

Structural use of steelwork in building —

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

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- British Constructional Steelwork Association Ltd.
- British Railways Board
- British Steel Industry
- Department of the Environment (Building Research Establishment)
- Department of the Environment (Housing and Construction Industries)
- Department of the Environment (Property Services Agency)
- Health and Safety Executive
- Institution of Civil Engineers
- Institution of Structural Engineers
- Royal Institute of British Architects
- Steel Construction Institute
- Welding Institute

The following bodies were also represented in the drafting of the standard, through subcommittees and panels:

- Cold Rolled Sections Association
- Construction Industry Research and Information Association
- Department of the Environment [Building Research Establishment (Fire Research Station)]
- Metal Roof Deck Association
- Storage Equipment Manufacturers' Association

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Publications referred to	Inside back cover

Foreword

This Part of BS 5950 has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. 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.

It 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 3¹⁾: Code of practice for design in composite construction;*
- *Part 4: Code of practice for design of floors with profiled steel sheeting;*
- *Part 5: Code of practice for design of cold formed sections;*
- *Part 6¹⁾: Code of practice for design of light gauge sheeting, decking and cladding;*
- *Part 7¹⁾: Specification for materials and workmanship: cold formed sections and sheeting;*
- *Part 8¹⁾: Code of practice for fire protection of structural steelwork;*
- *Part 9¹⁾: Code of practice for stressed skin design.*

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, at the same time, to be as self contained as possible.

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 in obtaining simple and rapid analyses, it is possible in many situations to use the various tables and graphs provided instead of calculation via the equations.

During the course of drafting, consultation and examination of a large number of other design documents and specifications was continuously carried out. Close attention was paid to BS 5950-1 and to the documents dealing with cold formed steel published by the following.

American Iron and Steel Institute
European Convention for Constructional Steelwork
Swedish Institute of Steel Construction
Standards Association of Australia
Canadian Standards Association
Association Française de Normalisation

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.

¹⁾ In preparation

The full list of organizations who have taken part in the work of the Technical Committee is given on the back cover. The Chairman of the Committee is Mr P R Brett and the following people have made a particular contribution in the drafting of the code.

Professor E R Bryan	Chairman of Drafting panel
Mr M D C Bratcher	
Professor J M Davies	
Dr R G Dawson	
Mr T W Giddings	
Mr R D Homer	
Dr J Rhodes	
Mr J S Tempest	
Professor A C Walker	
Mr A D Weller	

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 72, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

Section 1. General

1.0 Introduction

1.0.1 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 one 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 design.

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

While 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.

1.0.2 Overall stability

The designer responsible for the overall stability of the structure should ensure the compatibility of design and details of parts and components. There should be no doubt of this responsibility for overall stability when some or all of the design and details are not made by the same designer.

1.0.3 Accuracy of calculation

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

1.1 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 sections of thickness up to 8 mm.

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:

- a) by joining together two previously formed open sections by continuous welding;
- b) from a single flat strip, by forming the corners to make a box, and continuously welding the longitudinal joint.

Welded cold formed square and rectangular hollow sections complying with BS 6363 may also be designed in accordance with this Part of BS 5950.

NOTE 1 Requirements for materials and construction will be covered by BS 5950-7, which is in preparation.

NOTE 2 The titles of the publications referred to in this standard are listed on the inside back cover.

1.2 Definitions

For the purposes of this Part of BS 5950 the following definitions apply.

1.2.1

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

1.2.2

effective length

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

1.2.3

effective width

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

1.2.4

element

distinct portion of the section of a member

NOTE Types of elements are defined in 1.2.5 to 1.2.8.

1.2.5

stiffened element

a flat element adequately supported at both longitudinal edges

1.2.6

unstiffened element

a flat element adequately supported at only one longitudinal edge

1.2.7

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

1.2.8

multiple stiffened element

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

1.2.9

lateral buckling

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

NOTE This is also known as lateral-torsional buckling.

