

Structural use of steelwork in building —

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

ICS 91.080.10

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- British Industrial Fasteners' Federation
- British Iron and Steel Producers' Association
- Cold Rolled Sections' Association
- Department of the Environment (Building Research Establishment)
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- Department of the Environment (Specialist Services)
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- Steel Construction Institute
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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 It replaces BS 5950: Part 5:1987 which is withdrawn.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.

The start and finish of text introduced or altered by Amendment No. 1:2006 is indicated in the text by tags **[A]** **[A1]**. Minor editorial changes are not tagged. The principal change introduced by the amendment is a revision of subclause **2.3.5**.

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 (including additional pages 7a to 7d), 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	e_s	Distance between the geometric neutral axis and the effective neutral axis of a section
<i>or</i>	Gross area of a cross-section	F_c	Applied axial compressive load
A_e	Effective net area of a section	F_s	Shear force (bolts)
A_{eff}	Effective area	F_t	Applied tensile load
A_n	Net area of a section	F_v	Shear force
A_{st}	Area of an intermediate stiffener	F_w	Concentrated load on a web
A_t	Tensile stress area of a bolt	f_a	Average stress in a flange
a	Effective throat size of a fillet weld	f_c	Compressive stress on the effective element
a_1	Net sectional area of connected elements	f_w	Applied compressive stress
a_2	Gross sectional area of unconnected elements	G	Shear modulus of steel
B	Overall width of an element	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
B_f	Half the overall flange width of an element	h	Vertical distance between two rows of connections in channel sections
b	Flat width of an element	<i>or</i>	As defined in annex B
b_{eff}	Effective width of a compression element	I	Second moment of area of a cross-section about its critical axis
b_{er}	Reduced effective width of a sub-element	I_{min}	Minimum required second moment of area of a stiffener
b_{eu}	Effective width of an unstiffened compression element	I_s	Second moment of area of a multiple stiffened element
C_b	Coefficient defining the variation of moments on a beam	I_x, I_y	Second moment of area of a cross-section about the x and y axes respectively
C_T	Constant depending on the geometry of a T-section	J	St Venant torsion constant of a section
C_W	Warping constant of a section	K	Buckling coefficient of an element
c	Distance from the end of a beam to the load or the reaction as shown in Tables 7 and 8	L	Length of a member between support points
D	Overall web depth	L_E	Effective length of a member
D_c	Depth of the compression zone in a web	L_w	Length of a weld
D_e	Equivalent depth of an intermediately stiffened web	M	Applied moment on a beam
D_w	Equivalent depth of a stiffened web	M_b	Buckling resistance moment
D_1	Distance between the centre line of an intermediate web stiffener and the compression flange	M_c	Moment capacity of a cross-section (as determined from 5.2.2)
d	Diameter of a bolt	M'_c	Design moment capacity of a section utilizing plastic bending capacity (see 5.2.3)
<i>or</i>	Diameter of a spot weld	M_{cr}	Critical bending moment causing local buckling in a beam
<i>or</i>	Flat width of an element as shown in Tables C.1 and D.1	M_{cx}	Moment capacity in bending about the x axis in the absence of F_c and M_y
<i>or</i>	As otherwise defined in a clause	M_{cy}	Moment capacity in bending about the y axis in the absence of F_c and M_x
d_e	Distance from the centre of a bolt to the end of an element	M_E	Elastic lateral buckling moment of a beam
d_p	Peripheral diameter of an arc spot weld or elongated arc spot weld	M_p	Plastic moment capacity of a section
d_r	Recommended tip diameter of an electrode	M_x, M_y	Moment about x and y axes respectively
