

Steel, concrete and composite bridges —

Part 3: Code of practice for design of steel bridges

ICS 93.040

Committees responsible for this British Standard

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- British Cement Association
- British Constructional Steelwork Association
- British Precast Concrete Federation Ltd.
- British Railways Board
- Concrete Society
- County Surveyors' Society
- Highways Agency
- Institution of Civil Engineers
- Institution of Structural Engineers
- Steel Construction Institute
- UK Steel Association
- Welding Institute

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Foreword

This part of BS 5400 has been prepared by Subcommittee B/525/10. It supersedes BS 5400-3:1982, which is withdrawn.

This new edition of BS 5400-3 includes technical changes only. It does not represent a full review or revision of the standard, which will be undertaken in due course.

BS 5400 is a document combining codes of practice covering the design and construction of steel, concrete and composite bridges and specifications for loads, materials and workmanship. It comprises the following parts:

- *Part 1: General statement;*
- *Part 2: Specification for loads;*
- *Part 3: Code of practice for design of steel bridges;*
- *Part 4: Code of practice for design of concrete bridges;*
- *Part 5: Code of practice for design of composite bridges;*
- *Part 6: Specification for materials and workmanship, steel;*
- *Part 7: Specification for materials and workmanship, concrete, reinforcement and prestressing tendons;*
- *Part 8: Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons;*
- *Part 9: Bridge bearings*
 - *Section 9.1: Code of practice for design of bridge bearings;*
 - *Section 9.2: Specification for materials, manufacture and installation of bridge bearings;*
- *Part 10: Code of practice for fatigue.*

In the drafting of BS 5400 important changes have been made in respect of loading and environmental assumptions, design philosophy, load factors, service stresses and structural analysis. Furthermore, recourse has been made to recent theoretical and experimental research and several design studies have been made on components and on complete bridges.

The relationship between Part 3 and Part 5

The design of composite bridges requires the combined use of this part of BS 5400 and BS 5400-5.

This part of BS 5400 has been drafted on the assumption that for the design of steelwork in bridges with either steel or concrete decks, the methods of global analysis and all the procedures for satisfying the limit state criteria will be in accordance with this part of BS 5400. For beams, this part of BS 5400 may be used without any modification in conjunction with those provisions of BS 5400-5 that are applicable to the properties of the composite slab and its connection to the steel.

It should be noted that more serviceability checks are recommended for composite than for steel bridges. This difference is due to the special characteristics of composite construction, such as the large shape factor of certain composite sections, the addition of stresses in a two-phase structure (bare steel/wet concrete, and composite), and the effects of shrinkage and temperature on the girders and shear connectors.

Part 2 and Part 5

Attention is drawn to new editions of Part 2 and Part 5, which were issued in 2006 and 2005 respectively. The revisions of those parts took account of guidelines formerly given in documents produced by the central government and published as Departmental Standards BD 37/01 (DM RB 1.3) and BD 16/82 (DM RB 1.3) respectively.

Amendment No. 1 to Part 3

Amendment No. 1 was published in 2006 and introduced changes to Part 3 that were prepared by Subcommittee B/525/10. The majority of the changes concern the determination of the limiting moment of resistance of beams and in particular the determination of the effective length for lateral torsional buckling and the slenderness of beams with discrete restraints that are not fully effective. A small number of minor errors have also been corrected.

The changes are indicated by the symbols $\boxed{A_1}$ and $\langle A_1 \rangle$ enclosing the text affected.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its 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 190, an inside back cover and a back cover.

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1 Scope

This part of BS 5400 gives recommendations for the design of structural steelwork in bridges.

After stating general recommendations, procedures are given for the design of steelwork components, assemblies and connections. Such procedures are applicable to steelwork which is to be fabricated and erected in accordance with BS 5400-6. Recommendations for design against fatigue are contained in BS 5400-10. Recommendations for design of concrete components and shear connectors for their interaction with steelwork are contained in BS 5400-5. The partial factors of safety given are appropriate only for bridges designed to this standard.

Hybrid construction, using materials of different yield stress, is not generally within the scope of this part of BS 5400.

Parapets, safety fences and other ancillary items are not within the scope of this part of BS 5400.

2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this part of this British Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 4-1, *Structural steel sections — Part 1: Specification for hot-rolled sections.*

BS 2846-3:1975, *Guide to statistical interpretation of data — Part 3: Determination of a statistical tolerance interval.*

BS 3692, *Specification for ISO metric precision hexagon bolts, screws and nuts — Metric units.*

BS 4190, *Specification for ISO metric black hexagon bolts, screws and nuts.*

BS 4395 (all parts), *Specification for high strength friction grip bolts and associated nuts and washers for structural engineering.*

BS 4604 (all parts), *Specification for the use of high strength friction grip bolts in structural steelwork — Metric series.*

 Text deleted 

BS 5400-1, *Steel, concrete and composite bridges — Part 1: General statement.*

BS 5400-2, *Steel, concrete and composite bridges — Part 2: Specification for loads.*

BS 5400-5:1979, *Steel, concrete and composite bridges — Part 5: Code of practice for design of composite bridges.*

BS 5400-6:1999, *Steel, concrete and composite bridges — Part 6: Specification for materials and workmanship, steel.*

BS 5400-10:1980, *Steel, concrete and composite bridges — Part 10: Code of practice for fatigue.*

BS 7668, *Specification for weldable structural steels — Hot finished structural hollow sections in weather resistant steels.*

 BS EN 1011-1, *Welding — Recommendations for welding of metallic materials — Part 1: General guidance for arc welding.*

BS EN 1011-2, *Welding — Recommendations for welding of metallic materials — Part 2: Arc welding of ferritic steel.* 

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

BS EN 10029, *Specification for tolerances on dimensions, shape and mass for hot rolled steel plates 3 mm thick or above.*

BS EN 10034, *Structural steel I and H sections — Tolerances on shape and dimensions.*

BS EN 10056-1, *Specification for structural steel equal and unequal angles — Part 1: Dimensions.*

BS EN 10056-2, *Specification for structural steel equal and unequal angles — Part 2: Tolerances on shape and dimensions.*

BS EN 10067, *Hot rolled bulb flats — Dimensions and tolerances on shape, dimensions and mass.*

BS EN 10113 (all parts), *Hot-rolled products in weldable fine grain structural steels.*

BS EN 10137 (all parts), *Plates and wide flats made of high yield strength structural steels in the quenched and tempered or precipitation hardened conditions.*

BS EN 10155, *Structural steels with improved atmospheric corrosion resistance — Technical delivery conditions.*

BS EN 10210 (all parts), *Hot finished structural hollow sections of non-alloy and fine grain structural steels.*

BS EN ISO 2566-1, *Steel — Conversion of elongation values — Part 1: Carbon and low alloy steels.*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this part of BS 5400, the terms and definitions given in BS 5400-1 apply, together with the following. For the sake of clarity the factors which together comprise the partial safety factor for loads are restated as follows.

3.1.1

design load

load obtained by multiplying the nominal load by γ_{FL} , the partial safety factor for loads

NOTE γ_{FL} is the product of two individual factors, γ_{F1} and γ_{F2} , which take account of the following:

- for γ_{F1} , the possible unfavourable deviations in load from those considered in deriving their nominal values;
- for γ_{F2} , the reduced probability that, with combinations of loads, the individual loads would all be at their nominal values.

The relevant values of the function γ_{FL} ($= \gamma_{F1}\gamma_{F2}$) are given in BS 5400-2. The factor γ_{FB} takes account of inaccurate assessment of the effects of loading, unforeseen stress distribution in the structure, variation in dimensional accuracy achieved in construction and the importance of the limit state being considered.

The values of γ_{FB} are given in 4.3.3.

3.2 Symbols

NOTE Symbols are further clarified, as appropriate, in the text. Some additional symbols are used in the annexes.

3.2.1 Main symbols

A	cross-sectional area
A_o	area of box enclosed by perimeter
a	length of panel; longitudinal spacing of transverse stiffeners
B	overall width; spacing of beams
b	width of panel; width of element; transverse spacing of longitudinal stiffeners
C	weld shrinkage coefficient
C_v	Charpy energy absorption
c	centre of bearing
D	overall depth; overall diameter
d	depth of element; diameter of element
E	modulus of elasticity
e	the exponential function, to be taken as 2.718 3
e	eccentricity; offset
F	internal force
F_o	prestress force
F_R	lateral restraint force on compression flange
F_S	lateral force at support
f	rotational flexibility; force per unit length or width
G	shear modulus of elasticity
g	throat thickness of weld; gauge of holes
H	prying force
h	height of element
I	second moment of area
i	factor, as defined in text
J	torsional constant
j	width of bearing pad contact
K	buckling coefficient; elastic stress concentration factor
k	buckling coefficient; ratio of principal stresses
L	span; overall length
ℓ	length of element

ℓ_R	U-frame spacing
ℓ_w	half wavelength of buckling
M	moment
m	ratio, as defined in text
N	number of friction interfaces
n	number, as defined in text
P	applied force
p	penetration of weld
Q	shear force in diaphragms and battens
q	shear flow
R	reaction; radius of curvature
r	radius of gyration
S	shape factor
S_0	cross-sectional area of test piece
s	spacing
T	torque or torsional moment; Charpy test temperature
t	thickness of plate or section
U	temperature
V	shear force in webs
u, v	factors, as defined in text
W	total uniform load
w	width of element
x	distance longitudinally along member
y	distance from neutral axis or centroid
Z	section modulus
α	ratio of stiffener to flange area
α_{LT}	coefficient for torsional restraint
β	slope of web to vertical; factor
γ_{FL}, γ_{F3}	partial load factors
$\gamma_m, \gamma_{m1}, \gamma_{m2}$	partial material factors
Δ	imperfection to be assumed
δ	flexibility (deflection per unit force)
δ_e	unit force related displacement at end support
δ_i	unit force related displacement at internal support
δ_t	lateral displacement at end torsional restraint for unit force to each flange
δ_r	lateral displacement at intermediate restraint (other than internal support)
θ	rotation (rad)
θ_d	slope (degrees)
λ	slenderness parameter
λ_{LT}	slenderness parameter for lateral torsional buckling
μ	slip factor
ν	Poisson's ratio
π	to be taken as 3.141 6
ρ	proportion, as defined in text
σ	direct stress
σ_y	nominal yield stress (N/mm ²)
τ	shear stress

τ_o	shear stress at onset of tension field action
τ_y	shear yield stress
φ	panel aspect ratio
ψ	effective breadth ratio
η, ξ	factors, as defined in text

3.2.2 Subscripts

a	axial
b	cross beams; bending; bearing; battens
B	box; beam
c	cantilever; compressive
d	diaphragm
D	design resistance
e	effective; equivalent
E	Euler buckling
f	flange
g	gusset
h	horizontal; hole
i	buckling; instability
j	compact member
k	stocky member
ℓ	limiting
n	integer value to be taken, 1 to n
o	stiffener; outstand
p	plastic; lacing
q	shear
R	reference value
s	stiffener
t	tensile
T	torsion
u	U-frame
v	vertical
w	web; weld
x	about X-X axis
y	about Y-Y axis
z	centroid of plate
1	normal; longitudinal; primary
2	transverse; horizontal; secondary
	or 1 to 5 distinguishing subscript, as defined in text
x-x	summation about X-X axis
y-y	summation about Y-Y axis

4 Design objectives

4.1 General

4.1.1 Basis

The objectives of design should be those stated in BS 5400-1.

4.1.2 Design loads and combinations

The design loads and combinations should be those given in BS 5400-2. Design loads and combinations should be selected and applied in such a way that the most adverse total effect is caused in the element or structure under consideration.

NOTE Care is needed when some stress resultants are adverse and others are beneficial.

4.2 Limit states

4.2.1 Ultimate limit state

All structural steelwork should be checked to determine whether it is in accordance with the recommendations of this part of BS 5400 in relation to the ultimate limit state.

4.2.2 Serviceability limit state

Structural steelwork should be considered to have reached the serviceability limit state if either:

- a) deformation has occurred in one or more components or connections such as to cause either excessive permanent deflection, or damage to finishes or protective coatings; or
- b) buckling of one or more elements has occurred to such an extent that the maximum stress exceeds the yield stress, and excessive deformation occurs due to the spread of plasticity.

Except where otherwise stated in this part of BS 5400, structural steelwork may be deemed to satisfy the serviceability limit state if it has been designed in accordance with the provisions of this part of BS 5400 for the ultimate limit state.

A list of subclauses in which a separate check at the serviceability limit state is recommended is given in Table 1.

Table 1 — Clauses recommending serviceability check

Subclause	Items and consideration for which a serviceability check is recommended
9.2.3.1	Flange panel when maximum longitudinal stress is more than $1/\psi_R$ times the mean longitudinal stress
9.5.5	Whole beam cross-section, without redistribution, when yielding of tension flange occurs at a lower loading than the buckling or yielding of the compression flange and redistribution of tension flange stresses is assumed for the ultimate limit state
9.9.8	Unsymmetric sections
9.10.3.3	Stiffened flanges subjected to local bending when local bending stresses are neglected for the ultimate limit state
12.2.3	Compression members in trusses that are not compact, or certain compression members having length to width ratios of less than 12 for chord members and less than 24 for web members
14.5.4.1.2	Connections made with HSFG bolts when the capacity used for the ultimate limit state is based on the shear or bearing capacities of the HSFG bolts and the lower of these is in excess of the friction capacity

4.2.3 Fatigue

The fatigue endurance should be in accordance with the recommendations of BS 5400-10.

4.3 Partial safety factors to be used

4.3.1 General

For a satisfactory design of the structure, the provisions given in BS 5400-1 should be met using the format set out in 4.3.2.

4.3.2 Safety factor format

Stresses should be calculated from the effects of $\gamma_{fL}Q_k$.

