

Steel, concrete and composite bridges —

Part 3: Code of practice for design of steel bridges

UDC 624.21.01:624.21.014.2

Cooperating organizations

The Civil Engineering and Building Structures Standards Committee, under whose direction this British Standard was prepared, consists of representatives from the following:

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- Association of County Councils
- British Constructional Steelwork Association
- British Railways Board
- Constructional Steel Research and Development Organization
- Greater London Council
- Federation of Civil Engineering Contractors
- Institution of Highway Engineers
- Ministry of Defence
- Sand and Gravel Association Limited
- Welding Institute

This British Standard, having been prepared under the direction of the Civil Engineering and Building Structures Standards Committee, was published under the authority of the Board of BSI and comes into effect on 30 April 1982

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The following BSI references relate to the work on this standard:
Committee reference CSB/30
Draft for comment 79/13134 DC

ISBN 0 580 12464 9

Amendments issued since publication

Amd. No.	Date of issue	Comments
4051	August 1982	
6488	April 1991	Indicated by a sideline in the margin

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Foreword

BS 5400 is a document combining codes of practice to cover 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;*
- *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 bearings¹⁾;*
- *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.

It is to be expected that as design experience of different bridge types is accumulated, further modifications will be required.

The relationship between Part 3 and Part 5. The design of composite bridges requires the combined use of Part 5 and Part 3 of BS 5400.

Part 5 was published in 1979, the major decisions on scope and approach having been taken some years previously. It is natural therefore that some differences will exist between Part 3 and Part 5.

Part 3 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 as prescribed in this Part. For beams Part 3 may be used without any modification in conjunction with those provisions of Part 5 that are applicable to the properties of the composite slab and its connection to the steel section.

Part 5 also contains optional provisions for increased redistribution of longitudinal moments in compact members or for plastic analysis of continuous beams for the ultimate limit state, which could prove economical in some instances. These procedures require special consideration of increased transverse deformations of the slab, which is not covered in Part 5, and of stability of the bottom flange, which is not covered in Part 3: they should not be used unless proper account is taken of these considerations.

It will be noted that more serviceability checks are required 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 on the shear connectors.

¹⁾ In course of preparation.

The method given in 4.1.3 a) of Part 5 should not be used, when the relationship between loading and load effects is non linear, and the values of γ_m for structural steel given in Table 1 of Part 5 should not be used and reference made to Table 2 of Part 3.

It is intended to revise Parts 3 and 5 to coordinate them fully after there has been sufficient experience of their application.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

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

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xvi, pages 1 to 170, an inside back cover and a back cover.

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

1 Scope

This Part of this British Standard 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 Part 6. Recommendations for design against fatigue are contained in Part 10. Recommendations for design of concrete components and shear connectors for their interaction with steelwork are contained in Part 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. Parapets, safety fences and other ancillary items are not within the scope of this Part.

2 References

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

3 Definitions and symbols

3.1 Definitions

For the purposes of this Part of this British Standard the definitions given in Part 1 apply. For the sake of clarity the factors which together comprise the partial safety factor for loads are restated as follows

design loads

design loads are the loads obtained by multiplying the nominal loads by γ_{fL} , the partial safety factor for loads. γ_{fL} is a function of two individual factors, γ_{f1} and γ_{f2} , which take account of the following:

- γ_{f1} , the possible unfavourable deviations in load from those considered in deriving their nominal values
- γ_{f2} , the reduced probability that, with combinations of load, 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 Part 2. The factor γ_{f3} takes account of inaccurate assessment of 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_{f3} are given in 4.3.3

3.2 Symbols

3.2.1 General. The symbols used in this Part of this British Standard are as given in 3.2.2 and 3.2.3. Main symbols are given in 3.2.2 and subscripts are given in 3.2.3. Symbols are further clarified, as appropriate, in the text. Some additional symbols are used in the appendices.

3.2.2 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	Centres of bearings
D	Overall depth; overall diameter
d	Depth of element; diameter of element
E	Modulus of elasticity
e	Eccentricity; offset; exponential function = 2.7183
F	Internal force
F_o	Prestress force
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
k	Factor, as defined in text
L	Span; overall length
$=$	Length of element
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	Spacing
T	Torque or torsional moment
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/flange area
α_{LT}	Coefficient for torsional restraint
β	Slope of web to vertical
γ_{FL}, γ_{FB}	Partial load factors
$\gamma_m, \gamma_{m1}, \gamma_{m2}$	Partial material factors
Δ	Imperfection to be assumed
δ	Flexibility (deflection per unit force)
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	3.1416
ρ	Proportion, as defined in text

Σ	Sum
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

3.2.3 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. The objectives of design shall be those stated in Part 1.

4.2 Limit states

4.2.1 Ultimate limit state. All structural steelwork should be checked for compliance with the recommendations of this Part 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 to 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 spread of plasticity.

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

A list of clauses requiring a separate check at the serviceability limit state is given in Table 1.

Table 1 — Clauses requiring serviceability check

Clause	Items and consideration requiring serviceability check
9.2.3.1	Flange panel when maximum longitudinal stress is more than 1.67 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 Part 10.

4.3 Partial safety factors to be used

4.3.1 General. For a satisfactory design of the structure, the provisions given in Part 1 should be met using the format set out in 4.3.2 of this Part.

4.3.2 Safety factor format. Stresses shall be calculated from the effects of $\gamma_{fL}Q_k$.

The safety factor format to be used in applying this Part is:

$$(\text{the effects of } \gamma_{fL}Q_k) \leq \frac{1}{\gamma_{f3}\gamma_{m1}\gamma_{m2}} \left[\begin{array}{c} \text{(function } \sigma_y \text{, and other)} \\ \text{geometric variables} \end{array} \right]$$

where

$\gamma_{fL}\gamma_{f3}$ and Q_k are as described in Part 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:
- γ_{fl} the values of γ_{fl} are given in Part 2 for each type and combination of loading
 - γ_{f3} the factor γ_{f3} in this Part shall be taken as 1.1 for the ultimate limit state and 1.0 for the serviceability limit state
 - γ_{m1}, γ_{m2} for the sake of simplicity the expressions for strength in this Part 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 are given in Table 2.

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

a) Ultimate limit state

Where explicitly expressed in a strength requirement in this Part, γ_m shall be taken as 1.05, except in the following clauses for which the appropriate value of γ_m is tabulated

Structural component and behaviour	Clauses	γ_m
Bending resistance of beams related to the limiting compressive stress except for a composite beam section where the steel and concrete composite flange is in compression for which $\gamma_m = 1.05$	9.9.1.2, 9.9.1.3 a), 9.9.3.1, 9.9.5.3 a), 9.10.1.1 a)	1.20
Buckling resistance of stiffeners	9.10.2.3 a) and b), 9.11.5.2, 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	1.10

b) Serviceability limit state

Where explicitly expressed in a strength requirement in this Part, γ_m shall be taken as 1.00, except in the following clause for which the appropriate value of γ_m is tabulated.

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 above clauses should incorporate the appropriate γ_m factor tabulated.		

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 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 an 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 of steel or concrete, should be waterproofed and so designed as to protect supporting steelwork from corrosive attack by 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 the temperature of the air inside the enclosed space and 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 not be 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 required by 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) at industrial or marine sites, 6 mm;
- b) at other inland sites, 4 mm;
- c) where free drainage cannot be ensured, 1 mm, in addition to the excess under a) and b).

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 not be 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.

4.6 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 4.4.1 of Part 2 of BS 5400:1978, for the serviceability limit state.

5 Limitations on construction and workmanship

5.1 Workmanship. The design rules given in this Part are appropriate only to bridges fabricated and erected in accordance with Part 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.

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 Part 5.

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

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 1/800th 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 Performance. The mechanical properties of materials required by the Engineer are to be specified in accordance with Part 6.