d_s	Interface diameter of an arc spot weld or elongated arc spot weld	M_Y	Yield moment of a section
d_w	Visible diameter of an arc spot weld or width of elongated plug weld	N	Number of 90° bends in a section
E	Modulus of elasticity of steel	<i>or</i>	Length of bearing as shown in Tables 7 and 8
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	<i>or</i>	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	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
P_{Ex}, P_{Ey}	Elastic flexural buckling load (Euler load) for a column about x and y axes respectively	t	Net material thickness
P_{fs}	Shear capacity of a fastener	<i>or</i>	As otherwise defined in a clause
P_{ft}	Tensile capacity of a fastener	t_s	Equivalent thickness of a flat element to replace a multiple stiffened element for calculation purposes
P_s	Shear capacity of a connection	t_1, t_2	Thickness of thinner and thicker materials connected by spot welding or as defined in annex B
P_T	Torsional buckling load of a column	U_e	Nominal ultimate tensile strength of the electrode
P_{TF}	Torsional flexural buckling load of a column	U_f	Minimum tensile strength of fastener
P_t	Tensile capacity of a member or connection	U_s	Nominal ultimate tensile strength of steel (See 3.3.2)
P_v	Shear capacity or shear buckling resistance of a member	U_{ss}	Nominal ultimate tensile strength of the steel in the supporting members
P_w	Concentrated load resistance of a single web	u	Deflection of a flange towards the neutral axis due to flange curling
p_c	Compressive strength	W	Total distributed load on a purlin
p_{cr}	Local buckling stress of an element	W_d	Weight of cladding acting on a sheeting rail
p_0	Limiting compressive stress in a flat web	W_w	Wind load acting on a sheeting rail
p_s	Shear strength of a bolt	w	Flat width of a sub-element
p_v	Shear yield strength	<i>or</i>	Intensity of load on a beam
p_y	Design strength of steel	w_s	Equivalent width of a flat element to replace a multiple stiffened element for calculation purposes
p_w	Design strength of weld	x_o	Distance from the shear centre to the centroid of a section measured along the x axis of symmetry
Q	Factor defining the effective cross-sectional area of a section	Y_f	Minimum yield strength of a fastener
q_{cr}	Shear buckling strength of a web	Y_s	Nominal yield strength of steel (See 3.3.2)
$R_{d,i}$	Resistance predicted by the design expression for the specific parameters	Y_{sa}	Average yield strength of a cold formed section
R_{eH}	Upper yield strength of steel (as defined by BS EN 10002-1)	Y_{sac}	Modified average yield strength in the presence of local buckling
R_{eL}	Lower yield strength of steel (as defined by BS EN 10002-1)	y	Distance of a flange from the neutral axis
R_m	Tensile strength of steel (as defined by BS EN 10002-1)	Z_c	Compression modulus of a section in bending
$R_{p0.2}$	0.2 % proof stress (as defined by BS EN 10002-1)	α	Coefficient of linear thermal expansion
$R_{t0.5}$	Stress at 0.5 % total elongation (as defined by BS EN 10002-1)	<i>or</i>	Effective length multiplier for torsional flexural buckling
r	Inside bend radius	β	Ratio of end moments in a beam
<i>or</i>	Radius of gyration	<i>or</i>	Constant defined in 6.3.2
r_{cy}	Radius of gyration of a channel about its centroidal axis parallel to the web	γ_f	Overall load factor
r_I	Radius of gyration of a compound I-section	γ_I	Variability of loading factor
r_o	Polar radius of gyration of a section about the shear centre	γ_m	Material strength factor
r_x, r_y	Radii of gyration of a section about the x and y axes respectively	γ_p	Structural performance factor
S	Plastic modulus of a section	Δ	Beam deflection
S_o	Original cross-sectional area of the parallel length in a tensile test specimen (as defined in BS EN 10002-1)	Δ_c	Beam deflection at moment M_c
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	Δ_{cr}	Beam deflection at the point of local buckling
<i>or</i>	Leg length of a fillet weld	η	Perry coefficient
<i>or</i>	Standard deviation	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 $+35^{\circ}\text{C}$.

