table 14 — 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

10.8.3.2 *Design capacities for families of tests*

A series of tests carried out on a number of otherwise similar components or structural assemblies, in which one or more parameter is varied, may be treated as a single family of tests, provided that they all have the same failure mode. The parameters that are varied may include cross-sectional dimensions, spans, thickness and material strengths.

The design capacities of the members of a family may be determined on the basis of a suitable design expression that relates the test results to all the relevant parameters. This design expression may either be based on the appropriate equations of structural mechanics, or determined on an empirical basis.

The design expression should be modified to predict the mean measured resistance as accurately as practicable, by adjusting the coefficients to optimize the correlation.

In order to calculate the standard deviation, s, each test result should first be normalized by dividing it by the corresponding value predicted by the design expression. If the design expression has been modified as specified above, the mean value of the normalized results will be unity. The number of tests, n, should be taken as equal to the total number of tests in the family.

For a family of at least four tests, the design capacity should be determined using:

$$\text{design capacity} = 1.1R_{\mathrm{d},i}\ (1 - k \times s) \text{ but} \leq R_{\mathrm{d},i}$$

where:

$R_{\mathrm{d},i}$ is the resistance predicted by the design expression for the specific parameters;

k is a statistical factor obtained from Table 14 for the appropriate number of tests;

s is the standard deviation of the normalized test results, obtained from:

$$s = \sqrt{\frac{\sum_{i=1}^{n} (\bar{v}_i)^2 - \frac{1}{n}\left(\sum_{i=1}^{n} \bar{v}_i\right)^2}{n - 1}}$$

in which,

n is the number of tests;

$\bar{v}_i$ is the normalized result of test $i = \dfrac{v_i}{R_{\mathrm{d},i}}$;

10.9 Load tables

10.9.1 General

Load tables for components and systems may be based either completely on the results of tests in accordance with **10.8** or on a combination of testing and rational analysis.

Such tables may be used to represent the performance of a member or assembly when its behaviour is influenced by interaction with cladding and other structural components, provided the member or assembly forms part of a specific structural system in which the effect of those factors has been evaluated.

If the performance of a system relies on the stabilizing effect of associated materials, such as sheeting on roof purlin systems, load tables based on testing should clearly state the necessary conditions of validity in terms of the associated materials and the methods of fixing them.

Extrapolation should generally be avoided. However limited extrapolation may be used where this can be justified on the basis of a specific and appropriate analysis of test results, provided that it can be demonstrated that this extrapolation does not lead to conditions in which a different failure mode is likely.

In preparing load tables, account should be taken of the possibility that relevant serviceability limit state criteria, rather than ultimate limit state design resistance, might govern the load-carrying capacity.

Load tables should always clearly state whether they are for factored or unfactored load capacities.

In unfactored load tables, where the combination of loads is not specified, the tabulated values should be based on the most conservative combination of factored loads.

10.9.2 Tables based completely on testing

If the load tables are based completely on test results, sufficient tests should be carried out to determine the characteristic design capacity of the component or system using the statistical factors given in Table 14. The tests should cover the whole range of geometries and loading conditions to be published in the load tables in such a way that no extrapolation from the test results is contained in the published data. The tests should confirm that, when subjected to the stated service loads, the components do not exhibit significant local or permanent distortions. The forms of loading and support used in the tests should conform to those stated in the load tables.

10.9.3 Tables based on combined testing and analysis

As an alternative to **10.9.2**, load tables may be based on a rational analysis assisted by testing. The mathematical model of the resistance should take account of all failure modes that are possible within the range of the load tables. This mathematical model should be validated by testing.

The validation of the mathematical model may be carried out by means of full scale tests on a completely representative portion of the structure, comprising the structural components and connections, together with the associated materials and methods of fixing which will be used in service.

Alternatively the mathematical model may be validated by carrying out separate tests on all members, connections and other structural components to determine their strength and stiffness, and the rotational restraint given to the members by cladding.

The results derived should be adjusted to account for specimen variability using the relative strength coefficient given in **10.4**. The adjusted results may then be used in a rational analysis of the component or system to produce load tables. The analysis used should take into account all failure possibilities. If this is in doubt further tests in accordance with **10.8** should be carried out to check the validity of the analysis.