1.2.10

buckling resistance

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

1.2.11

local buckling

buckling of the walls of elements of a section characterized by a formation of waves or ripples along the member. It is treated separately from overall buckling resistance and modifies the capacity of cross sections

1.2.12

flexural buckling

buckling of a column due to flexure

1.2.13

torsional buckling

buckling of a column by twisting

1.2.14

torsional flexural buckling

buckling of a column by combined flexure and twisting

1.2.15

limit state

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

1.2.16**strength**

resistance to failure; specifically, limiting value for stress

1.3 Major symbols

A	Area
	<i>or</i> Gross area of a cross section
A_e	Effective 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	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 Table 7 and Table 8
D	Overall web depth
	<i>or</i> As shown in Table 13 and Table 14
D_e	Equivalent depth of an intermediately stiffened web
D_2	Distance between the centre line of an intermediate web stiffener and the tension element
d	Diameter of a bolt
	<i>or</i> Diameter of a spot weld
	<i>or</i> As otherwise defined in a clause
d_e	Distance from the centre of a bolt to the end of an element
d_{eff}	Effective diameter of a circular plug or elongated plug weld
d_r	Recommended tip diameter of an electrode
d_w	Visible diameter of a circular plug or elongated plug weld
E	Modulus of elasticity of steel
e	Distance between a load and a reaction as shown in Table 7 and Table 8
	<i>or</i> As shown in Table 14
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	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 Appendix 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
M_{cr}	Critical bending moment to cause 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 Table 7 and Table 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 bolt or Shear capacity of a spot weld
P_T	Torsional buckling load of a column
P_T	Tensile capacity of a member or connection
P_{TF}	Torsional flexural buckling load of a column
P_v	Shear capacity or shear buckling resistance

P_w	Concentrated load resistance of a single web
p_c	Compressive strength
p_{cr}	Local buckling stress of an element
p_o	Limiting compressive stress in a flat web
p_v	Shear 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	Inside bend radius
	<i>or</i> 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 an 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	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
	<i>or</i> Leg length of a fillet weld
	<i>or</i> 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
	<i>or</i> 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 Appendix B
U_s	Ultimate tensile strength of steel
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
	<i>or</i> 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_s	Yield strength of steel
Y_{sa}	Average yield strength of a cold formed section
Y_{sac}	Modified average yield strength in the presence of local buckling
$\bar{y}$	Distance of a flange from the neutral axis
Z_c	Compression modulus of a section in bending
α	Coefficient of linear thermal expansion

	<i>or</i> Effective length multiplier for torsional flexural buckling
β	Ratio of end moments in a beam
	<i>or</i> Constant defined in 6.3.2
γ_f	Overall load factor
γ_m	Material strength 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's ratio

Section 2. Limit state design

2.1 General principles and design methods

2.1.1 General

Structures should be designed by considering the limit states at which they would become unfit for their 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 will be 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.

Table 1 — Limit states relevant to steel structures

Ultimate		Serviceability	
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		

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

material strength:	γ_m
loading:	γ_l
structural performance:	γ_p

In this Part of BS 5950 γ_m is taken as 1.0 (see 3.3.2). Depending on the type of load, values of γ_l and γ_p are assigned. The product of γ_l and γ_p 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).

A detailed breakdown of γ factors is given in Appendix A of BS 5950-1:1985.

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 such as to realize 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.

It is necessary to maintain stability against sway and the provisions of 2.3.2.3 apply.

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 sidesway 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.
- 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 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, so as to ensure composite behaviour of the member or structure.

NOTE Composite design will be covered by BS 5950-3, which is in preparation.

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 Stressed skin design will be covered by BS 5950-9, which is in preparation.

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 natural frequency of the vibration and the structure should be checked, and consideration 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

Reference should be made to BS 6399-1 and CP 3: Chapter V-2 for the determination of the dead, imposed and wind loads²⁾. In the case of purlins and sheeting rails, local wind pressure and suction need not be considered.