The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in 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 $\times$ dead load; plus
- 1.6 $\times$ 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.

A1 Text deleted – see pages 7a to 7d for updated text. A1

2.3.5 Structural integrity

NOTE This subclause refers in a number of places to regulations. For England and Wales, these are taken to be the Building Regulations 2000 and subsequent amendments [1].

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**, **2.3.5.3**, **2.3.5.4**, **2.3.5.5** and **2.3.5.6** apply to buildings.

NOTE 1 In general, light steel structures only offer lateral support to masonry cladding and the self weight of the masonry is transferred directly to the foundations. Disproportionate collapse of the masonry cladding during an accidental event is unacceptable and the masonry designer is expected to take special measures to ensure this does not occur.

Where regulations stipulate that certain buildings should be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause, light steel framed buildings designed as recommended in this standard (including the recommendations of **1.1**) may be assumed to meet this requirement provided that:

a) buildings of Class 1 that require special provision are designed to conform to **2.3.5.2**;

NOTE 2 Buildings of this class do not normally require any special provision.

b) buildings of Class 2A are designed to conform to **2.3.5.2**;

c) buildings of Class 2B are designed to conform to **2.3.5.2** and **2.3.5.3**;

d) buildings of Class 3 are designed to conform to **2.3.5.2** and **2.3.5.3** in addition to resisting the design conditions that can reasonably be foreseen as possible during the life of the buildings (identified by a systematic risk analysis of normal and abnormal hazards) such that any collapse is not disproportionate to the cause;

where:

— *Class 1* includes houses not exceeding four storeys; agricultural buildings; buildings into which people rarely go, provided no part of another building, or area where people do go, is closer than a distance of 1.5 times the building height;

— *Class 2A* includes five-storey single occupancy houses; hotels not exceeding four storeys; flats, apartments and other residential buildings not exceeding four storeys; offices not exceeding four storeys; industrial buildings not exceeding three storeys; retailing premises not exceeding three storeys of less than 2 000 m² floor area in each storey; single storey educational buildings; all buildings not exceeding two storeys to which members of the public are admitted and which contain floor areas not exceeding 2 000 m² floor area at each storey;

— *Class 2B* includes hotels, flats, apartments and other residential buildings greater than four storeys but not exceeding 15 storeys; educational buildings greater than one storey but not exceeding 15 storeys; retailing premises greater than three storeys but not exceeding 15 storeys; hospitals not exceeding three storeys; offices greater than four storeys but not exceeding 15 storeys; all buildings to which members of the public are admitted which contain floor areas exceeding 2 000 m² but less than 5 000 m² at each storey; car parking not exceeding six storeys;

— *Class 3* includes all buildings defined above as Class 2A and 2B that exceed the limits on area or number of storeys; grandstands accommodating more than 5 000 spectators; buildings containing hazardous substances or processes.

2.3.5.2 Minimum provisions for tying

Every building frame should be effectively tied together at each principal floor and roof level. All wall studs 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 joists or tie members. Members such as steel decking, provided for other purposes, may be utilized as ties. When members are checked as ties, other loading may be ignored. The tie force should be not less than that recommended in **2.3.5.3a**). Joists 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.

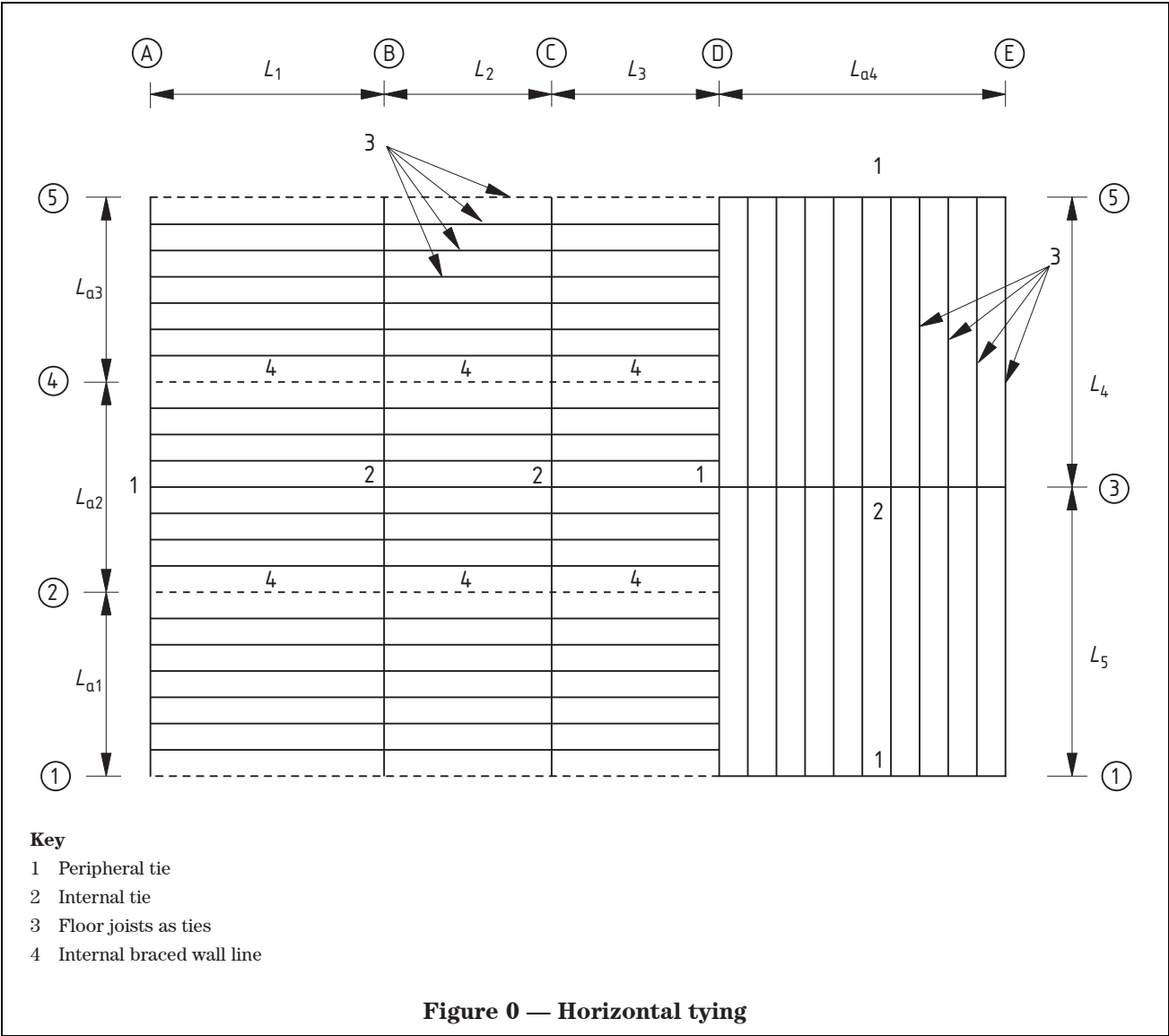
2.3.5.3 Limiting the effects of accidental removal of supports

