British Standard

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BS 5400-3:2000

Incorporating Corrigendum No. 1

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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Association of Consulting Engineers
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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Contents

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 It supersedes BS 5400-3:1982, which is withdrawn.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.

BS 5400-5 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 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.

BS 5400-5 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 BS 5400-5, and of the stability of the bottom flange, which is not covered in this part of BS 5400; they should not be used unless proper account is taken of these considerations.

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.

The method given in BS 5400-5:1979, **4.1.3**a) 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 BS 5400-5 should not be used and reference made to Table 2 of this part of BS 5400.

Guidelines for Part 2 and Part 5

Since the publication of BS 5400-2 in 1978 and its amendment in 1983 and the publication of BS 5400-5 in 1979 and its amendment in 1982, guidelines to those parts and implementation documents have been produced by the central government and published as Departmental Standards BD 37 (DM RB 1.3) and BD 16 (DM RB 1.3) respectively.[1)]

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.

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to viii, pages 1 to 186, 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.*

BS 5135:1984, *Specification for arc welding of carbon and carbon manganese steels.*

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 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_{f3} 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_{f3} 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
γ_m, γ_{m1}, γ_{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
υ	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:

$$(\text{effects of } \gamma_{fL}Q_k) \leq \frac{1}{\gamma_{f3}\gamma_{m1}\gamma_{m2}} (\text{function } \sigma_y, \text{ and other geometric variables})$$

where

$\gamma_{fL}\gamma_{f3}$, 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_{f3} 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 γ_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	**9.10.2.3**a) 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 SO_3 per 100 cm^2 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 $5.65\sqrt{S_o}$, where S_o 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 $5.65\sqrt{S_o}$ 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 $5.65\sqrt{S_o}$.

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: Potential fracture initiation site		Detail description: BS 5400-10[a, b] Detail type	k_d[c]
Non-welded part	All	As-rolled surface Ground or machined edge		1.1, 1.2 1.3*	2
		Flame cut edge Mechanically fastened joints (drilled or reamed holes)		1.4* 1.5 to 1.11	1.5
		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 Transverse weld toes at ends of short welded attachments		2.1 to 2.5 2.9, 2.10	1
		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)	2.7	0.5
			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									
	N	BS EN 10113-2, BS EN 10210	86	79	72	66	60	55	50	42	35
	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 (°C) (see **6.5.2**);
T_{27J}	is the test temperature in degrees Celsius (°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 $C_v = 30$ J, $T_{27J} \equiv T_{30J}$;

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

where T_{30J} and T_{40J} are the specified test temperatures for $C_v = 30$ J and $C_v = 40$ 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, $E = 205\,000$ N/mm^2;
- — shear modulus, $G = 80\,000$ N/mm^2;
- — Poisson's ratio, $\upsilon = 0.3$;
- — coefficient of thermal expansion = 12×10^{-6}/°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 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 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	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
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

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

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

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 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:

a) 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;

b) 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/mm^2;

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:

a) flexure, shear, torsion and, for box girders, distortion, 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 BS 5400-5 for composite structures);

d) 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:

a) shear lag;

b) restraint of torsional warping;

c) 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**;

d) items c) and d) of **9.2.1.2** provided that:

1) the section is compact throughout the span being considered in accordance with **9.3.7**; and

2) 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:

a) 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:

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

$$m_{fL} = \frac{(\gamma_{fL,\,DEAD}\alpha_{DEAD} + \gamma_{fL,\,SUP}\alpha_{SUP} + \gamma_{fL,\,LIVE}\alpha_{LIVE} + \gamma_{fL,\,O}\alpha_O)_{SLS}}{(\gamma_{fL,\,DEAD}\alpha_{DEAD} + \gamma_{fL,\,SUP}\alpha_{SUP} + \gamma_{fL,\,LIVE}\alpha_{LIVE} + \gamma_{fL,\,O}\alpha_O)_{ULS}}$$

where

α_{DEAD}, α_{SUP}, α_{LIVE}, α_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 **9.2.1.2**a) 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:

a) 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;

b) the longitudinal distance between the boundaries of adjacent openings is at least three times the maximum internal dimension;

c) 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) a value of 10; or

b) 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:

a) either $\frac{\ell_s}{h_s}$ does not exceed $3\sqrt{\frac{355}{\sigma_{ys}}}$; or

b) $\frac{(b + k_s h_s)}{t}\sqrt{\frac{\sigma_y}{355}}$ 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 limite 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*

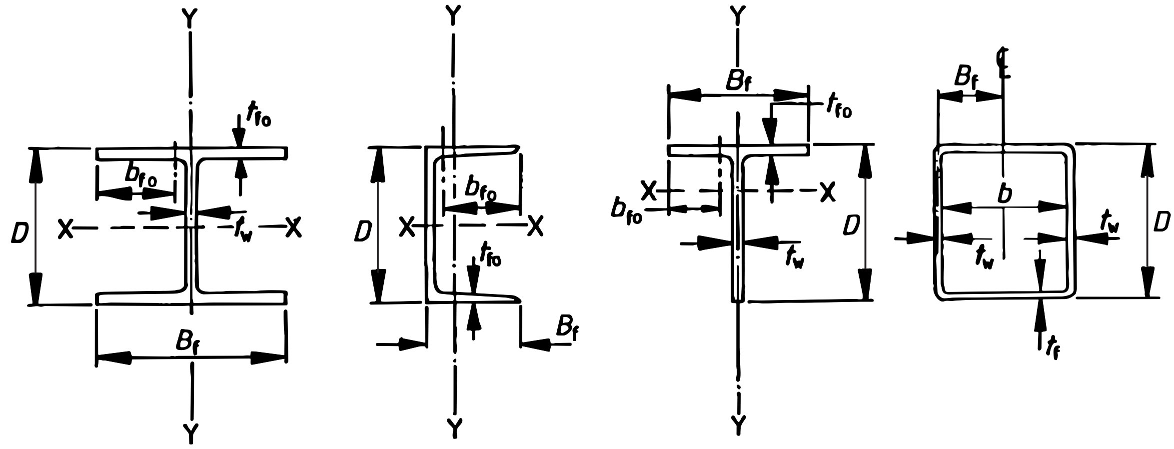
Closed stiffeners should be proportioned such that:

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.

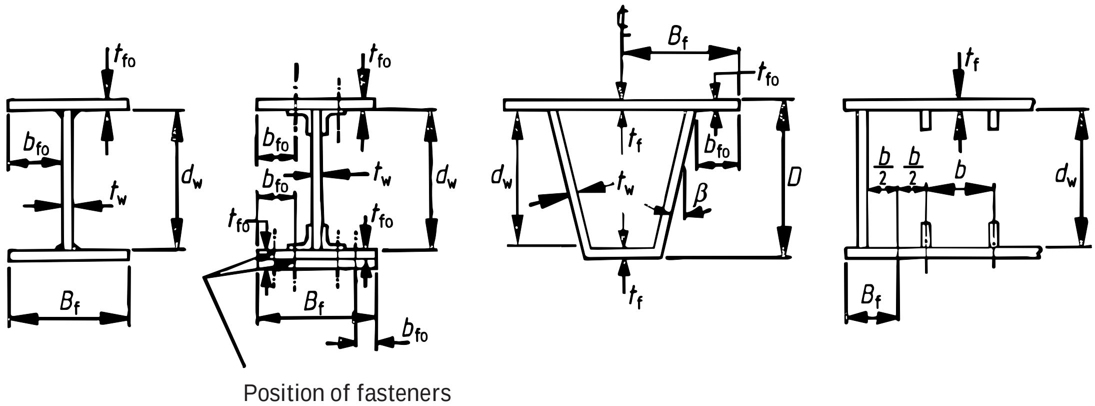
where

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**.



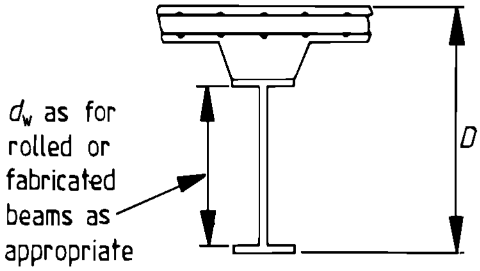
NOTE For tapered flanges, t_{fo} should be taken as the mean thickness of the flange, as defined in BS 4-1.

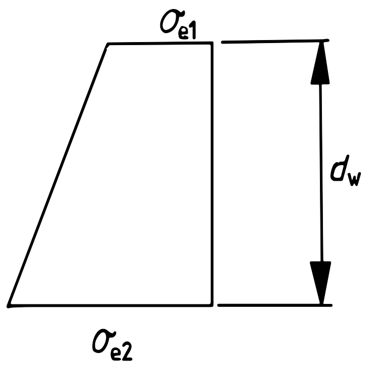
a) Rolled beam sections, $d_w = D$



b) Fabricated beam sections, d_w is as shown

Figure 1 — Geometric notation for beams

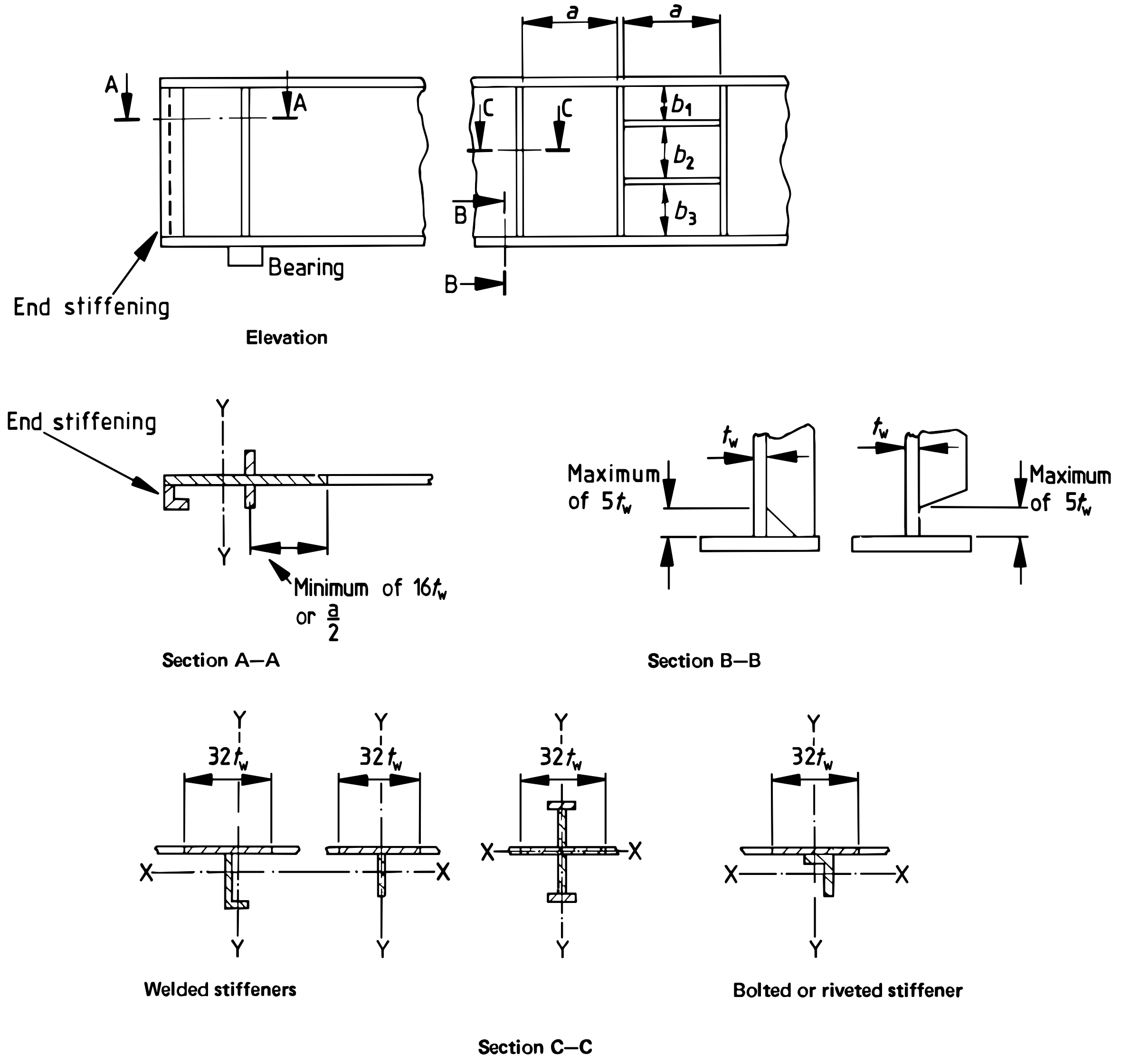




NOTE *D* is shown for concrete in compression. For concrete in tension *D* is the level of tension reinforcement.

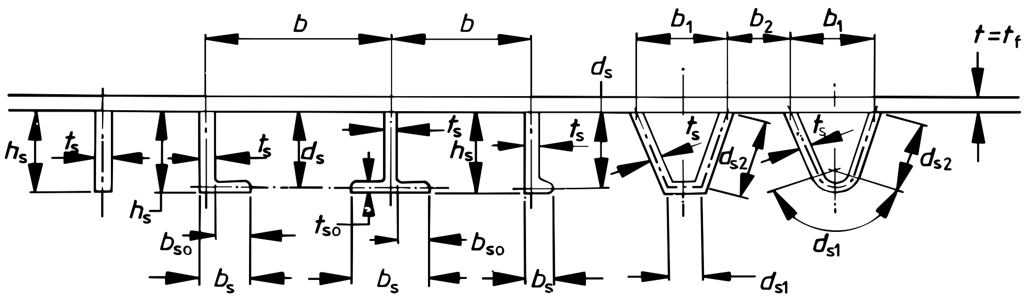
c) Composite beam section

d) Stress distribution in web



e) Stiffened beam

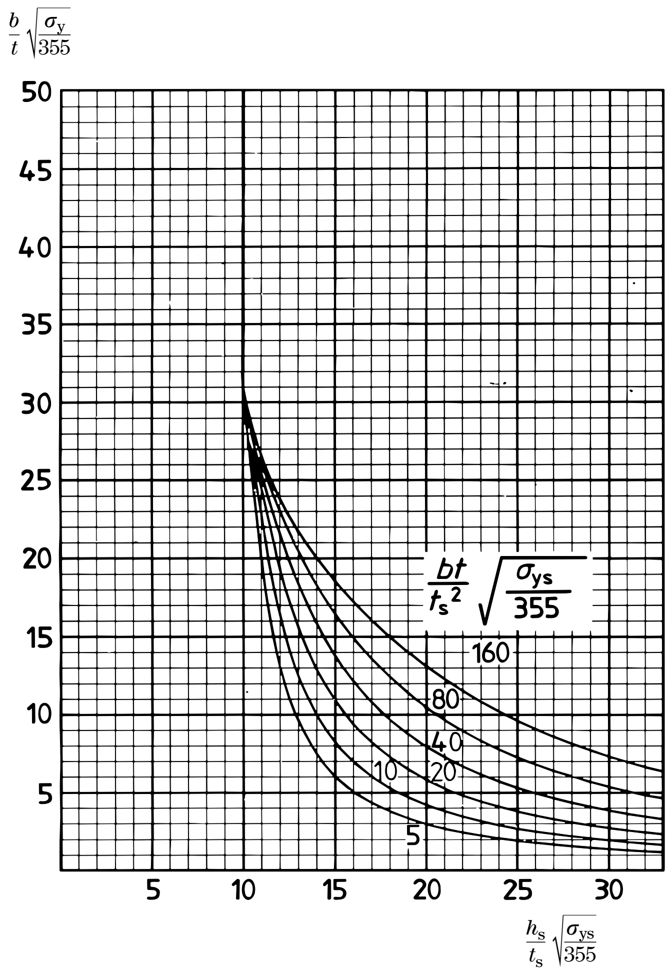
Figure 1 — Geometric notation for beams *(continued)*



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

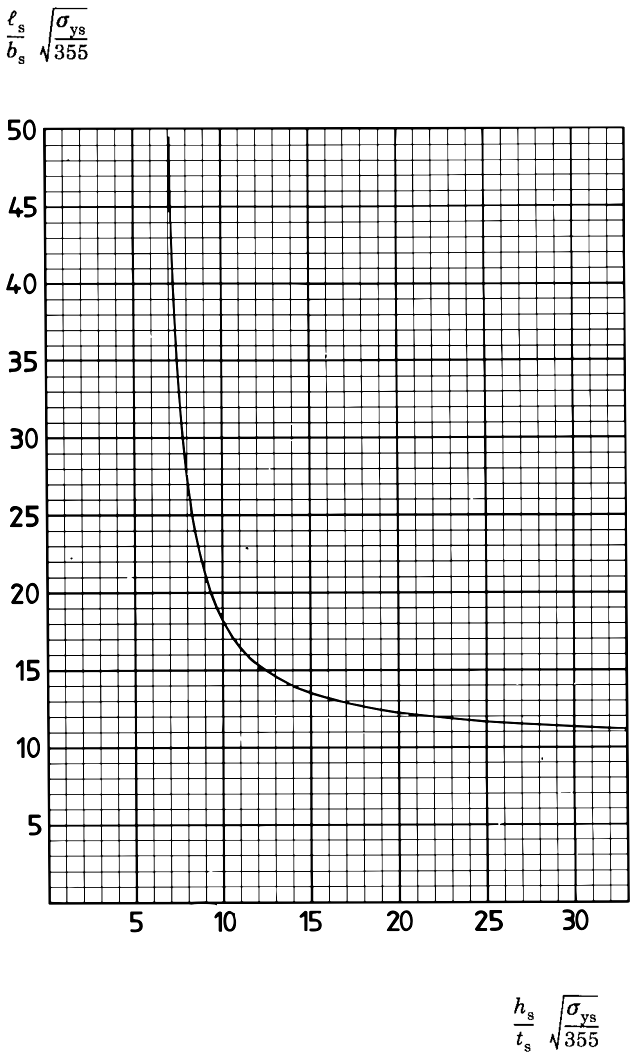
f) Flange and web stiffeners

Figure 1 — Geometric notation for beams *(continued)*



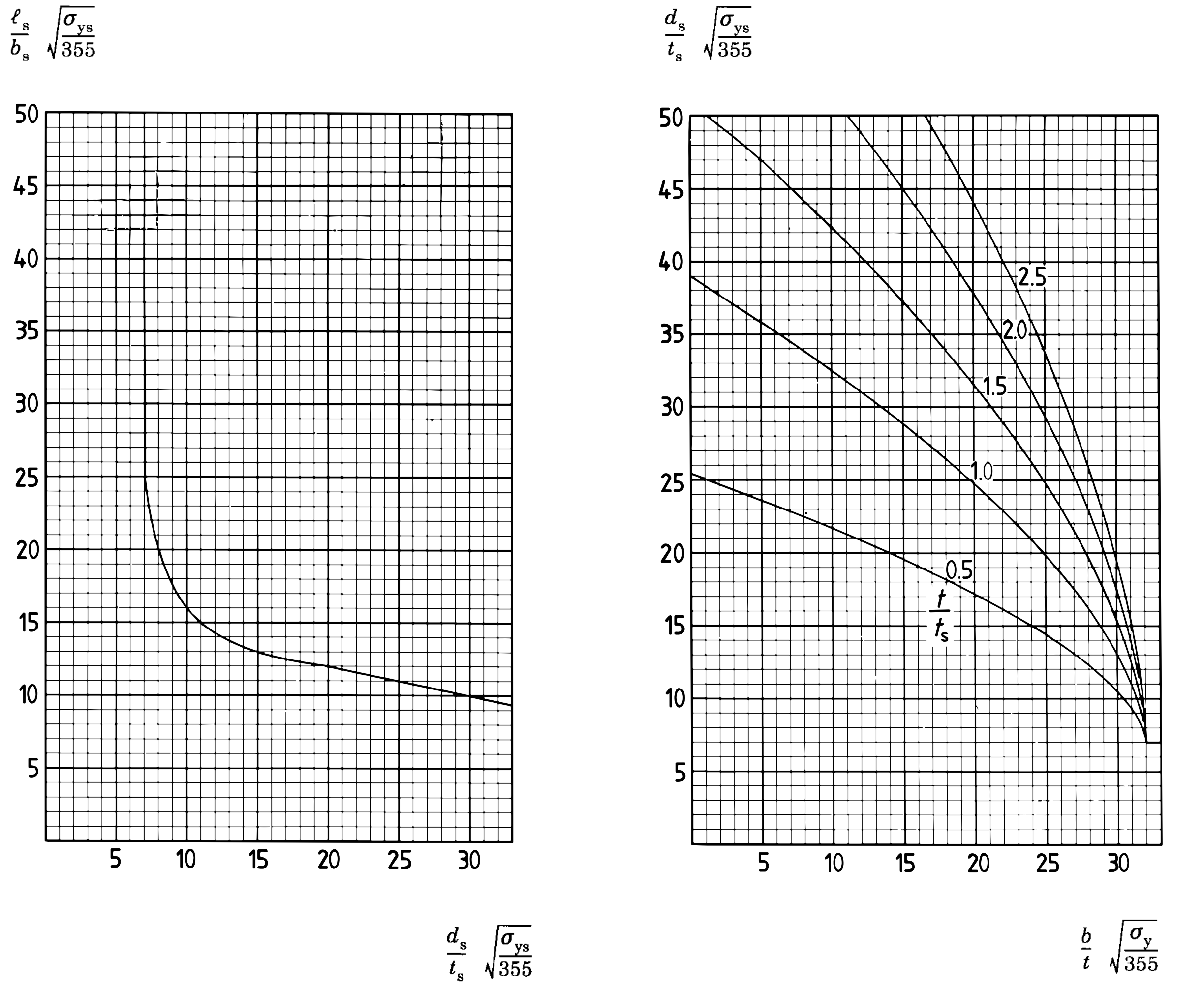
NOTE For basis of curves, see **G.2**.

Figure 2 — Limiting slenderness for flat stiffeners



NOTE For basis of curve, see **G.3**.

Figure 3 — Limiting slenderness for angle stiffeners



NOTE For basis of curve, see **G.4**a).

a) Related to ℓ_s/b_s

NOTE For basis of curves, see **G.4**b).

b) Related to *b*/*t*

Figure 4 — Limiting slenderness for tee stiffeners

9.3.5 *Flanges curved in elevation*

Flanges curved in elevation should be such that:

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

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}, 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:

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

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

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:

a) root fillets for rolled sections;

b) webs for welded constructions;

c) 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 $A_e \Sigma b_e / B_f$

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 $\Sigma K_c(k_h A_c)$;

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:

a) 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;

b) 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;

c) 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 = 1.0 for a section free from holes or for a section with one or more holes not greater than 40 mm in diameter; or

= 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;

Σb_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:

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

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) $t_{we} = \left(1.425 - 0.006\,25\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}\right)t_w$, 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.

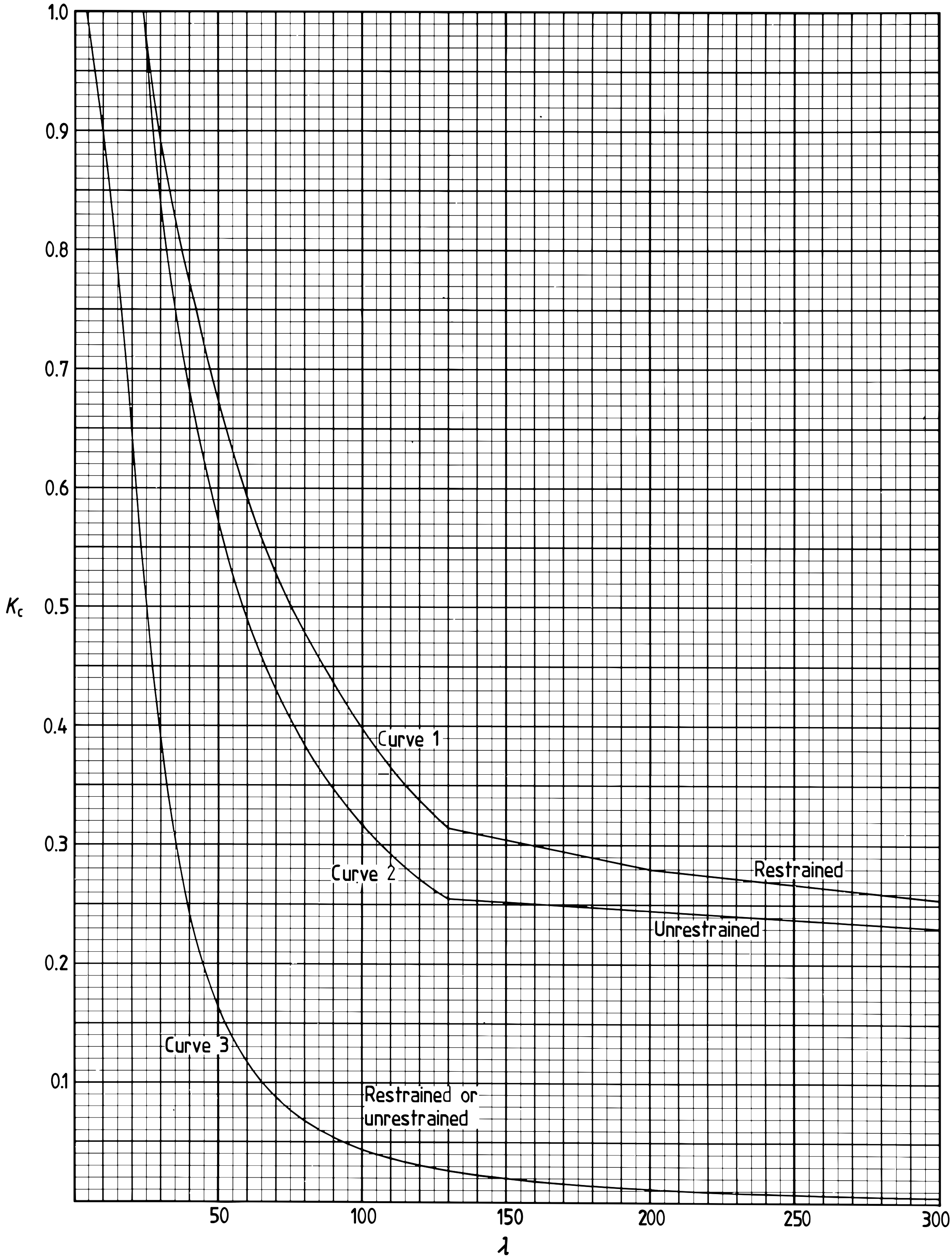


Figure 5 — Coefficient K_c for plate panels under direct compression

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 $2\sigma_y/E$;

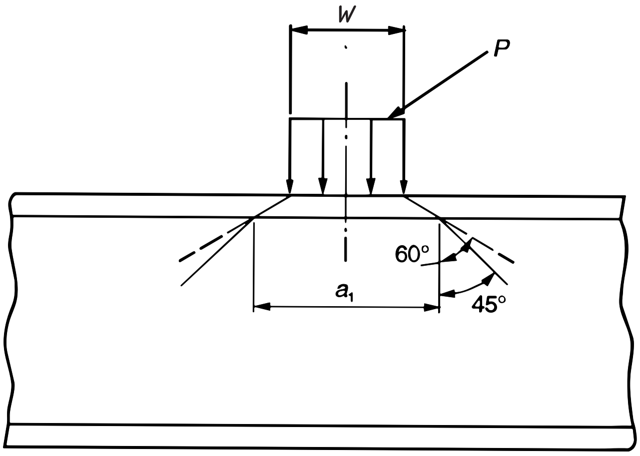
d) the web stresses obtained in b) should be in accordance with **9.11**. For the buckling check of a web plate panel, in accordance with **9.11.4**, an equivalent system of stress patterns, shown in Figure 20, may be assumed such that the net axial force along, and the bending moment about, the longitudinal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

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).



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:

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

$\dfrac{3\sigma_f b^2}{4R_f t_f}$ 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**.

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 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.

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;

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

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**, which may be derived using k_e = 1.0 in calculating λ_F;

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:

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

where

ΣL 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;

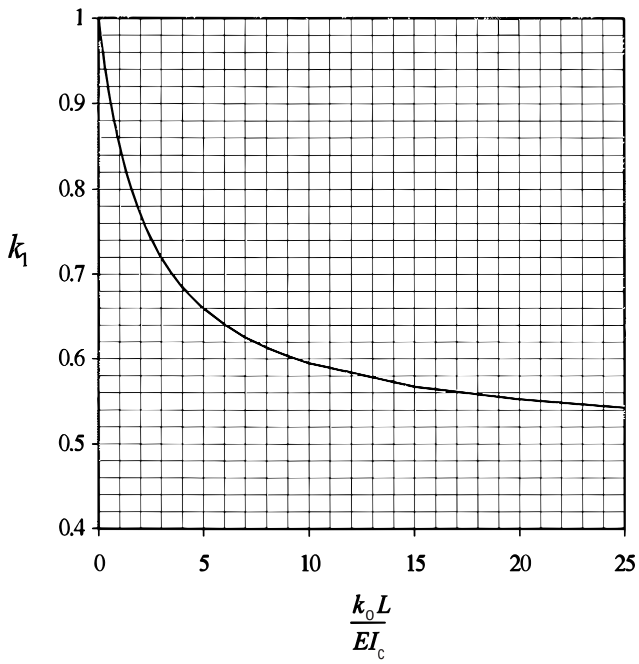
δ_i is the value of δ_t for an internal support;

δ_e is the value of δ_t for the end or internal supports at the opposite ends of the adjacent spans.

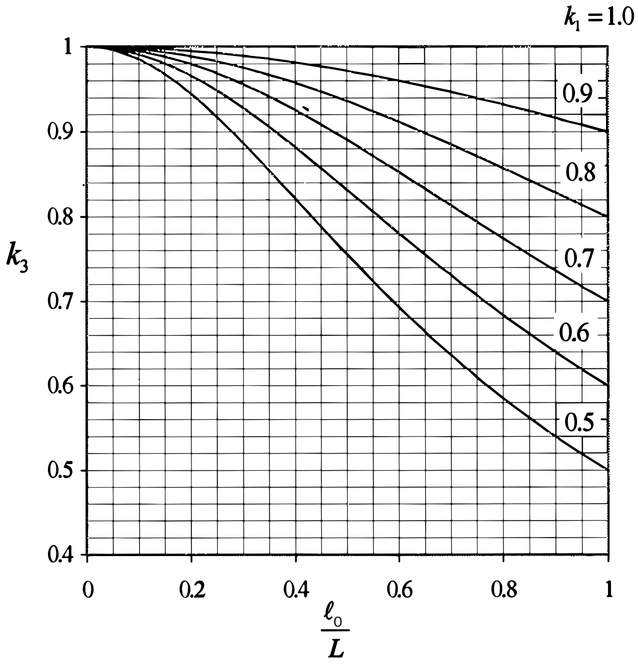
NOTE 4 The equations for ℓ_e may be used to determine the stiffness of the supports required to achieve a selected effective length.

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.



a) Effect of rotational end restraint



b) Effect of bending restraint

NOTE 1 k_0 is the smaller value, at either end, of the rotational stiffness to lateral bending of the compression flange, chord or strut;

I_c is as defined in **9.6.2** or **9.6.4.1.1.2**, as appropriate;

L is the span of the beam or truss or the length between the ends of a compression member effectively held in position;

ℓ_0 is the value of ℓ_e obtained from **9.6.4.1.1.2** but calculated with $k_3 = 1.0$.

NOTE 2 For basis of curves, see **G.6**.

Figure 7 — Influence on effective length of compression flange restraint

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.

A discrete lateral restraint may be taken as fully effective provided that it has a stiffness such that its deflection in a direction normal to the axis of the beam under unit force applied in the same direction is not greater than $\ell_R^3/40EI_c$ where 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.

When the restraint is provided by a plan bracing system with truss nodes attached to the compression flange at centres ℓ_R, to be fully effective the bracing system should have stiffness such that the deflection normal to the axis of the beam of a node due to a unit force applied to it relative to adjacent nodes is not greater than $\ell_R^3/40EI_c$.

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:

a) $\ell_e = k_2 k_3 k_5 \ell_1$ but not less than $k_3 \ell_R$

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

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

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 $= 2.22 + \dfrac{0.69}{X + 0.5}$;

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

ℓ_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 is the distance between points of intermediate restraint or between an intermediate restraint and a support restraint which results in the lowest limiting moment of resistance;

δ_R is the lateral deflection which would occur in the restraint, at the level of the centroid of the flange being considered, when a unit force acts laterally to the restraint only at this point;

δ_{e1}, δ_{e2} are the relative lateral deflections δ_t 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*

When a beam of uniform cross-section 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 parameter $v^4L^3/(EI_c\theta_R d_f^2)$ but should be not less than $(1.7 - 0.6v)\ \ell_R$. 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 the appropriate central restraint curves.

where

d_f is the vertical distance between the centroid of the compression and tension flanges respectively at the position of the torsional restraint;

θ_R is the rotation of the restraint, about the longitudinal axis of the beam of each of any number, n, of equally spaced torsional restraints in a span due to a torque, equal to a unit torque multiplied by $1/n$, applied to each restraint without contribution from the restrained beam. When such restraint is provided by diaphragms interconnecting two or more beams the rotations should be calculated assuming the above torque to be applied in the same rotational direction by equal and opposite horizontal forces on the diaphragms. In such cases the restraining torques have to be resisted by equal and opposite vertical forces on the connected beams, equal to the value of the torque divided by the beam spacing, and account should be taken of the deflections of the beams due to all the restraining torques, θ_R being taken as that at the restraint where there is the greatest total rotation;

L is as defined in **9.6.2**a);

ℓ_R is as defined in **9.6.4.1.1.2**;

v is calculated in accordance with **9.7.2** which should be derived using $\ell_e = L$ in calculating λ_F;

I_c is as defined in **9.6.4.1.1.2**.

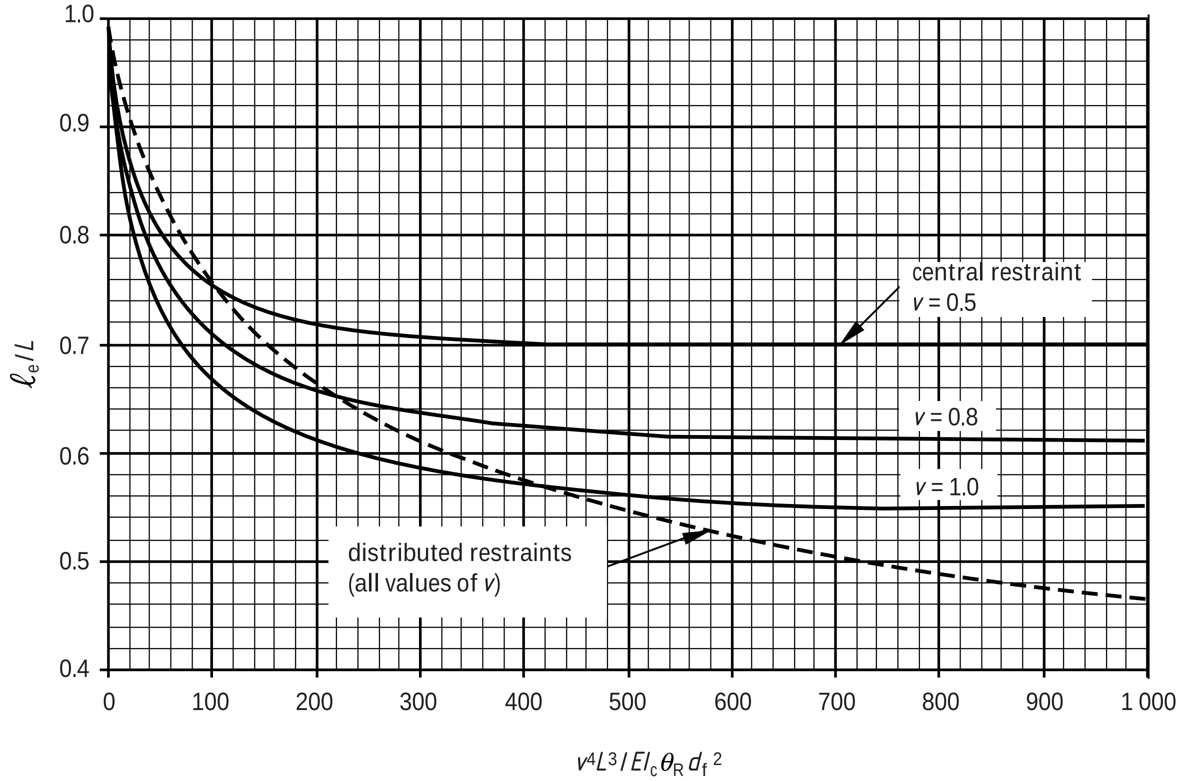


Figure 8 — Effective length of beams with discrete torsional restraints

9.6.4.1.3 *Beams with discrete U-frame restraints*

When restraint to a compression flange is provided only by U-frames in accordance with **9.12.3**, ℓ_e should be calculated in accordance with **9.6.4.1.1.2** with:

ℓ_R equal to the U-frame spacing;

δ_e equal to the lateral deflection of the end U-frame or stiffener, acting as a cantilever, when unit lateral forces act as given for the calculation of δ_R;

δ_R equal to the lateral deflection which would occur in any intermediate 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_R. 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:

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

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 9;

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 9;

I_1 is the second moment of area of the effective section of the vertical about its axis of bending perpendicular to the plane of the U-frames. 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 $\ell_R/2$, 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 u = 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;

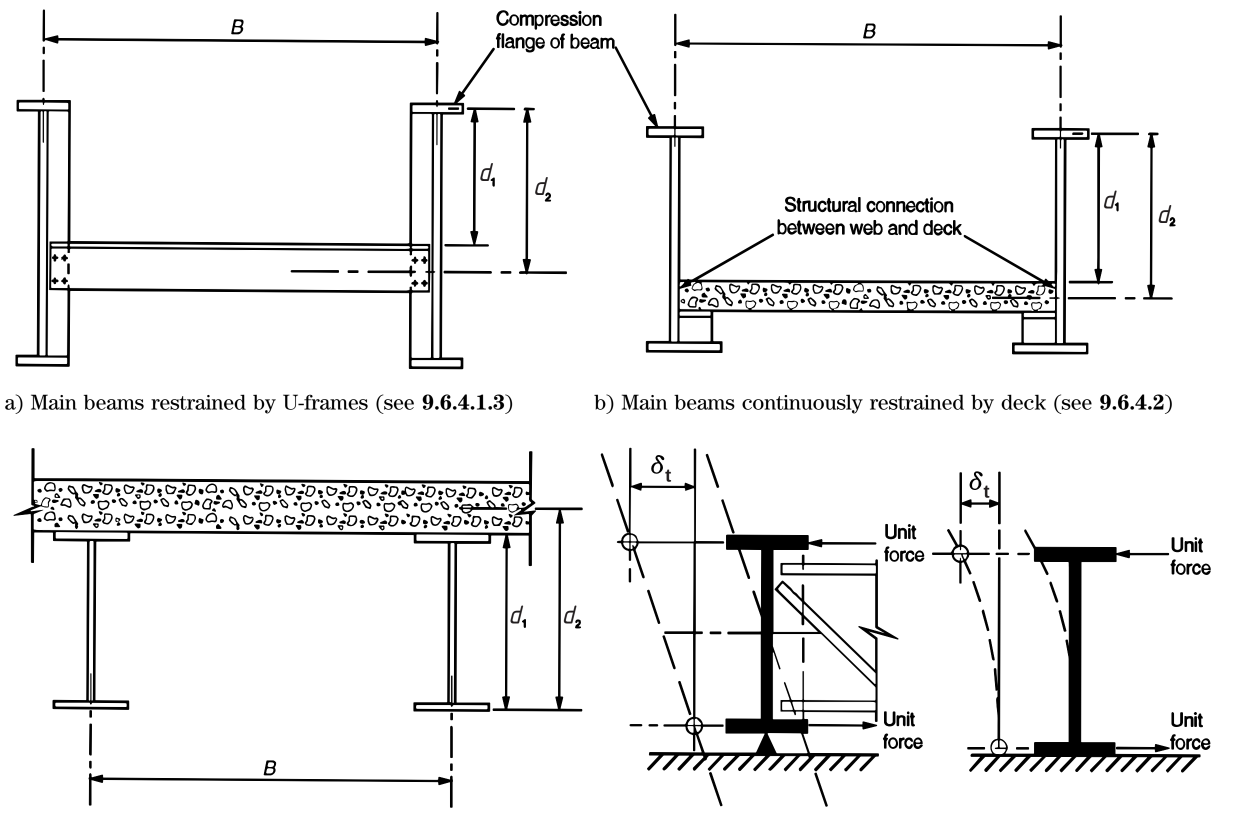
f is the flexibility of the joint between the cross member and the verticals of the U-frame, expressed in radians per unit moment; f may be taken as:

a) 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

b) 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

c) 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)].

NOTE Values of f may be determined experimentally or taken from available test results which should cover the required ultimate capacity of the joint.



a) Main beams restrained by U-frames (see **9.6.4.1.3**)

b) Main beams continuously restrained by deck (see **9.6.4.2**)

c) Composite deck main beams continuously restrained by deck (see **9.6.4.2**)

d) End torsional restraint (see **9.6.2**)

Figure 9 — Restraint of compression flange by U-frames or deck or end torsional restraint

9.6.4.2 *Beams continuously restrained by deck*

9.6.4.2.1 *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 **9.12.4.1**, ℓ_e may be taken as zero.

9.6.4.2.2 *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 **9.12.4.2**, 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 ℓ_e should be taken as in **9.6.4.1.3** with ℓ_R equal to unity and with δ_R equal to 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_R. 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:

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

where

d_1 is the distance from the centroid of the compression flange to the nearest surface of the structural deck (see Figure 9);

d_2 is the distance from the centroid of the compression flange to the centroidal axis of the deck (see Figure 9);

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, B are as defined in **9.6.4.1.3**.

9.7 Slenderness

9.7.1 *General*

The slenderness parameter λ_{LT} needed for the calculation of the limiting moment of resistance (see **9.8**) should be determined for all beams in accordance with **9.7.2** to **9.7.5** as appropriate to the type of beam, using the effective length for lateral torsional buckling ℓ_e obtained from **9.6**.

The half wavelength of buckling of the compression flange is needed in the application of the various clauses. It may be taken as:

a) the distance between supports of beams without intermediate restraints;
b) the distance between fully effective discrete intermediate restraints (see **9.6.4.1.1.1** and **9.6.4.1.1.2**);
c) ℓ_w as defined in **9.8** for beams with discrete restraints which are not fully effective.

The moduli of the cross-section are needed in the application of various clauses which follow. The derivation of these should be based on the following:

Z_{pe} is the plastic modulus of the effective section derived in accordance with **9.4.2**, and is defined as M_{pe}/σ_{yc}

where

M_{pe} is the plastic moment of resistance of the effective cross-section derived in accordance with **9.4.2** and based on rectangular stress blocks of intensity equal to the strength of the elements. In the case of elements in structural steel, the strength should be taken as the nominal yield stress of the elements, as defined in **6.2**. In the case of concrete flanges in compression, the area of reinforcement should be ignored and the strength should be taken as $0.4f_{cu}\gamma_m$. In the case of concrete flanges in tension the area of concrete should be ignored and the strength of the reinforcement taken as $0.87f_y\gamma_m$, where:

f_{cu} is the concrete cube strength in accordance with BS 5400-4;

f_y is the characteristic strength of the reinforcement in accordance with BS 5400-4.

NOTE 1 γ_m is the partial factor for structural steel and is given in **4.3.3**.

σ_{yc} is the nominal yield stress value, as defined in **9.3.1**, for the compression flange of the steel section;

Z_{xc}, Z_{xt} are the elastic moduli of the section with respect to the extreme compression and tension fibres respectively, based on the effective section derived in accordance with **9.4.2**.

Z_{xw} is the minimum elastic modulus of the section with respect to the web, based on the effective section derived in accordance with **9.4.2**.

NOTE 2 For composite sections Z_{xc}, Z_{xt} and Z_{xw} should be based on the transformed section. The transformed area of the concrete compression flange should be obtained using either the short-term or the long-term modular ratio of the concrete as appropriate to the type of loading. Concrete in tension should be ignored but the area of reinforcement in concrete subject to tension should be included.

9.7.2 *Uniform I, channel, tee or angle sections*

The value of λ_{LT} for overall lateral buckling of a beam of I, channel, tee or angle section, uniform within the half-wavelength of buckling of the compression flange, and bending about its X-X axis, as defined in Figure 1, should be taken as:

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

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 4-1 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, but where the bending moment varies substantially within the half-wavelength of buckling of the compression flange, advantage may be obtained by using η, from Figure 10a), if the loading is substantially concentrated within the middle-fifth of the half-wavelength or from Figure 10b), for other loading patterns;

v is dependent on the shape of the beam, and may be obtained from Table 9, using the parameters:

$$\lambda_F = \frac{\ell_e}{r_y}\left(\frac{t_f}{D}\right) \text{ and}$$

$$i = \frac{I_c}{I_c + I_t}$$

t_f is the mean thickness of the two flanges of an I- or channel section, or the mean thickness of the table of a tee or leg of an angle section;

D is the depth of the cross-section (see Figure 1);

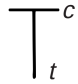
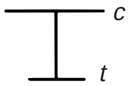
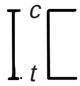
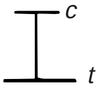
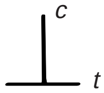
I_c, I_t are the second moments of area of the compression and tension flange, respectively, about their Y-Y axes, as defined in Figure 1, at the section being checked. For beams with $I_c \geq I_t$ or with $\lambda_F \geq 8$, λ_{LT} may conservatively be taken as ℓ_e/r_y.

NOTE 1 Where a flange is common to two or more beams (for example in a girder bridge with a composite deck) the properties r_y, I_y, I_c or I_t may be calculated by attributing a fraction $1/n$ of the lateral second moment of area and of the area of the common flange to the section of each beam, where n is the number of beams.

NOTE 2 In calculating t_f, I_c and I_t for composite beams, the equivalent thickness of the composite flange in compression should be based on the long term elastic modulus for concrete. Concrete in tension should be ignored and the equivalent thickness of tension reinforcement should be taken as the area of reinforcement divided by the flange width over which it is placed.

When, in accordance with **9.6.4.1.3** or **9.6.4.2**, ℓ_e is greater than ℓ_R, λ_{LT} should be taken as equal to ℓ_e/r_{yc} where r_{yc} is the radius of gyration about the Y-Y axis of the gross cross-section of the compression flange plus one-third of the height of the web.

Table 9 — Slenderness factor *v* for beams of uniform section

i	**1.0**	**0.8**	**0.6**	**0.5**	**0.4**	**0.3**	**0.2**	**0.1**	**0**
λ_F									
0.0	0.791	0.842	0.932	1.000	1.119	1.291	1.582	2.237	∞
1.0	0.784	0.834	0.922	0.988	1.102	1.266	1.535	2.110	6.364
2.0	0.764	0.813	0.895	0.956	1.057	1.200	1.421	1.840	3.237
3.0	0.737	0.784	0.859	0.912	0.998	1.116	1.287	1.573	2.214
4.0	0.708	0.752	0.818	0.864	0.936	1.031	1.162	1.359	1.711
5.0	0.679	0.719	0.778	0.817	0.878	0.954	1.055	1.196	1.415
6.0	0.651	0.688	0.740	0.774	0.824	0.887	0.966	1.071	1.219
7.0	0.626	0.660	0.705	0.734	0.777	0.829	0.892	0.973	1.080
8.0	0.602	0.633	0.674	0.699	0.736	0.779	0.831	0.895	0.977
9.0	0.581	0.609	0.645	0.668	0.699	0.736	0.780	0.832	0.896
10.0	0.562	0.587	0.620	0.639	0.667	0.699	0.736	0.779	0.831
11.0	0.544	0.567	0.597	0.614	0.639	0.666	0.698	0.735	0.778
12.0	0.528	0.549	0.576	0.591	0.613	0.638	0.665	0.697	0.733
13.0	0.512	0.533	0.557	0.571	0.590	0.612	0.636	0.664	0.695
14.0	0.499	0.517	0.539	0.552	0.570	0.589	0.611	0.635	0.662
15.0	0.486	0.503	0.523	0.535	0.551	0.568	0.588	0.609	0.633
16.0	0.474	0.490	0.509	0.519	0.534	0.550	0.567	0.586	0.607
17.0	0.463	0.478	0.495	0.505	0.518	0.533	0.548	0.566	0.585
18.0	0.452	0.466	0.482	0.492	0.504	0.517	0.531	0.547	0.564
19.0	0.442	0.456	0.471	0.479	0.491	0.503	0.516	0.530	0.546
20.0	0.433	0.446	0.460	0.468	0.478	0.489	0.502	0.515	0.529

NOTE 1 $\lambda_F = \dfrac{\ell_e t_f}{r_y D}$, $i = \dfrac{I_c}{I_c + I_t}$, i = 0.5 when flanges are equal.

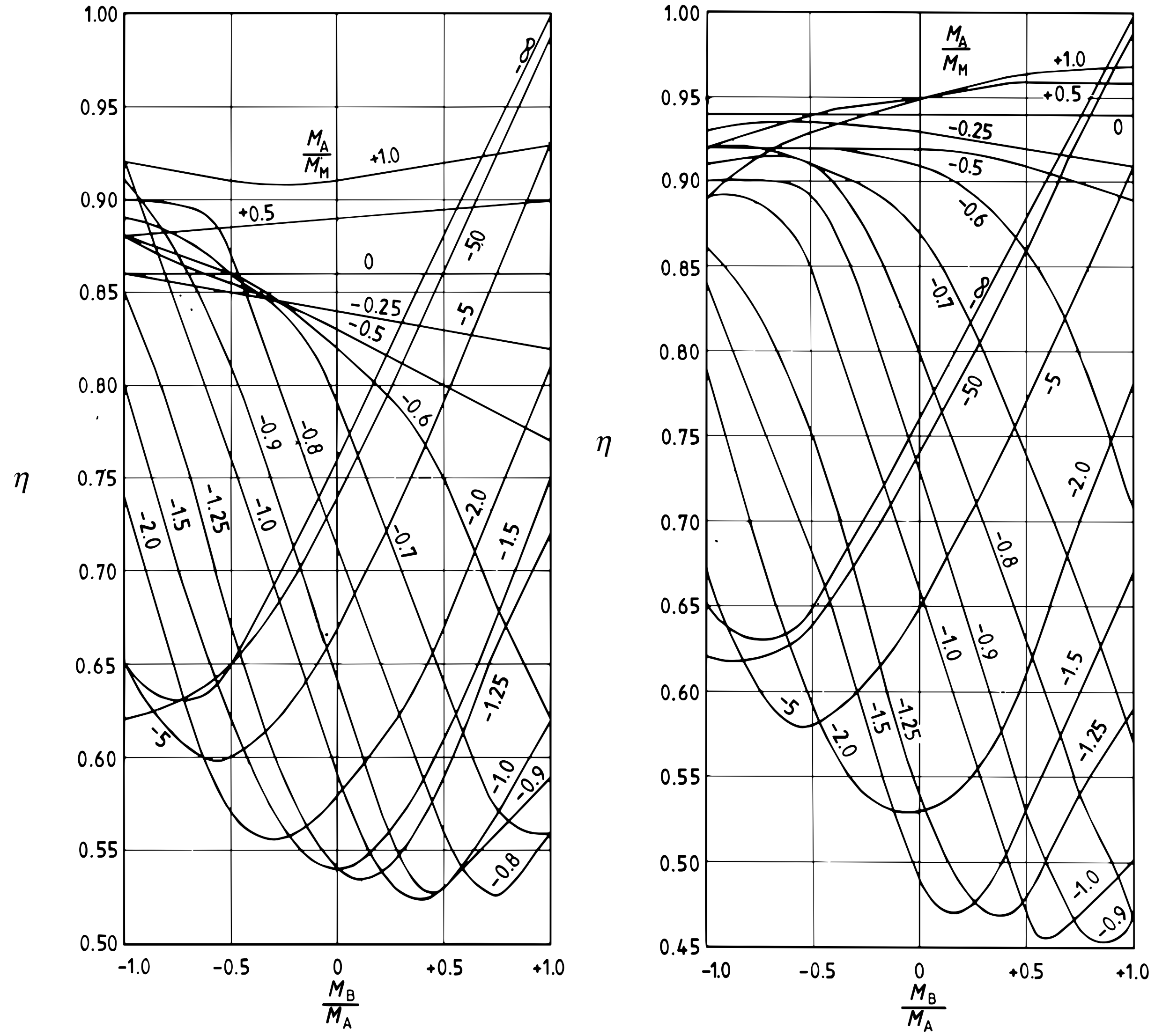
NOTE 2 Intermediate values to the right of the stepped line should be determined from the formula given in note 3 rather than by interpolation between the tabulated values.

NOTE 3 $v = [\{4i(1 - i) + 0.05\lambda_F^2 + \psi_i^2\}^{0.5} + \psi_i]^{-0.5}$

where

$\psi_i = 0.8\,(2i - 1)$, when $I_c \geqslant I_t$

$\psi_i = 2i - 1$, when $I_c < I_t$



a) Applied loading substantially concentrated within the middle fifth of the half wavelength of buckling

b) Applied loading other than for a)

Figure 10 — Slenderness factor η for variation in bending moment

NOTE 1 The procedure for using Figure 10 is as follows:

a) all hogging moments should be considered positive;

b) the ends A and B should be chosen such that $M_A \geq M_B$ regardless of sign;

c) M_M is the mid-span moment on a simply supported span equal to the half wavelength of buckling.

NOTE 2 Examples of the use of Figure 10 are as follows:

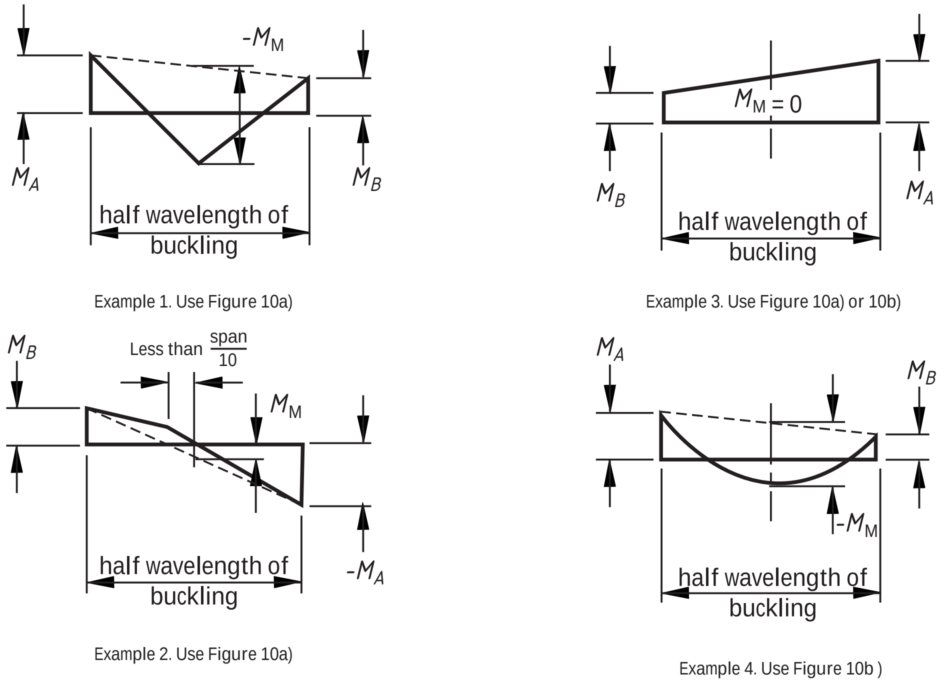


Figure 10 — Slenderness factor η for variation in bending moment *(continued)*

9.7.3 *Other uniform sections*

9.7.3.1 *Uniform rectangular or trapezoidal box sections*

The value of λ_{LT} for overall lateral buckling of a beam of rectangular or trapezoidal box section, uniform within the half-wavelength of buckling of the compression flange, should be taken as:

$$\lambda_{LT} = 2.25\eta\xi\sqrt{\frac{Z_{pe}\ell_e}{r_y\sqrt{AJ}}}$$

where

η, r_y are as defined in **9.7.2**;

Z_{pe} is as defined in **9.7.1**;

NOTE 1 For non-compact sections, Z_{pe} need not be calculated explicitly since (according to **9.7.1**) it may be replaced by M_{pe}/σ_{yc} and in the subsequent application of **9.8**, M_{pe} will cancel.

ℓ_e is as determined in accordance with **9.6**;

A is the area of the gross cross-section;

J is the torsional constant $4A_o^2/\Sigma\,(B/t)$;

A_o is the area enclosed by the median line of the perimeter material of the section;

B, t are the width and thickness, respectively, of each wall of the section forming the closed perimeter;

NOTE 2 In the case of a wall made from material other than steel, t should be taken as the actual thickness multiplied by the ratio of the shear modulus of the material used to the shear modulus of steel. Where the shear modulus varies with the load history, the long-term value should be used.

$$\xi = \left\{ \frac{(I_x - I_y)(I_x - 0.385J)}{I_x^2} \right\}^{0.25}$$

I_x, I_y are the second moments of area of the gross cross-section about axes through the centroid normal to the plane of bending, and in the plane of bending, respectively.

9.7.3.2 *Uniform solid rectangular sections*

The value of λ_{LT} for overall lateral buckling of a beam of homogeneous solid rectangular section, which is uniform within the half-wavelength of buckling of the compression flange, should be taken as:

$$\lambda_{LT} = 2.8\eta \frac{\sqrt{\ell_e D}}{B}$$

where

η is as defined in **9.7.2**;

ℓ_e is determined in accordance with **9.6**;

D is the depth of the section in the plane of bending;

B is the width of the section.

9.7.4 ***Varying sections***

For a beam of varying section within the half wavelength of buckling an effective slenderness λ_{LT} for overall lateral buckling should be obtained at each cross-section for which the bending resistance is to be checked. The value of λ_{LT} should be taken as $(1.5 - 0.5\rho_f)$ times the value obtained from **9.7.2** or **9.7.3** using the values of r_y and v appropriate to the section where the limiting moment of resistance is to be derived,

where

$$\rho_f = \frac{\text{minimum total area of two flanges at any section in } \ell_w}{\text{total area of two flanges at the section being considered}},$$

ℓ_w is the half-wavelength of buckling of the compression flange as defined in **9.8**.

NOTE The above expression is not applicable to beams with U-frame restraints.

9.7.5 ***Other cases and alternative method***

For cases not covered by **9.7.2**, **9.7.3** or **9.7.4**, or as an alternative, λ_{LT} for overall lateral buckling may be taken as:

$$\lambda_{LT} = \sqrt{\frac{\pi^2 E Z_{pe}}{M_{cr}}}$$

where

Z_{pe} is as defined in **9.7.1**;

NOTE 1 For non-compact sections, Z_{pe} need not be calculated explicitly since (according to **9.7.1**) it may be replaced by M_{pe}/σ_{yc} and in the subsequent application of **9.8**, M_{pe} will cancel.