6.2 Nominal yield stress. Irrespective of the thickness of a component, the nominal yield stress σ_y , for steel supplied to a standard grade complying with the requirements of BS 4360 and tested in accordance with that standard, should be taken as equal to the specified minimum yield strength appropriate to the thickness of 16 mm (12 mm for grade WR50A). When steel to specifications other than BS 4360 is used the nominal yield stress should be taken as:

$$\left(1 - \frac{\rho_t}{100}\right) (\sigma_{ym} - 1.65 \text{ standard deviations from } \sigma_{ym})$$

where

ρ_t is the percentage tolerance below the specified thickness permitted by the relevant British Standard for material for the relevant thickness

σ_{ym} is the mean yield stress of material of the relevant thickness.

6.3 Ultimate tensile stress. The ultimate tensile stress of steel plates and sections of steel not complying with the requirements of BS 4360 should be not less than $1.4\sigma_y$ when $\sigma_y < 390 \text{ N/mm}^2$, nor less than $1.2\sigma_y$ when $\sigma_y \geq 390 \text{ N/mm}^2$,

where

σ_y is the nominal yield stress of the material.

6.4 Ductility. Steel used in a bridge designed in accordance with this Part should have a ductility not less than that corresponding to an elongation of 15 %, based on the standard gauge length of 200 mm. Steels complying with the requirements of BS 4360 satisfy this recommendation.

NOTE If ductility is tested with other gauge lengths, the observed value should be converted to an equivalent ductility for a 200 mm gauge length in accordance with BS 3894.

Where use is made of plastic methods of design which permit redistribution of bending effects, the ductility of the steel used should not be less than that specified in BS 4360.

6.5 Notch toughness

6.5.1 General. Any part which is to be subjected to applied tensile stress either during erection or in service should be in accordance with the simple provisions of 6.5.4, or with the fuller energy absorption provisions of 6.5.5.

In applying either 6.5.4 or 6.5.5, the design minimum temperature, U (in °C), determined in accordance with 6.5.2, should be used, and the part should be classified in accordance with 6.5.3.

Special consideration is required where the geometrical shape is such as to induce large concentrations of stress (see 6.5.6).

6.5.2 Design minimum temperature. The design minimum temperature, U (in °C), to be used when applying either 6.5.4 or 6.5.5 should be as follows:

- a) in a part of which the primary function 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 Part 2 (in °C).

6.5.3 Classification. Each part subjected to applied tensile stress should be classified as type 1 or type 2 as follows.

- a) *Type 1.* Any part which is subjected to applied principal tensile stress at the ultimate limit state (ignoring geometric stress concentrations) greater than 100 N/mm^2 , and which, in addition, has either
 - 1) any welded connection or attachment; or
 - 2) welded repair of surface defects and has not been subsequently inspected by crack detection of at least a 10 % random sample; or
 - 3) punched holes which have not been subsequently reamed.
- b) *Type 2.* All parts subjected to applied tensile stress and which are not of type 1.

6.5.4 Simple provisions. To simplify selection, limiting thicknesses for certain steels complying with the requirements of BS 4360, to give the required energy absorption, are given in Table 3. For the use of greater thicknesses of these steels, the provisions of **6.5.5** should be met.

Unless dealt with in accordance with **6.5.5**:

- a) no part of type 1 shall be thicker than the appropriate limit given in Table 3;
- b) no part of type 2 shall be thicker than double the appropriate limit given in Table 3 nor exceed the maximum thickness specified in BS 4360.

6.5.5 Energy absorption. Unless the simple provisions of **6.5.4** are adopted, the material used to resist applied tensile stress should be such that:

$$\text{for type 1, } C_v \geq \frac{\sigma_y}{355} \left(\frac{t}{2} \right) \text{ (in joules);}$$

$$\text{for type 2, } C_v \geq \frac{\sigma_y}{355} \left(\frac{t}{4} \right) \text{ (in joules);}$$

where

C_v is the energy absorption in Charpy V-notch tests defined in BS 4360 and carried out at the design minimum temperature U (see **6.5.2**).

σ_y is the nominal yield stress

t is the thickness of the part (in mm).

6.5.6 Stress concentrations. The provisions of **6.5.4** and **6.5.5** do not apply where severe geometric stress concentrations occur. In such cases the material should be such that:

$$C_v \geq \frac{\sigma_y}{355} [0.3t(1 + 0.67k)]$$

where

C_v , σ_y and t are as defined in **6.5.5**

k is the stress concentration factor.

NOTE The stress concentration factor is the ratio of the peak principal tensile stress to the mean principal tensile stress at any section. Typical stress concentration factors for openings and notches are given in Part 10. For example, at the junctions between plated diaphragms and corners of box girders, k may be taken as 3.5 for the adjacent flanges and webs. The influence of stress concentrations around small holes and those for bolts or rivets may be considered to have been taken into account in **6.5.4** and **6.5.5**.

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. For stress analysis the modular ratio appropriate to the stage of construction and the type of loading should be adopted.

Table 3 — Limiting thickness of certain steels, complying with the requirements of BS 4360, for parts in tension

a) Plates

Grade in BS 4360	$U = 0\text{ }^{\circ}\text{C}$	$U = -10\text{ }^{\circ}\text{C}$	$U = -20\text{ }^{\circ}\text{C}$	$U = -30\text{ }^{\circ}\text{C}$	$U = -40\text{ }^{\circ}\text{C}$	$U = -50\text{ }^{\circ}\text{C}$
	Limiting thickness					
	mm	mm	mm	mm	mm	mm
40A, 43A, 43A1	0	0	0	0	0	0
40B, 43B	40	20	0	0	0	0
40C, 43C	50	50	30	15	0	0
40D, 43D	75	75	75	50	30	15
40E, 43E	75	75	75	75	75	75
50A, 50B	0	0	0	0	0	0
50C	75	70	45	25	15	0
50D	75	75	75	55	35	20
50D1	40	40	40	40	35	20
50E	63	63	63	63	63	55
50F	40	40	40	40	40	40
55C	19	19	15	0	0	0
55E	63	63	63	63	60	45
55F	40	40	40	40	40	40
WR50A	12	12	12	0	0	0
WR50A1, WR50B1	0	0	0	0	0	0
WR50B	50	35	20	0	0	0
WR50C, WR50C1	50	50	45	25	15	0

b) Sections (other than hollow sections)

Grade in BS 4360	$U = 0\text{ }^{\circ}\text{C}$	$U = -10\text{ }^{\circ}\text{C}$	$U = -20\text{ }^{\circ}\text{C}$	$U = -30\text{ }^{\circ}\text{C}$	$U = -40\text{ }^{\circ}\text{C}$	$U = -50\text{ }^{\circ}\text{C}$
	Limiting thickness					
	mm	mm	mm	mm	mm	mm
40A, 43A, 43A1	0	0	0	0	0	0
40B, 43B	40	20	0	0	0	0
40C, 43C	50	50	30	15	0	0
40D, 43D	50	50	50	40	20	0
40E, 43E	50	50	50	50	50	30
50A, 50B	0	0	0	0	0	0
50C	40	35	20	0	0	0
50D	40	40	35	20	10	0
55C	19	19	15	0	0	0
55E	63	63	63	63	55	45
WR50A	12	12	12	0	0	0
WR50A1, WR50B1	0	0	0	0	0	0
WR50B	50	35	20	0	0	0
WR50C, WR50C1	50	50	45	25	15	0

Table 3 — Limiting thickness of certain steels, complying with the requirements of BS 4360, for parts in tension

c) Hollow sections

Grade in BS 4360	$U = 0\text{ }^{\circ}\text{C}$	$U = -10\text{ }^{\circ}\text{C}$	$U = -20\text{ }^{\circ}\text{C}$	$U = -30\text{ }^{\circ}\text{C}$	$U = -40\text{ }^{\circ}\text{C}$	$U = -50\text{ }^{\circ}\text{C}$
	Limiting thickness					
	mm	mm	mm	mm	mm	mm
43C	16	16	16	15	0	0
43D	16	16	16	16	16	0
43E	16	16	16	16	16	16
50C	16	16	16	0	0	0
50D	16	16	16	16	10	0
50E	16	16	16	16	16	16
55C	16	16	15	0	0	0
55E	16	16	16	16	16	16
WR50C	16	16	16	0	0	0
WR50D	16	16	16	16	15	0

NOTE 1 Interpolation for limiting thicknesses for intermediate temperatures is permitted between data in adjacent columns in Table 3.

