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British Standard

Structural use of steelwork in building

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

Aciers de construction

Partie 1. Code de bonne pratique pour la conception des ouvrages de construction
simple continue : profilés laminés à chaud

Stahlkonstruktion im Bauwesen

Teil 1. Leitfaden für die Verwendung warmgewalzter Profilstähle in
einfacher und Endlosbauweise

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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 3* 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 in cold formed sections
- Part 6* Code of practice for design in light gauge sheeting, decking and cladding
- Part 7* Specification for materials and workmanship: cold formed sections
- Part 8* Code of practice for design of fire protection for structural steelwork
- Part 9* Code of practice for stressed skin design

Part 1 gives recommendations for the design of structural steelwork 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.

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

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

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 D W Lazenby and the following people have made a particular contribution in the drafting of the code.

Mr J C H Finlinson	Past Committee Chairman
Mr B L Hurst	Chairman of Drafting Panel
Prof. P J Dowling	
Mr R Dixon	
Mr H J Gettins	
Mr G Heath	
Mr F H Needham	
Mr G Parkes	
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Dr D Tordoff	
Mr A D Weller	
Dr F J Whitbread	

Compliance with a British Standard does not of itself confer immunity from legal obligations.

*In preparation.

Section one. 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. Reference should be made to 2.4.5.

While the ultimate strength requirements 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 with hot rolled steel sections, flats, plates and hollow sections in buildings and allied structures.

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

NOTE 2. 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 beam. A member predominately subject to bending.

1.2.2 brittle fracture. Brittle failure of steel at low temperature.

1.2.3 buckling resistance. Limit of force or moment which a member can withstand without buckling.

1.2.4 built-up. Constructed by interconnecting more than one plate to form a single member.

1.2.5 cantilever. A beam which is fixed at one end and is free to deflect at the other.

1.2.6 capacity. Limit of force or moment which may be applied without causing failure due to yielding or rupture.

1.2.7 column. A vertical member of a structure carrying axial load and possibly moments.

1.2.8 compact cross section. A cross section which can develop the plastic moment capacity of the section but in which local buckling prevents rotation at constant moment.

1.2.9 compound section. Constructed by interconnecting one or more sections or plates and sections to form a single member.

1.2.10 dead load. All loads of constant magnitude and position that act permanently, including self weight.

1.2.11 design strength. The yield strength of the material multiplied by the appropriate partial factor. See 3.1.1.

1.2.12 dynamic load. Part of an imposed load resulting from motion.

1.2.13 edge distance. Distance from the centre of a fastener hole to the nearest edge of an element.

1.2.14 effective length. Length between points of effective restraint of a member multiplied by a factor to take account of the end conditions and loading.

1.2.15 elastic design. Design which assumes no redistribution of moments due to plastic rotation of a section throughout the structure.

1.2.16 empirical method. Simplified method of design justified by experience or testing.

1.2.17 end distance. Distance from the centre of a fastener hole to the edge of an element parallel to the direction in which the fastener bears.

1.2.18 factored load. Specified load multiplied by the relevant partial factor.

1.2.19 fatigue. Damage to a structural member caused by repeated application of stresses that are insufficient to cause failure by a single application.

1.2.20 foundation. Part of a structure which distributes load directly to the ground.

1.2.21 friction grip connection. A bolted connection which relies on friction to transmit shear between components.

1.2.22 H-section. A section with one central web and two equal flanges which has an overall depth not greater than 1.2 times the width of the flange.

1.2.23 hybrid. Composed of elements of more than one strength grade of steel.

1.2.24 I-section. Section with central web and two equal flanges which has an overall depth greater than 1.2 times the width of the flange.

1.2.25 imposed load. Load on a structure or member, other than wind load, produced by the external environment and intended occupancy or use.

1.2.26 instability. Inability to carry further load due to vanishing stiffness.

1.2.27 lateral restraint

For a beam. Restraint which prevents lateral movement of the compression flange.

For a compression member. Restraint which prevents lateral movement of the member in a particular plane.

1.2.28 longitudinal. Along the length of the member.

1.2.29 pattern loading. Loading arranged in such a manner as to give the most severe effect on a particular element.

1.2.30 pitch. Distance between centres of fasteners lying in the direction of stress.

1.2.31 plastic cross section. A cross section which can develop a plastic hinge with sufficient rotation capacity to allow redistribution of bending moments within the structure.

1.2.32 plastic design. Design method assuming redistribution of moment in continuous construction.

1.2.33 plastic moment. Moment capacity allowing for redistribution of stress within a cross section.

1.2.34 pretensioned fastener. Fastener tensioned to a specified proportion of its proof stress before connected components are loaded.

1.2.35 semi-compact cross section. A cross section in which the stress in the extreme fibres should be limited to yield because local buckling would prevent development of the plastic moment capacity in the section.

1.2.36 serviceability limit states. Those limit states which when exceeded can lead to the structure being unfit for its intended use.

1.2.37 slender cross section. A cross section in which yield of the extreme fibres cannot be attained because of premature local buckling.

1.2.38 slenderness. The effective length divided by the radius of gyration.

1.2.39 slip resistance. Limit of shear that can be applied before slip occurs in a friction grip connection.

1.2.40 stability. Resistance of the structure or part of the structure to overturning or overall failure.

1.2.41 strength. Resistance to failure by yielding or buckling.

1.2.42 strut. A member of a structure carrying predominantly compressive axial load.

1.2.43 subframe. Part of a larger frame.

1.2.44 transverse. Direction perpendicular to the stronger of the rectangular axes of the member.

1.2.45 ultimate limit state. That state which if exceeded can cause collapse of part or whole of the structure.

1.3 Major symbols

A	Area
A_e	Effective area
A_g	Gross area
A_s	Shear area (bolts)
A_t	Tensile stress area (bolts)
A_v	Shear area (sections)
a	Spacing of transverse stiffeners or Effective throat size of weld
B	Breadth
b	Outstand or Width of panel
b_1	Stiff bearing length
C_v	Charpy impact value
D	Depth of section or Diameter of section or Diameter of hole
d	Depth of web or Nominal diameter of fastener
E	Modulus of elasticity of steel
e	End distance
F_c	Compressive force due to axial load
F_s	Shear force (bolts)
F_t	Tensile force
F_v	Average shear force (sections)
f_c	Compressive stress due to axial load
f_v	Shear stress
G	Shear modulus of steel
H	Warping constant of section
h	Storey height
I_x	Second moment of area about the major axis
I_y	Second moment of area about the minor axis
J	Torsion constant of section
L	Length of span
L_E	Effective length
M	Larger end moment
M_{ax}, M_{ay}	Maximum buckling moment about the major or minor axis in the presence of axial load
M_b	Buckling resistance moment (lateral torsional)
M_{cx}, M_{cy}	Moment capacity of section about the major and minor axes in the presence of axial load

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M_E	Elastic critical moment
M_o	Midspan moment on a simply supported span equal to the unrestrained length
M_{rx}, M_{ry}	Reduced moment capacity of the section about the major and minor axes in the presence of axial load
M_x, M_y	Applied moment about the major and minor axes
$\bar{M}_x, \bar{M}_y$	Equivalent uniform moment about the major and minor axes
m	Equivalent uniform moment factor
n	Slenderness correction factor
P_{bb}	Bearing capacity of a bolt
P_{bg}	Bearing capacity of parts connected by friction grip fasteners
P_{bs}	Bearing capacity of parts connected by ordinary bolts
P_{cx}, P_{cy}	Compression resistance about the major and minor axes
P_s	Shear capacity of a bolt
P_{sL}	Slip resistance provided by a friction grip fastener
P_t	Tension capacity of a member or fastener
P_v	Shear capacity of a section
p_b	Bending strength
p_{bb}	Bearing strength of a bolt
p_{bg}	Bearing strength of parts connected by friction grip fasteners
p_{bs}	Bearing strength of parts connected by ordinary bolts
p_c	Compressive strength
p_E	Euler strength
p_s	Shear strength of a bolt
p_t	Tension strength of bolt
p_w	Design strength of a fillet weld
p_y	Design strength of steel
q_b	Basic shear strength of a web panel
q_{cr}	Critical shear strength of web panel
q_e	Elastic critical shear strength of web panel
q_f	Flange dependent shear strength factor
r_x, r_y	Radius of gyration of a member about its major and minor axes
S_x, S_y	Plastic modulus about the major and minor axes
s	Leg length of a fillet weld
T	Thickness of a flange or leg
t	Thickness of a web
or	As otherwise defined in a clause
U_s	Specified minimum ultimate tensile strength of the steel
u	Buckling parameter of the section
V_b	Shear buckling resistance of stiffened web utilizing tension field action
V_{cr}	Shear buckling resistance of stiffened or unstiffened web without utilizing tension field action
v	Slenderness factor for beam
x	Torsional index of section
Y_s	Specified minimum yield strength of steel

Z_x, Z_y	Elastic modulus about major and minor axes
α	Coefficient of linear thermal expansion
α_e	Modular ratio
β	Ratio of smaller to larger end moment
γ_f	Overall load factor
γ_l	Load variation factor, i.e. function of γ_{l1} and γ_{l2}
γ_m	Material strength factor
γ_o	Ratio M/M_o , i.e. the ratio of the larger end moment to the midspan moment on a simply supported span
δ	Deflection
ϵ	Constant $\left(\frac{275}{p_y}\right)^{1/2}$
λ	Slenderness, i.e. the effective length divided by the radius of gyration
λ_{cr}	Elastic critical load factor
λ_{LO}	Limiting equivalent slenderness
λ_{LT}	Equivalent slenderness
λ_o	Limiting slenderness
μ	Slip factor
ν	Poisson's ratio

1.4 Other materials

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

NOTE. Attention is drawn to the necessity of referring to local regulations.

1.5 Design documents

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

The design documents should show the assumed behaviour of the structure, the design assumptions and whether the forces and reactions included are factored or unfactored.

1.6 Detailing

The connections between members should withstand the forces and moments to which they will be subjected, without undue deformation and without invalidating the design assumptions.

The detailing of the connections should take account of possible dimensional variations due to rolling margins and fabrication variations, leading to some degree of lack of fit.

1.7 References to BS 5400

In BS 5400 the breakdown of partial safety factors, the assessment of material strengths, etc. are different, and these differences should be recognized.

Section two. Limit state design

2.1 General principles and design methods

2.1.1 Limit state concept

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.

Examples of limit states relevant to steel structures are given in table 1.