Annex A (normative)
Screws, blind rivets and powder actuated fasteners

A.1 Connections with screws and blind rivets

A.1.1 ***General***

This clause applies to self-tapping screws, including thread-forming, thread-cutting or self-drilling screws, with 3.0 mm $\leq d \leq$ 8.00 mm and to blind rivets with 2.5 mm $\leq d \leq$ 7.5 mm where d is the diameter of the screw or rivet.

If members of different thickness are connected, the head of the screw or the preformed head of the rivet should be in contact with the thinner member.

The diameter of the pre-drilled holes should be strictly in accordance with the manufacturer's requirements.

In load-carrying connections, at least two fasteners should be used.

A.1.2 ***Minimum pitch***

The distance between centres of fasteners should be not less than $3d$.

A.1.3 ***Minimum edge and end distances***

The distance from the centre of a fastener to the edge of any part should be not less than $3d$. If the connection is subjected to force in one direction only which is such as to cause shear of the fastener, the minimum edge distance may be reduced to $1.5d$ or 10 mm whichever is the smaller, in a direction normal to the force.

A.1.4 ***Calculation of shear capacity in tilting and bearing***

The shear capacity, P_s, of a screw or rivet in tilting and bearing may be taken as:

a) for $\dfrac{t_4}{t_3} = 1.0$,

the smaller of	$P_s = 3.2\ (t_3{}^3 d)^{1/2} p_y$
and	$P_s = 2.1 t_3 d p_y$
b) for $\dfrac{t_4}{t_3} \geq 2.5$	$P_s = 2.1 t_3 p_y$
c) for $1.0 < t_4/t_3 < 2.5$	P_s may be determined by linear interpolation between a) and b)

where

- t_3 is the thickness of the member in contact with the screw head or the preformed rivet head;
- t_4 is the thickness of the member remote from the screw head or the preformed rivet head;
- d is the diameter of the fastener;
- p_y is the design strength of the member material.

The shear capacity, P_{fs}, of the fastener itself should be determined by testing and should normally be guaranteed by the manufacturer. P_{fs} should be greater than $1.25P_s$.

A.1.5 ***Calculation of tensile capacity***

Blind rivets should not normally be used to carry significant tensile forces.

For screws which carry significant tensile loads, the head of the screw, or washer if present, should have a diameter, d_s, of at least 8 mm and should have adequate rigidity.

The tensile capacity, P_t, at a screwed connection may be taken as the smallest of a), b) or c).

a) *Pulling of the connected material over the screw head or washer.* For connected material of thickness, t_3 less than 2.00 mm; and for head or washer size, d_s, less than 25 mm:

$P_t = 1.1 t_3 d_s p_y$

For other configurations the tensile capacity should be determined by testing.

b) *Pull out from the base material.* For base material thickness, t_4, greater than 0.9 mm:

$P_t = 0.65 t_4 d p_y$

c) *Tensile failure of the screw itself.* The tensile capacity, P_{ft} of the screw itself can only be determined by testing and should normally be guaranteed by the manufacturer. P_{ft} should be greater than $1.25P_t$.

A.2 Powder actuated fasteners

A.2.1 ***General***

This clause applies to fasteners with 3.5 mm $\leq d \leq$ 4.5 mm where d is the diameter of the fastener.

Limitations specified by the manufacturer regarding the thickness and yield stress of the material to be joined should be observed. The thickness of the base material into which the fastener is fixed should not be less than 6 mm.

A.2.2 ***Minimum pitch***

The distance between centres of fasteners should be not less than $4.5d$.

A.2.3 ***Minimum edge and end distances***

The distance from the centre of a fastener to the edge of any part should not be less than $4.5d$.

A.2.4 ***Calculation of shear capacity***

The shear capacity, P_s, of a powder actuated fastener with respect to tearing of the fastened material may be taken as:

$P_s = 3.2 t d p_y$

where

- t is the thickness of the member in contact with the fastener head;
- d is the diameter of the fastener;
- p_y is the design strength of the fastened material.

The shear capacity, P_{fs}, of the fastener itself should be determined by testing and should normally be guaranteed by the manufacturer. P_{fs}, should be greater than $1.25P_s$.

A.2.5 ***Calculation of tensile capacity***

Powder actuated fasteners should not be used to carry significant tensile forces unless they are used with a suitable washer of minimum diameter 8 mm and of adequate rigidity.