The safety factor format to be used in applying this part of BS 5400 is:

$$(\text{effects of } \gamma_{fL}Q_k) \leq \frac{1}{\gamma_{fB}\gamma_{m1}\gamma_{m2}} \text{ (function } \sigma_y, \text{ and other geometric variables)}$$

where

$\gamma_{fL}\gamma_{fB}$, Q_k are as described in BS 5400-1;

γ_{m1} is the partial factor on the characteristic yield stress σ_y ;

γ_{m2} is the partial factor for modelling uncertainties and other variables in the formulae for design resistance.

4.3.3 Values of partial safety factors

The values of partial safety factors are as follows.

- The values of γ_{fL} are given in BS 5400-2 for each type and combination of loading.
- The factor γ_{fB} in this part of BS 5400 should be taken as 1.1 for the ultimate limit state and 1.0 for the serviceability limit state.
- For the sake of simplicity the expressions for strength in this part of BS 5400 contain a single factor $\gamma_m (= \gamma_{m1}\gamma_{m2})$. Values of the factor to be used where γ_m is explicitly shown in the design strength equations in this part of BS 5400 are given in Table 2.

Table 2 — Partial safety factors, $\gamma_m = \gamma_{m1}\gamma_{m2}$

a) Ultimate limit state		
The value of γ_m for the ultimate limit state should be taken as 1.05, except in the following clauses for which the appropriate value of γ_m is given.		
Structural component and behaviour	Clauses	γ_m
Strength of longitudinal stiffeners	9.10.2.3a) and b) , 9.11.5.2	1.20 (fibre in compression) 1.05 (fibre in tension)
Buckling resistance of stiffeners	9.13.5.3 , 9.13.6 , 9.14.4.3 , 9.17.6.7 , 9.17.7.3.2 , 9.17.8	1.20
Fasteners in tension	14.5.3.2 , 14.5.3.3 , 14.5.3.5	1.20
Fasteners in shear	14.5.3.4	1.10
Friction capacity of HSFG bolts	14.5.4.2	1.30
Welds	14.6.3.11.1 , 14.6.3.11.2 , 14.6.3.11.3	1.20
b) Serviceability limit state		
The value of γ_m for the serviceability limit state should be taken as 1.00, except in the following clause for which the appropriate value of γ_m is given.		
Structural component and behaviour	Clause	γ_m
Friction capacity of HSFG bolts	14.5.4.2	1.20

NOTE Any other clause making cross-reference to any of the clauses listed in Table 2 should incorporate the appropriate γ_m factor as given in Table 2.

4.4 Structural support

Provision should be made in the design for the transmission of vertical, longitudinal and lateral forces to the bearings and supporting structures.

4.5 Corrosion resistance and protection

4.5.1 General

The basis for the design of components contained in this part of BS 5400 makes no allowance for any loss of material due to corrosion. All steelwork should be designed and detailed to minimize the risk of corrosion.

All parts should be accessible for inspection, cleaning and painting, or should be effectively sealed against corrosion. Where these methods are not possible, either the surface of the steel should be given a system of protective coating selected with due regard to the design life of the part, together with additional thickness of steel in accordance with 4.5.5.1, or the steel used should have corrosion resistant properties suitable for the design environment. Road decks, whether steel or concrete, should be waterproofed and so designed as to protect supporting steelwork from corrosive attack from salt from the road surface.

4.5.2 Provision of drainage

Drainage should be provided wherever water may collect and should be designed to carry the water to a point clear of the underside of adjacent parts of the structure.

4.5.3 Sealing

Box members and other hollow sections without access for internal inspection and maintenance should be effectively sealed against corrosion (e.g. by continuous welding).

Box members and other hollow sections accessible for maintenance should be provided with internal protection against corrosion unless measures are taken to ensure that they are airtight.

Box members designed to be completely airtight should be checked for structural adequacy under internal and external pressure due to changes in temperature of the air inside the enclosed space and due to changes in the pressure of the external atmosphere.

4.5.4 Narrow gaps and spaces

To permit inspection and maintenance, the clear space between parts not in contact should be not less than one-sixth of the width of the face of the smaller part, or 10 mm, whichever is the greater. Alternatively, steel packing should be inserted to fill the space. Where this is impracticable then a sealant should be used.

4.5.5 Thickness of steel with inaccessible surfaces

4.5.5.1 Where recommended in 4.5.1, inaccessible surfaces of steel should be provided with an extra thickness based on an estimate of probable corrosion. For a design life of 120 years, this provision may be considered to be met if the following extra thickness is provided at each inaccessible surface:

- a) 6 mm at industrial or marine sites;
- b) 4 mm at other inland sites;
- c) 1 mm in addition to the excess under a) and b), where free drainage cannot be ensured.

NOTE This provision need not apply to surfaces in contact, the edges of which are effectively sealed against corrosion.

4.5.5.2 Sealed hollow sections should be not less than 5 mm thick. For such components with a design life not greater than 50 years this value may be reduced to 4 mm.

4.5.6 Thickness of weathering steel

To cater for the loss of thickness of weathering steel due to slow rusting, a thickness additional to the minimum design requirement should be provided on each exposed surface equal to 2 mm if the atmospheric environment of the bridge contains an average sulfur content greater than 1 mg SO₃ per 100 cm² per day, as measured by the lead candle method, or 1 mm if the environment is less severe.

NOTE The lead candle method was described in BS 1747-4, which was withdrawn in 1988. However, the method remains in use.

4.6 Clearance gauges

Specified clearance gauges should be maintained without encroachment by any part of the structure under the action of load combination 1, specified in BS 5400-2:1978, 4.4.1, for the serviceability limit state.

5 Limitations on construction and workmanship

5.1 Workmanship

The design rules given in this part of BS 5400 are appropriate only to bridges fabricated and erected in accordance with BS 5400-6.

5.2 Robustness

Components should be sufficiently robust to facilitate handling and prevent accidental damage in service. Consideration should also be given to the possibility of vibration due to aerodynamic excitation of exposed slender members.

For compression and tension flange outstands, the ratio b_{fo}/t_{fo} should not exceed 20, where b_{fo} and t_{fo} are as defined in 9.3.2.1.

For flanges in welded beams of 12 mm thickness or less the ratio b_{fo}/t_{fo} should not exceed 16.

For flat stiffeners, the ratio h_s/t_s should not exceed 30, where h_s and t_s are as shown in Figure 1.

5.3 Handling and transport

Components should be designed with due regard to the limitations on shop and site handling capacity and to transport restrictions on bulk and/or weight.

5.4 Composite steel/concrete construction

Where steel construction is used in conjunction with concrete, composite action may be assumed provided that the design is in accordance with BS 5400-5.

5.5 Built-up members

The elements of any member built-up from parts should be joined with connections providing the rigidity assumed in design and sufficient to transmit all appropriate internal and external forces in accordance with the relevant clauses in this part of BS 5400.

5.6 Diaphragms and fixings required during construction

When, in addition to any diaphragms required for the proper functioning of the completed structure, diaphragms, bracings, brackets and cleats are provided to facilitate fabrication, transport and erection, the effects of such components on the adequacy of the structure in service (particularly in relation to fatigue) should be considered.

5.7 Camber

The structure may need to be cambered in order to satisfy the provisions of 4.6 or to achieve a satisfactory appearance of the bridge. In this connection a sagging deflection of a nominally straight soffit of 1/800 of the span should not be exceeded. The cambered shape of the structure under the action of the actual dead and superimposed dead loads should be as specified or approved by the Engineer.

5.8 End connections of beams

Where the end of a beam is required to be free to rotate, due allowance for the movement should be made in the detailed design. Where the end of a beam is restrained by its connection to adjacent parts of the structure, account should be taken of the resulting moment when designing the end of the beam, the connection and the adjacent parts of the structure.

5.9 Support cross beams

Where a deck is supported on cross beams, no part of the deck should rest directly on a pier or abutment but should be supported by cross beams in the appropriate positions.

6 Properties of materials

6.1 General

6.1.1 Performance

The mechanical properties of materials required by the Engineer should be specified in accordance with BS 5400-6. All steels should be in accordance with 6.3 and 6.4.

6.1.2 Steel grades

Unless specified otherwise, steels and steel grades should be “S” grades, no higher than S460, conforming to BS 7668, BS EN 10025, BS EN 10113, BS EN 10137, BS EN 10155 or BS EN 10210.

NOTE The design of steel elements of grades higher than S460 is not covered by the design rules for strength in this part of BS 5400.

6.2 Nominal yield stress

For steel conforming to the standards listed in **6.1.2**, and supplied to the tolerances specified in BS 4-1 (channel sections), BS EN 10029 Class A (plates), BS EN 10034 (I and H sections), BS EN 10056-2 (angles) and BS EN 10210-2 (hollow sections), the nominal yield stress, σ_y , should be taken as the minimum yield strength specified for the appropriate thickness.

When steel to specifications other than those listed in **6.1.2** is used the nominal yield stress should be taken as:

$$\left(1 - \frac{\rho_t}{100}\right)(\sigma_{ym} - k_2 \times \text{standard deviation from } \sigma_{ym})$$

where

- ρ_t is the percentage tolerance below the specified thickness permitted by BS EN 10029 Class A for plate material of the relevant thickness;
- σ_{ym} is the mean yield strength of material of the relevant thickness;
- k_2 is the coefficient as given in BS 2846-3:1975, Table 7, using the confidence level $(1 - \alpha) = 0.95$ and the proportion $P = 0.95$.

6.3 Ultimate tensile stress

The specified minimum ultimate tensile stress of steel plates and sections of steel should not be less than $1.2\sigma_y$, where σ_y is the nominal yield stress of the material as defined in **6.2**.

6.4 Ductility

Steel used in a bridge designed in accordance with this standard should have a ductility not less than that corresponding to an elongation of 15 %, based on the standard proportional gauge length of $5.65\sqrt{S_0}$, where S_0 is the cross-sectional area of the test piece.

NOTE If a non-proportional test piece is used, the percentage elongation value should be converted to the value for the proportional gauge length $5.65\sqrt{S_0}$ as above in accordance with BS EN ISO 2566-1.

Where the plastic moment capacity of a compact section is utilized or redistribution of tensile flange stresses is assumed, the ductility of the steel should be not less than that equivalent to an elongation of 19 % on the standard proportional gauge length of $5.65\sqrt{S_0}$.

6.5 Notch toughness

6.5.1 General

In order to avoid brittle fracture, the impact quality of steel parts should be selected taking into account the following:

- a) the design minimum temperature of the part (see **6.5.2**);
- b) the types of steel product and construction detail used in the part (see **6.5.3**);
- c) the stress level expected in service (see **6.5.3**);
- d) the strength grade of the steel (see **6.5.4**);
- e) the thickness of the stress carrying part (see **6.5.4**).

6.5.2 Design minimum temperature

The design minimum temperature U (in °C) to be used when applying **6.5.4** should be as follows:

- a) in a part the primary function of which is to resist thermal movement, $U = U_e - 5$;
- b) in all other parts, $U = U_e$;

where

- U_e is the minimum effective bridge temperature given in BS 5400-2 in degrees Celsius (°C).

6.5.3 Fracture classification

6.5.3.1 General

Steel parts should be classified for fracture purposes according to their k -factor:

$$k = k_d \times k_g \times k_\sigma \times k_s$$

where

- k determines the maximum permitted thickness (see 6.5.4);
- k_d depends on the detailed construction (see 6.5.3.2);
- k_g takes account of any gross stress concentration (see 6.5.3.3);
- k_σ takes account of the stress levels (see 6.5.3.4);
- k_s takes account of the rate of loading (see 6.5.3.5).

6.5.3.2 Constructional details

The value of k_d should take account of the potential fracture initiation site in accordance with Table 3a).

6.5.3.3 Geometrical stress concentrations

Account should be taken of any abrupt changes in cross-section of a part, such as at apertures, re-entrant corners or unstiffened connections between shaped members (see BS 5400-10:1980, Figures 21 and 22, and detail types marked with an asterisk in Table 3a).

In these situations the elastic stress concentration factor K should be determined at the point where the initiating detail occurs. This may not necessarily be at the point of maximum stress in the area of gross stress concentration.

The factor k_g should be determined from:

$$k_g = K^{-0.5}$$

where

- K is the ratio of the peak principal tensile stress at the initiation site (sometimes referred to as the “hot spot stress”) to the nominal principal stress at that section. Local stress concentration effects are already covered by the detail types given in Table 3a). Guidance on situations where gross stress concentrations need to be taken into account is given in BS 5400-10:1980, Figures 21 and 22 and Table 17 under “design stress parameter”.

6.5.3.4 Stress level

The factor k_σ may be selected in accordance with Table 3b), depending on the value of the maximum principal stress $\sigma_{\max}$ at the ultimate limit state.

6.5.3.5 Rate of loading

For parts particularly prone to risk of accidental impact forces, e.g. deck details close to parapet fixings or steel columns close to highway or railway traffic, such that the maximum strain in the part is likely to be experienced under this condition, a value of $k_s = 0.5$ is recommended. For all other types of loading a value of $k_s = 1$ may be used.

Table 3 — Fracture classification

a) k_d values					
Type of construction	Product forms	Detail description		k_d^c	
		Potential fracture initiation site	BS 5400-10 ^{a, b} Detail type		
Non-welded part	All	As-rolled surface	1.1, 1.2	2	
		Ground or machined edge	1.3*		
		Flame cut edge	1.4*	1.5	
		Mechanically fastened joints (drilled or reamed holes)	1.5 to 1.11		
		Mechanically fastened joints (punched holes as permitted by BS 5400-6:1999, 4.5.1)	—	1	
Welded part ^c	All	Longitudinal attachment welds on built-up members	2.1 to 2.5	1	
		Transverse weld toes at ends of short welded attachments	2.9, 2.10		
		Transverse weld toes at ends of long welded attachments ($\ell > 150$ mm)	Narrow ($w \leq 50$ mm)	2.6	0.7
			Wide ($w > 50$ mm)	$\frac{A_1}{A_2} \cdot 2.8 \cdot \frac{A_1}{A_2}$	
			On edge with unradiused ends	2.11	
		Plates	Transverse butt welds between single plates	3.1 to 3.4, 3.5*	1
	Transverse butt welds between members built up from plate		3.6, 3.7*, 3.8*, 3.9**, 3.10**	0.7	
	Sections	Transverse butt welds	3.6, 3.7*, 3.8*, 3.9**, 3.10**	0.5	
	All	Fillet welded cruciform and tee joints	3.11*	1	

^a See BS 5400-10:1980, Tables 17a, 17b and 17c.

