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BRITISH STANDARD

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1979

Reprinted, incorporating Amendment No. 1

Steel, concrete and composite bridges —

Part 5: Code of practice for design of composite bridges

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

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3998	May 1982	Indicated by a sideline in the margin

Contents

Foreword

BS 5400 is a document combining codes of practice to cover the design and construction of steel, concrete and composite bridges and specifications for loads, materials and workmanship.

It comprises the following Parts:

— *Part 1: General statement;*

— *Part 2: Specification for loads;*

— *Part 3: Code of practice for design of steel bridges*[1]*;*

— *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: Code of practice for bearings*[1]*;*

— *Part 10: Code of practice for fatigue.*

In the drafting of BS 5400 important changes have been made in respect of loading and environmental assumptions, design philosophy, load factors, service stresses and structural analysis. Furthermore, recourse has been made to recent theoretical and experimental research and several design studies have been made on components and on complete bridges. It is to be expected that as design experience of different bridge types is accumulated, further modifications will be required.

It should be noted that this Part of BS 5400 supersedes CP 117-2.

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

Part 5 was published in 1979, the major decision on scope and approach having been taken some years previously; Part 3 was published in 1982. It s natural therefore that some differences will exist between Part 3 and Part 5.

Part 3 has been drafted on the assumption that for the design of steelwork in bridges with either steel or concrete decks the methods of global analysis and all the procedures for satisfying the limit state criteria will be as prescribed in Part 3. For beams Part 3 may be used without any modification in conjunction with those provisions of Part 5 that are applicable to the properties of the composite slab and its connection to the steel section.

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

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

[1] In course of preparation.

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

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

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

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

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 46, an inside back cover and a back cover.

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

1 Scope

This Part of this British Standard supersedes CP 117-2 and augments the provisions of BS 5400-3, BS 5400-4 and BS 5400-10 for structural steel and reinforced or prestressed concrete when components of these materials are so interconnected that they act compositely.

It gives recommendations for rolled or fabricated steel sections, cased or uncased, and for filler beam systems. Consideration is given to simply supported and continuous composite beams, composite columns and to the special problems of composite box beams. The recommendations for the concrete element cover normal and lightweight aggregate, cast in situ and precast concrete. Prestressing and the use of permanent formwork designed to act compositely with in situ concrete are also covered.

2 References

The titles of the publications referred to in this Part of BS 5400 are listed in the inside back cover.

3 Definitions and symbols

3.1 Definitions

For the purposes of this Part of this British Standard the following definitions, and those given in Part 1, apply.

3.1.1
cased composite beam

a beam composed of either rolled or built-up structural steel sections, with a concrete encasement, which acts in conjunction with a concrete slab where the two elements are interconnected so as to form a composite section

3.1.2
uncased composite beam

a beam composed of either rolled or built-up structural steel sections, without a concrete encasement, which acts in conjunction with a concrete slab where the two elements are interconnected so as to form a composite section

3.1.3
composite box beam

a steel box girder acting compositely with a concrete slab

NOTE In a closed steel box the concrete is cast on the top steel flange whereas in an open steel box the box is closed by the concrete slab.

3.1.4
composite column

a column composed either of a hollow steel section with an infill of concrete or of a steel section cased in concrete so that in either case there is interaction between steel and concrete

3.1.5
composite plate

an in situ concrete slab cast upon, and acting compositely with, a structural steel plate

3.1.6
concrete slab

the structural concrete slab that forms part of the deck of the bridge and acts compositely with the steel beams. The slab may be of precast, cast in situ or composite construction

3.1.7
composite slab

an in situ concrete slab that acts compositely with structurally participating permanent formwork

3.1.8
participating permanent formwork

formwork to in situ concrete, when the strength of the formwork is assumed to contribute to the strength of the composite slab

3.1.9
non-participating permanent formwork

permanent formwork that may or may not act compositely with the in situ concrete but where the formwork is neglected in calculating the strength of the slab

3.1.10
filler beam construction

rolled or built-up steel sections that act in conjunction with a concrete slab and which are contained within the slab

3.1.11 Interaction

3.1.11.1
complete interaction

this implies that no slip occurs between the steel and the concrete slab or encasement

3.1.11.2
partial interaction

this implies that slip occurs at the interface between steel and concrete and a discontinuity in strain occurs

3.1.12
shear connector

a mechanical device to ensure interaction between concrete and steel

3.1.13
connector modulus

the elastic shear stiffness of a shear connector

3.2 symbols

The symbols used in this Part of this standard have been derived in accordance with Appendix F of CP 110-1:1972 and are as follows.

A_b	Cross-sectional area of transverse reinforcement in the bottom of the slab
A_{bs}	Cross-sectional area of other transverse reinforcement in the bottom of the slab
A_{bv}	Cross-sectional area of additional transverse reinforcement
A_c	Cross-sectional area of concrete
A_e	Effective cross-sectional area of transverse reinforcement
A_f	Cross-sectional area of top flange of steel section
A_r	Cross-sectional area of reinforcement
A_s	Cross-sectional area of the steel section
A_{st}	Area of the encased tension flange of the structural steel member
A_t	Area of tension reinforcement, cross-sectional area of transverse reinforcement near the top of the slab
a'	Distance from the compression face to the point at which the crack width is calculated
a_{cr}	Distance from the point considered to the surface of the nearest longitudinal bar
b	Width of section or portion of flange or least lateral dimension of a column
b_e	Effective breadth of portion of flange
b_f	Breadth of flange
b_s	External dimension of the wall of the RHS
b_t	Effective breadth of the composite section at the level of the tension reinforcement
b_w	Half the distance between the centre lines of webs
C	A constant (with appropriate subscripts)
c_{min}	Minimum cover to the tension reinforcement
D	Diameter
D_e	External diameter
d_c	Depth to neutral axis in composite column
d_r	Separation of symmetrically placed reinforcing bars
d_s	Thickness of the concrete cover of encased steel section
d_w	Depth of steel web in compression zone
E_c	Static secant modulus of elasticity of concrete
E_r	Modulus of elasticity of steel reinforcement
E_s	Modulus of elasticity of structural steel
F_T	Tensile force per unit length
f_c	Concrete strength
f_{cc}	Enhanced characteristic strength of triaxially contained concrete
f_{ci}	Concrete strength at (initial) transfer
f_{cu}	Characteristic concrete cube strength
f_L	Longitudinal stress
f_{max}	Maximum longitudinal stress in concrete flange
f_{ry}	Characteristic strength of reinforcement
f_{tc}	Tensile stress in uncracked concrete flange
f'_y	Reduced nominal yield strength of the steel casing
f_y	Nominal yield strength of structural steel
h	Thickness (with appropriate subscripts), greatest lateral dimension of a column
h_c	Thickness of the concrete slab forming the flange of the concrete beam
I	Second moment of area (with appropriate subscripts)
K	A constant (with appropriate subscripts)
k	A constant (with appropriate subscripts)
L_s	Length of the shear plane under consideration
l	Distance from face of support to the end of a cantilever, or effective span of a beam (distance between centres of supports) or length of column between centres of end restraints[a]
l_E	Length of column for which the Euler load equals the squash load
l_e	Effective length of a column or l_x or l_y as appropriate
l_S	Distance from end of beam
l_{ss}	One-fifth of the effective span
M	Bending moment (with appropriate subscripts)
M_{max}	Maximum moment
M_u	Ultimate moment of resistance
M_{ux}	Ultimate moment of resistance about the major axis
M_{uy}	Ultimate moment of resistance about the minor axis
M_x	Moment acting about the major axis, longitudinal bending moment per unit width of filler beam deck