The tensile capacity, P_t, of a fastener may be taken as the smaller of a) or b).

a) *Pulling of the connected material over the washer.* P_t should be determined in accordance with **A.1.5**a).

b) *Pull out from the base material.* P_t should be determined by test.

Annex B (informative)
K factors for some bending and compression elements

Values for buckling coefficients for elements of some common structural members are plotted in Figures B.1 to B.3 and approximate equations are included to aid calculation. The *K* factors given refer to the element of width b_1 in all cases and are thus termed K_1. Where K_1 is less than 4 in the case of a stiffened element and 0.425 in the case of an unstiffened element, the value 4 or 0.425 may be used.

In the case of uniformly compressed members the corresponding *K* factor for elements of width b_2, which is thus termed K_2, may be obtained as follows:

$$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$$

where

$h = b_2/b_1$

b_1, b_2 are the mid-line dimensions of the respective elements assuming rounded corners are replaced with the intersections of the flat elements as in **3.5.1**.

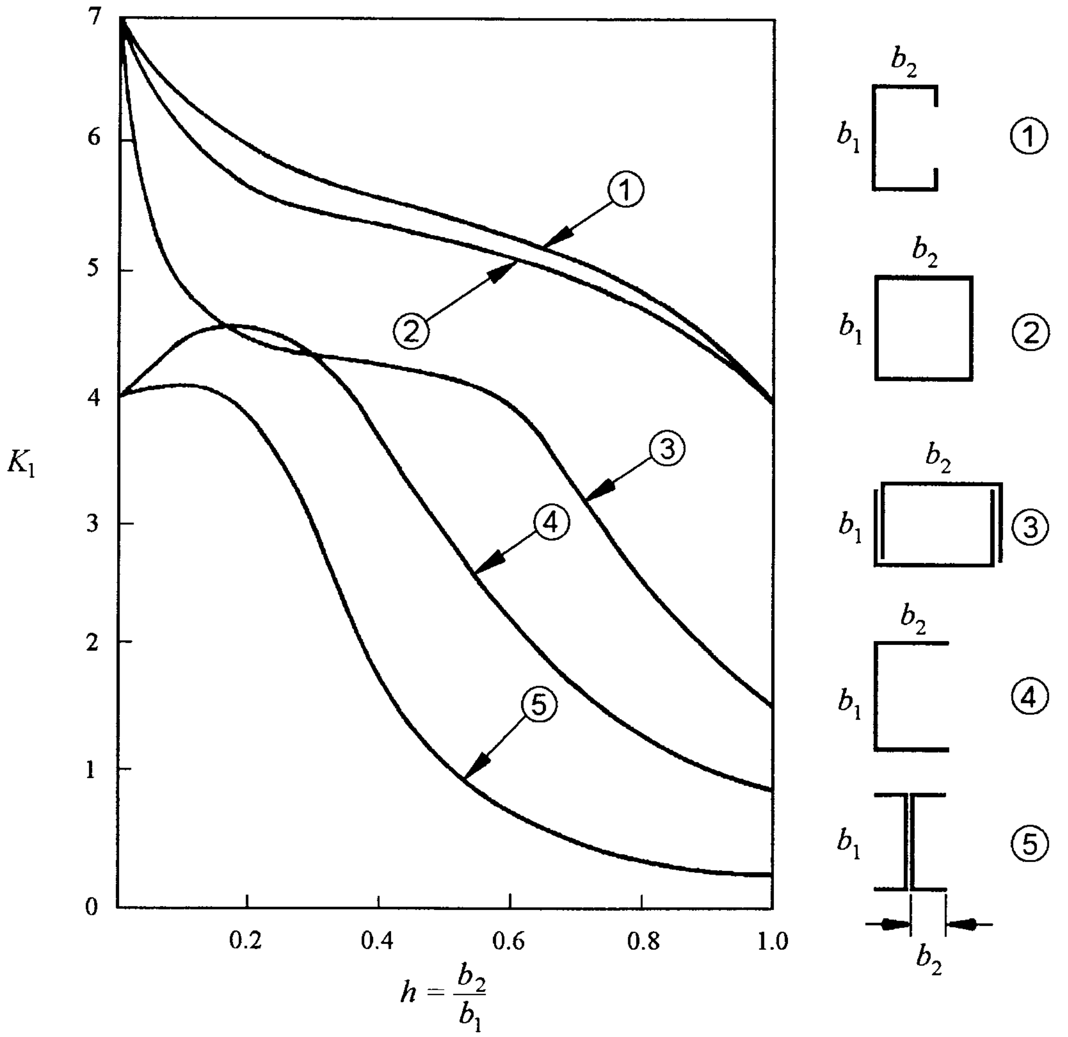
t_1, t_2 are the thicknesses of element widths b_1 and b_2 respectively.