^b Detail types marked with one asterisk * may require a k_g value of less than unity (see **6.5.3.3**). Detail types marked with two asterisks ** will require a k_g value of less than unity (see **6.5.3.3**).

^c k_d values of welded parts may be increased by 50 % if all welds have received full stress relief by post-weld heat treatment.

Table 3 — Fracture classification (continued)

b) k_{σ} values		
Stress limits	k_{σ}	
	$k_d > 0.7$	$k_d \leq 0.7$
$\sigma_{\max.} > 0.5 \sigma_y$ in tension	1	1
$0.25 \sigma_y < \sigma_{\max.} \leq 0.5 \sigma_y$ in tension	1	1.25
$\sigma_{\max.} \leq 0.25 \sigma_y$ in tension	1.5	1.5
All stresses compressive	2	2
NOTE Bridge parts may be stressed more severely in tension during erection than in service.		

Table 3 — Fracture classification (continued)

c) Thickness limits for steel parts, where $k = 1$											
Strength grade	Impact quality	Product standards	Maximum permitted thickness in millimetres according to design minimum temperature U in °C								
			$U = 0$	-5	-10	-15	-20	-25	-30	-40	-50
S235	J0	BS EN 10025, BS EN 10155	89	81	74	68	62	0	0	0	0
	J2	BS EN 10025, BS EN 10155	128	117	107	98	89	81	74	62	0
S275	J0	BS EN 10025, BS EN 10210	71	65	60	54	50	0	0	0	0
	J2	BS EN 10025, BS EN 10210	103	94	86	78	71	65	60	50	0
	N	BS EN 10113-2, BS EN 10210	124	113	103	94	86	78	71	60	50
	M	BS EN 10113-3									
	NL	BS EN 10113-2, BS EN 10210	178	162	148	135	124	113	103	86	71
	ML	BS EN 10113-3									
S355	J0	BS EN 10025, BS EN 10155, BS EN 10210	50	46	42	38	35	0	0	0	0
	J2	BS EN 10025, BS EN 10155, BS EN 10210	72	66	60	55	50	46	42	35	0
	K2	BS EN 10025, BS EN 10155	86	79	72	66	60	55	50	42	35
	N	BS EN 10113-2, BS EN 10210									
	M	BS EN 10113-3									
	NL	BS EN 10113-2, BS EN 10210	124	114	104	95	86	79	72	60	50
	ML	BS EN 10113-3									
S420	N	BS EN 10113-2	68	62	57	52	47	43	40	33	27
	M	BS EN 10113-3									
	NL	BS EN 10113-2	98	90	82	75	68	62	57	47	40
	ML	BS EN 10113-3									
S460	Q	BS EN 10137	50	46	42	38	35	32	29	24	0
	N	BS EN 10113-2, BS EN 10210	60	55	50	46	42	38	35	29	24
	M	BS EN 10113-3									
	QL	BS EN 10137	72	66	60	55	50	46	42	35	29
	NL	BS EN 10113-2, BS EN 10210	87	79	72	66	60	55	46	42	35
	ML	BS EN 10113-3									
	QL1	BS EN 10137	104	95	87	79	72	66	60	50	42

NOTE The Charpy impact specifications in the product standards do not apply above the following thicknesses: BS EN 10025: 100 mm; BS EN 10113: 150 mm (except S460N/NL, 100 mm and M/ML flat products, 63 mm); BS EN 10137: 150 mm; BS EN 10155: 100 mm; BS EN 10210: 65 mm. Thickness limits in the table above these thicknesses should only be used if agreement is reached with the supplier to extend the range of application of the impact specification.

6.5.4 Maximum permitted thickness

The thickness t of a steel part should be limited as follows:

where $U \geq T_{27J} - 20$

$$t \leq 50k \left(\frac{355}{\sigma_y} \right)^{1.4} 1.2^{\left(\frac{U - T_{27J}}{10} \right)}$$

$U < T_{27J} - 20$, not permitted.

where

- t is the maximum permitted thickness of the part under stress in millimetres;
- k is the k -factor from **6.5.3.1**;
- σ_y is the nominal yield stress of the part, as defined in **6.2**;
- U is the design minimum temperature of the part in degrees Celsius ($^{\circ}\text{C}$) (see **6.5.2**);
- T_{27J} is the test temperature in degrees Celsius ($^{\circ}\text{C}$) for which a minimum Charpy energy of 27 J is specified by the product standard for impact tests on longitudinal V-notch test pieces.

NOTE The importance of the correct signs for temperature in the formula is emphasized.

For the products and grades which do not have a required minimum Charpy energy, C_v , equal to 27 J, the following assumptions may be made:

- for $C_v = 30 \text{ J}$, $T_{27J} \equiv T_{30J}$;
- for $C_v = 40 \text{ J}$, $T_{27J} \equiv T_{40J} - 10$;

where T_{30J} and T_{40J} are the specified test temperatures for $C_v = 30 \text{ J}$ and $C_v = 40 \text{ J}$ respectively.

Maximum thickness limits according to steel grade and impact quality are given for a range of design minimum temperatures in Table 3c), for $k = 1$.

6.6 Properties of steel

The following properties of steel should be assumed in design:

- modulus of elasticity, $E = 205\,000 \text{ N/mm}^2$;
- shear modulus, $G = 80\,000 \text{ N/mm}^2$;
- Poisson's ratio, $\nu = 0.3$;
- coefficient of thermal expansion $= 12 \times 10^{-6}/^{\circ}\text{C}$.

6.7 Modular ratio

For global analysis of bridges of composite construction the modular ratio may be based on the long term value of the elastic modulus for concrete unless stated otherwise in this part of BS 5400. For stress analysis the modular ratio appropriate to the stage of construction and the type of loading should be adopted.

7 Global analysis for load effects

7.1 General

The global analysis of the structure should be in accordance with BS 5400-1 using an elastic method of analysis. For structures in which the load effects are not proportional to the loads and/or the secondary effects due to deformation are significant, the method of analysis should be suitable for treatment of non-linear behaviour.

7.2 Sectional properties

The sectional properties to be used in global analysis should generally be calculated for the gross cross-section assuming the specified sizes. The influence of shear lag on the stiffness of elements should, however, be taken into account in the following cases:

- a) for beams or trusses on flexible supports or in cable-stayed bridges;
- b) for analysis of conditions during erection of continuous girders of box construction or with integral decks;
- c) for calculating deflections of beams.

7.3 Allowance for shear lag

When the effects of shear lag are to be taken into account in global analysis, in accordance with 7.2, this should be allowed for by using an effective breadth of flange in calculating the cross-sectional properties of the members. This may be done in a manner similar to that described in 8.2 for stress analysis, using the values of ψ in Tables 4, 5, 6 and 7 and in annex A. The values for the quarter span may be adopted to calculate effective sectional properties for global analysis at all sections in the span.

8 Stress analysis

8.1 Longitudinal stresses in beams

The distribution of longitudinal stress between the flanges and web or webs of a beam may be calculated on the assumption that plane sections remain plane, but using the effective widths of flanges and the effective thickness of a deep web in accordance with 8.2 and 9.4.2.5, respectively. No further account need be taken of deformation of plating out of its plane. In composite construction the area of concrete in a tensile zone should be ignored.

Subject to the provisions of 9.5.4, plate panels in webs may be assumed to shed a proportion of their longitudinal stress to the flanges.

8.2 Allowance for shear lag

Where, in order to meet the provisions of 9.2.3, the effect of in-plane shear flexibility (i.e. shear lag) is to be allowed for in calculating the stress in a flange, either the elastic stress distribution should be calculated by finite element analysis or an equivalent flange may be assumed having an effective breadth equal to the sum of the effective breadths of the portions of flange on each side of the web. The effective breadth, b_e , of each portion should be taken as:

- a) ψb for portions between webs;

where

b is half the distance between centres of webs measured along the mid-plane of the flange plate;

- b) $k\psi b$ for portions projecting beyond an outer web;

where

b is the distance from the free edge of the projecting portion to the centre of the outer web, measured along the mid-plane of the flange plate;

$k = (1 - 0.15b/L)$;

and where in a) and b) and in Tables 4 to 7

ψ is the appropriate effective breadth ratio taken from Tables 4, 5, 6 and 7 for uniformly distributed loads which should be used for standard highway or railway loading as specified in BS 5400-2, including wheel and axle loads;

L is the span of a beam between centres of support, or in the case of a cantilever beam, between the support and the free end;

$a = 0$ if there are no stiffeners on the flange within the width b in the span direction, otherwise

$$= \frac{\text{sectional area of flange stiffeners in width } b}{\text{sectional area of flange plate in width } b}$$

Values of ψ for intermediate values of b/L and a and for intermediate positions in the span may be obtained by linear interpolation.

The value of ψ at an interior support should be taken as the mean of the values obtained for adjacent spans. For end spans of continuous beams the effective breadth ratios may be obtained by treating the end span as a propped cantilever of the same span.

For the purpose of calculating deflections of beams, the values of ψ given in Tables 4, 5, 6 and 7 and in annex A for the quarter span sections may be adopted for all sections in the span.

For intermediate structures, for point loads and for combinations of point and distributed loads, not specifically covered above, the effective breadth ratios ψ may be determined by the methods given in annex A.

When finite element analysis is used, the effective breadth for use with 9.4.2 should be taken as the breadth of each portion multiplied by the mean stress in the portion divided by the peak stress in the same portion.

Table 4 — Effective breadth ratio ψ for simply supported beams

b/L	Mid-span		Quarter span		Support	
	$a = 0$	$a = 1$	$a = 0$	$a = 1$	$a = 0$	$a = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.98	0.97	0.98	0.96	0.84	0.77
0.10	0.95	0.89	0.93	0.86	0.70	0.60
0.20	0.81	0.67	0.77	0.62	0.52	0.38
0.30	0.66	0.47	0.61	0.44	0.40	0.28
0.40	0.50	0.35	0.46	0.32	0.32	0.22
0.50	0.38	0.28	0.36	0.25	0.27	0.18
0.75	0.22	0.17	0.20	0.16	0.17	0.12
1.00	0.16	0.12	0.15	0.11	0.12	0.09

Table 5 — Effective breadth ratio ψ for interior spans of continuous beams

b/L	Mid-span		Quarter span		Support	
	$a = 0$	$a = 1$	$a = 0$	$a = 1$	$a = 0$	$a = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.96	0.91	0.85	0.76	0.58	0.50
0.10	0.86	0.72	0.68	0.55	0.41	0.32
0.20	0.58	0.40	0.42	0.31	0.24	0.17
0.30	0.38	0.27	0.30	0.20	0.15	0.11
0.40	0.24	0.18	0.21	0.14	0.12	0.08
0.50	0.20	0.14	0.16	0.11	0.11	0.07
0.75	0.15	0.10	0.10	0.08	0.09	0.06
1.00	0.13	0.09	0.09	0.07	0.07	0.05

Table 6 — Effective breadth ratio ψ for propped cantilever beams

b/L	Fixed end		Quarter span near fixed end		Propped end	
	$a = 0$	$a = 1$	$a = 0$	$a = 1$	$a = 0$	$a = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.62	0.54	1.00	1.00	0.79	0.70
0.10	0.45	0.38	1.00	1.00	0.63	0.52
0.20	0.27	0.21	0.92	0.76	0.44	0.32
0.30	0.18	0.14	0.72	0.53	0.33	0.23
0.40	0.13	0.10	0.46	0.35	0.24	0.16
0.50	0.11	0.08	0.31	0.25	0.19	0.13
0.75	0.10	0.07	0.21	0.16	0.12	0.09
1.00	0.09	0.06	0.19	0.15	0.08	0.07

Table 7 — Effective breadth ratio ψ for cantilever beams

b/L	Fixed end		Quarter span near fixed end		Free end	
	$a = 0$	$a = 1$	$a = 0$	$a = 1$	$a = 0$	$a = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.82	0.76	1.00	1.00	0.92	0.86
0.10	0.68	0.61	1.00	1.00	0.84	0.77
0.20	0.52	0.44	1.00	1.00	0.70	0.60
0.30	0.42	0.35	0.95	0.90	0.60	0.48
0.40	0.35	0.28	0.88	0.75	0.52	0.38
0.50	0.30	0.25	0.76	0.62	0.40	0.33
0.75	0.22	0.18	0.52	0.38	0.34	0.23
1.00	0.18	0.14	0.38	0.27	0.27	0.18

8.3 Distortion and warping stresses in box girders

Stresses in a box girder due to transverse bending of the walls of the box and torsional and distortional warping should, where needed for calculations in accordance with **9.2.1.2**, **9.2.1.3**, **9.2.3.1** and **9.2.3.2**, be calculated by linear elastic analysis. The simplified procedures given in **B.2** may be used for the calculation of torsional warping stresses and those in **B.4** for distortional bending stresses. Those given in **B.3** may be used for calculation of distortional warping stresses when torque is applied other than at a cross frame or diaphragm.

8.4 Shear stresses

The design values of the shear stress in webs of rolled or fabricated I, box or channel sections may be calculated in accordance with **9.5.1**. Shear stresses in other sections should be computed from the whole cross-section having regard to the distribution of flexural stress across the section.