2.2.3 Accidental loading

If in the design it is necessary to consider the probable effects of excessive loads caused by misuse or accident, the nature and magnitude of the accidental loads should be assessed using adequate engineering analysis and based on realistic estimates of any impact forces involved.

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⁾ Reference should be made to BS 5502 for loads on agricultural buildings.

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 average 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 state 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.

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

2.3.1.2 Overhead cranes. If overhead cranes are provided then reference should be made to BS 5950-1 for detailed design.

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.

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.

The notional forces should be assumed to act on all structures in any one orthogonal direction at a time and should be applied to each roof and floor level or their equivalent. They should be taken as acting simultaneously with vertical loads taken as:

$1.4 \times$ (unfactored dead load); and

$1.3 \times$ (unfactored vertical imposed load)

The notional force should not:

- a) be applied when considering overturning;
- b) be combined with horizontal loads;
- c) be combined with temperature effects;
- d) be taken to contribute to net shear 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 state clearly 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 and should accommodate all the imposed forces. 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 $-15\text{ }^{\circ}\text{C}$ 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 1.0 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 clause.

2.3.5.3 Additional recommendations for certain buildings. Where it is stipulated by appropriate regulations that buildings should be designed to localize accidental damage, a reference should be made to BS 5950-1 for the additional recommendations.

2.4 Servicability 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.

Table 3 — Deflection limits

<i>(a) Deflection of beams due to unfactored imposed loads</i>	
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
<i>(b) Deflection of columns other than portal frames due to unfactored imposed and wind loads</i>	
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.	

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 or DD 24 in determining suitable treatment.

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

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 steel supplied to BS 1449, BS 2989 and BS 4360. Other steels may be used, subject to approval of the engineer, provided due allowance is made for variation in properties, including ductility.

NOTE Requirements for materials will be covered by BS 5950-7, which is in preparation.

3.3.2 Strength of steel

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

Y_s is the minimum yield strength or in the case of material with no clearly defined yield, either the 0.2 % proof stress or the stress at 0.5 % total elongation;

U_s is the minimum ultimate tensile strength.

Y_s may normally be taken as specified in the relevant British Standard. For such cases, values of yield, ultimate and design strengths for some of the more common types of steel are given in Table 4.

Where the sheet material is supplied with a certificated minimum yield stress, Y_s may be taken as the certificated value and used for the formed section. Alternatively, for any steel, the strength of the sheet material or the formed section may be determined by testing in accordance with section 10.

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's 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 $\leq 5t$ (fractions of 90° bends should be counted as fractions of N);

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

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

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

Table 4 — Yield, ultimate and design strengths

British Standard	Grade	Minimum yield strength, Y_s	Minimum ultimate strength, U_s	Design strength, p_y
		N/mm ²	N/mm ²	N/mm ²
1449-1	HR3	(170)	(290)	170
	HR4	(170)	(280)	170
	34/20	200	340	200
	37/23	230	370	230
	43/25	250	430	250
	50/35	350	500	350
	40/30	300	400	300
	43/35	350	430	350
	40F30	300	400	300
	43F35	350	430	350
2989	Z1 and Z2	(210)	(270)	210
	Z22	220	(290)	220
	Z25	250	(350)	250
	Z28	280	(390)	280
	Z35	350	(450)	350
4360	40B, 40C	235	340	235
	43A, 43B, 43C	275	430	275
	50B, 50C	355	490	355
NOTE Figures in brackets in BS 1449-1 and BS 2989 are non-mandatory and are given for guidance only.				

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

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 section, and the actual round corners are replaced by intersections of the flat elements.

NOTE Reference may be made to BS 2994 for some profiles.

3.5.2 Gross section properties

When calculating gross 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.