M_{cr} is the maximum bending moment at which, under the given pattern of loading, the beam reaches its theoretical elastic buckling condition as determined by an elastic analysis.

NOTE 2 In the absence of elastic buckling analysis, suitable formulae for the calculation of M_{cr} in some commonly occurring situations may be found as follows:

a) for pure lateral torsional buckling, in DD ENV 1993-1-1:1992, **F.1**;

b) for lateral distortional buckling (e.g. such as may occur in the compression flanges of continuous composite beams in hogging moment regions), in DD ENV 1994-1-1:1994, **B.1.2** to **B.1.4**.

9.8 Limiting moment of resistance

The limiting moment of resistance, M_R, should be obtained from Figure 11a) for beams fabricated by welding (excluding local welding of stiffeners) or Figure 11b) for all other sections (including stress relieved welded sections) according to the value of:

$$\lambda_{LT} \sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult}}{M_{pe}}\right)}$$

where

λ_{LT}	is obtained from **9.7**;
M_{ult}	is the moment of resistance of the cross-section if lateral-torsional buckling is prevented, i.e.: $M_{ult} = M_{pe}$ for compact sections; M_{ult} = the least of $Z_{xc}\sigma_{yc}$, $Z_{xt}\sigma_{yt}$ or $Z_{xw}\sigma_{yw}$ for non-compact sections;
M_{pe}, Z_{xc}, Z_{xt}, Z_{xw}	are as defined in **9.7.1**;
σ_{yc}	is the nominal yield stress value, as defined in **9.3.1**, for the compression flange material except that, where gross section properties have been used in accordance with note 2 of **9.4.2.4**, σ_{yc} should be taken as $A_e \Sigma b_e / AB_f$ multiplied by this nominal yield stress value;
A_e, B_f, Σb_e	are as defined in **9.4.2.4**;
A	is the gross cross-sectional area of the flange;
σ_{yt}	is the nominal yield stress of the tension flange material as defined in **6.2**;
σ_{yw}	is the nominal yield stress of the web material as defined in **6.2**;
ℓ_w	is the half-wavelength of buckling. Where intermediate restraints are fully effective, in accordance with **9.6.4.1.1.1**, $\ell_w = \ell_e$. Where there are no intermediate restraints, or the intermediate restraints are not fully effective, ℓ_w is determined by taking L/ℓ_w as the next integer below L/ℓ_e, but not less than unity;
ℓ_e	is the effective length defined in **9.6**;
L	is the distance between the restraints at the supports of the span of the simply supported or continuous beam under consideration.

NOTE For beams without intermediate restraints in the span ℓ_e/ℓ_w should be taken as 1.

9.9 Beams without longitudinal stiffeners

9.9.1 *Bending resistance*

9.9.1.1 *General*

Beams should be designed in accordance with **9.9.1.2** except for beams with flanges curved in elevation, which should be designed in accordance with **9.10** and **9.11**. Beams constructed in stages in which the loading and section properties change should be in accordance with **9.9.5**. Effects due to differential temperature and concrete shrinkage should be taken into account in accordance with **9.9.7**. Unsymmetrical beams should be additionally checked for the serviceability limit state in accordance with **9.9.8**.

9.9.1.2 *Compact and non-compact sections*

The bending resistance M_D of a beam should be taken as:

$$M_D = \frac{M_R}{\gamma_m \gamma_{f3}}$$

where

M_R is the limiting moment of resistance derived in **9.8**.

9.9.2 *Shear resistance*

9.9.2.1 *General*

The shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners should be determined in accordance with **9.9.2.2** provided that:

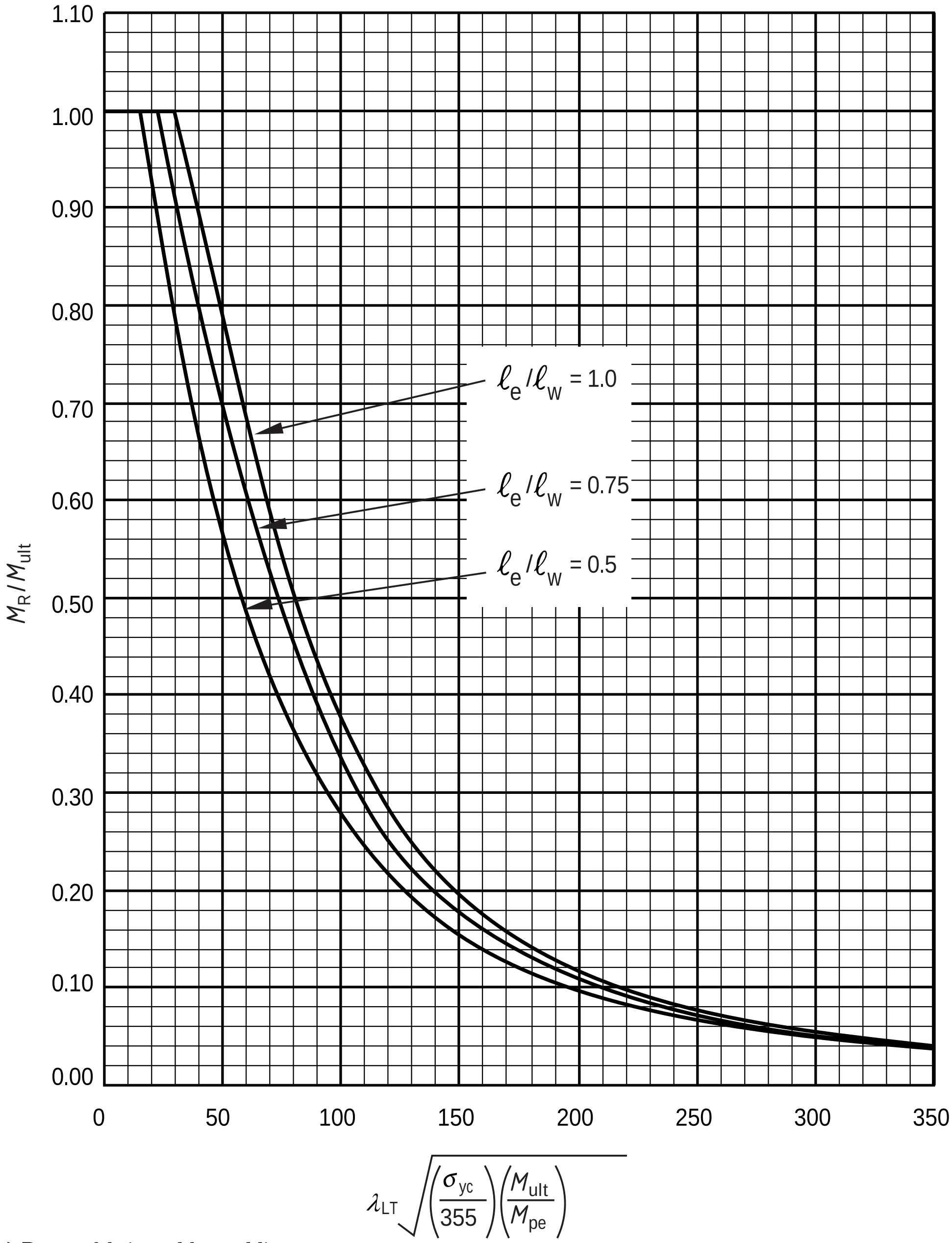
a) there are no longitudinal stiffeners on the web or compression flange;

b) the web panel considered has no openings other than those within the limits set out in **9.3.3.2**a), b) or c);

c) the provisions of **9.9.4** and **10.6** are met if the beam is subjected to axial load;

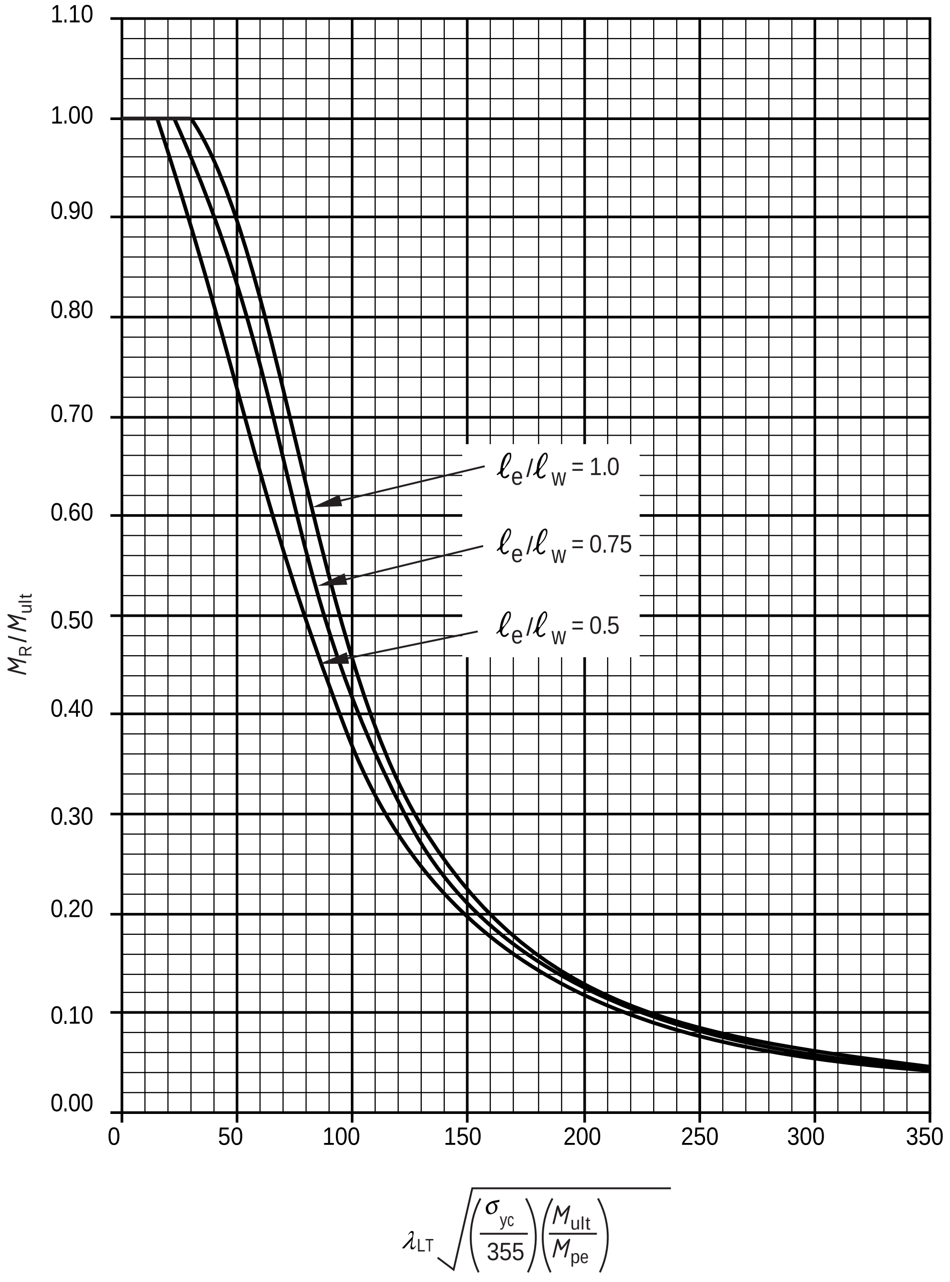
d) the flanges are parallel and straight in elevation.

Web panels which do not meet these conditions should be designed in accordance with **9.11**.



a) Beams fabricated by welding

Figure 11 — Limiting moment of resistance M_R



b) All other sections

Figure 11 — Limiting moment of resistance M_R (*continued*)

9.9.2.2 *Shear resistance under pure shear*

The shear resistance V_D of a web panel under pure shear should be taken as:

$$V_D = \left[\frac{t_w\left(d_w - h_h\right)}{\gamma_m \gamma_{f3}}\right]\tau_\ell$$

where

t_w is the thickness of the web;

d_w $= D$, the overall depth of a rolled section, or is the depth of the web measured clear between flanges of a fabricated section;

h_h is the height of the largest hole or cut-out, if any, within the panel being considered, but in the case of beams without intermediate transverse stiffeners the hole or cut-out may be ignored at sections further than $1.5h_h$ longitudinally from the edge of the hole;

τ_ℓ is the limiting shear strength of the web panel determined from Figures 12 to 18 corresponding to the values of τ_y, φ, m_{fw} and the slenderness ratio λ given by:

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

d_{we} is the depth of web clear between flange plates for welded sections, or the depth of web between toes of angles connecting the web to the flanges for riveted/bolted construction, or is the depth of web clear of root fillets for rolled sections;

τ_y $= \frac{\sigma_{yw}}{\sqrt{3}}$

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

φ $= \frac{a}{d_{we}}$, the aspect ratio of the panel;

a is the clear length of panel between transverse stiffeners;

m_{fw} $= \frac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} d_{we}^2 t_w}$ taking the smaller value at the top or the bottom flanges and ignoring any concrete;

b_{fe} is the smallest of:

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$; or

b) the distance from the mid-plane of the web to the nearer edge of the flange; i.e. taken as zero if there is no flange outstand; or

c) if there are two or more webs, half the clear distance between webs;

t_f is the flange plate thickness;

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

NOTE If the value of $\frac{\tau_\ell}{\tau_y}$ from Figures 12 to 18 is less than the shear coefficient K_q for an unrestrained panel (see Figure 23), the value of this ratio may be taken as K_q.

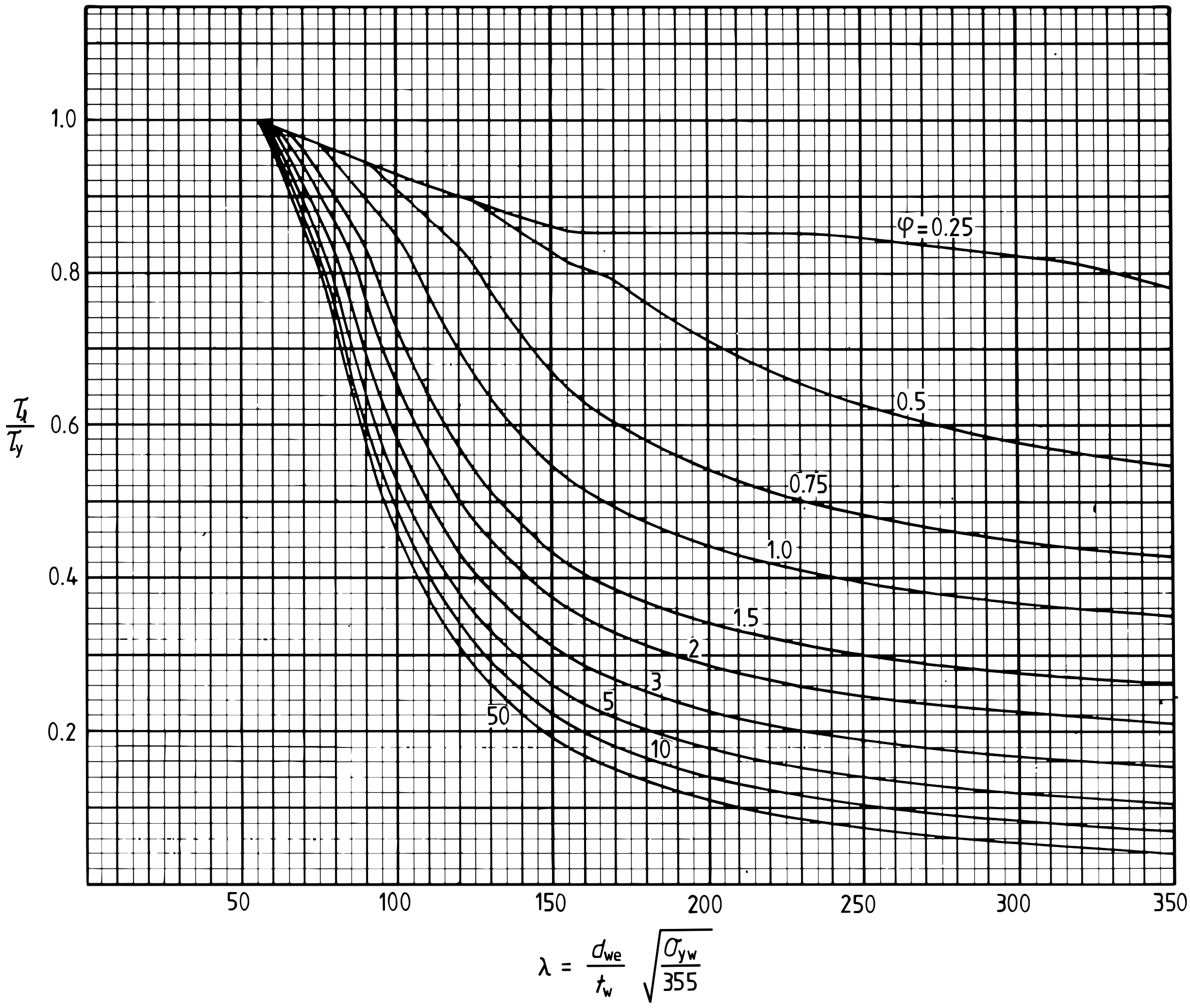
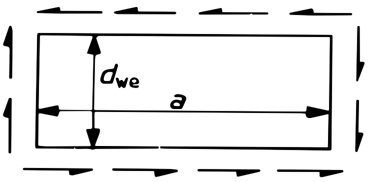


Figure 12 — Limiting shear strength τ_ℓ for $m_{fw} = 0$

Notes to Figures 12 to 18.

NOTE 1 $\varphi = \frac{a}{d_{we}}$ shown as follows:



NOTE 2 For definitions of a, d_{we}, t_w, σ_{yw}, τ_y and m_{fw}, see **9.9.2.2**.

NOTE 3 Values of τ_ℓ/τ_y above 1.0, plotted as dotted lines, are only to be used for interpolation purposes.

NOTE 4 For basis of curves, see **G.9**.

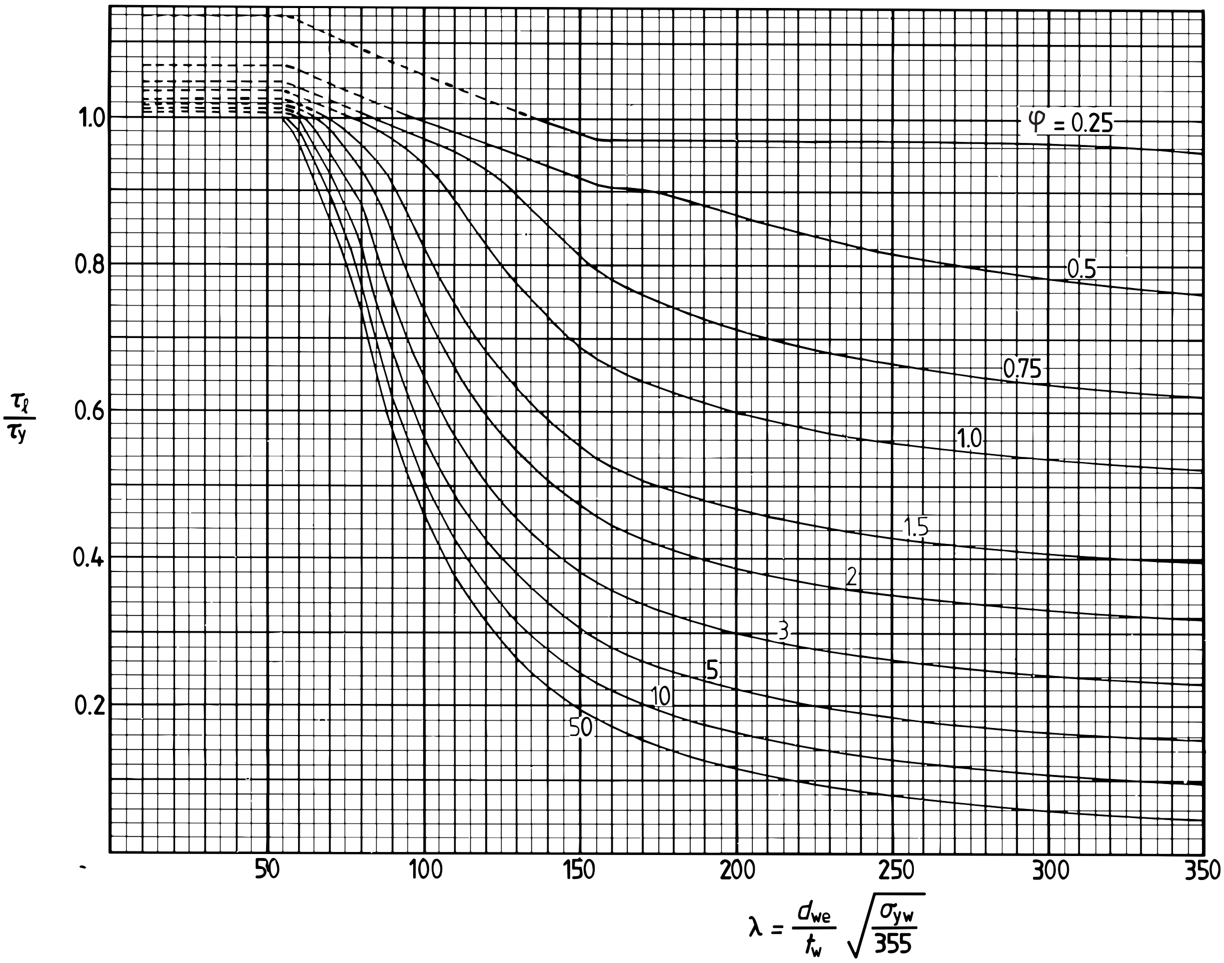


Figure 13 — Limiting shear strength τ_ℓ for m_{fw} = 0.005

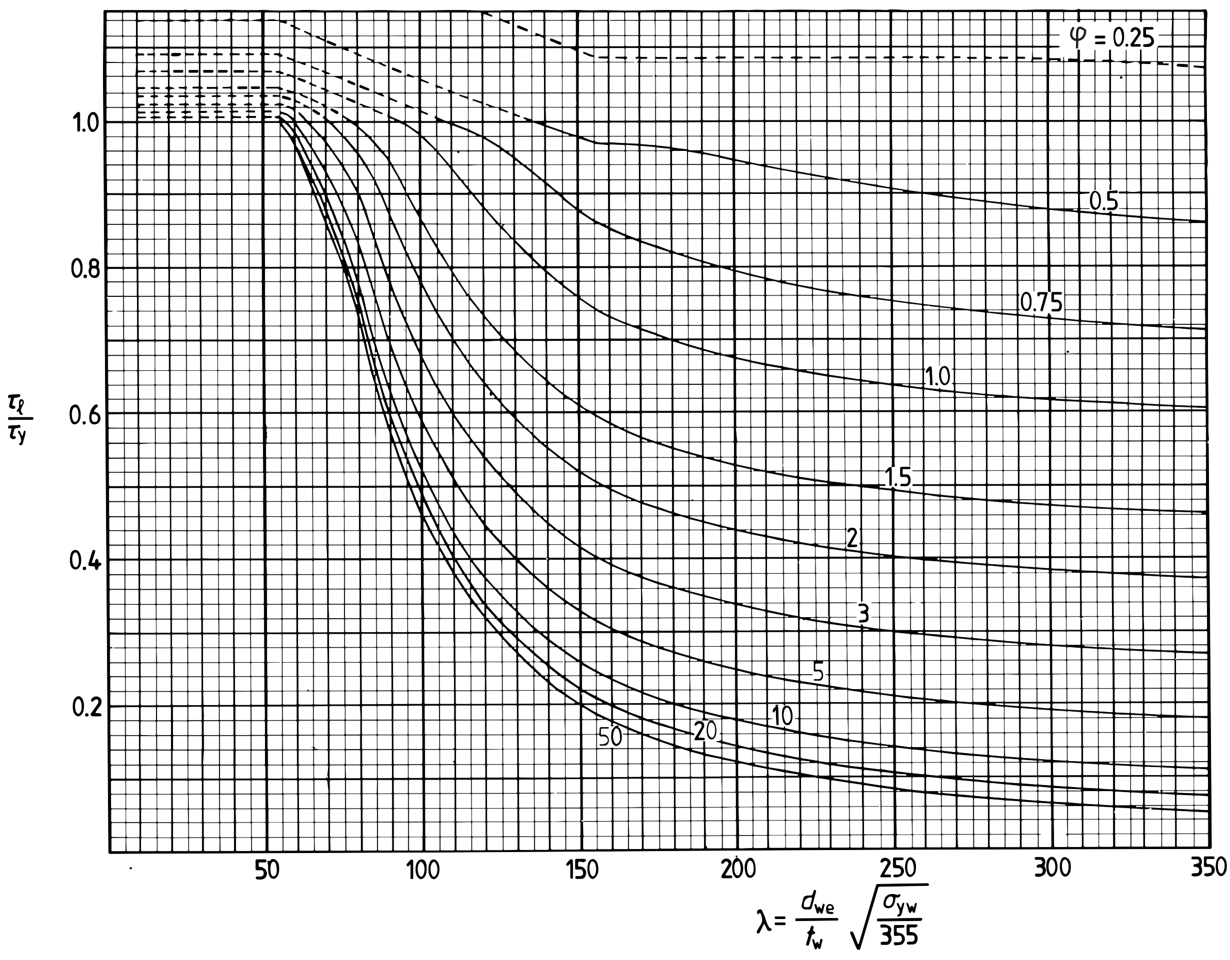


Figure 14 — Limiting shear strength τ_ℓ for m_{fw} = 0.010

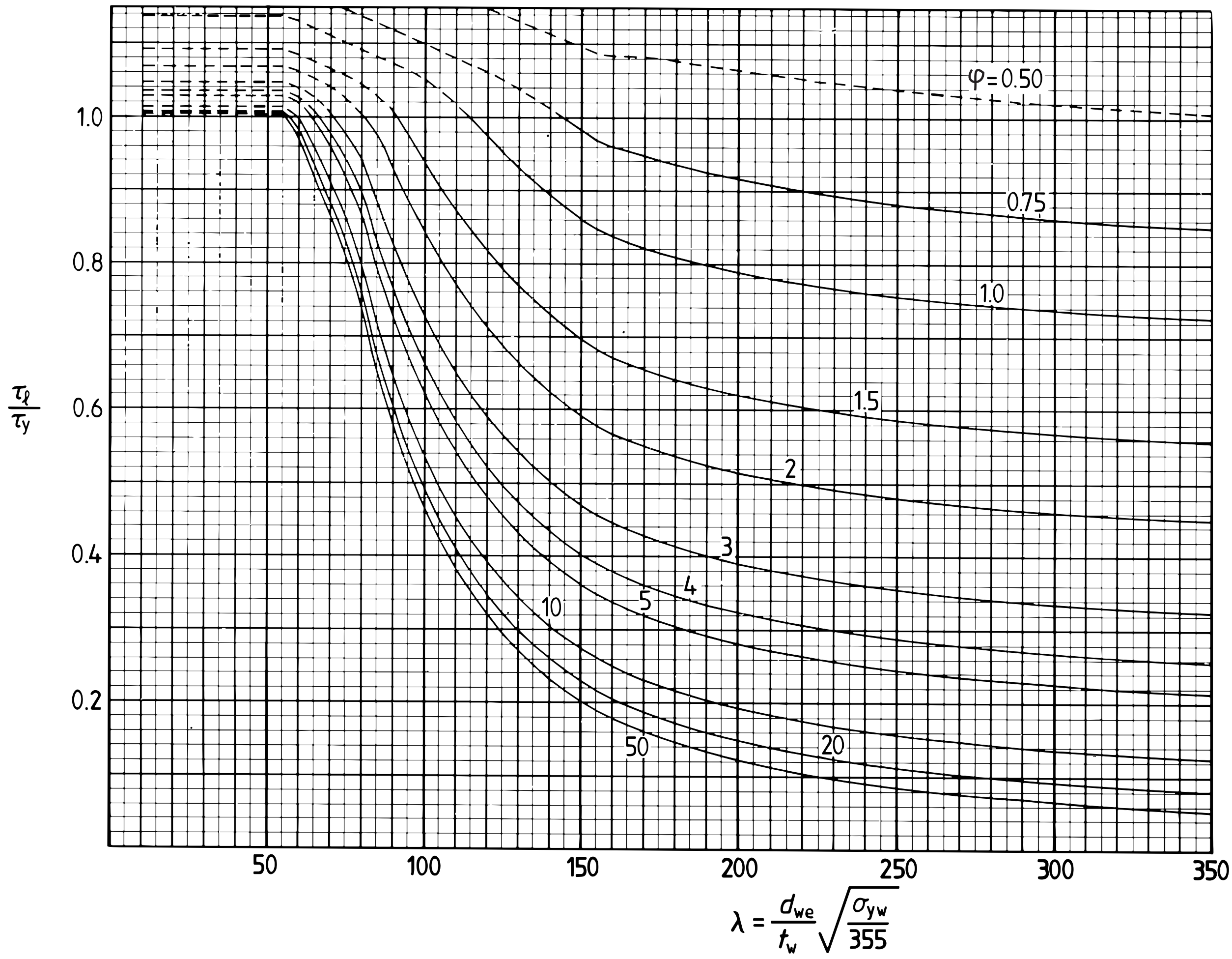


Figure 15 — Limiting shear strength τ_ℓ for m_{fw} = 0.020

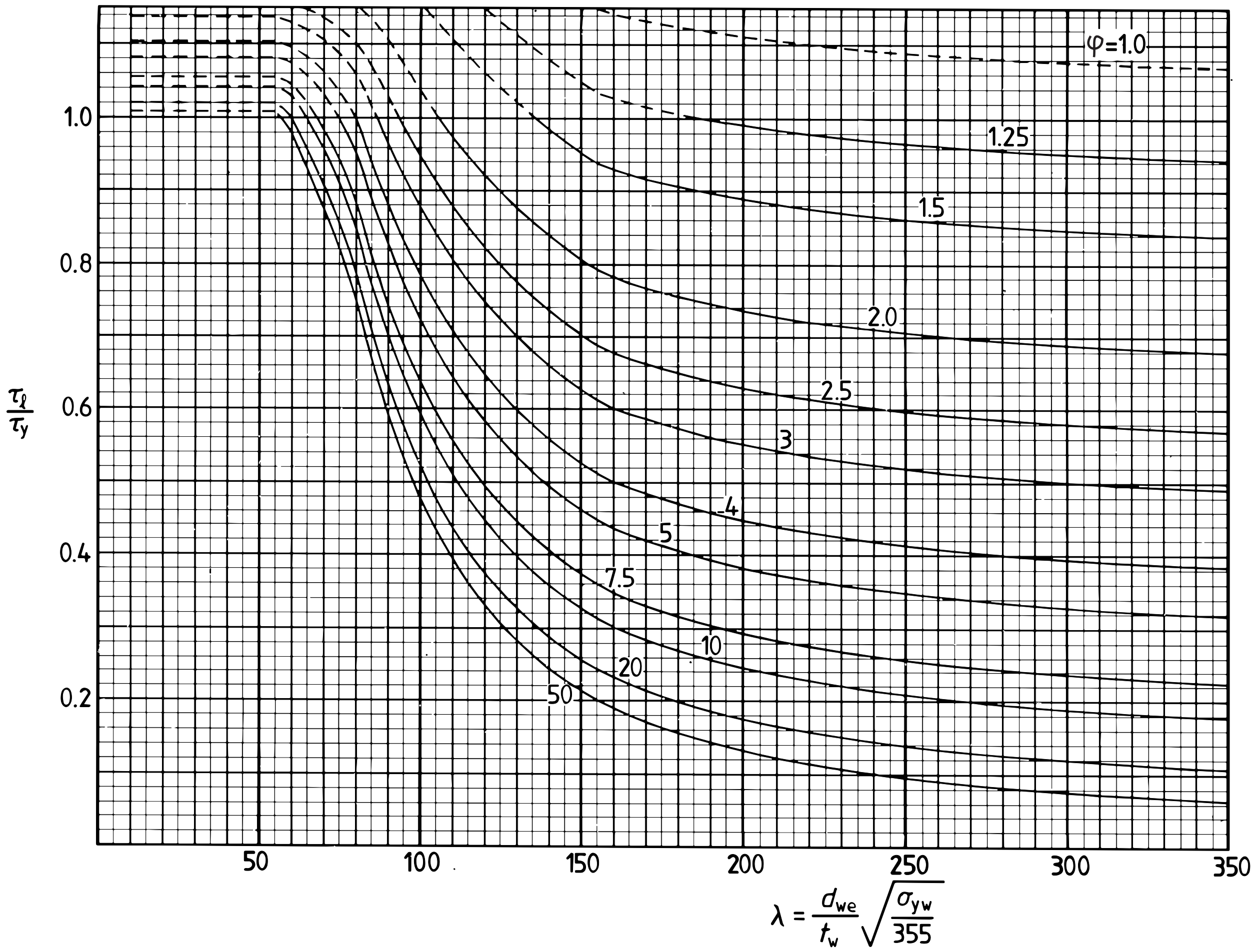


Figure 16 — Limiting shear strength τ_ℓ for m_{fw} = 0.060

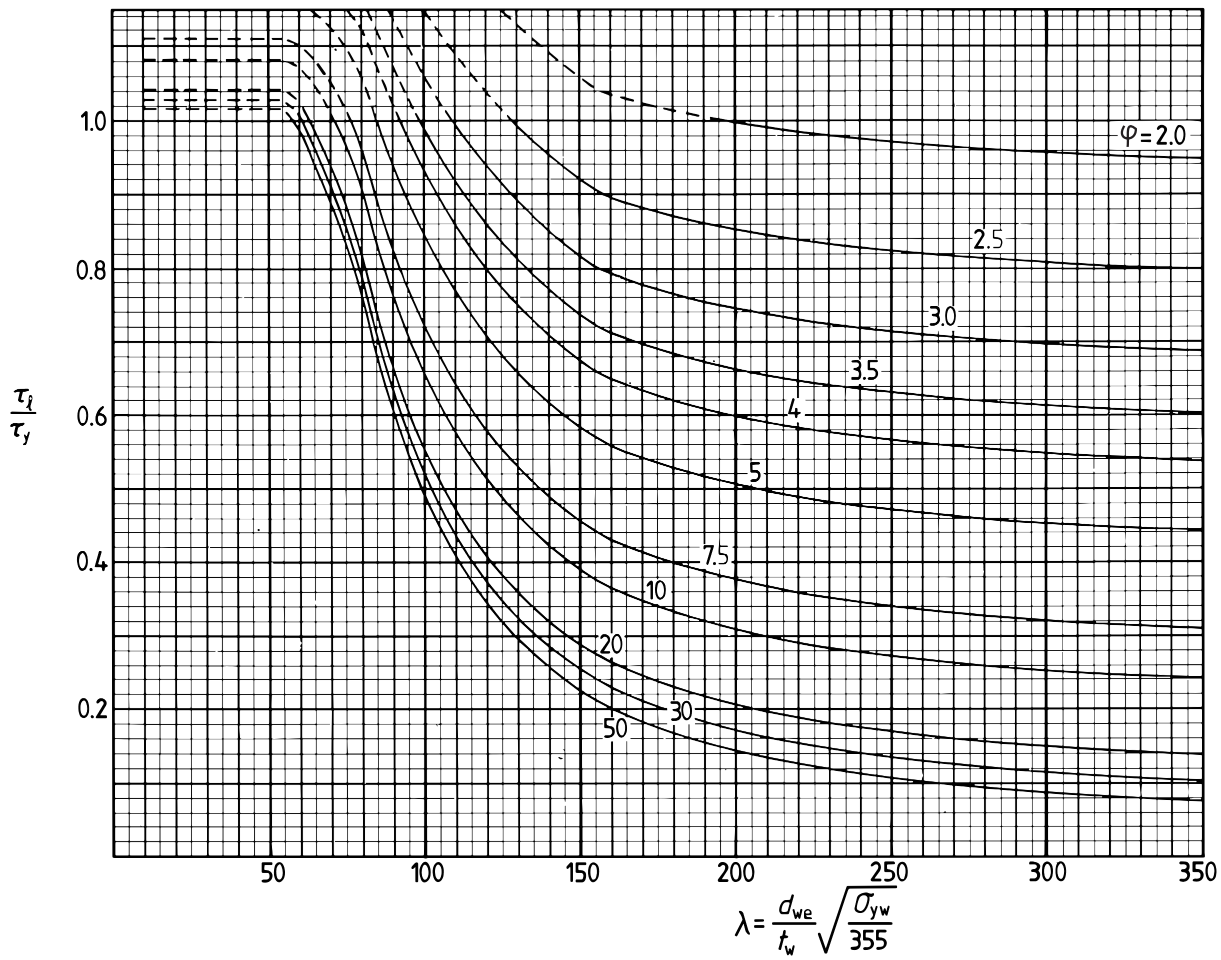


Figure 17 — Limiting shear strength τ_ℓ for m_{fw} = 0.120

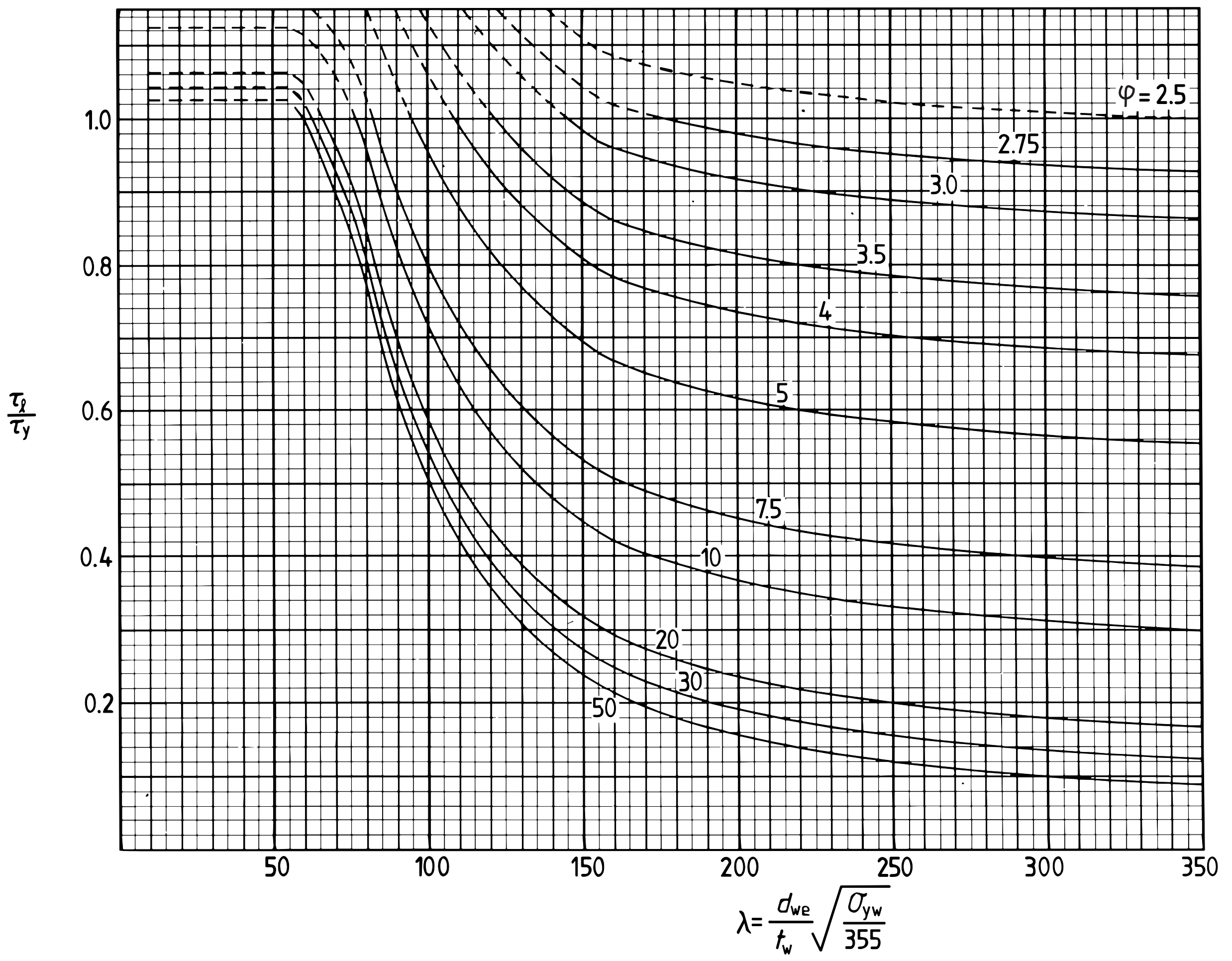


Figure 18 — Limiting shear strength τ_ℓ for m_{fw} = 0.180

9.9.3 *Combined bending and shear*

9.9.3.1 *Webs with intermediate transverse stiffeners*

Beams should be in accordance with the following:

a) $V \leq V_D$;

b) $M \leq M_D$;

c) if $M > M_f$, then $\frac{M}{M_D} + \left(1 - \frac{M_f}{M_D}\right)\left(\frac{2V}{V_R} - 1\right) \leq 1$;

d) if $V > V_R$, then $\frac{V}{V_D} + \left(1 - \frac{V_R}{V_D}\right)\left(\frac{2M}{M_f} - 1\right) \leq 1$;

For all rolled I- and channel sections of grade S275 and grade S355 steel in accordance with BS 4-1, and for other beams for which $V_D = V_R$, the conditions given in c) and d) are reduced to:

if $M > M_f$, then $\frac{M}{M_D} + \left(1 - \frac{M_f}{M_D}\right)\left(\frac{2V}{V_D} - 1\right) \leq 1$

where

V is the maximum shear force in the panel;

V_D is the shear capacity of the panel under pure shear determined in accordance with **9.9.2.2**;

V_R is the value of V_D obtained by taking m_{fw} = 0 when applying **9.9.2.2**;

M is the maximum bending moment within the length of the panel;

M_D is the bending resistance of the beam determined in accordance with **9.9.1**;

M_f	$= \dfrac{F_f d_f}{\gamma_m \gamma_{f3}}$ but not greater than M_D;
d_f	is the distance between the centroids of the two flanges; for a composite flange the distance should be measured from the centroid of the transformed flange section;
F_f	$= \sigma_f A_{fe}$, the limiting force in the flange, to be taken as the lower value for the two flanges;
σ_f	= (for the tension flange) σ_{yt} the nominal yield stress, as defined in **6.2**; or
	= (for the compression flange) the lesser of σ_{yc}, the nominal yield stress value as defined in **9.3.1**, and M_R/Z_{xc};
M_R	is as defined in **9.8**;
Z_{xc}	is as defined in **9.7.1**;
A_{fe}	is the area of the effective flange section derived in accordance with **9.4.2**.

NOTE 1 Bending moments up to a value of M_D can be resisted by a beam if the shear force V is less than $0.5V_R$.

NOTE 2 Shear forces up to a value V_D can be resisted by a beam if the bending moment M is less than $0.5M_f$.

9.9.3.2 *Webs with transverse stiffeners at the supports only*

For a web having transverse stiffeners at support positions only, the provisions of **9.9.3.1** should be applied at all sections of the beam with V and M defined as follows:

V	is the shear force at any section of the beam;
M	is the co-existent bending moment at the same section of the beam.

9.9.4 *Combined bending and axial load*

9.9.4.1 *Yielding of beam*

All points at all sections of a beam subjected to a combined bending and axial load should be such that:

$$\frac{P}{A_e} \pm \frac{M_x}{Z_x} \pm \frac{M_y}{Z_y} \leq \frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

P	is the axial load in the beam at the section under consideration;
M_x, M_y	are the co-incident bending moments about the X-X and Y-Y axes, respectively;
A_e	is the effective area of the beam, calculated at the section under consideration in accordance with **10.5** or **11.3**, as appropriate;
Z_x, Z_y	are the elastic moduli of the effective beam section about the X-X and Y-Y axes, respectively, at the section under consideration, derived from **9.4.2**;
σ_y	is the nominal yield stress value, as defined in **9.3.1**, for the part of the section under consideration.

9.9.4.2 *Buckling of a beam*

A beam subjected to combined bending and axial compression should be such that:

$$\frac{P_{max.}}{P_D} + \frac{M_{x,\,max.}}{M_{Dx}} + \frac{M_{y,\,max.}}{M_{Dy}} \leq 1$$

where

$P_{max.}$	is the maximum axial load within the middle third of the length of the beam between points of restraint;
$M_{x,\,max.}$, $M_{y,\,max.}$	are the maximum bending moments about the X-X and Y-Y axes, respectively (see Figure 1), within the middle third of the length of the beam between the points of restraint;
P_D	is the axial resistance derived in accordance with **10.6.1**;
M_{Dx}, M_{Dy}	are the corresponding bending resistances of the beam, determined in accordance with **9.9.1**.

A beam subjected to combined bending and axial tension should also be in accordance with **11.5.2**.

NOTE When the bending capacity of the cross-section may be increased by the presence of an axial compression the interaction above may be conservative. The interaction may then be calculated from first principles.

9.9.4.3 *Compact and stocky members*

As an alternative to **9.9.4.1** and **9.9.4.2**, compact sections subjected to combined bending and axial compression may be designed in accordance with **10.6.3**, provided that they also meet the provisions for slenderness for stocky members contained therein.

9.9.5 ***Beam built in several stages***

9.9.5.1 *General*

When the cross-section of a beam and the applied loading increase by stages, e.g. a steel section initially carrying self-weight and the weight of a concrete deck but acting compositely for subsequently applied loads, a check for adequacy should be made for each stage of construction as well as in service.

9.9.5.2 *Compact sections at the ultimate limit state*

For compact sections, as defined in **9.3.7**, at the ultimate limit state, the entire load at the stage considered may be assumed to act on the cross-section appropriate to that stage.

9.9.5.3 *Non-compact sections at the ultimate limit state and all sections at the serviceability limit state*

For non-compact sections, as defined in **9.3.7**, at the ultimate limit state and for all sections at the serviceability limit state, the stresses appropriate to the cross-section and the loading at each stage of construction should be calculated separately for bending about each axis and for axial load.

9.9.5.4 *Beam in pure bending*

The total bending moment at the ultimate limit state at any stage should not exceed the bending resistance M_D, as defined in **9.9.1.2**. Additionally, for non-compact sections at the ultimate limit state, and for compact sections at the serviceability limit state, the total accumulated stress at any fibre should not exceed:

a) $\dfrac{\sigma_{yc}}{\gamma_m \gamma_{f3}}$ in compression;

b) $\dfrac{\sigma_{yt}}{\gamma_m \gamma_{f3}}$ in tension.

9.9.5.5 *Beam in combined bending and shear*

As well as satisfying the provisions of **9.9.5.4**, the interaction formulae in **9.9.3** should be satisfied at all stages. In applying these formulae, V_D, V_R, M_D and M_f should be taken appropriate to the cross-section at the stage under consideration. For compact sections, the loading is as given by **9.9.5.2**. For non-compact sections, the value of M should be taken as the value $\sigma_{xx}Z_x$ where σ_{xx} is the total accumulated stress at the fibre that is most critical in determining M_{ult} for the stage being considered, M_{ult} is as given by **9.8** and Z_x is the elastic modulus of the effective section at the same fibre.

9.9.5.6 *Beam in combined bending and axial load*

The provisions of **9.9.4** should be satisfied at any stage.

In applying the formula in **9.9.4.1** to compact sections at the ultimate limit state, the ratios on the left-hand side should be calculated using the loading given in **9.9.5.2**.

In applying the formula in **9.9.4.1** to non-compact sections at the ultimate limit state, or to all sections at the serviceability limit state, the ratios on the left-hand side should be taken as the values accumulated at all stages up to and including the present stage.

In applying the formula in **9.9.4.2**, P_D, M_{Dx} and M_{Dy} should be taken appropriate to the cross-section at the stage under consideration. For beams that are of compact section, the loading is as given by **9.9.5.2**. For beams that are of non-compact section, the values of $M_{x,\ max.}$ and $M_{y,\ max.}$ should be taken as $\sigma_{xx}Z_x$ and $\sigma_{yy}Z_y$ respectively, for one fibre of the cross-section, where σ_{xx} and σ_{yy} are the total accumulated stresses at the fibre and Z_x and Z_y are the elastic moduli of the effective section at the fibre. The chosen fibre should be such that the left-hand side of the formula in **9.9.4.2** is a maximum.

9.9.6 ***Webs subjected to in-plane patch loading***

The effects of in-plane patch loading on a longitudinal edge of a web should be taken into account if the transverse stress σ_2 in the web plate caused by this loading is greater than:

$$3\sigma_{yw} \frac{t_w}{\sqrt{a_1 d_{we}}} \sqrt{\frac{355}{\sigma_{yw}}}$$

where

a_1 is the width of the patch loading along the span of the beam as dispersed to the edge of the web (see Figure 6);

t_w is the web plate thickness;

d_{we} is the depth of the web as defined in **9.9.2.2**;

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

In this case, either a transverse stiffener designed in accordance with the relevant provisions of **9.14** should be provided in the web below the patch load, or the web without transverse stiffeners should be checked in accordance with annex D.

9.9.7 ***Differential temperature and concrete shrinkage***

When, in accordance with **9.2.1** or **9.2.3**, differential temperature and shrinkage effects are to be taken into account, the effects should be separated into the following parts:

a) stresses forming the internal stress distribution through the section, ignoring any continuity over supports;

b) bending moments and shears due to requirements for continuity over supports in a continuous beam.

For the strength checks contained in **9.9.1** to **9.9.4** the values of bending moment and shear from b) should be combined with other load effects as appropriate.

For the serviceability limit state, the stresses calculated from a) should be added to the stresses due to load effects [including the moments from b) taking into account the effect of shear lag at appropriate points on the section]. The resultant total stresses should not exceed:

1) $\frac{\sigma_{yc}}{\gamma_m \gamma_{f3}}$ in compression;

2) $\frac{\sigma_{yt}}{\gamma_m \gamma_{f3}}$ in tension.

9.9.8 ***Serviceability check for unsymmetric cross-sections***

The smaller flange of unsymmetric beams designed as compact should be checked for the serviceability limit state, treating the beams as non-compact.

9.10 Flanges in beams with longitudinal stiffeners in the cross-section

9.10.1 ***Strength of unstiffened flanges***

9.10.1.1 *Flanges straight in elevation*

The stresses in the extreme fibres of a beam with longitudinal stiffeners on the web, including any redistribution of stresses from the web, should not exceed:

a) $\frac{M_R}{Z_{xc} \gamma_m \gamma_{f3}}$ in compression;

b) $\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}$ in tension;

where

M_R is as defined in **9.8**;

Z_{xc} is as defined in **9.7.1**;

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

9.10.1.2 *Flanges curved in elevation*

Flanges curved in elevation should be in accordance with **9.10.1.1**. Additionally the stresses in the flange plate including those due to flange curvature (as calculated in accordance with **9.5.7.1**) should be in accordance with **9.10.2.1**.

9.10.2 *Strength of stiffened flanges*

9.10.2.1 *Yielding of the flange plate*

The design of the flange plate should satisfy the following yield criterion:

$$\sigma_f^2 + \sigma_2^2 - \sigma_f\sigma_2 + 3\tau^2 \leq \left(\frac{\sigma_{yf}}{\gamma_m\gamma_{f3}}\right)^2$$

where

σ_f is the longitudinal stress at the mid-plane of the flange plate including any redistribution of stresses from the web, treated as positive when compressive;

σ_2 is the co-existent in-plane transverse stress at the mid-plane of the flange plate, treated as positive when compressive, due to the bending of cross beams or diaphragms, or due to curvature (see **9.5.7.1**);

τ $= \tau_1 + 0.5\tau_2$;

τ_1 is the in-plane shear stress in the flange plate due to torsion on a box beam;

τ_2 is the shear stress in the flange plate at the junction with the web of the beam due to the shear force on the beam;

σ_{yf} is the nominal yield stress, as defined in **6.2**, for the flange plate material.

9.10.2.2 *Effective section for longitudinal flange stiffeners*

The effective section of a longitudinal stiffener should be taken as the stiffener combined with a width of flange plate equal to $0.5K_cb$ on each side of the stiffener,

where

K_c is obtained from Figure 5 (in accordance with **9.4.2.4**) for a compression flange, or is equal to 1.0 for a tension flange;

b is the spacing of longitudinal stiffeners.

9.10.2.3 *Strength of longitudinal flange stiffeners*

The design of discontinuous and continuous longitudinal stiffeners should be such that:

a) $\sigma_a + 2.5\tau_1 k_{s1} < \dfrac{k_{\ell 1}\sigma_{ys}}{\gamma_m\gamma_{f3}}$

b) $\sigma_a + 2.5\tau_1 k_{s2} < \dfrac{k_{\ell 2}\sigma_{ye}}{\gamma_m\gamma_{f3}}$

where

σ_a is the longitudinal stress including any redistribution of stresses from the web, positive when compressive, at the centroid of the effective section of the stiffener. In designing any part of a stiffener which has not been included for stress analysis (see **9.10.5**), σ_a should be taken as that calculated on the assumption that the stiffener does not contribute to the effective section for bending analysis;

τ_1 is the in-plane shear stress in the flange plate due to torsion on the beam, always to be taken as positive;

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material;

σ_{ye} $= \sqrt{\sigma_{yf}^2 - 3\tau^2\gamma_m^2\gamma_{f3}^2}$;

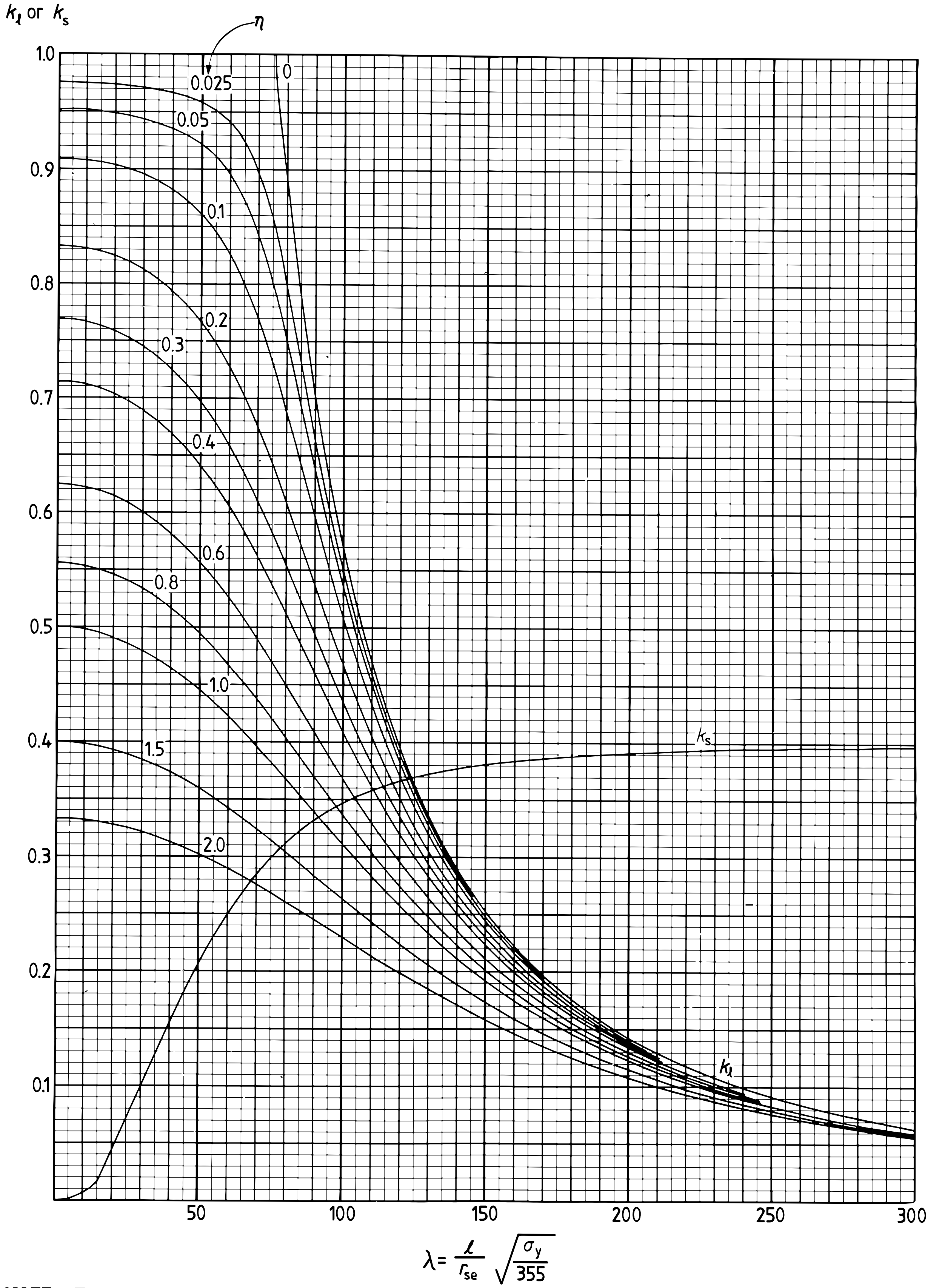
σ_{yf} is the nominal yield stress value, as defined in **9.3.1**, for the flange plate material;

τ is as defined in **9.10.2.1**;

k_{s1}, k_{s2} are coefficients obtained from Figure 19;

$k_{\ell 1}$, $k_{\ell 2}$ are values of the reduction factor k_ℓ obtained from Figure 19 when the left-hand side of the relevant expression a) or b) is positive, or taken as 1.0 when the left-hand side of the relevant expression a) or b) is negative;

γ_m = 1.2 when the left-hand side of the relevant expression a) or b) is positive;
= 1.05 when the left-hand side of the relevant expression a) or b) is negative.



NOTE 1 For appropriate values of η, σ_y and ℓ to be used in obtaining k_ℓ and k_s, see **9.10.2.3**.

NOTE 2 For basis of curves, see **G.10**.

Figure 19 — Parameters for the design of longitudinal flange stiffeners

When using Figure 19:

$$\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}} \text{ when obtaining } k_{\ell 1} \text{ and } k_{s1}$$

$$\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ye}}{355}} \text{ when obtaining } k_{\ell 2} \text{ and } k_{s2}$$

$$\eta = \frac{y_o \Delta}{r_{se}^2} \text{ when obtaining } k_{\ell 1}$$

$$\eta = \frac{y_z \Delta}{r_{se}^2} \text{ when obtaining } k_{\ell 2}$$

where

ℓ is the spacing of cross beams and/or diaphragms which restrain longitudinal stiffeners (for flanges not stiffened transversely see **9.10.4**);

r_{se} is the radius of gyration of the effective section of a longitudinal stiffener about the centroidal axis parallel to the flange plate;

y_o is the distance from the centroid of the effective stiffener section to the point on the stiffener furthest from the plate;

y_z is the distance from the centroid of the effective stiffener section to the mid-plane of the flange plate;

Δ $= \frac{\ell}{625} + \frac{\zeta r_{se}^2}{y_{Bs}} + \frac{e_f}{2}$

y_{Bs} is the distance from the centroid of the effective stiffener section to the neutral axis of the cross-section of the beam (determined in accordance with **9.4.2**);

NOTE The neutral axis occurs where the total longitudinal stress is zero.