NOTE 2 Some of the thicknesses in Table 3 are the limits set by the maximum thickness specified in BS 4360. The option in BS 4360 for specifying notch ductility requirements for grades 40B and 43B (plates and sections) and grade 43C (hollow sections) should be adopted.

NOTE 3 All thicknesses given are for type 1 parts. Thicknesses may be doubled for type 2 parts but should not exceed the maximum thickness specified in BS 4360.

NOTE 4 Alternative thicknesses may be derived in accordance with 6.5.5.

7 Global analysis for load effects

7.1 General. The global analysis of the structure should be in accordance with Part 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 section assuming the specified sizes. For beams or trusses on flexible supports or cable stayed, account should, however, be taken of the influence of shear lag on their stiffnesses. The effect of shear lag should also be taken into account in analysis of conditions during erection of continuous girders of box construction or with integral decks.

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 shall be ignored (see 9.4.2.7).

Subject to the provisions of 9.5.4, plate panels in webs may be assumed to shed a proportion of their longitudinal stresses 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, an equivalent flange may be assumed to have 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 Table 4 to Table 7,

ψ is the appropriate effective breadth ratio taken from Table 4, Table 5, Table 6 or Table 7 for uniformly distributed loads and which should be used for standard highway or railway loading as specified in Part 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

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

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

Values of ψ for intermediate values of b/L and α 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 Table 4, Table 5, Table 6 and Table 7 and in Appendix 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 Appendix A.

Table 4 — Effective breadth ratio ψ for simply supported beams

$\frac{b}{L}$	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
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

$\frac{b}{L}$	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
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

$\frac{b}{L}$	Fixed end		Quarter-span near fixed end		Propped end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
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

$\frac{b}{L}$	Fixed end		Quarter-span near fixed end		Free end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
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 required by **9.2.1.2**, be calculated by linear elastic analysis. The simplified procedure given in Appendix B may be used for highway bridges designed for loading in accordance with Part 2.

8.4 Shear stresses. The design values of 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 may be assumed to have made allowance for the following tolerances where these apply:

- a) bearing misalignment in plan, errors in level of single bearing or in the mean level of more than one bearing at any support, and bearing inclination within the tolerances given in Part 9²⁾;
- b) imperfect flatness and straightness of compression members and of stiffened and unstiffened plate panels within the tolerances given in Part 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 flat bearing surface beneath 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 may be deemed to allow for residual stresses due to roiling, 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-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.

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 state

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:

- a) flexure, shear and torsion due to any loads transverse to the longitudinal axis of the member;
- b) the effects of axial load;
- c) creep, shrinkage and differential temperature (see Part 5 for composite structures);
- d) settlement of supports.

²⁾ In course of preparation.

9.2.1.3 Effects that may be neglected. The effects of shear lag and restraint of torsional and distortional warping may be neglected for the ultimate limit state.

Items c) and d) of **9.2.1.2** may be neglected at the ultimate limit state provided that:

- a) the section is compact throughout the span being considered in accordance with the provisions of **9.3.7**; and
- b) the member is not prone to lateral instability; this may be deemed to be satisfied when the slenderness parameter, λ_{LT} is less than $45\sqrt{355/\sigma_y}$ (see **9.7**).

9.2.2 Fatigue. The fatigue endurance should be in accordance with the recommendations of Part 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.

- a) When the maximum longitudinal stress is more than 1.67 times the average longitudinal stress in the particular flange segment between adjacent webs, or in the overhang from an outside web as appropriate. For this purpose, a “flange segment” is the whole flange width between adjacent webs, or the whole overhang, whether or not stiffened by longitudinal stiffeners.

NOTE For plate girder construction this criterion may be deemed to be satisfied if the shear lag effective breadth ratio ψ , determined in accordance with **8.2**, is greater than 0.6.

- 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 required in **9.9.8** for unsymmetric beams.

9.2.3.2 Effects to be considered. Stresses at the serviceability limit state should be obtained for the relevant combinations given in **9.2.1.2** a) to d) together with:

- a) the effects of shear lag;
- b) the effects of restraint of torsional and distortional warping in a girder.

9.2.4 Composite beams. In the design of composite beams, the concrete, reinforcement and shear connectors should satisfy the limit state requirements of Parts 4 and 5.

9.3 Shape limitations

9.3.1 General. Figure 1 sets out the geometric notation used throughout this section.

In applying **9.3.2**, **9.3.4** and **9.3.6** to sections that are not fully stressed by the applied loading, the nominal yield stress values σ_{ys} or σ_y may be taken as the lesser of: the nominal yield stress of the material or 1.5 times the maximum stress at the appropriate section calculated for the factored loading at the ultimate limit state condition,

where

σ_{ys} relates to the stiffener

σ_y relates to the flange, the web or the circular hollow section, as appropriate.

9.3.2 Flanges

9.3.2.1 Outstands in compression. Unless a free edge of a plate or other outstand is stiffened, the ratio b_{fo}/t_{fo} should not exceed $12\sqrt{355/\sigma_y}$ or 16 whichever is the lesser.

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

t_{fo} is the mean thickness of the outstand

σ_y is as defined in **9.3.1**.

9.3.2.2 Outstands in tension. The ratio b_{f0}/t_{f0} should not exceed 16, where b_{f0} and t_{f0} are as defined in **9.3.2.1**.

NOTE Where a flange 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 the above provision should be satisfied for all the flange plates. For the flange plate connected to the web, b_{f0} should be taken as given above, but for all the other flange plates b_{f0} 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 the depth of the web, nor for longitudinally stiffened webs, one-third the depth of the panel containing the opening;
- the longitudinal distance between 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 Appendix 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:

- 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} and σ_y are as defined in **9.3.1**.

9.3.4.1.3 Bulb flat stiffeners. Bulb flat stiffeners should comply with the requirements of Part 5 of BS 4848 and should be proportioned such that:

- either $\frac{\bar{s}}{h_s}$ does not exceed $\frac{b}{t} \sqrt{\frac{\sigma_y}{355}}$
- or $\frac{(b + k_s h_s)}{t} \sqrt{\frac{\sigma_y}{355}}$ does not exceed 30

where

$\bar{s}$ 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 50 steel in accordance with BS 4360, or

$= 0.15$ for grade 43 steel in accordance with BS 4360

b and t are as defined in **9.3.4.1.2**

σ_{ys} and σ_y are as defined in **9.3.1**.