Where regulations intend that certain buildings should be designed to limit the effect of accidental removal of supports, light steel-framed buildings designed as recommended in this standard (including the recommendations of **1.1** and **2.3.5.2**) may be assumed to meet this requirement provided that the following four conditions a) to d) are met. If any of these conditions cannot be met, the recommendations given in **2.3.5.4** should be followed. **(A1)**

A1 a) *General tying*. Horizontal tying should be arranged in continuous lines wherever practicable throughout each floor and roof level in two directions approximately at right angles, as shown in Figure 0.

Light steel members acting as ties, and their end connections, should be capable of resisting the following factored tensile loads, which need not be

- considered as additive to other loads acting on the members:
- for floor or roof ties¹⁾: $0.5n (1.4g_k + 1.6q_k) c_j L$ but not less than $5c_j$ kN;
 - for internal ties²⁾: $0.5n (1.4g_k + 1.6q_k) s_t L_a$ but not less than 15 kN;
 - for peripheral ties²⁾: $0.25n (1.4g_k + 1.6q_k) s_t L_a$ but not less than 15 kN;



¹⁾ The equations for roof and floor ties are based on the assumption that ties are provided by members at close centres, e.g. floor or roof joists.

²⁾ Peripheral and internal ties will generally be provided as a member at the head of the stud wall supporting the joists. **A1**

[A₁] where:

- c_j is the spacing of the floor joists in metres (m) (typically 0.4 m or 0.6 m);
- g_k is the specified dead load per unit area of the floor or roof in kilonewtons per square metre (kN/m²);
- L is the span of the joist under consideration in metres (m);
- L_a is the largest value, anywhere within the length of the tie, of the mean of the distance between any two adjacent braced walls or similar form of support³⁾ in metres (m);
- n is a factor related to the number of storeys in the structure as shown in Table 2A;
- q_k is the specified imposed floor or roof load per unit area in kilonewtons per square metre (kN/m²);
- s_t is the mean transverse spacing of the ties⁴⁾ in metres (m).

Table 2A — Values of factor n

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

This recommendation may be assumed to be satisfied if, in the absence of other loading, the member and its end connections are capable of resisting a tensile force equal to its end reaction under factored loads multiplied by n , or the larger end reaction multiplied by n if they are unequal.

b) *Tying of perimeter walls and columns.* A tying member at the periphery of the building, for example at the head of a wall, should be connected back to the rest of the structure. If the vertical loads are resisted by a distributed assembly of closely spaced elements, the tying members should be similarly distributed to ensure that the entire assembly is effectively tied. The forces for anchoring the vertical elements at the periphery should be based on the spacing of the elements and taken as 1 % of the factored vertical load in the element at that level. The minimum value of 15 kN does not apply to the wall studs. However, the floor to wall connections should be designed for the tying forces in condition a).