8.5 Imperfections

8.5.1 Imperfections allowed for

The design strengths given in this part of BS 5400 may be assumed to have made allowance for the following tolerances where these apply:

- bearing misalignment in plan, errors in level of a single bearing or in the mean level of more than one bearing at any support, and bearing inclination within the tolerances given in BS 5400-9;
- imperfect flatness and straightness of compression members and of stiffened and unstiffened plate panels within the tolerances given in BS 5400-6.

8.5.2 Imperfections to be allowed for separately

8.5.2.1 Torsionally stiff girders

Imperfections in common planarity of bearings should be allowed for in the analysis of torsional moments and reactions for torsionally stiff superstructures and should be compatible with tolerances, which the designer should specify, and the construction method and sequence used.

8.5.2.2 Columns

Where a column is supported on a rocker bearing, the following eccentricities should be added to any calculated eccentricity of the bearing reaction:

- a) for a flat-topped rocker bearing in contact with a flat bearing surface beneath the column: half the width of the flat bearing surface plus 10 mm;
- b) for a radiused rocker bearing: 3 mm if the bearing is attached to the end of the column during fabrication in a position that is nominally central and 10 mm in all other cases.

8.6 Residual stresses

The design strengths given in this part of BS 5400 may be deemed to allow for residual stresses due to rolling, handling and transportation, and from those arising from normal welding procedures.

In order to make allowance for the relaxation of residual stresses due to welding when estimating deflections during erection, or on first loading, the effective area A_e of a flange in tension should, for this purpose, be taken as:

$$A_e = A_f \left(1 - \frac{CA_w}{\sigma_y A_f} \right)$$

where

- A_f is the gross cross-sectional area of flange in tension, inclusive of longitudinal stiffeners;
- A_w is the volume of longitudinal weld per unit length of flange;
- C is a weld shrinkage coefficient and may be taken as 7 000 N/mm²;
- σ_y is the nominal yield stress of the flange material as defined in 6.2.

9 Design of beams

9.1 General

Beams are defined as members with solid webs (or with openings in accordance with 9.3.3), subjected primarily to bending, including members of rolled and hollow section, plate girders and box girders.

9.2 Limit states

9.2.1 Ultimate limit states

9.2.1.1 General

Beams should be designed to satisfy the provisions of clause 9 for the ultimate limit state.

9.2.1.2 Effects to be considered

The effects at the ultimate limit state should be obtained for the relevant combinations of:

- flexure, shear, torsion and, for box girders, distortion, due to any loads transverse to the longitudinal axis of the member;
- the effects of axial load;
- creep, shrinkage and differential temperature (see BS 5400-5 for composite structures);
- settlement of supports.

9.2.1.3 Effects that may be neglected

The effects of the following may be neglected for the ultimate limit state:

- shear lag;
- restraint of torsional warping;
- for box girders, restraint of distortional warping provided that any torque is directly transmitted (by the deck system) to cross frames or diaphragms which are in accordance with 9.16 or 9.17;
- items c) and d) of 9.2.1.2 provided that:
 - the section is compact throughout the span being considered in accordance with 9.3.7; and
 - the member is not prone to lateral instability; this may be deemed to be satisfied when the slenderness parameter, λ_{LT} , is less than:

$$30 \sqrt{\frac{355M_{pe}}{\sigma_y M_{ult}}}$$

where

- λ_{LT} is defined in 9.7;
 σ_y is defined in 6.2;
 M_{pe} is defined in 9.7.1;
 M_{ult} is defined in 9.8.

9.2.2 Fatigue

The fatigue endurance should be in accordance with the recommendations of BS 5400-10.

9.2.3 Serviceability limit state

9.2.3.1 General

The serviceability limit state provisions should additionally be met where called for by the following:

- when the effective breadth ratio ψ , determined in accordance with 8.2, is less than the restricting value ψ_R for a particular flange portion.

For each portion of flange (i.e. between webs or between a web and a free edge of a flange) the restricting shear lag factor, ψ_R , is given by the lesser of 0.77 or $m_{f3}m_m m_{fl}$, where:

$$m_{f3} = \frac{\gamma_{f3, SLS}}{\gamma_{f3, ULS}} \quad m_m = \frac{\gamma_{m, SLS}}{\gamma_{m, ULS}}$$

$$m_{fl} = \frac{(\gamma_{fl, DEAD} a_{DEAD} + \gamma_{fl, SUP} a_{SUP} + \gamma_{fl, LIVE} a_{LIVE} + \gamma_{fl, O} a_O)_{SLS}}{(\gamma_{fl, DEAD} a_{DEAD} + \gamma_{fl, SUP} a_{SUP} + \gamma_{fl, LIVE} a_{LIVE} + \gamma_{fl, O} a_O)_{ULS}}$$

where

a_{DEAD} , a_{SUP} , a_{LIVE} , a_{O} are the fractions of nominal load effect (i.e. of the mean stress in the flange portion, calculated using gross section properties) due to dead load, superimposed load, live load and other loads respectively;

NOTE For plate and box girder construction, serviceability limit state checks are not needed when ψ is greater than 0.77.

- b) when redistribution of stresses from the tension flange is made in accordance with **9.5.5**;
- c) when stiffened flanges are subjected to bending by local loads (see **9.10.3.3**);
- d) when called for in **9.9.8** for unsymmetrical beams about the axis of bending;
- e) when torque is applied to a box girder other than at a cross frame or diaphragm and distortional warping stresses have been ignored in calculating the effects of torque at the ultimate limit state.

9.2.3.2 Effects to be considered

The effects at the serviceability limit state should be obtained for the relevant combinations given in **9.2.1.2a**) to d) together with:

- a) the effects of shear lag;
- b) the effects of restraint of torsional warping;
- c) the effects of restraint of distortional warping when torque is applied other than at an effective cross frame or diaphragm.

9.2.4 Composite beams

In the design of composite beams, the concrete, reinforcement and shear connectors should satisfy the limit state recommendations of BS 5400-4 and BS 5400-5.

9.3 Shape limitations

9.3.1 General

Figure 1 sets out the geometric notation used in clause **9**. Components should be in accordance with the recommendations for robustness given in **5.2**.

The recommendations for strength in clause **9** are defined by reference to the nominal yield stress value for the component.

The nominal yield stress value, σ_{ys} or σ_y , should be taken as either:

- a) the nominal yield stress of the material as defined in **6.2**; or
- b) a lesser value such that the component conforms to the shape limitations given in **9.3.2.1**, **9.3.4** or **9.3.6**, as appropriate.

NOTE 1 Where the strength is defined by reference to **9.3.1**, the symbols for nominal yield stress value are not always in accordance with those in **9.3.1**. The reference should then be taken to refer to σ_y or σ_{ys} as appropriate: σ_y for plates and outstands, or σ_{ys} for stiffeners.

NOTE 2 When the stress in the component is tensile, the nominal yield stress value is the nominal yield stress of the material as defined in **6.2**.

9.3.2 Flanges

9.3.2.1 Flange outstands in compression

Unless a free edge of a plate or other outstand in compression is stiffened, the ratio b_{fo}/t_{fo} should not exceed

$$12 \sqrt{355/\sigma_y}$$

When the edge of the outstand is stiffened the ratio b_{fo}/t_{fo} should not exceed

$$14 \sqrt{355/\sigma_y},$$

where

- b_{fo} is the width of the outstand measured from the edge to the nearest line of rivets or bolts connecting it to the supporting part of the member, or to the toe of a root fillet of a rolled section, or, in the case of a welded construction, to the surface of the supporting part of the member, or, in the case of composite construction, to the outer line of shear connectors. Where a flange is built-up from several plates it should, in addition, be in accordance with **9.3.2.2**;
- t_{fo} is the mean thickness of the outstand;
- σ_y is as defined in **9.3.1**.

9.3.2.2 Flanges of built-up plates

Where a flange (whether in tension or compression) consists of several flange plates built-up and connected to each other only by welds at their edges, an outer flange plate should not be thicker than an inner plate and 9.3.2.1 should be satisfied for all the flange plates. For the flange plate connected to the web, b_{fo} should be taken as given in 9.3.2.1 but for all the other flange plates b_{fo} should be taken as half the width between the welds connecting it to the adjacent inner plate.

9.3.3 Openings

9.3.3.1 General

Any openings in webs or compression flanges should be framed and the stiffened section designed for local load effects, including secondary bending. Alternatively, openings in webs may be unstiffened provided that they meet the provisions of 9.3.3.2.

All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.

9.3.3.2 Unstiffened openings in webs

Openings in a web may be unstiffened provided that:

- the overall greatest internal dimension does not exceed one-tenth of the depth of the web, nor, for longitudinally stiffened webs, one-third of the depth of the panel containing the opening;
- the longitudinal distance between the boundaries of adjacent openings is at least three times the maximum internal dimension;
- not more than one opening is provided at any cross-section.

Cut-outs provided for transverse stiffeners should either have at least one-third of their perimeters welded to the stiffeners, or the stiffeners should be cleated to the web with at least two bolts or rivets per side of the connection or by full perimeter welding of the cleat.

9.3.4 Stiffeners to webs and compression flanges

9.3.4.1 Open stiffeners to webs and compression flanges

9.3.4.1.1 General

Open stiffeners should either be in accordance with 9.3.4.1.2, 9.3.4.1.3, 9.3.4.1.4, or 9.3.4.1.5, as appropriate, or be checked in accordance with annex C.

9.3.4.1.2 Flat stiffeners

The stiffener proportions should be such that $\frac{h_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed:

- a value of 10; or
- a higher value obtained from Figure 2 when $\frac{b}{t} \sqrt{\frac{\sigma_y}{355}}$ is less than 31;

where, as shown in Figure 1:

- h_s, t_s are the depth and thickness, respectively, of the stiffener;
- b is the spacing of stiffeners or distance between the stiffener and the beam flange/web boundary, as appropriate;
NOTE In the case of non-uniform spacing the average value on the two sides may be taken.
- t is the plate thickness t_f or t_w ;
- σ_{ys}, σ_y are as defined in 9.3.1.

9.3.4.1.3 Bulb flat stiffeners

Bulb flat stiffeners should conform to BS EN 10067 and should be proportioned such that:

- either $\frac{\ell_s}{h_s}$ does not exceed $3 \sqrt{\frac{355}{\sigma_{ys}}}$; or
- $\frac{(b + k_s h_s)}{t} \sqrt{\frac{\sigma_y}{355}}$ does not exceed 30;

where

- ℓ_s is the span of the stiffener between supporting members;
- h_s is the overall depth of the bulb flat, as shown in Figure 1;
- k_s = 0.4 for grade S355 steel in accordance with the standards listed in 6.1.2, or $k_s = 0.15$ for grade S275 steel in accordance with the standards listed in 6.1.2;
- b, t are as defined in 9.3.4.1.2;
- σ_{ys}, σ_y are as defined in 9.3.1.

9.3.4.1.4 Angle stiffeners

Angle stiffeners should conform to BS EN 10056-1 and should be proportioned such that:

- a) b_s does not exceed h_s ;
- b) $\frac{b_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 11;
- c) $\frac{h_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed either:
 - 1) a value of 7; or
 - 2) a higher value obtained from Figure 3 when $\frac{\ell_s}{b_s} \sqrt{\frac{\sigma_{ys}}{355}}$ is less than 50;

NOTE When $\frac{b}{t} \sqrt{\frac{\sigma_y}{355}}$ is less than or equal to 30, there is no limitation on $\frac{h_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$

where

- h_s, b_s, t_s are the depth, width and thickness of the angle, respectively, as shown in Figure 1;
- b, t are as defined in 9.3.4.1.2;
- ℓ_s is as defined in 9.3.4.1.3;
- σ_{ys}, σ_y are as defined in 9.3.1.

9.3.4.1.5 Tee stiffeners

Tee stiffeners should be proportioned such that:

- a) $\frac{b_{so}}{t_{so}} \sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 10;
- b) $\frac{d_s}{t_s} \sqrt{\frac{\sigma_{ys} + \sigma_a}{355}}$ does not exceed 41;
- c) $\frac{d_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 7 or a higher value obtained from either:
 - 1) Figure 4a) when $\frac{\ell_s}{b_s} \sqrt{\frac{\sigma_{ys}}{355}}$ is less than 25; or
 - 2) Figure 4b) when $\frac{b}{t} \sqrt{\frac{\sigma_y}{355}}$ is less than 32 and $\frac{d_s}{b_s}$ does not exceed 4;

where

- b_s, t_s are as shown in Figure 1;
- b, t are as defined in 9.3.4.1.2;
- ℓ_s is as defined in 9.3.4.1.3;
- t_{so} is the average thickness of the flange outstand width b_{so} ;
- d_s is the effective stiffener depth, measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee (see Figure 1);
- σ_a is the longitudinal stress (positive when compressive) for the ultimate limit state at the centroid of the effective section of the stiffener, as given in 9.10.2.2 for flanges and 9.11.5.1 for webs, but may be taken conservatively as σ_{ys} ;
- σ_{ys}, σ_y are as defined in 9.3.1.