ζ $= -(1 - 1.35\sigma_a/\sigma_E)$ when obtaining $k_{\ell 1}$

ζ $= (1 + 1.35\sigma_a/\sigma_E)$ when obtaining $k_{\ell 2}$

σ_E $= \pi^2 E/(\ell/r_{se})^2$;

e_f is the greatest offset of the flange plate from a straight line of length ℓ, due to specified camber or curvature.

NOTE In determining Δ, the signs of the three components should be considered. Where a component would give a relieving effect on the fibre being considered, it may conservatively be ignored.

9.10.2.4 *Longitudinally varying moment*

If the longitudinal stress in the flange varies within the length ℓ, the provisions of **9.10.2.1** should be satisfied by all sections within the length ℓ, and the provisions of **9.10.2.3** should be satisfied with σ_a taken at a point 0.4 ℓ from the end where the stress is greater.

9.10.3 *Stiffened flanges subjected to local bending*

9.10.3.1 *Strength*

Stiffened flanges subjected to bending due to wheel or other local loads in addition to in-plane stresses should satisfy the provisions of **9.10.3.2** and **9.10.3.3**.

9.10.3.2 *Ultimate limit state*

Provided that the provisions of **9.10.3.3** are satisfied, no account need be taken of local bending stresses when checking a stiffened flange at the ultimate limit state. Under in-plane forces the provisions of **9.10.2.1** to **9.10.2.4** should be satisfied.

9.10.3.3 *Serviceability limit state*

9.10.3.3.1 *Flange plate*

The design of the flange plate should satisfy the following yield criterion at all sections:

$$(\sigma_{fz} + \sigma_f)^2 + (\sigma_2 + \sigma_{2b})^2 - (\sigma_{fz} + \sigma_f)(\sigma_2 + \sigma_{2b}) + 3\tau^2 \leq \left(\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}\right)^2$$

where

σ_f, σ_2, τ are as defined in **9.10.2.1** due to global effects;

σ_{fz} is the stress at the mid-plane of the flange plate due to local bending of the effective stiffener section spanning between transverse members;

σ_{2b} is the stress due to local bending at the extreme fibre of the flange plate spanning between longitudinal stiffeners and transverse membrane action;

σ_{yf} is the nominal yield stress value, as defined in **9.3.1**, for the flange plate material.

9.10.3.3.2 *Longitudinal stiffeners*

The design of the longitudinal stiffeners should be such that, at all points in the region subjected to the local moments, the following provisions are satisfied for the serviceability limit state, under the combined effects of in-plane forces and local bending:

a) stresses in the stiffener due to local bending and in-plane forces should not exceed $\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$;

b) $\frac{\sigma_a + 2.5\tau_1 k_{s1}}{k_{\ell 1}\sigma_{ys}} + \frac{\sigma_{fo}}{\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$; and

c) $\frac{\sigma_a + 2.5\tau_1 k_{s2}}{k_{\ell 2}\sigma_{ye}} + \frac{\sigma_{fz}}{\sigma_{ye}} \leq \frac{1}{\gamma_m \gamma_{f3}}$

where

σ_{fo} is the stress due to local bending at the point on the stiffener furthest from the flange plate;

σ_{fz} is as defined in **9.10.3.3.1**;

σ_a, τ_1, σ_{ys}, σ_{ye} are as defined in **9.10.2.3**;

k_{s1}, k_{s2} are coefficients obtained from Figure 19;

$k_{\ell 1}$, $k_{\ell 2}$ are values of the reduction factor determined from Figure 19.

For zones of local sagging moment, i.e. causing local compressive stresses in the plate, the values of λ to be used in Figure 19 are given in **9.10.2.3**.

For zones of local hogging moment, i.e. causing local tensile stresses in the plate, the values of λ to be used in Figure 19 are given by:

$$\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ys} - \gamma_m \gamma_{f3}\sigma_{fo}}{355}} \text{ for } k_{s1} \text{ and } k_{\ell 1};$$

$$\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ye} - \gamma_m \gamma_{f3}\sigma_{fz}}{355}} \text{ for } k_{s2} \text{ and } k_{\ell 2};$$

where

ℓ, r_{se} are as defined in **9.10.2.3**.

9.10.4 ***Longitudinally stiffened flange not stiffened transversely***

The longitudinal stiffeners should satisfy the provisions of **9.10.2** and **9.10.3**, but with ℓ taken as ℓ_e, given by:

$$\ell_e = 1.5\left(\frac{B}{t_f}\right)^{0.75}\left[I_{se}(n+1)\right]^{0.25}$$

where

B is the total width of the stiffened flange between the main beam webs;

I_{se} is the second moment of area of the effective section of each longitudinal stiffener;

t_f is the flange plate thickness;

n is the number of longitudinal stiffeners in width B.

9.10.5 ***Curtailment of longitudinal flange stiffeners***

Where longitudinal stiffeners are curtailed, the stiffener should be extended beyond the theoretical cut-off point. The attachment of this extension is required to develop the load in the stiffener calculated at its theoretical cut-off point and the extension should be terminated and adequately connected to a transverse member or cross frame (see **9.4.2.6**).

9.11 Webs in beams with longitudinal stiffeners in the cross-section

9.11.1 ***General***

Webs should either be solid or have openings within the limits set out in **9.3.3**.

A "web panel" is defined as an area of web plate bounded on each transverse edge by a transverse stiffener or a diaphragm, and on each longitudinal edge either by a longitudinal stiffener or a flange of a beam.

An "outer panel" is defined as a web panel adjacent to a flange of a beam.

9.11.2 ***Strength***

The design of web panels should be such that, at all points on the panel, the yield criterion of **9.11.3** and the buckling criterion of **9.11.4** are both satisfied.

Longitudinal web stiffeners, if any, should satisfy the provisions of **9.11.5** and **9.11.6**.

Intermediate transverse stiffeners, if any, should satisfy the provisions of **9.13**.

9.11.3 ***Yielding of web panels***

The following condition should be satisfied at all points on the panel:

$$\sigma_{1e}^2 + \sigma_2^2 - \sigma_{1e}\sigma_2 + 3\tau^2 \leq \left(\frac{\sigma_{yw}}{\gamma_m \gamma_{f3}}\right)^2$$

where

σ_1, σ_2, σ_b, τ are the co-existent components of stress shown in Figure 20;

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

a) In the absence of transverse stresses in the panel ($\sigma_2 = 0$):

$$\sigma_{1e} = \sigma_1 + 0.77\sigma_b$$

where

σ_1 is the mean longitudinal stress on a cross-section of the panel after any assumed redistribution in accordance with **9.5.4**, considered positive if compressive;

σ_b is the maximum longitudinal stress due to in-plane bending of the individual panel after any assumed redistribution in accordance with **9.5.4**, considered positive if compressive;

τ is the average shear stress due to the applied shear force and, in a closed section, due to the applied torsional moment.

NOTE In this case it is only necessary to satisfy the above condition at all points along the longitudinal edges of the panel.

b) In the presence of transverse stresses σ_2 in the panel:

$$\sigma_{1e} = \sigma_1 + k\sigma_b$$

where

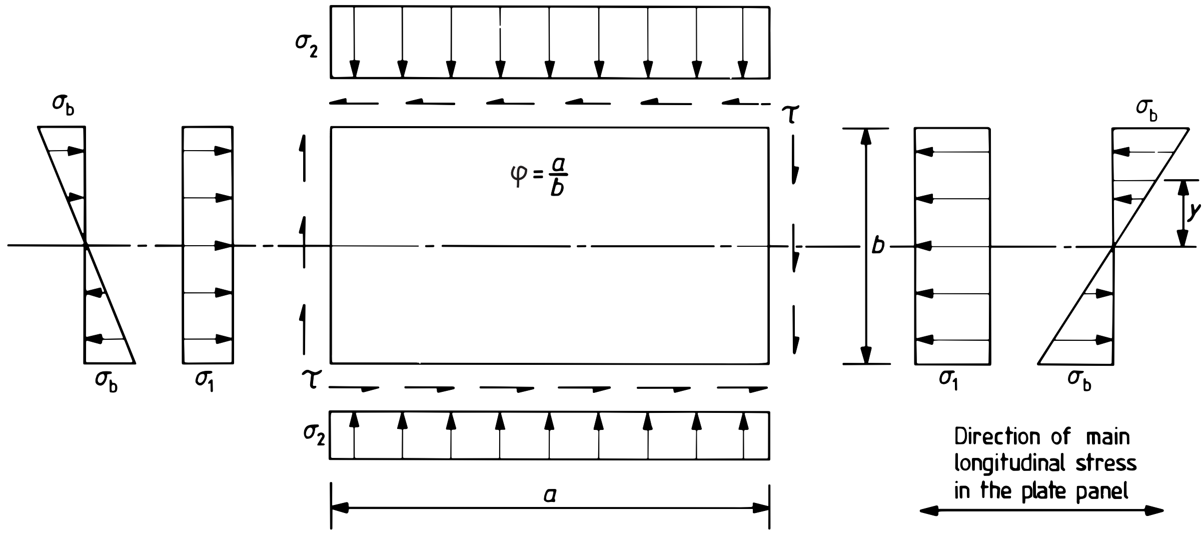
σ_2 is the tranverse stress, considered positive if compressive;

σ_1, τ, σ_b are as defined in a);

k = $2y/b$ or 0.77, whichever is smaller;

y is the perpendicular distance from the point being considered to the longitudinal centreline of the panel, to be taken always as positive;

b is the width of the panel (see Figure 20).



NOTE For transverse web stiffeners see **9.13.3.2**, for diaphragm plate panels see **9.17.6.5**.

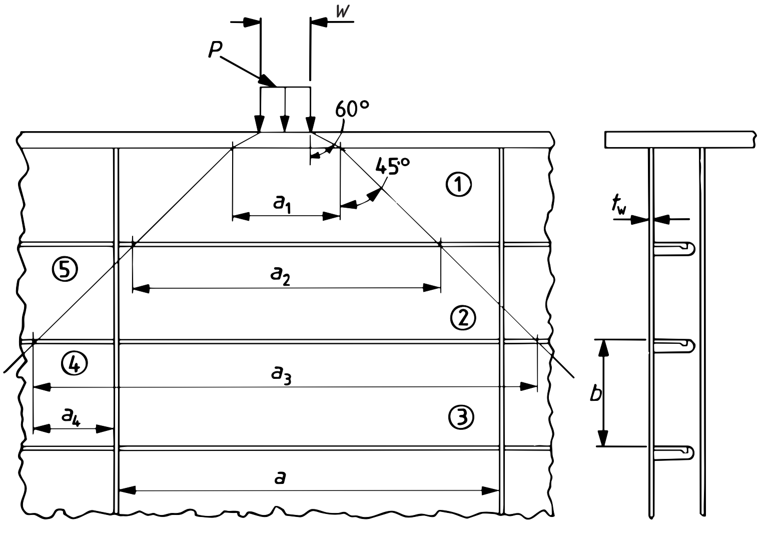
Figure 20 — Stresses on web panels

9.11.4 *Buckling of web panels*

9.11.4.1 *General*

In the case of webs subject to transverse stress as well as other in-plane stresses, the maximum intensity of transverse stress σ_2, acting over part of the length of a longitudinal edge, may conservatively be assumed to act over the whole length of the panel. Alternatively, the method set out in annex D may be used.

The transverse stress σ_2 in each panel of the web should be taken as that at the edge of the panel nearest to the load, calculated using the dispersion shown in Figure 21.



Panel	Transverse stress σ_2 to be used for each panel over length a	w_2 for use in D.2
①	$\frac{P}{a_1 t_w}$	a_1
②	$\frac{P}{a_2 t_w}$	a_2
③	$\frac{P}{a_3 t_w}$	a
④	$\frac{P}{a_3 t_w}$	a_4
⑤	0	0

a) Typical panels b) Treatment of panels

NOTE *P* is the component of the applied load in the plane of the web and may be compressive or tensile.

Figure 21 — Dispersal of load through a longitudinally stiffened web

9.11.4.2 *Restraint of web panels*

9.11.4.2.1 *General*

In order to calculate the buckling coefficients K_1, K_q, K_b and K_2 needed in **9.11.4.3**, the effective in-plane boundary restraint of the panel should be determined in accordance with **9.11.4.2.2** or **9.11.4.2.3** as appropriate.

Any panel not meeting the provisions given in **9.11.4.2.2** and **9.11.4.2.3** should be treated as unrestrained.

9.11.4.2.2 *Restraint for derivation of* K_1, K_q *and* K_b

All interior web panels (i.e. not adjacent to a flange) may be treated as restrained.

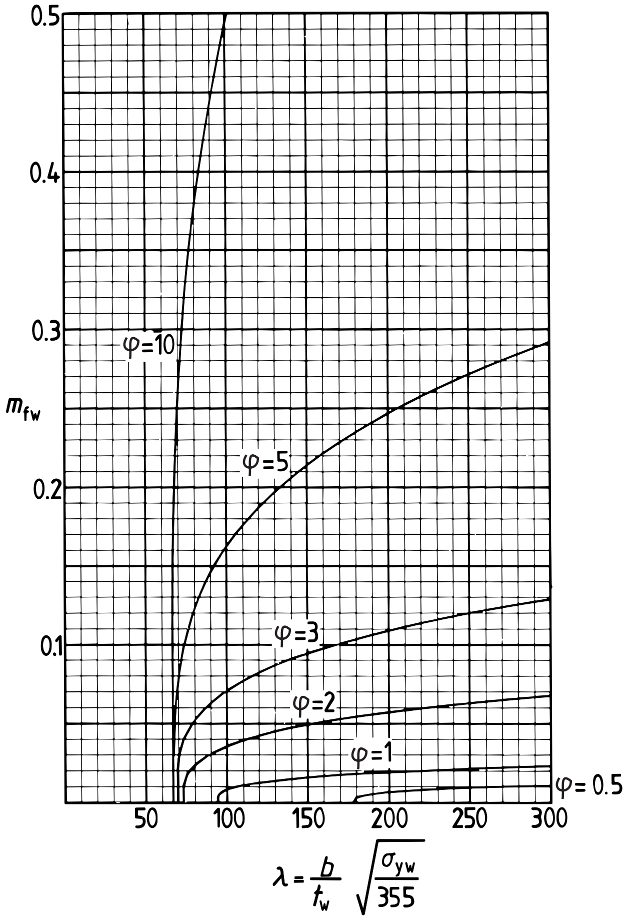
Any web panel adjacent to a flange may be treated as restrained provided that either:

a) its slenderness ratio λ is less than 24; or

b) the following conditions are satisfied:

1) $m_{fw} > \left[\dfrac{\sigma_{yf}^2}{\sigma_{yf}^2 - \gamma_m^2 \gamma_{f3}^2 \sigma_f^2}\right] \times [0.000\,25(\lambda - 24)]$ for $24 \leq \lambda \leq 84$, but λ taken as 84 for $\lambda > 84$ for this purpose only; and

2) if $\lambda > 66 + 28/\varphi^2$ then m_{fw} should be greater than the limiting value obtained from Figure 22;



NOTE 1 $\varphi = a/b$ (see Figure 20)

where

a is the dimension of the panel in the direction of main longitudinal stress;

b is the panel dimension normal to a.

NOTE 2 For basis of curves, see **G.11**.

Figure 22 — Minimum value of m_{fw} for outer panel restraint

where

λ $= \dfrac{b}{t_w}\sqrt{\dfrac{\sigma_{yw}}{355}}$

m_{fw} $= \dfrac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} b^2 t_w}$

t_w, b_{fe}, t_f, σ_{yf}, σ_{yw} are as defined in **9.9.2.2**;

φ is the aspect ratio a/b as shown in Figure 20;

a, b are the length and width of the panel, respectively (see Figure 20);

σ_f is the longitudinal stress in the flange plate, including any redistribution of stresses.

For a web without longitudinal stiffeners both flanges should satisfy the criteria given in this clause for the web to be taken as restrained.

9.11.4.2.3 *Restraint for derivation of K_2*

When the plate extends beyond the transverse stiffeners bounding a panel by at least a distance $a/2$, the panel may be assumed to be restrained.

9.11.4.3 *Buckling coefficents*

9.11.4.3.1 *General*

The coefficents K_1, K_q, K_b and K_2 should be obtained from **9.11.4.3.2**, **9.11.4.3.3**, **9.11.4.3.4** and **9.1.4.3.5** respectively, with the panel assumed to be restrained or unrestrained in-plane as determined from **9.11.4.2**.

9.11.4.3.2 *Axial coefficient K_1*

When the stress is compressive, K_1 should be taken as the greater of the values obtained as follows:

a) from Figure 23a) using curve 1 or 2, as appropriate, with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 24, when:

$$K_1 = \left(\frac{t_w}{b}\right)^2 \frac{204\ 500}{\sigma_{yw}};$$

b) from Figure 23a), using curve 3, with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 4.33, when:

$$K_1 = \left(\frac{t_w}{a}\right)^2 \frac{6\ 660}{\sigma_{yw}}$$

where

t_w is the web thickness;

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

a, b are as defined in Figure 23a).

When the stress is tensile, K_1 should be taken as 1.0.

9.11.4.3.3 *Shear coefficient K_q*

K_q should be taken from Figure 23b), unless:

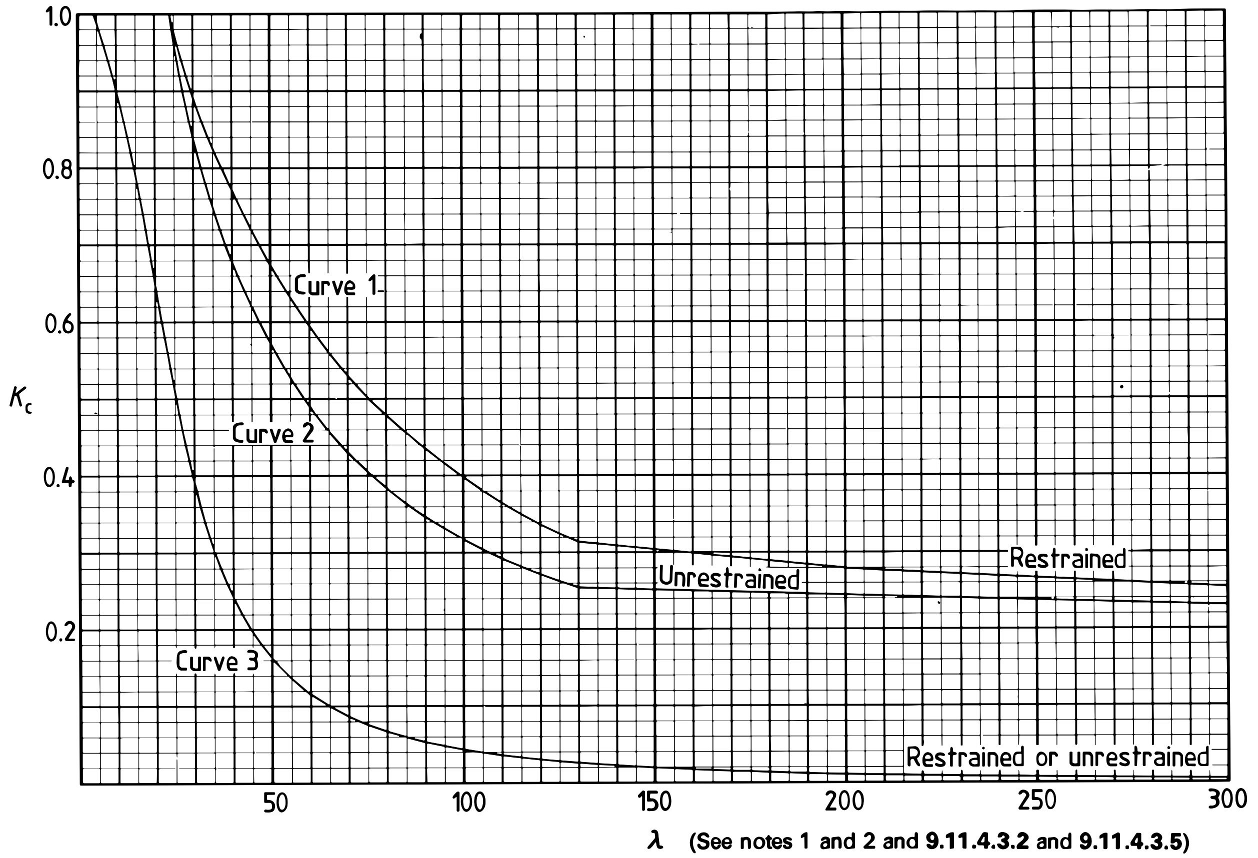
$\dfrac{b}{t_w}\sqrt{\dfrac{\sigma_{yw}}{355}}$ is less than $35\sqrt{1 + (b/a)^2}$

when $K_q = \left(\dfrac{t_w}{b}\right)^2 \dfrac{435\ 000[1 + (b/a)]^2}{\sigma_{yw}}$

where

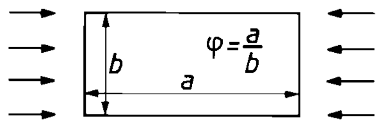
t_w, σ_{yw} are as defined in **9.11.4.3.2**;

a, b are as defined in Figure 23b).



a) K_1 and K_2

NOTE 1 For stresses shown as follows:



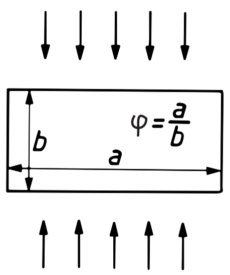
K_1 is the greater value of K_c obtained from curve 1 or 2, as appropriate, with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

or curve 3 with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

NOTE 2 For stresses shown as follows:



K_2 is the greater value of K_c obtained from curve 1 or 2, as appropriate, with:

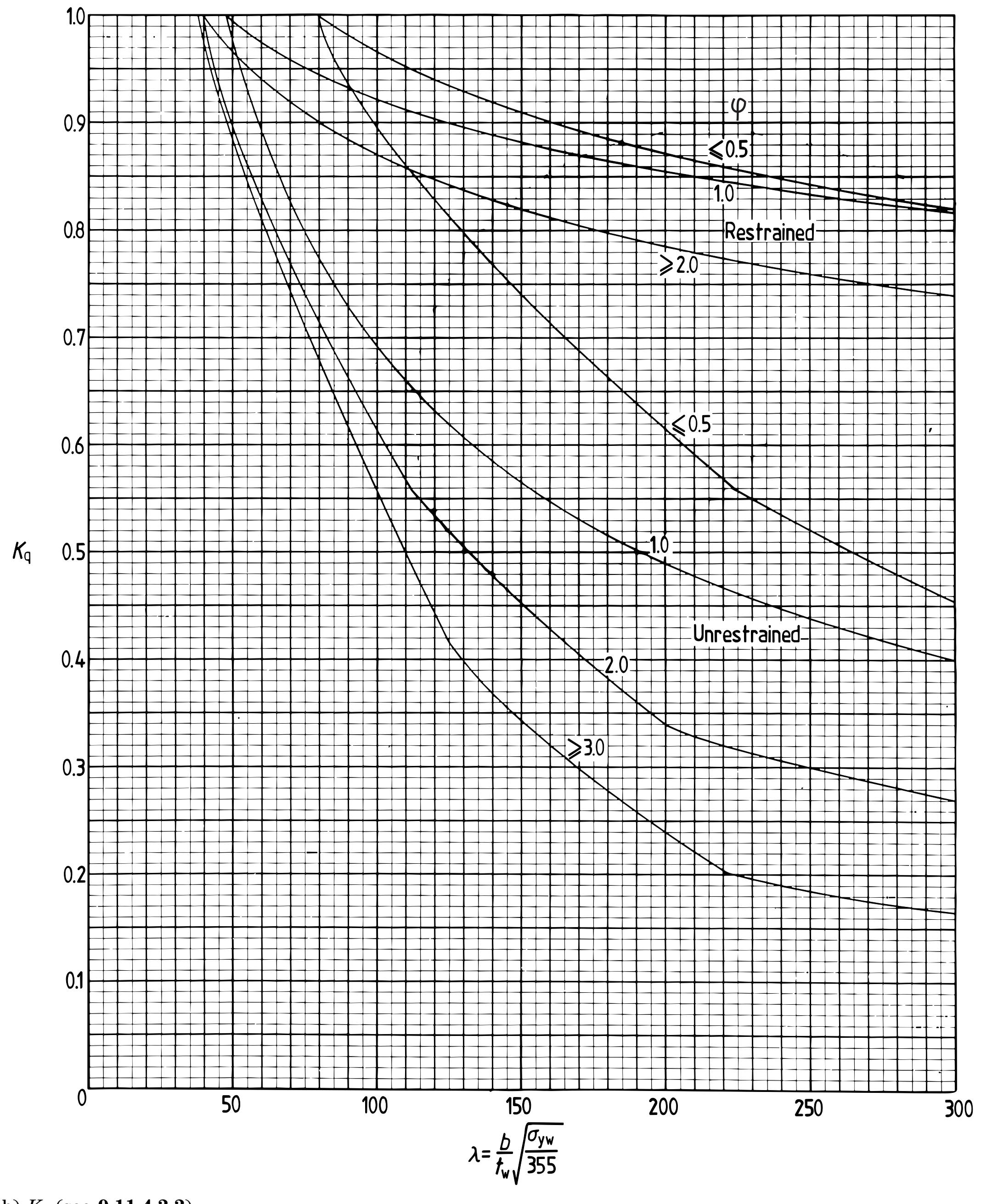
$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

or curve 3 with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

NOTE 3 For basis of curves, see **G.12**a).

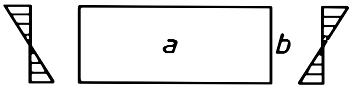
Figure 23 — Buckling coefficients K_1, K_2, K_q and K_b



b) K_q (see **9.11.4.3.3**)

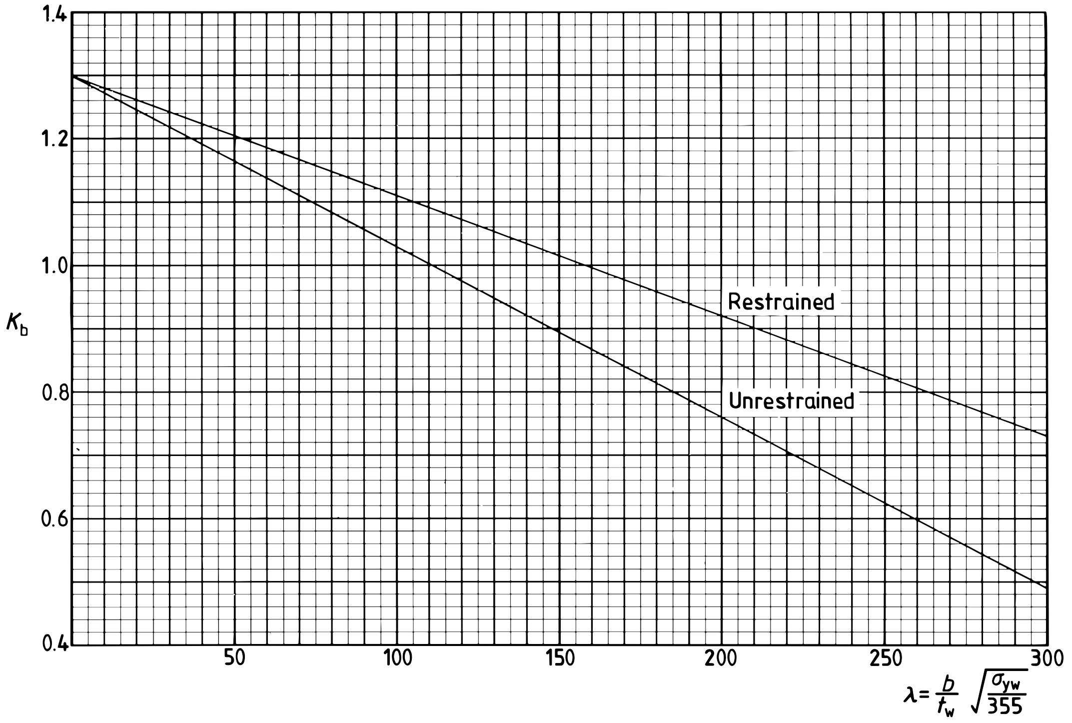
NOTE 1 For intermediate values of φ, K_q may be obtained by linear interpolation between two adjacent values of φ; for this purpose a curve may be extended beyond $K_q = 1$ by the expression in **9.11.4.3.3**.

NOTE 2 $\varphi = a/b$ shown as follows:



NOTE 3 For basis of curves, see **G.12**b).

Figure 23 — Buckling coefficients K_1, K_2, K_q and K_b *(continued)*



c) K_b (see **9.11.4.3.4**)

NOTE For basis of curves see **G.12**c).

Figure 23 — Buckling coefficients K_1, K_2, K_q and K_b *(continued)*

9.11.4.3.4 *Bending coefficient K_b*

K_b should be taken from Figure 23c).

9.11.4.3.5 *Transverse coefficient K_2*

K_2 should be taken as the greater of the values obtained as follows:

a) from Figure 23a) using curve 1 or 2 as appropriate with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 24, when:

$$K_2 = \left(\frac{t_w}{a}\right)^2 \frac{204\,500}{\sigma_{yw}}$$

b) from Figure 23a), using curve 3, with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 4.33, when:

$$K_2 = \left(\frac{t_w}{b}\right)^2 \frac{6\,660}{\sigma_{yw}}$$

where

t_w, σ_{yw} are as defined in **9.11.4.3.2**;

a, b are as defined in Figure 23a).

When the stress is tensile, K_2 should be taken as 1.0.

9.11.4.4 *Interaction buckling criterion*

The following condition should be satisfied for each panel:

$m_c + m_b + 3m_q \leq 1$

a) In the absence of transverse stresses in the panel:

$$m_c = \frac{\sigma_1 \gamma_m \gamma_{f3}}{\sigma_{yw} K_1 (1 - \rho)}$$

$$m_b = \left\{\frac{\sigma_b \gamma_m \gamma_{f3}}{\sigma_{yw} K_b (1 - \rho)}\right\}^2$$

$$m_q = \left(\frac{\tau \gamma_m \gamma_{f3}}{\sigma_{yw} K_q}\right)^2$$

where

σ_1	is as defined in **9.11.3** but the algebraically higher of the two values on the opposite edges should be used;
K_1, K_b, K_q	are coefficients derived in accordance with **9.11.4.3.2** to **9.11.4.3.4**;
σ_{yw}	is the nominal yield stress of the web material as defined in **6.2**;
ρ	= 0 for a restrained panel, irrespective of any redistribution of bending moment or axial force assumed under **9.5.4**,
	or is the proportion of stress assumed to be redistributed from the panel in accordance with **9.5.4** for an unrestrained panel;
σ_b, τ	are as defined in **9.11.3** but the average values over the whole panel should be used.

b) In the presence of transverse stresses in the panel:

$m_c = \sqrt{(m_1 + m_2)}$ for positive values of $(m_1 + m_2)$

$= -\left\{\sqrt{-(m_1 + m_2)}\right\}$ for negative values of $(m_1 + m_2)$

$m_1 = \left\{\frac{\sigma_1 \gamma_m \gamma_{f3}}{\sigma_{yw} K_1 (1 - \rho)}\right\}^2$ to be taken as negative if σ_1 is tensile

$m_2 = \left(\frac{\sigma_2 \gamma_m \gamma_{f3}}{\sigma_{yw} K_2}\right)^2$ to be taken as negative if σ_2 is tensile

m_b, m_q are as defined in a)

where

K_1, σ_1, σ_{yw}, ρ	are as defined in a);
σ_2	is as defined in **9.11.3**, but the algebraically higher of the two values on the opposite edges should be used;
K_2	is a coefficient derived in accordance with **9.11.4.3.5**.

9.11.5 ***Longitudinal web stiffeners***

9.11.5.1 *Effective section for longitudinal web stiffeners*

The effective stiffener section should comprise the stiffener with a width of web plate on each side of the stiffener connection centreline, not exceeding the lesser of:

a) $16t_w(1 - \rho)$; or

b) $\frac{b}{2}(1 - \rho)$

where

t_w	is the thickness of the web plate;
ρ	is the proportion of the longitudinal stress assumed to be redistributed from the relevant panel in accordance with **9.5.4**;
b	is the width of the relevant plate panel adjacent to the stiffener.

9.11.5.2 *Strength of longitudinal web stiffeners*

The design of discontinuous and continuous longitudinal stiffeners should be such that:

$$\sigma_{se} \leq \frac{\sigma_{\ell s}}{\gamma_m \gamma_{f3}}$$

where

$\sigma_{\ell s}$ is the limiting stiffener stress obtained from Figure 24 using the value of:

$\lambda = \frac{a}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$ when σ_{se} is compressive, or is taken as σ_{ys} when σ_{se} is tensile;

σ_{se} $= \sigma_1 + \left(2.5\tau + \frac{a^2}{b^2}\sigma_2\right)\frac{bt_w k_s}{A_{se}}$

σ_1 is the longitudinal stress in the plate along the stiffener connection centreline (derived in accordance with **9.5.2**), taken as positive if compressive;

γ_m = 1.20 when σ_{se} is compressive;
= 1.05 when σ_{se} is tensile;

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material;

σ_2 is the co-existent transverse stress, if any, taken as positive if compressive;

τ is the average shear stress;

a is the clear distance between transverse web stiffeners;

b is the mean of the clear widths of the web plate panels above and below the line of attachment of the stiffener under consideration;

t_w is the web plate thickness;

A_{se} is the area of effective stiffener action;

k_s is obtained from Figure 24 using the value of:

$\lambda = \frac{a}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$

r_{se} is the radius of gyration of the effective stiffener section about an X-X axis parallel to the web (see Figure 1).

9.11.6 ***Curtailment of longitudinal web stiffeners***

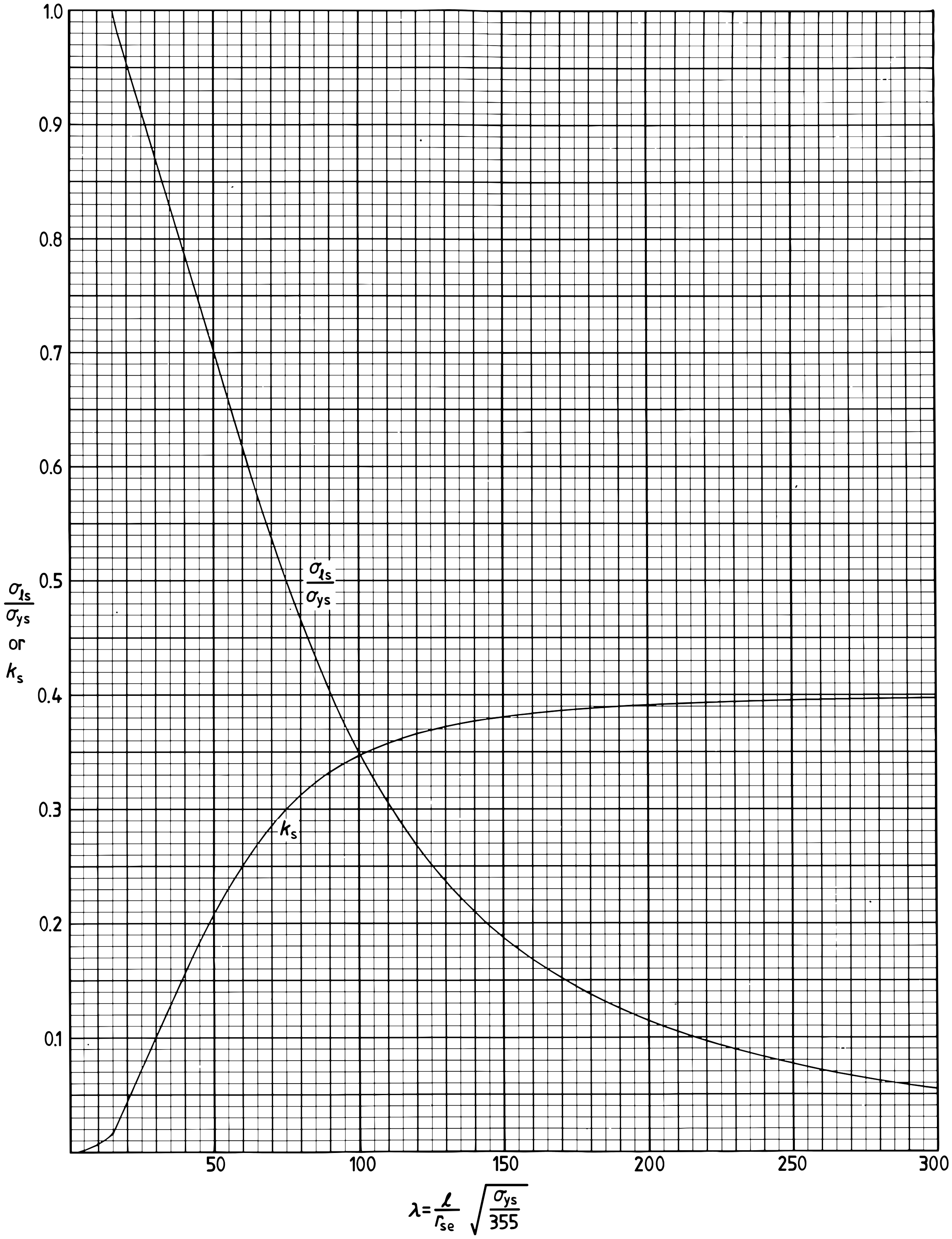
When continuous longitudinal stiffeners are curtailed, the stiffener should be extended beyond the theoretical cut-off point. The attachment of this extension is required to develop the load in the stiffener calculated at its theoretical cut-off point and the extension should be terminated and adequately connected to a transverse member, transverse stiffener or cross frame (see **9.4.2.6**).

Discontinuous longitudinal web stiffeners should be terminated at a transverse member, transverse stiffener or cross frame.

9.12 Restraints to compression flanges

9.12.1 ***General***

The load effects on restraints to a compression flange may be determined by non-linear analysis of representative structural models of beams and the restraint system under ultimate limit state-factored loads with imperfections in geometry of the beams and their supports corresponding to 1.5 times the relevant tolerances given in BS 5400-6. Alternatively, the recommendations of **9.12.2** to **9.12.5** may be applied.



NOTE 1 For the design of longitudinal web stiffeners (see **9.11.5.2**) use $\ell = a$.

NOTE 2 For the design of transverse web stiffeners (see **9.13.3**), bearing stiffeners (see **9.14.3** and **9.14.4**), and diaphragm stiffeners (see **9.17.6.3** and **9.17.6.7**) use $\ell = \ell_s$.

NOTE 3 For basis of curves, see **G.13**.

Figure 24 — Parameters for the design of web stiffeners

9.12.2 ***Elements providing discrete intermediate restraints***

Where the effective length of a compression flange is determined in accordance with **9.6.4.1.1.2** or **9.6.4.1.2**, the restraint system should be in accordance with the following.

a) *Lateral restraints*

Lateral restraints and any associated plan bracing should each be capable of resisting forces F_R in addition to any wind and other laterally applied forces.

For the plan bracing system to "effective" it should have a stiffness such that the lateral deflection which would occur at a node in the bracing system relative to its adjacent nodes when a unit force acts laterally to the restraint considered is no greater than $\ell_R^3/40EI_c$ where ℓ_R is the distance between the nodes and I_c is as defined in **9.6.4.1.1.2**.

NOTE Where two or more parallel beams require lateral restraint at intervals, it is not adequate merely to connect the compression flanges together.

b) *Torsional restraints*

Torsional restraint may be provided by means of a suitable diaphragm between two beams or by equivalent triangulated transverse bracing between two beams such that the beams transfer the restraint effects by equal and opposite vertical shears together with any associated horizontal shears to the supports. Such restraints may alternatively be provided by external means.

Torsional restraints should each be capable of resisting two equal and opposite transverse forces F_R applied normal to each beam at the levels of the centroid of each of its two flanges.

NOTE Intermediate torsional restraints used on their own do not rely on lateral restraint or plan bracing to provide restraint nor do they provide any lateral restraint to the beams. Hence, beams subjected to lateral forces, either external forces (e.g. wind, centrifugal) or sway forces, need to be provided with other means to resist such forces. Restraint against lateral forces may be provided by external lateral restraints, by the lateral bending of the beams, or by a system of plan bracing between the beams.

Each intermediate restraint should be capable of resisting forces F_R appropriate to the restraint under consideration, for which the assumed compression flange forces should be the maximum which occur at the restraint or, if greater, at a position midway between the restraint and the adjacent restraints (including any support restraint present which should be assumed to be an intermediate restraint for this purpose). Vertical component forces arising from the application of forces F_R should be taken into consideration in the design of the beams and restraints. Any eccentricity of the bracing members with respect to the line of action of forces F_R should be taken into account.

The forces F_R should be taken as:

$$F_R = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{\ell_w}{667\delta_R} \text{ but not greater than}$$

1) for lateral restraints

$$\left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{EI_c}{16.7\ell_R^2}$$

2) for torsional restraints

$$\left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{(n+1)^4 EI_c \ell_w}{10v^4 L^3}$$

where

ℓ_w, L are as defined in **9.8**;

I_c, ℓ_R are as defined in **9.6.4.1.1.2**;

δ_R is as defined in **9.6.4.1.1.2** for lateral restraints or should be taken as $n\theta_R d_f^2/2$ for torsional restraints;

σ_{fc} is the maximum compressive stress in the flange;

σ_{ci} $= \dfrac{\pi^2 ES}{\lambda_{LT}^2}$

S $= Z_{pe}/Z_{xc}$;

Z_{pe}	is the plastic modulus of the section (see **9.7.1**);
Z_{xc}	is the elastic modulus of the section with respect to the extreme compression fibre (see **9.7.1**);
θ_R, d_f, n	are as defined in **9.6.4.1.2**;
λ_{LT}	is as derived in **9.7**.

When there are several interconnected beams, two such forces F_R should be applied, in the same or opposite directions, in such a way as to produce the most severe effect in the part being considered.

Where the restraint forces are to be transmitted to end supports by a system of plan bracing, the bracings should be designed to carry the most severe effects arising from the sum of the forces, F_R for each of the restraints within a length ℓ_w in any flange in compression, but not less than the effects due to a lateral force equal to $\Sigma P_f/80$ plus the effects of wind and any other laterally applied forces, where ΣP_f is the sum of the greatest compression forces in two of the beams connected by the bracing.

9.12.3 ***Discrete intermediate U-frame restraints***

9.12.3.1 *General*

Intermediate U-frames may be used to provide lateral restraint in accordance with **9.6.4.1.3**; a plan bracing or decking system, in accordance with **9.12.2**, should be provided which restrains the cross member of each U-frame against movement in a direction normal to the compression flange; intermediate U-frames should be designed in accordance with **9.12.3.2** and **9.12.3.3**.

9.12.3.2 *Strength*

Where the effective length is determined in accordance with **9.6.4.1.3**, each intermediate U-frame and its connections and associated plan bracing system should be designed to resist, in addition to the effects of wind and other applied forces, the effect of horizontal forces F_R acting normal to the compression flange at the level of its centroid as defined in **9.12.2** with ℓ_e, ℓ_R and δ_R as defined in **9.6.4.1.3**.

NOTE When a concrete deck constitutes the whole or part of the cross member of the U-frame, in accordance with **9.6.4.2.2**, only those shear connectors on the main beam flange which are within half the effective width of the concrete deck acting with or as the cross member should be assumed to transmit the load effects at the corner of the U-frame.

9.12.3.3 *U-frames with cross members subjected to vertical loading*

The following additional effects should be included for U-frames with cross members subjected to vertical loading:

a) additional forces F_c applied to the U-frame, in the same manner as F_R in **9.12.3.2**, resulting from the interaction between the bending of the cross members and verticals, which may be taken as:

$$F_c = \frac{\theta d_2}{1.5\delta_R + \left\{ \ell_R^{\,3} \Big/ \left(12EI_c \right) \right\}}$$

where

d_2, δ_R, ℓ_R	are as defined in **9.6.4.1.3**;
I_c	is as defined in **9.6.4.1.1.2**;
θ	is the difference in rotation between the cross member of the U-frame under consideration, and the mean of the rotations of the cross members of the adjacent U-frames on either side. The rotations are calculated in radians under the loading, at the junction of the relevant cross member with the main beam under consideration, assuming that the cross member is simply supported;

b) for all highway and railway bridges the lateral flexure of a compression flange due to loading on a cross member should be taken into account. A method of determining the resulting transverse moment, and of combining it with other effects, may be obtained from annex E.

9.12.4 ***Continuous restraint provided by deck***

9.12.4.1 *Deck at compression flange level*

When a deck is continuously connected to the main beams at the level of the compression flange, the deck and its connections or friction forces should be capable of withstanding a lateral restraining force equal to the greater of 2.5 % of the force in the flange at the point of maximum bending moment in the absence of forces resulting from direct transverse loading, or 1.25 % of the force in the flange at the point of maximum bending moment plus all forces arising from direct transverse loading. This lateral restraining force should be uniformly distributed along the span of the main beams.

9.12.4.2 *Deck not at compression flange level*

Where the effective length of the main beams is determined in accordance with **9.6.4.2.2**, the deck and webs and their connections should be designed to resist, in addition to the effects of wind and other applied forces, the effects due to the following:

a) horizontal forces f_R per unit length, acting normal to the compression flange at the level of its centroid, given by:

$$f_R = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right) \frac{\ell_w}{667\delta_R}$$

where

δ_R	is as defined in **9.6.4.2.2**;
ℓ_w, σ_{fc}, σ_{ci}	are as defined in **9.12.2**.

When there are several interconnected beams, two such forces f_R should be applied, in the same or opposite direction, in such a way as to produce the most severe effect in the part being considered.

b) horizontal forces f_c per unit length, applied in the same manner as f_R in a), resulting from the interaction of bending of the deck and the main beam webs, which may be taken as:

$$f_c = \frac{\theta d_2}{1.5\delta_R}$$

where

δ_R, d_2	are as defined in **9.6.4.2.2**;
θ	is the rotation in radians of the deck at its junction with the web of the main beam under consideration, under the loading used in calculating σ_{fc}, the maximum compressive stress in the flange;

NOTE θ may be calculated neglecting any interaction between the deck and webs of the main beam. The average value of θ should be used within a non-uniformly loaded portion of the span.

c) for all highway or railway bridges the lateral flexure of a compression flange due to loading on a cross member should be taken into account. A method of determining the resulting transverse moment, and of combining it with other effects, may be obtained from annex E.

9.12.5 ***Restraint at supports***

9.12.5.1 *General*

All beams, including cantilever beams, designed in accordance with **9.6**, should be restrained against rotation about their own axes at each support in accordance with **9.12.5.2** and **9.12.5.3** as appropriate.

9.12.5.2 *Restraining forces*

9.12.5.2.1 *General*

The restraining system should be capable of resisting, in addition to the coexistent effects of wind, frictional and other applied forces, two equal and opposite forces F_S applied normal to the beam and in the planes of its two flanges.

Where two or more beams are restrained by a common lateral member interconnecting their ends, pairs of such forces, from the beam being considered and from one adjacent beam, should be taken at the end of each beam, applied so as to produce the most severe effect in the restraining member.

The value of each force F_S in a direction normal to the longitudinal axis of the beam should be taken as follows:

$$F_S = F_{S1} + F_{S2} + F_{S3} + F_{S4}$$

where F_{S1}, F_{S2}, F_{S3}, F_{S4} are as given in **9.12.5.2.2** to **9.12.5.2.5**.

9.12.5.2.2 *Force due to bow of compression flange*

The force F_{S1} on a support due to the end torque on the beam resulting from the initial bow of the compression flange equal to 1.5 times the tolerance in BS 5400-6 may be taken as:

$$F_{S1} = 0.005 \frac{M}{d_f \{1 - (\sigma_{fc}/\sigma_{ci})^2\}}$$

where M, d_f, σ_{fc}, σ_{ci} are as defined in **9.12.5.2.5**.

9.12.5.2.3 *Force due to non-verticality of web at supports*

The force F_{S2} on the support due to magnification of the initial departure from verticality of the supports resulting from compressive force in the flange may be taken as:

$$F_{S2} = \frac{\beta(\Delta_{e1} + \Delta_{e2})\,\sigma_{fc}}{(\sigma_{ci} - \sigma_{fc})\,\Sigma\delta}$$

where β, Δ_{e1}, Δ_{e2}, σ_{fc}, σ_{ci}, $\Sigma\delta$ are as defined in **9.12.5.2.5**.

9.12.5.2.4 *Force due to eccentricity of bearing reaction*

The force F_{S3} at a support due to the eccentricity of the lateral location of the centre of the applied loading relative to the centre of bearing reaction resulting from initial out of verticality of the support combined with that due to change in slope of an end of a beam bearing on a skew may be taken as:

$$F_{S3} = \frac{R d_L(\Delta/\mathrm{D} + \theta_L \tan\alpha)}{D}$$

where R, d_L, Δ, D, θ_L, α are as defined in **9.12.5.2.5**.

9.12.5.2.5 *Force due to twist at skew supports*

The force F_{S4} at a support due to twisting of a beam caused by changes in longitudinal slope of the beam on bearings aligned with skew supports may be taken as follows.

a) For a simply supported non-composite beam without intermediate restraints:

$$F_{S4} = \frac{GJ\,(\theta_{LA}\tan\alpha_A + \theta_{LB}\tan\alpha_B)}{d_f L\{1 + (GJ/Ad_f^2)\}}$$

b) For an end support to a two-span non-composite beam without intermediate restraints or one side of a support adjacent to an internal support in a multi-span non-composite continuous beam without intermediate restraints:

either

$$F_{S4} = \frac{KEI_c d_f\,(\theta_{LA}\tan\alpha_A + \theta_{LB}\tan\alpha_B)}{L^3\,\{1 + (GJ/Ad_f^2)\}}$$

or

$$F_{S4} = \frac{CGJ\,(\theta_{LA}\tan\alpha_A + \theta_{LB}\tan\alpha_B)}{d_f L\,\{1 + (GJ/Ad_f^2)\}}$$

NOTE The two expressions for F_{S4} give the same values and either may be used.

c) For beams rigidly attached to a composite deck by shear connectors the force to be resisted by an end support due to skew support should be taken as:

$$F_{S4} = \frac{D\theta_{LA}\tan\alpha_A}{2\lambda\delta'_R}$$

When the flanges not connected to the composite deck are restrained against plan rotation, the value of F_{S4} given above should be doubled.

NOTE When there is a flexible intermediate restraint only at midspan, it should be ignored for the purposes of deriving the value of F_{S4} according to the above expression.

d) For an internal support in a continuous beam F_{S4} is the algebraic difference between the values of F_{S4} on each side of the support;

where

A $= \dfrac{L}{(\delta_{e1} + \delta_{e2})}$ for a simply supported beam; or

$= \dfrac{L}{(2\delta_i + \delta_e)}$ for a continuous beam;

Δ is the initial lateral deflection of the compression flange relative to the tension flange at the support, which should be taken as $D/200$ if the specification is to BS 5400-6 or 1.5 times any alternative tolerance specification;

Δ_{e1}, Δ_{e2} are the values of Δ in opposite directions at the supports at each end of a simply supported span or at the end and internal supports or restraints, respectively, for a continuous beam;

δ_{e1}, δ_{e2} are the values of δ_e for an end support or for an internal support, as appropriate, as defined in **9.6.2**b);

$\Sigma\delta$ $= \delta_{t1} + \delta_{t2}$ for a simply supported span,

$= \delta_e + 2\delta_i$ for a continuous beam;

L is the distance between the support considered and either the support at the other end of the span considered or any effective intermediate torsional restraint within that span, whichever is the lesser;

d_L is the vertical distance between the levels of the bearing support and the applied loads, respectively. For composite beams the levels of the applied loads should be taken as those of the tops of the steel beams. For cambered beams d_L should be taken as the value given by the sum of the individual applied loads multiplied by their relevant values of d_L, divided by the total applied loads;

G is the shear modulus;

J is the St Venant torison constant for the beam;

I_c is as defined in **9.6.4.1.1.2**;

M is the largest bending moment occuring at a support or within lengths ℓ_w either side of it, whether hogging or sagging;

σ_{fc} is the maximum compressive stress in the flange, averaged over the whole flange width, either at the support under consideration or in the span either side of it;

δ_{t1}, δ_{t2} are the values of δ_t for an end support at each end of a simply supported beam as defined in **9.6.2**a);

δ_e, δ_i are the values of δ_t for an end support and internal support respectively for a continuous beam as defined in **9.6.2**b);

σ_{ci} is as defined in **9.12.2**. In determining σ_{ci} for use in calculating F_{S1}, ℓ_e should be derived in accordance with **9.6.2** for beams without intermediate restraints or **9.6.4** for beams with intermediate restraints. In determining σ_{ci} for use in calculating F_{S2}, ℓ_e should be derived from **9.6.4.1.1.2**b) for simply supported beams or **9.6.2**b) for continuous beams;

D is the overall depth of the beam at the support;

d_f is as defined in **9.6.4.1.2**;

R is the bearing reaction;

β = 1 for an end support,

= 2 for an internal suppport or restraint in continuous beams with lateral restraints;

θ_L is the change in the longitudinal slope of the beam adjacent to a support;

θ_{LA}, θ_{LB} are the coincident changes in the longitudinal slope of the beam adjacent to a support in the span considered, taken as positive when in a sagging direction away from the relevant support, and with θ_{LA} taken as the slope change adjacent to the support being considered and θ_{LB} as that at the opposite end of the span;

C, K are obtained from Figure 25.

NOTE For a two span beam the values of C and K are related to $(d_f/L)(I_c/J)^{0.5}$. For an internal support in a multi-span continuous beam they are obtained from Figure 25 by replacing $(d_f/L)(I_c/J)^{0.5}$ by $(2d_f/L)(I_c/J)^{0.5}$.

α is the angle of skew, i.e. that between the normal to the longitudinal axis of the beam and the axis of the support in plan for a skewed bridge;

α_A is the value of α at the support being considered;

α_B is the value of α at the other end of the span being considered;

λ $= \left(\dfrac{1}{4\delta'_R E I_F}\right)^{0.25}$

δ'_R is the value of δ_R as defined in **9.6.4.2.2** for beams without intermediate restraint or is the value of $\delta_R \ell_R$ as derived from **9.6.4.1.3** for beams with intermediate U-frame restraints;

I_F is the lateral second moment of area of the bottom flange.

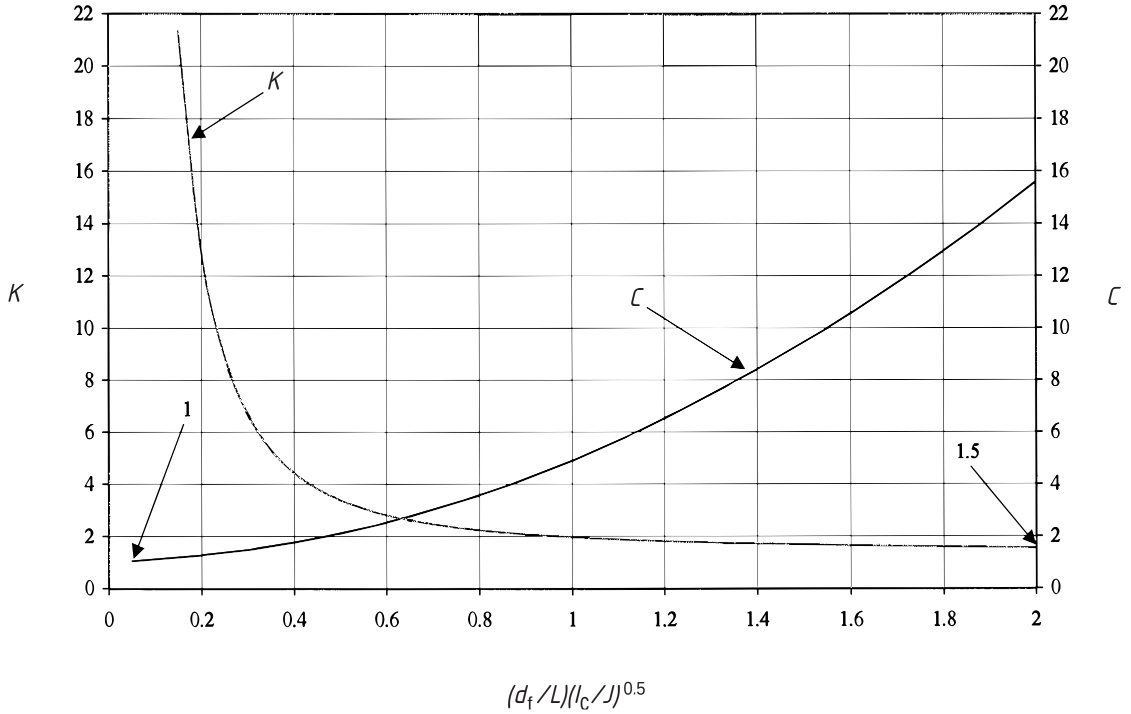


Figure 25 — Factors for determining restraint forces in continuous beams

9.12.5.2.6 *Additional force due to cross members in U-frames subjected to vertical loading*

When the compression flange of a beam is restrained laterally between points of a support by a system of U-frames with cross members subject to vertical loading, a force F_L should be added to F_S as defined above where F_L may be taken as follows:

$$F_L = \sum F_c \frac{(\ell_w - x)}{\ell_w}$$

where

- F_c is as defined in **9.12.3.3** and derived for each loaded U-frame within a length ℓ_w adjacent to the support considered;
- x is the distance of the loaded frame from the support;
- ℓ_w is the half-wavelength of buckling as defined in **9.12.2**;
- Σ denotes summation for all loaded frames.

F_L may alternatively be taken as:

$$F_L = \frac{d_2\theta}{2.5\delta_R + (\delta_e/2) + \left\{\ell_R^{\,3} \Big/ \left(3EI_c\right)\right\}}$$ for an end support to a beam with several internal U-frames;

$$F_L = \frac{d_2\theta}{2\delta_R + \delta_e + \left\{\ell_R^{\,3} \Big/ \left(3EI_c\right)\right\}}$$ for a support to a beam with a single internal U-frame;

$$F_L = \frac{d_2\theta}{(\delta_R/2) + \delta_e + \left\{\ell_R^{\,3} \Big/ \left(12EI_c\right)\right\}}$$ for an internal support;

where

θ	is as defined in **9.12.3.3** and is to be calculated for the most adverse distribution on cross beams as follows:
	for beams with multiple U-frames in a span: the difference between the rotation of the U-frame adjacent to an end support and the mean of the rotations at the end support and the second U-frame from the support, respectively;
	for a beam with only one internal U-frame: the difference between the rotation of the internal frame and the mean of the rotations at the supports at each end of the span;
	for an internal support: the difference between the rotation at the support and the mean of the coincident rotations at the U-frames on each side of the support;
	NOTE Where the support is not in the form of a U-frame, its rotation should be taken as zero.
d_2, δ_R, ℓ_R, δ_e	are defined in **9.6.4.1.3**;
I_c	is as defined in **9.6.4.1.1.2**.