9.3.4.1.4 Angle stiffeners. Angle stiffeners should comply with the requirements of Part 4 of BS 4848 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) 7; or
 - 2) a higher value obtained from Figure 3 when $\frac{t_s}{b_s} \sqrt{\frac{\sigma_{ys}}{355}}$ 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 and t_s are the depth, width and thickness of the angle, respectively, as shown in Figure 1

b and t are as defined in 9.3.4.1.2

σ_{ys} is as defined in 9.3.4.1.3

σ_{ys} and 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:
 - 1) either Figure 4 a) when $\frac{t_s}{b_s} \sqrt{\frac{\sigma_{ys}}{355}}$ is less than 25;
 - 2) or Figure 4 b) when both $\frac{b}{t} \sqrt{\frac{\sigma_{ys}}{355}}$ is less than 32, and $\frac{d_s}{b_s}$ does not exceed 4

where

b_s and t_s are as shown in Figure 1

b and t are as defined in 9.3.4.1.2

σ_{ys} 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} and 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:

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

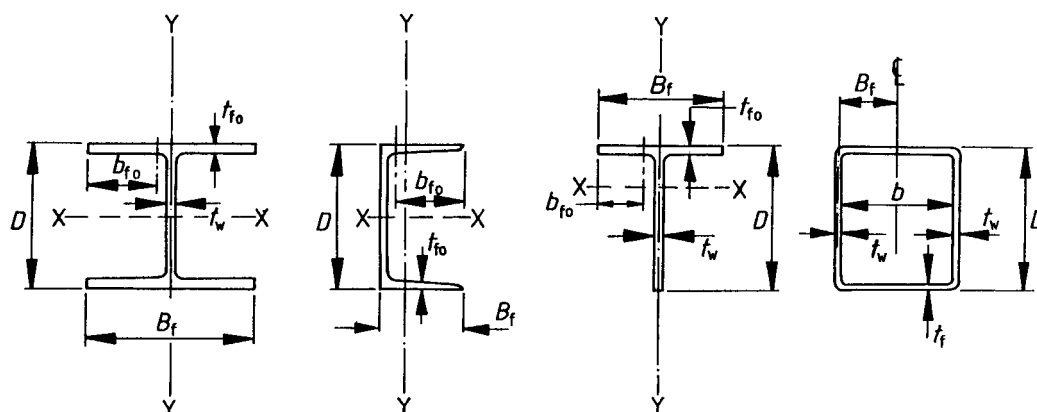
where

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

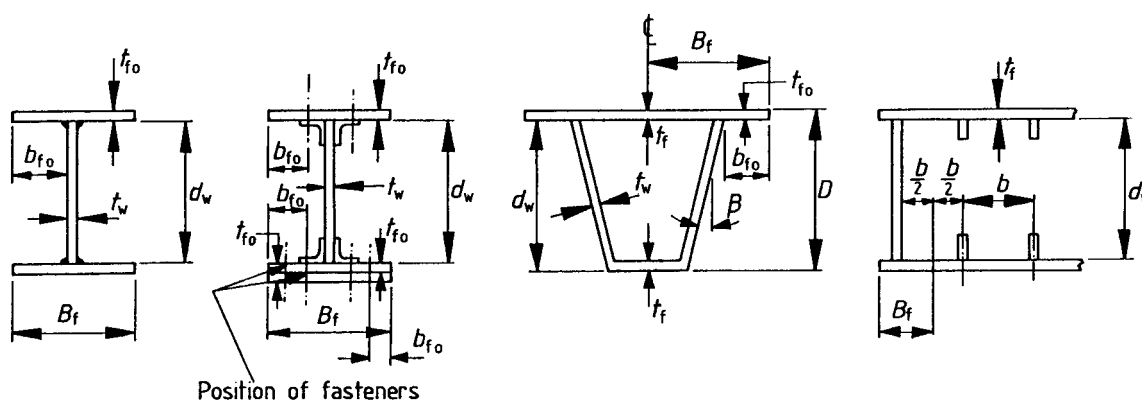
t_s is the thickness of the stiffener

σ_{ys} is as defined in 9.3.1.



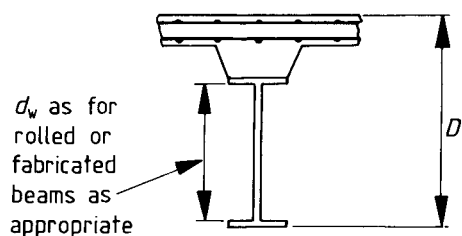
NOTE For tapered flanges, t_{fo} should be taken as the mean thickness of the flange, as defined in BS 4.

a) Rolled beam sections, $d_w = D$



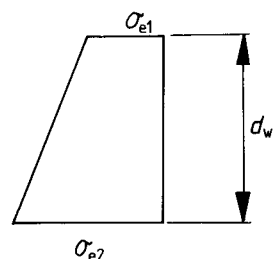
b) Fabricated beam sections, d_w is as shown

Figure 1 — Geometric notation for beams (continued)

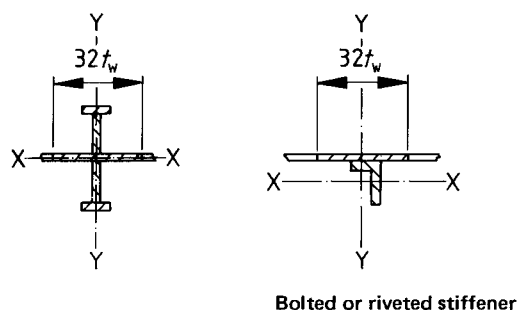
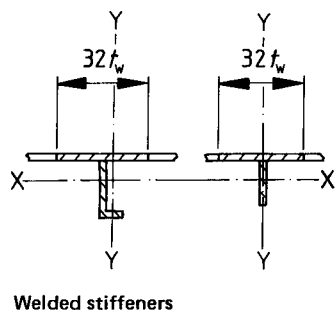
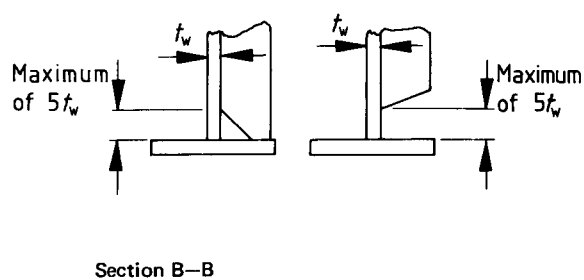
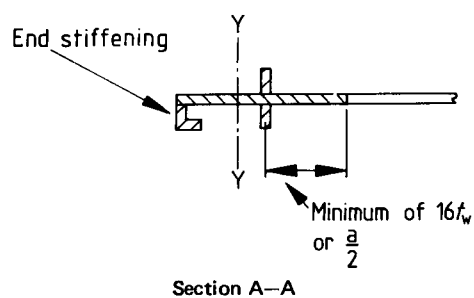
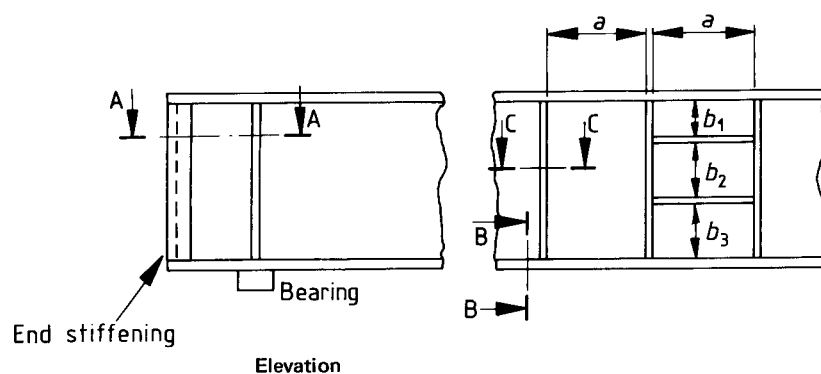


NOTE D is shown for concrete in compression. For concrete in tension D is taken to level of tension reinforcement.