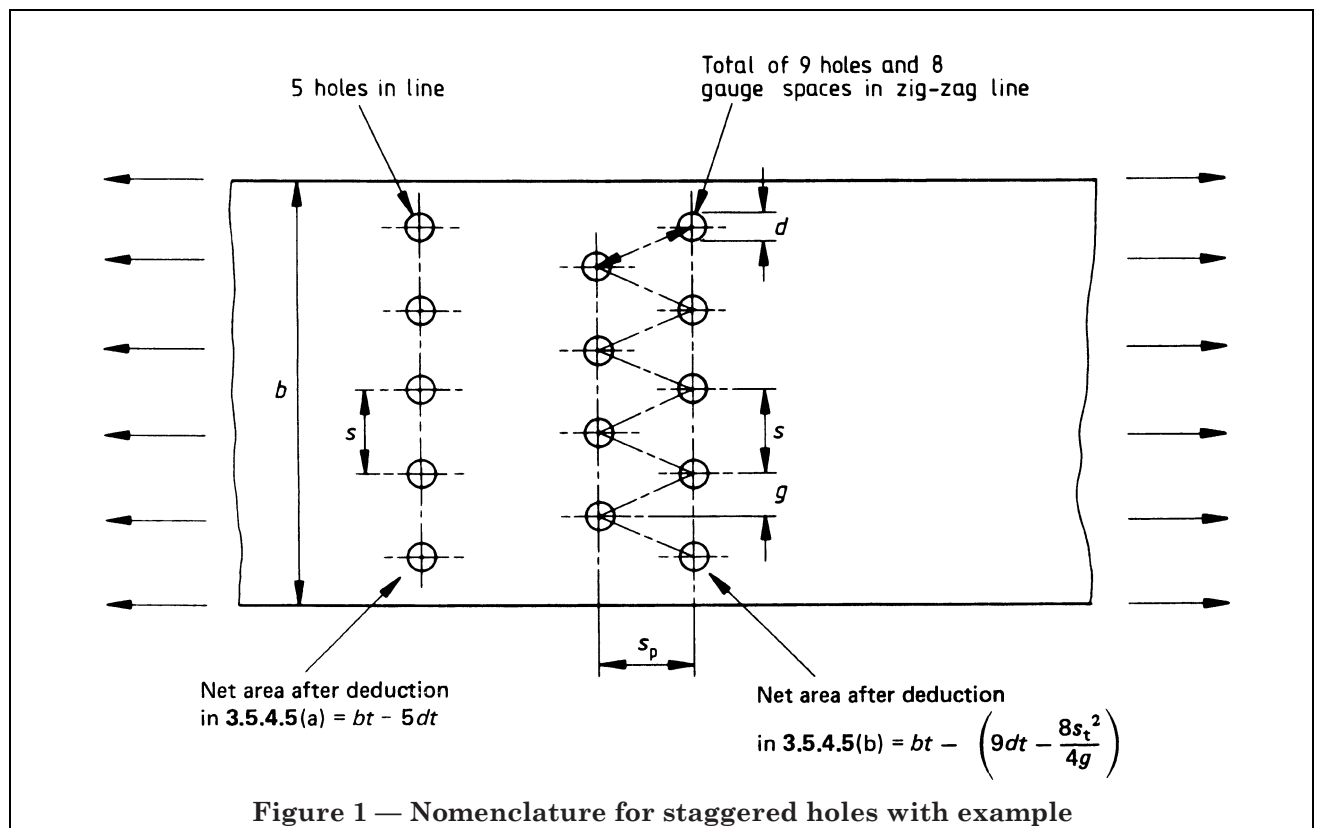
3.5.4.4 Holes not staggered. The area to be deducted from the gross sectional area should be the maximum sum of the sectional areas of the holes in any cross section at right angles to the direction of stress in the member.

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

- the deduction for non-staggered holes;
- the sum of the sectional areas of all holes in any zig-zag line extending progressively across the member or part of the member, less $s_p^2 t/4g$ 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).



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 satisfying 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 $f_c/p_{\text{cr}} < 0.123$

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

for $f_c/p_{\text{cr}} \geq 0.123$

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

where

f_c is the compressive stress on the effective element;

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

$$p_{\text{cr}} = 185\,000 K \left(\frac{t}{b} \right)^2$$

where

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