When a beam is continuously restrained by the deck so that its effective length is determined in accordance with **9.6.4.2**, F_L may be taken as:

$$F_L = \frac{\int_0^{\ell_w} f_c(\ell_w - x)\,dx}{\ell_w}$$ which may conservatively be taken as:

$$F_L = \frac{f_c(\ell_{w1} + \ell_{w2})}{2}$$

where

f_c	is as derived in **9.12.4.2**b);
ℓ_{w1}, ℓ_{w2}	are half wavelengths of buckling of the beam on each side of the support under consideraton, derived as in **9.12.2**;
x	is the distance along the span from the support under consideration.

9.12.5.3 *Stiffness*

The stiffness of restraints at supports against rotation about the longitudinal axis of a beam should be such that the values δ_t, calculated as in **9.6.2**, do not exceed the values adopted in determining the effective length of the compression flange.

Where bearing stiffeners are used to provide the sole torsional restraint they should meet the above stiffness criterion in addition to the criteria of **9.14** relevant to their function as bearing stiffeners.

9.13 Transverse web stiffeners other than at supports

9.13.1 *General*

Webs of plate girders, box girders and rolled beams should be provided with transverse stiffeners at all points where these are necessary for the adequacy of the web plate and the longitudinal stiffeners, if any.

A transverse web stiffener should be provided at all locations where a web connects with a cross beam and where a sloping flange changes direction.

Each end of the transverse stiffener should be stopped off or shaped to allow space for a root fillet or weld connecting the web to the flange where these occur, with a clearance not exceeding five times the thickness of the web, as shown in Figure 1. The stiffener should extend over the whole remaining depth of the web and should be fitted closely to the flange at each point of application of a concentrated load to the flange.

Where cut-outs are provided in transverse stiffeners to allow the passage of longitudinal stiffeners, at least one side of the opening in the transverse stiffener should be cleated to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat, or at least one-third of the perimeter of the cut-out should be connected to the longitudinal stiffener by welding.

A transverse web stiffener may form part of a cross beam, cross frame or U-frame.

9.13.2 ***Effective section of transverse web stiffeners***

The effective stiffener section should comprise the stiffener with a portion of web plate of width b_{we} on each side of the stiffener connection centreline taken as the lesser of:

$$16t_w \text{ or } \frac{a}{2}$$

where

t_w is the thickness of the web plate;

a is the spacing of transverse web stiffeners.

Where a stiffener outstand is stopped clear of the flange, the effective stiffener section between the end of the outstand and the flange should be taken as a portion of web plate only of width b_{we} on each side of the stiffener connection centreline for applying the provisions of **9.13.5.2**.

9.13.3 ***Loading on transverse web stiffeners***

9.13.3.1 *Effects to be considered*

A transverse stiffener should be designed to resist the following load effects, where these are present:

a) axial force due to tension field action, in accordance with **9.13.3.2**;

b) axial force representing the destabilizing influence of the web, determined in accordance with **9.13.3.3**;

c) axial force due to transfer of load through a cross frame or cross beam;

d) axial force due to load applied at flange level;

e) axial force due to curvature of flange, in accordance with **9.13.3.4**;

f) axial force due to change of slope of flange;

g) bending moment about the centroidal axis of the effective section parallel to the plane of the web due to eccentricity to that axis of axial forces described in a), c), d), e), and f) above, or from flexure of a cross beam, U-frame or deck.

NOTE When only b) is applicable the stiffener may be designed in accordance with **9.13.6**.

9.13.3.2 *Axial force due to tension field action*

Tension field action should be assumed to occur in any web panel when the average shear stress in the web panel, τ (see **9.5.1**), is greater than τ_o, given by:

$$\tau_o = 3.6E\left\{1 + \left(\frac{b}{a}\right)^2\right\}\left(\frac{t_w}{b}\right)^2\sqrt{1 - \frac{\sigma_1}{2.9E}\left(\frac{b}{t_w}\right)^2}$$

when $\sigma_1 < 2.9E\left(\frac{t_w}{b}\right)^2$, or

$$\tau_o = 0$$

when $\sigma_1 \geq 2.9E\left(\frac{t_w}{b}\right)^2$

where

a is the panel length, i.e. the dimension in the direction of the main longitudinal stress (see Figure 20);

b is the panel width, i.e. the dimension perpendicular to the direction of the main longitudinal stress (see Figure 20);

t_w is the thickness of the web;

σ_1 is the average longitudinal stress in the web panel, to be taken as positive when compressive, calculated without redistribution of moment or axial force to the flanges.

Tension field action should be assumed to cause a compressive force F_{tw} in the adjacent transverse stiffener over its entire length, given by:

$$F_{tw} = (\tau - \tau_o)t_w a \text{ or } (\tau - \tau_o)t_w \ell_s\text{, whichever is smaller;}$$

where

ℓ_s is the length of the transverse stiffener, measured along the stiffener as the clear distance between the flanges of the beam.

Where F_{tw} differs on the two sides of a transverse stiffener the average value may be taken.

When there are longitudinal stiffeners on the web, the average of the two smallest values of τ_o occuring in the web panels on one side of the transverse stiffener should be used in detemining the value of F_{tw} for that side.

The force F_{tw} is to be taken as acting in the mid-plane of the web.

9.13.3.3 *Axial force representing the destabilizing influence of the web*

In order to resist buckling of the web plate the effective stiffener section should be assumed to carry, along its centroidal axis, a compressive force F_{wi} given by:

$$F_{wi} = \frac{\ell_s{}^2}{a} \; t_w k_s \sigma_R$$

where

ℓ_s is as defined in **9.13.3.2**;

a is one half of the sum of the panel widths on each side of the stiffener;

t_w is the thickness of the web;

k_s is obtained from Figure 24 using the slenderness parameter λ determined from:

$$\lambda = \frac{\ell_s}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$$

r_{se} is the radius of gyration of the effective stiffener section about the centroidal axis X-X (see Figure 1);

σ_R $= \tau_R + \left(1 + \frac{\Sigma A_s}{\ell_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right)$

τ_R is equal to τ or τ_o, whichever is less (see **9.13.3.2**);

ΣA_s is the sum of the cross-sectional areas of all the longitudinal stiffeners on the web, whether continuous or discontinuous, not including any adjacent web plate;

σ_1 is the average longitudinal stress in the web, taken as a positive when compressive, calculated without any redistribution to the flanges (see Figure 26);

σ_b is the maximum value of the stress in the web due to bending alone, calculated without any redistribution of moment to the flanges, as permitted in **9.5.4**, and always taken as a positive (see Figure 26);

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material.

The force F_{wi} may be factored by η_s

where

$$\eta_s = \frac{1}{1 + \left(0.5\ell_s{}^3 / a^3 I\right)\left\{\Sigma I_s + \left(\ell_s t_w{}^3 / 12\right)\right\}}$$

which may conservatively be taken as unity;

ΣI_s is the sum of the moments of inertia of the effective section of all continuous longitudinal web stiffeners if any in the depth ℓ_s derived in accordance with **9.11.5.1**;

I is the moment of inertia of the effective section of the transverse stiffener.

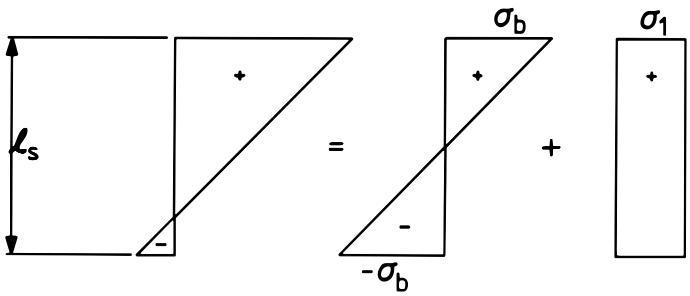


Figure 26 — Longitudinal stress in webs with transverse stiffeners

9.13.3.4 *Axial force due to curvature*

The web plate included in the effective stiffener section should be considered to be subjected to an axial force F_f given by:

$$F_f = \frac{\sigma_f B_f \, t_f b_{we}}{R_f \cos \beta}$$

where

σ_f, B_f, t_f, R_f, β are as defined in **9.5.7**;

b_{we} is as defined in **9.13.2**.

9.13.4 *Distribution of axial loading*

The force in a stiffener due to the load applied at flange level, or due to curvature or change of slope of a stressed flange, or due to transfer of load through a cross frame, should be assumed to vary uniformly along the length of the stiffener from the value at the point of application to zero at the remote end of the stiffener.

The force in a stiffener due to tension field action or restraint of web buckling should be assumed as constant over the length of the stiffener.

9.13.5 *Strength of transverse web stiffeners*

9.13.5.1 *Yielding of web plate*

The maximum equivalent stress σ_e in the portion of web plate included in the effective stiffener section due to all the relevant forces and moments listed in **9.13.3.1**, except that in **9.13.3.1**b), together with co-existent shear and longitudinal stresses, should not exceed:

$$\frac{\sigma_{yw}}{\gamma_m \gamma_{f3}}$$

where

σ_e $= \sqrt{(\sigma_1 + k\sigma_b)^2 + \sigma_{es2}^2 - \sigma_{es2}(\sigma_1 + k\sigma_b) + 3\tau_R^2}$;

σ_1 is as defined in **9.13.3.3**;

σ_b is the longitudinal bending stress in the web;

σ_{es2} is the transverse stress in the web plate forming part of the effective stiffener section;

τ_R is the lesser of τ and τ_o defined in **9.13.3.3**;

k is as defined in **9.11.3**b);

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

9.13.5.2 *Yielding of stiffener*

The maximum stress in the stiffener at every point along its entire length between the flanges due to all relevant forces and moments listed in **9.13.3.1**, except that in **9.13.3.1**b), should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material.

In areas where cut-outs are provided an appropriate reduced effective section should be taken.

Where the end of a stiffener is fitted closely to the flange of a beam, whether to meet the provisions of **9.13.1** or otherwise, the bearing stress over the area in contact should not exceed:

$$\frac{1.33\sigma_{ys}}{\gamma_m\gamma_{f3}}$$

In calculating this stress, the effective bearing area should be taken to consist of only those portions of the area of the stiffener and the web plate that are:

a) in contact with the flange;

b) clear of the weld or root fillet at the web flange junction;

c) within dispersal lines drawn at 60° from the line of application of any local load through the thickness of a flange plate (see Figure 27).

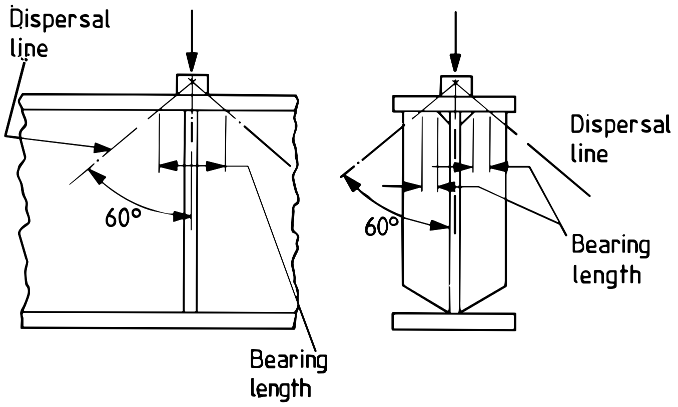


Figure 27 — Dispersal of load through a transversely stiffened web

9.13.5.3 *Buckling of effectice stiffener section*

The effective stiffener section defined in **9.13.2** should be such that:

$$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x\sigma_y} \leq \frac{1}{\gamma_m\gamma_{f3}}$$

where

P, M_{xs} are, respectively, the total maximum force on the effective stiffener section and the total maximum moment about the centroidal axis parallel to the web, due to all the relevant load effects fiven in **9.13.3.1** within the middle third of the stiffener length;

$\sigma_{\ell s}$ is taken as equal to the value of σ_c determined from:

a) for double sided stiffeners, curve D of Figure 37; or

b) for single sided stiffeners, curve B of Figure 37 when the web plate is in compression or curve D when the stiffener outstand is in compression;

where the slenderness parameter λ is determined from:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_y}{355}}$$

ℓ_s is as defined in **9.13.3.2**.

Z_x is the elastic section modulus of the effective section about the centroidal axis parallel to the web with reference to the extreme fibres under maximum compressive stress;

σ_y is the nominal yield stress value, as defined in **9.3.1**, for the web plate, or for the stiffener;

A_{se}, r_{se} are the area and radius of gyration, respectively, of the effective stiffener section.

9.13.6 ***Transverse web stiffeners without applied loading***

Where a transverse stiffener is subjected only to the load effect of **9.13.3.1**b), it need only be designed to satisfy the following:

$$\sigma_R \le \frac{A_{se} a \sigma_{\ell s}}{\eta_s k_s t_w \ell_s^{\ 2} \gamma_m \gamma_{f3}}$$

where

σ_R, a, k_s, t_w	are as defined in **9.13.3.3**;
A_{se}, $\sigma_{\ell s}$	are as defined in **9.13.5.3**;
ℓ_s	is as defined in **9.13.3.2**;
η_s	is as defined in **9.13.3.3**.

9.14 Load bearing support stiffeners

9.14.1 ***General***

Webs of plate girders and rolled beams should be provided with a system of load bearing stiffeners at each support position. Hereafter these will be referred to as bearing stiffeners.

The section of a bearing stiffener should be symmetrical about the mid-plane of the web. When this condition is not met, the effect of the resulting eccentricity should be taken into account.

The ends of a bearing stiffener should be adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web (see Figure 1). The stiffener should be fitted closely to the flange where the flange is subject to a concentrated load.

Where cut-outs are provided in bearing stiffeners, to allow the passage of longitudinal stiffeners, at least one side of the opening in the bearing stiffener should be cleated to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat, or at least one-third of the perimeter of the cut-out should be connected to the longitudinal stiffener by welding.

A bearing stiffener may be used to provide torsional restraint to a beam at its support, as recommended by **9.12.5.3**; it may also form part of a cross beam, cross frame or U-frame.

9.14.2 ***Effective section for bearing stiffeners***

9.14.2.1 *Single leg stiffener*

The effective stiffener section should be taken to comprise the stiffener with the portion of web plate on each side, having a width not exceeding the following [see Figure 28b)]:

a) half the spacing of transverse stiffeners;

b) the distance to the transverse edge of the web plate at the end of a beam;

c) $16t_w$ where t_w is the thickness of the web plate.

9.14.2.2 *Multileg stiffeners*

The effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs, and a portion of web plate not exceeding the widths given in **9.14.2.1** on the outer sides of the outer legs.

If the spacing of the legs of adjacent stiffeners is greater than $25t_w$, the legs should be treated as independent stiffeners in accordance with **9.14.2.1**.

9.14.3 ***Loading on bearing stiffeners***

9.14.3.1 *Effects to be considered*

A stiffener should be designed to resist the following load effects, where these are present:

a) an axial force equal to the algebraic sum of the reaction from the support bearing and the vertical components of the forces in the adjacent bottom flange or flanges, due to slope or curvature;

b) an axial force representing the destabilizing influence of the web, determined in accordance with **9.14.3.2**;

c) an axial force due to transfer of load through a cross frame or cross beam;

d) bending moments about the X-X or Y-Y axes (as shown in Figure 28) arising from eccentricity of the axial force about the relevant axes, determined in accordance with **9.14.3.3**, or from flexure of a cross beam, cross frame, U-frame or deck;

e) a bending moment about the X-X axis due to the transverse forces F_S determined and applied in accordance with **9.12.5.2**. This moment should be assumed to be applied to any bearing stiffener which, as a cantilever or part of a U-frame, is the sole means of providing the restraint necessary to meet the provisions of **9.12.5.1**;

f) a bending moment about the Y-Y axis due to shear in the web adjacent to an end bearing stiffener (see **9.14.3.4**) in the case of beams with intermediate transverse web stiffeners.

9.14.3.2 *Axial force representing the destabilizing influence of the web*

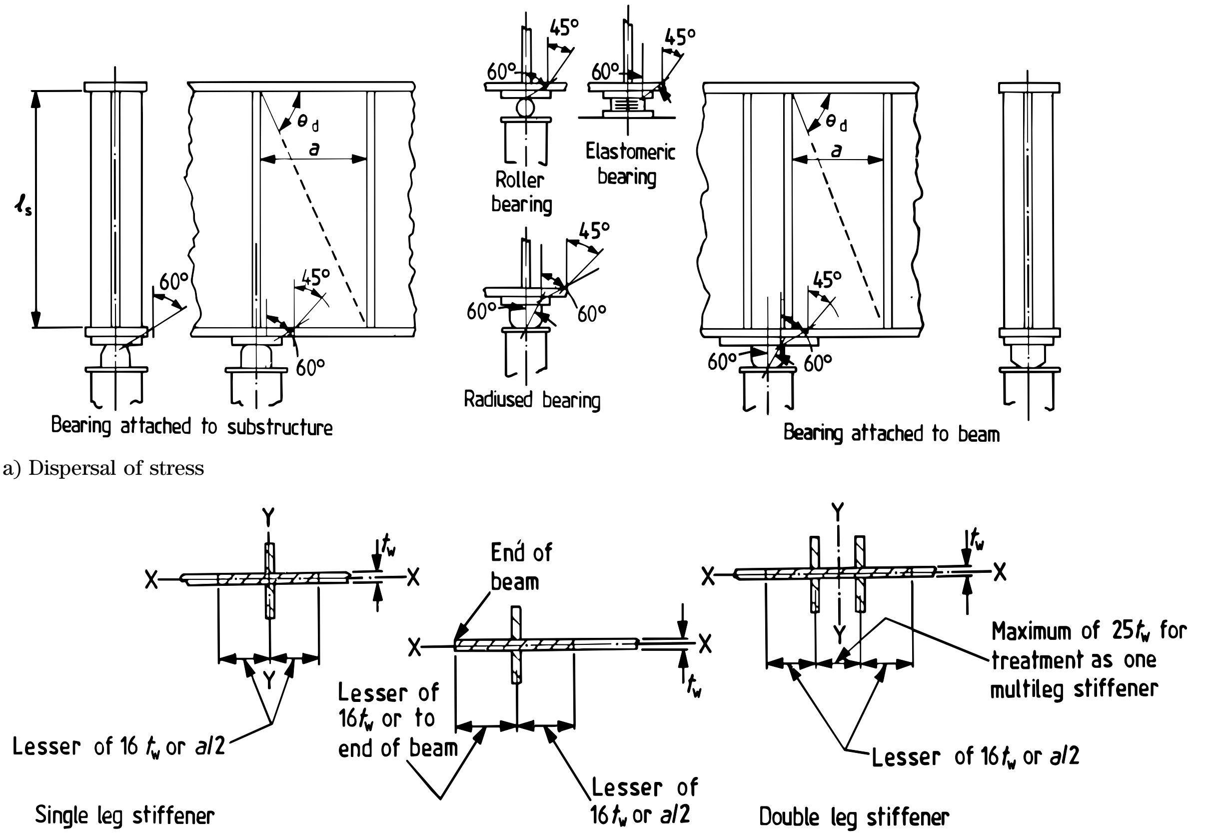
In order to resist web buckling, the effective section of a bearing stiffener at an internal support of a continuous beam should be assumed to carry an additional compressive force F_{wi} along its centroidal axis:

$$F_{wi} = \frac{\ell_s^2}{a} t_w k_s \sigma_R$$

where

σ_R	$= \left(1 + \frac{\Sigma A_s}{\ell_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right)$
t_w, a, k_s, ΣA_s, σ_1, σ_b	are as defined in **9.13.3.3**;
ℓ_s	is as defined in **9.13.3.2**.

The force F_{wi} may be factored by η_s, where η_s is as given in **9.13.3.3**.



a) Dispersal of stress

b) Effective section

NOTE Dispersal lines lie continuously in metal from their origin. Where a 60° dispersal angle is shown, this is the maximum that may be assumed.

Figure 28 — Bearing stiffeners

9.14.3.3 *Eccentricity*

Load effects due to eccentricities arising from the following causes should be taken into account, when relevant:

a) movements of the beam relative to the bearing due to changes in temperature;

b) changes in the point or line of contact at the spherical or cylindrical surface of a bearing due to slope of the beam when deflected by load;

c) uneven seating which may occur on a flat bearing surface;

d) inaccuracy which may occur in the positioning of the beam relative to the bearing.

The following values of eccentricity may be assumed to satisfy the provisions of c) and d):

1) half the width of the flat bearing surface plus 10 mm for a flat-topped rocker bearing in contact with a flat bearing surface; or

2) 3 mm for a radiused upper bearing resting on a flat or radiused lower part; or

3) 10 mm for a flat upper bearing resting on a radiused lower part.

9.14.3.4 *End supports*

When the shear stress τ in the web of a beam at a section adjacent to an end support stiffener is greater than τ_o, the support stiffener, together with any additional stiffening which may be provided at the end of the beam (see Figure 1) should be designed to resist an additional bending moment M_y about the axis through the centroid of the end section perpendicular to the web, i.e. the Y-Y axis shown in Figure 1, given by:

$$M_y = \frac{8(\tau - \tau_o) t_w \ell_s^2}{\theta_d}$$

where

τ_o $= 3.6E\left\{1 + \left(\frac{b}{a}\right)^2\right\}\left(\frac{t_w}{b}\right)^2$

t_w, a, b are as defined in **9.13.3.2**;

ℓ_s is the length of the bearing stiffener, measured along the stiffener as the clear distance between the flanges of the beam;

θ_d is the slope in degrees of the diagonal line joining the top of the bearing stiffener to the bottom of the adjacent transverse stiffener, as shown in Figure 28.

If there are longitudinal stiffeners on the web, the average of the two smallest values of τ_o in the web panels in the section should be used.

NOTE For beams without intermediate transverse web stiffeners, M_y may be taken as zero.

9.14.4 *Strength of bearing stiffeners*

9.14.4.1 *Yielding of the web plate*

The maximum equivalent stress σ_e, determined from **9.13.5.1**, in the portion of web plate included in the effective stiffener section due to all relevant forces and moments listed in **9.14.3.1**, except that in **9.14.3.1**b), together with co-existent longitudinal stresses but excluding co-existent shear in the web plate, should not exceed:

$$\frac{\sigma_{yw}}{\gamma_m \gamma_{f3}}$$

where

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

9.14.4.2 *Yielding of stiffener*

The maximum stress in the stiffener itself, at every point along its length, due to all relevant load effects listed in **9.14.3.1**, except that in **9.14.3.1**b), calculated on the basis of the effective section as determined under **9.14.2**, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material.

In areas where cut-outs are provided an appropriate reduced effective section should be taken.

The bearing stress over the area of the end of a stiffener which has been fitted to a flange should not exceed:

$$\frac{1.33\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

In calculating this stress, the effective bearing area should consist only of those portions of the area of the stiffener and the web that are within the uninterrupted dispersal lines drawn at a maximum of 60° to the line of application of the bearing reaction from the contact area (see Figures 27 and 28) and should comprise:

a) the fitted area of the stiffener, clear of the weld or root fillet at the web flange junction; and

b) for a rolled section beam, the web of the rolled section over the thickness of the web and the root fillets; or

c) for a fabricated beam where the web is fitted locally, the area of the web.

When considering the fatigue endurance, in accordance with **9.2.2**, the bearing forces and moments should be assumed to be carried entirely by the welds and, where the beam is a rolled section, the web and its root fillets; no bearing should be assumed over the fitted area.

9.14.4.3 *Buckling of effective stiffener section*

The effective stiffener section, as defined in **9.14.2**, should be such that:

$$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x \sigma_y} + \frac{M_{ys}}{Z_y \sigma_y} + \frac{M_y}{Z_{cy}\sigma_y} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P	is the maximum force on the stiffener within the middle third of its length;
M_{xs}, M_{ys}	are the maximum bending moments on the stiffener about the X-X and Y-Y axes, respectively, (see Figure 28) due to all the relevant load effects listed in **9.14.3.1**, except that in **9.14.3.1**f), within the middle third of the length of the stiffener;
A_{se}	is the area of the effective stiffener section;
$\sigma_{\ell s}$	is as defined in **9.13.5.3**;
Z_x, Z_y	are the appropriate section moduli of the effective stiffener section;
M_y	is the additional bending moment on an end section, as defined in **9.14.3.4**;
Z_{cy}	is the section modulus of the end section as described in **9.14.3.4**;
σ_y	is the nominal yield stress value, as defined in **9.3.1**, for the web plate or the stiffener, as appropriate.

9.14.5 ***Stiffness of bearing stiffeners***

A bearing stiffener, acting as a cantilever from the bottom flange level or as part of a U-frame, which is the sole means of providing the restraint necessary to meet the provisions of **9.12.5.1**, should be in accordance with **9.12.5.3**.

9.15 Cross beams and other transverse members in stiffened flanges

9.15.1 *General*

9.15.1.1 *Loading*

Where transverse members are provided on flanges or decks they should be designed for the following:

a) to transfer local loading from the flange or deck to the web of main beams;

b) to distribute loading transversely to the main beams;

c) to withstand forces arising from a longitudinal change of slope of a flange.

NOTE In box girders, a flange transverse member may also be part of an internal cross frame or diaphragm.

9.15.1.2 *Compression flanges*

In addition to the provisions of **9.15.1.1**, compression flange transverse members should have sufficient stiffness and strength to prevent overall buckling of the flange. A member designed in accordance with **9.15.3** and **9.15.5** may be deemed to satisfy this provision.

9.15.1.3 *Transverse member support*

All flange transverse members designed to meet the provisions of **9.15.1.1** and **9.15.1.2** should be supported by transverse web stiffeners at main beam webs, unless a special investigation is undertaken to show such stiffening to be unnecessary. Transverse members required for a flange projecting from an outer main beam should be continuous with the transverse member between main beam webs.

9.15.2 ***Effective section for transverse members***

9.15.2.1 *Effective section for stiffness*

In determining the stiffness of a transverse member for global analysis and for overall buckling of a compression flange, an effective width of attached flange should be assumed to act with the member, on each side of the web of the cross member where available, and should be taken as the lesser of:

a) half the spacing of transverse members; or

b) either:

1) one-eighth of the distance between main beam webs, for a portion between such webs; or

2) one-sixth of the cantilever length, for a cantilever portion.

This effective width should be taken as constant over each relevant portion of the transverse member.

NOTE When the flange consists of composite reinforced concrete, the reinforcement within the effective width, but not the concrete, may be taken into account in calculating the overall buckling strength of the compression flange, but both reinforcement and concrete may be taken into account for global analysis.

9.15.2.2 *Effective section for strength and stress calculation*

For calculating stresses in, or the strength of, a transverse member, the effective section should be taken as for stiffness (see **9.15.2.1**), but with the following modifications:

a) when, in a non-composite transverse member, the attached flange is in compression parallel to its axis (e.g. in the sagging moment zone of a top flange transverse member) the effective width on each side of the web of the cross member should not exceed one-quarter of the transverse member spacing;

b) between main beam webs, in the portion subjected to hogging bending moments, the effective width on each side of the web of the cross member should not locally exceed one-fourteenth of the distance between main beam webs;

c) when the effective widths on two sides of a main beam web are unequal, an average value should be taken for the section at the main beam web;

d) in the case of composite flanges, the area of concrete in tension should be ignored;

e) the effective thickness of the cross member web should be determined in accordance with **9.4.2.5**.

9.15.2.3 *Compact section*

Irrespective of the effective width of the flange, transverse members attached to a continuous deck may be taken as compact if their webs and flange outstands meet the appropriate provisions of **9.3.7**.

9.15.3 ***Stiffness of transverse members in compression flanges***

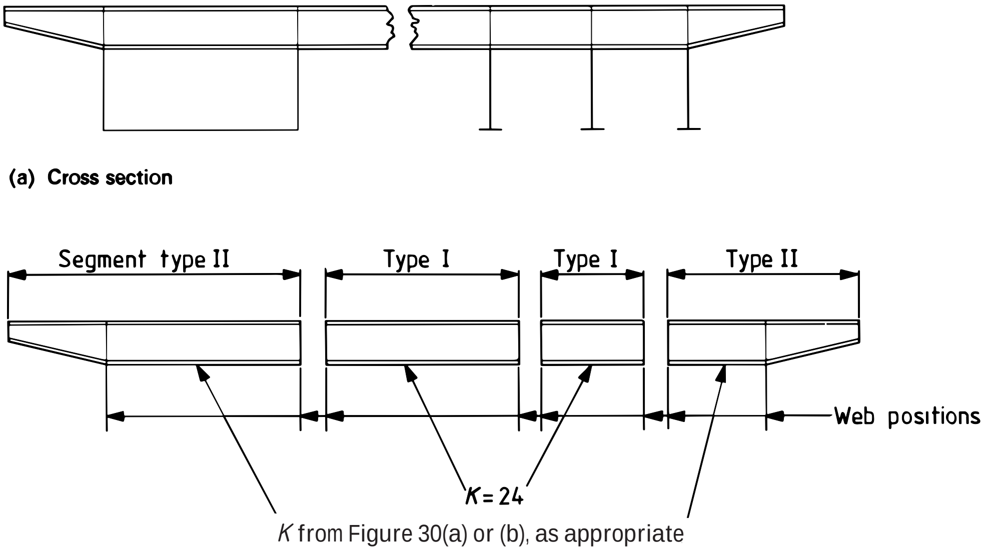
9.15.3.1 *Transverse member segment types*

In order to satisfy the stiffness provisions of **9.15.1.2** for a transverse member supporting a compression flange, the entire length of the effective member should be divided, for analysis, into the following segments (see Figure 29):

a) type I segments between interior webs of main beams;

b) type II segments comprising any cantilever and the adjacent length of the first interior beam web.

NOTE For cross-sections with only two main beam webs, one segment of type II will encompass the full length of the transverse member, including cantilevers (if any).



b) Cross-section divided into segments between webs, with cantilevers taken as integral parts of the adjacent internal segment (see **9.15.3.1**)

NOTE *K* should be taken from annex F when any of the provisions of **9.15.3.2** are not met.

Figure 29 — Segments of transverse members continuous over three or more webs

9.15.3.2 *Stiffness of transverse members*

Transverse members should be designed such that:

$$I_{be} \geq \frac{9{\sigma_f}^2 a B^4 {A_f}^2}{16 K E^2 I_f}$$

where

I_{be} is the average second moment of area of the effective transverse member (see **9.15.2.1**) between the main beam webs;

σ_f is the longitudinal compressive stress in the flange of the main beam, averaged across the width of the segment between main beam webs;

B is the spacing of the main beam webs at the level of the transverse member;

a is the transverse member spacing (or mean of adjacent spacings);

A_f is the area per unit width of the cross-section of the flange of the main beam;

I_f is the second moment of area per unit width of the flange of the main beam about the centroidal axis of the flange.

NOTE 1 A_f and I_f should include any longitudinal stiffeners, and, in the case of a concrete or composite flange, concrete areas divided by the modular ratio. Where there are longitudinal stiffeners, A_f and I_f should be calculated on the basis of the effective section of the member derived in accordance with **9.4.2**.

K = 24 for type I segments (see **9.15.3.1**) with open longitudinal stiffeners (but see note 2 for closed stiffeners), or,
for type II segments (see **9.15.3.1**), K should be obtained from Figure 30a) or b) for cantilever overhangs on one or both sides, respectively, for the appropriate values of I_{bc}/I_{be} and B_c/B provided that the following limitations are taken into account:

a) longitudinal stiffeners are of open type (but see note 2 for closed stiffeners); and

b) I_f and σ_f are constant over the whole segment including cantilevers; and

c) either:

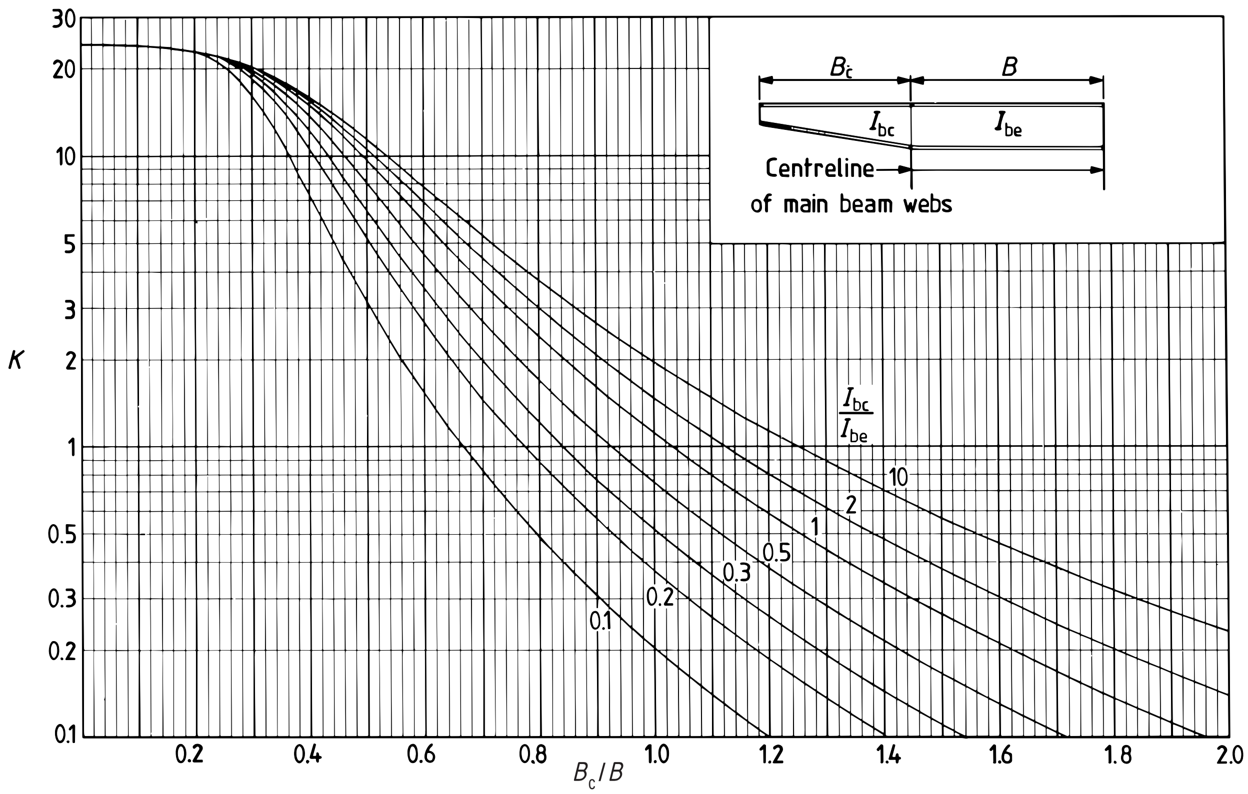
1) there are no edge members at the cantilever tips; or

2) the edge member is unable to carry longitudinal stress because of its structural detailing; or

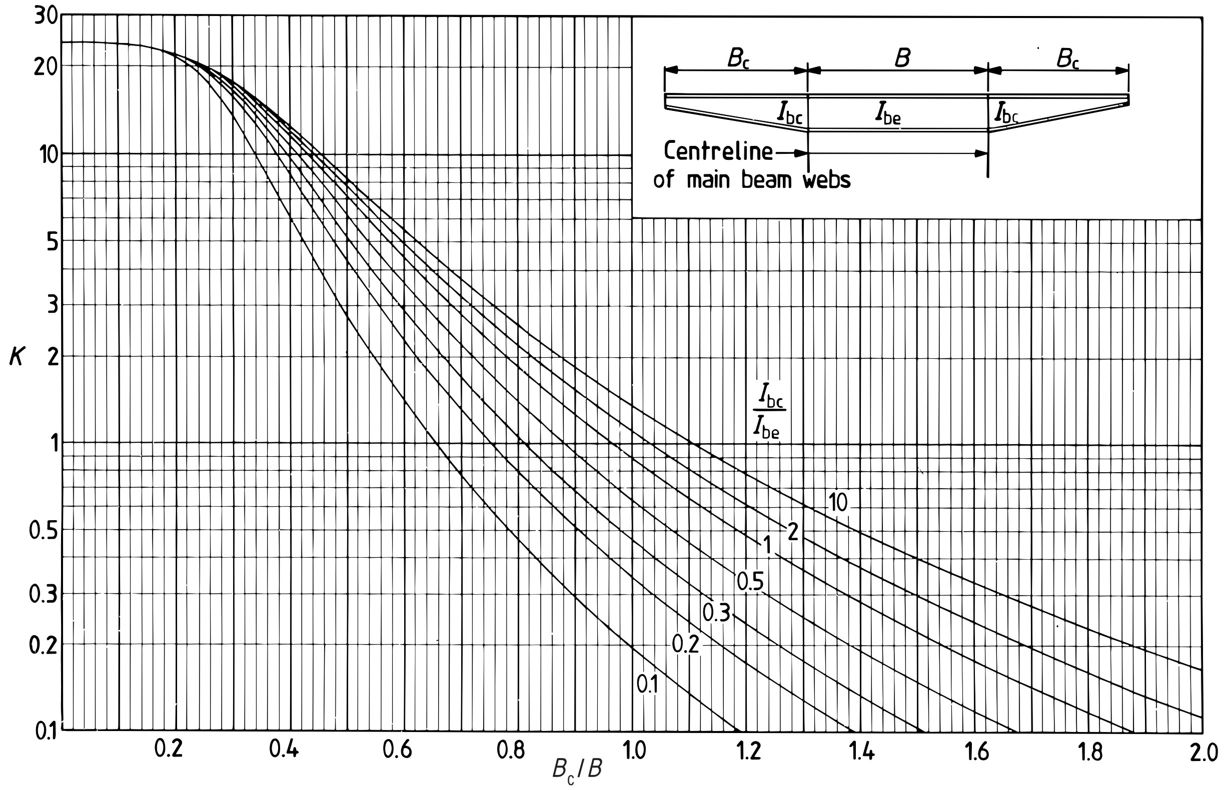
3) the radius of gyration of the edge member about its centroidal axis is not less than 1.65 times the radius of gyration of the flange of the main beam;

B_c, I_{bc} are the length and the average effective second moment of area, respectively, of the cantilever portion of the transverse member (see Figure 30).

NOTE 2 For closed longitudinal stiffeners, values of the buckling coefficient K may either be taken conservatively as given above or be determined more accurately from annex F. For cases in accordance with the limitations given in b) or c), values for the buckling coeffecient K may be determined from annex F.



a) With cantilevers on one side only (see **9.15.3**)



b) With cantilevers on both sides (see **9.15.3**)

NOTE For basis of curves, see **G.15**.

Figure 30 — Buckling coefficient *K* for transverse members

9.15.4 ***Loading on transverse members***

9.15.4.1 *Effects to be considered*

Transverse members should be designed to resist load effects due to the following:

a) dead and live loading placed locally over the transverse members;

b) transverse distribution of loading between the main beams (see also **9.15.4.5**);

c) restraint of distortion of box girders;

d) creep and shrinkage of concrete, and differential temperature (but see also **9.2.1.3** and **9.2.3.2**);

e) longitudinal curvature of the main flange in elevation (see **9.15.4.2**);

f) change of longitudinal slope of the main flange (see **9.15.4.3**);

g) profile deviations from the specified profile in elevation of a compression flange (see **9.15.4.4**).

Methods of application of the loadings given in a) to g) are given in **9.15.4.5**.

9.15.4.2 *Flanges curved in elevation*

When a flange of a main beam is curved in elevation, the transverse member should be designed to resist a load equal to:

$$\frac{P_f a}{R_f}$$

where

P_f is the longitudinal force in the flange;

a is the distance between transverse members;

R_f is the radius of curvature of the flange.

The direction of the load on the transverse member should be assumed to be such as to increase the flange curvature in a compression flange, or decrease it in a tension flange.

9.15.4.3 *Change of flange slope*

At any change in the longitudinal slope of the flange of the main beam, a transverse member should be designed to resist a load equal to the component of the longitudinal flange force in the plane of the web of the transverse member.

9.15.4.4 *Profile deviation in compression flanges*

A destabilizing force should be assumed to act on each portion of each transverse member, in either direction in the plane of its web, in such a way as to maximize the load effects at the section of the transverse member under consideration. The magnitude of this force should be taken as:

a) $P_f/200$ per unit width, distributed uniformly along the length of portions of transverse members between adjacent main beam webs; and

b) $P_{fc}/200$ per unit width, distributed uniformly along the length of any cantilever portions of transverse members; and

c) $P_{fc}A_{sc}/160A_{fc}$ concentrated at a longitudinal stiffening member at the cantilever tips, if this is capable of transmitting longitudinal stress;

where

P_f is the average longitudinal compressive force per unit width in the relevant portion of flange between adjacent webs of main beams;

P_{fc} is the average longitudinal compressive force per unit width in the flange attached to a cantilever portion of a transverse member;

A_{sc} is the area of the cross-section of the longitudinal stiffening member at the cantilever tip;

A_{fc} is the area of the cross-section per unit width of the flange attached to the cantilever portion of the transverse members, inclusive of any longitudinal stiffeners. In the case of a concrete or composite flange, the area of the concrete divided by the modular ratio should be included.

9.15.4.5 *Application of loading*

9.15.4.5.1 Loadings which cause substantially the same effects on a series of transverse members along the length of the flange (e.g. the effects of dead load, or of creep and shrinkage of concrete) should be considered to be applied directly to the transverse member under consideration.

9.15.4.5.2 Loadings which cause substantially different effects on adjacent transverse members (e.g. the effect of traffic loading on a deck supported by transverse members) should be distributed over several transverse members. Any suitable elastic method (e.g. a grillage analysis) may be used to determine the maximum effects on the transverse member under consideration, neglecting the destabilizing effect of longitudinal flange stress in compressive flanges.

NOTE A non-linear analysis including the destabilizing effect of longitudinal flange stress may be used to determine the maximum effects of the load distribution on the transverse member under consideration, in which case, when applying **9.15.4.5.4**, $i_2 = 1.0$.

9.15.4.5.3 For tension flanges, the effects from **9.15.4.5.1** and **9.15.4.5.2** should be superimposed.

9.15.4.5.4 For compression flanges, the effects from **9.15.4.5.1** and **9.15.4.5.2** should be multiplied by the destabilizing factors given below and then superimposed as follows:

a) factor for effects from **9.15.4.5.1**:

$$i_1 = 1 + \frac{\ell_i}{L_f} \frac{\sqrt{I_{be,\ min.}}}{3\sqrt{I_{be}} - \sqrt{I_{be,\ min.}}}$$

b) factor for effects from **9.15.4.5.2**:

$$i_2 = \frac{3\sqrt{I_{be}}}{3\sqrt{I_{be}} - \sqrt{I_{be,\ min.}}}$$

where

ℓ_i	$= B\left(\frac{24 I_f a}{K I_{be}}\right)^{0.25}$
L_f	is the length of compression flange of the main beam between the points of contraflexure;
$I_{be,\ min.}$	is the minimum value of I_{be} to satisfy **9.15.3.2**;
I_{be}, B, a, I_f, K	are as defined in **9.15.3.2**.

NOTE i_1 and i_2 may be conservatively taken as $(2L_f + \ell_i)/2L_f$ and 1.5, respectively.

9.15.5 ***Strength of transverse members***

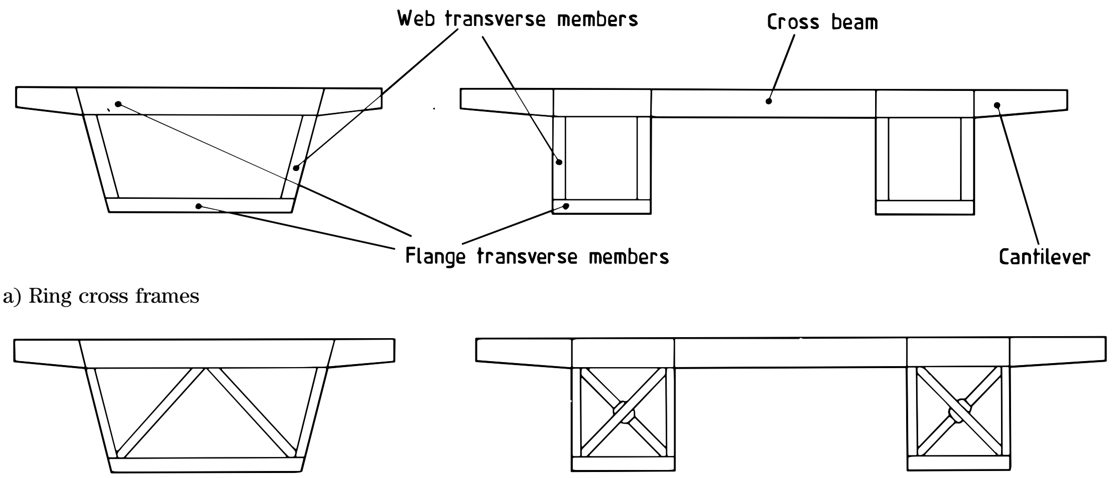
The capacity of a transverse member, or the limiting stress in the member under the action of the loadings, applied as in **9.15.4**, should be determined from **9.9** or **9.10** and **9.11**, as appropriate, using the effective section as defined in **9.15.2.2**.

9.16 Intermediate internal cross frames in box girders

9.16.1 *General*

Clause **9.16** applies to ring or braced intermediate cross frames provided in box girders to restrict distortion of the cross-section or to control the effects due to distortional warping (see Figure 31), subject to the limitations and provisions of **9.16.2**.

NOTE The design of plated intermediate diaphragms is not covered by this part of BS 5400, but they may be used provided special analysis is undertaken.



a) Ring cross frames

b) Braced cross frames

Figure 31 — Internal intermediate cross frames in box girders

9.16.2 ***Limitations***

9.16.2.1 *Girder layout*

Girders should be nominally symmetric (i.e. ignoring cross fall or superelevation), of rectangular or trapezoidal cross-section, and with webs in single planes inclined at less than 45° from the vertical.

Girders should be of single cell form, with or without interconnecting cross members and cantilevers.

Girders should not be subject to internal pressure effects due to sealing.

9.16.2.2 *Cross frames*

The cross frames should be in a single plane, within ±5° of the normal to the axis of the girder in elevation, within ±10° in plan, and within ±5° of a vertical plane.

9.16.2.3 *Corner stiffening*

Each corner of the cross frame (i.e. at the intersections of flange and web of main box) should be stiffened as necessary to withstand the bending moments acting at the corner. In the absence of such stiffening, no effective width of box web or flange (see **9.16.4.1**) should be assumed to act with the cross frame at this point.

9.16.2.4 *Stiffeners*

Where distortion due to torque applied to a girder between cross frames is to be resisted by the box walls, the cross frames should have dimensionless stiffness of at least that defined in **B.3.4.3**.

9.16.3 ***Load effects to be considered***

The design should be such as to resist the load effects given in **9.13.3** and **9.15.4** with the relevant loads applied as described in **9.13.4** and **9.15.4.5**.

NOTE The distortional effects to be considered may include those due to torque applied either at the cross frame, or at intermediate positions between the cross frame and the adjacent cross frames or diaphragms.

9.16.4 ***Ring cross frames***

9.16.4.1 *Effective section of transverse members*

When calculating the stiffness of a flange transverse member forming part of a ring frame, its effective section should be determined in accordance with **9.15.2.1**.

When calculating flexural stresses or moment capacity in the plane of the frame, the effective section of a flange transverse member should be determined in accordance with **9.15.2.2**.

When determining the effective section of a flange transverse member forming part of a ring frame for the purposes of calculating axial stresses or axial load capacity:

a) a width of flange plate equal to 16 times the flange plate thickness (if available) should be taken to act each side of the web of the transverse member, when the axial effect is compressive; or

b) the effective section should be determined in accordance with **9.15.2.2** when the axial effect is tensile.

When determining the effective section of a web transverse member forming part of a ring frame, the effective section should be determined in accordance with **9.13.2** for all purposes.

9.16.4.2 *Analysis*

Global load effects and stresses in ring cross frames should be derived from analysis undertaken in accordance with **9.4.1**, using the effective section properties described in **9.16.4.1**, and incorporating the local load effects referred to in **9.16.3**.

The effects of restraint of distortional warping of the box walls may be ignored in the analysis of distortional bending stresses in the frames.

9.16.4.3 *Strength of ring cross frame members*

Components of ring cross frames attached to the web or flange plates of main beams should satisfy the strength provisions of **9.13.5**, **9.13.6** and **9.15.5**, as appropriate.

9.16.5 ***Braced cross frames***

9.16.5.1 *General*

Braced cross frames should be designed to satisfy the provisions of **9.16.4**, **9.16.5.2** and **9.16.5.3**.

9.16.5.2 *Load effects to be considered*

Additional account should be taken of bending moments induced by eccentricities at connections, incomplete triangulation and application of loading other than at points of intersection between members.

9.16.5.3 *Design of braced cross frame members*

The design of compression and tension members of braced cross frames, not attached to the web or flange plate of main beams, should be in accordance with clauses **10** and **11**, respectively.

9.17 Diaphragms in box girders at supports

9.17.1 ***General***

Diaphragms should be provided at supports of box girders to transfer applied loads to the bearings. Subject to the limitations and provisions of **9.17.2**, unstiffened and stiffened diaphragms should be designed in accordance with **9.17.5** and **9.17.6**, respectively, on the basis of the loadings and effective sections given in **9.17.3** and **9.17.4**, respectively.

The diaphragm/web junctions should meet the provisions of **9.17.7**. Deck cross beams and/or cantilevers supporting the deck and located in the plane of a diaphragm should meet the provisions of **9.17.8**.

The geometric notation used is shown in Figure 32.

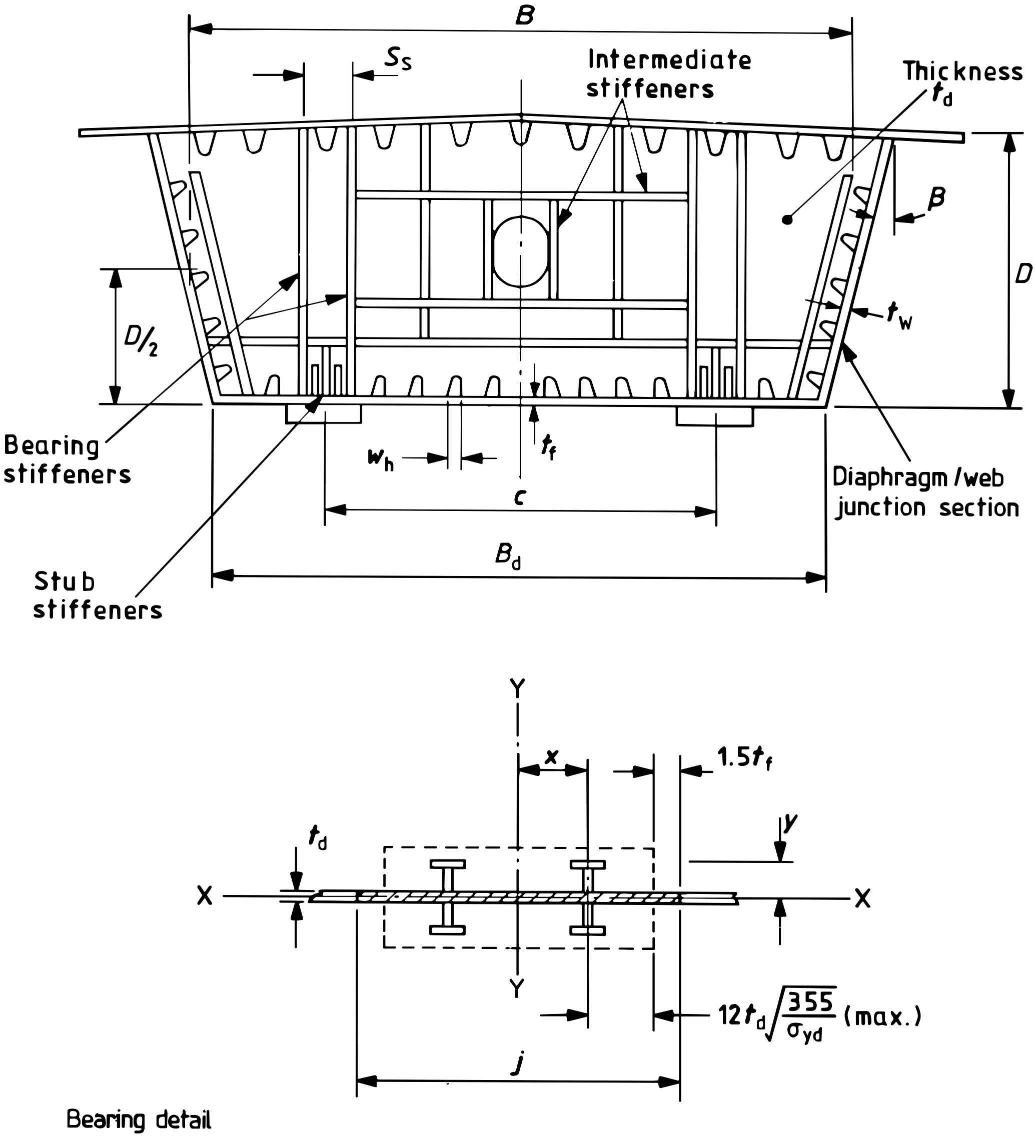


Figure 32 — Geometric notation for diaphragms

9.17.2 *Limitations*

9.17.2.1 *Box girders*

Box girders should be of nominally rectangular cross-section or of nominally trapezoidal cross-section with webs in single planes inclined at less than 45° from the vertical, and when unstiffened, should be nominally symmetrical about a vertical axis (i.e. ignoring cross fall or superelevation).

Box girders should be of a single cell form with or without interconnecting cross members or cantilevers and should not be subject to internal pressure effects due to sealing.

9.17.2.2 *Diaphragms and bearings*

The plane of the diaphragm should be within ±5° to the normal to the axis of the girder in elevation, within ±10° in plan, and within ±5° of a vertical plane.

The diaphragm should be in a single plane, except as permitted in **9.17.2.4** for starter plates.

Each diaphragm should be supported on a single bearing or on twin bearings under each box.

Bearings under unstiffened diaphragms should be symmetrically placed about the vertical axis of the diaphragm.

The contact width j of a stiffened diaphragm above a bearing, as defined in Figure 32, should not exceed half the depth of the diaphragm with a single bearing nor one-quarter of the depth of the diaphragm with twin bearings.

A bearing below a stiffened diaphragm should not extend across the width of the diaphragm beyond the line of attachment of a bearing stiffener by more than $12\sqrt{355/\sigma_{yd}}$ times the thickness of the diaphragm plate, where σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**.

9.17.2.3 *Cross beams and cantilevers*

Where the deck projects beyond the box web and is supported on cross beams and/or cantilevers which are in the plane of the diaphragm, the flanges of such members should provide a continuous load path through each box web and across the diaphragm for the forces they are required to carry. These members should be assumed to be supported by the diaphragm/box web junctions (see **9.17.7** and **9.17.8**).

9.17.2.4 *Starter plates*

Where starter plates are to be used to connect a diaphragm to the box walls, they should either be:

a) positioned in the plane of the diaphragm and butt-welded or connected by double cover plates to the diaphragm; or

b) lap jointed to the diaphragm, provided that a suitable system of stiffening is designed to withstand, in addition to any other load effects, all moments resulting from the eccentricity of connection.

9.17.2.5 *Stiffeners to diaphragms*

All stiffeners to plate diaphragms should be in accordance with **9.3.4**, with b taken as the spacing of stiffeners, or the distance between the stiffener and the box wall, as appropriate. For boxes with sloping walls, the distance between the stiffener and the box wall should be taken at the centre of the length of the stiffener between points of effective restraint.

Bearing stiffeners should be symmetrically placed about the diaphragm plate, unless a special analysis is made of the effects of eccentricity with respect to that plate.

9.17.2.6 *Plating in diaphragms*

The thickness of plating in an unstiffened diaphragm should be uniform throughout.

9.17.2.7 *Openings in unstiffened diaphragms*

Openings in unstiffened diaphragms should be in accordance with the following:

a) only one circular opening may be provided on each side of the vertical centreline of the diaphragm within the upper-third of the height of the diaphragm;

b) the diameter of any such opening should not exceed the least of:

1) $6t_d$

2) $D/20$ or

3) $B/20$

where

t_d is the diaphragm plate thickness;

D is the depth of the diaphragm (see Figure 32);

B is the width of the diaphragm taken as the average of the widths at the top and bottom flange levels for boxes with sloping webs;

c) cut-outs for longitudinal stiffeners on the box walls should have the stiffeners connected to the diaphragm plate by either:

1) welding, along at least one-third of the perimeter of the cut-out; or

2) cleating to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by the full perimeter welding of the cleat.

In addition, the length of the free edge of any cut-out should not exceed $8t_d\sqrt{355/\sigma_{yd}}$, when any part of this free edge is within a distance $10t_d\sqrt{355/\sigma_{yd}}$ from any part of the bearing plate,

where

σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**;

t_d is the diaphragm plate thickness.

9.17.2.8 *Openings in stiffened diaphragms*

Openings in stiffened diaphragms should be in accordance with the following.

a) With the exception of openings permitted in item d), openings should not be positioned within the areas shown shaded in Figure 33.

b) Unstiffened openings should be circular and of diameter not exceeding the least of:

1) $6t_d$, or

2) $a/20$, or

3) $b/20$

except when

$$\sigma_e \leq \frac{\sigma_{yd}}{2\gamma_m\gamma_{f3}}$$

for which the limiting diameter is twice the limits given in items 1), 2) and 3),

where

a, b are the panel dimensions;

σ_e $= \sqrt{\sigma_{d1}{}^2 + \sigma_{d2}{}^2 - \sigma_{d1}\sigma_{d2} + 3\tau^2}$;

σ_{d1}, σ_{d2}, τ are the stresses in the diaphragm plate derived in accordance with **9.17.6.2**;

σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**.

Not more than one such opening should be positioned in a single plate panel.

c) Stiffened openings should:

1) be framed on all sides by stiffeners;

2) have circular corners of radius at least one-quarter of the least dimension of the hole, with no re-entrant corners;

3) be positioned such that the distance of any edge from an adjacent wall of the box is at least 0.7 times the maximum dimension of the hole parallel to the wall, plus the distance from the wall to the tips of any cut-outs in the diaphragm for longitudinal stiffeners (see Figure 33), unless the adjacent plate is designed for secondary in-plane stresses.