Normally t_1 and t_2 are equal, but there are cases where the element has double thickness, e.g. element b_1 in case 3 of Figure B.1 where $t_1 = 2t_2$.

Where K_2 is less than 4 or 0.425 as the case may be then the values 4 or 0.425 may be used.

In the case of beams the K_1 factor refers to the element of width b_1, which is taken as the compression element except in the case of curve 2 in Figure B.3, in which cases the K_1 factor refers to the tip stress of the unstiffened bending element.

NOTE The curves and formulae given in Figures B.1, B.2 and B.3 apply only to sections having essentially similar geometries to those indicated in the figures, and only within the range of the geometries shown. Moreover, the edge stiffeners in lipped channels should conform to **4.6**, so that the adjacent flat compression element may be considered as stiffened. The use of the curves and formulae outside this range may lead to erroneous estimates of the *K* factors.



Curve 1 Lipped channel $K_1 \approx 7 - \dfrac{1.8h}{0.15 + h} - 1.43h^3$

Curve 2 Box with equal thickness sides $K_1 \approx 7 - \dfrac{2h}{0.11 + h} - 1.2h^3$

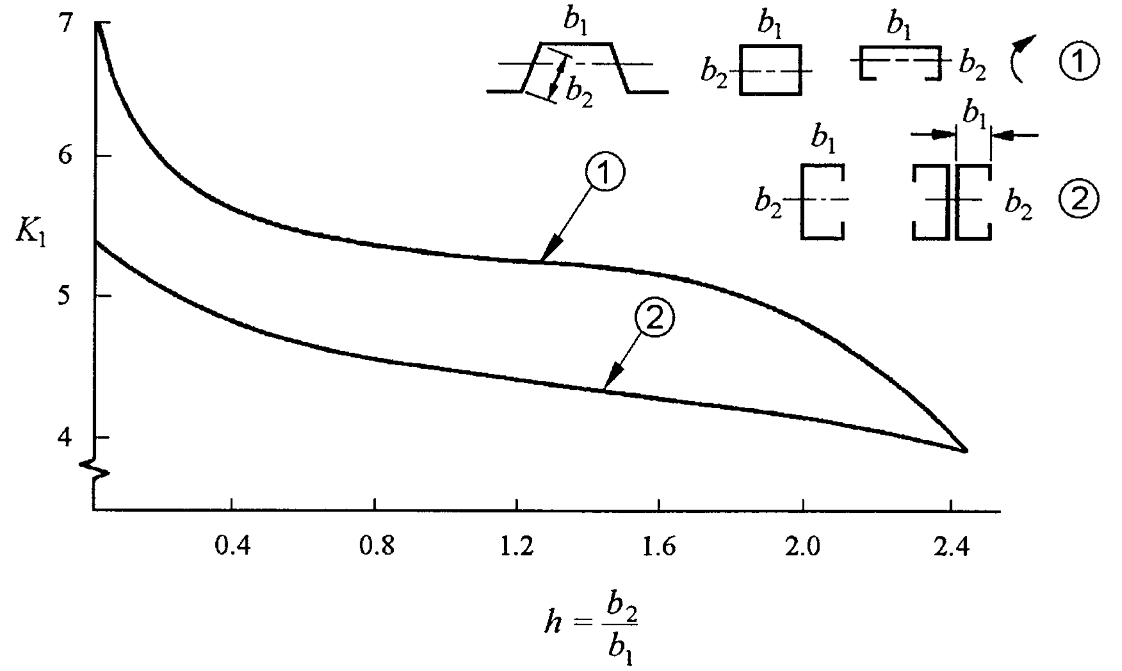
Curve 3 Box with unequal thickness sides $K_1 \approx 7 - 5.4h + 50h(h - 0.55)(h - 0.75)(h - 1)$

Curve 4 Plain channel $K_1 \approx \dfrac{2}{\beta} + \dfrac{2 + 4.8h}{\beta^2}$ where $\beta = (1 + 15h^3)^{1/2}$