Table 1. Limit states	
Ultimate	Serviceability
1 Strength (including general yielding, rupture, buckling and transformation into a mechanism)	5 Deflection
2 Stability against overturning and sway	6 Vibration (e.g. wind induced oscillation)
3 Fracture due to fatigue	7 Repairable damage due to fatigue
4 Brittle fracture	8 Corrosion and durability

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

Material strength: γ_m

Loading: γ_L

Structural performance: γ_p

In this code the material factor, γ_m , is taken as 1.0 (see 3.1.1). 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.

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 2.1.2.5.

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 2.4.2.3 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 should be designed for the actual moment capacity of the connection not the assumed moment.

2.1.2.5 Experimental verification. Where design of a structure or element by calculation in accordance with any of the preceding methods is not practicable, or is inappropriate, the strength, stability and stiffness may be confirmed by loading tests in accordance with section seven.

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2.2 Loading

2.2.1 General

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

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

2.2.2 Dead, imposed and wind loading

Reference should be made to BS 6399 : Part 1 and CP 3 : Chapter V : Part 2 for the determination of the dead, imposed and wind loads.

2.2.3 Dynamic loads and impact effects

These may be determined from BS 6399 : Part 1 in the case of cranes.

It is recommended that values for cranes of loading class Q3 and Q4 as defined in BS 2573 : Part 1 should be established in consultation with the crane manufacturer.

2.3 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 internal steelwork varies from -5°C to $+35^{\circ}\text{C}$. The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in other conditions, and in locations abroad subjected to different temperature ranges.

2.4 Ultimate limit states

2.4.1 Limit state of strength

2.4.1.1 General. In checking the strength 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 part or effect under consideration.

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

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

For cranes on outdoor gantries the wind loads on the gantry and supporting structure should be obtained from:

- (a) BS 2573 : Part 1, for cranes in the working condition;
- (b) CP 3 : Chapter V : Part 2, for cranes which are not working.

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 or crane load	1.2
Forces due to temperature effects	1.2
<i>Crane loading effects</i>	
Vertical load	1.6
Vertical load acting with horizontal loads (crabbing or surge)	1.4
Horizontal load	1.6
Horizontal load acting with vertical	1.4
Crane load acting with wind load*	1.2
*When considering wind or imposed load and crane loading acting together the value of γ_f for dead load may be taken as 1.2.	

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

2.4.2 Stability limit state

2.4.2.1 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.

2.4.2.2 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 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.

2.4.2.3 Sway stability. All structures, including portions between expansion joints, should be adequately stiff 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 at each roof and floor level or their equivalent. They should be taken as acting simultaneously with vertical loads taken as:

- 1.4 x (unfactored dead load); and
- 1.3 x (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 staircase, 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 (see 1.5).

2.4.2.4 Foundation design. The design of foundations should be in accordance with CP 2004 and should accommodate all the forces imposed on them. Attention should be given to the method of connecting the steel superstructure to the foundations and the anchorage of any holding down bolts as recommended in 6.7.

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

2.4.3 Fatigue

Fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to fluctuations in wind loading need not be considered but account should be taken of wind induced oscillations.

In the design of crane supporting structures only those structures which support cranes of utilization classes U7 to U9 as defined in BS 2573 need be checked for fatigue by reference to the fatigue design clauses of that standard.

When designing for fatigue a γ_f factor of 1.0 should be used.

2.4.4 Brittle fracture

2.4.4.1 General. Brittle fracture need not be considered except in locations subject to tensile stresses in service due to applied axial load or moment.

Where such tension exists then the maximum thickness may be determined from 2.4.4.2 if the service temperature does not fall below that normal in the UK, taken as -5°C for internal conditions and -15°C for external conditions.

Where the steel is subjected to lower temperatures or where the steel grade or thickness used is not covered by table 4 then the energy absorption should comply with 2.4.4.3, which may also be used in place of table 4.

Where severe restraint conditions occur due to welding then reference should be made to BS 5400 or specialist advice.

The recommendations of this clause do not apply to grade 43A base plates (see 4.13).

2.4.4.2 Maximum thickness. The maximum thickness should not exceed the value given in table 4 for the value of K determined from table 3.

2.4.4.3 Energy absorption. The Charpy impact value, C_v , in joules, at the service temperature should not be less than:

$$\frac{Y_s t}{710K}$$

where

Y_s is the minimum yield strength of the material (in N/mm^2);

t is the thickness of material from which the specimen is taken (in mm);

K is determined from table 3.

Table 3. Factor K for location of material and tensile stress

Tensile stress due to factored loads at the location being considered	Welded location	Unreamed punched holes	Non-welded location	Drilled or reamed holes
$\text{N/mm}^2 \leq 100$	2	2	2	2
> 100	1	1	2	2

2.4.5 Structural integrity

2.4.5.1 Requirements for all structures. All structures should follow the principles given in 1.0.1. The additional requirements in 2.4.5.2 to 2.4.5.5 apply to buildings.

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Table 4. Maximum thickness of parts subjected to applied tensile stress
(see note 1)

BS 4360 grade	Sections (except hollow sections (see note 2)) and flat bars		Plates, wide flats and universal wide flats	
Values for $K = 1$	Internal	External	Internal	External
	mm	mm	mm	mm
43A	25	15	25	15
43B see note 3	30	20	30	20
43C	50	40	50	40
43D	50	50	75	75
43E	50	50	75	75
50A	16	10	16	10
50B	20	12	20	12
50C see note 4	40	27	75	55
50D see note 4	40	40	75	75
55C	19	16	19	16
55E	63	63	63	63
WR 50A	12	12	12	12
WR 50B	45	27	45	27
WR 50C	50	50	50	50
Values for $K = 2$	Internal	External	Internal	External
	mm	mm	mm	mm
43A	50	30	50	30
43B see note 3	50	40	50	40
43C	50	50	50	50
43D	50	50	75	75
43E	50	50	75	75
50A	32	20	32	20
50B	40	25	40	25
50C see note 4	40	40	75	75
50D see note 4	40	40	75	75
55C	19	19	19	19
55E	63	63	63	63
WR 50A	12	12	12	12
WR 50B	50	50	50	50
WR 50C	50	50	50	50

NOTE 1. For sections with flanges the thickness is the flange thickness defined in BS 4.

NOTE 2. All BS 4360 grades to which hollow sections are normally supplied may be used up to 16 mm thickness.

NOTE 3. The optional impact test requirement of BS 4360 has to be specified when ordering material of grade 43B.

NOTE 4. The maximum thickness for wide flats and universal wide flats of grades 50C and 50D is 50 mm.

2.4.5.2 Requirements for all buildings. Every building frame should be effectively tied together at each principal floor and roof level. All columns should be effectively restrained 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. Where possible these should be arranged in continuous lines as close as practicable to the columns and to each edge. At re-entrant corners the peripheral tie should be anchored into the steel framework.

Ties may be either steel members or steel reinforcement embedded in concrete or masonry provided that they are properly anchored to the steel framework.

Steel members provided for other purposes may be utilized as ties. When 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.

All ties and their end connections should be of a standard of robustness commensurate with the structure of which

they form a part and should be capable of carrying a factored tensile load of not less than 75 kN at floors or 40 kN at roof level.

Ties are not required at a roof level where steelwork supports cladding weighing not more than 0.7 kN/m² and carries roof loads only.

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.

2.4.5.3 Additional requirements for certain multi-storey buildings. Local or national regulations may stipulate that tall multi-storey buildings be designed to localize accidental damage.

Steel-framed buildings which satisfy the recommendations of 1.0.1 and 2.4.5.2 may be assumed to meet this requirement provided that the five additional conditions given below are met.

A tall multi-storey building which is required to be designed to localize accidental damage but which does not satisfy these five additional conditions should be checked as recommended in 2.4.5.4.

(a) *Sway resistance.* The means of providing sway resistance as recommended in 2.4.2.3 (i.e. steel bracing, rigid joints, shear walls, staircase and lift cores, etc.) should be sufficiently distributed throughout the building so that no substantial portion of the structural frame is solely reliant on a single plane of bracing in each orthogonal direction.

(b) *Tying.* The ties referred to in 2.4.5.2 should be arranged in continuous lines wherever practicable throughout each floor and roof level in two directions approximately at right angles. These and their connections should be checked for the following factored tensile loads, which need not be considered as additive to other loads.

(1) *Generally.* $0.5w_f s_t L_a$ for any internal ties and $0.25w_f s_t L_a$ for edge ties but not less than 75 kN for floors or 40 kN at roof level

where

w_f is the total factored dead and imposed load per unit area of floor or roof;

s_t is the mean transverse spacing of the ties;

L_a is the greatest distance, in the direction of the tie, between adjacent lines of columns or other vertical supports.

(2) *At the periphery.* Ties anchoring columns at the periphery of a floor or roof should be checked for the force given in (1) but not less than 1 % of the factored vertical load in the column at that level.

(c) *Columns.* All column splices should be capable of resisting a tensile force of not less than two-thirds of factored vertical load applied to the column from the floor level next below the splice.

Except where the steel framework is of continuous construction in at least one direction, the columns should be carried through at each beam-to-column connection.

(d) *Integrity.* Any beam which carries a column should be checked, together with the members which support it, for localization of damage as recommended in 2.4.5.4.

(e) *Floor units.* Where precast concrete or other heavy floor or roof units are used they should be effectively anchored in the direction of their span either to each other over a support or directly to their supports as recommended in BS 8110.

2.4.5.4 Localization of damage. Where required by 2.4.5.3 a building should be checked to see whether at each storey in turn any single column, or beam carrying a column, could be removed without causing collapse of more than a limited portion of the building local to the member concerned. Where the removal of one of these members would cause failure in excess of appropriate limits that member should be designed as a key element as recommended in 2.4.5.5.

For the purposes of this provision, it may be assumed that substantial permanent deformation of members and their connections is acceptable.

In this check only one-third of the ordinary wind load and one-third of the ordinary imposed load need be considered 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. The γ_f factor should be taken as 1.05 except that when considering overturning the dead load supplying the restoring moment should be multiplied by a γ_f factor of 0.9.

2.4.5.5 Key elements. Where it is required by 2.4.5.4 to design a member as a key element, the accidental loading should not be less than that stipulated.

Accidental loads should be applied to members from appropriate directions together with the reactions from other building components attached to the member which are subject to the same loading but limited to the ultimate strength of these components or their connections.

In this check the effects of ordinary loads should also be considered, to the same extent and with the same γ_f factors as recommended in 2.4.5.4 for localization of damage.