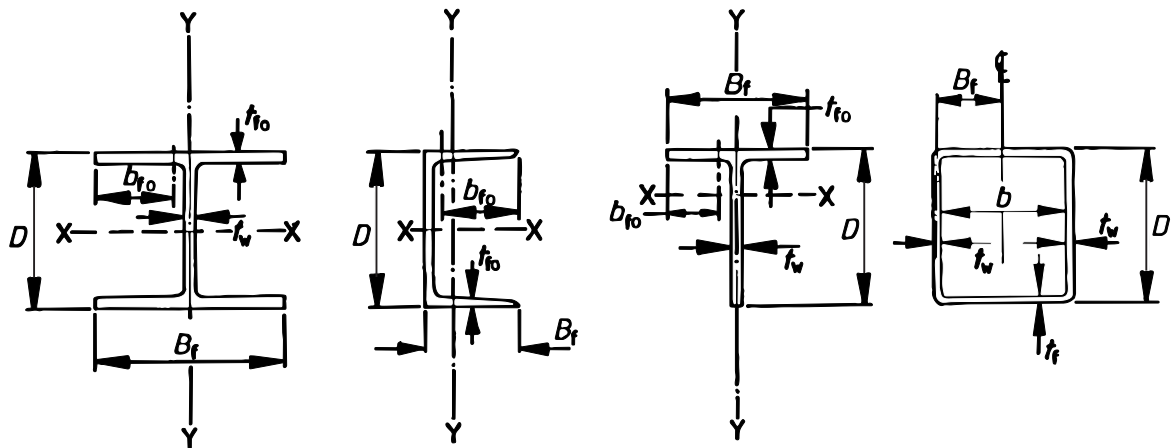
9.3.4.2 Closed stiffeners to webs and compression flanges

Closed stiffeners should be proportioned such that:

- $\frac{d_{s1}}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 29;
- $\frac{d_{s2}}{t_s} \sqrt{\frac{\sigma_{ys} + \sigma_a}{355}}$ does not exceed 41.

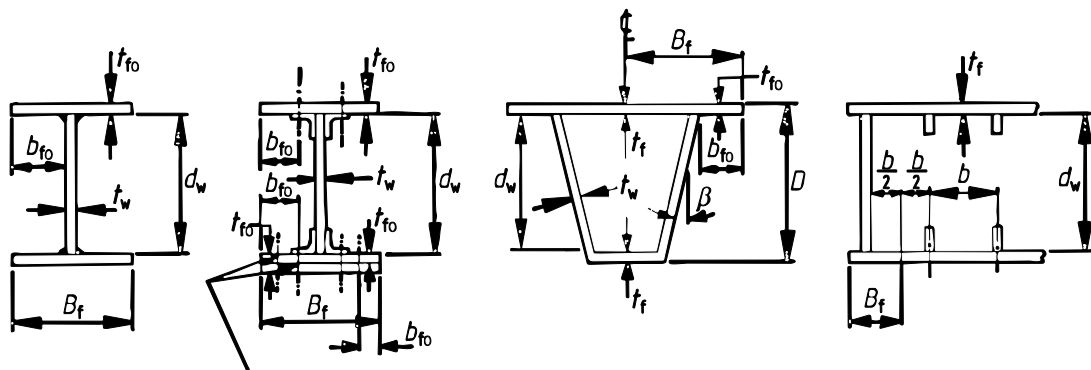
where

- σ_a is as defined in 9.3.4.1.5;
 d_{s1} , d_{s2} are the widths of the walls of the stiffener as shown in Figure 1;
 t_s is the thickness of the stiffener;
 σ_{ys} is as defined in 9.3.1.



NOTE For tapered flanges, t_{fo} should be taken as the mean thickness of the flange, as defined in BS 4-1.

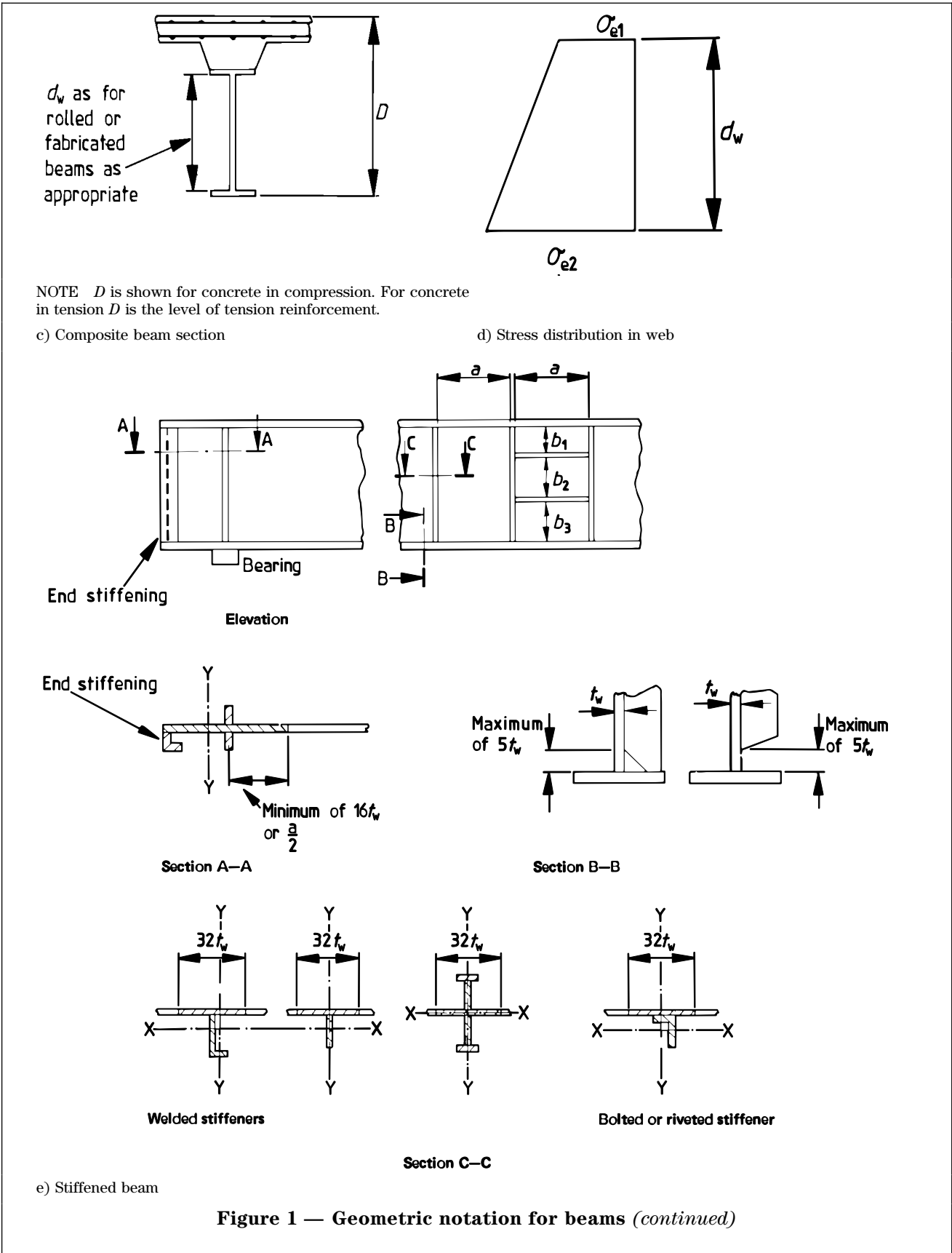
a) Rolled beam sections, $d_w = D$

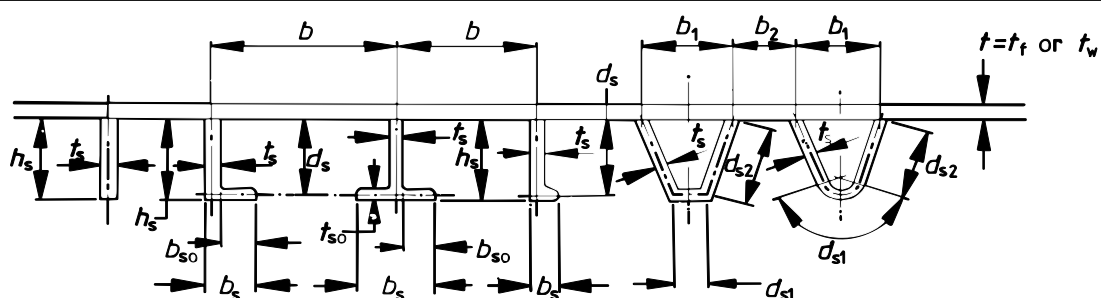


Position of fasteners

b) Fabricated beam sections, d_w is as shown

Figure 1 — Geometric notation for beams

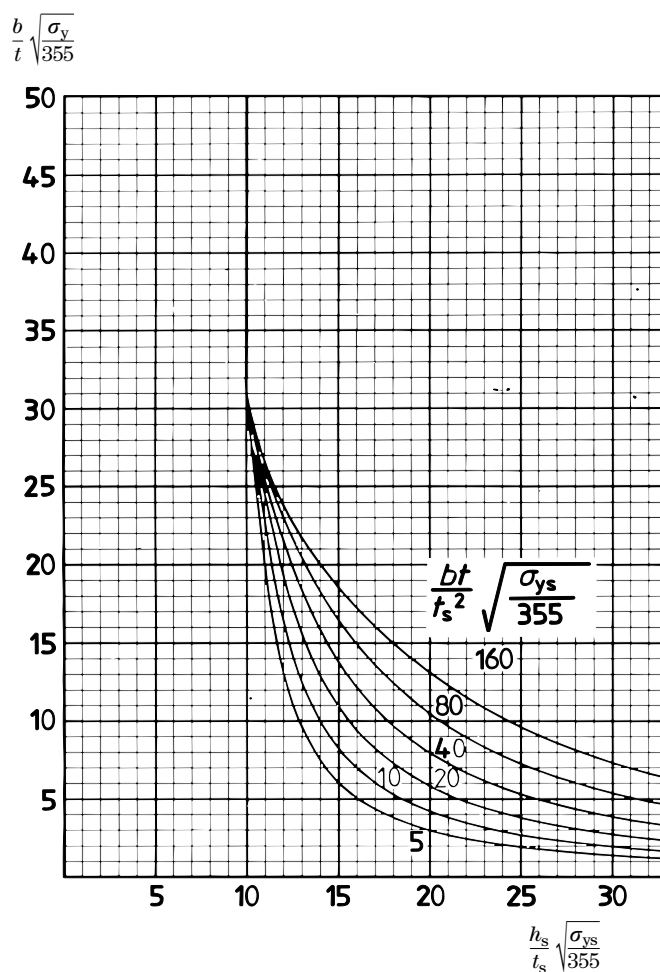




NOTE d_{s1} , d_{s2} measured on centreline.

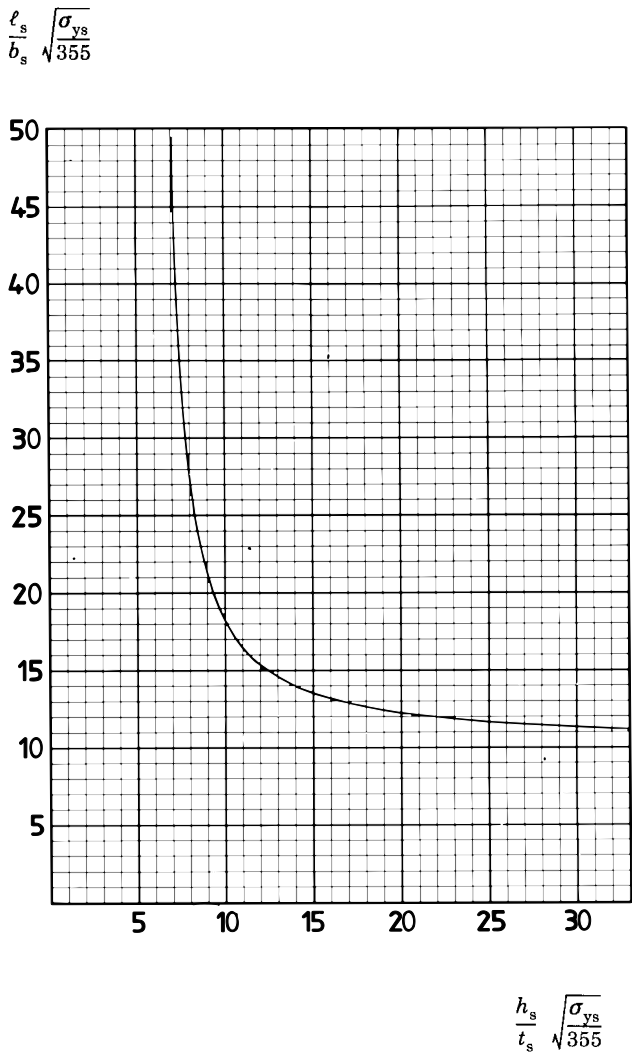
f) Flange and web stiffeners

Figure 1 — Geometric notation for beams (continued)