M_y	Moment acting about the minor axis, longitudinal bending moment per unit width of filler beam deck
m	Constant
N	Ultimate axial load at the section considered
N_{ax}, N_{ay}	Axial failure loads
N_u	Squash load of a column
N_x	Design failure load of a column subjected to a constant design moment M_x
N_{xy}	Strength of a column in biaxial bending
N_y	Design failure load of a column subjected to a constant design moment M_y
n	Total number of connectors per unit length of girder
n'	Number of connectors per unit length placed within 200 mm of the centre line of the web
P_u	Nominal static strength
P	Failure load of the connectors at concrete strength f_c
Q	Longitudinal shear force
Q^*	Design load
q	Longitudinal shear per unit length
q_p	Design longitudinal shear force per unit length of beam on the particular shear plane considered
S	Plastic section modulus
S^*	Design loading effects
s	A constant stress of 1 N/mm^2 re-expressed where necessary in units consistent with those used for other quantities
s_b	Spacing of bars
T_u	Tension
t	Wall thickness
t_f	Flange thickness
t_w	Web thickness
x	Neutral axis depth, coordinate
y	Coordinate
α_c	Concrete contribution factor
α_e	Modular ratio
α_L	Ratio of the product of the partial safety factors $\gamma_{fL}\,\gamma_{f3}$ for HB loading to the corresponding product for HA loading for the limit state being considered
β	Ratio of the smaller to the larger of the two end moments acting about each axis with appropriate subscripts
β_L	Coefficient of linear thermal expansion
γ_{fL}	Partial safety factor for loads and load effects
γ_{f1}, γ_{f2}, γ_{f3}	Partial safety factors for loads and load effects
γ_m	Partial safety factor for strength
Δf	Difference between the free strains at the centroid of the concrete slab and the centroid of the steel beam
ϵ_{cs}	Free shrinkage strain
ϵ_m	Average strain
ϵ_1	Strain at the level considered
η	Imperfection constant for composite columns
λ	Slenderness function (with appropriate subscripts)
λ_E	Euler slenderness function
μ	Coefficient of friction
ρ	Ratio of the average compressive stress in the concrete at failure to the design yield strength of steel, taken as $0.4\,f_{cu}/0.91\,f_y$
Φ	Bar size
ϕ	Creep coefficient
ϕ_c	Creep reduction factor
χ	Non-dimensional coordinate
Ψ	Effective breadth ratio, coefficient

[a] In Appendix A l is used as the distance between adjacent points at which the bending moment is zero.

4 Design: general

4.1 Design philosophy

4.1.1 ***General.*** Design should be in accordance with Part 1.

4.1.2 ***Design loads due to shrinkage of concrete.*** For shrinkage modified by creep the partial safety factor γ_{fL} should be taken as 1.0 for the serviceability limit state and 1.2 for the ultimate limit state.

NOTE For the definition of the partial safety factor, see Part 1.

4.1.3 ***Design loading effects***. The design loading effects S^* for design in accordance with this Part of this British Standard may be determined from the design loads Q^* from either:

a) S^* = effects of ($\gamma_{f3}Q^*$) or

b) $S^* = \gamma_{f3}$ (effects of Q^*).

The partial factor of safety γ_{f3} should be taken as 1.10 at the ultimate limit state and 1.0 at the serviceability limit state.

Where the relationship between loading and load effects is non-linear, as for example in slender columns or in beams designed using simple plastic theory, the method given in a) is the more simple.

4.2 Material properties

4.2.1 *General.* In analysing a structure to determine the load effects, the material properties associated with the unfactored characteristic strength should be used irrespective of the limit state being considered. For analysis of sections, the appropriate value of the partial factor of safety γ_m, to be used in determining the design strength, should be taken from Part 3 or Part 4 depending on the materials and the limit state. Except where specifically stated, it may be assumed that allowance has been made in the limiting stresses and expressions given in this Part of this British Standard, for the values of γ_m given in Table 1.

Table 1 — Values of the partial safety factor for materials γ_m

Material	Serviceability limit state	Ultimate limit state
Structural steel	1.0	1.10
Reinforcement	1.0	1.15
Concrete	1.0 or 1.3 (as appropriate)	1.5
Shear connectors	1.0	1.10

4.2.2 *Structural steel.* The characteristic or nominal properties of structural steel should be determined in accordance with Part 3.

4.2.3 *Concrete, reinforcement and prestressing steels.* The characteristic properties of concrete, reinforcement and prestressing steels should be determined in accordance with Part 4. For sustained loading, it should be sufficiently accurate to assume a modulus of elasticity of concrete equal to one half of the value used for short term loading.

4.3 Limit state requirements

4.3.1 *General.* Except where otherwise recommended in this Part of this British Standard the design of structural steelwork and structural concrete elements forming part of a composite structure should satisfy the recommendations of Parts 3, 4 and 10.

4.3.2 *Serviceability limit state.* A serviceability limit state is reached when any of the following conditions occur:

a) the stress in structural steel reaches the appropriate limit given in Part 3;

b) the stress in concrete, reinforcement or prestressing tendons reaches the appropriate limit given in Part 4;

c) the width of a crack in concrete, calculated in accordance with **5.2.6**, reaches the appropriate limit given in Part 4;

d) the slip at the interface between steel and concrete becomes excessive.

NOTE This is assumed to occur when the calculated load on a shear connector exceeds 0.55 times its nominal static strength;

e) the vibration in a structure supporting a footway or cycle track reaches the appropriate limit given in Part 2.

4.3.3 *Ultimate limit state.* General recommendations for composite structures at the ultimate limit state are as given in Part 1.

5 Design and detailing of the superstructure for the serviceability limit state

5.1 Analysis of structure

5.1.1 *Distribution of bending moments and vertical shear forces*

5.1.1.1 *General.* The distributions of bending moments and vertical shear forces, due to loading on the composite member, may be calculated by an elastic analysis assuming the concrete to be uncracked and unreinforced. The effects of shear lag may be neglected.

5.1.1.2 *Continuous beams.* In continuous beams, at each internal support, the apparent tensile stress in the concrete at the top surface of the slab due to the maximum design hogging (negative) moment obtained from **5.1.1.1** should be calculated. For this calculation, the composite section should be taken as the appropriate steel member acting compositely with a concrete flange equal in breadth to the effective breadth determined in accordance with **5.2.3**. The concrete should be assumed to be uncracked and unreinforced. If this tensile stress f_{tc} exceeds 0.1 f_{cu} then either:

a) a new distribution of bending moments should be determined as in **5.1.1.1** but neglecting the stiffening effect of the concrete over 15 % of the length of the span on each side of each support so affected. For this purpose, longitudinal tensile reinforcement in the slab may be included. Or, alternatively,

b) provided adjacent spans do not differ appreciably in length, the maximum design sagging moments in each span adjacent to each support so affected should be increased by 40 f_{tc}/f_{cu} % to allow for cracking of the concrete slab at the support. In this case, no reduction should be made in the support moment.

5.1.1.3 *Prestressing in continuous beams.* Where the concrete flange in the hogging (negative) moment region of a continuous composite beam is longitudinally prestressed the distribution of bending moments and vertical shear forces should be determined in accordance with **5.1.1.1**.

5.2 Analysis of sections

5.2.1 ***General.*** Composite elements should generally be designed to satisfy the requirements of the serviceability limit state given in **4.3.2** in accordance with **5.1** and **5.2**. However, for slender cross sections (see **6.2.2.2**) where both the distribution of bending moments and the stresses in the composite section are determined by elastic analysis at the ultimate limit state, temperature and shrinkage effects being included, no check need be made on the flexural stresses at the serviceability limit state, provided the stress limitations at the ultimate limit state given in **6.2.4.1** a) are not exceeded.

5.2.2 ***Analysis.*** Stresses due to bending moments and vertical shear forces may be calculated by elastic theory using the appropriate elastic properties given in **4.2** and effective breadths as given in **5.2.3**, assuming that there is full interaction between the steel beam and the concrete in compression. Vertical shear should be assumed to be resisted by the steel section alone and the tensile strength of concrete should be neglected.

5.2.3 *Effective breadth of concrete flange*

5.2.3.1 ***General.*** Except as provided in **5.3.1**, for longitudinal shear, and **5.5.2**, for deflections, and in the absence of a rigorous analysis, allowance for in-plane shear flexibility in the flange (shear lag effects) should be made by using an effective breadth of flange.

NOTE The total effective breadth of flange associated with each web should be taken as the sum of the effective breadths of the portions of flange considered separately on each side of that web.

The effective breadth b_e of each portion should be taken as:

a) ψb for portions between webs

where

b is equal to half the distance between the centre lines of webs, measured at the mid-plane of the concrete flange; and

b) 0.85 ψb for portions projecting beyond an outer web

where

b is equal to the distance from the free edge of the projecting portion to the centre line of the outer web, measured at the mid-plane of the concrete flange l is the distance from face of support to the end of a cantilever or the distance between the centres of supports of a beam (except as noted in **5.2.5.4**) ψ is the effective breadth ratio from Table 2, Table 3 or Table 4. Values of ψ at cross sections and for ratios b/l other than those covered by Table 2, Table 3 or Table 4 may be obtained by linear interpolation.