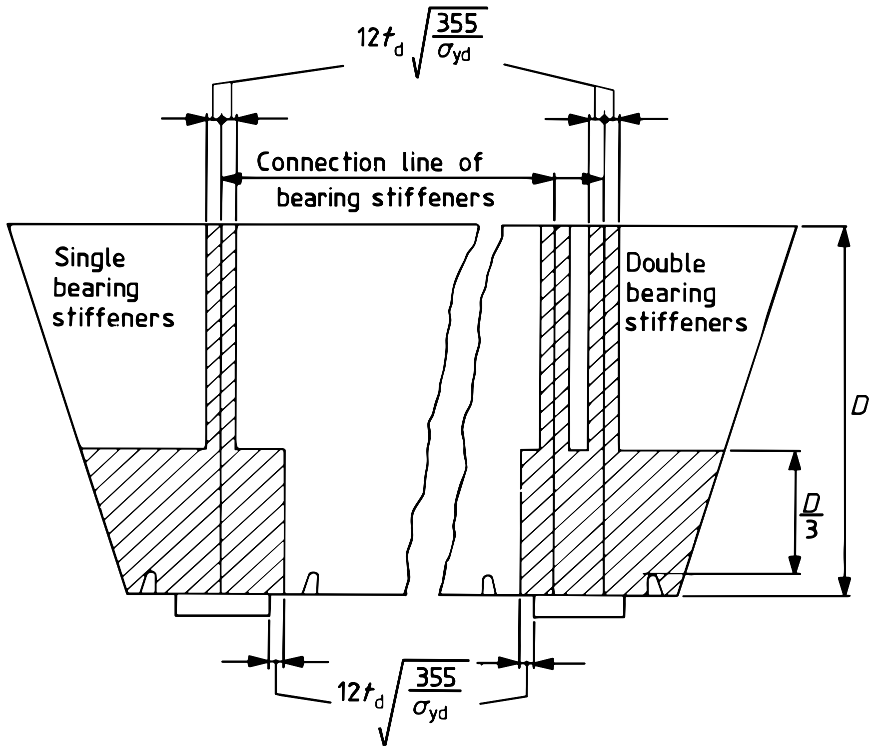
d) Cut-outs for longitudinal stiffeners should be in accordance with **9.17.2.7**c).

9.17.3 *Loading on diaphragms*

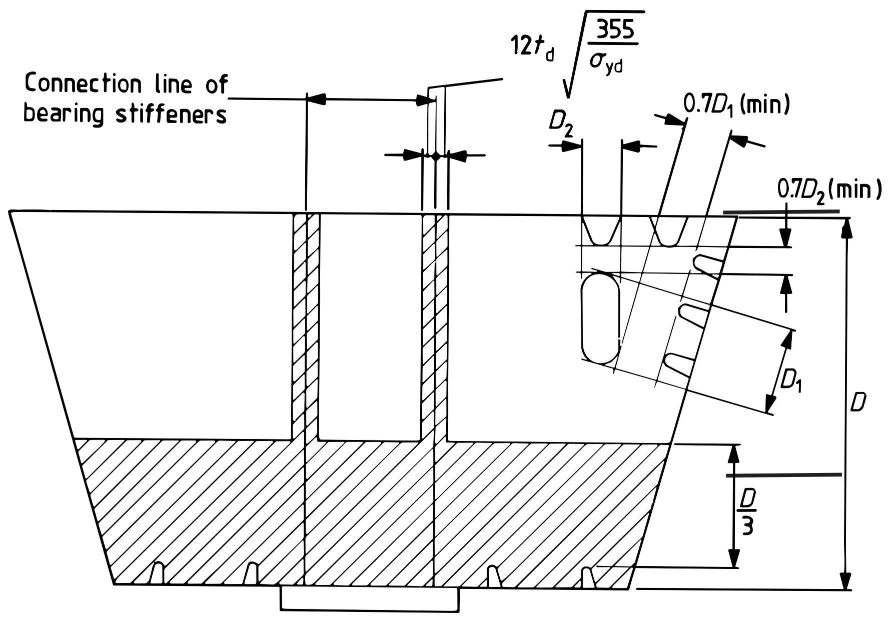
9.17.3.1 *Derivation*

The load effects in diaphragms and associated parts of box girders should be derived from global analysis undertaken in accordance with **9.4.1**.

The design methods of **9.17.5** and **9.17.6** use strength provisions that are compatible only with the assumed methods of stress derivation contained therein. Stresses derived by finite element analyses should not be substituted directly for these derived stresses.



a) Twin bearings



b) Single central bearing and positioning of large openings

NOTE 1 Openings are not permitted in shaded areas.

NOTE 2 Dimensions are taken from top of cut-outs where present.

Figure 33 — Openings in stiffened diaphragms

9.17.3.2 *Effects to be considered*

Diaphragms should be designed to resist, with due account being taken of any lack of symmetry in the cross-section or in the bearing arrangement, the combined effects of the following:

a) all externally applied loads and the associated bearing reactions;

b) changes in bearing reactions and web shears due to:

1) creep, shrinkage and differential temperature;

2) settlement and other movement of supports;

NOTE 1 Transverse effects due to item 1) may be neglected.

c) errors in installation of bearings, comprising:

1) bearing misalignment in plan;

2) errors in the level of a single bearing, or in the mean levels of more than one bearing at any support;

3) bearing inclination;

4) departure from common planarity of twin or multiple bearings;

NOTE 2 Installation errors in items 1), 2) and 3), within the tolerances given in BS 5400-9, are allowed for in **9.17.5** and **9.17.6** and their load effects need not be assessed separately.

d) changes in longitudinal slope of box flanges at the diaphragm;

e) errors in longitudinal camber in continuous construction. Allowance for this may be made by assuming, at the bearings, a vertical displacement of a support relative to two adjacent supports of 1/5 000 times the sum of the adjacent spans;

f) out-of-plane moments due to:

1) longitudinal movements of the bridge;

2) changes in slope of the bridge;

3) eccentricity due to bearing misalignment along the span or due to the shape of the bearing; the combined eccentricity for these may be taken as given in **9.14.3.3**;

4) interconnection between deck and diaphragm stiffeners;

5) any intended eccentricity of the centroidal axes of the effective section of the bearing stiffeners with respect to the diaphragm plate.

9.17.4 *Effective sections*

9.17.4.1 *General*

For determining the stresses in a diaphragm, the effective elastic section modulus and effective area of a vertical cross-section, and the effective vertical and horizontal shear areas, should be derived in accordance with **9.17.4.2** and **9.17.4.3**. For determining the stresses in stiffeners, their effective sections should be derived in accordance with **9.17.4.4** or **9.17.4.5**, as appropriate.

In **9.17.4.2** and **9.17.4.3** the references to transverse tension and compression apply to directions normal to the longitudinal axis of the girder.

9.17.4.2 *Vertical sections*

9.17.4.2.1 *General*

The determination of the effective area A_e and the effective section modulus Z_e of a vertical cross-section of a diaphragm should be based on effective areas of box flanges and diaphragm plate as given in **9.17.4.2.2** to **9.17.4.2.5**.

9.17.4.2.2 *Effective flange width*

In calculating an effective area of a box flange, an effective width w_e should be determined separately for each side of the diaphragm and should not exceed any of the following:

a) one-quarter of the distance of the section under consideration from the nearest web/flange junction; or

b) half the distance to an adjacent diaphragm or cross beam for any flange in transverse tension, or for a composite flange in transverse compression; or

c) outside an end diaphragm, the actual width of plate provided; or

d) $12t_f\sqrt{355/\sigma_{yf}}$ for a non-composite flange in transverse compression. This limit may be increased to one-quarter of the distance to an adjacent diaphragm or cross beam provided that the transverse compressive stress (using the increased width) does not exceed the lesser of:

1) one-quarter of the longitudinal compressive strength of the flange; or

2) $0.5\left(\frac{t_f}{b}\right)^2 E$

where

t_f is the thickness of the flange plate;

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

b is the spacing of the longitudinal flange stiffeners or the distance between box webs for an unstiffened flange.

9.17.4.2.3 *Effective flange area*

The effective area of a box flange should be determined as follows.

a) The effective area of steel plate on each side of the diaphragm should be taken as:

$K_c t_f w_e$

where

t_f is the flange thickness;

w_e is the effective width on the appropriate side of the diaphragm derived from **9.17.4.2.2**;

K_c is a coefficient taken as 1.0, except in the case of a non-composite flange in transverse compression with an effective width greater than $12t_f\sqrt{355/\sigma_{yf}}$, when the value of K_c should be obtained from Figure 5 with dimension a taken as the spacing of longitudinal flange stiffeners and dimension b taken as the distance from the diaphragm to an adjacent cross beam or diaphragm. In using Figure 5, the coefficient should be determined from curve 1 or curve 3, whichever gives the greater value, for diaphragms at internal supports, and from curve 2 or curve 3, whichever gives the greater value, for diaphragms at end supports.

b) Any transverse flange stiffeners within the effective width should be ignored.

c) In composite construction, the effective flange area may include the area of steel reinforcement within the total effective width, and, if subjected to transverse compression, may also include the transformed area of concrete within the total effective width.

9.17.4.2.4 *Diaphragm plate*

Holes within the vertical section of a diaphragm should be deducted. When a stiffened opening is provided, diaphragm plating extending within the framing stiffeners by more than $8t_d\sqrt{355/\sigma_{yd}}$ should be ignored, where t_d is the thickness of the diaphragm plate and σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**.

9.17.4.2.5 *Inclined webs*

In the case of box girders with inclined webs, no part of the webs should be included in the vertical section of the diaphragm.

9.17.4.3 *Shear area*

The effective vertical and horizontal shear areas, A_{ve} and A_{he}, should be taken as the net areas of a vertical and horizontal cross-section, respectively, of diaphragm plating only.

9.17.4.4 *Diaphragm stiffeners*

The effective section of a stiffener on a diaphragm should be taken to comprise the stiffener with widths of diaphragm plate on each side of the stiffener where available not exceeding the lesser of:

a) half the distance from the stiffener to an adjacent stiffener or to the wall of the box; or

b) $12\sqrt{355/\sigma_{yd}}$ times the thickness of the diaphragm plate, where σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**.

Additionally, for a bearing stiffener, the effective width of plate assumed on the side towards the web should not exceed half the distance from the stiffener to the web/bottom flange junction.

The sectional area of discontinuous diaphragm stiffeners should be ignored.

9.17.4.5 *Diaphragm/web junction*

The effective section of this part should be taken to comprise a) below, plus b), c) or d) as appropriate:

a) a width of plate each side of the diaphragm (where available) of up to 16 times the web thickness;

b) where there is a load bearing stiffener on the diaphragm, within a distance of $25t_d$ from the web: a width of diaphragm plate equal to half the distance between the web and the load bearing stiffener;

c) where there is a stiffener, other than a load bearing stiffener, on the diaphragm parallel to, and within a distance of $25t_d$ from, the web: the area of this stiffener together with a width of the diaphragm plate equal to $25t_d$;

d) when there is no stiffener on the diaphragm parallel to, and within a distance of $25t_d$ from, the web: a width of diaphragm plate equal to $12t_d\sqrt{355/\sigma_{yd}}$;

where

t_d is the thickness of the diaphragm plate;

σ_{yd} is the nominal yield stress of the diaphragm plate, as defined in **6.2**.

9.17.5 ***Unstiffened diaphragms***

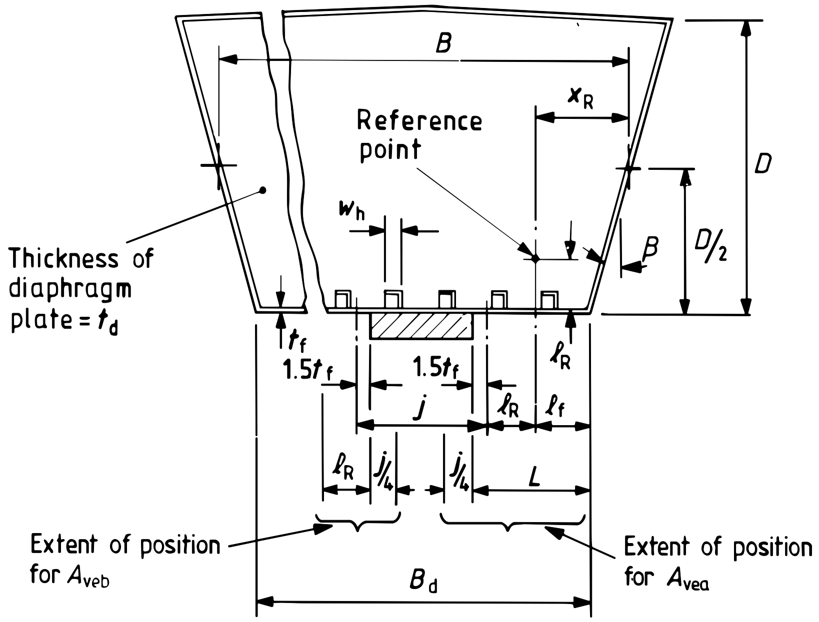
9.17.5.1 *General*

Unstiffened diaphragms in accordance with **9.17.2.1** to **9.17.2.4**, **9.17.2.6** and **9.17.2.7**, should be designed to meet the yield criterion of **9.17.5.4** and the buckling criterion of **9.17.5.5** using the reference stress values of **9.17.5.2** and the buckling coefficients of **9.17.5.3**. Web/flange junctions should additionally be in accordance with **9.17.7.3**.

9.17.5.2 *Reference values of in-plane stresses*

9.17.5.2.1 *General*

The stresses in an unstiffened diaphragm, resulting from the load effects given in **9.17.3**, should be determined at the reference point indicated in Figure 34, in accordance with **9.17.5.2.2** to **9.17.5.2.4**, for each of the appropriate reference stresses.



NOTE ℓ_R is taken as the least of:

$6t_d\sqrt{\frac{355}{\sigma_{yd}}}$, $\frac{D}{8}$ or $\frac{L}{2}$

Figure 34 — Reference point and notation for unstiffened diaphragms

9.17.5.2.2 *Vertical stresses*

The reference value of the in-plane vertical stress σ_{R1} should be taken as follows:

a) for a diaphragm with a single central bearing:

$$\sigma_{R1} = \frac{R_v(1 + 4e/t_d)}{(j - \sum w_h)t_d} + 0.77\left(\frac{T_b j}{2I_{yd}}\right)$$

b) for a diaphragm with a pair of twin symmetrical bearings:

$$\sigma_{R1} = \frac{R_v(1 + 4e/t_d)}{(j - \sum w_h)t_d}$$

where

R_v	is the total vertical load transmitted by the diaphragm to one bearing (including the effects of torque on twin bearings);
T_b	is the torsional reaction at a single central bearing;
j	is the width of contact of the bearing pad plus 1.5 times the thickness of the bottom flange at each end if available (see Figure 32);
Σw_h	is the sum of the widths of any cut-outs for stiffeners within the width j at the level immediately above the flange;
t_d	is the diaphragm thickness;
I_{yd}	is the second moment of area of the diaphragm plate of width j, excluding cut-outs, about the Y-axis (see Figure 32);
e	is the eccentricity of bearing reaction along the span (see **9.14.3.3**).

9.17.5.2.3 *Horizontal stresses*

The reference value of the in-plane horizontal stress σ_{R2} should be taken as:

$$\sigma_{R2} = \left\{\left(\frac{K_d \Sigma R_v}{2} + \frac{T}{B}\right)x_R + Q_{fv}\frac{\ell_f}{2}\right\}\frac{1}{Z_e} + \frac{\Sigma R_v \tan\beta}{2A_e}$$

where

K_d	is a factor allowing for the effects of boundary shears and should be taken as 2.0 in the absence of any special analysis;
ΣR_v	is the total vertical force transmitted by the diaphragm to the bearings;
Q_{fv}	is the vertical force transmitted to the diaphragm by the portion of the bottom flange over a width ℓ_f when there is a change of flange slope;
ℓ_f	is the horizontal distance from the reference point to the nearest edge of the bottom flange;
B	is as defined in **9.17.2.7**;
T	is the torque transmitted to the diaphragm in shear through the box walls and from cross beam and/or cantilever loading;
x_R	is the distance parallel to the bottom flange from the reference point to the web mid-point (see Figure 34);
Z_e, A_e	are the effective section modulus and the effective area, respectively, of the diaphragm and flanges at the vertical cross-section through the reference point, derived in accordance with **9.17.4.2**;
β	is the inclination of the box web to the vertical.

9.17.5.2.4 *Shear stresses*

a) Except as recommended in item b), the reference value of the in-plane shear stress τ_R should be taken as follows:

$$\tau_R = \left(\frac{\Sigma R_v}{2} + Q_{fv} + \frac{T}{2B}\right)\frac{1}{A_{ve,\,a}} + \frac{Q_h}{A_{he}}$$

where

ΣR_v, Q_{fv}, T	are as defined in **9.17.5.2.3**;
B	is as defined in **9.17.2.7**;
Q_h	is the shear force due to transverse horizontal loads on the bridge transmitted from the top flange to the diaphragm;
$A_{ve,\,a}$	is the minimum value of the effective vertical shear area, as given in **9.17.4.3**, for any section of diaphragm plating taken between the web and a point j/4 inside the outer edge of the bearing (see Figure 34);
j	is as defined in **9.17.5.2.2**;
A_{he}	is the effective horizontal shear area, as given in **9.17.4.3** for the section of diaphragm plating through the reference point.

b) In addition, in the case of diaphragms on twin symmetrical bearings where there is a change in slope of the bottom flange, an alternative value τ_{Rf} should be derived from:

$$\tau_{Rf} = \left(\frac{T}{c} + \frac{Q_{bv}}{2} - \frac{T}{2B}\right)\frac{1}{A_{ve,\,b}} + \frac{Q_h}{A_{he}}$$

where

T is as defined in **9.17.5.2.3**;

Q_{bv} is the total vertical force transmitted to the diaphragm by the portion of the bottom flange between the inner edges of the bearings when there is a change in flange slope;

c is the distance between centres of bearings;

$A_{ve,\ b}$ is the minimum value of the effective vertical shear area, as given in **9.17.4.3**, for any section of diaphragm plating taken within a distance ℓ_R from the inner edge of a bearing (i.e. towards the diaphragm centreline) and a distance $j/4$ inside the same inner edge of the bearing (see Figure 34);

ℓ_R is as defined in Figure 34.

This value of τ_{Rf} should be adopted if it exceeds the value of τ_R determined in a).

9.17.5.3 *Buckling coefficient*

In checking the adequacy of an unstiffened plate diaphragm, a coefficient K is required which is given by:

$$K = K_1 K_2 K_3 K_4$$

where

$$K_1 = 3.4 + \frac{2.2D}{B_d}$$

$$K_2 = 0.4 + \frac{j}{2B_d} \text{ for single central bearings}$$

$$= 0.4 + \frac{c - j/3}{B_d} \text{ for twin bearings}$$

$$K_3 = 1.0 - \frac{\beta}{100}$$

$$K_4 = 1.0 - \frac{fP_d}{\Sigma R_v + T/\ell_b}\left(\frac{2B}{B_d} - 1\right)$$

D, B_d, B are as defined in Figure 34;

β (in degrees) is as defined in Figure 34;

j is as defined in **9.17.5.2.2**;

f = 0.55 when $D/B \leq 0.7$, or
= 0.86 when $D/B \geq 1.5$ with intermediate values found by linear interpolation;

ℓ_b = $j/2$ for single central bearings;
= c for twin bearings;

ΣR_v, T are as defined in **9.17.5.2.3**;

c is the distance between centres of bearings;

P_d $= W_d + \Sigma\left(\frac{P}{K_5}\right)$;

W_d is the total uniformly distributed load applied to the top of the diaphragm;

P is any local load applied to the top of the diaphragm;

K_5 $= 0.4 + \frac{w}{2B - B_d}$;

w is the actual width of the load P, plus an allowance for the dispersal through a concrete flange at an angle of 45° to the vertical, and through a steel flange at an angle of 60° to the vertical.

9.17.5.4 *Yielding of diaphragm plate*

The value of σ_{R1} and $\sqrt{\sigma_{R2}^2 + 3\tau_R^2}$ should not exceed the lesser of:

$$\frac{\sigma_{yd}}{\gamma_m \gamma_{f3}}, \text{ or}$$

$$\frac{\sigma_{yd}}{\gamma_m \gamma_{f3}} \left\{ 1.2 - \frac{(\Sigma R_v + T/\ell_b)D}{1.25KEt_d^{\ 3}} \right\}$$

where

σ_{R1}, σ_{R2}, τ_R	are the reference values of stress as derived in **9.17.5.2.2**, **9.17.5.2.3** and **9.17.5.2.4**, respectively;
ΣR_v, T	are as defined in **9.17.5.2.3**;
ℓ_b, K	are as derived in **9.17.5.3**;
D	is as defined in Figure 34;
t_d	is the thickness of the diaphragm plate;
σ_{yd}	is the nominal yield stress of the diaphragm plate as defined in **6.2**.

9.17.5.5 *Buckling of diaphragm plate*

The value of $\Sigma R_v + T/\ell_b$ should not exceed:

$$\frac{0.7KEt_d^3}{D\gamma_m \gamma_{f3}}$$

where

ℓ_b, K	are as derived in **9.17.5.3**;
t_d	is the thickness of the diaphragm plate;
D	is as defined in Figure 34;
ΣR_v, T	are as defined in **9.17.5.2.3**.

9.17.6 *Stiffened diaphragms*

9.17.6.1 *General*

Diaphragms in accordance with **9.17.2.1** to **9.17.2.6** and **9.17.2.8** and stiffened by an orthogonal system of stiffeners, generally as indicated in Figure 32, should be designed such that the diaphragm plate meets the yield criterion of **9.17.6.4** and the buckling criterion of **9.17.6.5**, using the appropriate stresses determined from **9.17.6.2**.

In addition, all types of stiffeners, as defined in a), b) and c) below, should be designed such that they meet the yield criterion of **9.17.6.6** and the buckling criterion of **9.17.6.7**, using the appropriate stresses determined from **9.17.6.3**.

Web/flange junctions should, additionally, be in accordance with **9.17.7.3** and **9.17.7.4**.

Stiffening may consist of (see Figure 32):

a) bearing stiffeners, which span from a box flange immediately above a bearing, to the flange at deck level;

b) stub stiffeners, which are short vertical stiffeners above bearings;

c) intermediate stiffeners, which may be either primary or secondary. Stiffeners spanning between box walls or, if horizontal, between a box web and bearing stiffener, or between bearing stiffeners, should be treated as primary. All other stiffeners should be treated as secondary.

9.17.6.2 *Stresses in diaphragm plates*

9.17.6.2.1 *General*

Relevant stress components should be calculated at the corners of each plate panel, using the appropriate section properties obtained from **9.17.4**, in accordance with **9.17.6.2.2** to **9.17.6.2.4**. When secondary bending stresses have been calculated in accordance with **9.17.2.8**c)3), they should be added to these components.

9.17.6.2.2 *Vertical stresses*

Vertical stresses σ_{d1} may be neglected with the exception of those due to:

a) a change in slope of the main girder flange; and

b) local wheel loads applied above the diaphragm, which should be calculated in accordance with **9.11.4.1**.

9.17.6.2.3 *Horizontal stresses*

Horizontal stresses σ_{d2} should be calculated under the action of the following:

a) the in-plane primary moment M on the diaphragm, which should be taken as:

$$M = (K_d Q_v + 2Q_T)x_w + K_d Q_c x_c + \sum_{i=1}^{n} (P_i x_i) - R_v x_b + \left(\frac{Q_{fv} \ell_f}{2}\right)$$

where (as shown in Figure 35)

Q_v is the vertical component of the symmetric shear transmitted into the diaphragm from one web;

Q_T is the vertical component of torsional shear transmitted into the diaphragm from one web, given by $T/2B$, where B is as defined in **9.17.2.7**;

NOTE In calculating the effects in plate panels occurring in a region bounded by an inclined girder web, a girder top flange and a vertical line passing through the bottom flange/web junction, an appropriate portion of Q_V and Q_T should be taken on the assumption that they are uniformly distributed over the depth of the girder web.

x_w is the horizontal distance from the section under consideration to the mid-point of the web;

Q_c is the vertical component of any cross beam or cantilever shear;

x_c is the horizontal distance from the section under consideration to the root of the cross beam or cantilever;

P_i is a locally applied deck load between the section under consideration and the web;

x_i is the horizontal distance from the section under consideration to the locally applied deck load P_i;

R_v is the total vertical load transmitted to one bearing by the diaphragm;

x_b is the distance from the section under consideration to the inner edge of the nearest bearing plus $j/4$, for sections between twin bearings, or is zero for all other sections and for diaphragms with a single bearing;

K_d, Q_{fv}, ℓ_f are as defined in **9.17.5.2.3**;

j is as defined in **9.17.5.2.2**.

The horizontal bending stress σ_{2b} should be taken as:

$$\sigma_{2b} = \frac{M}{Z_e}$$

where

Z_e is the effective section modulus of a vertical cross-section of the diaphragm and flanges at the point under consideration, derived in accordance with **9.17.4.2**;

b) the horizontal component of girder shear when the webs are inclined. The horizontal stress σ_{2q} from this component should be taken as:

$$\sigma_{2q} = \frac{Q_v \tan \beta}{A_e}$$

where

Q_v is as defined in a);

A_e is the effective area of a cross-section of the diaphragm and flanges, at the point under consideration, derived in accordance with **9.17.4.2**;

β is the inclination of the box web to the vertical.

The total horizontal stress σ_{d2} at the point under consideration should be taken as:

$$\sigma_{d2} = \sigma_{2b} + \sigma_{2q}$$

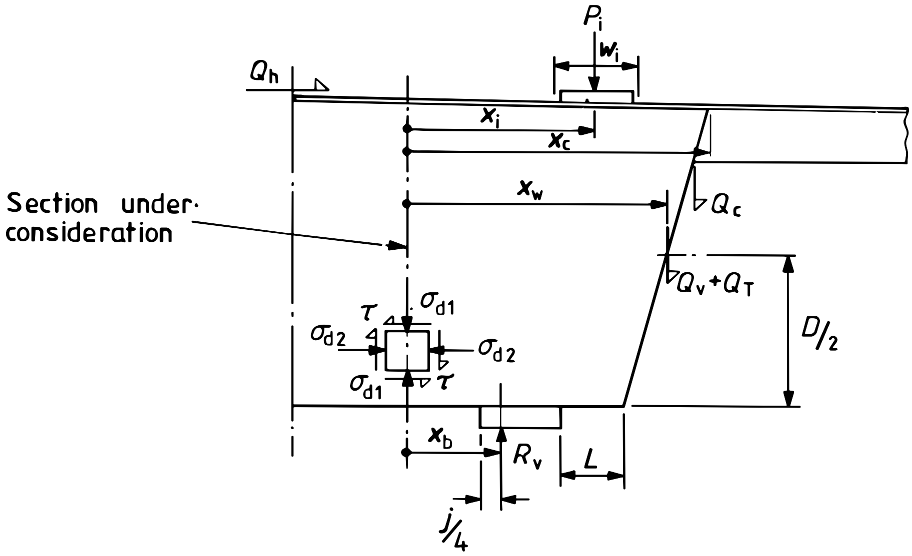


Figure 35 — Load effects and notation for stiffened diaphragms

9.17.6.2.4 *Shear stresses*

Shear stresses should be calculated under the action of the shear flow q at the section of the diaphragm under consideration. This shear flow q should be taken as constant over the net depth or width of the diaphragm, and should be calculated as follows:

a) in sections between a box web and an outer bearing stiffener:

$$q = \frac{Q_v + Q_T + Q_{fv} + Q_c + \Sigma P_i}{D_e} + \frac{Q_h}{B_e};$$

b) in sections between inner bearing stiffeners where there are twin bearings:

$$q = \left(\frac{Q_v}{4} + \frac{Q_{bv}}{2} + \frac{T}{c} - Q_T\right)\frac{1}{D_e} + \frac{Q_h}{B_e};$$

c) in sections between pairs of bearing stiffeners above one of a pair of bearings:

$$q = \left(\frac{5Q_v}{8} + \frac{T}{2c}\right)\frac{1}{D_e} + \frac{Q_h}{j - \Sigma w_h};$$

up to the level of the tops of any longitudinal bottom flange stiffeners above the bearing or to a height j in the absence of cut-outs; and

$$q = \left(\frac{5Q_v}{8} + \frac{T}{2c}\right)\frac{1}{D_e} + \frac{Q_h}{B_e};$$

above that level;

d) in sections between pairs of bearing stiffeners above a single bearing:

$$q = \left(\frac{Q_v}{4} + \frac{T}{s_s} - Q_T\right)\frac{1}{D_e} + \frac{Q_h}{j - \Sigma w_h}$$

up to the tops of any longitudinal stiffener, bottom flange stiffener cut-outs above the bearing or to a height j in the absence of cut-outs;

$$q = \left(\frac{Q_v}{4} + \frac{T}{s_s} - Q_T\right)\frac{1}{D_e} + \frac{Q_h}{B_e};$$

above that level;

where

Q_v, Q_T, Q_c, P_i	are as defined in **9.17.6.2.3**;
Q_{fv}, T	are as defined in **9.17.5.2.3**;
Q_h, Q_{bv}, c	are as defined in **9.17.5.2.4**;
D_e, B_e	are the net depth and width of the diaphragm at the point under consideration;
j, Σw_h	are as defined in **9.17.5.2.2**;
s_s	is the distance between stiffener centroids.

The shear stress τ in the sections referred to in a), b), c) or d) should be taken as:

$$\tau = \frac{q}{t_d}$$

where

t_d is the thickness of the diaphragm plate in the panel under consideration.

In sections other than those referred to in a), b), c) or d), τ may be neglected.

9.17.6.3 *Stresses in diaphragm stiffeners*

9.17.6.3.1 *General*

Stresses in stiffeners should be determined in accordance with **9.17.6.3.2** to **9.17.6.3.4**, using the appropriate effective stiffener sections obtained from **9.17.4.4**.

The stiffener types: bearing, stub, primary intermediate and secondary intermediate, are as defined in **9.17.6.1**.

9.17.6.3.2 *Vertical stresses in bearing stiffeners*

The vertical stress σ_{1s} in a bearing stiffener should be taken as:

$$\sigma_{1s} = \frac{P_s}{A_{se}}$$

where

P_s is the total vertical force in the group of bearing stiffeners;

A_{se} is the effective cross-sectional area of the group of bearing stiffeners, derived in accordance with **9.17.4.4**.

NOTE 1 Both values are taken at the level under consideration.

In the absence of openings in the diaphragm between the group of bearing stiffeners and the adjacent web, the vertical force P_s may be assumed to vary linearly from the value of the reaction at the bearing to the value of any reaction transmitted from the deck to the top of the bearing stiffener.

If there are any openings in the diaphragm between the group of bearing stiffeners and the adjacent web, no variation of load over the depth of such openings should be assumed. The variation over the remaining parts of the diaphragm should be assumed to be linear, of constant slope.

In the case of a diaphragm above a single bearing, an additional vertical stress σ_{1sT} in a bearing stiffener should be taken as:

$$\sigma_{1sT} = \frac{T_s x}{I_{yse}}$$

where

T_s is the value of the moment in the plane of the diaphragm on the group of bearing stiffeners;

x is the horizontal distance of the stiffener under consideration from the centroidal axis, normal to the plane of the diaphragm, of the stiffener group (see Figure 32);

I_{yse} is the effective second moment of area of the stiffener group about the same centroidal axis, derived in accordance with **9.17.4.4**.

NOTE 2 All values are taken at the point under consideration. T_s may be assumed to vary linearly, from the torsional reaction above the bearing, to zero at the top flange level.

Where stub stiffeners are used, the stress calculated as above may be reduced locally by including the area of such stiffeners, provided their connections to the diaphragm plate are adequate to transfer their share of the bearing reaction.

9.17.6.3.3 *Bending stresses in bearing stiffeners*

The bending stress σ_{bs} in a bearing stiffener due to an out-of-plane moment should be taken as:

$$\sigma_{bs} = \frac{M_s y}{I_{xse}}$$

where

M_s is the proportion of the out-of-plane moment carried by the group of bearing stiffeners;

y is the distance of the extreme fibre of the stiffener under consideration from the centroidal axis, parallel to the plane of the diaphragm, of the stiffener group (see Figure 32);

I_{xse} is the effective second moment of area of the stiffener group about the same centroidal axis, derived in accordance with **9.17.4.4**.

NOTE All values are taken at the point under consideration.

A proportion of the out-of-plane moment may be assumed to be carried by the flange longitudinal stiffeners, provided due account is taken of this in their design.

Stub stiffeners should not be considered to carry any part of the out-of-plane moment carried by a bearing stiffener group unless they have an adequate out-of-plane shear connection to the bearing stiffeners and/or the box walls.

9.17.6.3.4 *Equivalent stress for buckling check*

The equivalent axial stress σ_{se}, to be used in the buckling check of all stiffeners, should be taken as the maximum value within the middle third of the length ℓ_s of the stiffener, calculated from:

$$\sigma_{se} = \sigma_a + \frac{1}{A_{se}}\left\{\frac{\sigma_q \ell_s^{\,2} t_d k_s}{a}\left(1 + \frac{\Sigma A_s}{\ell_s t_d}\right) + \tau_h t_d h_h\right\}$$

where, for all stiffeners:

A_{se}	is the effective cross-sectional area of the stiffener derived in accordance with **9.17.4.4**;
ℓ_s	is the length of the stiffener between points of effective restraint;
t_d	is the thickness of the diaphragm plate;
k_s	is obtained from Figure 24 using the slenderness parameter: $\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$
r_{se}	is the radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm, derived in accordance with **9.17.4.4**;
σ_{ys}	is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material;
ΣA_s	is the sum of the cross-sectional areas of all stiffeners which intersect the stiffeners being designed within the length ℓ_s not including any adjacent diaphragm plate;
σ_{d2}	is as derived in **9.17.6.2.3**, for the level being considered, and is taken as positive when compressive;
σ_{2s}	is the average value of σ_{d2} within the middle third of the length ℓ_s;
a	is one half of the sum of the panel widths on either side of the stiffener. Where widths vary over the length ℓ_s the average value within the middle third should be used;

σ_a, σ_q, τ_h, h_h are as follows for the appropriate type of stiffener.

For bearing stiffeners:

σ_a	$= \sigma_{1s} + \sigma_{1sT}$;
σ_{1s}, σ_{1sT}	are as derived in **9.17.6.3.2**;
σ_q	$= \sigma_{2s}$;
τ_h, h_h	are taken as zero.

For all intermediate stiffeners:

τ	is the average shear stress in the panels on either side of the stiffener;
τ_h	is zero except in the case of the stiffeners framing openings, where τ_h is the shear stress which would occur in the plating adjacent to the stiffener if the opening had been fully plated;
h_h	is zero except in the case of the stiffeners framing openings, where h_h is the dimension of the opening parallel to the stiffener.

NOTE In calculating a and σ_q no account should be taken of any opening in the diaphragm adjacent to the stiffener (i.e. it should be assumed that a plate of thickness t_d fills the opening).

For horizontal intermediate stiffeners only:

$\sigma_a = \sigma_{d2}$

$\sigma_q = \tau$

For vertical intermediate stiffeners only:

$\sigma_a = 0$

$$\sigma_q = \tau + \sigma_{2s} + \frac{\sigma_{2b,\ max.} - \sigma_{2b,\ min.}}{12}$$

$\sigma_{2b,\ max.}$, $\sigma_{2b,\ min.}$ are the maximum and minimum values of σ_{2b}, derived as in **9.17.6.2.3**, within the length ℓ_s and are taken as positive when compressive.

For bearing stiffeners and for vertical intermediate stiffeners the stress σ_q may be factored by η_s where:

$$\eta_s = \frac{1}{1 + \left(0.5\ell_s^{\,3} / a^3 I\right)\left\{\Sigma I_s + \left(\ell_s t_d^{\,3} / 12\right)\right\}}$$

where

ΣI_s is the sum of the second moments of area of the effective stiffener section of all continuous stiffeners, if any, within the depth ℓ_s derived in accordance with **9.17.4.4**;

I is the second moment of area of the effective section of the vertical stiffener.

9.17.6.4 *Yielding of diaphragm plate*

Plate panels between stiffeners, or between stiffeners and the box walls, should be designed such that at all points in every panel:

$$\sigma_{d1}^2 + \sigma_{d2}^2 - \sigma_{d1}\sigma_{d2} + 3\tau^2 \leq \left(\frac{\sigma_{yd}}{\gamma_m \gamma_{f3}}\right)^2$$

where

σ_{d1} $= \sigma_{1s} + \sigma_{1sT}$ for parts of plate panels forming part of the effective section of any bearing stiffener, or is the vertical in-plane stress due to local deck loads and change in flange slope, if relevant, for all remaining parts of plate panels;

σ_{1s} is as defined in **9.17.6.3.2**;

σ_{1sT} is as derived in **9.17.6.3.2**, but with the value of x in that clause taken as the dimension from the centroidal axis to the extreme fibre of the effective section of the stiffener group;

σ_{d2} is as defined in **9.17.6.2.3**;

τ is as defined in **9.17.6.2.4**;

σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**.

9.17.6.5 *Buckling of diaphragm plate*

9.17.6.5.1 Plate panels need not be checked for buckling provided that:

a) the cross-section of the girder is nominally rectangular;

b) the ratio of the depth of the diaphragm D to the minimum plate thickness t_d is less than $80\sqrt{\frac{355}{\sigma_{yd}}}$;

c) the overhang L (see Figure 34 or 35) from the outer edge of the bearing to the box web is less than $D/2$;

d) stiffening is limited to the bearing stiffeners themselves, and any member providing continuity of cross beam or cantilever flanges through the diaphragm;

e) there is no change in flange slope at the diaphragm.

9.17.6.5.2 If any of the provisions of **9.17.6.5.1** are not satisfied, all plate panels should meet the buckling criterion given in **9.11.4**, but with the following qualifications.

a) For panels adjacent to an inclined web the panel dimension a should be taken as the maximum horizontal dimension of the panel.

b) A plate panel of non-constant thickness should be assumed to be of its minimum thickness throughout.

c) All plate planes adjacent to the box webs or flanges or to boundary stiffeners not more than $25t_d$ from the box walls or to large cut-outs should be treated as unrestrained. Other panels may be treated as restrained.

d) For a plate panel without horizontal stiffeners, bounded on three sides by the main beam web and the two main beam flanges, the shear stress coefficient K_q should not be taken higher than:

$$\frac{1.3 \times 10^6}{\sigma_{yd}} \left(\frac{t_d}{b}\right)^2 \left\{1 + \left(\frac{b}{a}\right)^2\right\}$$

where

a, b are the length and width of the panel, respectively;

t_d is the thickness of the diaphragm plate;

σ_{yd} is the nominal yield stress of the diaphragm plate as defined in **6.2**.

e) σ_{d2} should be taken as the main longitudinal stress in the plate panel, and hence for the purposes of meeting the buckling criterion of **9.11.4**, σ_{d1} and σ_{d2} as derived in **9.17.6.4** should be taken as σ_2 and σ_1 respectively, in **9.11.4**.

9.17.6.6 *Yielding of diaphragm stiffeners*

A bearing stiffener section should be designed such that, at any point along its length:

$$\sigma_{1s} + \sigma_{1sT} + \sigma_{bs} \leq \frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{1s}, σ_{1sT} are as defined in **9.17.6.3.2**;

σ_{bs} is as defined in **9.17.6.3.3**;

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material.

The bearing stress at the point of contact with a flange should be in accordance with **9.14.4.2**.

9.17.6.7 *Buckling of the diaphragm stiffeners*

The stiffener section should be such that, at any point within the middle third of the length of the stiffener:

$$\frac{\sigma_{se}}{\sigma_{\ell s}} + \frac{\sigma_{bs}}{\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{se} is as defined in **9.17.6.3.4**;

σ_{bs} is as defined in **9.17.6.3.3** for a bearing stiffener, or is taken as zero for an intermediate stiffener;

$\sigma_{\ell s}$ is obtained from Figure 24 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}};$$

σ_{ys} is the nominal yield stress value, as defined in **9.3.1**, for the stiffener material;

ℓ_s is the length of the stiffener between points of effective restraint;

r_{se} is the radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm derived in accordance with **9.17.4.4**.

9.17.7 ***Diaphragm/web junctions***

9.17.7.1 *General*

The diaphragm/web junction should be designed as a stiffener to the box web, spanning between box flanges, unsupported in the plane of the diaphragm and with an effective section derived as in **9.17.4.5**.

9.17.7.2 *Loading effects to be considered*

The junction should withstand the effects of the following:

a) all loads transmitted to the diaphragm from the cross beams and/or cantilevers in the plane of the diaphragm. Such loads should be assumed to be applied at the centroidal axis of the effective section, and to vary linearly from a maximum at the top of the junction, to zero at the bottom;

b) any forces resulting from tension field action in the adjacent web panels (see **9.13.3.2**). Such forces should be assumed to be applied in the plane of the box web, and to be constant over the height of the junction;

c) an axial force representing the destabilizing influence of the web (see **9.14.3.2**). This force should be assumed to be applied at the centroidal axis of the effective section, and to be constant over the height of the junction.

9.17.7.3 *Strength of diaphragm/web junction*

9.17.7.3.1 The maximum stress at any point on the cross-section of the junction, at any section in its length, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the junction section as defined in **6.2**.

9.17.7.3.2 The effective junction section should be such that:

$$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M}{Z_{se}\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P, M are, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects listed in **9.17.7.2**, within the middle third of the length of the junction;

A_{se} is the effective area of the junction section (see **9.17.4.5**);
Z_{se} is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (see **9.17.4.5**);

$\sigma_{\ell s}$ is obtained from Figure 24 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}};$$

ℓ_s is the total length of the junction section;
r_{se} is the radius of gyration of the effective junction section about its centroidal axis parallel to the web derived in accordance with **9.17.4.5**;
σ_{ys} is the nominal yield stress of the junction section as defined in **6.2**.

9.17.7.4 *Junction restraint provided by diaphragm stiffeners*

Diaphragm/web junctions should be designed in accordance with **9.17.7.1** to **9.17.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress σ_{se} in such stiffeners (see **9.17.6.3.4**) is increased by an amount equal to:

$$\frac{0.025P}{nA_{se}}$$

where

- P is as defined in **9.17.7.3.2**;
- n is the number of full width horizontal stiffeners;
- A_{se} is the area of the effective section of the horizontal stiffeners derived in accordance with **9.17.4.4**.

NOTE In this case ℓ_s in **9.17.7.3.2** may be taken as the distance between such stiffeners.

9.17.8 ***Continuity of cross beams and cantilevers***

When continuity of cross beams and cantilevers is provided in the plane of a diaphragm, in accordance with **9.17.2.3**, that portion within the box walls should be in accordance with the following.

a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

b) If the member providing the continuity in a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **9.17.6**, it should be designed to withstand, in addition to the load given in a), an axial force equal to $A_{se}\sigma_{se}$,

where

- A_{se} is the area of the effective section of the continuity member derived in accordance with **9.17.4.4**;
- σ_{se} is as specified in **9.17.6.3.4**.

c) The member providing continuity in a) should be designed as a compression member in accordance with **10.1** to **10.6**, and should be assumed to be unrestrained out of the plane of the diaphragm unless provided with effective intermediate restraints. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **9.17.6.3**, a force equal to 2.5 % of the maximum axial load in the continuity member including that given in b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{\sigma_{\ell s}} + \frac{\sigma_{bs} + \sigma_{b2}}{\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

- σ_{b2} is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle third of the length of the stiffener;
- σ_{bs}, $\sigma_{\ell s}$, σ_{se}, σ_{ys} are as defined in **9.17.6.7**.

10 Design of compression members

10.1 General

This clause covers the design of straight members of uniform cross-section subjected to axial compression or to combined compression and bending.

10.2 Limit state

10.2.1 *Ultimate limit state*

Members subjected to axial compression or to combined compression and bending should be designed to satisfy the provisions of clause **10** for the ultimate limit state.

10.2.2 *Fatigue*

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

10.2.3 *Serviceability limit state*

Non-compact truss members (see **10.6.3**) which are not in accordance with item b) of **12.2.3** should also satisfy the provisions of clause **10** for the serviceability limit state.

10.3 Limitations on shape

10.3.1 *General*

Figure 36 sets out the geometric notation used in clause **10**. Components should be in accordance with the robustness recommendations given in **5.2**.

The recommendations for strength in clause **10** 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 meets the shape limitations of **10.3.2**, **10.3.3**, or **10.3.4** as appropriate.

10.3.2 *Unstiffened outstand*

Unless the free edge of a plate or other outstand is stiffened, the ratio b_o/t_o should not exceed:

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

where

b_o 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 the root fillet of a rolled section, or, in the case of a welded construction, to the surface of the supporting part of the member [see Figure 36a)];

t_o is the mean thickness of the outstand, or the aggregate thickness of the outstand where two or more parts are joined together in accordance with **14.5** or **14.6**;

σ_y is as defined in **10.3.1**.

10.3.3 *Stiffened outstand*

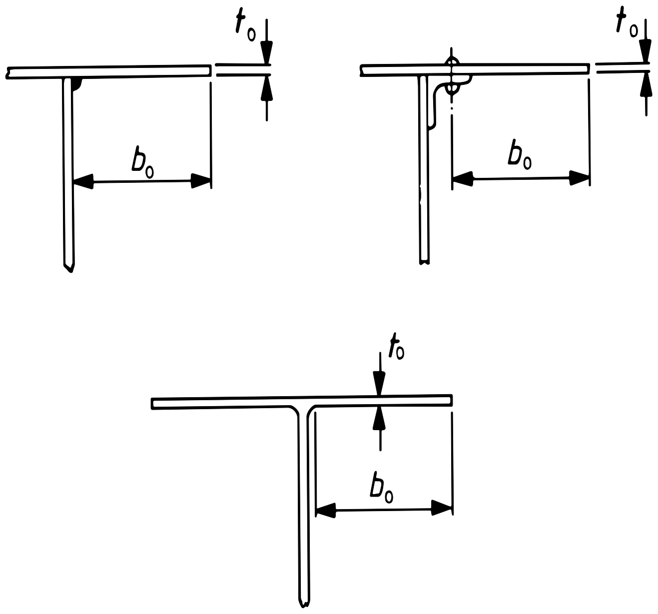
Unless the free edges of stiffened outstands are interconnected transversely by means of battens, lacing or perforated plates in accordance with **10.8**, **10.9** or **10.10**, respectively, the ratio b_o/t_o should not exceed:

$$14\sqrt{\frac{355}{\sigma_y}}$$

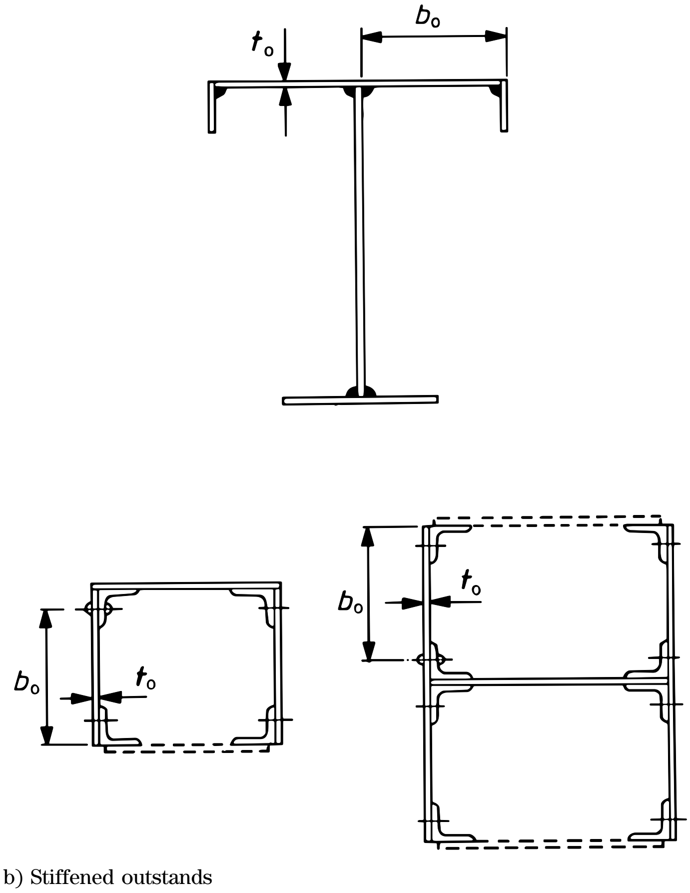
where

b_o, t_o are as defined in **10.3.2** [see Figure 36b)];

σ_y is as defined in **10.3.1**.



a) Unstiffened outstands



b) Stiffened outstands

Figure 36 — Limitations on shape for compression members

10.3.4 *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 **10.3.1**.

10.4 Effective lengths

10.4.1 *General*

The effective length of a compression member ℓ_e may be determined either from Table 10 or from **10.4.2** for single angles and **12.4** or **12.5** for trusses, or may be determined by an elastic critical buckling analysis. Alternatively, for a strut which is effectively held in position and fully or partially restrained in direction, ℓ_e may be taken as $k_1 L$

where

k_1 is obtained from Figure 7a) taking I_c as the second moment of area of the member about its appropriate centroidal axis;

L is the length of the member between end restraints.

10.4.2 *Single angle members*

10.4.2.1 *Discontinuous members*

The effective length ℓ_e of a single angle discontinuous member connected by bolts, rivets or welds to a gusset or to a section, provided that they offer effective restraint in the plane considered, should be taken as the length of the member measured between centres of fastenings or groups of fastenings at the ends.

10.4.2.2 *Intersecting members*

The effective length ℓ_e of a single angle bracing member intersected by, and connected to, another such member should be taken as:

a) *in the plane of bracing:*

0.85 × (the greatest distance between centres of adjacent intersections);

b) *in any other plane:*

0.7 × (the distance along the bracing member between centroids of the main members).

Table 10 — Effective length ℓ_e for compression members

Restraint condition	Effective length ℓ_e
Effectively held in position and restrained in direction at both ends	0.7L
Effectively held in position at both ends and restrained in direction at one end	0.85L
Effectively held in position at both ends, but not restrained in direction	L
Effectively held in position and restrained in direction at one end; partially restrained in direction but not held in position at the other end	1.5L
Effectively held in position and restrained in direction at one end; not held in position or restrained in direction at the other end	2.0L

10.5 Effective section

10.5.1 *General*

In determining the effective section of a member, consideration should be given to the adequacy of the end fixings to distribute the load effects into all parts of the section.

10.5.2 *Effective areas*

10.5.2.1 *Members other than circular hollow sections*

The effective area of a compression member, other than a circular hollow section, should be taken as $\sum A_e$, where A_e is the effective area of each component part as defined in **9.4.2.4**.

10.5.2.2 *Circular hollow sections*

The effective area A_e of a circular hollow section should be taken as A_c, where A_c is the net area calculated in accordance with **9.4.2.4**.

10.6 Compression members without longitudinal stiffeners

10.6.1 *Axial compression*

10.6.1.1 *Strength*

A member subjected to axial compression should be such that the axial load does not exceed the resistance P_D given by:

$$P_D = \frac{A_e \sigma_c}{\gamma_m \gamma_{f3}}$$

except for single angles connected by one leg (see **10.6.1.2**)

where

A_e is the effective area of a section as defined in **10.5**;

σ_c is the least ultimate compressive stress for buckling about any axis to be obtained from σ_c/σ_y, in accordance with Figure 37.

NOTE In using Figure 37 the values of $\frac{r}{y}$ and $\frac{\ell_e}{r}\sqrt{\frac{\sigma_y}{355}}$ are required,

where

ℓ_e is the effective length for buckling about any centroidal axis, as defined in **10.4**;

r is the radius of gyration of the section about the same axis, based on the gross section of the member, but ignoring battening or lacing;

y is the distance from the same axis to the extreme fibre of the section. In any unsymmetrical section the larger value of y should be used;

σ_y is the nominal yield stress value, as defined in **10.3.1**, for the material.

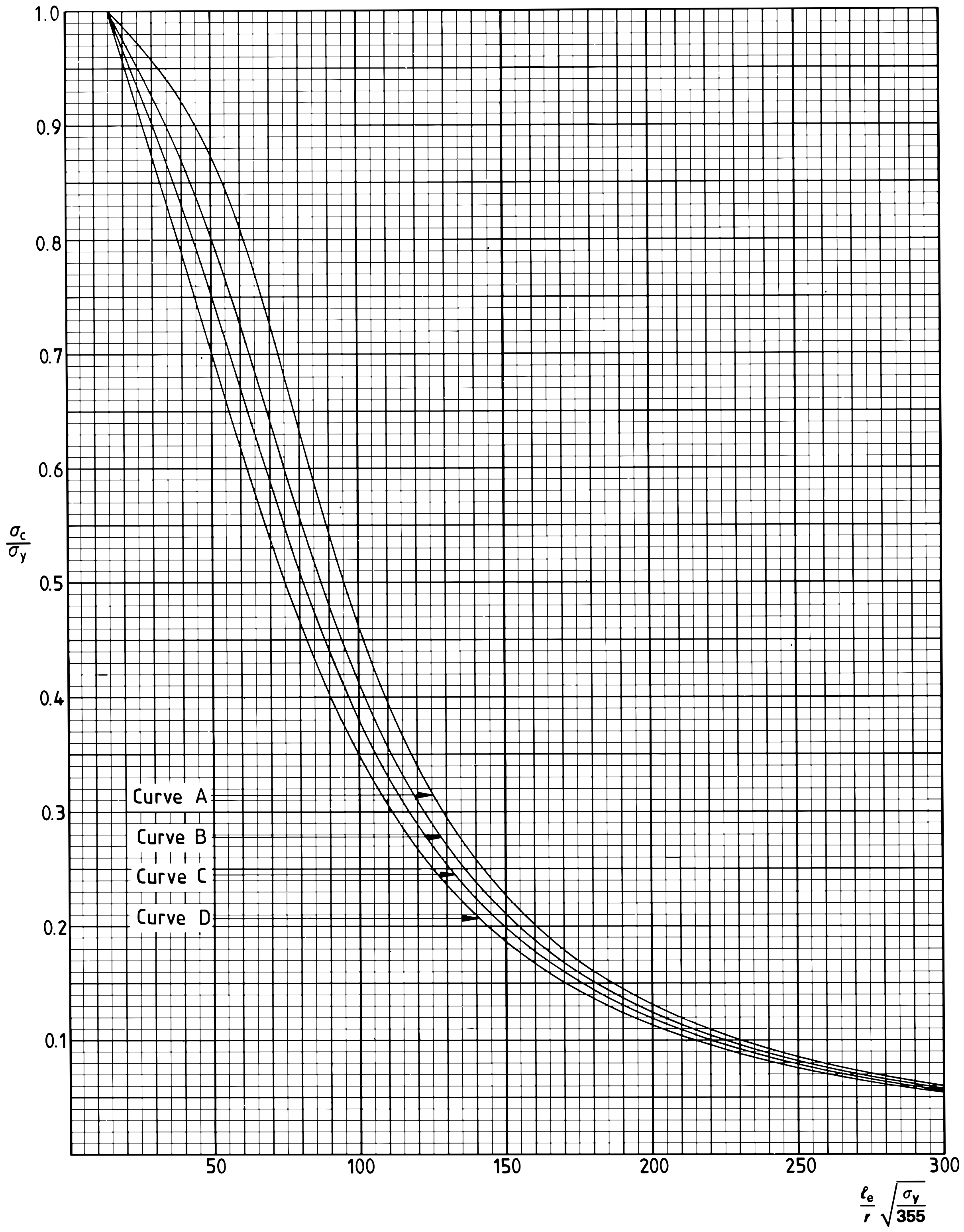


Figure 37 — Ultimate compressive stress σ_c

NOTE 1 Curves should be selected as follows:

	Members fabricated by welding (excluding local welding of battens, lacing, etc.)	All other members (including stress relieved welded members)
$r/y \geqslant 0.7$	Curve B	Curve A
$r/y = 0.60$	Curve C	Curve B
$r/y = 0.50$	Curve C	Curve B
$r/y \leqslant 0.45$	Curve C	Curve C
All rolled sections with flange thickness > 40 mm	Curve D	
Hot finished hollow sections	Curve A	

NOTE 2 For intermediate values of r/y, linear interpolation may be used between the curves given.

NOTE 3 For the basis of curves, see **G.16**.

Figure 37 — Ultimate compressive stress σ_c (*continued*)

10.6.1.2 *Single angles*

A single angle member connected by one leg may be designed ignoring the eccentricity of the connections with respect to the centroidal axis, unless it is connected at either end by a single bolt or rivet, when its resistance should be taken as 0.8 times the value derived in **10.6.1.1**. However, for a lacing bar, the full value, without the reduction, may be used.

10.6.2 *Combined compression and bending*

10.6.2.1 *Strength*

A member subjected to coexistent compression and bending should be such that at all cross-sections within the middle third of the length of the member:

$$\frac{P_{max.}}{P_D} + \frac{M_{x,\ max.}}{M_{Dxc}} + \frac{M_{y,\ max.}}{M_{Dyc}} \leq 1.0$$

where

$P_{max.}$, $M_{x,\ max.}$, $M_{y,\ max.}$	are the maximum axial load, and bending moments about the X-X and Y-Y axes, respectively (see Figure 1) anywhere within the middle third of the length of the member between points of restraint;
P_D	is as defined in **10.6.1**;
M_{Dxc}, M_{Dyc}	are the corresponding bending resistances of the member determined in accordance with **9.9.1**.

NOTE When the bending capacity of the cross-section may be increased by the presence of an axial compression the above interaction may be conservative. The interaction may then be calculated from first principles.

In addition, at all sections of the member, the maximum stress due to the applied load P and coexistent bending moments M_x and M_y should be such that:

$$\frac{P}{A_e} \pm \frac{M_x}{Z_x} \pm \frac{M_y}{Z_y} \leq \frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

A_e	is the effective area of the section, as defined in **10.5**;
σ_y	is the nominal yield stress value, as defined in **10.3.1**, for the material;
Z_x, Z_y	are the appropriate elastic moduli of the effective section derived in accordance with **9.4.2**.

10.6.2.2 *Eccentricity of end connections*

The bending moment resulting from any eccentricity of the end connections of the member or its components should be taken into account in determining the values of $M_{x,\ max.}$, $M_{y,\ max.}$, M_x and M_y referred to in **10.6.2.1**.

10.6.3 ***Compact and stocky members***

As an alternative to the provisions of **10.6.2**, when a member is stocky and of compact cross-section, the resistance in combined compression and bending may be determined on the basis of any assumed distribution of stress over the effective area of cross-section, provided that the stresses so assumed are in equilibrium with the load effects and nowhere exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

and provided that:

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

where

σ_y is the nominal yield stress of the material as defined in **6.2**;

M_{pe} is as defined in **9.7.1**;

M_{ult} is as defined in **9.8**;

λ_{LT} is derived in accordance with **9.7**.