d) Stress distribution in web

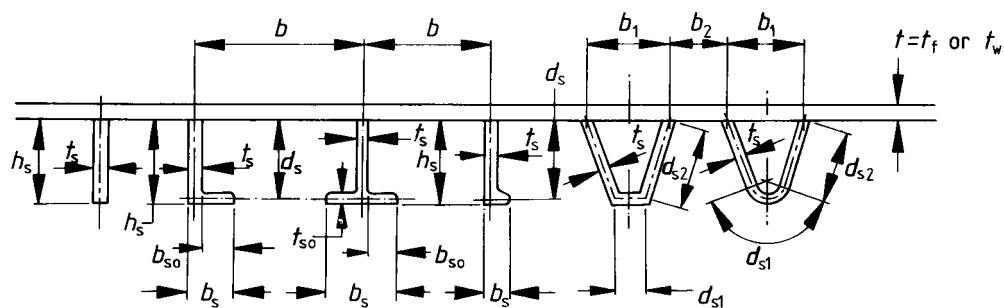


c) Composite beam section



e) Stiffened beam

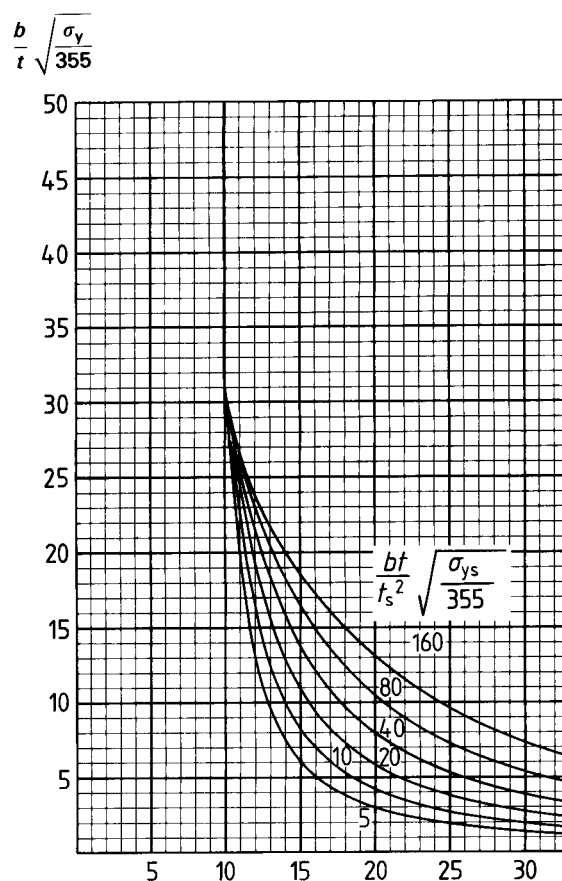
Figure 1 — Geometric notation for beams (continued)



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

f) Flange and web stiffeners

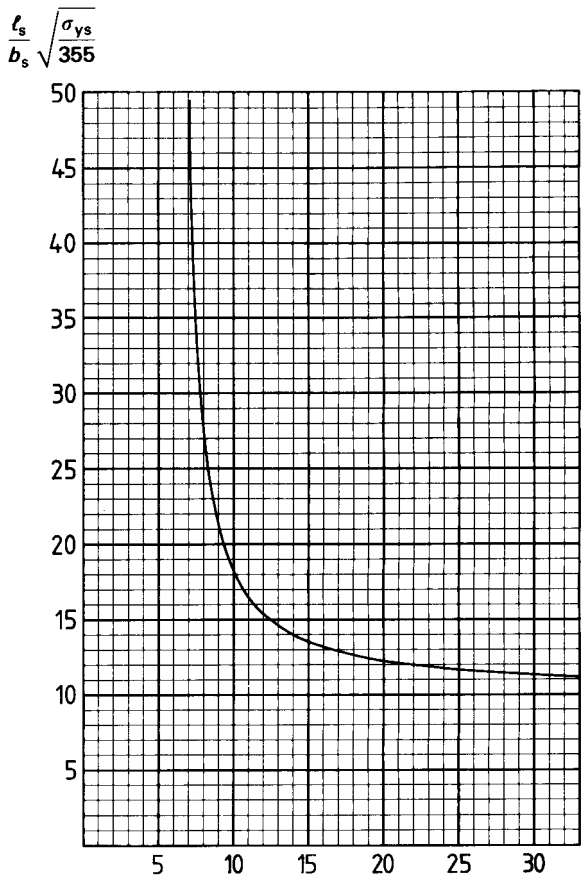
Figure 1 — Geometric notation for beams (concluded)



NOTE For basis of curves, see G.2.

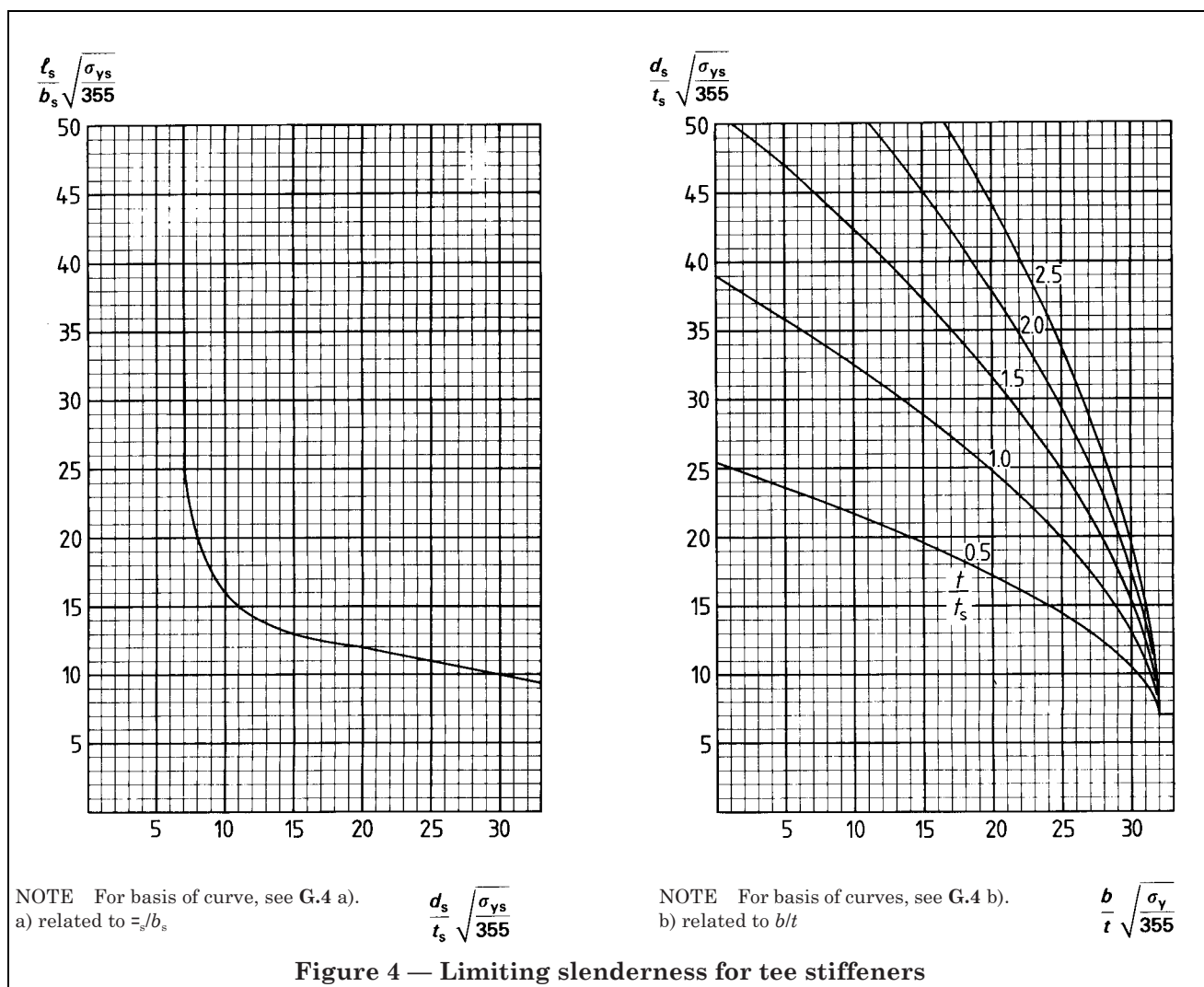
$$\frac{h_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$$

Figure 2 — Limiting slenderness for flat stiffeners



NOTE For basis of curve, see G.3. $\frac{h_s}{t_s} \sqrt{\frac{\sigma_{ys}}{355}}$

Figure 3 — Limiting slenderness for angle stiffeners



9.3.5 Flanges curved in elevation. Flanges curved in elevation should be such that:

$$a) \frac{b_{fo}}{t_{fo}} \leq \frac{R_f}{6b_{fo}}$$

$$b) \frac{b}{t_f} \leq \frac{R_f}{2b}$$

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} and 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:

$$100 \sqrt{\frac{355}{\sigma_y}}$$

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 requirements should be applied to the cross section of the beam appropriate to the stage considered (see 9.9.5).