Table 2 — Effective breadth ratios ψ for simply supported beams

b/l	Loading uniformly distributed over a length not less than 0.5 l			Point loading at midspan[a]		
	Midspan	Quarter span	Support	Midspan	Quarter span	Support
0	1.0	1.0	1.0	1.0	1.0	1.0
0.02	0.99	0.99	0.93	0.91	1.0	1.0
0.05	0.98	0.98	0.84	0.80	1.0	1.0
0.10	0.95	0.93	0.70	0.67	1.0	1.0
0.20	0.81	0.77	0.52	0.49	0.98	1.0
0.30	0.65	0.60	0.40	0.38	0.82	0.85
0.40	0.50	0.46	0.32	0.30	0.63	0.70
0.50	0.38	0.36	0.27	0.24	0.47	0.54

[a] To be used only for point loads or reactions of significant magnitude not for wheel loads or axle loads.

Table 3 — Effective breadth ratios ψ for cantilever beams

b/l	Loading uniformly distributed over a length not less than 0.5 l			Point loading at free end[a]		
	Support	Quarter point near support	Free end	Support	Quarter point near support	Free end
0	1.0	1.0	1.0	1.0	1.0	1.0
0.05	0.82	1.0	0.92	0.91	1.0	1.0
0.10	0.68	1.0	0.84	0.80	1.0	1.0
0.20	0.52	1.0	0.70	0.67	0.84	1.0
0.40	0.35	0.88	0.52	0.49	0.74	1.0
0.60	0.27	0.64	0.40	0.38	0.60	0.85
0.80	0.21	0.49	0.32	0.30	0.47	0.70
1.00	0.18	0.38	0.27	0.24	0.36	0.54

[a] To be used only for point loads or reactions of significant magnitude not for wheel loads or axle loads.

Table 4 — Effective breadth ratios ψ for internal spans of continuous beams

b/l	Loading uniformly distributed over a length not less than 0.5 l			Point loading at midspan[a]		
	Midspan	Quarter span	Internal support	Midspan	Quarter span	Internal support
0	1.0	1.0	1.0	1.0	1.0	1.0
0.02	0.99	0.94	0.77	0.84	1.0	0.84
0.05	0.96	0.85	0.58	0.67	1.0	0.67
0.10	0.86	0.68	0.41	0.49	1.0	0.49
0.20	0.58	0.42	0.24	0.30	0.70	0.30
0.30	0.38	0.30	0.15	0.19	0.42	0.19
0.40	0.24	0.21	0.12	0.14	0.28	0.14
0.50	0.20	0.16	0.11	0.12	0.20	0.12

[a] To be used only for point loads or reactions of significant magnitude not for wheel loads or axle loads.

5.2.3.2 *Standard highway or railway loading.* The effective breadth ratios ψ to be used in stress calculations on structural elements subjected to standard highway or railway loading, as specified in Part 2, should be the appropriate values for uniformly distributed loading given in Table 2, Table 3 or Table 4.

5.2.3.3 *Width over which slab reinforcement is effective.* Only reinforcement placed parallel to the span of the steel beam within the effective breadth of the concrete slab should be assumed to be effective in analysing cross sections.

5.2.3.4 *Longitudinal stiffening.* The effective breadth ratios given in Table 2, Table 3 and Table 4 take no account of the increase in shear lag in the flange of a composite beam due to the presence of longitudinal stiffening members. Table 2, Table 3 and Table 4 may be taken to apply where the cross-sectional area of longitudinal stiffening members within breadth b does not exceed one quarter of the cross-sectional area of the flange within breadth b and where these areas are calculated for steel and concrete on the basis of the modular ratio. Effective breadth ratios for heavily stiffened steel flanges are given in Part 3.

5.2.3.5 *End spans of continuous beams.* For end spans of a continuous beam, the effective breadth ratios ψ, for the portion between the end support and the adjacent point of contraflexure, may be calculated in accordance with **5.2.3.1** for simply supported spans by considering the end span as an equivalent span of length 0.9 l.

5.2.3.6 *Transverse members.* The effective breadths given in Table 2, Table 3 and Table 4 take no account of any contribution to the in-plane shear stiffness of a flange that may be made by transverse members connected to it.

5.2.3.7 *Effective breadth at internal supports.* The effective breadth ratio at an internal support may be taken as the mean value of ψ obtained at that support for each span adjacent to that support. For a concrete flange in tension that is assumed to be cracked the mean effective breadth ratio ψ obtained from Table 3 or Table 4 as appropriate, may be modified by adding $(1 - \psi)/3$.

5.2.3.8 *Indeterminate structures and special loadings.* For indeterminate structures not specifically covered by **5.2.3.1** to **5.2.3.7** and for load positions other than those given in Table 2, Table 3 or Table 4, the effective breadth ratios ψ may be determined by the method given in Appendix A.

5.2.4 ***Deck slabs forming flanges of composite beams***

5.2.4.1 *Effects to be considered.* The slab should be designed to resist:

a) the effects of loading acting directly on it, and

b) the effects of loading acting on the composite member or members of which it forms a part.

These effects should be considered separately and, where they arise together, in conjunction. Coexistent stresses acting in the same direction should be added algebraically.

NOTE Recommendations for the design of slabs subject to longitudinal shear are given in **5.3** and **6.3**.

5.2.4.2 *Serviceability requirements.* Except as directed in **5.2.1**, the stresses in the concrete slab and reinforcement should be determined by elastic analysis and should not exceed the appropriate limits given in Part 4. Crack widths should be controlled in accordance with **5.2.6**.

5.2.4.3 *Coexistent stresses.* In calculating coexistent stresses in a deck slab, which also forms the flange of a composite beam, account may be taken of the effects of shear lag to reduce the longitudinal bending stress in regions of the flange remote from the web/flange junction. The longitudinal stress f_L, at any point in the flange distance x from the centre line of the web, may be calculated from:

$$f_L = f_{max}\,[\chi^4 + k(1 - \chi^4)]$$

where

$$\chi = \left(\frac{b - x}{b}\right)$$

$k = \frac{1}{4}\,(5\,\psi - 1)$ for portions between web centre lines, or

$k = \frac{1}{4}\,(4.25\,\psi - 1)$ for portions projecting beyond an outer web

f_{max} is the maximum stress in the concrete flange due to longitudinal bending of the composite section calculated by elastic analysis using the effective flange breadth determined in accordance with **5.2.3**

ψ, b are as defined in **5.2.3**, and

x is as given in Figure 1

If the calculated value of f_L turns out to be negative it should be taken as zero.

5.2.5 ***Steel section***

5.2.5.1 *General.* The steel section should be designed in accordance with the recommendations of Parts 3 and 10.

Consideration should be given to the effects noted in **5.2.5.2** to **5.2.5.4**.

5.2.5.2 *Unpropped construction.* Except as noted in **5.2.5.4**, where the steel section carries load prior to the development of composite action, the resulting stresses and deflections should be added algebraically to those later induced in the composite member, of which the steel section forms a part, and the appropriate limit states should be satisfied.

5.2.5.3 *Propped construction.* Where composite action has been assumed for the whole of the design load, consideration should be given to the nature and layout of the props to ensure that the assumptions made in the design will be achieved. Where significant prop settlement cannot be avoided the reduction in propping force should be taken into account.

5.2.5.4 *Slab cast in specified sequence.* Where the deck slab is cast in a specified sequence the dead load stresses may be calculated on the composite section in accordance with **12.1**, using the effective breadth determined from **5.2.3** and the relevant design procedures.

NOTE For the purpose of estimating the effective breadth of the flange, l, in **5.2.3**, should be taken as the continuous length of concrete in the flange containing the section under consideration which is assumed to act compositely.

5.2.6 ***Control of cracking in concrete***

5.2.6.1 *General.* Adequate reinforcement should be provided in composite beams to prevent cracking from adversely affecting the appearance or durability of the structure.

NOTE Special recommendations for cased beams and filler beams are given in clause **8**.

5.2.6.2 *Loading.* In calculating crack widths in reinforced concrete flanges of highway bridges the loading should only comprise dead loading, superimposed dead loading, HA loading (with the HA wheel load excluded except for cantilever slabs, the top flange of beam and slab decks and open-type composite steel box girder bridges) and/or pedestrian loading. In arriving at the design value for the various loads the partial safety factors γ_{f1} and γ_{f2} appropriate to load combination 1 (see Part 2) should be used with γ_{f3} equal to 1.0, except that for HA and HB loading the total partial safety factor on loads $\gamma_{f1}\,\gamma_{f2}\,\gamma_{f3}$ should be taken as 1.0, i.e. the nominal load is adopted. For spans less than 6.5 m 25 units of HB loading with associated HA loading should be considered.