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

2.5 Serviceability limit states

2.5.1 Deflection

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

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

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Table 5. Deflection limits other than for pitched roof portal frames

(a) Deflection on beams due to unfactored imposed load

Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
All other beams	Span/200
Purlins and sheeting rails	See 4.12.2

(b) Horizontal 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

(c) Crane gantry girders

Vertical deflection due to static wheel loads	Span/600
Horizontal deflection (calculated on the top flange properties alone) due to crane surge	Span/500

NOTE 1. On low-pitched and flat roofs the possibility of ponding needs consideration.

NOTE 2. For limiting deflections in runway beams refer to BS 2853.

Table 5 gives recommended limitations for certain structural members. Circumstances may arise where greater or lesser values would be more appropriate. Other members may also need a deflection limitation to be established, e.g. sway bracing.

Generally the serviceability loads may 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. In the case of crane surge and wind, only the greater effect of either need be considered in any load combination.

2.5.2 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 in determining adequate methods of protection where applicable. Weather resistant steel to BS 4360 may also be used.

Section three. Properties of materials and section properties

3.1 General

3.1.1 Strength of steel

This standard covers the design of structures fabricated from steels supplied to BS 4360; other steels may be used provided due allowance is made for variations in properties, including ductility.

The design strength, p_y , may be taken as $1.0 Y_s$ but not greater than $0.84 U_s$ where Y_s and U_s are the minimum yield strength and the minimum ultimate tensile strength respectively as specified in BS 4360 (or agreed with steelmaker).

For the more common types of steel p_y may be obtained from table 6.

For rolled sections the thickness should be taken as the specified flange thickness from BS 4.

Table 6. Design strengths, p_y , for steel to BS 4360		
BS 4360 Grade	Thickness, less than or equal to	Sections, plates and hollow sections p_y
43 A, B and C	mm	N/mm ²
	16	275
	40	265
	100	245
50 B and C	16	355
	63	340
	100	325
55 C	16	450
	25	430
	40	415

Additional properties of steel are required for use in plastic design and reference should be made to 5.3.3.

3.1.2 Other properties of steel

The following values for the elastic properties should be used:

Modulus of elasticity	$E = 205 \text{ kN/mm}^2$
Poisson's ratio	$\nu = 0.30$
Coefficient of linear thermal expansion	$\alpha = 12 \times 10^{-6} \text{ per } ^\circ\text{C}$

3.1.3 Steel castings and forgings

Steel castings and forgings may be used for components in bearings, junctions and other similar parts. Castings should comply with BS 3100 and forgings with BS 29. Design strengths corresponding to grade 43 steel may be adopted where no other information is available.

3.2 Welds and fasteners

3.2.1 Welding consumables

All welding consumables (i.e. electrode wires, filler rods, flux, shielding gas, etc.) should comply with BS 5135.

3.2.2 Ordinary bolts, nuts and washers

Bolts and nuts should comply with BS 4190 or BS 3692. Bolts and nuts of material complying with BS 3692 made to the size and tolerances of BS 4190 are permitted.

Countersunk or cup headed bolts should comply with BS 4933. High strength friction grip bolts complying with BS 4395 may be used untorqued. Nuts should be of a strength grade equal to or higher than the grade of bolt.

Washers should comply with BS 4320.

3.2.3 Friction grip fasteners

High strength friction grip bolts and associated nuts and washers should comply with BS 4395.

Other types of friction grip fasteners may be used provided they have mechanical properties not inferior to bolts complying with BS 4395 and provided they can be reliably tightened to the minimum shank tensions specified in BS 4604.

3.3 Section properties

3.3.1 Gross section

Gross section properties should be determined using the specified size and profile of the member or elements, but allowance should be made for openings larger than required for fasteners. Battens or splices should not be included.

3.3.2 Net area

The net area of a section or element of a section should be taken as its gross area less deductions for fastener holes as given in 3.4.

3.3.3 Effective area at connections

The effective area, A_e , of each element of a member at a connection, where fastener holes occur may be taken as K_e times its net area, but not more than its gross area, where for steels complying with BS 4360:

$$K_e = 1.2 \text{ for grade 40 or 43}$$

$$K_e = 1.1 \text{ for grade 50 or WR 50}$$

$$K_e = 1.0 \text{ for grade 55}$$

$$\text{for other steels, } K_e = 0.75 \frac{U_s}{Y_s}$$

$$\text{but } \leq 1.2$$

where

U_s is the specified minimum ultimate tensile strength;

Y_s is the specified minimum yield strength.

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3.4 Deductions for holes

3.4.1 Hole area

In deducting for holes for fasteners (including countersunk holes), the gross area of the hole in the plane of its axis and not that of the fastener should be deducted.

3.4.2 Holes not staggered

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

3.4.3 Staggered holes

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

- (a) the deduction for non-staggered holes (see 3.4.2);
- (b) 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.

For sections such as angles with holes in both legs the gauge should be taken as the sum of the back marks to each hole, less the leg thickness. See figure 2.

3.5 Limiting proportions of cross sections

3.5.1 General

Local buckling can be avoided by limiting the width to thickness ratios of each element of a cross section subject to compression due to moment or axial load. Elements and cross sections are classified as plastic, compact, semi-compact or slender. Cross sections may be composed of elements of different classes.

3.5.2 Classification of cross sections

Class 1. Plastic cross sections are those in which all elements subject to compression comply with the values given in table 7 for plastic elements. A plastic hinge can be developed with sufficient rotation capacity to allow redistribution of moments within the structure.

Only class 1 sections may be used for plastic design.

Class 2. Compact cross sections are those in which all elements subject to compression comply with the values given in table 7 for compact elements. The full plastic moment capacity can be developed but local buckling may prevent development of a plastic hinge with sufficient rotation capacity to permit plastic design.

Class 2 sections can be used without restriction except for plastic design.

Class 3. Semi-compact sections are those in which all elements subject to compression comply with the values given in table 7 for semi-compact elements. The stress at the extreme fibres can reach the design strength but local buckling may prevent the development of the full plastic moment.

Class 3 sections are subject to limitations on their capacity which are given in section four.

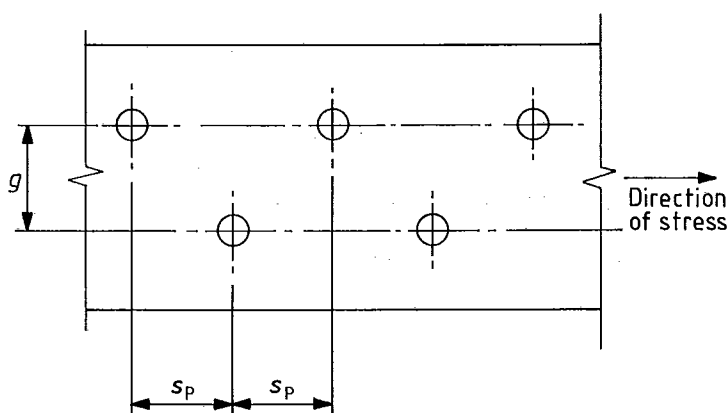


Figure 1. Staggered holes

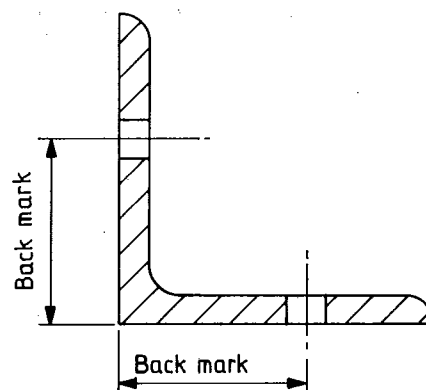


Figure 2. Angles with holes in both legs

Table 7. Limiting width to thickness ratios

(Elements which exceed these limits are to be taken as class 4, slender cross sections.)

Type of element	Type of section	Class of section		
		(1) Plastic	(2) Compact	(3) Semi-compact
Outstand element of compression flange	Built-up by welding	$\frac{b}{T} \leq 7.5\epsilon$	$\frac{b}{T} \leq 8.5\epsilon$	$\frac{b}{T} \leq 13\epsilon$
	Rolled sections	$\frac{b}{T} \leq 8.5\epsilon$	$\frac{b}{T} \leq 9.5\epsilon$	$\frac{b}{T} \leq 15\epsilon$
Internal element of compression flange	Built-up by welding	$\frac{b}{T} \leq 23\epsilon$	$\frac{b}{T} \leq 25\epsilon$	$\frac{b}{T} \leq 28\epsilon$
	Rolled sections	$\frac{b}{T} \leq 26\epsilon$	$\frac{b}{T} \leq 32\epsilon$	$\frac{b}{T} \leq 39\epsilon$
Web, with neutral axis at mid-depth	All sections	$\frac{d}{t} \leq 79\epsilon$	$\frac{d}{t} \leq 98\epsilon$	$\frac{d}{t} \leq 120\epsilon$
Web, generally	All sections	$\frac{d}{t} \leq \frac{79\epsilon}{0.4+0.6\alpha}$	$\frac{d}{t} \leq \frac{98\epsilon}{\alpha}$	See 3.5.4
Web subject to compression throughout	Built-up by welding			$\frac{d}{t} \leq 28\epsilon$
	Rolled sections			$\frac{d}{t} \leq 39\epsilon$
Legs of single angle and double angle members with components separated	Rolled angle sections	$\frac{b}{t} \leq 8.5\epsilon$ and $\frac{d}{t} \leq 8.5\epsilon$	$\frac{b}{t} \leq 9.5\epsilon$ and $\frac{d}{t} \leq 9.5\epsilon$	$\frac{b}{t}$ and $\frac{d}{t} \leq 15\epsilon$ and $\frac{b+d}{t} \leq 23\epsilon$
Outstand legs of double angle members with angles in contact back-to-back	Rolled angle sections	$\frac{b}{t} \leq 8.5\epsilon$ $\frac{d}{t} \leq 8.5\epsilon$	$\frac{b}{t} \leq 9.5\epsilon$ $\frac{d}{t} \leq 9.5\epsilon$	$\frac{b}{t} \leq 15\epsilon$ $\frac{d}{t} \leq 15\epsilon$
Stems of T-sections	T-section	$\frac{d}{t} \leq 8.5\epsilon$	$\frac{d}{t} \leq 9.5\epsilon$	$\frac{d}{t} \leq 19\epsilon$
Circular tube subject to moment or axial compression	CHS or built-up by welding	$\frac{D}{t} \leq 40\epsilon^2$	$\frac{D}{t} \leq 57\epsilon^2$	$\frac{D}{t} \leq 80\epsilon^2$

NOTE 1. Dimensions b , D , d , T , t are as defined in figure 3.

$$\alpha = \frac{2y_c}{d}$$

where y_c is the distance from the plastic neutral axis to the edge of the web connected to the compression flange. But if $\alpha > 2$ the section should be taken as having compression throughout.NOTE 2. Check webs for shear buckling in accordance with 4.4 when $d/t > 63\epsilon$.

$$\text{NOTE 3. } \epsilon = \left(\frac{275}{p_y} \right)^{1/2}$$

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Class 4. Slender sections are those which contain slender elements subject to compression due to moment or axial load. Local buckling may prevent the stress in a slender section from reaching the design strength.