If the main structural elements are discrete columns, the horizontal ties anchoring the columns nearest to the edge of a floor or roof should be capable of resisting a factored tensile load, acting perpendicular to the edge, equal to the greater of the load for an internal tie [given in condition a)], or 1 % of the factored vertical dead and imposed load in the column acting at that level.

c) *Continuity of walls and columns.* All splices in primary vertical elements should be capable of resisting a tensile force of not less than two thirds of the factored design vertical dead and imposed load applied to the vertical element from the floor level below the splice.

Unless the steel frame is fully continuous in at least one direction, the primary vertical load bearing structural elements, whether discrete columns or panel walls, should be continuous at each beam-to-column/wall connection.

d) *Resistance to horizontal forces.* The system of bracing, whether discrete members or diaphragm panels, providing resistance to horizontal forces, should be distributed throughout the building such that, in each of two directions approximately at right angles, no substantial portion of the building is connected to a means of resisting horizontal forces at only one point.

2.3.5.4 Notional removal of supports

If any of the conditions 2.3.5.3a) to 2.3.5.3c) for tying and continuity cannot be met, the designer should check each storey to ensure that disproportionate collapse would not be precipitated by the notional removal, one at a time, of vertical load-bearing elements. If condition d) is not met, a check should be made in each storey in turn to ensure that disproportionate collapse would not be precipitated by the notional removal, one at a time, of each element of the systems providing resistance to horizontal forces.

Where the light steel frame has discrete columns, the approach should be identical to that in BS 5950-1.

For steel stud walls, the situation is considered to be similar to that of masonry, and requires consideration of the optional removal of a wall panel. The length of an external stud wall that is considered to be removed is the length measured between vertical lateral supports. For internal steel stud walls, the length considered to be removed is 2.25 times the storey height.

³⁾ For grids A, B, C and D, L_a would be the larger of $(L_{a1} + L_{a2})/2$ or $(L_{a2} + L_{a3})/2$. For grids 1, 3 and 5, $L_a = L_{a4}$.

⁴⁾ For gridline A, $s_t = L_1/2$; for gridline B, $s_t = (L_1 + L_2)/2$. [A₁]

[A1] In all cases, the test for localization of damage is that the portion of the building at risk of collapse should not exceed 15 % of the area of the storey or 70 m² (whichever is less) within that storey and the immediately adjoining storey above. A similar area of the immediately adjoining storey below could be loaded with debris from above. Hence, up to a total of 140 m² of floor area may be assumed to be at risk of collapse. If the notional removal of a column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area, that column or element should be designed as a key element, as recommended in **2.3.5.5**.

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

2.3.5.5 Key elements

In a multi-storey building that is required by regulations to be designed to avoid disproportionate collapse, a member that is recommended in **2.3.5.4** to be designed as a key element should be designed for the accidental loading specified in BS 6399-1.

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

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

In this check the effects of ordinary loads should also be considered, to the same extent and with the same partial factor γ_f as recommended in **2.3.5.4**. **[A1]**

^{A1} Text deleted – see pages 7a to 7d for updated text. ^{A1}

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.

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.

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 350 G	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, $R_{p0.2}$, or the stress at 0.5 % total elongation, $R_{t0.5}$, 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} , $R_{p0.2}$, $R_{t0.5}$ 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} , $R_{p0.2}$, $R_{t0.5}$ 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/mm².

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} , $R_{p0.2}$, $R_{t0.5}$ 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	$E = 205 \text{ kN/mm}^2$
Shear modulus	$G = 79 \text{ kN/mm}^2$
Poisson ratio	$\nu = 0.30$
Coefficient of linear thermal expansion	$\alpha = 12 \times 10^{-6} \text{ per } ^\circ\text{C}$

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/mm²);
- A is the gross area of the cross-section in square millimetres (mm²).

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:

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

$$Y_{sac} = Y_{sa}$$

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

$$Y_{sac} = Y_s$$

For unstiffened elements:

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

$$Y_{sac} = Y_{sa}$$

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

$$Y_{sac} = Y_s$$

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.