For railway loading the values of $\gamma_{f1}\,\gamma_{f2}$ given in Part 2 with γ_{f3} equal to 1.0 should be used.

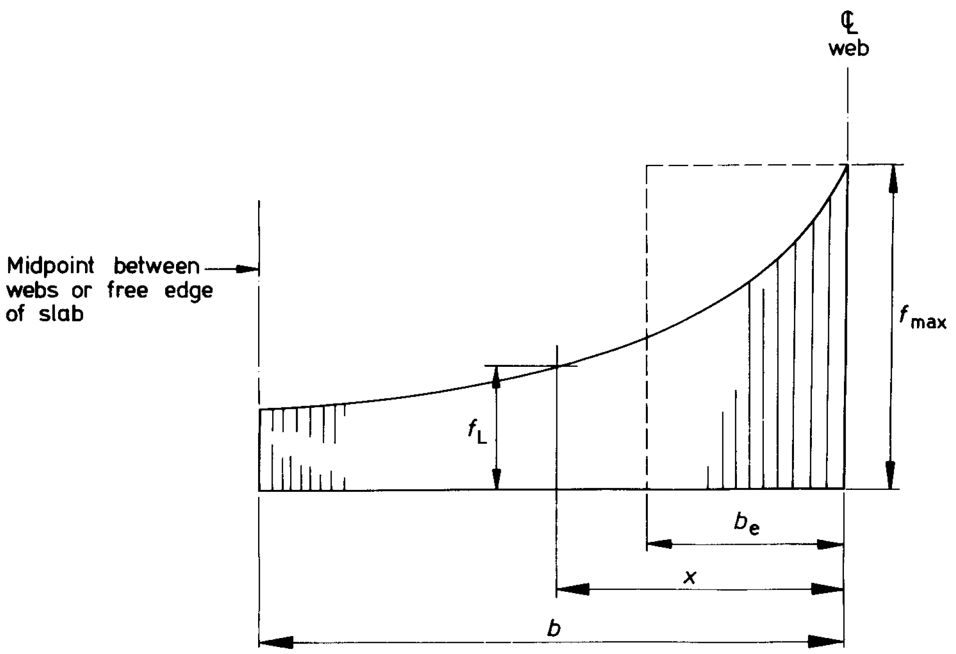


Figure 1 — Distribution of longitudinal stress in the concrete flange of a composite beam

5.2.6.3 *Limiting crack width.* The engineer should satisfy himself that cracking will not be excessive with regard to the requirements of the particular structure, its environment and the limits to the widths of cracks given in Part 4. Surface crack widths in a composite beam under the action of the loadings specified in **5.2.6.2** may be calculated by the method given in Appendix B. In a concrete flange where the effects of global and local loading coexist the crack width due to global longitudinal bending should be determined in accordance with **5.2.6** and Appendix B. The crack width due to longitudinal local bending in the slab should be determined in accordance with Part 4. The sum of the crack widths due to longitudinal global and local bending, calculated in this manner, should not exceed the appropriate limit. In calculating the strain due to global longitudinal bending account may be taken of the beneficial effect of shear lag in regions remote from the webs in accordance with **5.2.4.3**.

5.2.6.4 *Maximum distance between bars in tension in highway bridges designed for HA and/or HB loading.* Reinforcement provided in accordance with this clause may be deemed to satisfy the recommendations of **5.2.6.3**, in respect of composite beams in moderate conditions of exposure, provided the nominal cover to the reinforcement is not greater than 30 mm.

The clear distance between adjacent bars near the tension face of a composite beam should be not greater than the spacing given in Table 5 or Table 6, as appropriate, depending on the stress in the reinforcement at the ultimate limit state and the amount of redistribution carried out in the analysis at the ultimate limit state.

Reinforcement bars of diameter less than 0.45 times the diameter of the maximum size of tension bar at the section considered should be neglected for the purpose of this clause.

5.3 Longitudinal shear

5.3.1 ***General.*** Longitudinal shear per unit length of the composite beam q, whether simply supported or continuous, should be calculated for the serviceability limit state on the basis of elastic theory using the properties of the transformed composite cross section calculated assuming the concrete flange to be uncracked and unreinforced. The effective breadth of concrete flange may be assumed to be constant over any span and may be taken as the quarterspan value for uniformly distributed loading given in Table 2, Table 3, or Table 4, as appropriate.

Where the second moment of area of the composite section, thus obtained, varies significantly along the length of any span account should be taken of the variation of stiffness in calculating the longitudinal shear flow.

Table 5 — Clear distance (mm) between bars in tension for propped construction

Stress in reinforcement at ultimate limit state, N/mm^2	Type of analysis at ULS[a]	% redistribution to section considered						
	Uncracked	– 30	– 25	– 20	– 15	– 10	– 5	0
	Cracked	– 20	– 15	– 10	– 5	– 0	+ 5	+ 10
$250/\gamma_m$		200	210	225	235	250	260	275
$410/\gamma_m$		120	130	135	145	150	160	165
$460/\gamma_m$		110	115	120	130	135	140	150

[a] Ultimate limit state.

Table 6 — Clear distance (mm) between bars in tension for unpropped construction

Stress in reinforcement at ultimate limit state, N/mm^2	Type of analysis at ULS[a]	% redistribution section considered						
	Uncracked	– 30	– 25	– 20	– 15	– 10	– 5	0
	Cracked	– 20	– 15	– 10	– 5	0	– 5	+ 10
$250/\gamma_m$		230	240	255	270	285	300	300
$410/\gamma_m$		140	150	160	165	175	185	190
$460/\gamma_m$		125	130	140	145	155	165	170

[a] Ultimate limit state.

5.3.2 *Shear Connectors*

5.3.2.1 *Nominal strengths of shear connectors embedded in normal density concrete*

a) *Static strengths.* Table 7 gives the nominal static strengths of commonly used types of connectors, which are illustrated in Figure 2, in relation to the specified characteristic cube strengths of the normal grades of concrete. The nominal strengths given in Table 7 may be used where the slab is haunched provided that the haunch complies with **6.3.2.1**. For other haunches reference should be made to **5.3.2.3**.

b) *Fatigue strengths.* The fatigue strength of connectors should be determined in accordance with Part 10.

c) *Strengths of connectors not included in Table 7.* Static strengths should be determined experimentally by push-out tests in accordance with **5.3.2.4**. Where the connector type is included in Table 7, but the appropriate size is not given, the fatigue strength should be determined in accordance with Part 10.

5.3.2.2 *Nominal strengths of shear connectors embedded in lightweight concrete.* The strengths given in a) and b) may be used where the slab is haunched provided that the haunch complies with **6.3.2.1**.

NOTE For other haunches see **5.3.2.3**.

a) *Static strengths.* The nominal static strengths of headed stud connectors embedded in lightweight concrete of density greater than 1 400 kg/m^3 may be taken as 15 % less than the values given in Table 7. Static strengths of other sizes of stud and of other types of connectors should be determined experimentally by push-out tests made in accordance with **5.3.2.4**.

b) *Fatigue strengths.* The fatigue strength of shear connectors embedded in lightweight concrete of density greater than 1 400 kg/m^3 should be determined in accordance with Part 10.

5.3.2.3 *Nominal strengths of shear connectors in haunched slabs.* Where the haunch does not comply with **6.3.2.1** the nominal static strength of the shear connectors P_u should be determined experimentally by push-out tests (see **5.3.2.4**).

The fatigue strength should be determined in accordance with Part 10.

5.3.2.4 *Tests on shear connectors*

a) *Nominal strength.* The nominal static strength of a shear connector may be determined by push-out tests. Not less than three tests should be made and the nominal static strength P_u may be taken as the lowest value of $f_{cu}P/f_c$ for any of the tests, where P is the failure load of the connectors at concrete strength f_c, and f_{cu} is the specified characteristic cube strength at 28 days.

b) *Details of tests.* Suitable dimensions for the push-out specimen are given in Figure 4. Bond at the interfaces of the flanges of the steel beam and the concrete should be prevented by greasing the flange or by other suitable means. The slab and reinforcement should be either as given in Figure 4 or as in the beams for which the test is designed.