Design of class 4 sections is considered in 3.6.

3.5.3 Classification of elements

Flat elements in a cross section are either:

- (a) *internal elements* attached on both longitudinal edges to other elements or to longitudinal stiffeners which are connected at suitable intervals to transverse stiffeners, or
- (b) *outside elements* attached on only one horizontal edge to an adjacent element, the other edge being free.

Flat elements are generally of constant thickness. Tapered elements may be treated as flat elements having the average thickness defined in BS 4.

Elements may be classified as plastic, compact or semi-compact if they meet the limits given in table 7. Those which do not meet the limits for semi-compact elements are classified as slender.

3.5.4 Webs of semi-compact sections

The limiting width to thickness ratio, d/t , for the web of a semi-compact section is given by the following:

- (a) where $R > 0.5$

- (1) for sections built-up by welding

$$\frac{d}{t} \leq (104 - 76R)\epsilon$$

- (2) for rolled sections

$$\frac{d}{t} \leq (93 - 54R)\epsilon$$

- (b) when $R \leq 0.5$ but > -0.45

$$\frac{d}{t} \leq \frac{120\epsilon}{1 + 1.6R}$$

in which R is the ratio of the mean longitudinal stress in the web to p_y , compression being taken as positive and tension as negative.

3.5.5 Compound flanges

The following width to thickness ratios should be considered:

- (a) the outstand of the compound flange compared to the thickness of the original flange;
- (b) the internal width of each added plate between the lines of welds or fasteners which connect it to the original flange, compared to its own thickness;
- (c) any outstand of an added plate beyond the lines of welds or fasteners which connect it to the original flange, compared to its own thickness.

3.5.6 Longitudinally stiffened flanges

The unsupported width, b , of a flange or part of a flange, which is effectively supported along both edges either by a web or a longitudinal stiffener, should be taken as the width between adjacent lines of welds or fasteners connecting it to the supporting elements. In rolled sections b should be measured as shown in figure 3.

Where the free edge of the flange is supported by a longitudinal stiffener, the flange thickness, T , should not be less than $b/20$ unless the edge stiffener is itself supported at suitable intervals.

Where the flange is stiffened by transverse as well as longitudinal stiffeners or a web, the thickness, T , should not be less than $1/100$ of the smaller panel dimension a or b , where a is a spacing of transverse stiffeners on the flange.

3.6 Slender cross sections

3.6.1 General

Local buckling may become the design criterion when the proportions of elements in a cross section exceed those given for semi-compact elements in table 7. Such cross sections are defined as slender and their capacity is limited.

3.6.2 Sections with thin webs required to carry shear

Where thin webs which have a d/t ratio > 63 are required to carry shear the capacity of the cross section should be calculated from 4.4.4.2.

3.6.3 Webs subject to moments and axial loads and circular hollow sections

In the absence of a more rigorous method of analysis a value of the design strength, p_y , should be assumed such that the limiting proportions for semi-compact sections are met (see table 7). The same reduced value of p_y should be used for that element throughout the design of the section whenever it is in compression, except that such reduction need not be made in the design of connections to that element.

3.6.4 Other sections

Where a slender outstand is in compression the design strength, p_y , should be reduced by the factor given in table 8. The same reduced value of p_y should be used for that element throughout the design of the section whenever it is in compression except that such reduction need not be made in the design of connections to that element.

Table 8. Stress reduction factors for slender cross sections

Type of element	Type of section	Stress reduction factor
Outstand element of compression flange	Built up by welding	$\frac{10}{\frac{b}{T\epsilon} - 3}$
	Rolled sections	$\frac{11}{\frac{b}{T\epsilon} - 4}$
Internal element of compression flange	Build up by welding	$\frac{21}{\frac{b}{T\epsilon} - 7}$
	Rolled sections	$\frac{31}{\frac{b}{T\epsilon} - 8}$
Legs of single angle and double angle members with components separated	Rolled angle section	The lesser of $\frac{11}{\frac{d}{T\epsilon} - 4}$ and $\frac{19}{\frac{(b+d)}{T\epsilon} - 4}$
Outstand legs of double angle members with angles in contact back-to-back	Rolled angle	$\frac{11}{\frac{b}{T\epsilon} - 4}$
Stems of T-sections		$\frac{14}{\frac{d}{T\epsilon} - 5}$

NOTE 1. Dimensions b , d , t , T are as defined in figure 3.NOTE 2. $\epsilon = \left(\frac{275}{p_y}\right)^{1/2}$

NOTE 3. The strength of slender cross sections may be obtained from 3.6.2, 3.6.3 and 3.6.4 as appropriate. Alternatively the more rigorous approach given in BS 5950 : Part 5 (in preparation) may be used.

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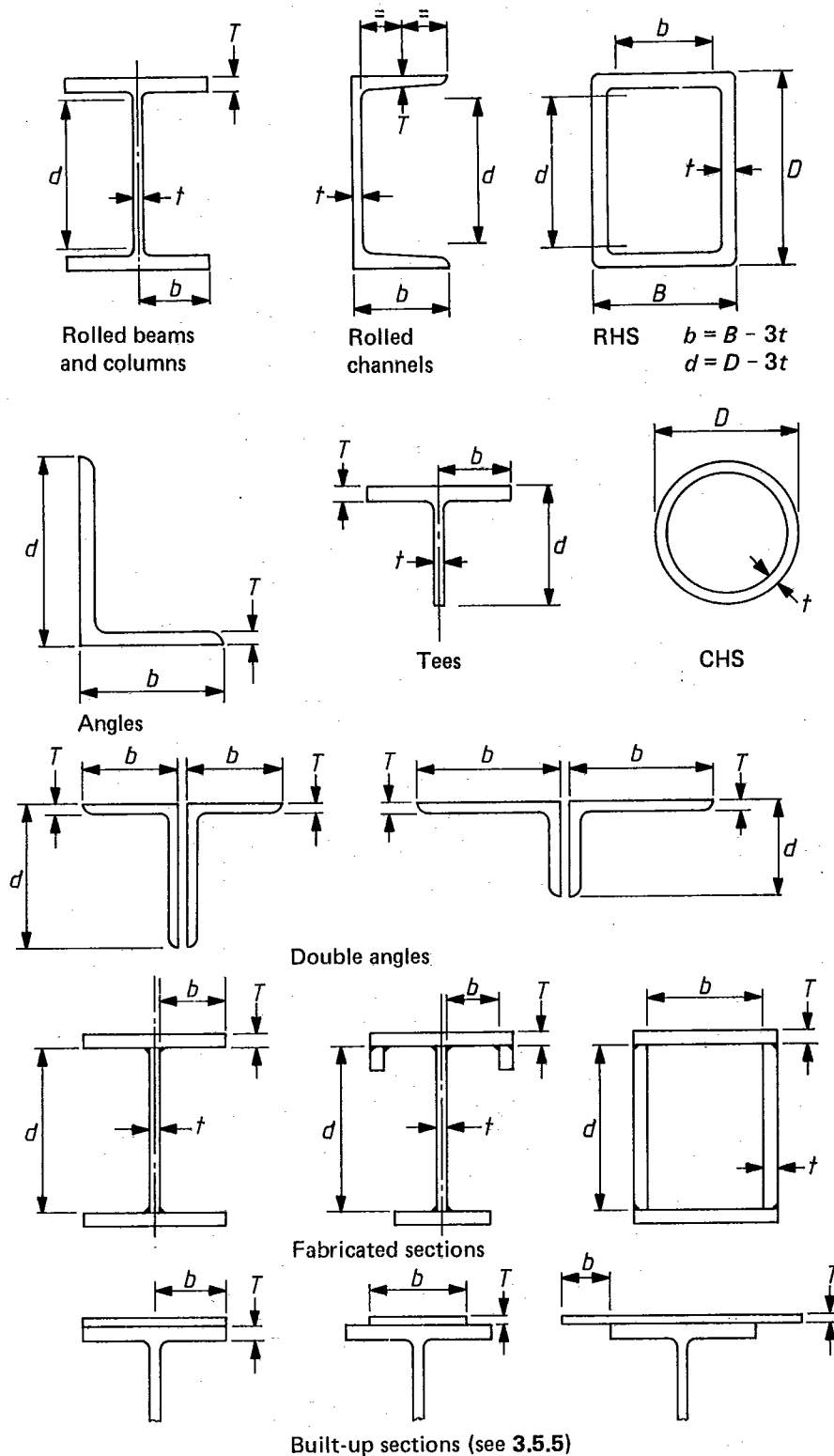


Figure 3. Dimensions of sections

Section four. Design of structural elements

4.1 General

4.1.1 Scope

This section gives rules for the design of simple elements or elements comprising parts of frames.

4.1.2 Class of cross section

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

4.1.3 Design strength

The design strength should be obtained from table 6. The reduction of 20 N/mm² given in 4.7.5 for welded elements in compression does not apply to the element when it is checked for other effects such as bending, shear and connections.

4.2 Members in bending

4.2.1 General

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

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

4.2.1.3 General conditions. All members in bending should meet the following conditions.

- (a) At critical points the combination of maximum moment and co-existent shear, and the combination of maximum shear and co-existent moment should be checked.
- (b) The deflection limits given in 2.5.1 should be considered.
- (c) Unless the compression flange has full lateral restraint, as indicated in 4.2.2, the resistance of the member to lateral torsional buckling should be checked in accordance with 4.3.
- (d) Local buckling should be considered as given in 3.6.
- (e) When loads or reactions are applied through the flange to the web the conditions of 4.5 for bearing and buckling should be met.

4.2.2 Full lateral restraint

When full lateral restraint is provided no reference need be made to 4.3 which deals with the lateral torsional buckling resistance of the member.

Full restraint exists if the frictional or positive connection of a floor or other construction to the compression flange of the member is capable of resisting a lateral force of not less than 1 % of the maximum factored force in the compression flange of the member, under factored loading. This load should be considered as distributed uniformly

along the flange, provided that the dead load of the floor and the imposed load it supports together constitute the dominant loading on the beam. The floor construction should be capable of resisting this lateral force.