Curve 5 I-section made from two plain channels fixed back-to-back

$K_1 \approx \dfrac{2}{\beta} + \dfrac{2 + h}{\beta^2}$, where $\beta = (1 + 90h^4)^{1/2}$

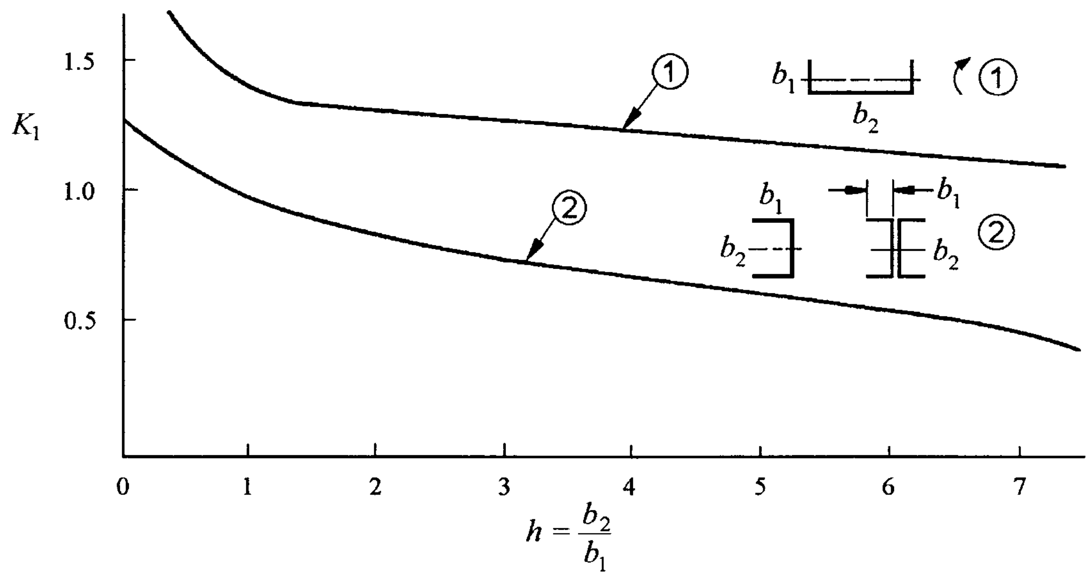
Figure B.1 — *K* factors for uniformly compressed members.



Curve 1 Element connection to webs on both edges $K_1 \approx 7 - \frac{1.8h}{0.15 + h} - 0.091h^3$

Curve 2 Element connected to web on one edge and lip on other edge $K_1 \approx 5.4 - \frac{1.4h}{0.6 + h} - 0.02h^3$

Figure B.2 — *K* factors for stiffened compression elements of beams



Curve 1 Channel flanges as bending elements $K_1 \approx \frac{3.4}{\left(2 + \frac{h}{1 + h}\right)}$

Curve 2 Unstiffened element $K_1 \approx 1.28 - \frac{0.8h}{2 + h} - 0.0025h^2$

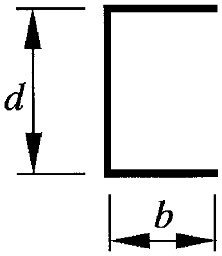
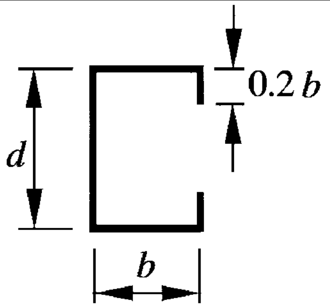
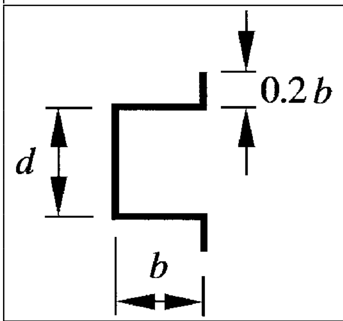
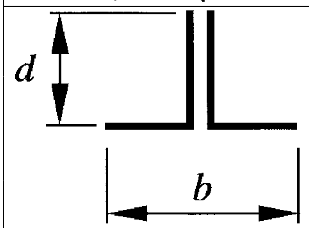
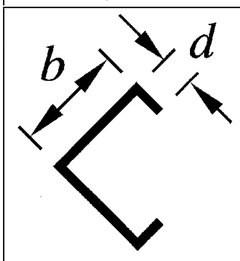
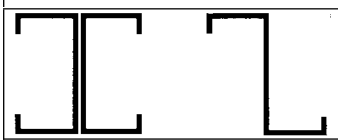
Figure B.3 — *K* factors for unstiffened elements of beams

Annex C (informative)

α factors for members in compression

Values of the α factor for members in compression and restrained as in **6.3.1** are given in Table C.1 For definition of symbols see **6.3.2**.