The strength of the concrete f_c, at the time of testing, should not differ from the specified cube strength f_{cu} of the concrete in the beams by more than ± 20 %. The rate of application of load should be uniform and such that failure is reached in not less than 10 min.

c) *Resistance to separation.* Where the connector is composed of two separate elements, one to resist longitudinal shear and the other to resist forces tending to separate the slab from the girder, the ties which resist the forces of separation may be assumed to be sufficiently stiff and strong if the separation measured in push-out tests does not exceed half of the longitudinal slip at the corresponding load level. Only load levels up to 80 % of the nominal static strength of the connector need be considered.

Table 7 — Nominal static strengths of shear connectors for different concrete strengths

Type of connector		Connector material	Nominal static strengths in kN per connector for concrete strengths f_{cu}, Nmm^2			
			20	**30**	**40**	**50**
Headed studs [see Figure 2(a)]		Material with a characteristic yield stress of 385 N/mm^2 minimum elongation of 18 % and a characteristic tensile strength of 495 N/mm^2				
Diameter	**Overall height**					
mm	mm					
25	100		139	154	168	183
22	100		112	126	139	153
19	100		90	100	109	119
19	75		78	87	96	105
16	75		66	74	82	90
13	65		42	47	52	57
Bars with hoops [see Figure 2(b) and Figure 2(c)]		Grade 43 of BS 4360:1972				
50 mm × 40 mm × 200 mm bar			697	830	963	1 096
25 mm × 25 mm × 200 mm bar			348	415	482	548
channels [see Figure 2(d)]		Grade 43 of BS 4360:1972				
127 mm × 64 mm × 14.90 kg × 150 mm			351	397	419	442
102 mm × 51 mm × 10.42 kg × 150 mm			293	337	364	390
76 mm × 38 mm × 6.70 kg × 150 mm			239	283	305	326
Friction grip bolts		BS 4395	see clause **10**			

NOTE 1 f_{cu} is the specified characteristic cube strength at 28 days.

NOTE 2 Strengths for concrete of intermediate grade may be obtained by linear interpolation.

NOTE 3 For bars (see Figure 2(b) and Figure 2(c), and channels (see Figure 2(d) of lengths different from those quoted above, the capacities are proportional to the lengths for lengths greater than 100 mm.

NOTE 4 For stud connectors of overall height greater than 100 mm the nominal static strength should be taken as the values given in Table 7 for 100 mm high connectors unless the static strength is determined from push-out tests in accordance with **5.3.2.4**.

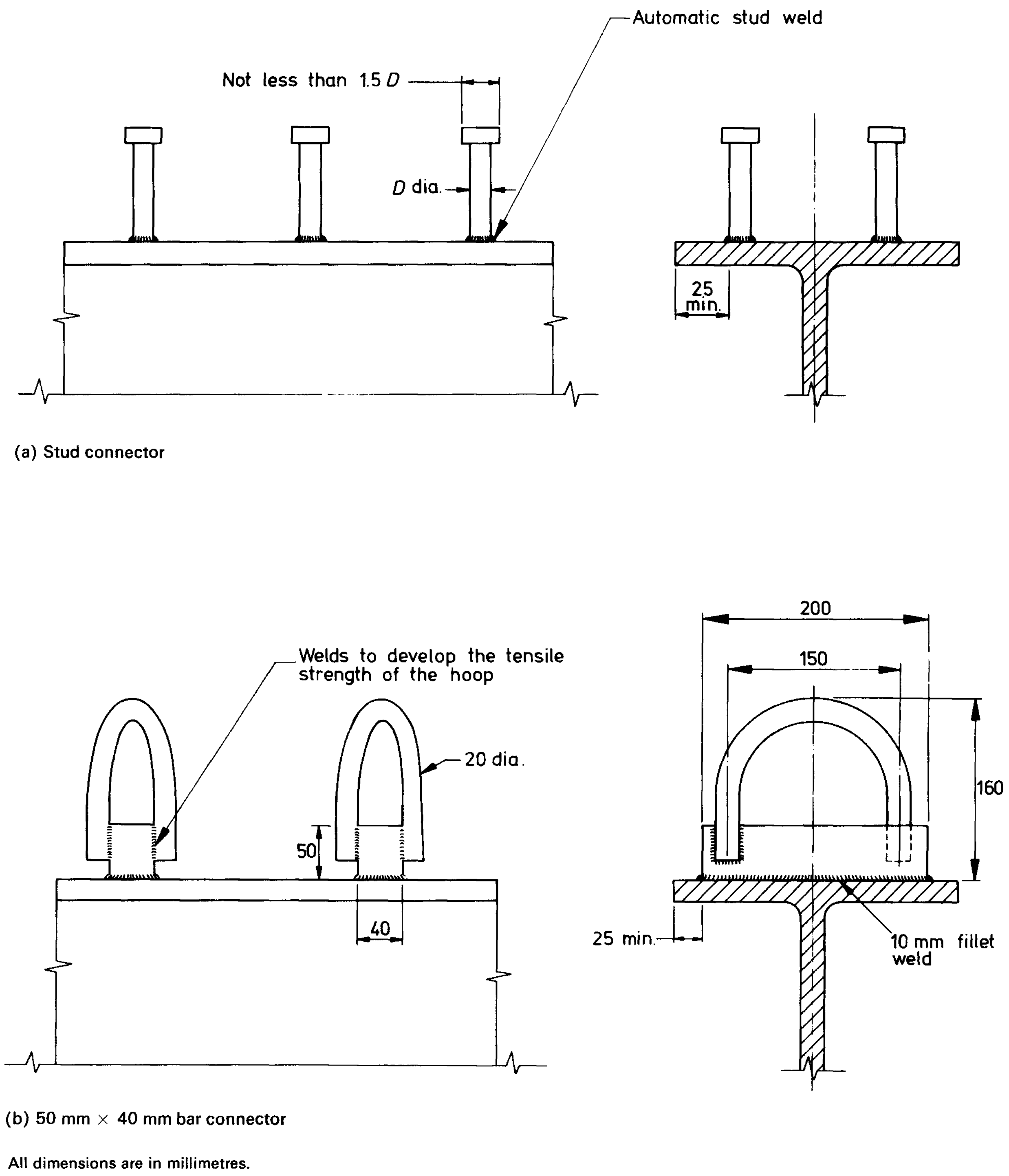
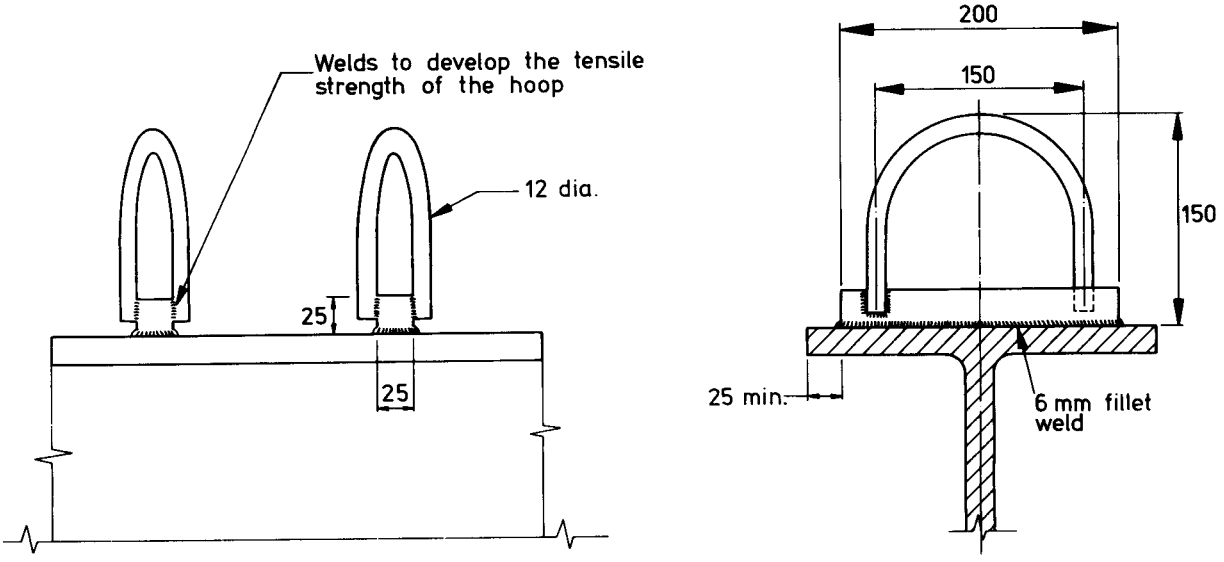
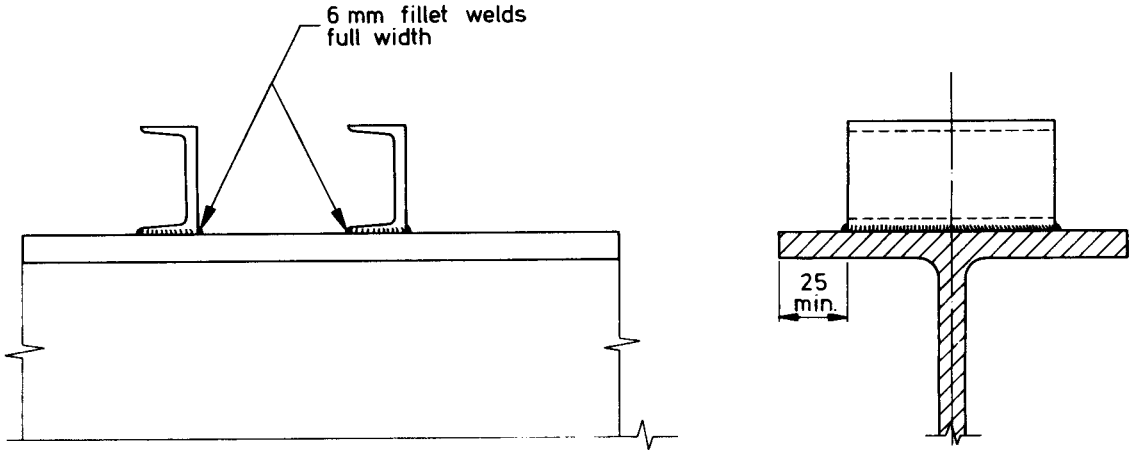