4.2.3 Shear

Shear force F_v not greater than shear capacity P_v , where $P_v = 0.6p_v A_v$ and A_v is the shear area taken as follows:

- (a) rolled I, H and channel sections,
load parallel to web tD
- (b) built-up sections and boxes,
load parallel to webs td
- (c) solid bars and plates $0.9A$
- (d) rectangular hollow sections,
load parallel to webs $\left(\frac{D}{D+B}\right) A$
- (e) circular hollow sections $0.6A$
- (f) any other case $0.9A_o$

where

- t is the total web thickness;
- B is the breadth;
- D is the overall depth;
- d is the depth of the web;
- A is the area of the section;
- A_o is the area of the rectilinear element of the section which has the largest dimension in the direction parallel to the load.

When the depth to thickness ratio, d/t , of a web exceeds 63 ϵ then it should be checked for shear buckling in accordance with 4.4.5.

4.2.4 Elastic shear stress

In sections where webs vary in thickness or have holes significantly larger than those required for fasteners, the shear stress should be calculated from first principles assuming elastic behaviour. In such cases the maximum shear stress, f_v , should not exceed 0.7 p_v .

4.2.5 Moment capacity with low shear load

Where $F_v \leq 0.6P_v$ the moment capacity, M_c , should be taken as follows.

For plastic or compact sections:

$$M_c = p_v S \text{ but } \leq 1.2p_v Z$$

NOTE. The elastic limitation is to prevent plasticity at working load. If $S \geq 1.2Z$ then the 1.2 constant may be replaced by the ratio of the factored load to the unfactored load.

For semi-compact sections:

$$M_c = p_v Z$$

For slender sections:

$$M_c = p_v Z$$

where

p_v is the design strength (reduced for slender sections, see 3.6);

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S is the plastic modulus of the section about the relevant axis;

Z is the elastic modulus of the section about the relevant axis.

When the depth to thickness ratio, d/t , of a web exceeds 63ϵ then it should be checked for shear buckling in accordance with 4.4.5.

4.2.6 Moment capacity with high shear load

Where $F_v > 0.6P_v$ the moment capacity, M_c , should be taken as follows.

(a) For plastic or compact sections:

$$M_c = p_y(S - S_v\rho_1) \text{ but } \leq 1.2p_yZ$$

$$\text{where } \rho_1 = \frac{2.5F_v}{P_v} - 1.5$$

and S_v is taken as follows:

For sections with equal flanges: the plastic modulus of the shear area, A_v (see figure 4(a));

For sections with unequal flanges: the plastic modulus of the gross section less the plastic modulus of that part of the section remaining after deduction of the shear area (see figure 4(b)).

(b) For semi-compact sections:

$$M_c = p_yZ$$

(c) For slender sections:

$$M_c = p_yZ$$

where p_y and Z are as defined in 4.2.5.

When the depth to thickness ratio, d/t , of a web exceeds 63ϵ then it should be checked for shear buckling in accordance with 4.4.5.

4.3 Lateral torsional buckling

4.3.1 General

When a member is not provided with full lateral restraint as defined in 4.2.2 the resistance of the member to lateral torsional buckling should be checked.

This clause covers the design of members between lateral restraints preventing sideways movement, or torsional restraints preventing twisting at points along the longitudinal axis of the beam. Beams should also satisfy the requirements of 4.2.

4.3.2 Lateral restraints

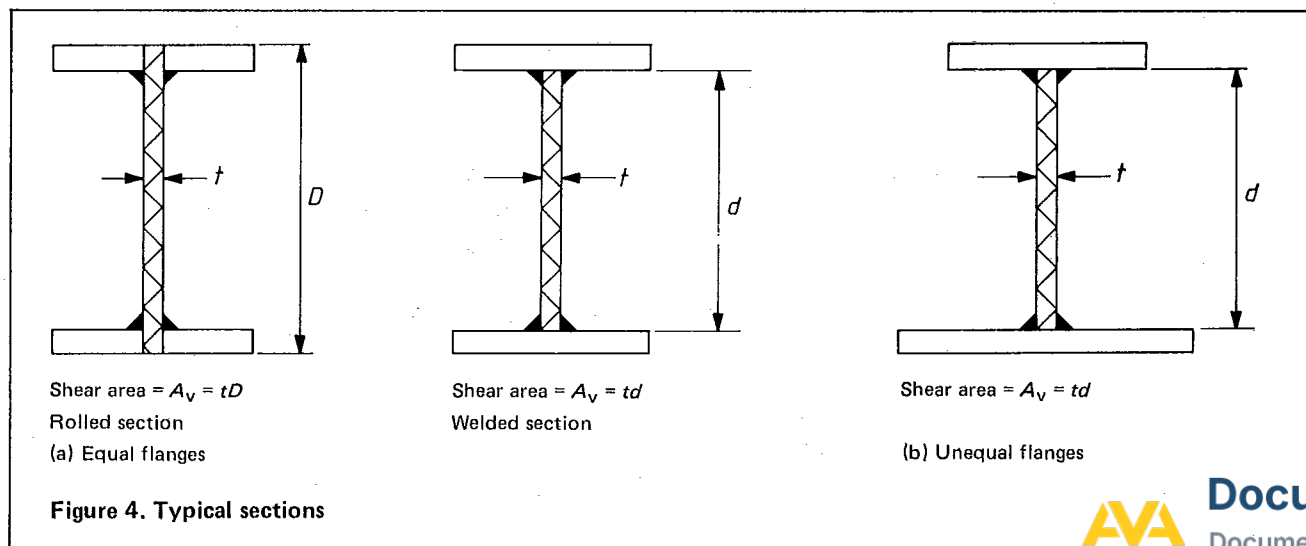
Lateral restraints have to be of adequate stiffness and strength. Restraints may be deemed to provide adequate strength if they are capable of resisting a lateral force of not less than 1 % of the maximum factored force in the compression flange or chord, divided equally between the points of restraint. Where several members share a common restraint the minimum total lateral force may be taken as the sum of those derived from the largest three members.

Purlins adequately restrained by sheeting need not normally be checked for the forces arising from the restraining of rafters of roof trusses or portal frames carrying predominantly roof loads, provided there is suitable bracing in the plane of the rafters.

Where a series of two or more parallel flexural members require lateral restraint at intervals, it is not adequate merely to tie the compression flanges such that the members become mutually dependent. Adequate restraint will be provided only by anchoring the ties at some point to an independent robust part of the structure or, alternatively, by providing a system of triangulated bracing.

4.3.3 Torsional restraints

A flexural member may be considered as torsionally restrained (about the longitudinal axis) at any point in its length where both the flanges are effectively held in position relative to each other. Such restraints should be capable of resisting a couple derived from the lever arm equal to the depth between centroids of flanges and a force equal to not less than 1 % of the maximum factored compression flange force. Alternatively, torsional restraint may be provided by bearing stiffeners acting in conjunction with the bearing of the beam (see 4.5.8).



4.3.4 Destabilizing load

Destabilizing load conditions exist when a load is applied to the compression flange of a beam and both the load and the flange are free to deflect laterally (and possibly rotationally also) relative to the centroid of the beam.

4.3.5 Effective length

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

- (a) Where a beam is restrained at the ends only, L_E should be obtained from table 9.
- (b) Where a beam is restrained at intervals by other steel members, the value of L_E should be taken as L .
- (c) For beams between supports and intermediate restraints, the value of L_E should be taken from the greater of (a) or (b).

4.3.6 Effective lengths of cantilevers

4.3.6.1 With intermediate lateral restraint. If a cantilever does have intermediate restraints to the compression flange, or when a moment is applied at the tip, the lengths between restraints should be treated as beams; the effective length, L_E , should be taken from 4.3.5.

4.3.6.2 Without intermediate lateral restraint. When a cantilever has no intermediate restraint to the compression

flange and no moment is applied to the tip, the effective length, L_E , should be taken from table 10.

4.3.7 Lateral torsional buckling resistance of members subject to bending

4.3.7.1 General. Equal flanged rolled sections may be checked using the conservative approach in 4.3.7.7. For other members, or a portion of a member, between adjacent lateral restraints, subject to bending about their major axis the following condition should be satisfied:

$$\bar{M} \leq M_b$$

where

$\bar{M}$ is the equivalent uniform moment;

M_b is the lateral torsional buckling resistance moment.

NOTE. For box sections of uniform thickness (including structural hollow sections) this check is unnecessary within the limits given in B.2.5.1.

4.3.7.2 Equivalent uniform moment, $\bar{M}$. The equivalent uniform moment, $\bar{M}$, is given by:

$$\bar{M} = m M_A$$

where

M_A is the maximum moment on the member or the portion of the member under consideration;

m is an equivalent uniform moment factor, determined from 4.3.7.6.

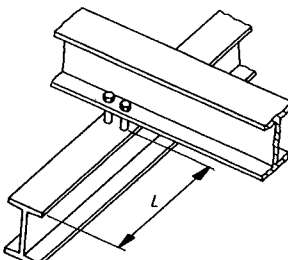
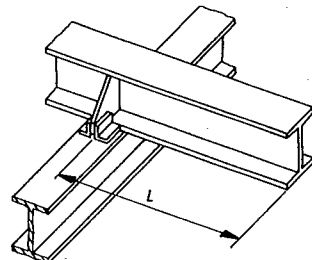
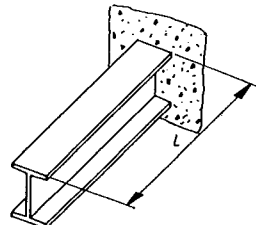
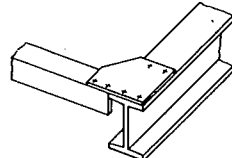
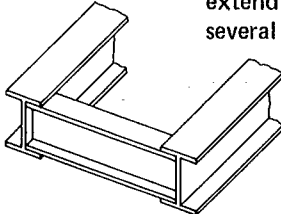
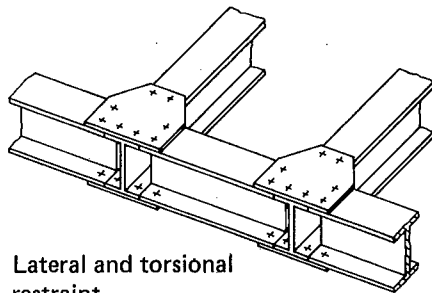
Table 9. Effective length, L_E , for beams, between supports

Conditions at supports	Loading conditions	
	Normal	Destabilizing
Beam torsionally unrestrained Compression flange laterally unrestrained Both flanges free to rotate on plan	$1.2(L+2D)$	$1.4(L+2D)$
Beam torsionally unrestrained Compression flange laterally unrestrained Compression flange only free to rotate on plan	$1.0(L+2D)$	$1.2(L+2D)$
Beam torsionally restrained Compression flange laterally restrained Compression flange only free to rotate on plan	$1.0L$	$1.2L$
Beam torsionally restrained Compression flange laterally restrained Both flanges partially free to rotate on plan (i.e. positive connections to both flanges)	$0.85L$	$1.0L$
Beam torsionally restrained Compression flange laterally restrained Both flanges NOT free to rotate on plan	$0.7L$	$0.85L$

L is the length of the beam between restraints;
 D is the depth of the beam.