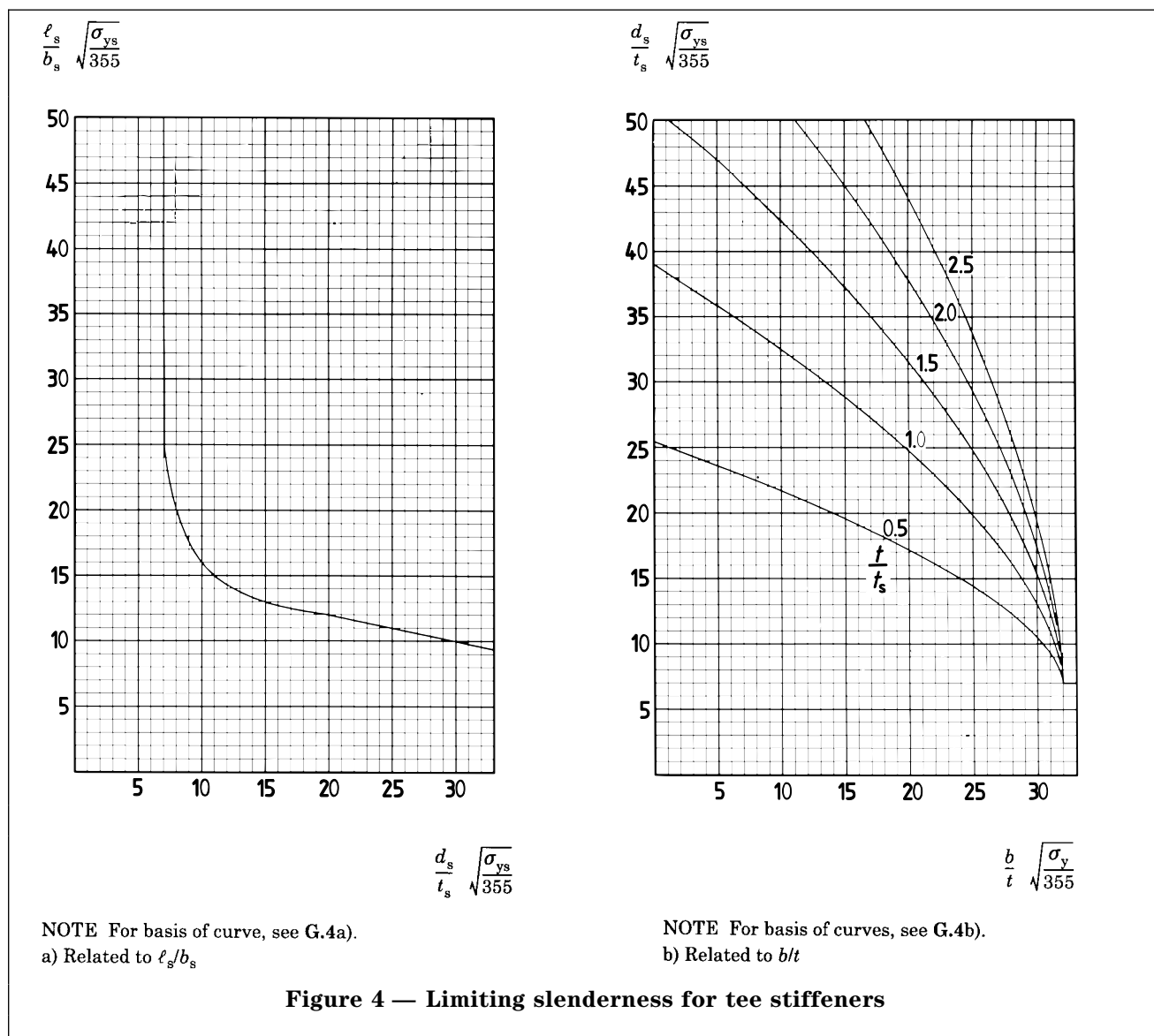
NOTE For basis of curves, see G.2.

Figure 2 — Limiting slenderness for flat stiffeners



NOTE For basis of curve, see G.3.

Figure 3 — Limiting slenderness for angle stiffeners



9.3.5 Flanges curved in elevation

Flanges curved in elevation should be such that:

- $\frac{b_{fo}}{t_{fo}} \leq \frac{R_f}{6b_{fo}}$; and
- $\frac{b}{t_f} \leq \frac{R_f}{2b}$

where

- R_f is the radius of curvature of the flange;
- b is the distance between adjacent longitudinal stiffeners and/or webs;
- t_f is the thickness of the flange in the panel between such longitudinal stiffeners and/or webs;
- b_{fo} , t_{fo} are as defined in 9.3.2.

9.3.6 Circular hollow sections

The ratio of outside diameter to wall thickness of a circular hollow section should not exceed:

$$60 \left(\frac{355}{\sigma_y} \right)$$

where

σ_y is as defined in 9.3.1.

9.3.7 Compact sections

9.3.7.1 General

Compact sections are those in which the full plastic moment can be developed before, and maintained after, the onset of local buckling. Rolled or fabricated I-beams, channels or hollow sections are defined as compact provided that they meet the provisions of 9.3.7.2 and 9.3.7.3, or 9.3.7.4, as appropriate.

Longitudinal stiffeners, if any, should be ignored in calculating the section properties and in deriving the strength of a compact beam.

NOTE For beams built in stages, the recommendations should be applied to the cross-section of the beam appropriate to the stage considered (see 9.9.5).

9.3.7.2 Webs

The depth of the web should not exceed:

$$\frac{34t_w}{m} \sqrt{\frac{355}{\sigma_{yw}}} \text{ when } m \text{ does not exceed } 0.5; \text{ and}$$

$$\frac{374t_w}{(13m - 1)} \sqrt{\frac{355}{\sigma_{yw}}} \text{ when } m \text{ exceeds } 0.5;$$

where

m is the ratio of the depth of the web plate which is on the compressive side of the plastic neutral axis of the beam to the depth of the web plate;

t_w is the thickness of the web plate;

σ_{yw} is the nominal yield stress of the web material as defined in 6.2.

NOTE The depth of the web referred to in this clause should be measured in its plane and taken clear of root fillets for rolled sections and welds or flange angles for fabricated sections.

9.3.7.3 Compression flanges

9.3.7.3.1 Compression flange outstands

The projection of a compression outstand, b_{fo} , should not exceed:

$$7t_{fo} \sqrt{\frac{355}{\sigma_{yf}}}$$

where

σ_{yf} is the nominal yield stress of the flange material as defined in 6.2;

t_{fo} , b_{fo} are as defined in 9.3.2.1.

9.3.7.3.2 Compression flanges in box sections

The clear width of the compression flange should not exceed:

$$24t_f \sqrt{\frac{355}{\sigma_{yf}}}$$

where

t_f is the thickness of the compression flange plate;

σ_{yf} is the nominal yield stress of the flange material as defined in 6.2.

The clear width is taken as between:

- root fillets for rolled sections;
- webs for welded constructions;
- lines of rivets or bolts connecting the flange to the web for other fabricated sections.

9.3.7.3.3 Composite compression flanges

In composite compression flanges, where the flange plate is connected to the concrete by shear connectors in accordance with BS 5400-5, the limits of 9.3.7.3.1 or 9.3.7.3.2 may be exceeded, provided that the spacing of the shear connectors perpendicular to the direction of compression does not exceed:

$$30t_f \sqrt{\frac{355}{\sigma_{yf}}}$$

and the spacing in the direction of compression does not exceed:

$$15t_f \sqrt{\frac{355}{\sigma_{yf}}}$$

but may be increased to:

$$22t_f \sqrt{\frac{355}{\sigma_{yf}}}$$

in any line of shear connectors where the adjacent lines are staggered,

where

t_f and σ_{yf} are as defined in 9.3.7.3.2.

9.3.7.4 Circular hollow sections

The ratio of the outside diameter to the wall thickness of a circular hollow section should not exceed:

$$46 \left(\frac{355}{\sigma_y} \right)$$

where

σ_y is the nominal yield stress of the material of the hollow circular section as defined in 6.2.

9.4 Effective section

9.4.1 Effective section for global analysis

Gross section properties may be used for global analysis except for transverse members, for which reference should be made to 9.15.2.1. For composite construction the procedure set out in BS 5400-5:1979, 5.1 should be adopted for all limit states.

9.4.2 Effective section for bending stress analysis of beams

9.4.2.1 General

The elastic modulus of a section, or the plastic modulus of a section, should be determined by taking account of the provisions of 9.4.2.2 to 9.4.2.7.

NOTE Additional or alternative provisions are given elsewhere for specific elements such as stiffeners.

9.4.2.2 Shear lag effects

When the effects of shear lag are to be taken into account in accordance with 9.2.3 an effective breadth of a flange should be used, determined in accordance with 8.2.

9.4.2.3 Effective areas of tension flanges

The effective area of a tension flange should be taken as the net area after allowance for any holes in accordance with 11.3.3 and for shear lag in accordance with 9.4.2.2.

The net area should be taken as the gross area within the effective breadth multiplied by A_t/A ,

where

A_t is as defined in 11.3.3;

A is the gross area of the whole flange.

9.4.2.4 Effective areas of compression flanges or plates in compression

The effective area of a compression flange or a plate in compression should be taken as the combined effective net areas of plating and longitudinal stiffeners (if any), with allowance for shear lag in accordance with 9.4.2.2.

The effective area should be taken as $A_e \sum b_e / B_f$

where

- A_e is the sum of the effective net areas of the component parts of the flange or plate in compression and is given by $\sum K_c (k_h A_c)$;
- K_c is taken as 1.0 for all outstands which are in accordance with 9.3.2, and for all stiffeners which are in accordance with 9.3.4, or as a coefficient, to be determined from Figure 5, for plates supported by adjacent components as follows:
- when the number of open stiffeners is three or more, or the number of closed stiffeners is two or more, with any type of connection to the boundaries or to the stiffeners, from curve 1 or curve 3, whichever is the greater;
 - when there is only one longitudinal stiffener, or the flange is unstiffened, from curve 1 for panels with bolted or riveted connections to the boundaries and the stiffeners or curve 2 for rolled sections or panels in welded construction, or curve 3 for any form of construction, whichever is the greater;
 - when the number of open stiffeners is two, from curve 1 for panels with bolted or riveted connections to the boundaries and stiffeners, or as the mean of the values from curves 1 and 2 for panels in welded construction, or from curve 3 for any form of construction, whichever is the greater;
- k_h $\overline{A_1} = 1.0$ for a section free from holes or for a section with one or more holes greater than 40 mm in diameter; or $\overline{A_1}$
 $= 1.2$ for a section in which holes do not exceed 40 mm in diameter, provided that $k_h A_c$ in no case exceeds the gross area of the component part;
- A_c is the net area of each component part, equal to its gross area less a deduction across a section perpendicular to the longitudinal axis for open holes or clearance holes for pins, black bolts or countersunk bolts. Holes filled with rivets, HSFG bolts, close tolerance or turned barrel bolts, or fully filled plugged holes need not be deducted;
- $\sum b_e$ is the sum of the effective widths of the flange or plate in compression (see 8.2);
- B_f is the total width of the flange or plate.

NOTE 1 In using Figure 5 the value of the slenderness parameter λ should be taken as:

$$\lambda = \frac{b}{t_f} \sqrt{\frac{\sigma_{yf}}{355}}$$

for panels in a stiffened compression flange or plate, or for an unstiffened box.

In the case of closed stiffeners, when b_1 and b_2 are different, an average value may be taken provided:

$$0.67 \leq \frac{b_1}{b_2} \leq 1.5$$

where

- σ_{yf} is the nominal yield stress of the plate material as defined in 6.2;
- b_1, b_2 are as shown in Figure 1;
- b is the unsupported width of plate between adjacent lines of bolts or rivets connecting the plate to the supporting parts, or, for welded construction, between the surfaces of the supporting parts, or, for rolled sections, clear between toes of root fillets;
- t_f is the thickness of the plate, or, if two or more plates are adequately connected together in accordance with 14.5 or 14.6, the aggregate thickness of such plates. (See Figure 1.)

NOTE 2 For rolled sections or unstiffened flanges, gross cross-sectional properties may be used, provided that the appropriate allowances are incorporated in the calculation of the limiting moment of resistance. (See 9.8.)

9.4.2.5 Effective web

9.4.2.5.1 Beams without longitudinal stiffeners

An effective web thickness t_{we} should be used as follows:

- a) $t_{we} = t_w$, if $\frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} \leq 68$;
- b) $t_{we} = \left(1.425 - 0.00625 \frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} \right) t_w$, if $68 < \frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} < 228$;
- c) $t_{we} = 0$, if $\frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} \geq 228$;

where

- y_c is the depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. For beams built and loaded in stages, y_c should be calculated for the cross-section of the beam appropriate to the stage considered (see 9.9.5);
- σ_{yw} is the nominal yield stress of the web material as defined in 6.2;
- t_w is the thickness of the web.

9.4.2.5.2 Beams with effective longitudinal stiffeners

An effective web thickness t_{we} should be used as follows:

$t_{we} = t_w$ for beams with longitudinal stiffeners designed in accordance with 9.11.5;

where

t_w is as defined in 9.4.2.5.1.

9.4.2.6 Stiffener continuity

The area of longitudinal stiffeners should be included for stress analysis only if they are either continuous longitudinal stiffeners or connected discontinuous longitudinal stiffeners extending over a distance on either side of the section under consideration equal to the depth of the beam.

The cross-section of continuous longitudinal stiffeners should be uninterrupted. Any splices should be in accordance with 14.4. Any cut-outs provided at intersections should be in accordance with 9.3.3.2 with the connection designed in accordance with clause 14. For curtailment of longitudinal stiffeners see 9.10.5 and 9.11.6.

Connected discontinuous longitudinal stiffeners should be welded or otherwise adequately connected to a transverse member or a cross frame using connections designed in accordance with clause 14.

Unconnected discontinuous longitudinal stiffeners should not be used in flanges.

9.4.2.7 Composite construction

For composite construction the area of concrete in a tensile zone should be ignored.

9.5 Evaluation of stresses

9.5.1 General

Longitudinal stresses due to bending and to axial force, if any, should be calculated on the basis of an effective section in accordance with 9.4, 10.5 or 11.3, as appropriate.

The shear flow in a web due to applied shear force should be determined by taking into account the distribution of flexural stresses in the cross-section. However, the shear flow may be taken as the average value throughout the net depth of the web, where the net depth is equal to:

$$(d_w - h_h)$$

where

- d_w is the full depth of a rolled section and the depth of a web plate between flanges in a fabricated section, both as shown in Figure 1;
- h_h is the height of any hole in the section.

9.5.2 Stresses in longitudinally stiffened webs

The longitudinal stress in a longitudinally stiffened web should be calculated by the elastic theory without any assumption of redistribution of stresses. If the stress varies within the length a between transverse stiffeners, the value appropriate for stiffener design should be taken as that on the line of the stiffener at a distance $0.4a$ from the more severely stressed end.