Figure 2 — Shear connectors



(c) 25 mm × 25 mm bar connector



(d) Channel connector

All dimensions are in millimetres.

Figure 2 — Shear connectors *(concluded)*

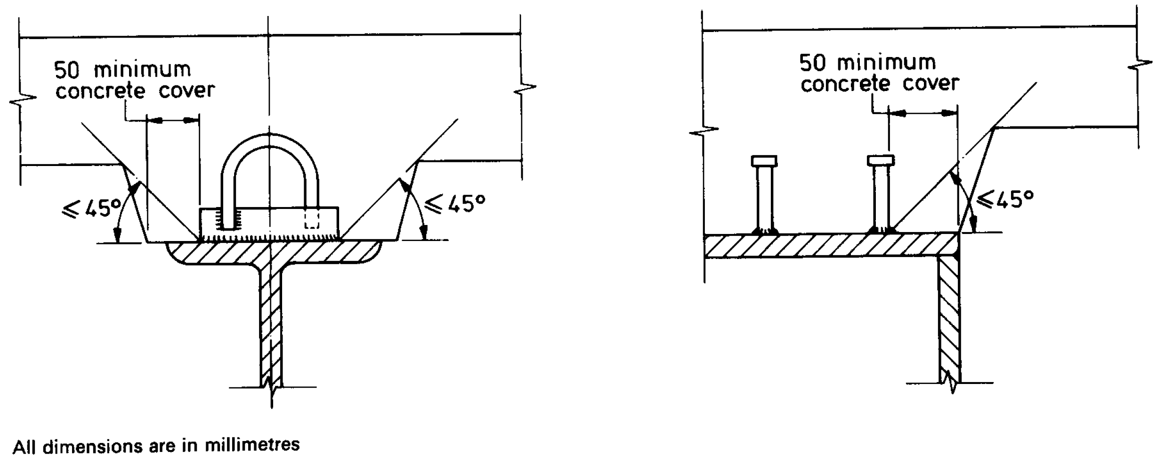
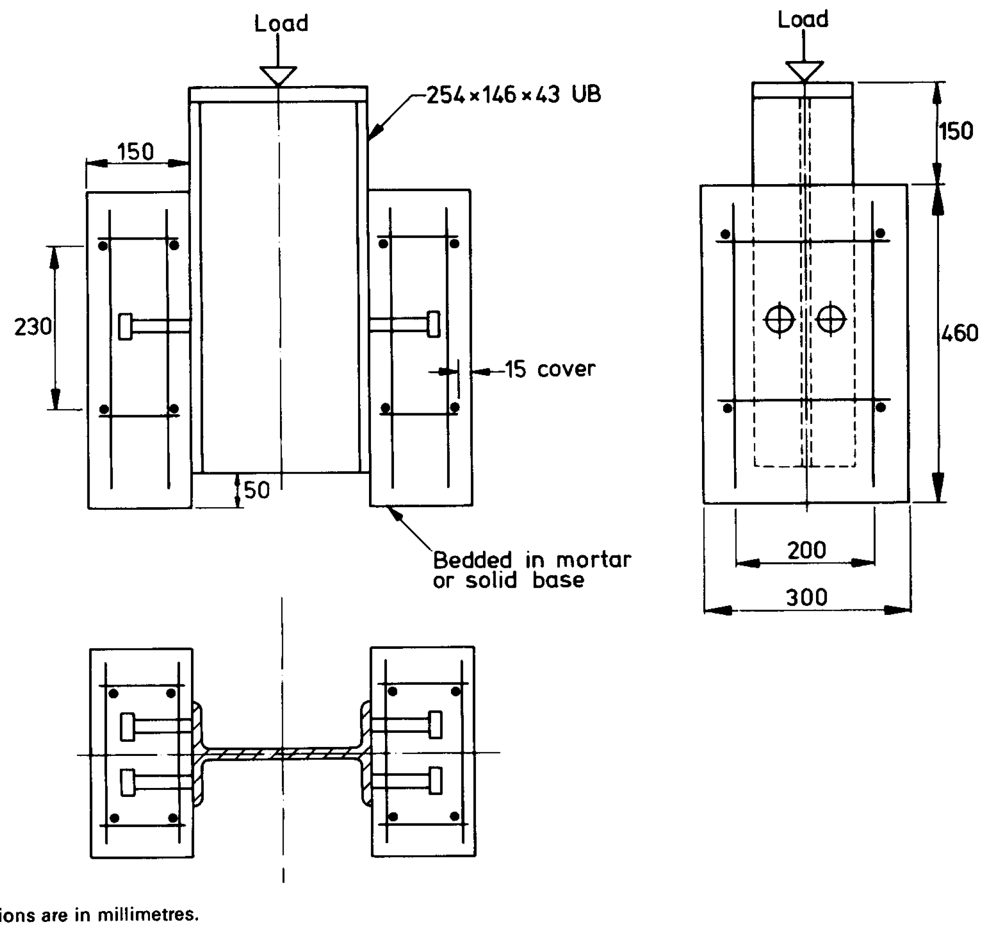


Figure 3 — Dimensions of haunches



NOTE Reinforcement should be of 10 mm diameter mild steel.

Figure 4 — Dimensions of specimens for test on shear connectors

5.3.3 ***Design of shear connection***

5.3.3.1 *General.* The longitudinal spacing of the connectors should be not greater than 600 mm or three times the thickness of the slab or four times the height of the connector, including any hoop which is an integral part of the connector, whichever is the least, except that:

a) in negative (hogging) moment regions of continuous cross girders no connector should be placed within a distance $2b_f$ of the nearest connectors on the main girder; where b_f is the breadth of the tension flange of the steel cross-girder;

b) connectors may be placed in groups, with the group spacing greater than that specified for individual connectors, provided consideration is given in design to the non-uniform flow of longitudinal shear and of the greater possibility of slip and vertical separation between the slab and the steel member.

The distance between the edge of a shear connector and the edge of the plate to which it is welded should be not less than 25 mm (see Figure 2).

The diameter of stud connectors welded to a flange plate, which is subject to tensile stresses, should not exceed one and a half times the thickness of the plate. Where a plate is not subject to tensile stresses the diameter of stud connectors should not exceed twice the plate thickness.

The leg length of the weld joining other types of connectors to the flange plate should not exceed half the thickness of the flange plate.

Except where otherwise permitted for encased and filler beams shear connectors should be provided throughout the length of the beam.