A member is defined as stocky if its slenderness ratio ℓ_e/r does not exceed:

$$15\sqrt{\frac{355}{\sigma_y}}$$

where

ℓ_e, r are as defined in **10.4** and **10.6.1.1**, respectively.

A cross-section of a member is defined as compact if:

a) $\frac{b}{t} \leq 24\sqrt{\frac{355}{\sigma_y}}$ for plates between supports

where

b is the unsupported width of plate between adjacent lines of bolts or rivets connecting the plate to supporting parts of the member, or, for welded members, between the surfaces of the supporting parts, or for rolled sections, clear between root fillets;

t 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.

b) $\frac{b_o}{t_o} \leq 7\sqrt{\frac{355}{\sigma_y}}$ for outstands

where

b_o, t_o are as defined in **10.3.2**;

c) the ratio of the outside diameter to the wall thickness for circular hollow sections does not exceed

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

10.7 Compression members with longitudinal stiffeners

10.7.1 ***Strength***

The stress in plate panels, longitudinally stiffened by discrete stiffeners (i.e. other than corner stiffeners) and forming walls of box-type compression members, should satisfy the provisions of **9.10.2.1** and the stresses at the centroids of the longitudinal stiffeners should satisfy the provisions of **9.10.2.3** when both these stresses are determined in accordance with **10.7.2**. In determining the stresses at the centroid of the longitudinal stiffeners, the value of y_{Bs} should be appropriate to the combined stress diagram at the section considered, where y_{Bs} is as defined in **9.10.2.3**.

NOTE When the maximum tensile stress intensity due to the bending moment is smaller than the compression stress intensity due to the axial load, the axis of zero stress will be outside the cross-section of the member and in that case y_{Bs} may be greater than the depth of the member.

10.7.2 ***Evaluation of stresses***

The stresses in the member should be evaluated for the following coexistent loads and moments:

a) the applied axial load;

b) the applied bending moments about the X-X and Y-Y axes each multiplied by a factor:

$$\frac{\sigma_E}{\sigma_E - \sigma_a}$$

where

σ_E is the Euler buckling stress of the member about the relevant axis, given by $\pi^2 E r^2/\ell_e^2$;

ℓ_e, r are as defined in **10.4** and **10.6.1**, respectively;

σ_a is the axial stress based on the effective section of the member in accordance with **10.5**;

c) additional bending moments acting about the X-X and Y-Y axes, respectively, equal to:

$$P\Delta_i \left(\frac{\sigma_E}{\sigma_E - \sigma_a}\right)$$

where

P is the applied axial load;

Δ_i is 0.001 25 times the length of member between the appropriate points of restraint, determined separately for the X-X and Y-Y axes.

NOTE Stresses due to a) should be evaluated on the basis of the effective area determined in accordance with **10.5.2**, and stresses due to b) and c) should be evaluated on the basis of the effective section determined in accordance with **9.4.2**.

10.7.3 ***Shape of longitudinal stiffeners***

The shape of the longitudinal stiffeners should satisfy the provisions of **9.3.4**.

10.7.4 ***Transverse stiffeners***

10.7.4.1 Transverse stiffeners may be assumed to combine with an effective width of the plating on each side not exceeding either:

a) one-quarter of the stiffener spacing; or

b) one-eighth of the length of the stiffener.

10.7.4.2 The effective section of the stiffener should satisfy both:

a) the stiffness provision in **9.15.3**; and

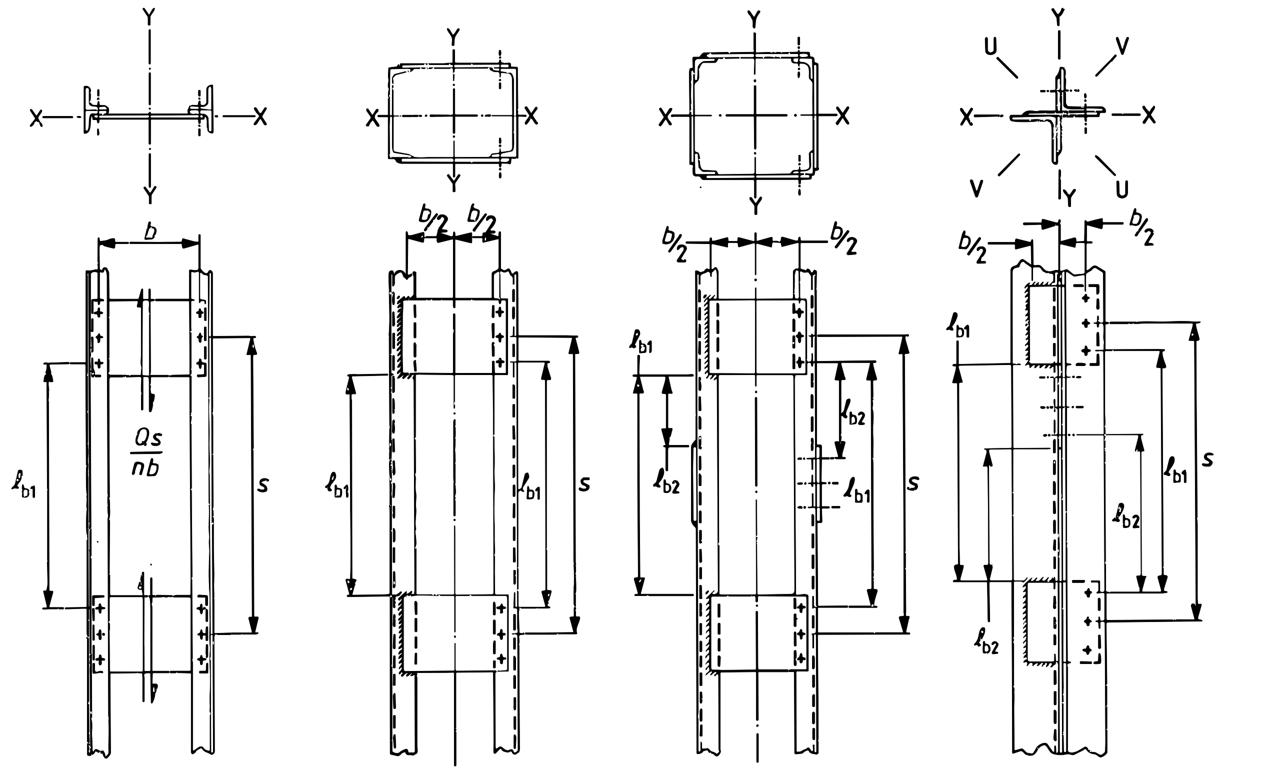
b) the strength provision of **9.15.5** under the action of a uniformly distributed load equal to 1/300 times the longitudinal compressive force in the stiffened panel at the cross-section under consideration due to the loads and moments given in **10.7.2**.

10.8 Battened compression members

10.8.1 ***General***

A compression member consisting of two or more main components may have battens connecting the components either in one plane, in two or more parallel planes, or in two perpendicular planes or sets of parallel planes, as shown in Figure 38.

The strength of the individual components, including battens, and their connections, should be in accordance with the provisions of **9.1** to **9.9**, **10.1** to **10.7**, **11.1** to **11.5** and clause **14**, as appropriate.



NOTE Pairs of battens are shown staggered to illustrate ℓ_{b1} and ℓ_{b2}. Normally all four battens should be at the same cross-section.

Figure 38 — Battened members

10.8.2 ***Radius of gyration of the member***

The radius of gyration of the member about the Y-Y axis, in the case of a single plane of battens or of parallel battens (see Figure 38), and about any axis in the case of a member with battens in planes at right angles, or of cruciform section, should be taken as 0.9 times the actual radius of gyration.

10.8.3 ***Spacing of battens***

Battens should generally be spaced uniformly throughout the length of the member, except as required for intermediate restraint in accordance with **10.8.5.1**c).

If the member as a whole is such that $\ell_y/r_y \leq 0.8\ (\ell_x/r_x)$, the spacing of battens in a plane parallel to the X-X axis should be such that each main component of the member satisfies the following:

$$\frac{\ell_{b1}}{r_{b1}} \leq 0.7\lambda_{max.} \text{ and}$$

$$\frac{\ell_{b2}}{r_{b2}} \leq 0.7\lambda_{max.}$$

where

$\ell_x/r_x, \ell_y/r_y$ are the slenderness ratios of the member about its X-X and Y-Y axes, respectively (as shown in Figure 38), r_x and r_y should be calculated on the basis of the gross cross-section of the member;

ℓ_{b1} is the distance between end fastenings of successive battens in planes parallel to the X-X axis;

ℓ_{b2} is the distance between end fastenings of successive battens in any plane;

r_{b1} is the radius of gyration of the gross cross-section of the component about an axis, parallel to the Y-Y axis, through the centroid of the component;

r_{b2} is the least radius of gyration of the gross cross-section of the component;

NOTE For determining r_{b1} and r_{b2} in the case of partially enclosed sections, as shown in Figure 36b), the component should be considered to consist of the angle or outstanding plate to which the batten is attached together with half of the depth of the web between the batten and the flange plate parallel thereto.

$\lambda_{max.}$ is the largest value of the slenderness ratio ℓ/r with which the design resistance of the member would be sufficient to resist the applied load (see **10.6.1**).

If the member as a whole is such that $\ell_y/r_y > 0.8(\ell_x/r_x)$, the spacing of battens in a plane parallel to the X-X axis should be such that each main component of the member satisfies the following:

$$\frac{\ell_{b1}}{r_{b1}} \leq 0.6\lambda_{max.} \text{ and}$$

$$\frac{\ell_{b2}}{r_{b2}} \leq 0.6\lambda_{max.}$$

The spacing of battens (if any) in a plane perpendicular to the X-X axis should be determined similarly by transposing the axes.

10.8.4 *Dimensions of battens*

10.8.4.1 *Length*

The length of each batten, measured between end fastenings in a direction parallel to the axis of the member, should not be less than three-quarters of the distance between the centroids of adjacent main components.

Furthermore, the length of each batten, measured as in **10.8.4.1**, should not be less than twice the width of the smallest main component measured parallel to the plane of the battens.

10.8.4.2 *Thickness*

The thickness of each batten should not be less 1/50 times the distance between the innermost lines of fastenings, except that, where both transverse edges of a batten are effectively stiffened by stiffeners having a slenderness ratio not exceeding 170, the thickness of the batten need not exceed 8 mm.

10.8.5 *Members with single or parallel planes of battens*

10.8.5.1 *Arrangement of battens*

In any battened member, other than a member of cruciform section, battens should be placed in each battened plane as follows:

a) at each end of the member;

b) at not less than two intermediate positions, inclusive of any positions where battens are provided under c);

c) at each intermediate point, if any, where in the plane being considered, the member is provided with restraint against lateral displacement or has another member connected to it.

Battens should be placed opposite one another wherever there are two or more planes of parallel battens.

10.8.5.2 *Loads and moments on battens*

Each batten, and its fixings to the main components of the member, should be proportioned to resist, simultaneously:

a) a longitudinal shear force equal to Qs/nb (see Figure 38);

b) a bending moment, acting in the plane of the batten, equal to $Qs/2n$;

c) the effects of any external transverse loads on the member;

where

Q is a transverse shear force acting parallel to the plane or planes of the batten, which should be taken as:

1) $Q = \dfrac{PP_{Ey}}{200(P_{Ey} - P)}$

for battens in a plane parallel to the X-X axis;

2) $Q = \dfrac{PP_{Ex}}{200(P_{Ex} - P)}$

for battens in a plane parallel to the Y-Y axis;

P is the axial force in the member;

P_{Ey} = $\dfrac{\pi^2 EA_e}{(\ell_y/r_y)^2}$;

P_{Ex} = $\dfrac{\pi^2 EA_e}{(\ell_x/r_x)^2}$;

$\ell_x/r_x, \ell_y/r_y$ are as defined in **10.8.3**;

A_e is the effective area of the whole member determined in accordance with **10.5**.

In using Figure 38:

s is the longitudinal spacing of battens measured between centres;

n is the number of parallel planes of battens;

b is the lateral distance between centroids of fastenings to the components.

10.8.6 *Cruciform members*

10.8.6.1 *Arrangement of battens*

Battens in cruciform members should either be placed in pairs in two perpendicular planes in contact at a transverse edge and fixed successively to the two legs of both angles, or be of cruciform section and fixed to both legs of both angles.

A pair of battens, or a cruciform batten, should be placed:

a) at each end of the member;

b) at not less than two intermediate positions, inclusive of any positions where battens are provided under c);

c) at each intermediate point, if any, where the member is provided with restraint against lateral displacement or has another member connected to it.

10.8.6.2 *Loads and moments on battens*

Each batten, and its connections to the main components, should be proportioned to resist simultaneously:

a) a longitudinal shear force equal to $\dfrac{Q_r s}{b}$;

b) a bending moment, acting in the plane of the batten, equal to $\dfrac{Q_r s}{2}$

where

Q_r = Q if the largest slenderness ratio ℓ/r of the member as a whole occurs about the X-X or Y-Y axis; or

= $Q/\sqrt{2}$ if the largest slenderness ratio ℓ/r of the member as a whole occurs about a diagonal axis V-V;

Q, s, b are as defined in **10.8.5.2**;

c) the effects of any transverse external loads on the member.

10.8.6.3 *Strength of components of the member*

Each main component of a cruciform member should be designed to resist, in addition to the axial force, a bending moment about each of the X-X and Y-Y axes equal to:

$$\frac{Q_r s}{4}$$

together with the effects of transverse external forces, if any,

where

s is as defined in **10.8.5.2**;

Q_r is as defined in **10.8.6.2**.

10.8.7 ***Welding of battens***

10.8.7.1 The aggregate length of weld connecting each longitudinal edge of a batten to a main component of a member should not be less than half the length of the batten. At least one-third of the longitudinal weld should be placed at each end of the edge of the batten. A further length of weld, equal to at least four times the thickness of the batten, should be returned along the end of the batten from each longitudinal edge.

10.8.7.2 Where batten plates are fitted between main components they should be connected to each component either by fillet welds on each side of the plate, at least equal in length to that given in **10.8.7.1**, or by complete penetration butt welds along the whole length of the plate.

10.9 Laced compression members

10.9.1 ***General***

A compression member consisting of two or more main components may have lacing connecting the components, either in one plane, in two or more parallel planes, or in two perpendicular sets of parallel planes.

The lacing should form a fully triangulated system and should be uniform throughout the length of the member.

A laced compression member should be provided with a batten, in accordance with **10.8.4**, in each plane of lacing at each end of the member, and at each point where the system of lacing is interrupted or where another member is connected to the laced member. These battens should be designed to resist the forces stated in **10.8.5.2**.

The strength of a member as a whole and of individual components, including lacings and their connections, should be in accordance with **10.1** to **10.7**, **11.1** to **11.5** and clause **14**, as appropriate.

10.9.2 ***Inclination of lacing bars***

If a single system of lacing bars is used, the bars should be inclined at an angle between 50° and 70° to the axis of the member; if a double system of intersecting bars is used, the bars should be inclined at an angle between 40° and 50°.

10.9.3 ***Spacing of lacing bars***

The spacing of lacing bars should be such that each main component of the member satisfies the following:

$$\frac{\ell_{p1}}{r_{p1}} \leq 0.7\lambda_{max.} \text{ and}$$

$$\frac{\ell_{p2}}{r_{p2}} \leq 0.7\lambda_{max.}$$

where

ℓ_{p1}	is the distance between the centroids of successive end fastenings of lacing bars in one plane;
ℓ_{p2}	is the distance between the centroids of successive end fastenings of lacing bars in any plane;
r_{p1}	is the radius of gyration of a main component of the member about an axis perpendicular to the plane of lacing based on the gross cross-section of the member;
r_{p2}	is the least radius of gyration of a main component of the member based on the gross cross-section of the member;
$\lambda_{max.}$	is as defined in **10.8.3**.

10.9.4 ***Slenderness of lacing bars***

In a single system the effective length of a lacing bar should be taken as the clear length along the bar between innermost fixings to the main components of the member, and, in a double intersecting system, 0.7 of this clear length.

10.9.5 ***Loads on lacing***

Lacing bars and their fixings should be designed to resist, at any point along the length of the member, a transverse shear force Q, as defined in **10.8.5.2** for battened members, together with the effects of any external transverse loads on the member. The shear force Q should be considered as divided equally between all the systems of lacing or plates connecting the components in the appropriate parallel planes.

10.9.6 ***Double lacing***

In a double system of intersecting lacing bars the effects of axial deformation of the member on the lacing bars and their connections should be considered.

Except for battens, in accordance with **10.9.1**, a double system of intersecting lacing bars should not be combined with diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for.

10.9.7 ***Welding of lacing bars to main components***

Where a lacing bar to be connected by welding is lapped on to a main component of a member, the length of lap, measured along the centreline of the lacing bar, should not be less than four times the thickness of the bar, or four times the mean thickness of the flange of the main component to which it is attached, whichever is less. The bar should be welded along the whole length of lap on both sides of the bar, and the weld should be returned across the end of the bar for an aggregate distance of not less than the width of the bar, or four times its thickness, whichever is less.

Where a welded lacing bar is fitted between main components of a member, it should be attached to each component either by welding all round, or by a full penetration butt weld.

10.10 Compression members connected by perforated plates

10.10.1 ***General***

A compression member consisting of two or more main components may have continuous perforated plates connecting the components, either in one plane, or in two or more parallel planes, or in two perpendicular sets of parallel planes. The thickness of a perforated plate should not be less than one-fiftieth of the unsupported distance between innermost attachments to the main components.

The overall length of a perforation, measured in the direction of stress, should not be more than twice its width. Each end of a perforation should be rounded.

The clear distance between perforations, and the clear length beyond the perforation at each end of the member, should not be less than three-quarters of the unsupported distance between the innermost attachments to the main components.

The unsupported width of a plate at a perforated section, between the inner edge of the perforation and the nearest attachment to a main component, should be in accordance with **10.3.2**.

10.10.2 ***Strength of member***

The net section of a perforated plate may be included as part of the effective section of the member when computing the strength of the member in accordance with **10.1** to **10.7**.

10.10.3 ***Loads on perforated plates***

Perforated plates and their fixings should be designed to resist, at any point along the length of a member, a transverse shear force Q, as defined in **10.8.5.2**, together with the effects of any transverse external loads. The shear force Q should be considered as divided equally between all the parallel perforated and other plates connecting the components of the member.

10.11 Compression members with components back to back

10.11.1 *General*

A compression member may consist of two angles, channels or tees, connected together, back to back, either in contact or separated by packs or washers. The components should preferably be in contact; when not in contact, either adequate space between the components should be provided in accordance with **4.5.4**, or the thickness should be increased to meet the provisions of **4.5.5**. In no case should the clear distance between the components exceed 50 mm.

Members with components not in contact should not be used to resist loads or moments applied in a plane perpendicular to the connected faces.

10.11.2 *Slenderness of components*

The components should be connected together so that:

$$\frac{\ell_p}{r_p} \leq 0.5\lambda_{max.}$$

where

ℓ_p is the distance between the centroids of successive connections;

r_p is the least radius of gyration of the unsupported length of a component of the member between successive connections based on the gross cross-section of the member;

$\lambda_{max.}$ is as defined in **10.8.3**.

10.11.3 *Connections between components*

Connections between components should be spaced so as to divide the overall length of the member into at least three approximately equal parts.

Where these connections are made by welding, solid packings should be used to effect the jointing unless the components are sufficiently close together to permit welding along both pairs of edges.

Where the components are separated, connections made by bolts or rivets should pass through solid washers or packs. At least two connectors should be provided side by side transversely at each connected point if the width of connected face is more than 130 mm in the case of angles or tees, or more than 150 mm in the case of channels.

Connections should be designed to resist the shear force Q, given for battens in **10.8.5.2**, and the effects of any transverse external loads, and should be designed in accordance with clause **14**.

11 Design of tension members

11.1 General

This clause covers the design of straight members subjected to axial tension or to combined tension and bending.

11.2 Limit state

11.2.1 *Ultimate limit state*

Tension members should be designed to satisfy the provisions of clause **11** for the ultimate limit state.

11.2.2 *Fatigue*

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

11.2.3 *Serviceability limit state*

The serviceability limit state need not be considered.

11.3 Effective section

11.3.1 *General*

In determining the effective section of a member, consideration should be given to the adequacy of the end fixings to distribute the load effects into all parts of the section.

11.3.2 ***Effective area***

The effective area A_e should be taken as:

$A_e = k_1 k_2 A_t$,

but $A_e \leq A$

where

k_1 is 1.0 except at a section through a pin hole when it should be taken as 0.65;

k_2 should generally be taken as 1.0, except that where the steel specified conforms to one of the standards listed in **6.1.2**, k_2 may be taken as $0.8\sigma_u/\sigma_y$, but not less than 1.0 nor greater than 1.2;

σ_u is the minimum tensile strength specified for the appropriate grade and thickness;

NOTE Where a range of tensile strengths is specified, the minimum value should be used in the formula for k_2.

σ_y is the nominal yield stress as defined in **6.2**;

A_t is the net cross-sectional area of the member or part given in **11.3.3**;

A is the gross cross-sectional area of the member or part.

11.3.3 ***Net cross-sectional area of members with bolt or rivet holes***

The net cross-sectional area A_t of a member or of any of its components should be taken as the lesser of either the gross cross-sectional area A minus the area of all the holes (including all plug holes and countersunk heads) in a section perpendicular to the direction of primary stress, or the least net cross-sectional area of any diagonal or zig-zag section through a chain of holes, taken as:

$$A_t = A - \Sigma A_h + \Sigma \frac{s^2 t}{4g} \qquad \text{(see Figure 39)}$$

where

ΣA_h is the sum of the cross-sectional areas of the holes lying on the zig-zag or diagonal section, including any plug holes;

s is the spacing of consecutive holes measured parallel to the direction of primary stress in the member;

g is the spacing of the same holes measured at right angles to the direction of primary stress in the member. In an angle or similar part having holes in more than one plane, g should be measured along the centre of the thickness of the part;

t is the thickness of the part. When the thickness varies between consecutive holes, as around the heel of a channel section, the mean thickness should be taken.

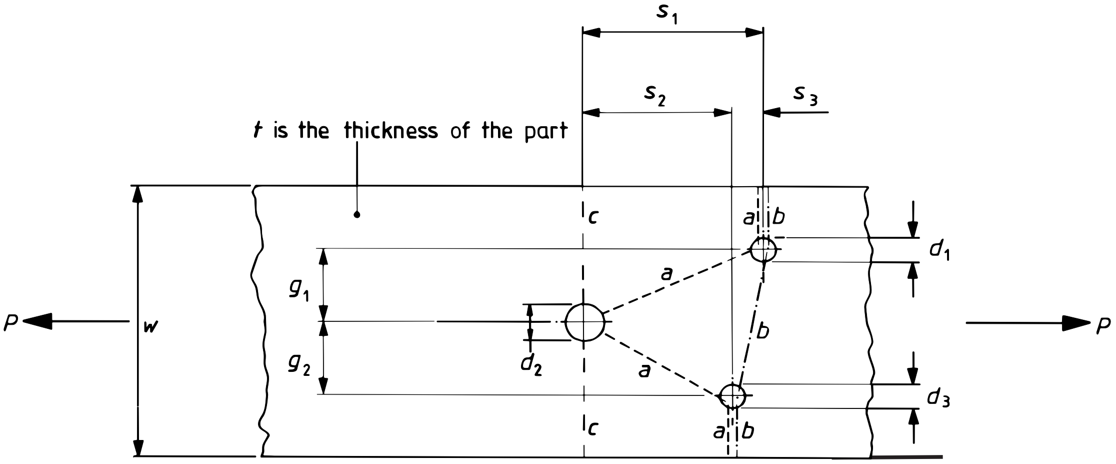
NOTE Where the critical chain of holes in a component does not coincide with that for the member as a whole, the load resisted by the fasteners joining the components between the two critical chains of holes should be taken into account in determining the strength of the member.

11.3.4 ***Screwed rods***

The net sectional area of the screwed rod should be taken as either the area at the root of the thread, or the tensile stress area given in BS 3692, BS 4190 or BS 4395, as appropriate.

11.3.5 ***Pin-connected members***

In a pin-connected member, the net area of the longitudinal section beyond the pin-hole, parallel to the axis of the member, should not be less than the required net cross-sectional area of the member (see Figure 40).



NOTE The net area A_t of the part is the least of the areas of sections a, b and c as follows:

a - - - - - $\quad A_t = t\left\{w - (d_1 + d_2 + d_3) + 0.25\left(\frac{S_1^2}{g_1} + \frac{S_2^2}{g_2}\right)\right\}$ considering all three holes;

b — · — · — $\quad A_t = t\left\{w - (d_1 + d_3) + 0.25\left(\frac{S_3^2}{g_1 + g_2}\right)\right\}$ considering two holes;

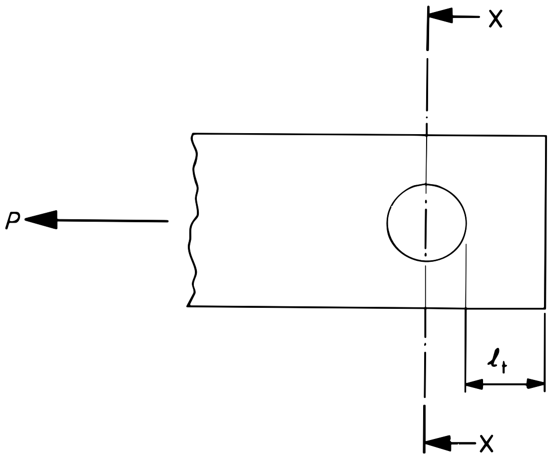
c — — — $\quad A_t$ is given by the smallest of:

$t(w - d_1)$

$t(w - d_2)$

$t(w - d_3)$ considering any one hole.

Figure 39 — Net area



NOTE 1 The net area within the length ℓ_t should not be less than the required net cross-sectional area of the member.

NOTE 2 The thickness of section X-X should not be less than one-sixteenth of the net width at X-X, unless the edges are stiffened.

Figure 40 — Pin-connected member

11.4 Thickness at pin-holes

If the edges of the member are unstiffened, the thickness at a pin-hole of a member or part should not be less than one-sixteenth of the net width at a pin-hole perpendicular to the axis of the member (see Figure 40).

11.5 Strength

11.5.1 *Axial tension*

A member or part subjected to axial tension should be such that the axial load does not exceed the resistance P_D given by:

$$P_D = \frac{\sigma_y A_e}{\gamma_m \gamma_{f3}}$$

where

A_e	is the effective cross-sectional area of the member or part as given in **11.3.2**;
σ_y	is the nominal yield stress of the member or part as defined in **6.2**.

11.5.2 *Combined tension and bending*

11.5.2.1 *Yielding*

A member subject to coexistent tension and bending should be checked for yielding in accordance with **9.9.4.1**.

11.5.2.2 *Buckling*

Where, at any section within the middle third of the length of the member, the maximum compressive stress due to bending exceeds the tensile stress due to axial load, the following condition should be met:

$$\frac{M_{x,\,max.}}{Z_{xc}} + \frac{M_{y,\,max.}}{Z_{yc}} - \frac{P}{A_e} \le \frac{M_R}{Z_{xc}\,\gamma_m \gamma_{f3}}$$

where

$M_{x,\,max.}$, $M_{y,\,max.}$	are the maximum bending moments anywhere within the middle third of the length of the member about the X-X and Y-Y axes respectively (see Figure 1);
Z_{xc}, Z_{yc}	are the elastic moduli of the effective member section for compression at the cross-section under consideration about the X-X and Y-Y axes respectively (see Figure 1);
P	is the axial tensile force in the member;
A_e	is as defined in **11.5.1**;
M_R	is as defined in **9.8**.

11.5.2.3 *Alternative method*

As an alternative to **11.5.2.1** and **11.5.2.2**, where all parts are in tension, or where the part of the cross-section which is in compression is compact, the resistance in combined bending and tension may be determined on the basis of any assumed distribution of stress over the effective area of the cross-section, provided that the stresses are in equilibrium with the load effects and nowhere exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

and provided that:

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

where

σ_y	is the nominal yield stress, as defined in **6.2**;
λ_{LT}	is derived for the whole cross-section in accordance with **9.7**;
M_{pe}	is as defined in **9.7.1**;
M_{ult}	is as defined in **9.8**.

11.5.3 *Eccentricity of end connections*

The bending moment resulting from any eccentricity of the end connections of a member or its components should be taken into account in determining the values of the bending moments referred to in **11.5.2**.

In the case of a member consisting of a single angle connected only by one leg, or of a rolled or built-up tee-section connected only by the table of the tee, or a single channel section connected only through the web, these provisions may be considered to be met if the effective area of the unconnected legs is taken as:

$\left(\frac{3A_1}{3A_1 + A_2}\right)A_2$ for angles, and

$\left(\frac{5A_1}{5A_1 + A_2}\right)A_2$ for tees and channels

where

A_1 is the net area of the connected leg of the angle, or of the table of the tee, or of the web of the channel;

A_2 is the net area of the unconnected leg of the angle, or of the stalk of the tee, or of both unconnected flanges of the channel.

NOTE The table of a tee-section or web of a channel should be connected without eccentricity about the centroidal axis perpendicular to the table of the tee or the web of the channel.

11.6 Battened tension members

11.6.1 *General*

A tension member consisting of two or more main components may have battens connecting the components, as described in **10.8.1**.

11.6.2 *Spacing of battens*

Battens should generally be spaced uniformly throughout the length of the member.

11.6.3 *Dimensions of battens*

The length of each batten, measured between end attachments in a direction parallel to the axis of the member, should not be less than three-quarters of the distance between the centroids of adjacent main components in the case of end battens, or half such distance in all other cases.

The thickness of each batten should not be less than 1/60 times the distance between the innermost lines of attachments, except that, where both transverse edges of a batten are effectively stiffened by stiffeners having a slenderness ratio not exceeding 170, the thickness of the batten need not exceed 8 mm.

11.6.4 *Arrangement of battens in single or parallel planes*

In any battened member, other than a member of cruciform section, battens should be arranged in accordance with **10.8.5.1**.

11.6.5 *Arrangement of battens in cruciform members*

Battens in cruciform members should be arranged in accordance with **10.8.6.1**.

11.6.6 *Connection of battens*

A batten attached by bolts or rivets should be connected to each main component by at least two bolts or rivets. Welds used to attach battens to the components should be in accordance with **10.8.7**.

11.6.7 *Loads on battens*

Battens and their fixings should be designed to resist the effects of any external transverse loads on the member.

11.7 Laced tension members

11.7.1 *General*

A tension member consisting of two or more main components may have lacing connecting the components, as described in **10.9.1**.

A laced tension member should be provided with a batten, in accordance with **11.6**, in each plane of lacing, at each end of the member, and at each point where the system of lacing is interrupted or where another member is connected to the laced member.

11.7.2 *Inclination of lacing*

The inclination of lacing bars should be in accordance with **10.9.2**.

11.7.3 *Loads on lacing bars*

Lacing bars and their fixings should be designed to resist the effects of any external loads applied transversely to the member. Such loads should be considered to be divided equally between all systems of lacing or plates connecting the main components in the appropriate parallel planes. The slenderness of bars should be determined in accordance with **10.9.4**.

11.7.4 ***Double lacing***

A double system of intersecting lacing bars should be in accordance with **10.9.6**.

11.7.5 ***Welding of lacing bars to main components***

Where a lacing bar is connected by welding to a main component of a member, the welding details should be in accordance with **10.9.7**.

11.8 Tension members connected by perforated plates

11.8.1 ***General***

A tension member consisting of two or more main components may have continuous perforated plates connecting the components, either in one plane, in two or more parallel planes or in two perpendicular sets of parallel planes.

The thickness of a perforated plate should not be less than 1/60 times of the unsupported distance between innermost attachments to the main components. Each end of a perforation should be rounded.

The clear distance between perforations should not be less than half the unsupported distance between innermost attachments to the main components, and the clear length beyond the perforation at each end of the member should not be less than three-quarters of such unsupported distance.

The unsupported width of a plate at a perforated section, between the inner edge of the perforation and the nearest attachment to a main component, should not be more than 20 times the thickness of the plate.

11.8.2 ***Strength of member***

The net section of a perforated plate may be included as part of the effective section of the member when computing the strength of the member in accordance with **11.1** to **11.5**.

11.8.3 ***Loads on perforated plates***

Perforated plates and their fixings should be designed to resist the effects of any external transverse loads on the member. Such loads should be considered to be divided equally between all the perforated plates and the other plates connecting the main components in the appropriate parallel planes.

11.9 Tension members with components back to back

Tension members with components back to back should be in accordance with **10.11.1**.

Connections between components should satisfy the provisions of **10.11.3**, except that the shear forces given for battens need not be considered.

12 Design of trusses

12.1 General

"Trusses" are defined as triangulated skeletal girders. The design of individual members and connections should be made in accordance with **12.2** to **12.7** in conjunction with clauses **10**, **11** and **14** as appropriate.

12.2 Limit states

12.2.1 ***Ultimate limit state***

All members and components of a truss should satisfy the provisions of clause **12** for the ultimate limit state. In trusses with stiff joints, stresses due to axial deformation of members may be ignored.

12.2.2 ***Fatigue***

Fatigue endurance should be in accordance with the recommendations of BS 5400-10 taking into account coexistent axial and bending stresses in members in accordance with **12.3.2**.

12.2.3 ***Serviceability limit state***

Tension members need not be checked for the serviceability limit state. All compression members should be checked for the serviceability limit state except:

a) members of compact section throughout their length, as defined in **10.6.3**; or

b) members meeting at a joint of a simply supported Warren truss, or modified Warren truss (see Figure 41), provided the ratio of length to width, in the plane of the truss, of each of these members is equal to or greater than 12 for the chord members and 24 for the web members; the length being taken between the centres of intersection and the width in the plane of the truss.

12.3 Analysis

12.3.1 *General*

The effects of interaction between the members of the main trusses and the lateral bracing system of a bridge structure should be considered.

12.3.2 *Global load effects*

The global load effects on the structure should be calculated in accordance with elastic theory, based on the assumption that all members are straight, and that either:

a) all members are pin jointed and each joint lies at the intersection of the centroidal axes of the relevant members and all loads, including the self-weight of members, are applied at the joints; or

b) the joints are stiff.

When considering the limit state of fatigue, or the limit state of serviceability, either method b) should be used, or method a), modified by the inclusion of flexural stresses due to axial deformation, self-weight of the members and the stiffness of joints. If any prestressing of the structure is adopted to counteract these stresses, this should be taken into account for the serviceability limit state, but only 90 % of the relieving effects of the prestressing should be considered.

12.3.3 *Local load effects*

12.3.3.1 *Loads not applied at truss joints*

Account should be taken of the following:

a) the resulting stresses when a load is applied to a member in the plane of a truss other than at a joint;

b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross member, the effect of interaction between the cross member so loaded and the truss and adjacent cross members should be taken into account.

12.3.3.2 *Eccentricities at joints*

If, at a joint, the centroidal axes of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account.

12.4 Effective length of compression members

The effective length ℓ_e of a compression member should either be obtained from Table 11 or be determined by an elastic critical buckling analysis of the truss. In applying Table 11, the end raker of a truss should be considered as a web member.

A compression chord may be considered to be effectively braced provided that the restraint system is in accordance with **9.12**, with the chord treated as a flange.

Table 11 — Effective length ℓ_e for compression members in trusses

Member		Effective length ℓ_e		
		Buckling in plane of truss	Buckling normal to plane of truss	
			When compression chord is effectively braced by lateral system	When compression chord is unbraced
Chord		0.85 times the distance between intersections with web members	0.85 times the distance between intersections with lateral bracing members or rigidly connected cross beams	See **12.5.1**
Web	Single triangulated system	0.70 times the distance between intersections with chords	0.85 times the distance between intersections with chords	Distance between intersections with chords
	Multiple intersection system with adequate connections at all points of intersection	0.85 times the greatest distance between any two successive intersections	0.70 times the distance between intersections with chords	0.85 times the distance between intersections with chords

12.5 Unbraced compression chords

12.5.1 *Effective length*

Where a compression chord is not provided with a system providing effective lateral restraint, the effective length ℓ_e of the chord member may be derived as for a beam with intermediate lateral restraints in accordance with **9.6.4.1.1.2** or **9.6.4.1.3**, as appropriate, with the chord treated as a compression flange.

Where the lateral restraint to a chord is provided by U-frames comprising cross members and web members (see Figure 41), the effective length may be determined in accordance with **9.6.4.1.3** with δ_R taken as follows:

$$\delta_R = \frac{1}{1/\delta_v + \sum 1/\delta_i}$$

where

δ_v is the deflection for a U-frame component with a vertical truss web member, given by:

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

δ_i is the deflection for a U-frame component with an inclined truss web member, given by:

$$\delta_i = \frac{d_3^3}{3EI_3} + \frac{uBd_2^2}{EI_2} + fd_2^2 + \theta s$$

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;

d_2 is the distance from the centroid of the compression flange to the centroidal axis of the cross member of the U-frame;

d_3 is the length of the diagonals measured as the distance sloping from the centroid of the chord to the top face of the cross member of the U-frame as shown in Figure 41;

I_1 is the second moment of area of the web member forming an arm of the U-frame in its plane of bending;

I_2 is the second moment of area of the cross member of the U-frame in its plane of bending;

I_3 is the second moment of area of the diagonals forming an arm of a U-frame about its axis perpendicular to the plane of the U-frame;

s is the horizontal offset between the top and bottom of the inclined web member. For a Warren truss, s is equal to the half spacing of the nodes on the bottom chord. For a modified Warren truss or a Pratt truss, s is equal to the spacing of the nodes on the truss;

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 main girders connected by the U-frame;

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:

a) 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

b) 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

c) 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 or a stiffened section of the chord [see Figure 42 Type (c)].

θ = 0 when the bottom truss chords are fully restrained throughout their lengths by an integral deck; otherwise θ may conservatively be taken as:

$$\frac{s}{\{(2nEI_4/B) + (mEI_5/s)\}}$$ for an end diagonal; or

$$\frac{s}{\{(nEI_4/B) + (mEI_5/s)\}}$$ for an intermediate diagonal

I_4 is the second moment of area of the cross member about its vertical axis;

I_5 is the second moment of area of the bottom chord about its vertical axis;

m = 1.5 for a Warren truss with cross members only at the positions of the nodes on the bottom chord;

= 4 for a modified Warren truss or Pratt truss with cross members at each node on the bottom chord;

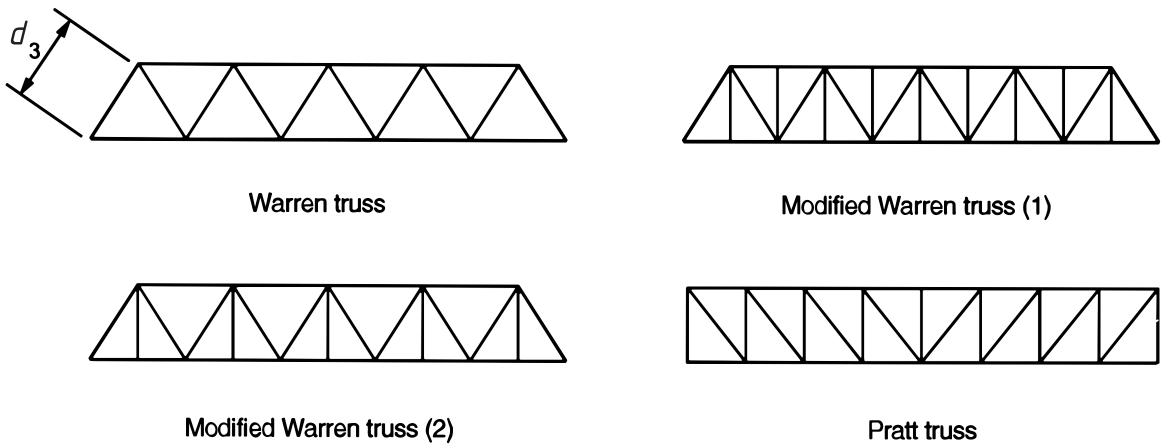
n = 1 when the connection between the chord and cross member is rigid;

= 0 when the connection between the chord and cross member is flexible.

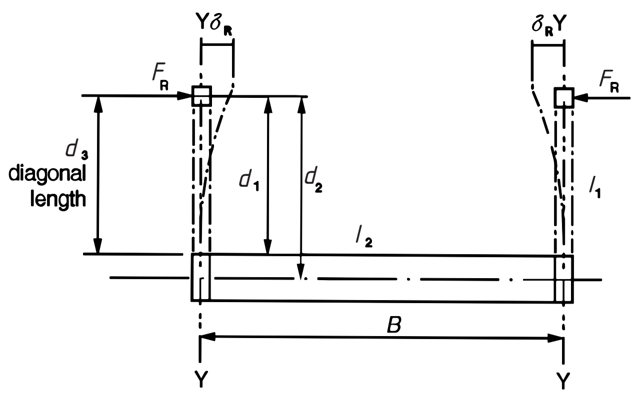
NOTE 1 A U-frame restraint should be taken into account at each connection of a web member to the compression chord. At any restraint position, more than one web member may be connected, and the members may be diagonal or vertical. The U-frame restraint assumed in design may include all the web members at each position, or may conservatively neglect the more flexible web members, such as the tension diagonals in a Pratt truss. Web members should be included only when they are adequately connected to the cross members either directly or by appropriate stiffening.

NOTE 2 Where a cross member acts with components of more than one U-frame, the moment of area I_2 should be proportioned between the components concerned.

NOTE 3 Where more than one type of intermediate U-frame occurs alternately, such as in the modified Warren truss in Figure 41 when all the web members are taken into account, then the average value of δ_R may be assumed.



a) Truss arrangement (see **12.5.1**)



b) U-frame configurations

Figure 41 — Lateral restraint of trusses by U-frame action

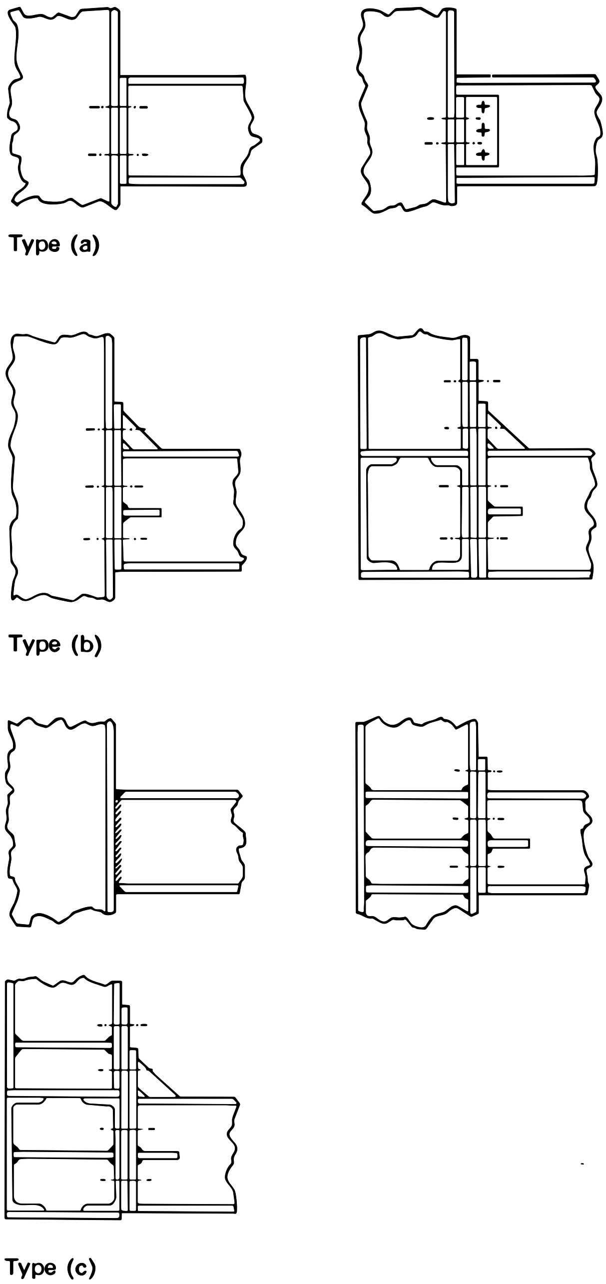


Figure 42 — U-frame joints

12.5.2 *Restraints to compression chords*

Restraints to compression chords should be in accordance with **9.12**, with chords treated as flanges.

In calculating F_R in accordance with **9.12.2**:

a) S should be taken as 1.0;
b) λ_{LT} should be taken as ℓ_e/r_y

where

r_y is the radius of gyration of the chord about the Y-Y axis (see Figure 41).

12.6 Lateral bracing

Sufficient bracing should be provided between main trusses to ensure that all external and stabilizing loads and load effects can be transmitted to the supporting structures, and that restraint is provided at all intersection points where such restraint is assumed in determining the effective length of compression members, and also at each point where a compressive force is applied to a web member, owing to change of direction of a chord (whether the chord is in tension or compression).

Bracing members and their connections to compression chords, or to U-frames restraining compression chords, should be designed to resist the forces given in **9.12.2**, with the chords treated as flanges. U-frames should be in accordance with **9.12.3** or **9.12.5**, as appropriate, with the cords treated as flanges.

12.7 Curved members

A tension or compression member curved to a circular arc (see Figure 43) may be designed in accordance with this part of BS 5400, provided that:

a) the deviation δ from the straight line joining the points of intersection at the ends of the member does not exceed one-twelfth of the length of the straight line;

b) the cross-section is compact as defined in **10.6.3**;

c) a flange outstand, if any, is such that:

$$\frac{b_o}{t_o} \leq \frac{R}{6b_o}$$

where

b_o is the width of the outstand measured from the edge to the nearest line of bolts or rivets connecting it to a supporting part of the member, or to the surface of such a supporting part in the case of welded construction, or to the root fillet of a rolled section;

t_o is the mean thickness of the outstand, or the aggregate thickness where two or more parts are connected in accordance with **14.5** or **14.6**;

R is the radius of curvature;

d) the unsupported width of the flange is such that:

$$\frac{b}{t_f} \leq \frac{R}{2b}$$

where

b is the unsupported width of flange between lines of bolts or rivets connecting the plate to supporting parts of the member, or between the surfaces of such supporting parts in the case of welded construction, or between root fillets of rolled sections;

t_f is the mean thickness of the flange over width b, or the aggregate thickness where two or more parts are connected in accordance with **14.5** or **14.6**;

e) a transverse load of uniform intensity is considered to be applied in the plane of the curve throughout the length of the member, acting on the convex side of a tension member or the concave side of a compression member, and having a value P/R, where P is the axial force in the member.

Bending moments in the member should be calculated from this load on the assumption that the member is pin-ended, and should be superimposed on the bending moments due to the stiffness of the joints determined in accordance with **12.3.2**b).

12.8 Gusset plates

12.8.1 *Strength*

Gusset plates should be capable of resisting actions from connected members in such a way that the maximum equivalent stress does not exceed:

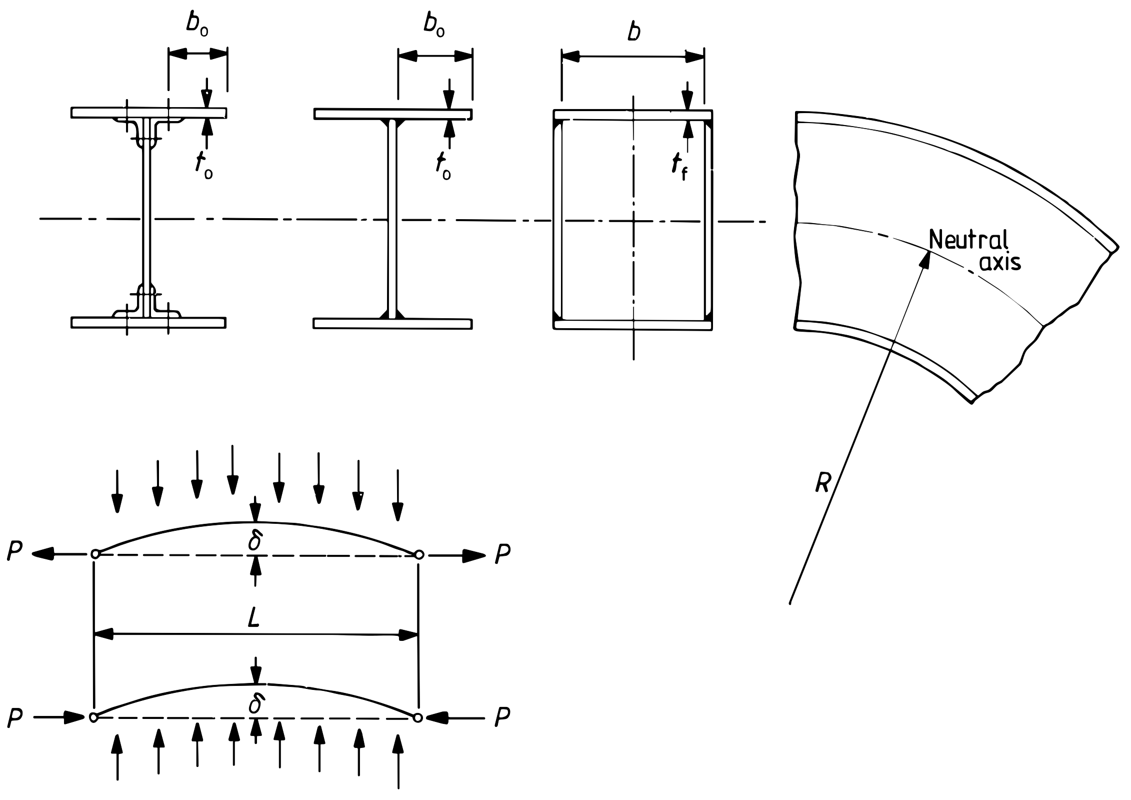
$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_y is the nominal yield stress of the gusset material as defined in **6.2**.

Any reasonable assumption as to the distribution of stresses may be made, provided that the assumed stresses are in equilibrium with the forces in the connecting members, and that the connections are in accordance with clause **14**.

In assessing the fatigue life, the stresses in a gusset plate should be determined by elastic analysis.



NOTE δ is not greater than $L/12$.

Figure 43 — Curved members

12.8.2 ***Detailing***

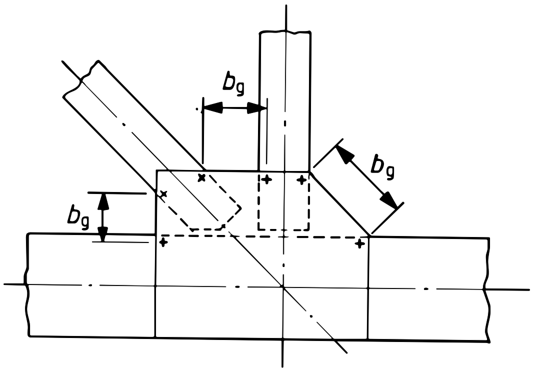
Gusset plates should be so shaped, and connectors so arranged, as to avoid severe stress concentrations. The lengths b_g of unstiffened unsupported edges shown in Figure 44 should be such that:

$$\frac{b_g}{t} \leq 50\sqrt{\frac{355}{\sigma_y}}$$

where

t is the thickness of the gusset;

σ_y is the nominal yield stress of the gusset material as defined in **6.2**.



NOTE The unsupported edge distance b_g is measured clear between fixings.

Figure 44 — Gusset plates

13 Design of base plates, cap plates and end plates

Base plates, cap plates and end plates should be designed at the ultimate limit state to satisfy the following yield criterion:

$$\frac{16M^2}{t^4} + \frac{3V^2}{t^2} \leq \left(\frac{\sigma_y}{\gamma_m \gamma_{f3}}\right)^2$$

where

- M is the maximum bending moment per unit width of the plate;
- V is the coexistent shear force per unit width;
- t is the plate thickness;
- σ_y is the nominal yield stress of the plate material as defined in **6.2**.

In addition, the calculated deformation of such a plate and the contact pressure distribution assumed should be compatible with the assumptions made in the design of the adjacent structural elements.

14 Design of connections

14.1 General

The term "connection" applies to all joints between different components of a structural member, joints between separate structural members and splices in members.

The term "fasteners" applies to bolts, rivets and pins.

14.2 Limit states

14.2.1 *Ultimate limit state*

All connections should satisfy the provisions of clause **14** for the ultimate limit state.

14.2.2 *Fatigue*

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

14.2.3 *Serviceability limit state*

Connections made with HSFG bolts in accordance with BS 4395-1 and BS 4395-2 in normal clearance holes should also satisfy the provisions of clause **14** for the serviceability limit state (see **14.5.4.1**). This limit state should be deemed to be reached when the design shear load on a bolt equals its friction capacity.

14.3 Basis of design

14.3.1 *General*

Connections should be designed on the basis of the strengths of the individual fasteners or welds, to transmit at least the design loads and moments communicated by the members.

14.3.2 *Alignment of members*

The centroidal axes of members meeting at a joint or at a splice should preferably meet at a point. When this is not the case, the moment on the connection due to any eccentricity should be taken into account.

14.3.3 *Distribution of loads between fasteners or welds*

14.3.3.1 *Elastic analysis*

The distribution of forces between individual fasteners in a bolted or riveted connection and between welds in a welded connection may be determined on the assumption that:

a) all the fasteners and all the welds share the design axial load in proportion to their respective strengths;

b) the force on a fastener or a length of weld due to a moment on the connection is proportional to its distance from the centroid of the connection.

14.3.3.2 *Plastic analysis*

Except in the case of connections made with HSFG bolts, for which the friction capacity of the fasteners is taken as the design strength in accordance with **14.5.4.2**, any reasonable distribution of the forces on the fasteners and stresses in the welds may be assumed provided that:

a) they are in equilibrium with the applied load effects;

b) the implied deformations are within the capacity of the fasteners or welds and of the connected parts;

c) each element in the connection is capable of resisting the forces or stresses assumed in the analysis.

14.3.4 ***Distribution of load to the connected members***

As far as possible, members should be so connected that the load in the connected member is appropriately distributed over its whole effective section. Where any part of a member cannot be connected so as to meet this provision, the manner in which the load effects are distributed should be considered. For this purpose, it may be assumed that the load is dispersed from a fastener into a connected part within an angle of ±45° from the direction of the force.

Groups of fasteners should be as compact as possible.

14.3.5 ***Connection of restraints to parts in compression***

A connection between a part in compression and any intermediate restraints should be designed to resist:

a) a force equal to 2.5 % of the axial force in the member acting in a direction opposite to that of the restraint; and

b) the effects of any other external load/moment on the member.

14.3.6 ***Prying force***

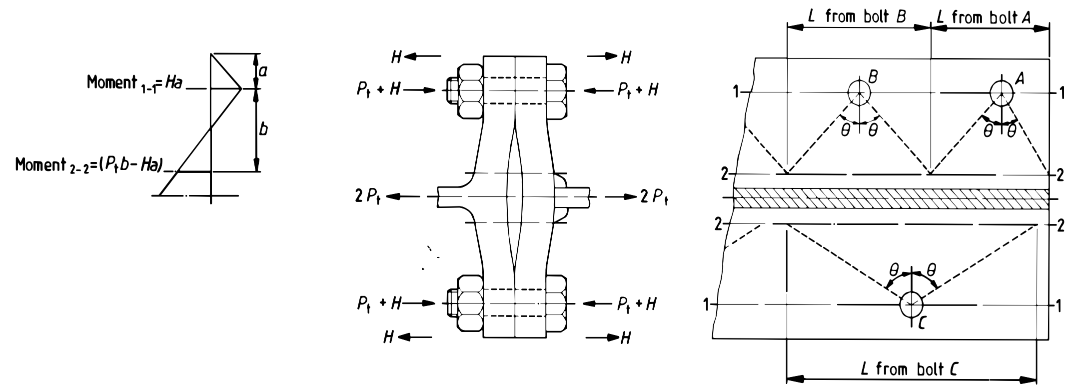
Where bolts or rivets are required to carry an applied tensile load P_t, they should be proportioned to resist an additional force H due to prying action where this can occur (see Figure 45). The force H should be assigned a value not smaller than $P_t/10$, such that the bending moments at sections 1 and 2 do not exceed:

$$\frac{M_p}{\gamma_m \gamma_{f3}}$$

where

M_p is the plastic moment capacity of the respective section based on the net area of the section.

In determining the moment capacity at section 2, the length of the section may be determined on the assumption of a dispersion from the fasteners at angles not exceeding ±60° from the normal to section 2, but not beyond half of the distance to the adjacent fasteners in the line. The same gross length may be assumed to apply when determining the moment capacity at section 1.



NOTE L is the limitation on the length of sections 1-1 and 2-2. θ is the angle of dispersal of P_t and should not exceed 60°.

Figure 45 — Prying forces

14.4 Splices

14.4.1 ***Cover material***

14.4.1.1 *General*

Where cover material is used to transmit load through a splice the following conditions should be satisfied whenever practicable:

a) both surfaces of the spliced parts should be provided with cover material;

b) cover material should be so disposed, with respect to the cross-section of the member, as to communicate the proportional load in the respective parts of the section.

Where the provisions of either a) or b) are not met, the effect of any eccentricity of cover material with respect to the centroid of the spliced section, or any part of such a section, should be considered when determining the strength both of the cover material and of the member or part.

14.4.1.2 *Welded stiffener splices*

The edges of cover material in a welded stiffener splice should be welded to the plate which is being stiffened.

14.4.2 ***Compression members***

14.4.2.1 *Loads to be transmitted*

A splice located at or near an effectively braced joint should be capable of transmitting at least the design load effects in the member.

All other splices should be capable of transmitting at least the load effects and the stresses at the spliced section due to initial imperfections. To achieve this, the splice should be designed to transmit a force equal to:

$$\frac{P}{P_D} P_{Dk}$$

in addition to any coincident moments and shears;

where

P is the load in the member or part;

P_D is the resistance of the member or part;

P_{Dk} is the resistance of the member or part determined as if it were a stocky member (see **10.6.3**).

14.4.2.2 *Design stresses*

The maximum stress σ_a in a spliced part and that in the cover material should not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_a is the axial stress or, where shear is present, the equivalent stress, based on the effective area of the cross-section determined in accordance with **10.5**;

σ_y is the nominal yield stress, as defined in **6.2**, of the spliced part, or of the cover material, as appropriate.

14.4.2.3 *Machined abutting ends of parts in compression*

A splice which has machined abutting ends in contact over the whole area of the section may be assumed to carry 75 % of any compressive load directly through the abutting ends. If the abutting area is increased by means of machined end plates, the whole compressive load may be assumed to be transmitted through the abutting faces. The alignment of the abutting ends should be maintained by cover plates or other means. The cover material and its fastenings should be proportioned to carry a force at the abutting ends, acting in any direction perpendicular to the axis of the member, equal to 2.5 % of the compressive force in the member.

14.4.3 ***Tension members***

14.4.3.1 *Loads to be transmitted*

A splice in a member or part subjected to tension should be designed to transmit at least the load in the member or part.