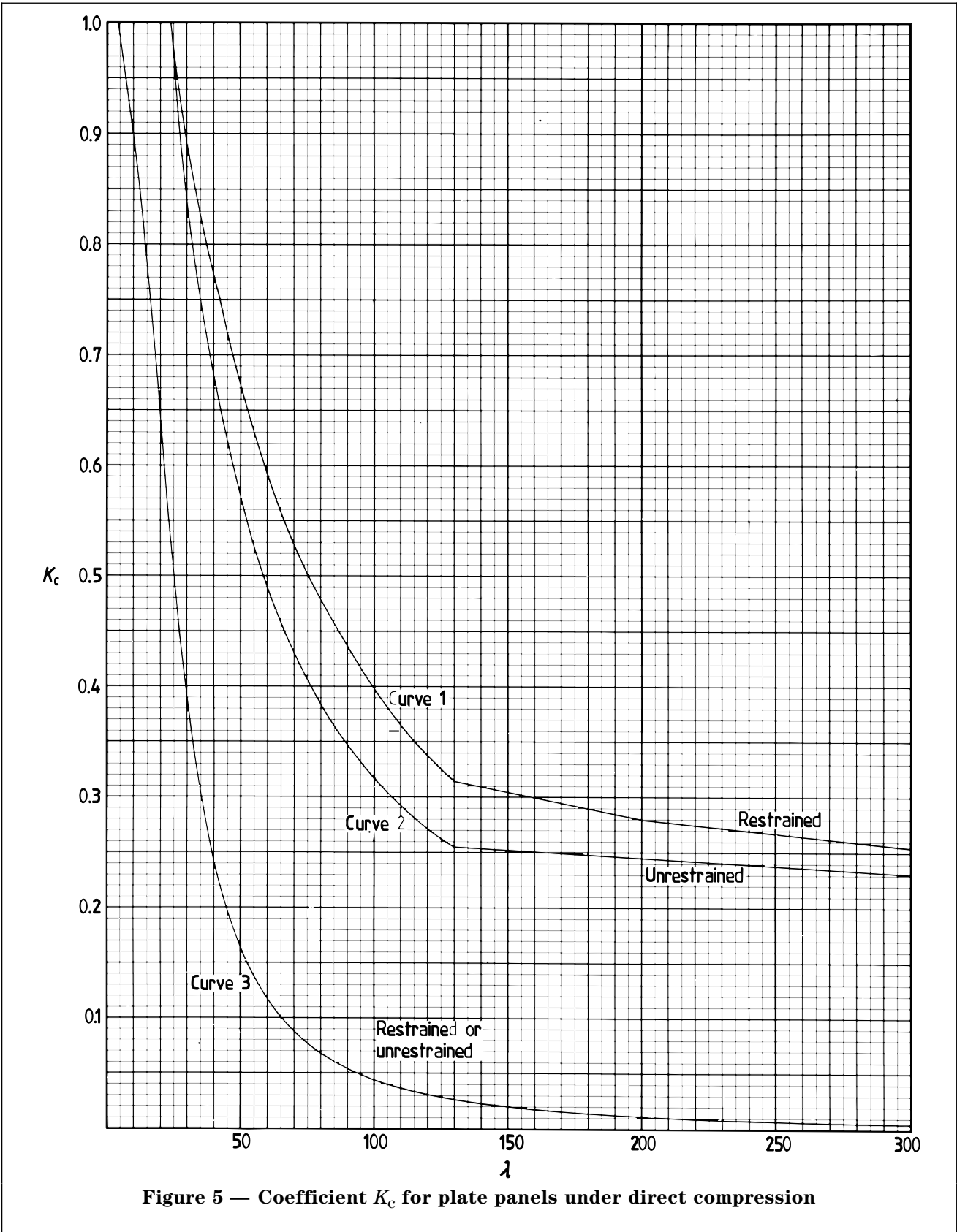


Figure 5 — Coefficient K_c for plate panels under direct compression

NOTE 1 The value of K_c to be used is the higher of the values obtained using either:

a) curve 1 or curve 2 as relevant, with $\lambda = \frac{b}{t} \sqrt{\frac{\sigma_y}{355}}$; or

b) curve 3 with $\lambda = \frac{a}{t} \sqrt{\frac{\sigma_y}{355}}$;

where

a is the panel dimension in the direction of stress under consideration;

b is the panel dimension normal to the direction of stress.

Item a) will always give the higher value for K_c when $a/b \geq 0.5$. For $a/b < 0.5$, item a) or item b) may give a higher value.

NOTE 2 For basis of curves, see G.5.

Figure 5 — Coefficient K_c for plate panels under direct compression (*continued*)

9.5.3 Stresses in longitudinally stiffened compression flanges

Longitudinal stresses in longitudinally stiffened compression flanges should be determined both at the mid-plane of the flange plate (when checking for yield) and at the centroid of the effective section of the stiffener (when checking for buckling).

If the stress varies substantially within the length a between transverse stiffeners, the stresses should be calculated at all sections (when checking for yield) and at a point $0.4a$ from the higher stressed end (when checking for buckling).

9.5.4 Redistribution of web stresses in a longitudinally stiffened beam

The longitudinal stresses in any web panels of a beam with longitudinal stiffeners in the compression flange or the web or both may be assumed to be reduced by not more than 60 % by shedding any appropriate part of the moment and/or axial force to the flanges. The effective longitudinal stiffness of the web panels, from which stress is assumed to be redistributed, should be reduced by a fraction, ρ_w , [equivalent to using a modular ratio for the panels of $(1 - \rho_w)$]. The modified properties of the cross-section should then be used to calculate the revised longitudinal stress distribution either from the load effects calculated by use of the gross section properties in the global analysis, or by use of those calculated using the modified properties for those portions of the beam in which redistribution is assumed. Due account should be taken of any change in bending moment due to longitudinal loads resulting from change in effective centroid position.

The value of ρ_w assumed should be uniform within any one panel of web plate bounded on each side by a longitudinal stiffener, or by a flange, but may vary from panel to panel.

No shedding should be made from any panel containing a hole or opening having a diameter in any direction greater than six times the thickness of the web, or one-fifth of the smaller dimension of the panel, whichever is less; nor from any panel any part of which is within a distance from the nearest edge of such hole or opening equal to the largest diameter of the hole or opening.

9.5.5 Redistribution of tension flange stresses in a longitudinally stiffened beam

When elastic distribution of stresses in a cross-section in a longitudinally stiffened beam for the ultimate limit state causes yielding of the tension flange but not buckling or yielding of the compression flange, redistribution of the stresses may be assumed subject to the following:

- a) redistribution may only be made at the ultimate limit state, and the whole cross-section should satisfy the serviceability limit state, without redistribution;
- b) a linearly varying pattern of longitudinal strains should be assumed over the whole cross-section, such that the stresses produced are in equilibrium with the load effects at the ultimate limit state. Longitudinal stresses at any point should be taken as the lesser of:

- 1) the assumed strain multiplied by the modulus of elasticity E ;

- 2) $\sqrt{\left(\frac{\sigma_y}{\gamma_m \gamma_{f3}}\right)^2 - 3\tau^2}$

where

τ is the elastic shear stress at the point under consideration;

σ_y is the nominal yield stress of the section under consideration, as defined in 6.2;

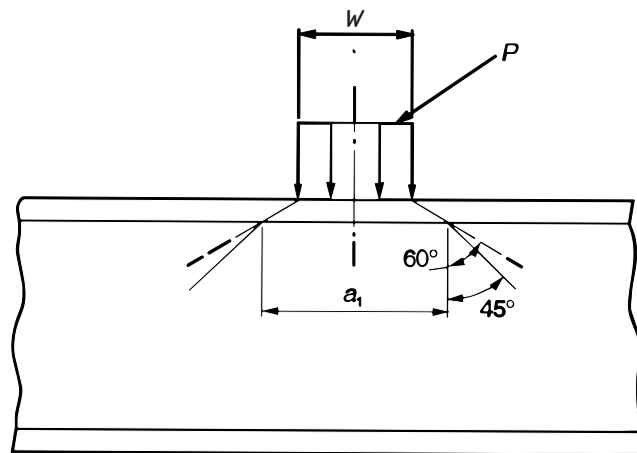
- c) the strain assumed in the compression flange should not exceed $1/E$ times the stress capacity obtained from 9.10, and the strain assumed in the tension flange should not exceed $2\sigma_y/E$;

d) the web stresses obtained in b) should be in accordance with 9.11. For the buckling check of a web plate panel, in accordance with 9.11.4, an equivalent system of stress patterns, shown in Figure 20, may be assumed such that the net axial force along, and the bending moment about, the longitudinal centreline of the panel remain unchanged.

9.5.6 Transverse stresses in webs

The transverse stress in the plane of a web due to the load applied to a flange may be calculated on the assumption that the load is dispersed uniformly:

- at an angle of 60° from the line of application of the load through the thickness of any plate against which the load is bearing; and
- at an angle of 45° from the line of application of the load through the web plate itself (see Figure 6).



NOTE Load P may be compressive or tensile.

Figure 6 — Dispersal of a load through an unstiffened web

9.5.7 Flanges curved in elevation

9.5.7.1 Stresses in flanges

For flanges curved in elevation, a transverse bending stress due to the radial component of the longitudinal force in the flange should be taken as:

$$\frac{3\sigma_f b_{fo}^2}{R_f t_{fo}} \text{ in a flange outstand}$$

$$\frac{3\sigma_f b^2}{4R_f t_f} \text{ in a plate panel of a flange between longitudinal stiffeners and/or webs}$$

where

- σ_f is the longitudinal stress in the flange;
- b is the distance between successive longitudinal stiffeners and/or webs;
- t_f is the thickness of the flange in the panel being considered;
- b_{fo}, t_{fo} are as defined in 9.3.2 (see also Figure 1);
- R_f is the radius of curvature at the flange.

NOTE The stresses are not applicable when the section is unsymmetrical about a vertical axis or curved in plan or in any plane other than the vertical, such a section being outside the scope of this clause.

9.5.7.2 Stresses in webs

9.5.7.2.1 Shear force

The vertical components of the forces in flanges should be taken into account in computing the shear force carried by the web.

9.5.7.2.2 Edge force

The edge of a web attached to a curved portion of a flange should be considered to be subjected to a force, acting in plane with the web, equal to:

$$\frac{\sigma_f B_f t_f}{R_f \cos \beta} \text{ per unit length}$$

where, as shown in Figure 1,

- B_f is the width of an unstiffened flange, in a beam having only one web, or half the distance between successive longitudinal stiffeners or webs, together with any adjacent outstand;
- β is the slope of the web to the vertical;
- σ_f , t_f , R_f are as defined in 9.5.7.1.

9.5.8 Flanges sloping in elevation

In computing the shear force carried by the web at any section of a beam, the vertical components of the longitudinal forces in sloping flanges should be taken into account.

9.6 Effective length for lateral torsional buckling

9.6.1 General

All beams should be restrained at their supports against rotation about their longitudinal axis in accordance with 9.12.5, unless the design is based on a different restraint system in accordance with 9.6.4.1.3.

[A1] In all cases the effective length for lateral torsional buckling ℓ_e should be determined in accordance with 9.6.2 to 9.6.4, as appropriate. However, if the second moment of area of a cross-section about the axis of bending is equal to or smaller than that about an axis perpendicular to it, the cross-section as a whole is stable against overall lateral torsional buckling and its effective length ℓ_e may be taken as zero. For beams with restraints at levels other than those of compression flanges the effective length should be determined in accordance with 9.6.4.1.3 or 9.6.4.2.2, as appropriate. **[A1]**

9.6.2 Beams (other than cantilevers) without intermediate restraints

When there is no intermediate restraint to a compression flange ℓ_e should be taken as the greater of the values calculated in accordance with a) and b) below where relevant.

- a) For single-span or continuous beams:

$$\ell_e = k_1 k_2 k_e L$$

where

- L is the span of the beam (i.e. between lateral restraints at supports). For continuous beams it is the larger of the adjacent spans;
- k_1 may conservatively be taken as:
 - 1.0 if the compression flange is free to rotate in plan at the points of support; or
 - 0.85 if the compression flange is partially restrained against rotation in plan at the points of support, or where it is fully restrained against rotation in plan at one support, and free to rotate in plan at the other; or
 - 0.7 if the compression flange is fully restrained against rotation in plan at the points of support;

NOTE 1 A more accurate value of k_1 allowing for the degree of restraint in plan may be obtained from Figure 7a).

- k_2
 - = 1.0, or
 - = 1.2 if the load is applied to the top flange and both the flange and the load are free to move laterally;

$$k_e = \sqrt{1 - \frac{60Et_{f, \max} \beta \delta_t R_v}{W \left(L/r_y \right)^3 v^4}}$$

where

- $t_{f, \max.}$ is the maximum thickness of the compression flange in the span;
- R_v is the vertical reaction at a support;
- W is the total vertical load on the span;
- δ_t is the largest relative lateral deflection at a support, at either end of the span under consideration, of the centroid of one flange of the beam with respect to the centroid of the other flange which would occur when equal and opposite unit forces act laterally on the torsional restraint at a support only at the same levels as shown in Figure 9d). Where the end restraint to two or more beams is provided by a common lateral member interconnecting their ends, the value of δ_t should be calculated for equal and opposite unit forces applied at the end of each beam applied in directions to produce the greatest value of δ_t at the end of any of the beams;
- β = 1.0 for a simply supported beam, or for an internal support to a continuous beam without restraint in plan to the compression flange at the support,
= 2.0 for an internal support to a continuous beam with restraint in plan at that support;
- v is the value of v calculated in accordance with 9.7.2; A1 Text deleted A1
- r_y is as defined in 9.7.2.

NOTE 2 k_e should be taken as the greater of the values obtained for either support.

NOTE 3 The restraint should be such that the denominator has a positive sign.

b) for continuous beams only:

$$\ell_e = k_1 k_2 \sum L \sqrt{1 + \frac{2(\sum L)^3}{\pi^4 E I_c \{\delta_i + (\delta_e/2)\}}}$$

where

- $\sum L$ is the sum of the lengths of the adjacent spans;
- I_c is the second moment of area of the compression flange about its centroidal axis parallel to the web of the beam at the point of maximum bending moment;
- δ_i is the value of δ_t for an internal support;
- δ_e is the value of δ_t for the end or internal supports at the opposite ends of the adjacent spans.