5.3.3.2 *Horizontal cover to connectors.* The horizontal distance between a free concrete surface and any shear connector should be not less than 50 mm (see Figure 3). At the end of a cantilever, as for example in a cantilever-suspended span structure, sufficient transverse and longitudinal reinforcement should be positioned adjacent to the free edge of the concrete slab to transfer the longitudinal shear connector loads back into the concrete slab.

5.3.3.3 *Resistance to separation.* The slab should be positively tied to the girder in accordance with the following recommendations.

a) The overall height of a connector, including any hoop which is an integral part of the connector, should be not less than 100 mm or the thickness of the slab less 25 mm whichever is the lesser.

b) The surface of a connector that resists separation forces, i.e. the inside of a hoop, the inner face of the top flange of a channel or the underside of the head of a stud, should neither extend less than 40 mm clear above the bottom transverse reinforcement (see Figure 6) nor less than 40 mm into the compression zone of the concrete flange in regions of sagging longitudinal moments. Alternatively, where a concrete haunch is used between the steel girders and the soffit of the slab, transverse reinforcing bars, sufficient to satisfy the requirements of **6.3.3**, should be provided in the haunch at least 40 mm clear below the surface of the connector that resists uplift. Where the shear connection is adjacent to a longitudinal edge of a concrete slab, transverse reinforcement provided in accordance with **6.3.3** should be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

c) Where the slab is connected to the girder by two separate elements, one to resist longitudinal shear and the other to resist forces tending to separate the slab from the girder, the ties which resist the forces of separation should be in accordance with a) and b).

5.3.3.4 *Uplift on shear connectors.* Where the shear connectors are subject to significant calculable direct tension due either to:

a) forces tending to separate the slab from a girder caused, for example, by differential bending of the girders or of the two sides of a box girder or tension-field action in a web, or

b) transverse moments on a group of connectors resulting from transverse bending of the slab particularly in the region of diaphragms or transverse cross bracing,

then additional ties, suitably anchored, should be provided to resist these forces. Alternatively, stud shear connectors may be used and should be checked for the ultimate limit state and, if applicable, for fatigue.

The effect of axial tension on the static or fatigue shear strength of a connector should be taken into account as follows, unless the reduction in P_u or the increase in Q_{max} is less than 10 %.

For stud connectors the nominal static ultimate shear strength P_u' in the presence of tension T_u may be taken as

$$P_u' = P_u - T_u/\sqrt{3}$$

where

P_u is the nominal static ultimate shear strength as defined in **5.3.2.1**

Where a stud is subject to shear Q and tension T_u the value of Q_{max}, to be used for calculating the shear range in fatigue calculations, should be taken as

$$Q_{max} = \sqrt{Q^2 + \frac{T_u^2}{3}}$$

5.3.3.5 *Design procedure: general.* Shear connectors should be designed initially to satisfy the serviceability limit state in accordance with **5.3.3.6**. The initial design should be checked in accordance with Part 10 for fatigue. Except as directed in **5.3.3.4** and **6.1.3** no check on the static strength of shear connectors need be made at the ultimate limit state.

5.3.3.6 *Design procedure, static loading.* The size and spacing of the connectors at each end of each span should be not less than that required for the maximum loading considered. This size and spacing should be maintained for at least 10 % of the length of each span. Elsewhere, the size and spacing of connectors may be kept constant over any length where, under the maximum loading considered, the maximum shear force per unit length does not exceed the design shear flow by more than 10 %. Over every such length the total design longitudinal shear force should not exceed the product of the number of connectors and the design static strength per connector (0.55 × nominal static strength).

5.4 Temperature effects and shrinkage modified by creep

5.4.1 ***General.*** Longitudinal stresses due to the effects of temperature and shrinkage modified by creep need only be considered at the serviceability limit state in composite beams where the cross section of the steel member is compact, as defined in **6.2.2.1**, or in slender sections as directed in **6.2.4.1** b). Account should be taken of the longitudinal shear forces arising from these effects in the design of all composite beams for the serviceability limit state. Where appropriate, variations in the stiffness of a composite beam along its length, e.g. due to changes in the cross section of the steel member or where the concrete flange is cast in stages, should be taken into account when calculating the longitudinal shear force per unit length.

5.4.2 ***Temperature effects***

5.4.2.1 *Effects to be considered.* Longitudinal stresses and longitudinal shear forces due to temperature effects should be taken into account where appropriate. The effects to be considered are:

a) primary effects due to a temperature difference through the depth of the cross section of the composite member;

b) primary effects due to a uniform change of temperature in a composite member where the coefficients of thermal expansion of the steel and concrete are significantly different; and

c) secondary effects, in continuous members, due to redistribution of the moments and support reactions caused by temperature effects of the types described in a) or b).

In the absence of a partial interaction analysis, longitudinal stresses and shear forces due to temperature effects should be calculated by elastic theory assuming that full interaction exists between the concrete slab and the steel beam. The stiffness should be based on the transformed composite cross section using a modular ratio α_e appropriate to short term loading and assuming the concrete slab to be of effective breadth as given in Table 8.

Table 8 — Properties of concrete flange for calculation of temperature effects

Purpose of calculation	Effective breadth of concrete flange to be determined from clause	State in which concrete slab is assumed to be
Longitudinal bending stresses due to primary effects	**5.2.3**	Uncracked
Longitudinal shear force due to primary effects	**5.3.1**	Uncracked
Moments and reactions due to secondary effects	**5.1.1.1**	Uncracked
Longitudinal bending stresses due to secondary effects	**5.2.3**	Cracked in tension

5.4.2.2 *Coefficient of linear expansion*

a) *Structural steel and reinforcement.* The coefficient of linear expansion β_L may be taken as 12 × 10^{-6}/°C.

b) *Concrete.* The coefficient of linear expansion βL, of normal density concrete (2 300 kg/m^3 or greater) made with aggregates other than limestone or granite, may be taken as 12 × 10^{-6}/°C. The use of limestone or certain granite aggregates may reduce the coefficient of linear expansion of the concrete to as low as 7 × 10^{-6}/°C. In these circumstances a value appropriate to the particular aggregate should be used. For lightweight aggregate concrete (density 1 400 kg/m^3 to 2 300 kg/m^3) the coefficient of linear expansion may normally be taken as 8 × 10^{-6}/°C.

5.4.2.3 *Longitudinal shear.* The longitudinal shear force Q, due to either a temperature difference through the depth of the cross section or differential thermal expansion between the concrete and steel beam, may be assumed to be transmitted from the concrete slab to the steel beam by connectors at each end of the beam ignoring the effects of bond. The forces on the connectors should be calculated on the basis that the rate of transfer of load varies linearly from $2Q/l_s$ at each end of the beam to zero at a distance l_s from each end of the beam, where

$$l_s = 2\sqrt{KQ/\Delta f}$$

where

Q is the longitudinal shear force due to the primary effects of temperature

Δf is the difference between the free strains at the centroid of the concrete slab and the centroid of the steel beam, and

$$K = \frac{\text{spacing of the connectors (mm)}}{\text{connector modulus (N/mm)}}$$

The value of K in mm^2/N will vary with the connector and concrete type and may be taken as follows:

	Stud connectors	Other connectors
Normal density concrete	0.003	0.0015
Lightweight aggregate concrete	0.006	0.003

Alternatively, where stud shear connectors are used the rate of transfer of load may be assumed to be constant for a distance l_{ss} from each end of the beam where l_{ss} is equal to one-fifth of the effective span.

5.4.2.4 *Longitudinal stresses.* Longitudinal stresses due to temperature effects may be calculated using the assumptions given in **5.4.2.1**.

5.4.3 ***Shrinkage modified by creep.*** When the effects of shrinkage modified by creep adversely affect the maximum resultant forces on the shear connectors or the maximum resultant stresses in the concrete slab and the steel beam, they should be calculated in the manner described for temperature effects in **5.4.2.1**, **5.4.2.3** and **5.4.2.4**, but using values of ϵ_{cs} the free shrinkage strain and a modular ratio α_e, appropriate to long term loading, which may be taken approximately as $2\,E_s/E_c$ or more accurately as $E_s/\phi_c E_c$.

where

E_c is the static secant modulus of elasticity of concrete

E_s is the elasticity of structural steel.

NOTE Values of ε_{cs} and ϕ_c are given in Table 9.