14.4.3.2 *Design stresses*

The maximum stress σ_a in the spliced part and that in the cover material should not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_a is the axial stress or, where shear is present, the equivalent stress, based on the effective section determined in accordance with **11.3** or 0.8 times the effective section for outer plies in connections made with HSFG bolts acting in friction;

σ_y is the nominal yield stress, as defined in **6.2**, of the spliced part, or of the cover material, as appropriate.

14.4.4 *Members in bending*

14.4.4.1 *General*

A splice in a member or part subjected to bending and axial load effects should satisfy **14.4.4.2** to **14.4.4.4** and **14.4.2** or **14.4.3** as appropriate.

14.4.4.2 *Compression flanges*

Compression flanges should be treated as compression members and spliced in accordance with **14.4.2**. In determining the load to be transmitted at a splice that is not effectively braced, the following definitions should be adopted:

P is the force in the compression flange at the splice position;

P_D is the flange compression calculated from the bending resistance of the beam at the position of the maximum bending moment;

P_{Dk} is the flange compression calculated from the bending resistance of the beam at the position of the maximum bending moment, assuming that the slenderness parameter λ_{LT} is equal to zero.

In applying **14.4.2.2**, the value of γ_m should be taken as that used for the compression flange being spliced.

14.4.4.3 *Tension flanges*

Tension flanges should be treated as tension members and spliced in accordance with **14.4.3**.

14.4.4.4 *Parts subject to shear*

A splice in a web or other part subjected to shear should be designed to transmit at least the total of:

a) the shear force at the splice;

b) the moment resulting from the eccentricity, if any, of the centroids of the groups of fasteners on each side of the splice;

c) the proportion of moment carried by the web or part, irrespective of any shedding of stress into adjoining parts assumed in the design of the member or part.

14.5 Connections made with bolts, rivets or pins

14.5.1 *Spacing of bolts and rivets*

14.5.1.1 *Minimum pitch*

The distance between centres of bolts or rivets should not be less than 2.5 times the diameter of the shank of the bolt or rivet.

14.5.1.2 *Maximum pitch*

14.5.1.2.1 *In any direction*

Except as noted in **14.5.1.2.2**, the distance between centres of two adjacent bolts or rivets should not exceed $32t$ or 300 mm, whichever is the lesser, where t is the thickness of the thinner of the outer parts joined (see Figure 46).

14.5.1.2.2 *In the direction of stress*

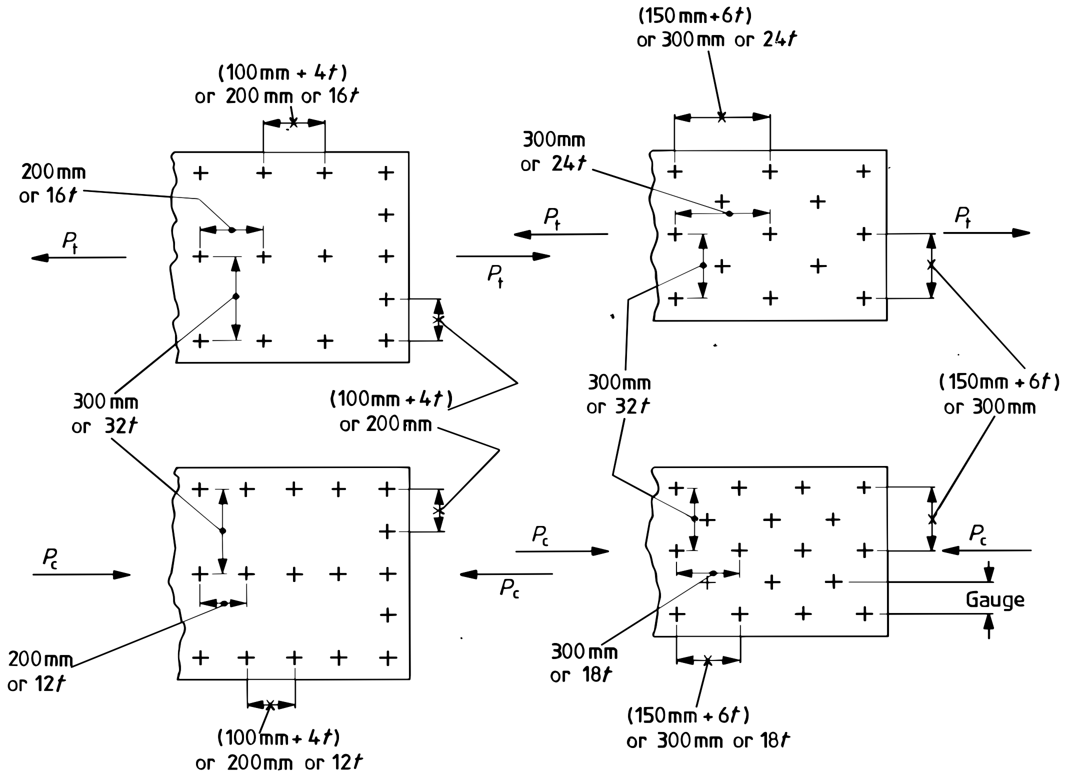
Except as noted in **14.5.1.3**, the distance between centres of two consecutive bolts or rivets in a line lying in the direction of stress should not be greater than:

a) $16t$ or 200 mm, whichever is the lesser, if the parts joined are in tension or shear;

b) $12t$ or 200 mm, whichever is the lesser, if the parts joined are in compression. Where compressive forces are transferred through abutting faces, this distance should not exceed 4.5 times the diameter of the bolts or rivets for a distance from the abutting faces equal to 1.5 times the width of the member.

14.5.1.2.3 *Adjacent to an edge*

Except as noted in **14.5.1.3**, the distance between centres of two consecutive bolts or rivets in a line adjacent to, and parallel to an edge of, an outside connected part should not be greater than (100 mm + $4t$) or 200 mm, whichever is the lesser.



a) Normal spacing b) Staggered spacing

NOTE 1 Staggered spacing values are for 75 mm gauge (maximum).

NOTE 2 Spacing of fasteners should not exceed the least, relevant value. Spacings labelled • are appropriate away from an edge, those labelled × are appropriate at an edge.

Figure 46 — Maximum pitch of bolts and rivets

14.5.1.3 *Staggered spacing*

Where bolts or rivets are staggered at equal intervals, and the gauge is not greater than 75 mm, the maximum distance between centres of bolts or rivets, as given in **14.5.1.2.2** and **14.5.1.2.3**, may be increased by 50 %.

14.5.1.4 *Spacing in stiffener attachment*

The distance between centres of two consecutive bolts or rivets connecting a stiffener to a plate or other part subjected to compression or shear should not be greater than $b/4$, or the limit obtained in **14.5.1.2.2**, whichever is lesser; where b is the maximum distance between adjacent stiffeners or between a stiffener and other support to the plate.

14.5.2 ***Edge and end distance***

The distance from the centre of the fastener hole to the edge of a part should not be less than:

a) 1.2d for bolts (other than HSFG bolts acting in friction) or rivets, or such larger distance as may be necessary to meet the provisions of **14.5.3.6**;

b) 1.5d for HSFG bolts acting in friction;

where

d is the diameter of the hole.

A line of bolts or rivets should be placed at a distance of not more than (40 mm + 4t) from any edge, where t is the thickness of the thinner outside part.

14.5.3 ***Strength of fasteners other than HSFG bolts acting in friction***

14.5.3.1 *General*

The ultimate strengths given for bolts and rivets in shear and bearing apply only to bolts and rivets in holes not larger than the sizes given in BS 5400-6:1999, **4.5**.

Black bolts should not be used in permanent main structural connections of highway and railway bridges.

For fasteners other than HSFG bolts, the ultimate limit state of a connection should be deemed to be reached when the design load on any fastener equals its ultimate capacity, determined in accordance with **14.5.3.2** to **14.5.3.11**.

14.5.3.2 *Bolts subjected to axial tension*

In a bolt subjected to applied axial tension, the tensile stress:

$$\sigma = \frac{P_t + H}{A_{et}}$$

should not exceed

$$\frac{\sigma_t}{\gamma_m \gamma_{f3}}$$

where

P_t is the externally applied tensile load;

H is the prying force determined in accordance with **14.3.6**;

A_{et} is the tensile stress area of the bolt given in BS 3692, BS 4190 or BS 4395, as appropriate;

σ_t is the lesser of 0.7 times the minimum ultimate tensile stress, and either the yield stress or the stress at a permanent set of 0.2 %, as appropriate.

14.5.3.3 *Rivets subjected to axial tension*

The use of rivets in tension should be avoided wherever possible. When unavoidable, the tensile stress:

$$\sigma = \frac{P_t + H}{A_{er}}$$

should not exceed:

$$\frac{\sigma_t}{\gamma_m \gamma_{f3}}$$

where

P_t, H are as defined in **14.5.3.2**;

A_{er} is the area of the rivet hole;

σ_t = $0.8\sigma_y$ for normal rivets;

= $0.5\sigma_y$ for countersunk rivets;

σ_y is the yield stress of the rivet material.

14.5.3.4 *Fasteners subjected to shear only*

In a fastener subjected to shear, the average shear stress:

$$\tau = \frac{V}{nA_{eq}}$$

should not exceed

$$\frac{\sigma_q}{\gamma_m \gamma_{f3}\sqrt{2}}$$

where

V	is the applied load on the fastener;
n	is the number of shear planes resisting the applied shear;
A_{eq}	for a bolt is the sectional area of the unthreaded shank, provided the shear plane or planes pass through the unthreaded part, or
	= A_{et}, if any shear plane passes through the threaded part;
A_{eq}	for a rivet is the area of the hole;
A_{eq}	for a pin is the cross-sectional area of the pin;
A_{et}	is as defined in **14.5.3.2**;
σ_q	= σ_y for all fasteners except black bolts and hand-driven rivets, or
	= $0.85\sigma_y$ for black bolts and hand-driven rivets;
σ_y	is the yield stress of the fastener material.

14.5.3.5 *Fasteners subjected to tension and shear*

Fasteners subjected to coexistent tensile and shear forces should be in accordance with **14.5.3.4** and **14.5.3.2** or **14.5.3.3**, as appropriate, and the tensile stress σ and the shear stress τ in combination should be such that:

$$\sqrt{\left(\frac{\sigma}{\sigma_t}\right)^2 + 2\left(\frac{\tau}{\sigma_q}\right)^2} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ, σ_t, τ, σ_q	are as defined in **14.5.3.2**, **14.5.3.3** and **14.5.3.4**, as appropriate.

14.5.3.6 *Bolts and rivets in bearing*

The bearing pressure σ_b between a fastener and each of the connected parts, equal to V/A_{eb}, should not exceed:

$$\frac{k_1 k_2 k_3 k_4 \sigma_y}{\gamma_m \gamma_{f3}}$$

where

V	is the load transmitted to each connected part by the fastener;
A_{eb}	for a bolt is the product of the shank diameter of the bolt and the thickness of each connected part loaded in the same direction, irrespective of the location of the thread;
A_{eb}	for a rivet is the product of the diameter of the hole and the thickness of each connected part loaded in the same direction;
A_{eb}	for countersunk bolts or rivets is as stipulated above with half the depth of the countersink deducted from the thickness of the part joined;
k_1	= 0.85 for black bolts and hand-driven rivets, or
	= 1.0 for all other fasteners;
k_2	= 2.5 except that when the force transmitted by the connector is towards the edge of the part connected (see Figure 47), the following values of k_2 should be taken:
	= 2.5 when the edge distance is greater than or equal to $3d$, or
	= 1.7 when the edge distance is $1.5d$ (minimum for HSFG bolts), or
	= 1.2 when the edge distance is $1.2d$;
d	is the diameter of the hole, and the edge distance is measured from the centre of the hole;
	NOTE For intermediate values of edge distance, k_2 may be obtained by linear interpolation.
k_3	= 1.2 if the part being checked is enclosed on both faces with the fastener acting in double shear, or
	= 0.95 in all other cases;
k_4	= 1.0 except when the fasteners are HSFG bolts acting in friction, or
	= 1.5 when the fasteners are HSFG bolts acting in friction;
σ_y	is the nominal yield stress, as defined in **6.2**, of the fastener material or of the connected part, whichever is the lesser.

For fasteners adjacent to an edge where k_2 is less than 2.5, the reduced capacity applies only to fasteners adjacent to the edge. Subject to the provisions of **14.5.5**, the total bearing capacity of the fasteners in a connection should be the sum of the full bearing capacities of fasteners away from the edge and the reduced strength to those adjacent to the edge.

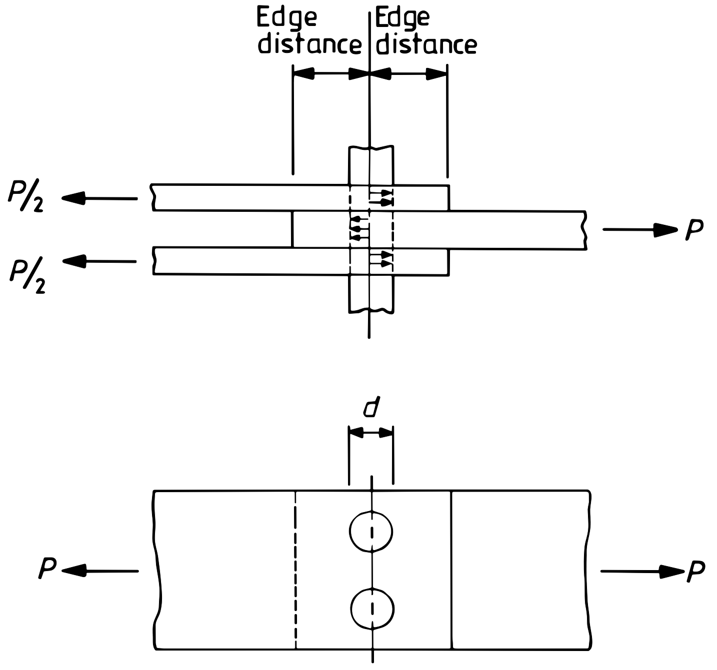


Figure 47 — Fastener force towards edge of part

14.5.3.7 *Pins in bearing*

The bearing area of a pin is the product of the pin diameter and the thicknesses of the parts in bearing loaded in the same direction. Consideration should be give to bending stresses in pins. For this purpose, the effective span should be taken as the distance between centres of the appropriate bearing areas. In addition, if the pin passes through bearing plates having a thickness greater than half the diameter of the pin, consideration should be given to the variation of bearing pressure across the thickness of the plate, and the effective span.

The following stresses should not be exceeded:

a) bending stress in the pin:

$$\frac{1.5\sigma_y}{\gamma_m\gamma_{f3}}$$

where

σ_y is the yield stress of the pin material;

b) bearing pressure where relative rotation occurs between the pin and the connected element:

$$\frac{0.75\sigma_y}{\gamma_m\gamma_{f3}}$$

where

σ_y is the nominal yield stress, as defined in **6.2**, of the connected part or of the pin, whichever is the lesser.

c) bearing pressure where such rotation does not occur:

$$\frac{1.5\sigma_y}{\gamma_m\gamma_{f3}}$$

where

σ_y is the nominal yield stress, as defined in **6.2**, of the connected part or of the pin, whichever is the lesser.

14.5.3.8 *Long grip rivets*

The grip of a rivet should not be greater than eight times the diameter of the hole.

Where the grip length of a rivet exceeds six times the diameter of the hole, the number of rivets calculated to satisfy the provisions of **14.5.3.3** to **14.5.3.6** should be increased by 1 % for each additional 2 mm of grip.

14.5.3.9 *Securing nuts*

Wherever there is a risk of nuts becoming loose they should be secured. Nuts of friction grip bolts need not be further secured after tightening.

14.5.3.10 *Bolts and rivets through packings*

The number of bolts or rivets transmitting load in bearing with packings thicker than 6 mm should be increased above the number calculated in accordance with **14.5.3.4** and **14.5.3.6** by 1.25 % for each additional millimetre thickness of packing.

For double shear connections with packings on both sides of the splice the number of additional rivets or bolts may be determined from the thickness of the thicker packing.

The additional rivets or bolts may be placed in an extension of the packing.

14.5.3.11 *Pin plates*

Pin plates may be attached to the ends of a pin-connected member in order to ensure that the provisions of **14.5.3.7** are met.

Pin plates should be of sufficient thickness to make up the required bearing or cross-sectional area and should be so arranged as to reduce the eccentricity to a minimum. Their length measured from the centre of the pin to the end (on the reaction side) should be at least equal to their width at the pin, and at least one plate on each side should be as wide as the dimensions will allow. Pin plates should be connected with enough rivets, bolts or welds to transmit the bearing pressure on them and should be so arranged as to distribute it uniformly over the full section of the member (see Figure 48).

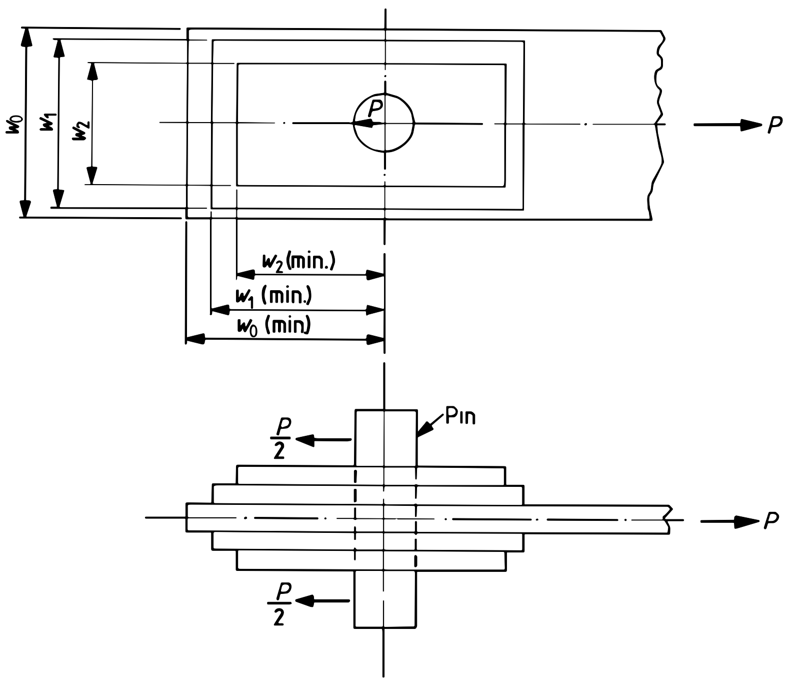


Figure 48 — Pin plates

14.5.4 *Strength of HSFG bolts acting in friction*

14.5.4.1 *General*

14.5.4.1.1 *Ultimate limit state*

For HSFG bolts in accordance with BS 4395-1 and BS 4395-2 in normal clearance holes, as specified in BS 4604, the design ultimate capacity is the greater of:

a) the friction capacity determined in accordance with **14.5.4.2** and;

b) the lesser of either the shear capacity determined in accordance with **14.5.3.4** or the bearing capacity determined in accordance with **14.5.3.6**.

For HSFG bolts in accordance with BS 4395-1 and BS 4395-2 in oversize or slotted holes, the ultimate capacity is the friction capacity determined in accordance with **14.5.4.2**.

14.5.4.1.2 *Serviceability limit state*

In a connection made with HSFG bolts in accordance with BS 4395-1 and BS 4395-2 in normal clearance holes, as specified in BS 4604, the serviceability limit state is reached when slip occurs between the parts joined, which should be deemed to occur when the shear load applied to any bolt equals its friction capacity, determined in accordance with **14.5.4.2**.

14.5.4.2 *Friction capacity*

The friction capacity P_D of a HSFG bolt should be taken as:

$$P_D = k_h \frac{F_v \mu N}{\gamma_m \gamma_{f3}}$$

where

F_v is the prestress load, as defined in **14.5.4.3**;

μ is the slip factor, having a value in accordance with **14.5.4.4**;

N is the number of friction interfaces;

k_h is 1.0 where the holes in all the plies are of normal size, as specified in BS 4604, otherwise k_h is as stated in **14.5.4.5**.

14.5.4.3 *Prestress*

The prestress load F_v of a HSFG bolt should be taken as:

$$F_v = F_o - F_t$$

where

F_o is the initial load, i.e. the proof load given in BS 4395, except that for bolts in accordance with BS 4395-2 it should be 0.85 times the proof load;

F_t is the externally applied tensile load, if any.

14.5.4.4 *Slip factor*

Unless determined by test, the slip factor μ at friction surfaces should be taken as:

a) μ = 0.45 for weathered surfaces clear of all mill scale and loose rust;

b) μ = 0.50 for surfaces blasted with shot or grit and with loose rust removed;

c) μ = 0.50 for surfaces sprayed with aluminium;

d) μ = 0.40 for surfaces sprayed with zinc;

e) μ = 0.35 for surfaces treated with zinc silicate paint;

f) μ = 0.25 for surfaces treated with etch primer.

The slip factors given in a) to f) should be reduced by 10 % where higher grade bolts in accordance with BS 4395-2 are used.

If the friction surfaces are not any of those listed in a) to f), the characteristic value of the slip factor should be established by testing in accordance with BS 4604 or obtained from other reliable sources to the satisfaction of the Engineer.

14.5.4.5 *Oversized and slotted holes*

Where there are three or more plies, oversized or slotted holes may be used in the inner plies in accordance with Table 12, subject to a reduction of the friction capacity given in **14.5.4.2** by a factor of k_h, where k_h = 0.85 for oversized and short slotted holes, or k_h = 0.70 for long slotted holes.

Table 12 — Oversized and slotted holes

Diameter of bolt mm	Maximum size of hole		
	Diameter of oversized hole mm	Short slotted hole mm	Long slotted hole mm
16	21	18 × 22	18 × 40
20	25	22 × 26	22 × 50
22	27	24 × 28	24 × 55
24	30	26 × 32	26 × 60
27	33	30 × 35	30 × 70
30	38	33 × 40	33 × 80
36	44	39 × 46	39 × 90

NOTE The recommendations given in **14.5.4** apply only to bolts tightened in accordance with BS 4604-1, BS 4604-2 and BS 4604-3.

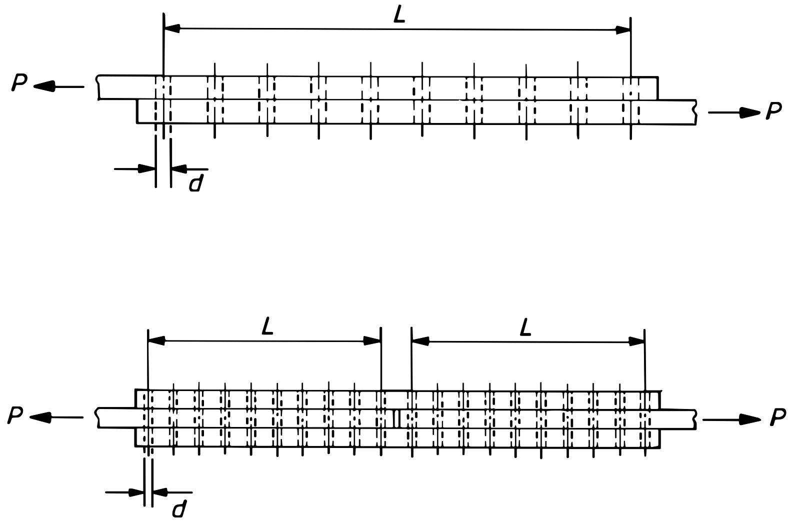
14.5.5 Long connections

Where the distance L between centres of end fasteners, measured in the direction of the load transmitted, in splices or end connections of tension and compression members (see Figure 49) is more than $15d$, the strength of all the fasteners determined in accordance with **14.5.3** or **14.5.4** should be reduced by a multiplying factor k_5 given by:

$$k_5 = 1 - \left(\frac{L - 15d}{200d}\right), \text{ but not less than } 0.75,$$

where

d is the diameter of the fasteners.



NOTE Long connections are those for which L is greater than $15d$.

Figure 49 — Long connections

14.6 Welded connections

14.6.1 *General*

Welds should be detailed in accordance with the appendices of BS 5135:1984, unless otherwise recommended in this part of BS 5400. The design strengths given in **14.6.2.3** and **14.6.3.11** are valid only when the yield stress of the weld metal is at least equal to that of the parent metal. Welds made in accordance with BS 5400-6 may be deemed to satisfy this provision.

14.6.2 *Butt welds*

14.6.2.1 *Intermittent butt welds*

Intermittent butt welds should not be used.

14.6.2.2 *Partial penetration butt welds*

Partial penetration butt welds should not be used to transmit tensile forces, nor to transmit a bending moment about the longitudinal axis of the weld.

Unless determined by procedure trials, the throat thickness of a partial penetration butt weld should be taken as:

a) the depth of weld preparation where this is of the J or U type;

b) the depth of the weld preparation minus 3 mm where the preparation is of the V or bevel type.

Where determined by procedure trials, the throat thickness should not be taken as more than the penetration consistently achieved, ignoring weld reinforcement.

14.6.2.3 *Strength of butt welds*

The strength of a full penetration butt weld should be taken as equal to the strength of the weaker of the parts joined.

The strength of a partial penetration butt weld, together with its reinforcing fillet weld, if any, should be calculated as for a full penetration fillet weld.

14.6.3 *Fillet welds*

14.6.3.1 *Intermittent fillet welds*

The clear unconnected gap between the ends of the welds, whether in line or staggered, should not be more than 200 mm, and also should not be more than:

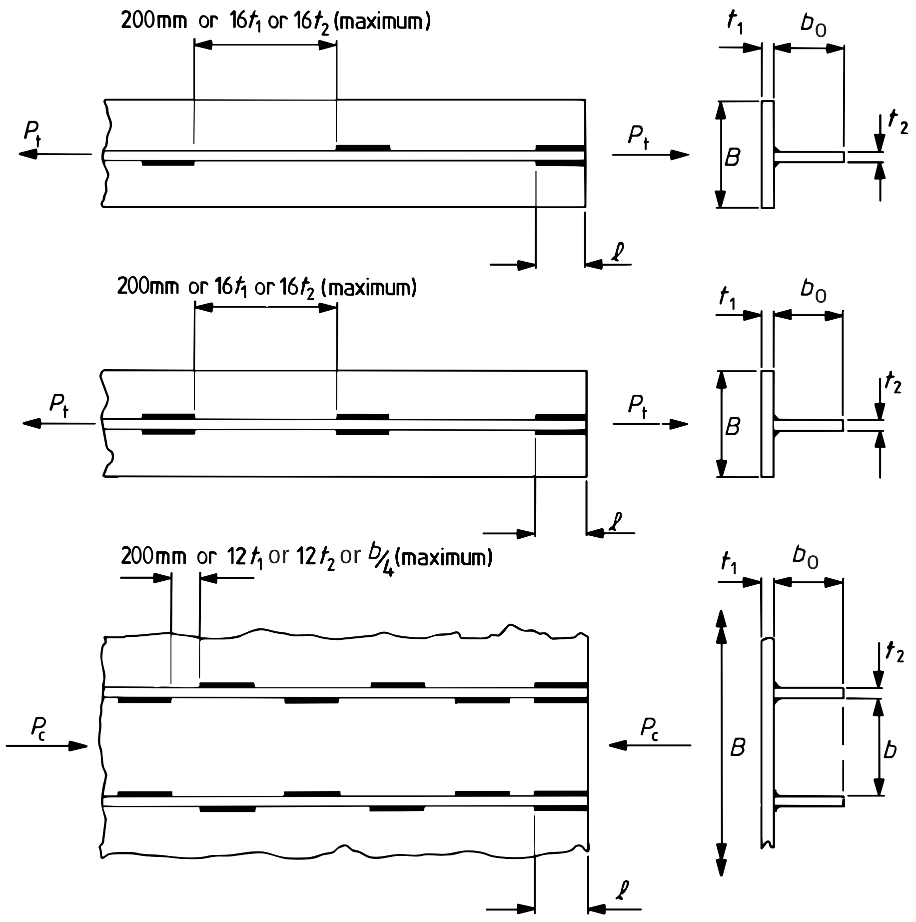
a) 12 times the thickness of the thinner part when the part is in compression;

b) 16 times the thickness of the thinner part when the part is in tension;

c) one-quarter of the distance between stiffeners when used to connect stiffeners to a plate or other part subjected to compression or shear.

In a line of intermittent welds there should be a weld at each end of the part connected.

In built-up members in which plates are connected by intermittent welds, continuous side fillet welds should be provided at the ends of each side of the plate for a length at least equal to three-quarters of the width of the narrower plate concerned (see Figure 50).



NOTE ℓ should not be shorter than the least of $0.75B$ or $0.75b_o$ or $0.75b$.

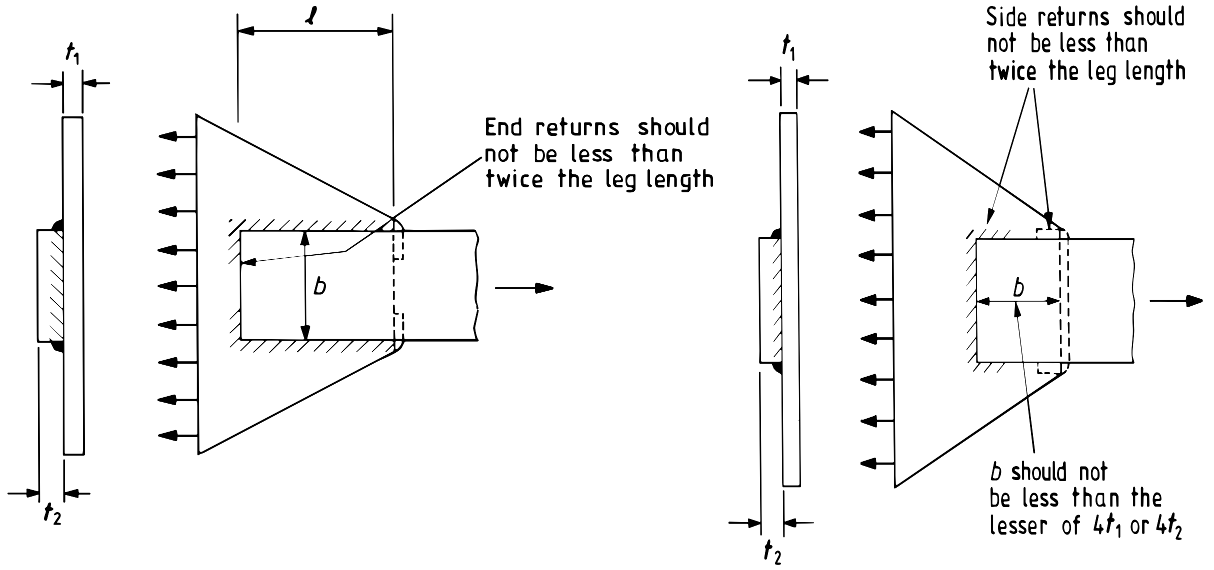
Figure 50 — Intermittent fillet welds

14.6.3.2 *End returns*

A fillet weld should be returned continuously around the corner at the end or the side of a part, for a length beyond the corner of not less than twice the leg length of the weld.

14.6.3.3 *End connections by side fillets*

If the end of a part is connected by side fillet welds only, both sides of the part should be welded and, where possible, the length of weld on each side should be not less than the distance between the welds b on the two sides [see Figure 51a)], nor less than four times the thickness of the thinnest part connected. Where the distance between the welds exceeds 16 times the thickness of the thinnest part connected, intermediate plug or slot welds should be provided to prevent separation.



a) Side fillets

b) Transverse fillets

NOTE 1 ℓ should be greater than b when practicable, and should not be less than the lesser of $4t_1$ or $4t_2$.

NOTE 2 Intermediate plug or slot welds are needed when b is greater than the lesser of $16t_1$ or $16t_2$.

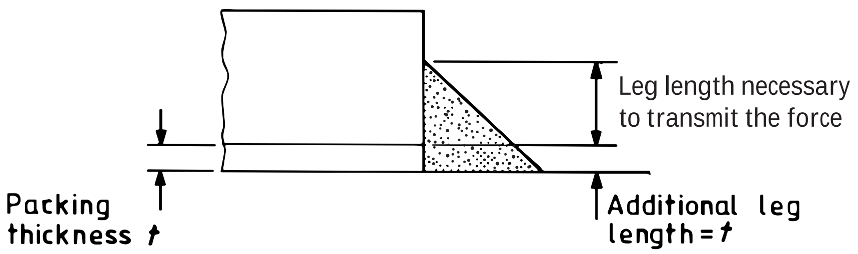
Figure 51 — End connections by fillet welds

14.6.3.4 *End connections by transverse welds*

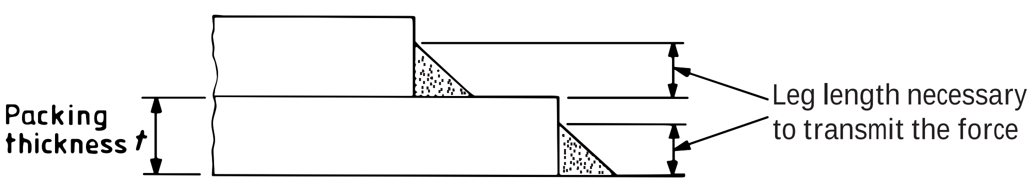
The overlap between the connected parts should not be less than four times the thickness of the thinnest part and the parts should be connected by two transverse lines of welds [see Figure 51b)]. Where the distance between the welds exceeds 16 times the thickness of the thinnest part connected, intermediate plug or slot welds should be provided to prevent separation.

14.6.3.5 *Welds with packings*

Where two parts connected by welding are separated by packing having a thickness less than the leg length of weld necessary to transmit the force, the leg length needed should be increased by the thickness of the packing [see Figure 52a)]. The packing should be trimmed flush with the edge of the part which is to be welded. Where two parts connected by welding are separated by packing having a thickness equal to, or greater than, the leg length of weld necessary to transmit the force, each of the parts should be connected to the packing by a weld capable of transmitting the design force [see Figure 52b)].



a) t less than necessary leg length



b) t greater than or equal to the necessary leg length

Figure 52 — Welds with packings

14.6.3.6 *Welds in holes and slots*

Fillet welds in holes or slots may be used to transmit shear in lap joints, or to prevent the buckling or separation of lapped parts, or to join the components of built-up members. For the purposes of strength calculations the effective area of such welds should be determined in accordance with **14.6.3.10**.

14.6.3.7 *Longitudinal welds in members subject to bending*

The distribution of longitudinal shear per unit length of a weld connecting parts of members subjected to bending (e.g. flange and web of beams) should be determined in accordance with linear elastic theory.

14.6.3.8 *Effective length of fillet welds*

When applicable, the effective length of welds should be taken as specified in BS 5135.

The effective length of a continuous weld round the perimeter of a hole or slot should be taken as the length of the centroidal axis of the throat of the weld.

The effective length of welds connecting webs of deep beams (including diaphragms) to other parts of the structure, and of the longitudinal welds in the end connections and splices of axially loaded members, should be taken as ηL,

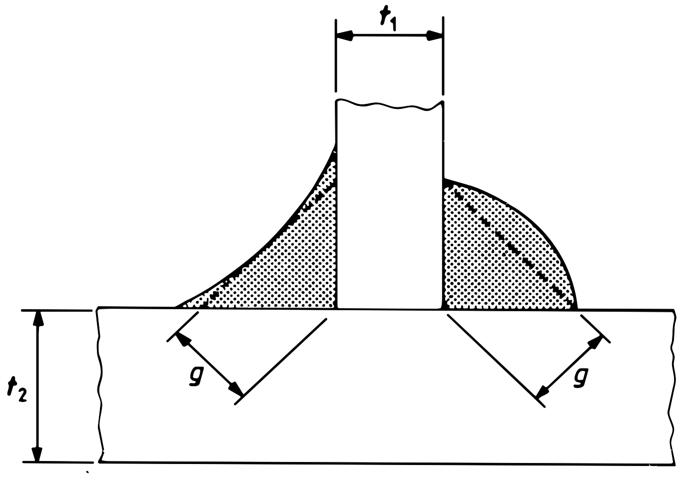
where

η	is given by $1.10 - (0.05\xi - 0.04)L$, but is not greater than 1.0;
L	is the length of the weld in metres or 8 m, whichever is the lesser;
ξ	is the ratio of the maximum to the average longitudinal shear stress in the weld. ξ may be taken as equal to 2.0 unless determined by linear elastic analysis.

14.6.3.9 *Effective throat of a fillet weld*

The throat of a fillet weld g is the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side (see Figure 53).

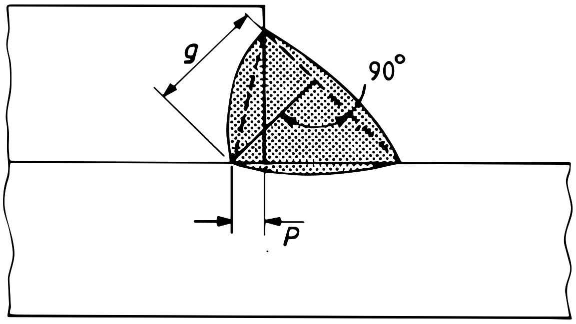
For this purpose the weld should be taken to include any specified penetration, provided that it is shown by procedure trials to the satisfaction of the Engineer that the required penetration can be consistently achieved. In a fillet weld made by the submerged arc process the penetration p may be assumed, without trials, to be 2 mm or $0.2g$, whichever is the lesser (see Figure 54).



NOTE 1 g is the effective throat of the weld.

NOTE 2 If either t_1 or t_2 is greater than 4 mm, g should be a minimum of 3 mm.

Figure 53 — Effective throat of fillet weld



NOTE *p* is the penetration of the weld.
g is the throat of the weld.

Figure 54 — Penetration of fillet weld

14.6.3.10 *Effective area of a fillet weld*

The effective area of a fillet weld is its throat dimension multiplied by its effective length, except that for fillet welds in holes or slots the effective area should not be taken as greater than the area of the hole or slot.

14.6.3.11 *Capacity of a fillet weld*

14.6.3.11.1 *Weld subject to longitudinal shear only i.e. shear in the direction of its length* [see Figure 55a)]

The stress in a weld, calculated as the longitudinal shear force per unit length P_L divided by the effective throat g, should not exceed:

$$\frac{\sigma_w}{\sqrt{3}\gamma_m\gamma_{f3}}$$

where

σ_w is the yield stress of the deposited weld metal and may be taken as:

$1/2(\sigma_y + 455)$;

σ_y is the smaller nominal yield stress value, as defined in **9.3.1** or **10.3.1**, of the two parts joined, as appropriate.

14.6.3.11.2 *Weld subject to transverse force only i.e. force at right angles to its length* [see Figure 55b)]

The stress in a weld, calculated as the transverse force per unit length P_{T1} (or P_{T2}) divided by the effective throat g, should not exceed:

$$\frac{K\sigma_w}{\sqrt{3}\,\gamma_m\gamma_{f3}}$$

where

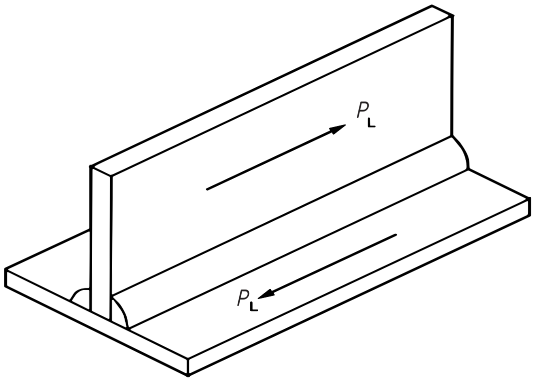
σ_w is as defined in **14.6.3.11.1**;

K depends on the angle θ between the direction of the applied force and the throat and is given by:

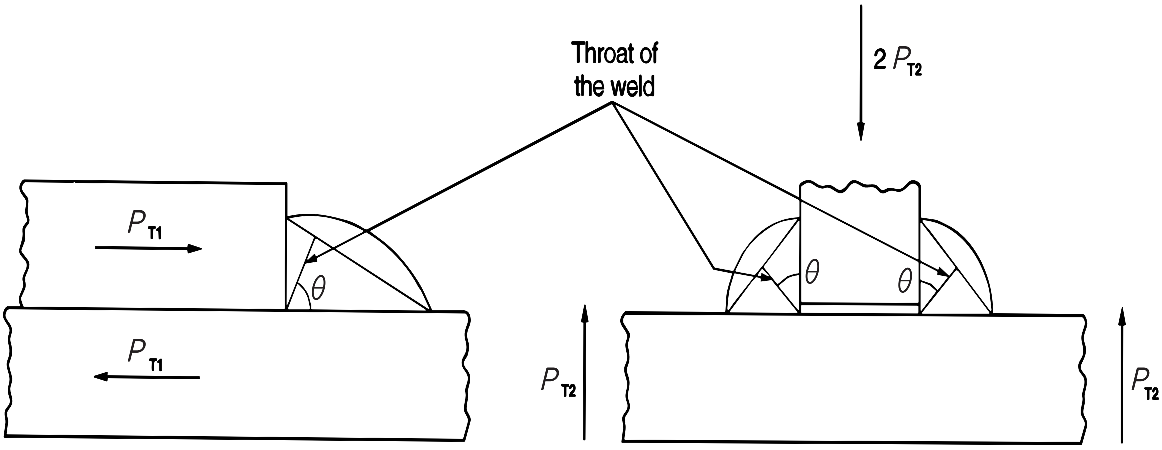
$$K = \sqrt{\frac{3}{1 + 2\cos^2\theta}}$$

but is not greater than 1.4.

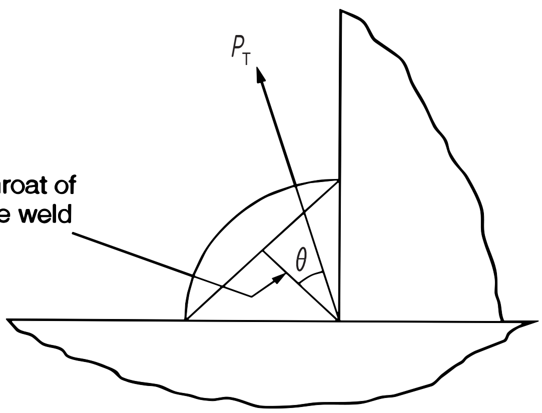
NOTE For equal leg fillets between components at right angles $\theta = 45°$ and $K = 1.225$.



a) Weld subjected to longitudinal shear



b) Weld subjected to transverse force



c) Resultant transverse force at weld

Figure 55 — Forces in fillet weld

14.6.3.11.3 *Weld subject to forces in both transverse and longitudinal directions*

The following condition should be satisfied for welds subject to forces in both transverse and longitudinal directions:

$$\frac{1}{g}\sqrt{P_L^2 + \frac{P_T^2}{K^2}} \leq \frac{\sigma_w}{\sqrt{3}\,\gamma_m \gamma_{f3}}$$

where

P_L is the longitudinal shear force per unit length of the weld;

P_T is the resultant of transverse forces per unit length of the weld [see Figure 55c)];

g is the effective throat of the weld;

σ_w is as defined in **14.6.3.11.1**;

K is given by:

$$\sqrt{\frac{3}{1 + 2\cos^2\theta}}$$

but is not greater than 1.4;

θ is the angle between the resultant transverse force and the throat.

14.6.4 ***Plug welds***

Plug welds should not normally be used for transmitting applied load. Plug welds in circular or slotted holes may be used to prevent the buckling or separation of lapped parts, or to join the components of built-up members. The entire area of the hole or slot should be filled with weld metal having a thickness which is either:

a) equal to the thickness of the holed or slotted part, where this is 16 mm or less; or

b) in other cases, not less than any of the following:

1) 16 mm;

2) 0.45 times the diameter of the hole or the width of the slot;

3) one-tenth of the length of the slot;

but which need not be greater than the thickness of the holed or slotted part.

The diameter of a hole, or the width of a slot, should not be less than the thickness of the holed or slotted part plus 8 mm.

The distance between centres of holes, or between the centrelines of slots, should not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of slots measured in the direction of their length should not be less than double the length of a slot.

The ends of a slot should be semicircular, except where the slot terminates at the edges of the part, when it can be square.

14.6.5 ***Load transfer by parts in contact***

Where a good fit is ensured between a flat surface and an edge of a section abutting it, the forces applied to either part transmitted to the other in direct bearing may be taken as follows:

a) the whole of such forces if the surfaces are machined;

b) 75 % of such forces if the surfaces are sawn or flame cut by machine.

14.7 Hybrid connections

14.7.1 ***Allowable combinations***

The following combinations of fasteners and welds in a connection may be taken as sharing the loads, transmitted by the connection, in proportion to their respective strengths at the ultimate limit state:

a) rivets, close tolerance bolts and turned barrel bolts when acting in shear or bearing;

b) welds and HSFG bolts acting in friction, provided that the ultimate capacity for the bolts is in accordance with **14.5.4.2** and that the procedure of making the joint is such that there is no distortion of the faying surfaces. However, the ultimate strength of the connection should not be taken as greater than 90 % of the combined strengths.

14.7.2 ***Other combinations***

With all other combinations of fasteners and welds in a connection, one type of the fasteners or welds should be assumed to transmit the loads, unless the deformation capacities of the different fasteners or welds have been proved to the satisfaction of the Engineer to be compatible and sufficient to share the loads.

14.8 Lug angles

Lug angles connecting angle members and their fastenings to the gusset or other supporting part should be designed, in accordance with clause **10** or clause **11** as appropriate, to transmit a force 20 % greater than the force in the outstand of the angle connected. The fastenings connecting the lug angle to the outstand of the angle member should be designed to transmit a force 40 % greater than the force in the outstand of the angle member.

Lug angles connecting a channel or similar member should be disposed symmetrically about the axis of the member, and, together with their fastenings to the gusset or other supporting part, should be designed to transmit a force 10 % greater than the force in the component of the member not directly connected. The fastenings connecting the lug angles to the member should be designed to transmit a force 20 % greater than such excess force.

In no case should less than two bolts or rivets be used to attach a lug angle to a gusset or other supporting part.

The connection of the lug angle to the gusset or other supporting part should terminate at the end of the member connected. The connection of the lug angle to the member should run from the end of the member to a point beyond the direct connection of the member to the gusset or other supporting part.

14.9 Other attachments

The dimensions of any other attachments such as brackets, stools and cleats should be such that:

a) the maximum equivalent stress does not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_y is the nominal yield stress of the material of the attachment as defined in **6.2**;

b) their deformation under load is compatible with the distribution of forces assumed in the design of the connection;

c) buckling does not occur in any component or in a free edge.

Annex A (informative)
Evaluation of effective breadth ratios

A.1 General

The procedures given in this annex may be used to determine the effective breadth ratios for conditions not specifically covered by **8.2** or where a special study is warranted. Examples are where point loads of significant magnitude act on a bridge deck, either in isolation or in combination with other loads, and where there are single spans with cantilevered projections continuous over the supports.

The notation used in this annex is the same as that used in **8.2** except that L may be taken as the distance between adjacent points at which the bending moment is zero.

A.2 Equivalent simply supported spans

In structures other than simply supported beams, the effective breadth ratio ψ may be obtained by treating each portion of a continuous beam between adjacent points of zero moment as an equivalent simply supported span, and by using Table 4 or Table A.1, as appropriate.

The positions of the points of zero moment should be those corresponding to the particular loading under consideration. In the special case of a portion of a span between a fixed end and an adjacent point of zero moment, the equivalent span should be obtained by considering a fictitious symmetrical span extending beyond the fixed end with the loading and reactions applied symmetrically about the fixed end.

A.3 Point loads at mid-span

For point loads and other concentrated loads at mid-span, or at the free end of a cantilever, the effective breadth ratio ψ may be obtained from Tables A.1, A.2, A.3 or A.4. These tables should only be used for point loads and reactions of significant magnitude and should not be used for standard highway wheel or axle loads.

A.4 Point loads not at mid-span

For point loads on a simply supported beam at positions other than mid-span, the effective breadth ratio ψ under the point of application of the load may be determined from:

$$\psi = 0.33\,\{2\psi_x + \psi_{(L-x)}\}$$

where

- ψ_x is the value of ψ from Table A.1 for a point load at mid-span with $L = 2x$;
- $\psi_{(L-x)}$ is the value of ψ from Table A.1 for a point load at mid-span with $L = 2(L - x)$;
- x is the shorter distance from the end of the span to the point of application of the load.

In the special case of a simply supported beam with $b/L < 0.1$, the effective breadth ratio ψ, under a point load anywhere in the span, may be taken as the effective breadth ratio ψ from Table A.1 for a point load at mid-span.

The effective breadth ratio at all points in the span or equivalent simply supported span, at a distance of more than $L/4$ from the point load, may be assumed to be the value of ψ given in Table A.1 at quarter-span. Within a distance $L/4$ of the point load, the effective breadth ratio may be assumed to vary linearly between the value at the load and the value at $L/4$ from the point load.

Where the distance between the point load and the support is less than $L/4$, the effective breadth ratio throughout that distance may be taken as the value under the load point.

A.5 Combination of loads

Under a combination of distributed and/or point loads the values of ψ may be derived from:

$$\psi = \frac{\Sigma M}{(M_1/\psi_1) + (M_2/\psi_2) \ldots + (M_n/\psi_n)}$$

where

- $M_1 \ldots\ldots M_n$ are the bending moments at the cross-section considered due to each component load;
- ΣM is the total bending moment at the same section due to load components 1 to n;
- $\psi_1 \ldots\ldots \psi_n$ are the effective breadth ratios for the same section appropriate to each load component, using Tables 4 to 7 for distributed loads and Tables A.1 to A.4 for concentrated loads.

NOTE In calculating the value of ψ, due account should be taken of the algebraic sign of the bending moments.

A.6 Transverse distribution of stress

The longitudinal stress σ_1 at any point in the flange at a distance x from the centreline of the web may be calculated from:

$$\sigma_1 = \sigma_{max.} \{\chi^4 + k\,(1 - \chi^4)\}$$

where

χ is given by $(b - x)/b$;

k is given by:
$0.25(5\psi - 1)$ for portions between web centrelines, or
$0.25\left\{5\psi\left(1 - 0.15\,\dfrac{b}{L}\right) - 1\right\}$ for portions projecting beyond an outer web;

$\sigma_{max.}$ is the maximim stress in the flange due to longitudinal bending of the section, calculated by elastic analysis using the effective flange breadth determined in accordance with **8.2**;

ψ, b, L are as defined in **8.2**;

x is as given in Figure A.1.

NOTE If the calculated value of σ_1 is negative it should be taken as zero.

Table A.1 — Effective breadth ratio ψ for simply supported beams for point load at mid-span

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	0.80	0.75	1.00	1.00	1.00	1.00
0.10	0.67	0.59	1.00	0.99	1.00	1.00
0.20	0.49	0.40	0.98	0.84	1.00	0.93
0.30	0.38	0.30	0.80	0.61	0.87	0.69
0.40	0.30	0.23	0.63	0.44	0.70	0.51
0.50	0.24	0.17	0.48	0.32	0.54	0.37
0.75	0.15	0.10	0.25	0.19	0.31	0.22
1.00	0.12	0.08	0.19	0.14	0.20	0.15

Table A.2 — Effective breadth ratio ψ for interior spans of continuous beams for point load at mid-span

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	0.67	0.59	1.00	1.00	0.67	0.59
0.10	0.49	0.40	1.00	0.93	0.49	0.40
0.20	0.30	0.23	0.70	0.51	0.30	0.23
0.30	0.19	0.14	0.42	0.29	0.19	0.14
0.40	0.14	0.10	0.28	0.20	0.14	0.10
0.50	0.12	0.08	0.20	0.15	0.12	0.08
0.75	0.09	0.06	0.08	0.07	0.09	0.06
1.00	0.08	0.04	0.02	0.05	0.08	0.04

Table A.3 — Effective breadth ratio ψ for propped cantilever beams for point load at mid-span

b/*L*	Fixed end		Quarter-span near fixed end[a]		Mid-span	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.68	0.61	1.00	1.00	0.69	0.62
0.10	0.51	0.42	1.00	0.98	0.53	0.44
0.20	0.33	0.25	0.77	0.58	0.34	0.27
0.30	0.21	0.15	0.48	0.32	0.23	0.18
0.40	0.15	0.11	0.32	0.23	0.19	0.14
0.50	0.12	0.08	0.23	0.17	0.15	0.10
0.75	0.10	0.06	0.10	0.09	0.10	0.07
1.00	0.08	0.05	0.05	0.04	0.07	0.05

[a] May also be used for a propped end.

Table A.4 — Effective breadth ratio ψ for cantilever beams for point load at free end

b/*L*	Fixed end		Mid-span		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	0.89	0.86	1.00	1.00	1.00	1.00
0.10	0.80	0.75	1.00	1.00	1.00	1.00
0.20	0.67	0.59	1.00	0.99	1.00	1.00
0.30	0.56	0.47	1.00	0.94	1.00	1.00
0.40	0.49	0.40	0.98	0.84	1.00	0.93
0.50	0.43	0.35	0.88	0.71	0.97	0.81
0.75	0.32	0.25	0.66	0.47	0.74	0.55
1.00	0.24	0.17	0.48	0.33	0.54	0.37

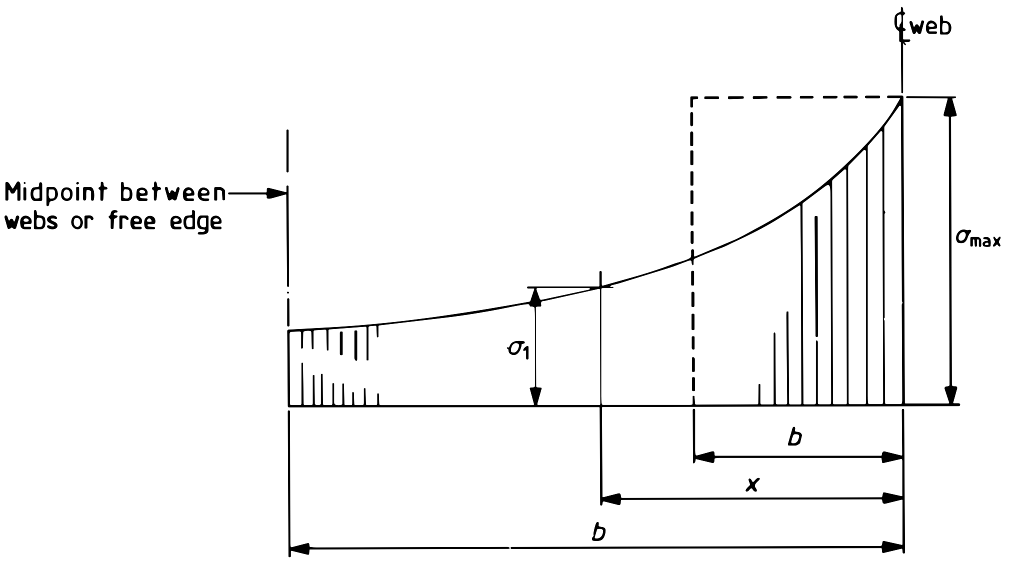


Figure A.1 — Distribution of longitudinal stress in the flange of a beam

Annex B (informative)

Distortion and warping stresses in box girders

B.1 General

When a bridge is subject to live loading, as specified in BS 5400-2, and comprises of one or more single-cell box girders, the simplified procedure given in **B.2** to **B.4** may be used to calculate transverse and longitudinal stresses due to restraint of warping. This procedure may also be used for multi-cell box girders, provided that interior webs are ignored for this purpose.

B.2 Restraint of torsional warping

When an increment of torque, T, is applied at a section of a box girder (other than at a free end) the resulting maximum longitudinal stress at this section due to restraint of torsional warping, σ_{TW}, may be calculated as follows.

a) *Stresses at the junction between the bottom flange and web*
At the junction between the bottom flange and a web, at the section where an increment of torque, T, is applied:

$$\sigma_{TWB} = \frac{DT}{J}$$

where

D	is the depth of the box at its centreline measured between centres of flange plates, or, in composite construction, between the effective centroid of the composite top flange and the centre of the bottom flange plate;
J	is the torsional constant $4A_o^2/\Sigma(w/t)$;
A_o	is the area enclosed by the median line of the perimeter material of the section;
w, t	are the width and thickness, respectively, of each wall of the section forming the closed perimeter.

NOTE In the case of a wall made from material other than steel, t should be taken as the actual thickness multiplied by the ratio of the shear modulus of the material used to the shear modulus of steel. Where the shear modulus varies with the load history, the long-term value should be used.

b) *Stresses at the junction between the top flange and the web*
At the junction between the top flange and a web, at the section where an increment of torque T is applied:

$$\sigma_{TWT} = \left(\frac{B_B}{B_T}\right)^2 \left(\frac{B_T}{B_T + 2B_c}\right)^3 \frac{DT}{J}$$

where

B_c	is the width of flange projection beyond the centre of the web;
B_B, B_T	are the widths of the bottom and top of the flanges, respectively, measured between the centres of webs;
D, J	are as defined in a).

NOTE Where there are two or more box girders in a single structure, σ_{TWT} may be taken as zero.

c) *Distribution of stresses*
At a distance x from the section where an increment of torque is applied:

$$\sigma_{TWx} = \sigma_{TW}\, e^{-(2x/B_B)}$$

The distribution across the section of the longitudinal stress due to restraint of torsional warping may be assumed to be as shown in Figure B.1.

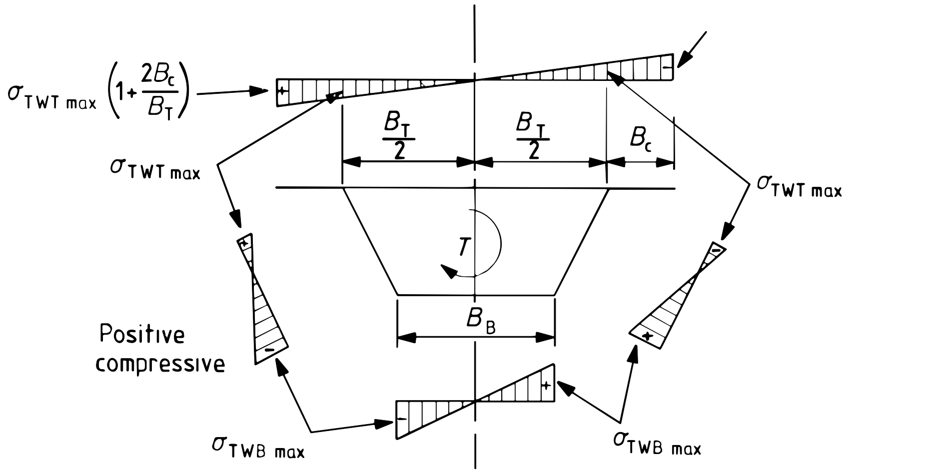


Figure B.1 — Longitudinal stresses due to restraint of torsional warping

B.3 Restraint of distortional warping

B.3.1 *General*

When torque is applied to a box girder other than at a cross frame or a diaphragm, the resulting longitudinal stress due to restraint of distortional warping σ_{DW} may be calculated in accordance with **B.3.2**, provided that the cross frames or diaphragms are in accordance with **B.3.4**.