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Restraint conditions		Loading conditions	
At support	At tip	Normal	Destabilizing
Continuous with lateral restraint only 	Free	$3.0L$	$7.5L$
	Laterally restrained on top flange only	$2.7L$	$7.5L$
	Torsionally restrained only	$2.4L$	$4.5L$
	Laterally and torsionally restrained	$2.1L$	$3.6L$
Continuous with lateral and torsional restraint 	Free	$1.0L$	$2.5L$
	Laterally restrained on top flange only	$0.9L$	$2.5L$
	Torsionally restrained only	$0.8L$	$1.5L$
	Laterally and torsionally restrained	$0.7L$	$1.2L$
Built-in laterally and torsionally 	Free	$0.8L$	$1.4L$
	Lateral restraint on top flange only	$0.7L$	$1.4L$
	Torsionally restrained only	$0.6L$	$0.6L$
	Laterally and torsionally restrained	$0.5L$	$0.5L$
Face beams extending over several bays  Top flange restraint  Torsional restraint  Lateral and torsional restraint			
NOTE. When values from this table are used for L_E the equivalent uniform moment factor, m , and the slenderness correction factor, n , should be taken as 1.0.			

4.3.7.3 Buckling resistance, M_b . For members with at least one axis of symmetry the buckling resistance moment, M_b , is given by:

$$M_b = S_x p_b$$

where

S_x is the plastic modulus of the section about the major axis;

p_b is the bending strength determined from 4.3.7.4.

4.3.7.4 Bending strength, p_b . The bending strength, p_b , is related to the equivalent slenderness, λ_{LT} , the design strength of the material, p_y , and member type, i.e. rolled or fabricated by welding. p_b can be obtained from tables 11, 12 or from the formulae in B.2 on which these tables are based.

Table 11. Bending strength, p_b , (in N/mm²) for rolled sections

λ_{LT} \ p_y	245	265	275	325	340	355	415	430	450
30	245	265	275	325	340	355	408	421	438
35	245	265	273	316	328	341	390	402	418
40	238	254	262	302	313	325	371	382	397
45	227	242	250	287	298	309	350	361	374
50	217	231	238	272	282	292	329	338	350
55	206	219	226	257	266	274	307	315	325
60	195	207	213	241	249	257	285	292	300
65	185	196	201	225	232	239	263	269	276
70	174	184	188	210	216	222	242	247	253
75	164	172	176	195	200	205	223	226	231
80	154	161	165	181	186	190	204	208	212
85	144	151	154	168	172	175	188	190	194
90	135	141	144	156	159	162	173	175	178
95	126	131	134	144	147	150	159	161	163
100	118	123	125	134	137	139	147	148	150
105	111	115	117	125	127	129	136	137	139
110	104	107	109	116	118	120	126	127	128
115	97	101	102	108	110	111	117	118	119
120	91	94	96	101	103	104	108	109	111
125	86	89	90	95	96	97	101	102	103
130	81	83	84	89	90	91	94	95	96
135	76	78	79	83	84	85	88	89	90
140	72	74	75	78	79	80	83	84	84
145	68	70	71	74	75	75	78	79	79
150	64	66	67	70	70	71	73	74	75
155	61	62	63	66	66	67	69	70	70
160	58	59	60	62	63	63	65	66	66
165	55	56	57	59	60	60	62	62	63
170	52	53	54	56	56	57	59	59	59
175	50	51	51	53	54	54	56	56	56
180	47	48	49	51	51	51	53	53	53
185	45	46	46	48	49	49	50	50	51
190	43	44	44	46	46	47	48	48	48
195	41	42	42	44	44	44	46	46	46
200	39	40	40	42	42	42	43	44	44
210	36	37	37	38	39	39	40	40	40
220	33	34	34	35	35	36	36	37	37
230	31	31	31	32	33	33	33	34	34
240	29	29	29	30	30	30	31	31	31
250	27	27	27	28	28	28	29	29	29

Table 12. Bending strength, p_b , (in N/mm²) for welded sections

λ_{LT} \ p_y	245	265	275	325	340	355	415	430	450
30	245	265	275	325	340	355	401	412	427
35	245	265	272	307	317	328	368	378	391
40	231	244	250	282	292	301	337	346	358
45	212	224	230	259	268	276	308	316	327
50	196	207	212	238	246	253	282	288	297
55	180	190	195	219	225	232	257	263	275
60	167	176	180	201	207	212	245	253	264
65	154	162	166	188	196	204	235	242	251
70	142	150	155	182	189	196	224	230	238
75	135	145	151	175	182	188	212	218	225
80	131	141	146	168	174	179	201	205	211
85	127	136	140	160	165	171	188	190	194
90	123	131	135	152	157	162	173	175	178
95	118	125	129	144	147	150	159	161	163
100	113	120	123	134	137	139	147	148	150
105	109	115	117	125	127	129	136	137	139
110	104	107	109	116	118	120	126	127	128
115	97	101	102	108	110	111	117	118	119
120	91	94	96	101	103	104	108	109	111
125	86	89	90	95	96	97	101	102	103
130	81	83	84	89	90	91	94	95	96
135	76	78	79	83	84	85	88	89	90
140	72	74	75	78	79	80	83	84	84
145	68	70	71	74	75	75	78	79	79
150	64	66	67	70	70	71	73	74	75
155	61	62	63	66	66	67	69	70	70
160	58	59	60	62	63	63	65	66	66
165	55	56	57	59	60	60	62	62	63
170	52	53	54	56	56	57	59	59	59
175	50	51	51	53	54	54	56	56	56
180	47	48	49	51	51	51	53	53	53
185	45	46	46	48	49	49	50	50	51
190	43	44	44	46	46	47	48	48	48
195	41	42	42	44	44	44	46	46	46
200	39	40	40	42	42	42	43	44	44
210	36	37	37	38	39	39	40	40	40
220	33	34	34	35	35	36	36	37	37
230	31	31	31	32	33	33	33	34	34
240	29	29	29	30	30	30	31	31	31
250	27	27	27	28	28	28	29	29	29

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4.3.7.5 Determination of λ_{LT} . For flanged members symmetrical about at least one axis and uniform through its length:

$$\lambda_{LT} = nuv\lambda$$

For members of other cross section, or of non-uniform section, refer to **B.2.6**, **B.2.7** and **B.3**.

In the above equation:

λ is the minor axis slenderness: $= L_E/r_y$ where

L_E is the effective length from **4.3.5** to **4.3.6**;

r_y is the radius of gyration about the minor axis of the member.

u is a buckling parameter:

(a) for a ROLLED I, H, or channel section, u may be taken from appendix B, published tables or conservatively as 0.9;

(b) for any other section, u may be taken from appendix B or conservatively as 1.0.

v is a slenderness factor:

(1) for flanged members symmetric about one axis of uniform section v may be determined from table 14 using N and λ/x in which:

$$N = \frac{I_{cf}}{I_{cf} + I_{tf}}$$

where

I_{cf} , I_{tf} are the second moments of area of the compression and tension flanges respectively about the minor axis of the section;

λ is defined above;

x is the torsional index and may be determined from **B.2.5** or published tables;

NOTE. $N = 0.5$ for members with equal flanges.

(2) For other sections v may be determined from **B.2.5** using the formulae on which table 14 is based.

n is a slenderness correction factor: determined from **4.3.7.6**.

4.3.7.6 Factors m and n . Factors m and n should be determined from the following.

(a) Members of uniform cross section, see table 13.

(b) Members of non-uniform cross section, reference should be made to **B.3**.

Table 13. Equivalent moment and slenderness correction factors

Description	Members <i>not</i> subject to de-stabilizing loads		Members subject to de-stabilizing loads (see note)	
	m	n	m	n
Members loaded in the length between adjacent lateral restraints	1.0	From table 15 or 16	1.0	1.0
Members <i>not</i> loaded in the length between adjacent lateral restraints	From table 18	1.0	1.0	1.0
Cantilevers	1.0	1.0	1.0	1.0
Alternatively (conservative)	1.0	1.0	1.0	1.0

NOTE. De-stabilizing load as defined in **4.3.4**.