Table C.1 — α factors for members in compression

Section	d/b	$L_E/r_{min} \times t/b$													
		2	3	4	5	6	7	8	9	10	11	12	13	14	15
	**1.75**	1.02	1.00												
	1.50	1.15	1.11	1.06	1.02	1.00									
	1.25	1.33	1.27	1.21	1.14	1.08	1.02	1.00							
	1.00	1.60	1.51	1.41	1.31	1.23	1.15	1.09	1.04	1.00					
	0.75	2.04	1.87	1.70	1.56	1.45	1.36	1.30	1.24	1.20	1.17	1.14	1.12	1.10	1.09
	0.50	2.14	1.95	1.77	1.62	1.51	1.41	1.34	1.28	1.24	1.20	1.18	1.15	1.13	1.12
	**1.75**	1.08	1.06	1.03	1.00										
	1.50	1.21	1.18	1.14	1.10	1.06	1.02	1.00							
	1.25	1.39	1.35	1.30	1.24	1.18	1.13	1.08	1.04	1.00					
	1.00	1.65	1.58	1.51	1.43	1.36	1.30	1.24	1.19	1.15	1.11	1.08	1.06	1.04	1.02
	0.75	1.81	1.73	1.65	1.57	1.49	1.42	1.36	1.31	1.27	1.24	1.21	1.18	1.16	1.14
	0.50	1.67	1.62	1.56	1.50	1.45	1.39	1.34	1.30	1.27	1.24	1.21	1.19	1.17	1.15
	**2.25**	1.01	1.00												
	2.00	1.11	1.08	1.05	1.01	1.00									
	1.75	1.23	1.20	1.15	1.10	1.05	1.00								
	1.50	1.40	1.35	1.28	1.22	1.15	1.08	1.03	1.00						
	1.25	1.64	1.55	1.46	1.36	1.27	1.19	1.12	1.06	1.01	1.00				
	1.00	1.98	1.83	1.68	1.55	1.43	1.33	1.25	1.18	1.13	1.09	1.05	1.02	1.00	
	0.75	2.39	2.15	1.93	1.74	1.60	1.49	1.40	1.33	1.28	1.24	1.20	1.18	1.15	1.14
	0.50	2.37	2.13	1.91	1.73	1.59	1.48	1.40	1.33	1.28	1.24	1.20	1.18	1.15	1.14
	**0.25**	1.10	1.00												
	0.33	1.12	1.00												
	0.50	1.33	1.00												
	0.75	1.80	1.34	1.17	1.10	1.06	1.05	1.03	1.03	1.02	1.02	1.01	1.01	1.01	1.01
	1.00	2.30	1.64	1.36	1.22	1.14	1.10	1.08	1.06	1.05	1.04	1.03	1.03	1.02	1.02
	1.33	3.05	2.11	1.68	1.44	1.30	1.22	1.16	1.13	1.10	1.08	1.07	1.06	1.05	1.04
	**0.00**	2.55	1.72	1.31	1.07	1.00									
	0.20	2.15	1.72	1.41	1.20	1.04	1.00								
	α = 1 For all values														

Annex D (informative)

Warping constants for some common sections

Approximate values for the location of the shear centre and value of the warping constant, C_w, are given in Table D.1 for some common sections. Mid-line dimensions of the elements should be used in the expressions assuming that the radiused corners are replaced by the projected intersections of the flat elements as given in **3.5.1.**

Table D.1 — Location of shear centre and approximate values of warping constant C_w