B.3.2 *Corner stresses*

The distortional warping stress σ_{DW} may be calculated as follows.

a) At a junction between a flange and a web, under a uniformly distributed applied torque T_{UD} per unit length of span:

$$\sigma_{DW} = \frac{T_{UD}\bar{y}L_D^2}{4.5B_T I_x} \text{ when } \beta L_D < 1.6$$

$$\sigma_{DW} = 0.6\,\frac{T_{UD}\bar{y}L_D^2}{\beta^2 L_D^2 B_T I_x} \text{ when } \beta L_D \geq 1.6$$

where

$\bar{y}$	is the distance from the horizontal neutral axis to the flange/web junction;
I_x	is the second moment of area of the girder, inclusive of its effective flanges, about the horizontal neutral axis;
L_D	is the spacing of cross frames or diaphragms in accordance with **B.3.4**;
B_T	is as defined in **B.2**b);
βL_D	is given by $\left(\frac{KL_D^4}{EI_x}\right)^{0.25}$;
K	is given by $\frac{24D_{YT}R_D}{B_T^3}$;
D_{YT}	is the transverse flexural rigidity, EI, of the top flange, including transverse stiffeners if any, per unit length of span;
R_D	may be obtained from Figure B.2a) for a rectangular box, and from Figure B.2b) to d) for a trapezoidal box with webs inclined at 30° from the vertical. Values of R_D for trapezoidal boxes in which the webs are inclined at less than 30° from the vertical may be obtained by linear interpolation, or from:

$$R_D = \frac{B_B + B_T}{B_B\left(\frac{B_B}{B_T + B_B}\right)\left(\frac{2D_{YT}}{D_{YC}}\frac{d}{B_T} + 1\right) - B_B V_D\left\{\left(2 + \frac{B_B}{B_T}\right)\frac{D_{YT}}{D_{YC}}\frac{d}{B_T} + 1\right\}}$$

where

V_D is as defined in **B.4.2**;

B_B, B_T are as defined in **B.2**.

In Figure B.2,

φ_T is given by $\frac{d}{B_T}$;

D_{YB} is the transverse flexural rigidity, EI, of the bottom flange, including transverse stiffeners, if any, per unit length of span;

d is the clear depth of the web measured in the plane of the web, or, if corner stiffening is provided, the distance between the centres of connections of such stiffening to the web;

D_{YC} is the transverse flexural rigidity, EI, of the web, including its transverse stiffeners, if any, per unit length of span.

b) At a junction between a flange and a web, due to a concentrated applied torque T where the axle or concentrated load is applied mid-way between diaphragms:

$$\sigma_{DW} = \frac{T\,\bar{y}L_D}{B_T I_x} \text{ when } \beta L_D \leq 1.0;$$

$$\sigma_{DW} = \frac{T\,\bar{y}L_D}{\beta L_D I_x B_T} \text{ when } \beta L_D > 1.0.$$

c) Under uniformly distributed loading (UDL) and concentrated loading, the effects due to each of them should be separately calculated as described in a) and b), and the sum of the resulting stresses taken.

d) Under a series of concentrated torques due to axle loads:

$$\Sigma\sigma_{DW} = \sigma_{DW1}\ \Sigma(C_{\beta x})$$

where

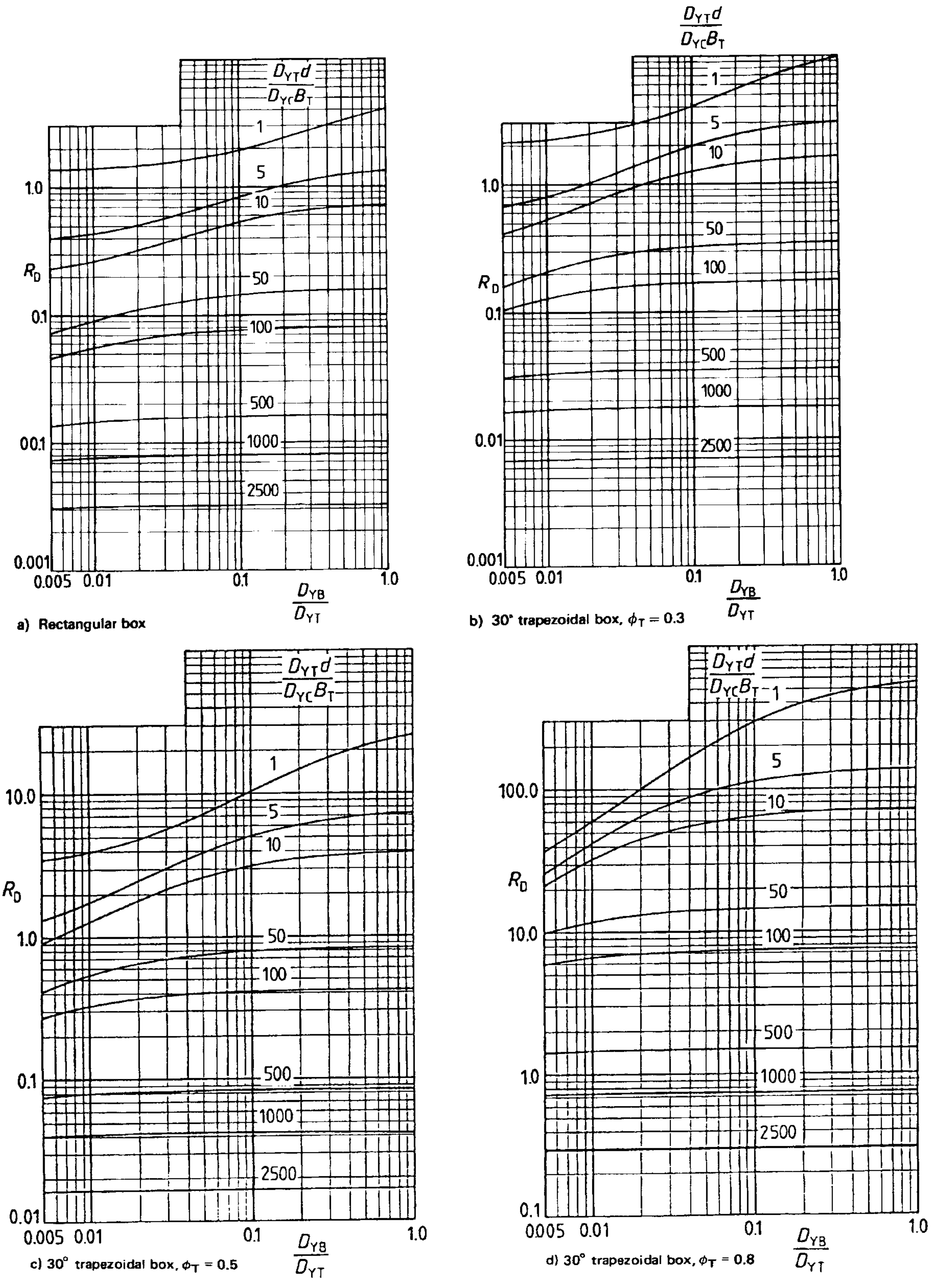
σ_{DW1} is the value of σ_{DW} obtained under b) above for a unit axle load;

$C_{\beta x}$ is given by $P_n(\cos \beta x - \sin \beta x)\ e^{-\beta x}$;

P_n is the load on an axle at distance x from the mid-point between diaphragms;

βx $= \left(\frac{Kx^4}{EI_x}\right)^{0.25}$;

K is as defined in a).



NOTE For basis of curves see **G.17**.

Figure B.2 — Distortional warping stress parameters

B.3.3 ***Distribution of distortional warping stress***

Stress due to restraint of distortional warping should be assumed to be distributed over the cross-section as shown in Figure B.3.

In Figure B.3, B_c is the width of the flange projection beyond the web, or, where there are two or more boxes in one structure, half the clear width of flange between boxes.

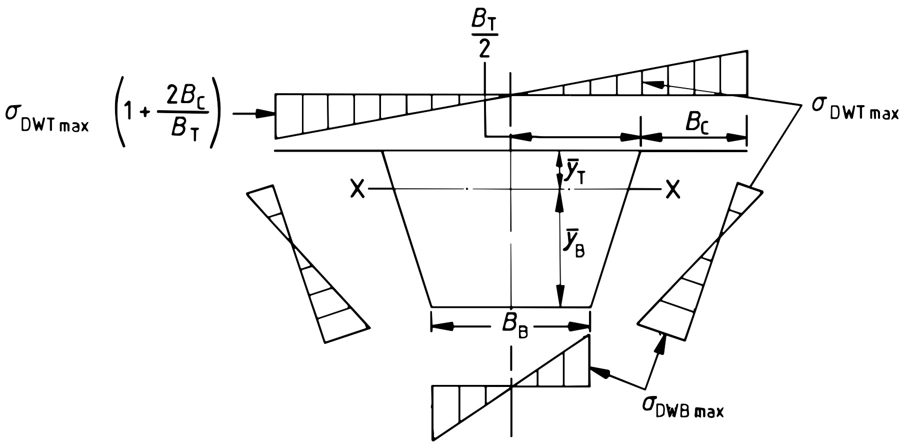


Figure B.3 — Longitudinal stresses due to distortional warping

B.3.4 ***Effective diaphragm***

B.3.4.1 *General*

To be effective for the purposes of this clause, a cross frame or a diaphragm should be such as to satisfy the conditions given in **B.3.4.2** and **B.3.4.3**, where the load effects mentioned should be considered as acting in combination with all other simultaneously acting loading effects.

B.3.4.2 *Strength*

a) A plate diaphragm should be capable of resisting a shear stress τ_D given by:

$$\tau_D = T/2BDt_d$$

where

t_d is the thickness of the diaphragm plate;

B is as defined in **9.17.2.7**, i.e. the average of the widths at the top and bottom flanges;

D is as defined in **B.2**;

T is the torque due to loads applied at the diaphragm and between the adjacent diaphragms or cross frames on either side. Any torque applied between the diaphragm and an adjacent diaphragm or cross frame may be apportioned by simple static analysis.

b) A cross frame consisting of a pair of cross braces connecting both pairs of opposite corners of the box, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, given by:

$$F_B = \frac{TL_pB_B}{4BDB_T}$$

where

T, B, D are as defined in a);

B_B, B_T are as defined in **B.2**;

L_p is as defined in **B.3.4.3**.

c) A vee-braced cross frame with the vee centred in the top or bottom flange, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, as given by:

$$F_B = \frac{TL_bB_B}{2BDB_T}$$

where

T, B, D	are as defined in a);
L_b	is as defined in **B.3.4.3**;
B_B, B_T	are as defined in **B.2**.

B.3.4.3 *Stiffness*

A cross frame or a diaphragm should have a dimensionless stiffness S not less than the value obtained from Table B.1;

where

$S = \dfrac{Gt_dL_p^2\delta_b^2K}{2A_pL_D}$ for a plated diaphragm;

$S = \dfrac{EA_b\delta_b^2K}{L_DL_p}$ for a cross-braced cross frame;

$S = \dfrac{EA_bL_p^2\delta_b^2K}{4L_DL_b^3}$ for a vee-braced cross frame, irrespective of whether the centre of the vee is at the top or bottom flange;

$S = \dfrac{K_R}{KL_D}$ for an unbraced ring cross frame with constant section framing members;

alternatively,

$S = \dfrac{P_p\delta_b^2K}{\Delta_pL_D}$ for any type of cross frame including a ring cross frame.

Δ_p	is the change in length of the diagonal L_p calculated to occur under the system of diagonal forces P_p as shown in Figure B.4 (this method of deriving stiffness may be used for any type of frame including those given above);
L_p	is the length of the diagonal, given by $\sqrt{D^2 + B^2}$;
L_b	is the length of the brace;
A_p	is the surface area of the plated diaphragm, given by BD;
A_b	is the area of the cross-section of the brace;
δ_b	is the flexibility per unit length, given by $\dfrac{4BD}{KB_BL_p}$;
B	is the average width of the box girder, given by $(B_T + B_B)/2$;
L_D, K	are as defined in **B.3.2**;
K_R	is the value of K derived by taking D_{YB}, D_{YC} and D_{YT} as the flexural rigidities of the effective framing members attached to the top and bottom flanges and webs respectively;
B_T, B_B, D	are as defined in **B.2**.

Table B.1 — Diaphragm stiffness, S

βL_D	Single torque		Uniformly distributed torque	
	for σ_{DW}	for σ_{DB}	for σ_{DW}	for σ_{DB}
3.0	0	0	0	5
2.0	0	5	0	30
1.5	2	10	10	200
1.0	20	50	100	500
0.8	50	100	200	1 000
0.5	500	1 000	200	10 000
0.3	2 000	10 000	200	20 000
NOTE For intermediate values of βL_D, values of S may be obtained by logarithmic interpolation.				

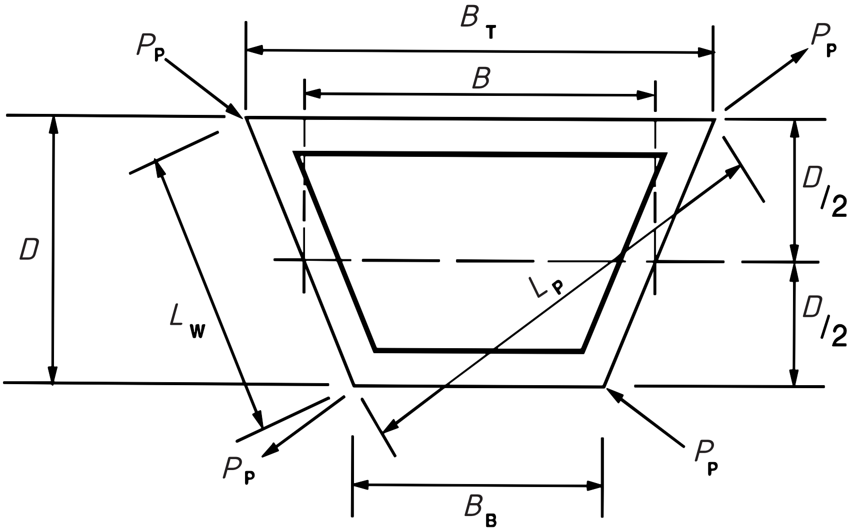


Figure B.4 — System of diagonal forces

B.4 Transverse distortional bending stresses

B.4.1 *General*

When loads are applied to a box girder other than at a cross frame or a diaphragm, the resulting transverse bending stresses in the walls of a box should be calculated on the basis of linear elastic theory. The simplified procedure given in **B.4.2** may be used provided that the cross frames or the diaphragms are in accordance with **B.3.4**.

B.4.2 *Corner stresses*

The transverse bending stress σ_{DB} may be calculated as follows.

a) At a junction between a transverse stiffener on a web and a transverse stiffener or cross beam on a flange, under a uniformly distributed torque, T_{UDL}:

$$\sigma_{DB} = \frac{T_{UDL} F_D}{B_T Z} \text{ when } \beta L_D > 2.65$$

$$\sigma_{DB} = \frac{T_{UDL} F_D}{2 B_T Z} \left(\frac{\beta L_D}{2.2}\right)^{3.7} \text{ when } \beta L_D \leq 2.65$$

where

Z is the elastic section modulus, per unit of span length, of the flange or web inclusive of transverse stiffeners or cross beams;

σ_{DB} is the maximum distortional bending stress in the part to which Z refers;

F_D is given by $\frac{B_T}{2}\left(\frac{B_B}{B_T + B_B} - V_D\right)$ at a top flange junction; or

$B_B V_D/2$ at a bottom flange junction;

βL_D is as defined in **B.3.2**;

V_D is obtained from Figure B.5;
or from

$$V_D = \frac{\left\{\frac{D_{YT}}{D_{YC}}\frac{d}{B_T}\left(2 + \frac{B_B}{B_T}\right) + 1\right\}}{\left(\frac{B_T}{B_B} + 1\right)\left(1 + 2\left[\frac{D_{YT}}{D_{YC}}\frac{d}{B_T}\left\{1 + \frac{B_B}{B_T} + \left(\frac{B_B}{B_T}\right)^2\right\}\right] + \frac{D_{YT}}{D_{YB}}\left(\frac{B_B}{B_T}\right)^3\right)}$$

φ_T, D_{YB}, D_{YC}, D_{YT}, B_B, B_T, d are as defined in **B.3.2**.

b) At a junction between a transverse stiffener on a web and a transverse stiffener or cross beam on a flange, due to a concentrated torque T where an axle or concentrated load is applied mid-way between diaphragms:

$$\sigma_{DB} = \frac{TF_D}{2B_T L_D Z}\beta L_D \text{ when } \beta L_D > 2$$

$$\sigma_{DB} = \frac{TF_D}{15.5 B_T L_D Z}(\beta L_D)^{3.9} \text{ when } \beta L_D \leq 2$$

where all symbols are as defined in a).

c) Under UDL and concentrated loading, the effects of the uniformly distributed and concentrated loads should be separately calculated as described in a) and b), and the sum of the resulting stresses used.

d) Under a series of concentrated torques due to axle loads:

$$\Sigma\sigma_{DB} = \sigma_{DB1}\Sigma(P_{\beta x})$$

where

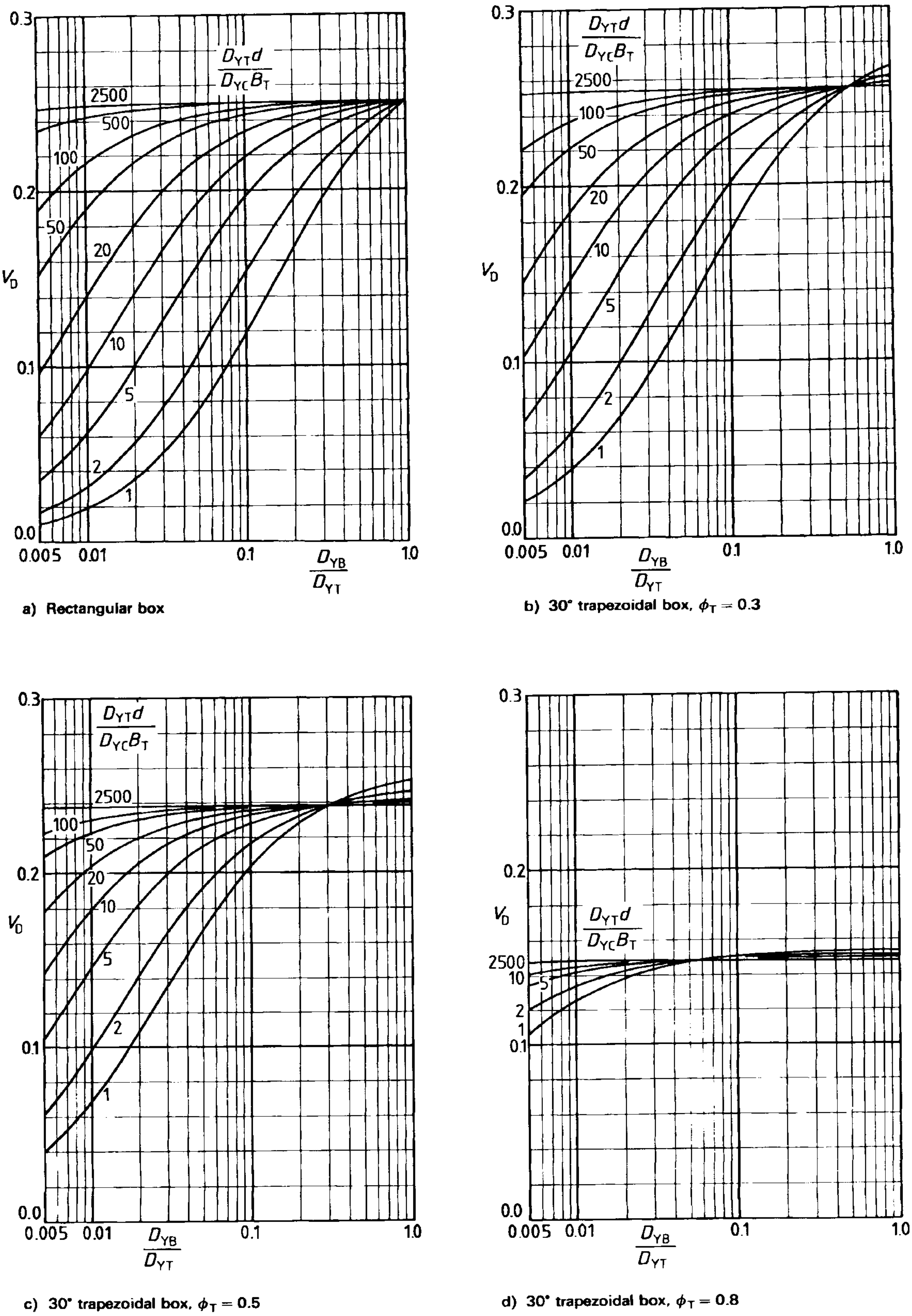
σ_{DB1} is the value of σ_{DB} obtained from b) for a unit axle load;

$P_{\beta x}$ is given by $P_n(\cos \beta x + \sin \beta x)\, e^{-\beta x}$

P_n, βx are as defined in **B.3.2**d).

B.4.3 ***Distribution of distortional bending stress***

Transverse distortional bending moments should be assumed to be distributed over the cross-section, as shown in Figure B.6, and the resulting stresses calculated using the appropriate value of Z at each section.



NOTE For basis of curves, see **G.17**.

Figure B.5 — Distortional bending stress parameters

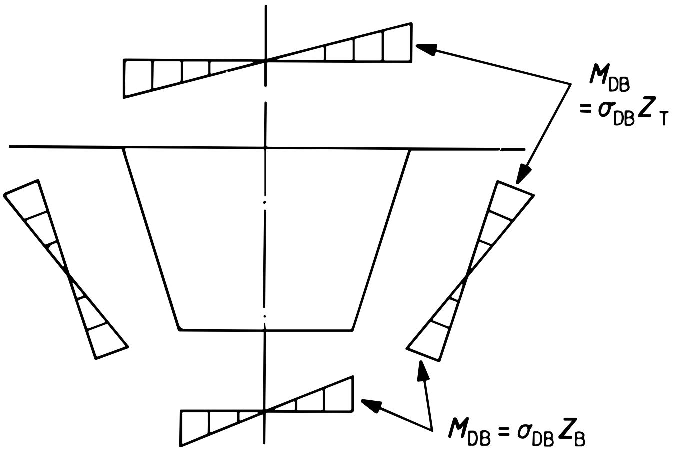


Figure B.6 — Transverse distortional moments

Annex C (informative)

Slenderness limitations for open stiffeners

As an alternative to the provisions of **9.3.4.1**, the limiting proportions of open stiffeners may be determined as follows.

a) For all open stiffeners:

1) d_s/t_s should not exceed $1.7\sqrt{\dfrac{E}{\sigma_{ys} + \sigma_a}}$;

2) additionally, if $m\left\{F_1 + F_2\left(\dfrac{d_s}{t_s}\right)^2\right\}$ is less than $2.25\sigma_{ys}/E$

then d_s/t_s should not exceed

$$\left(\frac{4F_3\sigma_{ys}}{\left[\left(2.25\sigma_{ys}/E\right) - m\left\{F_1 + F_2(d_s/t_s)^2\right\}\right]\left[3\sigma_{ys} + \sigma_a\right]}\right)^{0.5}$$

b) In addition, for angle and tee stiffeners:

$\dfrac{b_{so}}{t_{so}}\sqrt{\dfrac{\sigma_{ys}}{355}}$ should not exceed 10

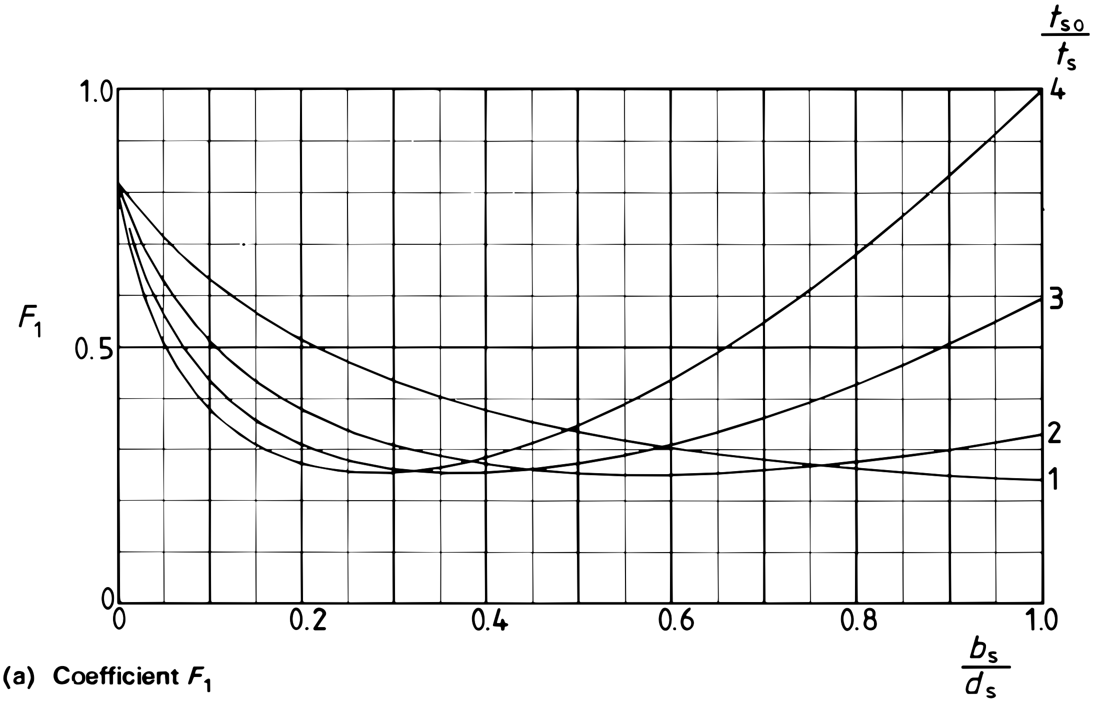
where

m	is given by $\dfrac{\sqrt{a_1^2 + 40a_2 - 4a_3} - a_1}{50}$ but is not less than $(t_s/\ell_s)^2$;
a_1	is given by btt_s^2/I_{ps};
a_2	is given by $\dfrac{b_t}{A_s}\left(\dfrac{t}{b}\right)^2 \dfrac{t_s^4}{d_s^2 r_{ys}^2}$;
a_3	is given by $\dfrac{bt}{A_s}\left(\dfrac{J_s}{I_{ps}}\right)\dfrac{t_s^4}{d_s^2 r_{ys}^2}$;
A_s	is the cross-sectional area of the stiffener;
I_{ps}	is the polar moment of inertia of the stiffener about its point of attachment to the plate;
r_{ys}	is the radius of gyration of the stiffener about its centroidal axis normal to the plate;
J_s	is the St Venant torsional constant of the stiffener;
F_1, F_2, F_3	are coefficients given in Figure C.1 for the ratios t_{so}/t_s and b_s/d_s;

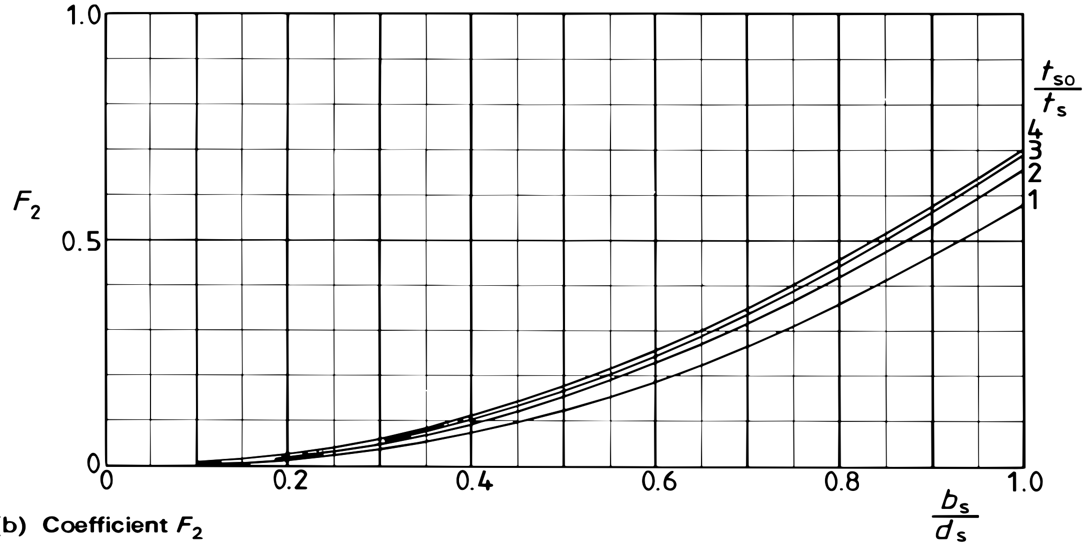
σ_a, b, t, b_s, t_s, b_{so}, t_{so}, ℓ_s, d_s are as defined in **9.3.4** and as shown in Figure 1;

σ_{ys} is the nominal yield stress value as defined in **9.3.1**.

NOTE For flats, d_s should be taken as h_s, as shown in Figure 1.



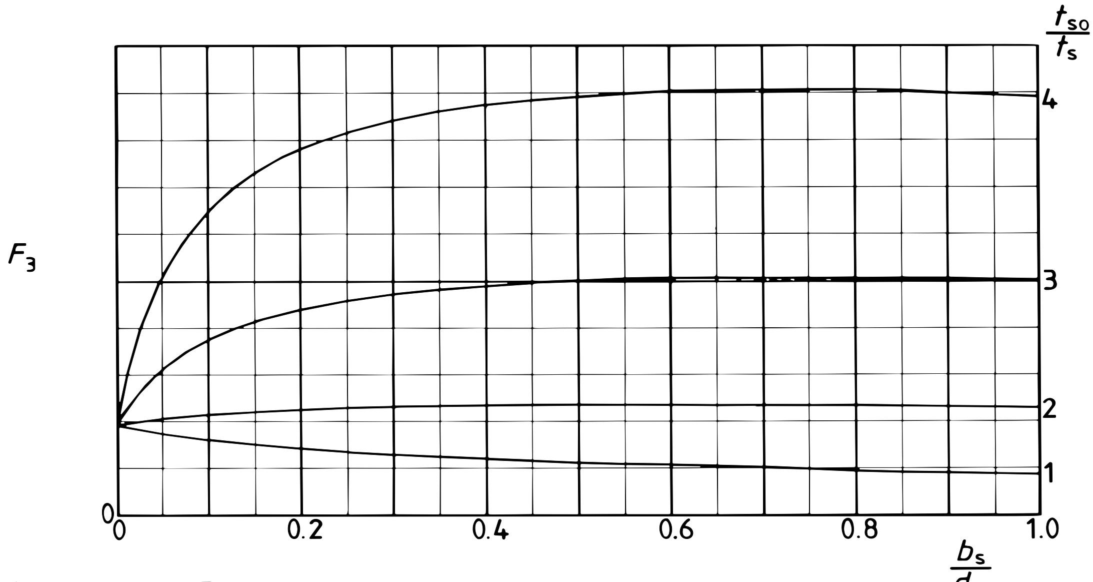
(a) Coefficient F_1



(b) Coefficient F_2

NOTE For basis of curves, see **G.18**.

Figure C.1 — Coefficients for torsional buckling



(c) Coefficient F_3

NOTE For basis of curves, see **G.18**.

Figure C.1 — Coefficients for torsional buckling (*continued*)

Annex D (normative)
Patch loading on webs

D.1 Beams without longitudinal stiffeners on web

The limiting value of patch load P on each web in its plane should be taken as the lesser of:

a) web buckling criterion:

$$\frac{0.5 t_w{}^2 \sqrt{E \sigma_{yw} (t_f/t_w)}}{\gamma_m \gamma_{f3}} \left\{ 1 + \frac{3w}{d} (t_w/t_f)^{1.5} \right\} \sqrt{1 - \left(\frac{\gamma_m \gamma_{f3} \sigma_f}{\sigma_{yw}} \right)^2}$$

b) web yielding criterion:

$$\frac{\left(2 t_f \sqrt{\sigma_{yf} \sigma_{yw} B_f t_w} + \sigma_{yw} t_w w \right)}{\gamma_m \gamma_{f3}} \sqrt{1 - \left(\frac{\gamma_m \gamma_{f3} \sigma_f}{\sigma_{yw}} \right)^2}$$

where

t_f	is the flange plate thickness;
t_w	is the web plate thickness;
w	is the width of the patch load (see **9.5.6** and Figure 6) but to be taken as not greater than $0.2d$;
B_f	is the width of the flange plate;
d	is the depth of the web in its plane;
σ_{yf}, σ_{yw}	are the nominal yield stresses of the material of the flange and web, respectively, as defined in **6.2**;
σ_f	is the longitudinal stress in the flange due to bending moment and/or axial force on the beam;
γ_m	is taken as 1.05 for the ultimate limit state.

D.2 Beams with longitudinal stiffeners on web

The limiting value of patch load P on each web in its plane should be taken as the lesser of the limiting values given in **D.1**a) multiplied by a factor K, or **D.1**b)

where

K	is given by $1.28 - 0.7(b/d)$ but is not less than 1.0 nor greater than 1.21;
b	is the clear distance between the flange and the longitudinal web stiffener nearest to the flange which is in accordance with **9.11.5**.

Annex E (informative)
Transverse moments in compression flanges: U-frame restraints

The maximum value M_y of the moment in the plane of the flange needed in **9.12.3.3**b) or **9.12.4.2**c) to be applied to the compression flange of the beam may be taken as follows:

$$M_y = \frac{5EI_c\theta d_2}{L\ell_e\left(1-\sigma_{fc}/\sigma'_{ci}\right)}\left\{1+\frac{\left(L/\ell_e\right)-1.25}{2.8+3.5\left(\sigma_{fc}/\sigma'_{ci}\right)^2}\right\}$$

where

I_c is as defined in **9.6.4.1.1.2**;

d_2 is as defined in **9.6.4.1.3**;

θ is as defined in **9.12.3.3**a) or **9.12.4.2**b) as appropriate;

ℓ_e is as given by **9.6.4.1.3** or **9.6.4.2.2** as appropriate;

σ_{fc} is the maximum compressive stress in the flange;

σ'_{ci} is taken as follows:

a) if ℓ_w is less than three times the spacing of U-frames, then $\sigma'_{ci} = \sigma_{ci}$, as defined in **9.12.2**;

b) if ℓ_w is more than four times the spacing of U-frames, or if ℓ_e has been calculated in accordance with **9.6.4.1.1.2**, then $\sigma'_{ci} = 1.25\sigma_{ci}$; or

c) for intermediate values of ℓ_e, σ'_{ci} is obtained by linear interpolation;

ℓ_w is as defined in **9.8**;

L is as defined in **9.8**.

For uniformly distributed loading, HA loading and RL loading, placed over the whole span, the maximum moment M_y derived above should be assumed to act anywhere within a horizontal distance ℓ_e from each bearing support of the beam.

Elsewhere the bending moment may be assumed to be $M_y/2$.

For all other loading cases it should be assumed that M_y acts anywhere within the span.

The moment M_y thus obtained should be combined with other effects, giving M'_y, for checking compliance with the following:

a) for flanges in beams without longitudinal stiffeners (see **9.9**):

$$\frac{M}{M_D}+\frac{M'_y}{M'_{Dy}} \leq 1$$

where

M, M_D are the applied moment and the moment capacity, respectively, for bending parallel to the web of the main beam;

M'_{Dy} is the transverse moment capacity of the compression flange with λ_{LT} taken as zero;

b) for flanges in longitudinally stiffened beams (see **9.10**):

$$\frac{\sigma_f Z_{xc}}{M_R}+\frac{\sigma_b Z_{xc}}{M_{ult}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_f is the longitudinal stress in part of the flange plate under consideration due to the applied bending moment in the plane of the web, based on the plastic section modulus when the design is for a compact section (see **9.3.7**);

σ_b is the longitudinal stress in that part of the flange plate under consideration when M'_y is applied to the compression flange alone;

M_R, M_{ult} are as defined in **9.8**;

Z_{xc} is as defined in **9.7.1**;

c) for chords in trusses (see **10.6.2**).

NOTE Annex E is applicable to simply supported spans having torsional end restraints which are infinitely rigid, and will generally be conservative where the torsional restraint is not fully rigid.

Annex F (informative)

Buckling coefficients for transverse members in compression flanges

For cases not in accordance with the limitations given in a), b) or c) of **9.15.3.2**, values of the buckling coefficient K may be determined from:

$$K = 24\frac{\{a_1 + (kf_c/f)\Omega\lambda_B a_2\}\{a_1^2 + (2k/3)\lambda_B\xi a_2^2\}}{\{a_1^2 + (2k/3)(f_c/f)\lambda_B(3\Omega - 2)a_2^2\}^2}$$

where

a_1 is given by $3k\Omega(f_c/f)\lambda_B^2 + 1$;

a_2 is given by $16\left(\frac{\Omega f_c I_{be}}{f I_{bc}}\right)\lambda_B^4 + 8\left\{(1+k)\frac{\Omega f_c}{f}\right\}\lambda_B^3 + 3\lambda_B$;

k = 2 if there are cantilever brackets on both sides of the segment, or = 1 if on one side only;

f_c is the longitudinal force per unit width in the cantilever portion of the flange;

f is the longitudinal force per unit width in the portion of the flange between main beam webs;

λ_B is given by B_c/B;

B, B_c are shown in Figure 30;

Ω is given by $1 + \frac{E_E A_E}{B_c E_{fc} A_{fc}}$;

ξ is given by $\left(1 + \frac{3E_E I_E}{B_c E_{fc} I_{fc}}\right)\frac{E_{fc} I_{fc}}{E_f I_f}$;

E_f is the modulus of elasticity for the portions of flange between the main beam webs;

E_{fc} is the modulus of elasticity for the cantilever portion of the flange;

E_E is the modulus of elasticity for the edge member;

A_{fc} is the area of the flange under consideration per unit width of the cantilever portions of the flange;

A_E is the area of the cross-section of the edge member;

I_{be} is the average second moment of area of the portion of transverse member between centrelines of main beam webs;

I_{bc} is the average second moment of area of the cantilever portions of the transverse member;

I_E is the second moment of area of the edge member about its centroidal axis;

I_f is the second moment of area of the flange under consideration per unit width of the portions of flange between main beam webs;

I_{fc} is the second moment of area of the flange under consideration per unit width of cantilever portions of the flange.

For a compression flange with closed longitudinal stiffeners: I_f in this annex and in **9.15.3** may be increased by multiplying by a factor:

$$1 + \frac{G_f J_f a^2}{E_f I_f B^2}$$

and I_{fc} in this annex may be increased by multiplying by a factor:

$$1 + 0.3\frac{G_{fc} J_{fc}}{E_{fc} I_{fc}}\left(\frac{a^2}{B_c^2}\right)$$

where

a is as defined in **9.15.3.2**;

J_f is the torsional constant of longitudinal stiffeners per unit width of the flange between main beam webs;

J_{fc} is the torsional constant of longitudinal stiffeners per unit width of the cantilever overhang;

G_f, G_{fc} are the shear moduli of the longitudinal stiffeners between main beam webs and in the cantilever overhang respectively.

Annex G (informative)
Equations used for production of curves in figures

G.1 General

As an alternative to obtaining values from the graphs given in the figures of the main document, the following equations may be used for calculation of the required values; these equations should only be used within the bounds of the figures themselves.

G.2 Figure 2 — Limiting slenderness for flat stiffeners

$$\frac{1}{q^2} = \frac{1}{10p^2} + \frac{5.21r^2p^3}{S} + 0.625r, \text{ when } q < 31.0$$

$$p = 10 \text{ when } q \geq 31.0$$

where

p is taken as $\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$;

q is taken as $\frac{b}{t}\sqrt{\frac{\sigma_y}{355}}$;

r is taken as $0.00474 - \frac{0.468}{p^2}$;

S is taken as $\frac{bt}{t_s^2}\sqrt{\frac{\sigma_{ys}}{355}}$;

σ_y, σ_{ys} are as defined in **9.3.1**;

h_s, t_s, b, t are as defined in **9.3.4.1.2**.

G.3 Figure 3 — Limiting slenderness for angle stiffeners

$$\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 6.2 + \frac{31.6}{\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} - 10}$$

when

$$\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} < 50$$

where

σ_{ys} is as defined in **9.3.1**;

h_s, t_s, b_s, ℓ_s are as defined in **9.3.4.1.4**.

G.4 Figure 4 — Limiting slenderness for tee stiffeners

a) *Figure 4a) related to* ℓ_s/b_s

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 80 - 5\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} \text{ when } \frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} \leqslant 12$$

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 5 + \frac{30}{\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} - 10} \text{ when } 12 < \frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} < 25$$

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 7 \text{ when } \frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} \geqslant 25$$

b) *Figure 4b) related to b/t*

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 7 + 4\left(\frac{t}{t_s}\right)^{0.8}\left(32 - \frac{b}{t}\sqrt{\frac{\sigma_y}{355}}\right)^{0.6} \text{ when } \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} < 32, \text{ and } \frac{d_s}{b_s} \leq 4$$

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 7 \text{ when } \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} \geq 32$$

where

σ_y, σ_{ys} are as defined in **9.3.1**;

d_s, t_s, b_s, ℓ_s, b, t are as defined in **9.3.4.1.5**.

G.5 Figure 5 — Coefficient K_c for plate panels under direct compression

a) *Curve 1*

$K_c = 1$ when $\lambda \leq 24$;

$K_c = \left(\frac{24}{\lambda}\right)^{0.50}$ when $24 < \lambda \leq 43$;

$K_c = \left(\frac{28}{\lambda}\right)^{0.68}$ when $43 < \lambda \leq 59$;

$K_c = \left(\frac{30}{\lambda}\right)^{0.75}$ when $59 < \lambda \leq 90$;

$K_c = \left(\frac{36}{\lambda}\right)^{0.90}$ when $90 < \lambda \leq 130$;

$K_c = 0.38 - \frac{\lambda}{2\,000}$ when $130 < \lambda \leq 200$;

$K_c = 0.33 - \frac{\lambda}{4\,000}$ when $200 < \lambda \leq 300$.

b) *Curve 2*

$K_c = 1$ when $\lambda \leq 24$;

$K_c = \left(\frac{24}{\lambda}\right)^{0.75}$ when $24 < \lambda \leq 47$;

$K_c = \left(\frac{26}{\lambda}\right)^{0.85}$ when $47 < \lambda \leq 130$;

$K_c = 0.274 - \frac{\lambda}{7\,000}$ when $130 < \lambda \leq 300$;

c) *Curve 3*

$K_c = 1$ when $\lambda \leq 4.33$

$$K_c = 0.5\left[\left\{1 + (1 + \eta)\frac{475}{\lambda^2}\right\} - \sqrt{\left\{1 + (1 + \eta)\frac{475}{\lambda^2}\right\}^2 - \frac{1\,900}{\lambda^2}}\right]$$

when $4.33 < \lambda \leq 300$

where

η is given by $0.015\,6(\lambda - 4.33)$;

λ is as defined in **9.4.2.4** and in Figure 5.

G.6 Figure 7 — Influence on effective length of compression flange restraint

a) *Figure 7a) Effect of rotational end restraint*

$$k_1 = 0.5 + \frac{0.5}{1 + 0.425\left(\frac{k_o L}{E I_c}\right)}$$

where

- L is the span of the beam or truss, or the length between the ends of a compression member effectively held in position;
- k_o is as defined in Figure 7a);
- I_c is as defined in **9.6.2** or **9.6.4.1.1.2**, as appropriate.

b) *Figure 7b) Effect of bending restraint*

$$\frac{1}{k_3} = \sqrt{1 + \left(\frac{\ell_o}{L}\right)^2\left(\frac{1}{k_1{}^2} - 1\right)}$$

where

- k_1 is as derived in a) above;
- ℓ_o is the value of ℓ_e obtained from **9.6.4.1.1.2** with $k_3 = 1.0$;
- L is as defined in a) above.

G.7 Figure 8 — Effective length of beams with discrete torsional restraints

$\frac{\ell_e}{L} = \sqrt{\frac{1}{c}}$ but, for central restraints, not less than $0.85 - 0.3v$.

For beams with central restraint

$$c = \frac{kL}{\pi}\sqrt{\frac{1 + (kL)^2\alpha}{1 + \pi^2\alpha}}$$

$$\theta_R = \frac{2v^4L^3}{\pi^4 E I_c d_f{}^2\left\{\frac{-2c^2\cot(kL/2)}{kL}\right\}}$$

where $\alpha = \frac{v^4}{\pi^2(1 - v^4)}$

NOTE Solve by trials of values of kL.

For beams with n equally spaced restraints

$$c = \sqrt{\left(1 + \frac{2v^4L^3}{\pi^4 EI_c d_f^2 \theta_R}\right)}$$

G.8 Figures 11a) and 11b) — Limiting moment of resistance M_R

$$\frac{M_R}{M_{ult.}} = 0.5\left[\left\{1 + (1+\eta)\frac{5\,700}{\beta^2}\right\} - \sqrt{\left\{1 + (1+\eta)\frac{5\,700}{\beta^2}\right\}^2 - \frac{22\,800}{\beta^2}}\right] \text{when } \beta\ell_w/\ell_e > 30$$

$$\frac{M_R}{M_{ult.}} = 1 \text{ when } \beta\ell_w/\ell_e \leq 30$$

where

η is given by:

$0.008\left(\frac{\beta\ell_w}{\ell_e} - 30\right)$ for Figure 11a) and

$0.003\,5\left(\frac{\beta\ell_w}{\ell_e} - 30\right)$ for Figure 11b);

β is given by $\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult.}}{M_{pe}}\right)}$;

$M_{ult.}$, λ_{LT}, ℓ_w, σ_y are as defined in **9.8**;
M_{pe} is as defined in **9.7.1**;
ℓ_e is as defined in **9.6.4**.

G.9 Figures 12 to 18 — Limiting shear strength τ_ℓ

The following iterative procedure is needed.

a) Calculate:

$$\beta = \frac{\lambda}{\sqrt{5.34 + (4/\varphi^2)}} \text{ when } \varphi \geq 1;$$

$$\beta = \frac{\lambda}{\sqrt{(5.34/\varphi^2) + 4}} \text{ when } \varphi < 1.$$

b) Calculate:

$\frac{\tau_c}{\tau_y} = 1$ when $\beta \leq 24.55$;

$\frac{\tau_c}{\tau_y} = 1.54 - 0.022\beta$ when $24.55 < \beta < 33.62$;

$\frac{\tau_c}{\tau_y} = \frac{904}{\beta^2}$ when $33.62 \leq \beta$.

c) Assume a value of θ such that:

$$0.33 \cot^{-1}\varphi \leq \theta \leq 1.33 \cot^{-1}\varphi \text{ and also } \theta \leq \frac{\pi}{4}$$

d) Calculate:

$$\frac{\sigma_t}{\tau_y} = \sqrt{\left\{3 + (2.25 \sin^2 2\theta - 3)\left(\frac{\tau_c}{\tau_y}\right)^2\right\}} - 1.5\frac{\tau_c}{\tau_y}\sin 2\theta.$$

e) Calculate:

$$\frac{\tau_u}{\tau_y} = f\left\{\frac{\tau_c}{\tau_y} + 5.264\sqrt{m_{fw}\frac{\sigma_t}{\tau_y}}\sin\theta + \frac{\sigma_t}{\tau_y}(\cot\theta - \varphi)\sin^2\theta\right\}$$

when $m_{fw} \leq \frac{\varphi^2}{4\sqrt{3}}\frac{\sigma_t}{\tau_y}\sin^2\theta$;

$$\frac{\tau_u}{\tau_y} = f\left(\frac{4\sqrt{3}m_{fw}}{\varphi} + \frac{\sigma_t}{2\tau_y}\sin 2\theta + \frac{\tau_c}{\tau_y}\right)$$

when $m_{fw} > \frac{\varphi^2}{4\sqrt{3}}\frac{\sigma_t}{\tau_y}\sin^2\theta$;

where

$f = 1$ when $\lambda \leq 56$;

$f = \frac{1.15}{1.15 + 0.002(\lambda - 56)}$ when $56 < \lambda < 156$;

$f = \frac{1.15}{1.35}$ when $156 \leq \lambda$;

φ, λ, d_{we}, t_w, τ_y, m_{fw} are as defined in **9.9.2.2**.

f) Repeat c) to e) with different values of θ until the maximum value $(\tau_u/\tau_y)_{max.}$ is obtained, where

τ_ℓ/τ_y is equal to the lesser of $(\tau_u/\tau_y)_{max.}$ and 1.0.

G.10 Figure 19 — Parameters for the design of longitudinal flange stiffeners

$$k_\ell = 0.5\left[\left\{1 + (1 + \eta)\frac{5\ 700}{\lambda^2}\right\} - \sqrt{\left\{1 + (1 + \eta)\frac{5\ 700}{\lambda^2}\right\}^2 - \frac{22\ 800}{\lambda^2}}\right]$$

$k_\ell = \frac{1}{1 + \eta}$ when $\lambda = 0$;

$$k_s = 0.4\ k'_\ell\left(\eta' + 1.754\ 6 \times 10^{-4}\lambda^2\right);$$

where

k'_ℓ $= k_\ell$ with $\eta = \eta'$;

η' $= 0.008\ 3(\lambda - 15)$ when $\lambda > 15$;

η' $= 0$ when $\lambda \leq 15$;

η, λ are as defined in **9.10.2.3**.

G.11 Figure 22 — Minimum value of m_{fw} for outer panel restraint

$$m_{fw} = 0.0187\ 5\ \varphi^{1.6}\left\{\frac{\lambda - 66 - (28/\varphi^2)}{134 - (28/\varphi^2)}\right\}^{0.3}$$

where

φ, λ, m_{fw} are as defined in **9.11.4.2.2**.

G.12 Figure 23 — Buckling coefficients K_1, K_2, K_q and K_b

a) *Figure 23a) K_1 and K_2.*

The equations to be used are as given for K_c in **G.5**a) to c), but where λ is as defined in **9.11.4.3.2** and **9.11.4.3.5** and in Figure 23a).

b) *Figure 23b) K_q*

1) Restrained panels:

for $\varphi \leq 0.5$: $K_q = 1$ when $\lambda \leq 80$

$K_q = \left(\frac{80}{\lambda}\right)^{0.15}$ when $80 < \lambda \leq 300$;

for $\varphi = 1.0$: $K_q = 1$ when $\lambda \leq 48$

$K_q = \left(\frac{48}{\lambda}\right)^{0.11}$ when $48 < \lambda \leq 300$;

for $\varphi > 2.0$: $K_q = 1$ when $\lambda \leq 40$

$K_q = \left(\frac{40}{\lambda}\right)^{0.15}$ when $40 < \lambda \leq 300$;

NOTE For intermediate values of φ, K_q may be obtained by linear interpolation between two adjacent values of φ. For this purpose values may be obtained beyond $K_q = 1.0$ from the equation given in **9.11.4.3.3**.

2) Unrestrained panels:

for $\varphi \leq 0.5$:

$K_q = 1$ when $\lambda \leq 80$

$K_q = 1 - 0.385\left(\frac{\lambda - 80}{120}\right)^{0.743}$ when $80 < \lambda \leq 224$

$K_q = 1 - 0.660\left(\frac{\lambda - 80}{320}\right)^{0.505}$ when $224 < \lambda \leq 300$;

for $\varphi = 1.0$:

$K_q = 1$ when $\lambda \leq 48$

$K_q = \left(\frac{48}{\lambda}\right)^{0.5}$ when $48 < \lambda \leq 300$;

for $\varphi = 2.0$:

$K_q = 1$ when $\lambda \leq 40$

$K_q = 1 - 0.385\left(\frac{\lambda - 40}{60}\right)^{0.743}$ when $40 < \lambda \leq 112$

$K_q = 1 - 0.660\left(\frac{\lambda - 40}{160}\right)^{0.505}$ when $112 < \lambda \leq 200$

$K_q = 0.34 - 0.07\left(\frac{\lambda - 200}{100}\right)^{0.8}$ when $200 < \lambda \leq 300$;

for $\varphi \geq 3.0$:

$K_q = 1$ when $\lambda \leq 38$

$K_q = 1 - 0.555\left(\frac{\lambda - 38}{82}\right)^{0.823}$ when $38 < \lambda \leq 125$

$K_q = 0.445 - 0.205\left(\frac{\lambda - 120}{80}\right)^{0.725}$ when $125 < \lambda \leq 221$

$K_q = 0.24 - 0.075\left(\frac{\lambda - 200}{100}\right)^{0.44}$ when $221 < \lambda \leq 300$.

NOTE For interpolation see the note in 1).

where

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

$$\varphi = \frac{a}{b}$$

σ_{yw}, t_w are as defined in **9.11.4.3.2**;

a, b are as defined in Figure 23.

c) *Figure 23c)* K_b

1) Restrained panels:

$K_b = 1.3 - 0.001\,9\lambda$ when $\lambda \leq 300$;

2) Unrestrained panels:

$K_b = 1.3 - 0.002\,7\lambda$ when $\lambda \leq 300$;

where

λ is given by $\frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$

σ_{yw}, t_w are as defined in **9.11.4.3.2**.

G.13 Figure 24 — Parameters for the design of web stiffeners

$$\frac{\sigma_{\ell s}}{\sigma_{ys}} = 0.5\left[\left\{1 + (1+\eta)\frac{5\ 700}{\lambda^2}\right\} - \sqrt{\left\{1 + (1+\eta)\frac{5\ 700}{\lambda^2}\right\}^2 - \frac{22\ 800}{\lambda^2}}\right]$$

$$k_s = 0.4\frac{\sigma_{\ell s}}{\sigma_{ys}}\left(\eta + 1.754\ 6 \times 10^{-4}\lambda^2\right)$$

where

$\eta = 0.008\,3(\lambda - 15)$ when $\lambda > 15$;

$\eta = 0$ when $\lambda \leq 15$;

λ is as defined in **9.11.5.2**, **9.13.3.3**, **9.13.5.3**, **9.17.6.3.4**, **9.17.6.7** and **9.17.7.3.2** as appropriate, and in Figure 24.

G.14 Figure 25 — Factors for determining restraint forces in continuous beams

$$C = \frac{1}{1 - 1.12\frac{d_f}{L}\sqrt{\frac{I_c}{J}}\tanh\left(0.89\frac{L}{d_f}\sqrt{\frac{J}{I_c}}\right)}$$

$$K = \frac{0.4\ CL^2J}{d_f^2 I_c}$$

G.15 Figure 30 — Buckling coefficient *K* for transverse members

a) *Figure 30a) — Cantilevers on one side only*

$$K = \frac{24\left[(3b^2 + 1) + b^2\{16b^3/a + 16b^2 + 3\}\right]}{\left[(3b^2 + 1)^2 + (2b^3/3)\{16b^3/a + 16b^2 + 3\}^2\right]}$$

b) *Figure 30b) — Cantilevers on both sides*

$$K = \frac{24\left[(6b^2 + 1) + 2b^2\{16b^3/a + 24b^2 + 3\}\right]}{\left[(6b^2 + 1)^2 + (4b^3/3)\{16b^3/a + 24b^2 + 3\}^2\right]}$$

where

a	is given by $\frac{I_{bc}}{I_{be}}$;
b	is given by $\frac{B_c}{B}$;
I_{be}, I_{bc}, B, B_c	are as defined in **9.15.3.2**.

NOTE The equations given in a) and b) are only valid provided that the provisions of **9.15.3.2**a), b) and c) are satisified. For other cases, the equations given in annex F should be used.

G.16 Figure 37 — Ultimate compressive stress σ_c

$$\frac{\sigma_c}{\sigma_y} = 0.5\left[\left\{1 + (1 + \eta)\frac{5\,700}{\lambda^2}\right\} - \sqrt{\left\{1 + (1 + \eta)\frac{5\,700}{\lambda^2}\right\}^2 - \frac{22\,800}{\lambda^2}}\right]$$

where

η	$= 0$ when $\lambda \leq 15$
	$= \alpha(\lambda - 15)$ when $\lambda > 15$
α	= 0.002 5 for curve A
	= 0.004 5 for curve B
	= 0.006 2 for curve C
	= 0.008 3 for curve D
λ	$= \frac{\ell_e}{r}\sqrt{\frac{\sigma_y}{355}}$
ℓ_e, r, σ_y	are as defined in **10.6.1.1**.

NOTE Guidance on the appropriate use of curves A, B, C or D is given in note 1 to Figure 37.

G.17 Figure B.2 — Distortional warping stress parameters. Figure B.5 — Distortional bending stress parameters

a) *Figures B.2a) and B.5a) — Rectangular box*

$$R_D = \frac{2.0}{\left(\frac{D_{YT}d}{D_{YC}B_T} + 0.5\right) - V_D\left[3\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{3\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{2 + 12\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 2\left(\frac{D_{YT}}{D_{YB}}\right)}$$

b) *Figures B.2b) and B.5b) — 30° trapezoidal box,* $\varphi_T = 0.3$

$$R_D = \frac{2.43}{0.412\left[2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right] - V_D\left[2.7\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{2.7\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{2.43 + 10.62\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 0.834\left(\frac{D_{YT}}{D_{YB}}\right)}$$

c) *Figures B.2c) and B.5c) — 30° trapezoidal box,* $\varphi_T = 0.5$

$$R_D = \frac{3.0}{0.333\left[2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right] - V_D\left[2.5\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{2.5\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{3 + 10.5\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 0.375\left(\frac{D_{YT}}{D_{YB}}\right)}$$

d) *Figures B.2d) and B.5d) — 30° trapezoidal box,* $\varphi_T = 0.8$

$$R_D = \frac{6.0}{0.167\left[2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right] - V_D\left[2.2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{2.2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{6 + 14.88\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 0.048\left(\frac{D_{YT}}{D_{YB}}\right)}$$

where

φ_T $= \frac{d}{B_T}$;

B_T is as defined in **B.2**b);

d, D_{YT}, D_{YB}, D_{YC} are as defined in **B.3.2**.

G.18 Figure C.1 — Coefficients for torsional buckling

$$F_1 = \frac{\pi^2}{12}\left(\frac{K_1^3K_2^3 + 4}{4 + K_1K_2^3 + 12K_1K_2}\right)$$

$$F_2 = \pi^2\left(\frac{K_1K_2^3}{4 + K_1K_2^3 + 12K_1K_2}\right)$$

$$F_3 = 1.54\left(\frac{1 + K_1^3K_2}{4 + K_1K_2^3 + 12K_1K_2}\right)$$

where

K_1 $= \frac{t_{so}}{t_s}$;

K_2 $= \frac{b_s}{d_s}$;

t_s, t_{so}, b_s, d_s are as defined in **9.3.4** and shown in Figure 1.

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