9.3.7.2 Webs

9.3.7.2.1 Webs partly in compression and partly in tension. The depth between the elastic neutral axis of the beam and the compressive edge of the web should not exceed:

$$28t_w \sqrt{\frac{355}{\sigma_{yw}}}$$

where

t_w is the thickness of the web plate

σ_{yw} is the nominal yield stress of the web material.

9.3.7.2.2 Webs wholly in compression. The depth of the web should not exceed:

$$\left\{ 24 + 4 \left[1 - \left(\frac{\sigma_{e1}}{\sigma_{e2}} \right)^3 \right] \right\} t_w \sqrt{\frac{355}{\sigma_{yw}}}$$

where

t_w and σ_{yw} are as defined in 9.3.7.2.1

σ_{e1} , σ_{e2} are the longitudinal stresses at the two edges

of the web, as defined in Figure 1, such that $\sigma_{e1} < \sigma_{e2}$

NOTE The depth of the web referred to in 9.3.7.2.1 and 9.3.7.2.2 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 flange 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

t_{fo} and 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 of the flange material.

The clear width is taken as between:

- root fillets for rolled sections;
- webs for welded construction;
- 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, the spacing of shear connectors perpendicular to the direction of compression should not exceed:

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

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

$$12t_f \sqrt{\frac{355}{\sigma_{yf}}}$$

but may be increased to:

$$18t_f \sqrt{\frac{355}{\sigma_{yf}}}$$

in any line of shear connectors where 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:

$$50 \sqrt{\frac{355}{\sigma_y}}$$

where

σ_y is the nominal yield stress of the material of the circular hollow section.

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 5.1 of Part 5 of BS 5400:1979 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 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 Deduction for holes. Holes should be deducted in accordance with 11.3.3, where the longitudinal stress is tensile, and in accordance with 10.5.2 where the longitudinal stress is compressive.

9.4.2.3 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.

9.4.2.4 Effective compression flange. The effective area of each panel of a compression flange should be taken as equal to $K_c b t$;

where

b and t are the unsupported width and the thickness, respectively, of the flange panel (see Figure 1 and note 1 below)

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, respectively, or as a coefficient, to be determined from Figure 5, for plates with edges 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, from the appropriate curve for restrained panels;
- when there is only one longitudinal stiffener, or the flange is unstiffened from the appropriate curve for unrestrained panels;
- when the number of open stiffeners is two, by taking the mean of the values obtained from the two curves 1 and 2, or from curves 3, as appropriate.

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 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 flange plate

t , t_{f0} , b_{f0} , b , b_1 , b_2 are as shown in Figure 1.

NOTE 2 For rolled sections or unstiffened flanges gross properties may be used, providing the reduction due to K_e is incorporated in the calculation of the limiting compressive stress (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:

$$\text{a) } t_{we} = t_w, \text{ if } \frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} \leq 68$$

$$\text{b) } t_{we} = \left(1.425 - 0.00625 \frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} \right) t_w,$$

$$\text{if } 68 < \frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} < 228$$

$$\text{c) } t_{we} = 0, \text{ if } \frac{y_c}{t_w} \sqrt{\frac{\sigma_{yw}}{355}} \geq 228$$

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

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:

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

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 continuous on either side of the section under consideration over a distance equal to the depth of the beam.

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 may be taken as the average value throughout the net depth of the web, 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.

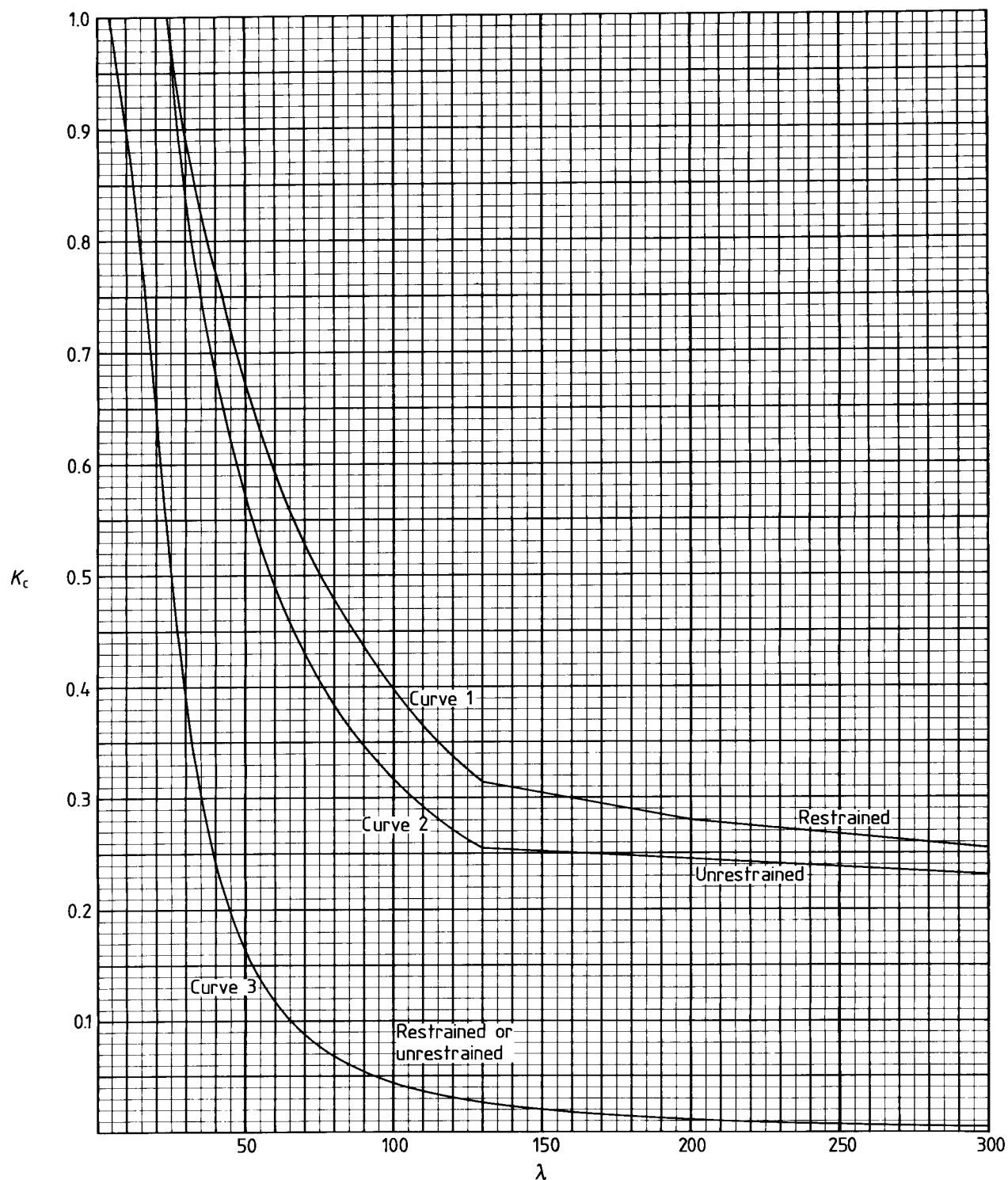
9.5.2 Stresses in longitudinally stiffened webs. The longitudinal stress in a longitudinally stiffened web should be calculated by the elastic theory without any assumption of redistribution of stresses. If the stress varies within the length a between transverse stiffeners, the value appropriate for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

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 panel or panels, of a beam with longitudinal stiffeners either 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, provided that the assumed stress distribution, after such shedding, is such that the whole of the applied bending moment and axial force is transmitted and equilibrium is maintained. The percentage reduction in stress so assumed should be uniform within any one panel of web plate bounded on each longitudinal edge 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.