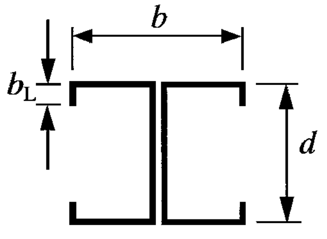
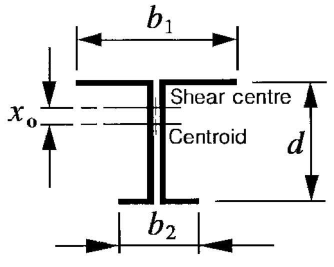
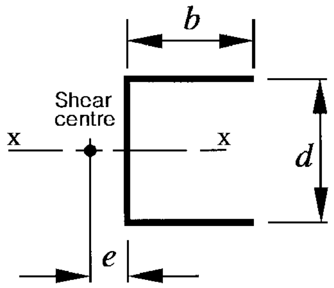
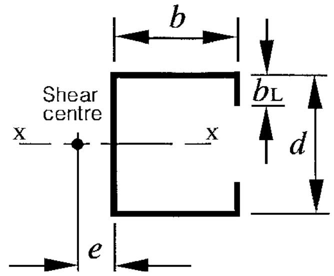
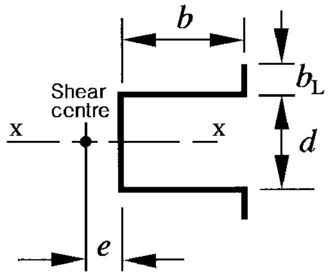
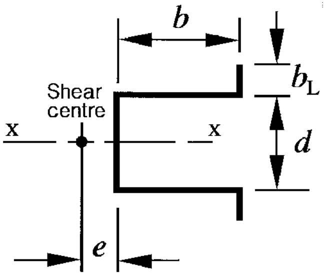
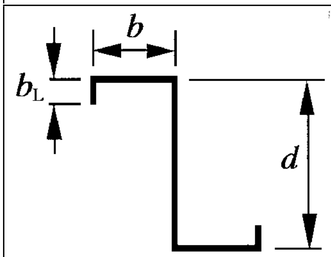
Section	Shear centre position	Warping constant, C_w
	Coincides with centroid	$\frac{b^3d^2t}{24}\left[1+6\frac{b_L}{b}\left\{1+2\frac{b_L}{d}+\frac{4}{3}\left(\frac{b_L}{d}\right)^2\right\}\right]$
	$x_0 = \frac{d^2\left(b_1^3-b_2^3\right)+db_1b_2(b_1^2-b_2^2)}{\left(b_1^3+b_2^3\right)(b_1+b_2+2d)}$	$\frac{b_1^3d^2t}{12\left\{1+\left(\frac{b_2}{b_1}\right)^3\right\}}$
	$e = \frac{3b^2}{6b+d}$	$\frac{b^3d^2t}{12}\left(\frac{2+3\frac{b}{d}}{1+6\frac{b}{d}}\right)$
	$e = \frac{d^2bb_Lt}{I_x}\left(\frac{1}{2}+\frac{b}{4b_L}-\frac{2}{3}\frac{b_L^2}{d^2}\right)$	$\frac{b^2t}{6}(4b_L^3+3d^2b_L-6db_L^2+bd^2)-I_xe^2$
	$e = \frac{d^2bb_Lt}{I_x}\left(\frac{1}{2}+\frac{b}{4b_L}-\frac{2}{3}\frac{b_L^2}{d^2}\right)$	$\frac{b^2t}{6}(4b_L^3+3d^2b_L-6db_L^2+bd^2)-I_xe^2$

Table D.1 — Location of shear centre and approximate values of warping constant C_w (*continued*)

Section	Shear centre position	Warping constant, C_w
	$e = \dfrac{bd^2t(3d - 2b)}{3\sqrt{2}I_x}$	$\dfrac{b^4d^3t^2}{18I_x}(4b + 3d)$
	Coincides with centroid	$\dfrac{X}{12(2b + d + 2b_L)}$ where $X = b^2t\left\{\begin{array}{l} d^2(b^2 + 2bd + 4bb_L + 6db_L) \\ + 4b_L^2(3db + 3d^2 + 4bb_L + 2db_L \\ + b_L^2) \end{array}\right\}$

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[2] EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK, BRUSSELS.(ECCS) Publication No. 21 (1983): European recommendations for steel construction: the design and testing of connections in steel sheeting and sections.[1)]

[3] EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK, BRUSSELS.(ECCS) Publication No. 42 (1983): European recommendations for steel construction: mechanical fasteners for use in steel sheeting and sections.[1)]

[1)] Available from Steel Construction Institute, Silwood Park, Ascot, Berkshire SL5 7QN. Tel: 01344 623345.

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