NOTE 4 The equations for ℓ_e may be used to determine the stiffness of the supports required to achieve a selected effective length.

9.6.3 Cantilevers without intermediate lateral restraints

When a cantilever beam is not provided with lateral restraint between its support and tip, ℓ_e may be taken from Table 8.

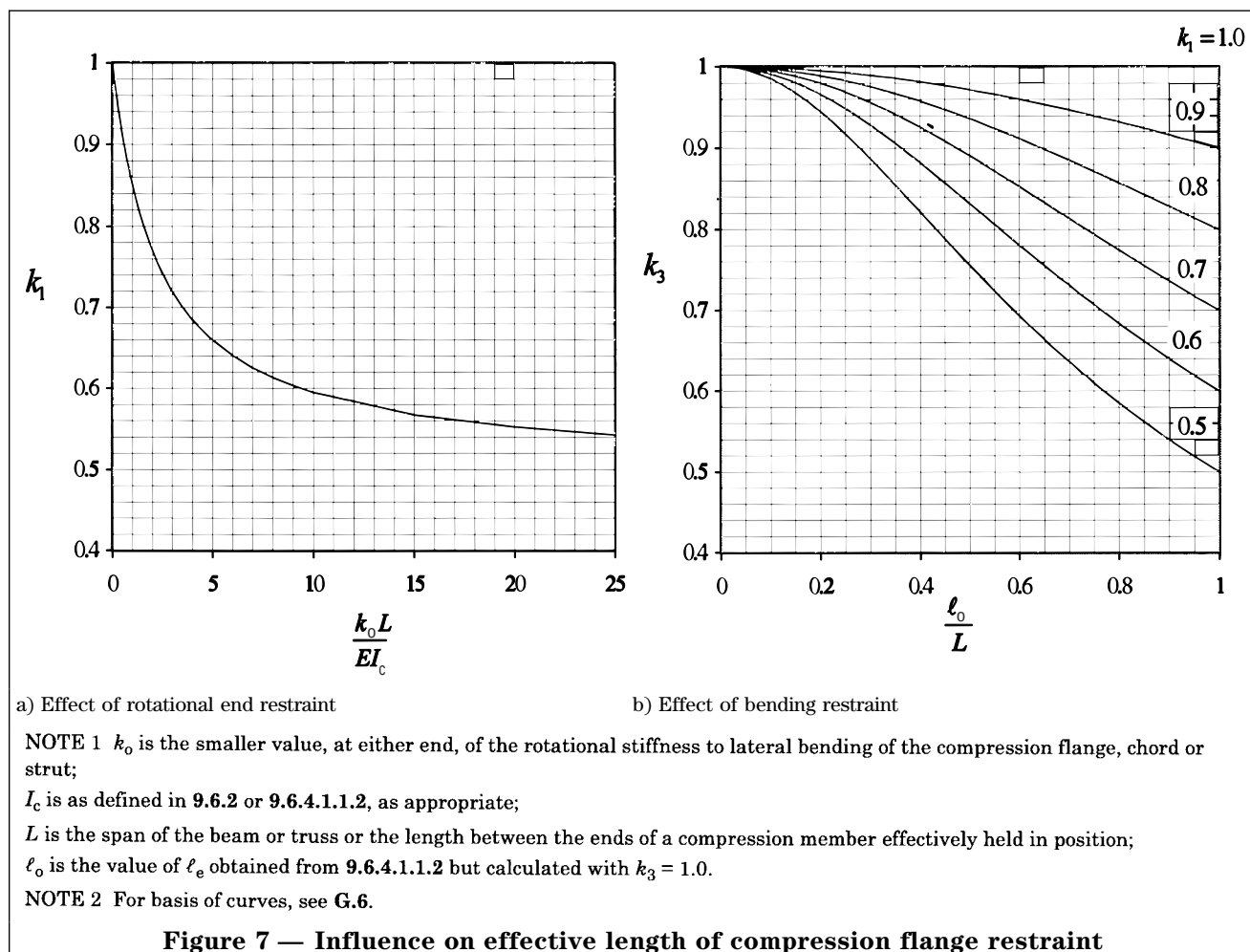


Table 8 — Effective length ℓ_e for a cantilever beam without intermediate lateral restraint

Restraint conditions		Position of load	
At support	At tip	On tension flange where there is no lateral restraint to load or flange	All other positions
1. Built in	a) Free	$1.4L$	$0.8L$
	b) Tension flange held against lateral displacement	$1.4L$	$0.7L$
	c) Both flanges held against lateral displacement	$0.6L$	$0.6L$
2. Continuous, and both flanges held against lateral displacement	a) Free	$2.5L$	$1.0L$
	b) Tension flange held against lateral displacement	$2.5L$	$0.9L$
	c) Both flanges held against lateral displacement	$1.5L$	$0.8L$
3. Continuous, with tension flange only held against lateral displacement	a) Free	$7.5L$	$3.0L$
	b) Tension flange held against lateral displacement	$7.5L$	$2.7L$
	c) Both flanges held against lateral displacement	$4.5L$	$2.4L$

NOTE L is the length of the cantilever.

9.6.4 Beams with intermediate restraints

9.6.4.1 Beams with discrete restraints

9.6.4.1.1 Beams with discrete lateral restraints at the level of a compression flange

9.6.4.1.1.1 Beams with fully effective lateral restraints at the level of a compression flange

When a compression flange is provided with end and intermediate lateral restraints in accordance with 9.12.2 and which are fully effective, ℓ_e should be taken as the distance ℓ_R , between such points of intermediate lateral restraint or between an intermediate restraint and a support which results in the lowest limiting moment of resistance. Where such restraint is provided by interconnecting bracing between two or more beams, consideration should be given to the possibility of lateral instability of the combined cross-section.

[A1] A discrete lateral restraint or a plan bracing system may be taken as fully effective provided that it has a stiffness such that $\delta_R \leq \ell_R^3/(40 EI_c)$ where δ_R , ℓ_R , and I_c are as defined in 9.6.4.1.1.2.

NOTE: Less conservative stiffness requirements can be obtained for a beam with n intermediate restraints by first deriving the value of M_R/M_{ult} from 9.8 for failure by buckling between restraints, with $\ell_e/\ell_w = 1.0$ and λ_{LT} calculated with $\ell_e = \ell_R$. The restraint stiffnesses can also be derived by selecting a value of ℓ_e such that with λ_{LT} calculated using the ℓ_e and with $\ell_e/\ell_w = \ell_e/L$ the same resistance for overall buckling is achieved. The required limit to δ_R is then $\ell_e^4/[EI_c \ell_R (k_2 k_3 k_5)^4]$ in which the terms are as defined in 9.6.4.1.1.2. **[A1]**

9.6.4.1.1.2 Beams with discrete lateral restraints to a compression flange which are not fully effective

When a compression flange of a beam is provided with lateral restraints in accordance with 9.12.2 but not fully effective in accordance with 9.6.4.1.1.1, ℓ_e should be taken as the greater of the values calculated in accordance with a) and b) below:

[A1] a) $\ell_e = k_2 k_3 k_5 \ell_1$ but not less than $k_3 \ell_R$ nor greater than L **[A1]**

b) for simply supported spans $\ell_e = \pi k_2 \sqrt{\frac{EI_c}{L} (\delta_{e1} + \delta_{e2})}$ or

for continuous beams, the value of ℓ_e given by 9.6.2b)

where

k_2 is as defined in 9.6.2;
 k_3 may be taken as 1.0, but where the compression flange is restrained against rotation in plan at the supports, a lower value of k_3 may be obtained from Figure 7b);

$$k_5 = 2.22 + \frac{0.69}{X + 0.5};$$

$$X = \frac{\ell_1^3}{\sqrt{2} EI_c \delta_{e, \max.}};$$

$$\ell_1 = (EI_c \ell_R \delta_R)^{0.25};$$

I_c is the second moment of area of the compression flange about its centroidal axis parallel to the web of the beam at the point of maximum bending moment;

ℓ_R **[A1]** is the distance between points of intermediate restraint or nodes in a plan bracing system or between an intermediate restraint or nodes in a plan bracing system and a support restraint which results in the lowest limiting moment of resistance;

δ_R is the lateral deflection which would occur in a lateral restraint, at the level of the centroid of the flange being considered, when a unit force acts laterally to a lateral restraint only at this point. With a plan bracing system the deflection is to be taken as the deflection of a node relative to adjacent nodes; **[A1]**

- δ_{e1}, δ_{e2} are the relative lateral deflections δ_i at the support at ends 1 and 2 respectively, of the centroid of one flange of the beam with respect to the centroid of the other flange which would occur when equal and opposite unit forces act laterally on the torsional restraint at a support only at the same levels as shown in Figure 9d). Where the end restraint to two or more beams is provided by a common lateral member interconnecting their ends, the value of δ_e should be calculated for equal and opposite unit forces applied at the end of each beam applied in directions to produce the greatest value of δ_e at the end of any of the beams;
- $\delta_{e, \max.}$ is the maximum value of δ_{e1} and δ_{e2} for the span being considered.

9.6.4.1.2 Beams with discrete torsional restraints

[A1] When a beam of uniform cross-section without lateral restraints is provided with a central torsional restraint or a number of equally spaced torsional restraints of the same stiffness in a span in accordance with 9.12.2 b), ℓ_e for that span may be derived from Figure 8, using the restraint parameter $v_{eq}^4 \ell_w^3 / [EI_c \theta_R d_f^2 (1-i)]$ but, for beams with distributed restraints, ℓ_e should be not less than $(1.7 - 0.7v_{eq}) \ell_R$. In using Figure 8, where there are several intermediate torsional restraints, the distributed restraints curve should be used; where there is only a central torsional restraint, the value should be determined by interpolation between appropriate central restraint curves.

where

- ℓ_w is the assumed half wavelength of buckling. The value of ℓ_w should be taken as the span L or a sub multiple of the span, whichever gives the lowest value of limiting moment of resistance M_R in accordance with 9.8;
- d_f is the vertical distance between the centroid of the compression and tension flanges respectively at the position of the torsional restraint;
- I_c is as defined in 9.6.4.1.1.2;
- ℓ_R is as defined in 9.6.4.1.1.2;
- θ_R is the greatest value of the rotation of a restraint about the longitudinal axis of the beam, due to a torque equal to a unit torque multiplied by $1/m$, applied to each restraint. When restraint is provided by uniform diaphragms interconnecting beams, the value of θ_R should be taken as θ_{R1} and $\theta_{R1} + \theta_{R2}$,

where

- θ_{R1} is the rotation due to the flexibility of the diaphragm calculated as the greatest rotation about the longitudinal axis of a beam at a connection between the diaphragm and the beam under unit moments in the plane of the diaphragm multiplied by $1/m$, applied to each connection in the same sense on each beam (see Figure 8A);
- θ_{R2} is the greatest value of rotation of a beam at the middle of a half wavelength of buckling due to the vertical deflections of the beams. A unit torque multiplied by $1/m$ should be applied to each beam at each diaphragm connection, the diaphragms being assumed to be rigid for this calculation, in directions of opposite sense in consecutive waves and in the same sense on each beam;
- m is the number of restraints in the half wavelength of buckling ($= 1$ for a single central restraint with a buckling half wavelength equal to L); **[A1]**

A1) v_{eq} is a parameter that takes account of warping properties of the section and is calculated as follows:

- for sections symmetrical about both their major and minor axis, $v_{eq} = v$, where the value of v is determined in accordance with **9.7.2**,
- for sections symmetrical about their minor axis only,

$$v_{eq} = \left[\frac{2 i \chi}{\left[\sqrt{4 + \tau \chi} + \psi_i \sqrt{\chi} \right]^2} \right]^{0.25}$$

where

$$\tau = 1 \text{ when } I_c \leq I_t$$

$$\tau = 4i(1-i) + \psi_i^2 \text{ when } I_c > I_t;$$

i , ψ_i , I_c and I_t are as defined in **9.7.2**;

$$\chi = \frac{\pi^2 d_t^2 E I_y}{G J \ell_w^2};$$

I_y is the second moment of area of the beam about its minor axis;

J is the St Venant Torsional Constant.

NOTE 1 The effective length relationships in Figure 8 do not apply when considering buckling with a half-wavelength equal to ℓ_R (within which there are no restraints), for which $\ell_w = \ell_e = \ell_R$.

NOTE 2 The resistance of a beam with $\ell_e < \ell_R$ and $\ell_w > \ell_R$ may be less than for buckling between restraints with $\ell_w = \ell_e = \ell_R$.

NOTE 3 For beams with a single central restraint, ℓ_w should be taken as equal to L , unless the restraint parameter derived from that value is such that it is to the right of the diagonal arrow on the appropriate curve for the value of v_{eq} , in which case either:

- ℓ_e and ℓ_w should be taken as $L/2$

or

- ℓ_w should be taken as L , and ℓ_e should be derived from Figure 8, whichever gives the lower moment of resistance.

NOTE 4 Where, for distributed restraints, the value of $v_{eq}^4 \ell_w^3 / [E I_c \theta_R d_t^2 (1-i)]$ exceeds the maximum value shown in Figure 8, the equation given in **G.7** may be used to derive the appropriate value of $\ell_e \ell_w$.

NOTE 5 When only pairs of beams are interconnected by diaphragms, θ_{R2} may be taken as $(m+1) \ell_w^3 / 24 n E I_2 B^2$ where I_2 is the second moment of area of each beam about its major axis, B is the spacing of the beams and m is the number of restraints within the half wavelength.

NOTE 6 Consideration should be given to the alternative strengths of the beams when buckling in modes in which adjacent beams rotate in opposite senses (see Figure 8A). These should be treated as beams restrained by the cross-members in accordance with **9.6.4.1.3** or **9.6.4.2**. **A1)**