Table 14. Factor ν for flanged beams of uniform section

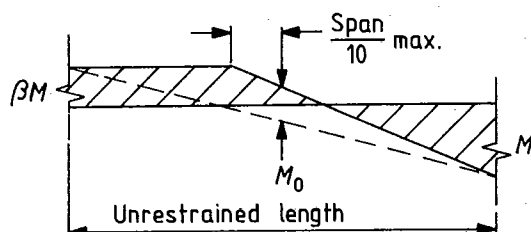
	T Compression I					I [	I Compression L				
	Tension						Tension				
N $\frac{\lambda}{x}$	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
0.5	0.79	0.81	0.84	0.88	0.93	1.00	1.11	1.28	1.57	2.20	12.67
1.0	0.78	0.80	0.83	0.87	0.92	0.99	1.10	1.27	1.53	2.11	6.36
1.5	0.77	0.80	0.82	0.86	0.91	0.97	1.08	1.24	1.48	1.98	4.27
2.0	0.76	0.78	0.81	0.85	0.89	0.96	1.06	1.20	1.42	1.84	3.24
2.5	0.75	0.77	0.80	0.83	0.88	0.93	1.03	1.16	1.35	1.70	2.62
3.0	0.74	0.76	0.78	0.82	0.86	0.91	1.00	1.12	1.29	1.57	2.21
3.5	0.72	0.74	0.77	0.80	0.84	0.89	0.97	1.07	1.22	1.46	1.93
4.0	0.71	0.73	0.75	0.78	0.82	0.86	0.94	1.03	1.16	1.36	1.71
4.5	0.69	0.71	0.73	0.76	0.80	0.84	0.91	0.99	1.11	1.27	1.55
5.0	0.68	0.70	0.72	0.75	0.78	0.82	0.88	0.95	1.05	1.20	1.41
5.5	0.66	0.68	0.70	0.73	0.76	0.79	0.85	0.92	1.01	1.13	1.31
6.0	0.65	0.67	0.69	0.71	0.74	0.77	0.82	0.89	0.97	1.07	1.22
6.5	0.64	0.65	0.67	0.70	0.72	0.75	0.80	0.86	0.93	1.02	1.14
7.0	0.63	0.64	0.66	0.68	0.70	0.73	0.78	0.83	0.89	0.97	1.08
7.5	0.61	0.63	0.65	0.67	0.69	0.72	0.76	0.80	0.86	0.93	1.02
8.0	0.60	0.62	0.63	0.65	0.67	0.70	0.74	0.78	0.83	0.89	0.98
8.5	0.59	0.60	0.62	0.64	0.66	0.68	0.72	0.76	0.80	0.86	0.93
9.0	0.58	0.59	0.61	0.63	0.64	0.67	0.70	0.74	0.78	0.83	0.90
9.5	0.57	0.58	0.60	0.61	0.63	0.65	0.68	0.72	0.76	0.80	0.86
10.0	0.56	0.57	0.59	0.60	0.62	0.64	0.67	0.70	0.74	0.78	0.83
11.0	0.54	0.55	0.57	0.58	0.60	0.61	0.64	0.67	0.70	0.73	0.78
12.0	0.53	0.54	0.55	0.56	0.58	0.59	0.61	0.64	0.66	0.70	0.73
13.0	0.51	0.52	0.53	0.54	0.56	0.57	0.59	0.61	0.64	0.66	0.69
14.0	0.50	0.51	0.52	0.53	0.54	0.55	0.57	0.59	0.61	0.63	0.66
15.0	0.49	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59	0.61	0.63
16.0	0.47	0.48	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59	0.61
17.0	0.46	0.47	0.48	0.49	0.49	0.50	0.52	0.53	0.55	0.57	0.58
18.0	0.45	0.46	0.47	0.47	0.48	0.49	0.50	0.52	0.53	0.55	0.56
19.0	0.44	0.45	0.46	0.46	0.47	0.48	0.49	0.50	0.52	0.53	0.55
20.0	0.43	0.44	0.45	0.45	0.46	0.47	0.48	0.49	0.50	0.51	0.53

NOTE 1. For beams with EQUAL flanges, $N = 0.5$.NOTE 2. ν should be determined from the general formulae given in B.2.5, on which this table is based:

- (a) for sections with LIPPED flanges (e.g. gantry girders composed of channel + universal beam); and
 (b) for intermediate values to the right of the stepped line in the table.

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Table 15. Slenderness correction factor, n , for members with applied loading substantially concentrated within the middle fifth of the unrestrained length



NOTE 1. All hogging moments are +ve.

NOTE 2. Ends chosen so that β is always less than, or equal to, 1.0.

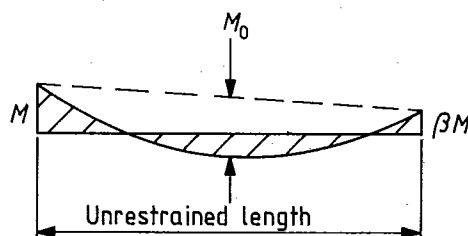
NOTE 3. M_0 is the midspan moment on a simply supported span equal to the unrestrained length (see table 17).

$\gamma = M/M_0$	β positive					0.0	β negative				
	1.0	0.8	0.6	0.4	0.2		-0.2	-0.4	-0.6	-0.8	-1.0
+50.00	1.00	0.96	0.92	0.87	0.82	0.77	0.72	0.67	0.66	0.66	0.65
+10.00	0.99	0.99	0.94	0.90	0.85	0.80	0.75	0.69	0.68	0.68	0.67
+5.00	0.98	0.98	0.97	0.93	0.89	0.84	0.79	0.73	0.71	0.70	0.70
+2.00	0.96	0.95	0.95	0.95	0.94	0.94	0.89	0.84	0.79	0.77	0.76
+1.50	0.95	0.95	0.94	0.94	0.93	0.93	0.92	0.90	0.85	0.80	0.80
+1.00	0.93	0.92	0.92	0.92	0.92	0.91	0.91	0.91	0.91	0.92	0.92
+0.50	0.90	0.90	0.90	0.89	0.89	0.89	0.89	0.89	0.88	0.88	0.88
0.00	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86	0.86
-0.10	0.85	0.85	0.85	0.85	0.85	0.86	0.86	0.86	0.86	0.86	0.86
-0.20	0.83	0.83	0.83	0.84	0.84	0.85	0.85	0.85	0.86	0.86	0.86
-0.30	0.81	0.82	0.82	0.83	0.83	0.84	0.85	0.85	0.86	0.86	0.87
-0.40	0.79	0.80	0.81	0.81	0.82	0.83	0.84	0.85	0.85	0.86	0.87
-0.50	0.77	0.78	0.79	0.80	0.82	0.83	0.85	0.86	0.86	0.87	0.88
-0.60	0.62	0.66	0.72	0.77	0.80	0.82	0.84	0.85	0.86	0.87	0.88
-0.70	0.56	0.56	0.61	0.67	0.73	0.79	0.83	0.85	0.87	0.88	0.89
-0.80	0.56	0.53	0.54	0.59	0.65	0.71	0.77	0.83	0.89	0.90	0.90
-0.90	0.59	0.57	0.54	0.53	0.57	0.64	0.71	0.77	0.84	0.88	0.91
-1.00	0.62	0.58	0.54	0.52	0.54	0.59	0.66	0.72	0.80	0.85	0.92
-1.10	0.66	0.62	0.57	0.54	0.54	0.57	0.63	0.68	0.76	0.83	0.89
-1.20	0.70	0.66	0.60	0.55	0.54	0.55	0.60	0.65	0.73	0.80	0.87
-1.30	0.73	0.69	0.63	0.57	0.55	0.54	0.57	0.61	0.69	0.77	0.83
-1.40	0.74	0.70	0.64	0.58	0.56	0.54	0.55	0.60	0.66	0.74	0.81
-1.50	0.75	0.70	0.64	0.59	0.56	0.54	0.55	0.59	0.65	0.73	0.80
-1.60	0.76	0.72	0.65	0.60	0.57	0.55	0.55	0.58	0.64	0.72	0.80
-1.70	0.77	0.74	0.66	0.61	0.58	0.56	0.55	0.58	0.63	0.70	0.78
-1.80	0.79	0.77	0.68	0.63	0.59	0.56	0.56	0.57	0.62	0.69	0.76
-1.90	0.80	0.79	0.69	0.64	0.60	0.57	0.56	0.57	0.61	0.67	0.75
-2.00	0.81	0.81	0.70	0.65	0.61	0.58	0.56	0.56	0.60	0.66	0.74
-5.00	0.93	0.89	0.83	0.77	0.72	0.67	0.64	0.61	0.60	0.62	0.65
-50.00	0.99	0.95	0.90	0.86	0.79	0.74	0.70	0.67	0.64	0.63	0.65
Infinity	1.00	0.96	0.91	0.86	0.82	0.77	0.72	0.68	0.65	0.65	0.65

NOTE 4. The values of n in this table apply only to members of UNIFORM section.

NOTE 5. Values for intermediate values of β and γ may be interpolated.

NOTE 6. When n from this table is used, $m=1.00$.

Table 16. Slenderness correction factor, n , for members with applied loading other than as for table 15

NOTE 1. All hogging moments are +ve.

NOTE 2. Ends chosen so that β is always less than, or equal to, 1.0.NOTE 3. M_0 is the midspan moment on a simply supported span equal to the unrestrained length (see table 17).

$\gamma = M/M_0$	β positive					0.0	β negative				
	1.0	0.8	0.6	0.4	0.2		-0.2	-0.4	-0.6	-0.8	-1.0
+50.00	1.00	0.96	0.92	0.87	0.83	0.77	0.72	0.67	0.66	0.66	0.65
+10.00	0.99	0.98	0.95	0.91	0.86	0.81	0.76	0.70	0.68	0.68	0.67
+5.00	0.99	0.98	0.97	0.94	0.90	0.85	0.80	0.75	0.71	0.70	0.70
+2.00	0.98	0.98	0.97	0.96	0.94	0.92	0.90	0.86	0.82	0.78	0.76
+1.50	0.97	0.97	0.97	0.96	0.95	0.93	0.92	0.89	0.86	0.83	0.79
+1.00	0.97	0.97	0.97	0.96	0.96	0.95	0.94	0.93	0.93	0.91	0.89
+0.50	0.96	0.96	0.96	0.96	0.96	0.95	0.94	0.94	0.94	0.93	0.92
0.00	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
-0.10	0.93	0.93	0.93	0.93	0.94	0.94	0.94	0.94	0.94	0.94	0.94
-0.20	0.92	0.92	0.92	0.92	0.93	0.93	0.93	0.93	0.94	0.94	0.93
-0.30	0.91	0.91	0.92	0.92	0.93	0.93	0.93	0.93	0.94	0.94	0.94
-0.40	0.90	0.90	0.91	0.91	0.92	0.92	0.92	0.92	0.93	0.93	0.93
-0.50	0.89	0.90	0.91	0.91	0.92	0.92	0.92	0.92	0.92	0.92	0.92
-0.60	0.71	0.77	0.84	0.87	0.89	0.91	0.92	0.92	0.92	0.92	0.92
-0.70	0.57	0.64	0.70	0.77	0.82	0.87	0.89	0.91	0.92	0.92	0.91
-0.80	0.47	0.52	0.59	0.67	0.73	0.80	0.86	0.90	0.92	0.92	0.92
-0.90	0.47	0.46	0.50	0.58	0.65	0.73	0.80	0.87	0.90	0.90	0.90
-1.00	0.50	0.48	0.46	0.51	0.58	0.66	0.73	0.81	0.87	0.89	0.89
-1.10	0.54	0.51	0.48	0.49	0.54	0.61	0.69	0.77	0.83	0.87	0.88
-1.20	0.57	0.54	0.50	0.47	0.51	0.56	0.64	0.73	0.80	0.84	0.87
-1.30	0.61	0.56	0.52	0.47	0.49	0.53	0.61	0.70	0.77	0.82	0.86
-1.40	0.64	0.59	0.55	0.49	0.48	0.51	0.58	0.67	0.74	0.79	0.85
-1.50	0.67	0.62	0.57	0.51	0.47	0.49	0.56	0.64	0.71	0.77	0.84
-1.60	0.69	0.64	0.59	0.52	0.48	0.50	0.55	0.63	0.69	0.76	0.83
-1.70	0.71	0.66	0.60	0.54	0.50	0.51	0.55	0.61	0.68	0.74	0.82
-1.80	0.74	0.69	0.62	0.55	0.51	0.51	0.54	0.60	0.66	0.73	0.81
-1.90	0.76	0.71	0.63	0.57	0.53	0.52	0.54	0.58	0.65	0.71	0.80
-2.00	0.78	0.73	0.65	0.58	0.54	0.53	0.53	0.57	0.63	0.70	0.79
-5.00	0.91	0.86	0.80	0.74	0.70	0.65	0.62	0.59	0.58	0.61	0.67
-50.00	0.99	0.95	0.89	0.84	0.79	0.74	0.70	0.66	0.63	0.62	0.65
Infinity	1.00	0.96	0.91	0.86	0.82	0.77	0.72	0.68	0.65	0.65	0.65

NOTE 4. The values of n in this table apply only to members of UNIFORM section.NOTE 5. Values for intermediate values of β and γ may be interpolated.NOTE 6. When n from this table is used, $m = 1.00$.