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

a) curve 1 or 2 as relevant, with

$$\lambda = \frac{b}{t} \sqrt{\frac{\sigma_y}{355}} \text{ or}$$

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

a is the panel dimension in the direction of stress considered
 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 $a/b < 0.5$, items a) or 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

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 times 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 point under consideration

σ_y is the nominal yield stress of the section under consideration.

- 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 19, 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 load applied to a flange may be calculated on the assumption that the load is dispersed uniformly:

- a) 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
- b) at an angle of 45° from the line of application of the load through the web plate itself (see Figure 6).

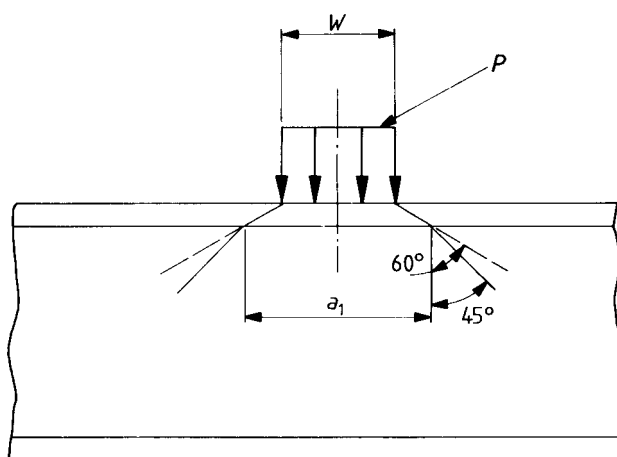


Figure 6 — Dispersal of load through an unstiffened web

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} and 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 any plane other than the vertical, such a section being outside the scope of this clause.

9.5.7.2 Stresses in webs

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

b) *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 the plane of 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 **9.12.4**.

In all cases the effective length for lateral torsional buckling λ_e should be determined in accordance with **9.6.2** to **9.6.6**, as appropriate. However, if the second moment of area of a cross section about the axis of bending is smaller than that about an axis perpendicular to it, the cross section as a whole is stable against lateral torsional buckling and its effective length λ_e may be taken as zero. Consideration should, however, be given to possible instability of constituent parts of the cross section.

9.6.2 Beams with intermediate lateral restraints When a compression flange of a beam is provided with effective discrete lateral restraints in accordance with **9.12.1**, λ_e should be taken as the greatest distance between such points of lateral restraint or between a restraint and a support. 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.

9.6.3 Beams (other than cantilevers) without intermediate lateral restraints. When there is no intermediate lateral restraint to a compression flange, ℓ_e should be taken as:

$$\ell_e = k_1 k_2 L$$

where

L is the span of the beam (i.e. between lateral restraints at supports)

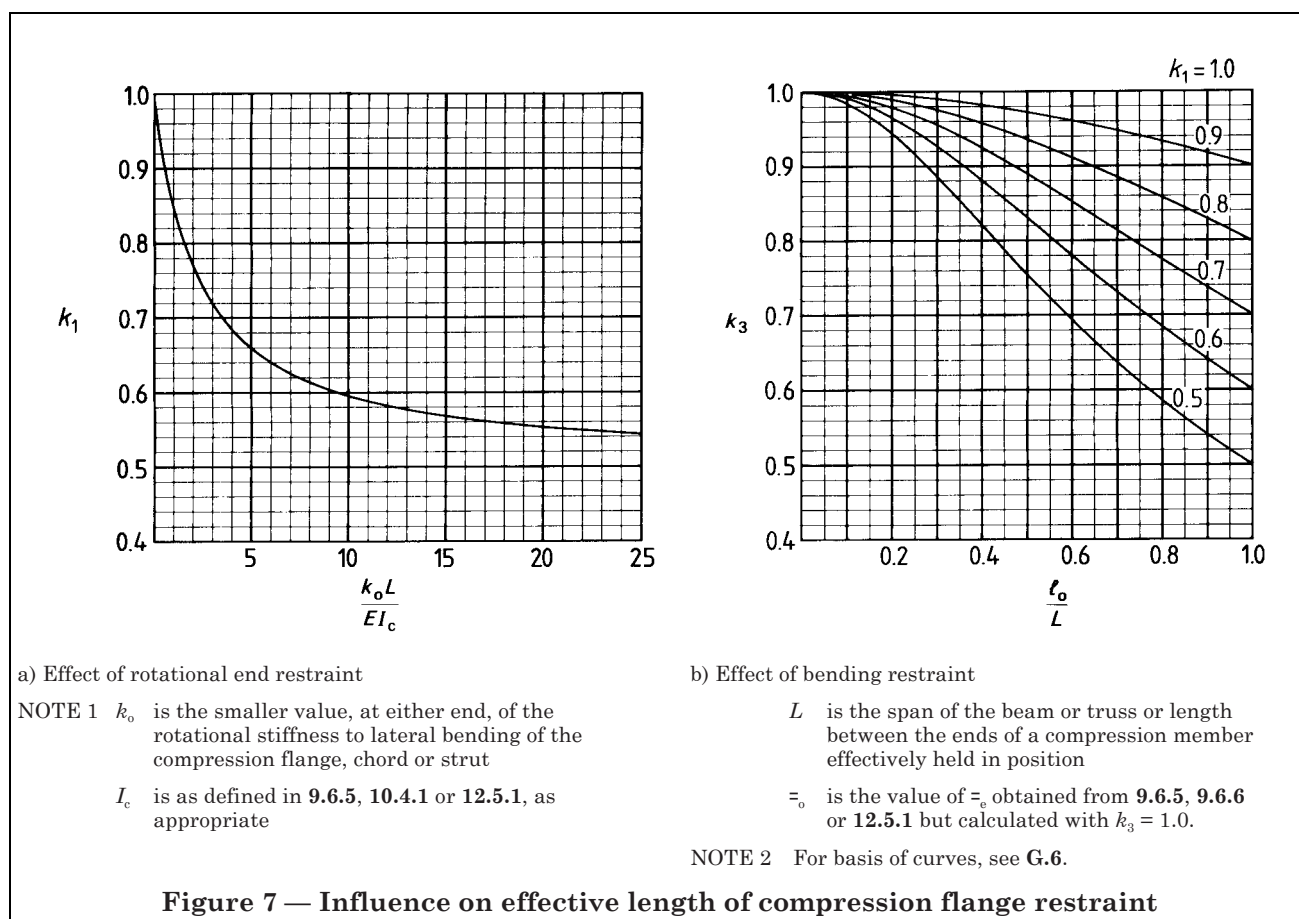
k_1 may conservatively be taken as:

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

$k_2 = 1.0$, or

1.2 if the load is applied to the top flange and both the flange and the load are free to move laterally.



9.6.4 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, in which L is the length of the cantilever.

9.6.5 Beam restrained by U-frames. When intermediate restraint to the compression flange is provided only by U-frames in accordance with 9.12.2, τ_e should be taken as:

$$\tau_e = 2.5k_3 (EI_c \tau_u \delta)^{0.25} \text{ but not less than } \tau_u$$

where

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 7 b)

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

τ_u is the distance between U-frames

δ is the lateral deflection which would occur in the U-frame at the level of the centroid of the flange being considered, when a unit force acts laterally to the U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ . The U-frame should be taken as fixed in position at each point of intersection between the cross member and a vertical, and as free and unconnected at all other points.