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Table 17. Moment diagram between adjacent points of lateral restraint

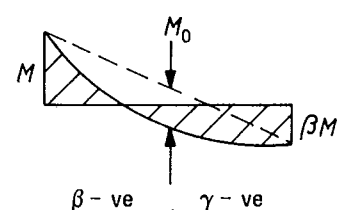
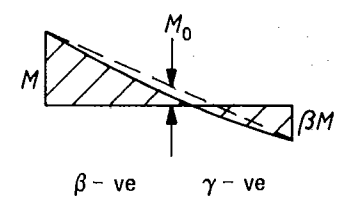
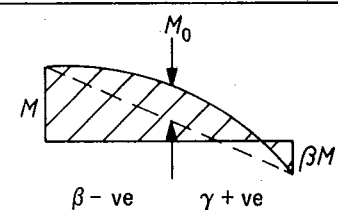
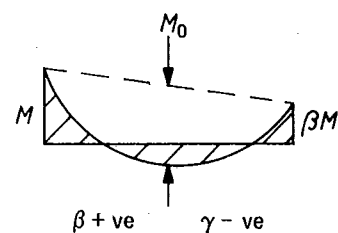
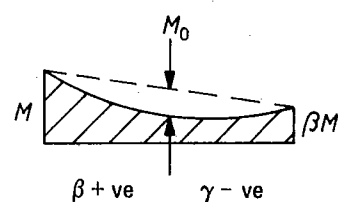
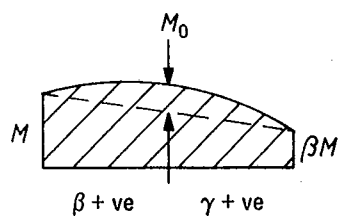
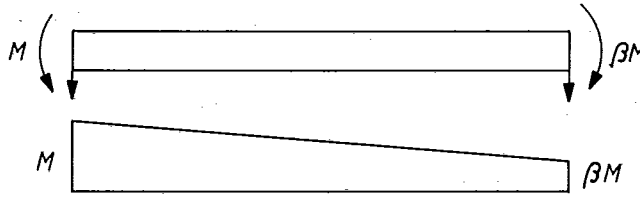
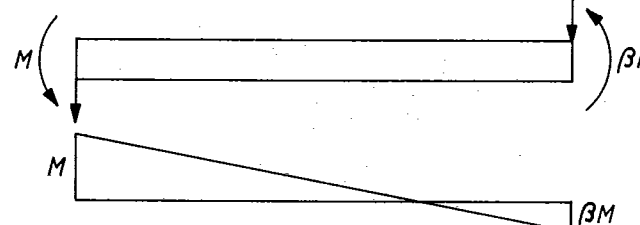


Table 18. Equivalent uniform moment factor, m

	β	m
Beta positive 	1.0	1.00
	0.9	0.95
	0.8	0.90
	0.7	0.85
	0.6	0.80
	0.5	0.76
	0.4	0.72
	0.3	0.68
	0.2	0.64
	0.1	0.60
Beta negative 	0.0	0.57
	-0.1	0.54
	-0.2	0.51
	-0.3	0.48
	-0.4	0.45
	-0.5 to -1.0	0.43

NOTE 1. The values of m in this table apply only to end moments applied to beams of UNIFORM section with EQUAL flanges. In all other cases, $m = 1.0$.

NOTE 2. Values of m for intermediate values of β may be interpolated, or determined from the equation:
 $m = 0.57 + 0.33\beta + 0.10\beta^2$; but not less than 0.43.

4.3.7.7 Equal flanged rolled sections. This clause gives a conservative approach. If a full analysis is required or the sections are not rolled universal sections or joists, then the provisions of 4.3.7.1 to 4.3.7.6 should be used.

The maximum moment between lateral restraints should not exceed the buckling resistance moment, M_b , of the section taken as $P_b S_x$.

Where

- P_b is determined from table 19(a), (b), (c) or (d), for λ and x ;
- S_x is the plastic modulus of the section about the x-x axis;
- λ is the slenderness of section = L_E/r ;
- L_E is the effective length from 4.3.5 or 4.3.6;
- r is the radius of gyration about the member minor axis;
- x is the torsional index (see 4.3.7.5) which may be taken as D/T .

When using table 19, the equivalent uniform moment factor should be taken as 1. The effect of the loads and moments between restraints may be considered by multiplying the effective length by a slenderness correction factor, n , taken from table 20.

4.3.8 Buckling resistance moment for single angles

The buckling resistance moment for a single angle should be taken as:

$$M_b = 0.8p_y Z \text{ for } L/r_{vv} \leq 100$$

$$M_b = 0.7p_y Z \text{ for } L/r_{vv} \leq 180$$

$$M_b = 0.6p_y Z \text{ for } L/r_{vv} \leq 300$$

where

- Z is the elastic modulus about the appropriate axis;
- r_{vv} is the radius of gyration about the weakest axis;
- L is the unrestrained length.

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Table 19. Bending strength, p_b , (in N/mm^2) for rolled sections with equal flanges

(a) $p_y = 265 \text{ N/mm}^2$

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

Table 19 (continued)											
(b) $p_y = 275 \text{ N/mm}^2$											
λ	x	5	10	15	20	25	30	35	40	45	50
30		275	275	275	275	275	275	275	275	275	275
35		275	275	275	275	275	275	275	275	275	275
40		275	275	275	275	274	273	272	272	272	272
45		275	275	269	266	264	263	263	262	262	262
50		275	269	261	257	255	253	253	252	252	251
55		275	263	254	248	246	244	243	242	241	241
60		275	258	246	240	236	234	233	232	231	230
65		275	252	239	232	227	224	223	221	221	220
70		274	247	232	223	218	215	213	211	210	209
75		271	242	225	215	209	206	203	201	200	199
80		268	237	219	208	201	196	193	191	190	189
85		265	233	213	200	193	188	184	182	180	179
90		262	228	207	193	185	179	175	173	171	169
95		260	224	201	186	177	171	167	164	162	160
100		257	219	195	180	170	164	159	156	153	152
105		254	215	190	174	163	156	151	148	146	144
110		252	211	185	168	157	150	144	141	138	136
115		250	207	180	162	151	143	138	134	131	129
120		247	204	175	157	145	137	132	128	125	123
125		245	200	171	152	140	132	126	122	119	116
130		242	196	167	147	135	126	120	116	113	111
135		240	193	162	143	130	121	115	111	108	106
140		238	190	159	139	126	117	111	106	103	101
145		236	186	155	135	122	113	106	102	99	96
150		233	183	151	131	118	109	102	98	95	92
155		231	180	148	127	114	105	99	94	91	88
160		229	177	144	124	111	101	95	90	87	84
165		227	174	141	121	107	98	92	87	84	81
170		225	171	138	118	104	95	89	84	81	78
175		223	169	135	115	101	92	86	81	78	75
180		221	166	133	112	99	89	83	78	75	72
185		219	163	130	109	96	87	80	76	72	70
190		217	161	127	107	93	84	78	73	70	67
195		215	158	125	104	91	82	76	71	68	65
200		213	156	122	102	89	80	74	69	65	63
210		209	151	118	98	85	76	70	65	62	59
220		206	147	114	94	81	72	66	62	58	55
230		202	143	110	90	78	69	63	58	55	52
240		199	139	106	87	74	66	60	56	52	50
250		195	135	103	84	72	63	57	53	50	47

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Table 19 (continued)

(c) $p_v = 340 \text{ N/mm}^2$

λ	$\times$	5	10	15	20	25	30	35	40	45	50
30		340	340	340	340	340	340	340	340	340	340
35		340	340	340	340	340	340	339	339	339	339
40		340	333	333	320	328	327	327	326	326	326
45		340	333	323	318	316	315	314	314	313	313
50		340	322	312	307	304	302	301	301	300	300
55		340	315	303	296	292	290	288	287	286	286
60		337	308	293	285	280	277	275	274	273	272
65		333	301	283	273	268	264	262	260	259	258
70		329	294	274	263	256	251	248	246	245	244
75		325	287	265	252	244	239	235	233	231	230
80		321	281	257	242	232	227	223	220	218	216
85		318	275	248	232	222	215	211	207	205	203
90		314	269	240	223	211	204	199	196	193	191
95		311	263	232	213	201	194	188	185	182	180
100		307	257	225	205	192	184	178	174	171	169
105		304	252	218	197	184	175	169	165	161	159
110		301	246	211	189	176	166	160	156	152	150
115		297	241	205	182	168	159	152	147	144	142
120		294	236	199	176	161	151	145	140	136	134
125		291	231	193	170	155	145	138	133	129	127
130		288	227	188	164	148	138	131	126	123	120
135		285	222	183	158	143	133	125	120	117	114
140		282	218	178	153	138	127	120	115	111	108
145		279	213	173	148	133	122	115	110	106	103
150		276	209	168	144	128	118	110	105	101	99
155		273	205	164	139	124	113	106	101	97	94
160		270	207	160	135	120	109	102	97	93	90
165		267	197	156	132	116	106	98	93	89	86
170		265	194	153	128	112	102	95	90	86	83
175		262	190	149	125	109	99	92	86	82	79
180		259	187	146	121	106	96	88	83	79	76
185		257	184	142	118	103	93	86	80	77	74
190		254	180	139	115	100	90	83	78	74	71
195		251	177	136	113	98	87	80	75	71	68
200		249	174	134	110	95	85	78	73	69	66
210		244	168	128	105	90	81	74	69	65	62
220		239	163	123	101	86	77	70	65	61	58
230		234	158	119	96	82	73	66	61	58	55
240		230	153	115	93	79	70	63	58	55	52
250		225	149	111	89	76	67	60	56	52	49