In cases of symmetrical U-frames, where cross members and verticals are each of constant moment of inertia throughout their own length, it may be assumed that:

$$\delta = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2} + fd_2^2$$

where

d_1 is the distance from the centroid of the compression flange to the nearer face of the cross member of the U-frame, as shown in Figure 8

Table 8 — Effective length τ_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 b) Tension flange held against displacement c) Both flanges held against lateral displacement	1.4L 1.4L 0.6L	0.8L 0.7L 0.6L
2. Continuous, and both flanges held against lateral displacement	a) Free b) Tension flange held against lateral displacement c) Both flanges held against lateral displacement	2.5L 2.5L 1.5L	1.0L 0.9L 0.8L
3. Continuous, with tension flange only held against lateral displacement	a) Free b) Tension flange held against lateral displacement c) Both flanges held against lateral displacement	7.5L 7.5L 4.5L	3.0L 2.7L 2.4L
NOTE L is the length of the cantilever.			

d_2 is the distance from the centroid of the compression flange to the centroidal axis of the cross member of the U-frame, as shown in Figure 8

I_1 is the second moment of area of the effective section of the vertical about its axis of bending. A width of web plate of up to 16 times the web thickness may be included on each side of the centreline of its connection when determining the effective section of the vertical

- I_2 is the second moment of area of the cross member of the U-frame about an axis perpendicular to the plane of the U-frame. A width of deck on either side of the U-frame, equal to $B/8$ or $\ell_u/2$, whichever is less, may be taken as the effective cross member when no other discrete member is present, or may be taken together with a cross member if structurally connected to it. In calculating the transformed area of a concrete deck, the gross area of concrete within this effective width may be considered
- u = 0.5 for an outer beam, or
 = 0.33 for an inner beam, if there are three or more beams interconnected by U-frames.
- B is the distance between centres of consecutive beams, or the maximum distance when the beams are not exactly parallel
- f is the flexibility of the joint between the cross member and the verticals of the U-frame, expressed in radians per unit moment; f may be taken as:
- 0.5×10^{-10} rad/N mm, when the cross member is bolted or riveted through unstiffened end-plates or cleats [see Figure 42 type(a)], or
 - 0.2×10^{-10} rad/N mm, when the cross member is bolted or riveted through stiffened end plates [see Figure 42 type(b)], or
 - 0.1×10^{-10} rad/N mm, when the cross member is welded right round its cross section or the connection is by bolting or riveting between stiffened end-plates on the cross member and a stiffened part of the vertical [see Figure 42 type(c)].

9.6.6 Beams continuously restrained by deck

9.6.6.1 Deck at compression flange level. When restraint to the compression flange is provided by a deck connected to the flange over the length of the beam, in accordance with 9.12.3.1, τ_e may be taken as zero.

9.6.6.2 Deck not at compression flange level. When restraint to the compression flange is provided by a deck connected to an unstiffened web, either directly or via the tension flange, over the length of the beam in accordance with 9.12.3.2, the effect of this restraint should be allowed for by assuming that the deck and webs of the main beams comprise a continuous series of effective U-frames of unit length longitudinally. In this case, τ_e should be taken as:

$$\tau_e = 2.5k_3 (EI_c \delta)^{0.25}$$

where

k_3 and I_c are as defined in 9.6.5

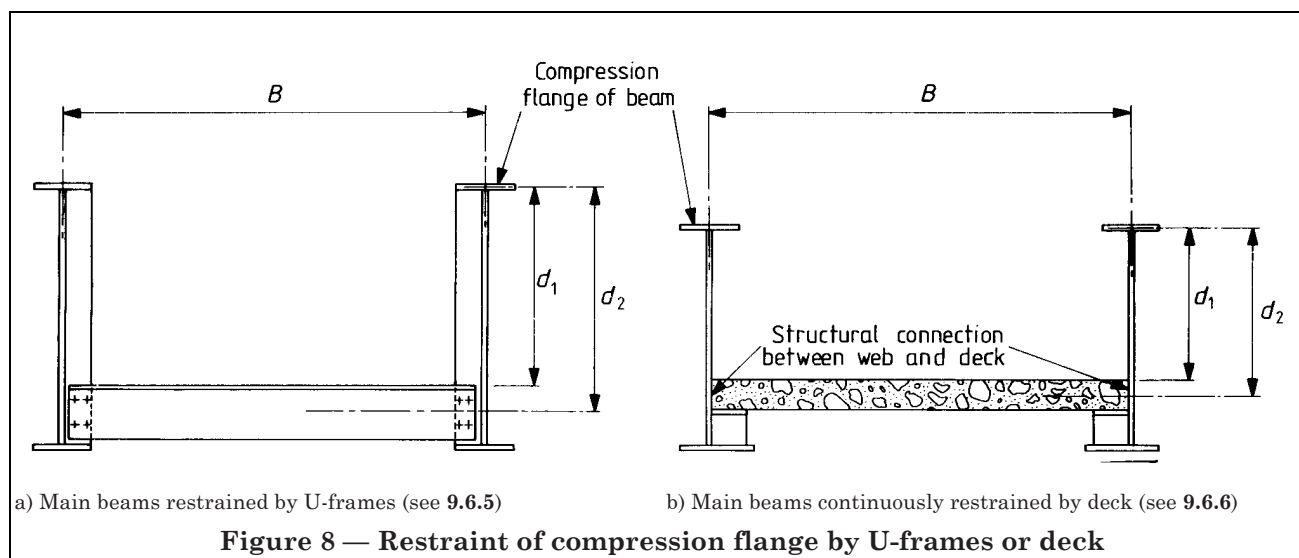
δ is the lateral deflection which would occur, in an effective U-frame, at the level of the centroid of the flange being considered, when a unit force acts laterally to the effective U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same effective U-frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ . The effective U-frame should be taken as fixed in position at each point of intersection of deck and web, and as free and unconnected at all other points.

In cases where the deck and webs of the beam are of constant thickness throughout the span, and the beam is of constant depth, it may be assumed that:

$$\delta = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2}$$

where

d_1 is the distance from the centroid of the compression flange to the nearest surface of the structural deck (see Figure 8)



d_2 is the distance from the centroid of the compression flange to the centroidal axis of the deck (see Figure 8)

$$I_1 = \frac{t_w^3}{12}$$

t_w is the thickness of the web of the beam

I_2 is the second moment of area of the deck per unit length, about its axis of bending, with the gross concrete area being transformed in terms of steel

u and B are as defined in 9.6.5.

9.7 Slenderness

9.7.1 General. The slenderness parameter λ_{LT} required for the calculation of the limiting compressive stress (see 9.8) should be determined for all beams in accordance with 9.7.2 to 9.7.5 appropriate to the type of beam, using the effective length for lateral torsional buckling obtained from 9.6.

9.7.2 Uniform I, channel, tee or angle sections. The value of λ_{LT} for a beam of I, channel, tee or angle section, uniform between points of effective lateral restraint to the compression flange, and bending about its X-X axis, as defined in Figure 1, should be taken as:

$$\lambda_{LT} = \frac{\ell_e}{r_y} k_4 \eta \nu$$

where

ℓ_e is the effective length determined in accordance with 9.6

r_y is the radius of gyration of the gross cross section of the beam about its Y-Y axis (see Figure 1)

k_4 = 0.9 for rolled I or channel section beams in accordance with BS 4 or BS 4848 or any I section symmetrical about both axes with t_f not greater than twice the web thickness, or
= 1.0 for all other beams

η = 1.0 for beams restrained by U-frames or continuously restrained by a deck (see 9.6.5 or 9.6.6), or may be conservatively taken as 1.0 for other beams, but where the bending moment varies substantially between points of lateral restraint, advantage may be obtained by using η , from Figure 9 a), if the loading is substantially concentrated within the middle-fifth of the length between full restraints or from Figure 9 b), for other loading patterns