The values in Table 9 should only be used where the concrete specification complies with the limits given in Figure 5. For situations outside the scope of Figure 5 and Table 9 or where a better estimation of the effect of shrinkage modified by creep is required, the value of free shrinkage strain ϵ_{cs} and the creep coefficient ϕ may be determined in accordance with Appendix C of Part 4.

The value of ϕ_c should then be taken as

$$\phi_c = \frac{1}{1+\phi}$$

Table 9 — Shrinkage strains and creep reduction factors

Environment	ε_{cs}	ϕ_c
Very humid, e.g. directly over water	-100×10^{-6}	0.5
Generally in the open air	-200×10^{-6}	0.4
Very dry, e.g. dry interior enclosures	-300×10^{-6}	0.3

5.5 Deflections

5.5.1 ***General.*** Recommendations for deflections and general guidance on their calculation are given in Part 1. The partial load factor γ_{fL} is given in Part 2 and γ_{f3} is given in **4.1.3**.

5.5.2 ***Calculation of deflections.*** In calculating deflections consideration should be given to the sequence of construction and, where appropriate, proper account should be taken of the deflections of the steel section due to loads applied to it prior to the development of composite action and of partial composite action where deck slabs are cast in stages.

Deflections may be calculated by elastic theory using the elastic properties given in **4.2** and assuming full interaction between the concrete and steel beam and neglecting concrete in tension. Allowance for in-plane shear flexibility (shear lag effects) in the flange should be made in calculations based on the elementary theory of bending by using an effective breadth of flange.

To determine the effective breadth of flange the effective breadth ratio for all types of loading may be taken as constant along any equivalent simply supported span, as defined in **5.2.3.5**, **A.1** and **A.2**, and equal to the value given in Table 2 for the quarter span under uniformly distributed load, except that for cantilevers the value given in Table 3 for the quarter point near the support under uniformly distributed load should be used.

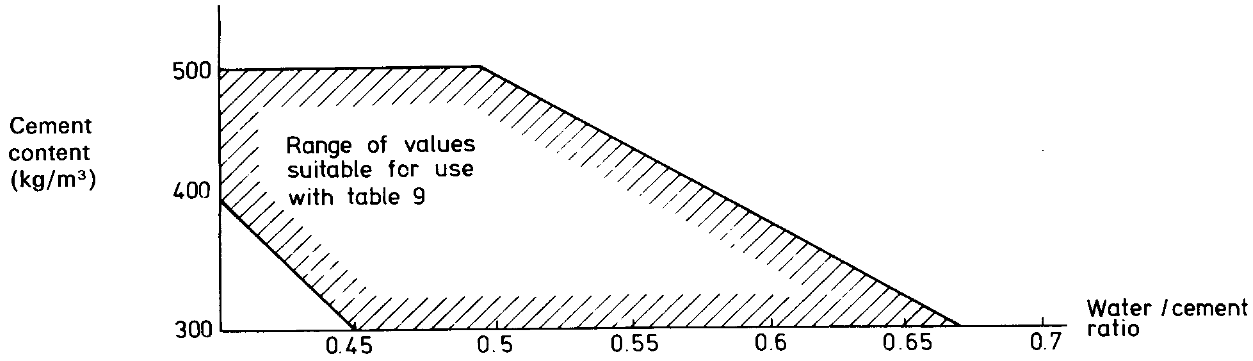


Figure 5 — Range of concrete mixes for which Table 9 can be used

In the absence of a more rigorous analysis of the effects of creep, the deflections due to sustained loading may be calculated by using a modulus of elasticity of concrete appropriate to sustained loading determined in accordance with **4.2.3**. Alternatively, under sustained loading, the modulus of elasticity may be taken as $1/(1 + \phi)$ times the short term modulus given in **4.2.3** where ϕ is the creep coefficient determined in accordance with Appendix C of Part 4.

6 Design and detailing of superstructure for the ultimate limit state

6.1 Analysis of structure

6.1.1 ***General.*** Except where alternative methods are given in **6.1.2** and **6.1.4.3**, elastic analysis should be used to determine the distribution of bending moments, shear forces and axial loads due to the design ultimate loadings specified in Part 2. The use of alternative methods should be in accordance with **8.2** of Part 1.

6.1.2 ***Deck slabs forming the flanges of composite beams.*** The deck slab should be designed to resist the effects of loading given in **5.2.4.1** but design loads relevant to the ultimate limit state should be used. In general, the effects of local wheel loading on the slab should be determined by elastic analysis. Alternatively, an inelastic method of analysis, e.g. yield line theory, may be used where an appropriate solution exists.

Where slabs form the flanges of composite beams which have slender steel sections (as defined in **6.2.2.2**) the stresses in the concrete and reinforcement should be determined by elastic analysis. The coexistent stresses in the concrete and reinforcement due to the effects of local wheel loading and global bending of the composite beam should not exceed the limits given in **6.2.4.1**. Advantage may be taken of the effects of shear lag in the manner described in **5.2.4.3**.

Where slabs form the flanges of composite beams which have compact steel sections (as defined in **6.2.2.1**) the design of the slab cross section should be in accordance with Part 4.

Proper account should be taken of the interaction between longitudinal shear forces and transverse bending of the slab in the region of the shear connection. The methods given in **6.3** may be deemed to satisfy these recommendations.

6.1.3 ***Composite action.*** Where the cross section is compact (as defined in **6.2.2.1**), and premature failure of the steel compression flange by lateral-torsional buckling is prevented in accordance with Part 3, composite action may be assumed to exist for the whole of the loading at the ultimate limit state, even when unpropped construction is used, provided that the shear connectors and transverse reinforcement are designed in accordance with **6.3** for the corresponding longitudinal shear. Where the cross section is slender (as defined in **6.2.2.2**) the requirements of **5.2.5** should be observed.

6.1.4 ***Distribution of bending moments and vertical shear forces***

6.1.4.1 Elastic analysis. The design envelopes of bending moments and vertical shear forces which are produced by the whole of any particular combination of loads applied to the composite member, may be found by elastic analysis, assuming the concrete to be uncracked. The effects of shear lag may be neglected.

Alternatively, the stiffening effect of the concrete over 15 % of the length of the span on each side of each internal support may be neglected but tensile reinforcement may be taken into account.

6.1.4.2 Redistribution of moments in principal longitudinal members. Redistribution of the bending moments obtained from a particular combination of loads by either of the methods given in **6.1.4.1** may be carried out from the supports to the span by amounts not exceeding the values given in Table 10, as appropriate, provided that:

a) equilibrium between the internal forces and external loads is maintained under each appropriate combination of ultimate loads;

b) the ultimate moment of resistance provided at any section of a member should be not less than 70 % of the elastic moment obtained from **6.1.4.1** covering all appropriate combinations of loads nor less than the maximum moment obtained from the redistributed moment diagram, whichever is the greater;

c) premature failure of the steel compression flange by lateral-torsional buckling is prevented in accordance with Part 3;

d) proper account is taken of the effects of redistribution of longitudinal moments on cross members (if any) and their connections.

Table 10 — Maximum percentage redistribution of bending moments at the ultimate limit state

Slenderness of steel cross section[a]	Type of analysis used in 6.1.4.1	
	Cracked	Uncracked
Slender	0	10
Compact	20	30

[a] As defined in **6.2.2**.

6.1.4.3 *Plastic analysis.* Plastic analysis may be used to determine the distribution of bending moments and vertical shear forces in simply supported and continuous composite superstructures provided that:

a) all cross sections of the steel member at which, according to calculations, inelastic behaviour will occur are compact (as defined in **6.2.2.1**);

b) premature failure of the steel compression flange by lateral torsional buckling is prevented in accordance with Part 3;

c) the length of an end span in a continuous beam does not differ from that of an adjacent span by more than 15 % nor do the lengths of two adjacent interior spans differ by more than 25 %;

d) the concrete slab is of normal density concrete having a characteristic strength within the range 20 N/mm^2 to 45 N/mm^2;

e) not more than half the design ultimate load for any span is concentrated within a length l/5, where l is the effective span.

6.1.5 *Temperature effects and shrinkage modified by creep.* When the steel section is slender (as defined in **6.2.2.2**), the effects of temperature and shrinkage modified by creep on the longitudinal stresses in the composite section need only be considered at the ultimate limit state except as directed in **6.2.4.1** b). The methods given in **5.4.2** and **5.4.3** may be used but the partial factors of safety should be appropriate to the ultimate limit state.

No account need be taken of the effects of temperature and shrinkage modified by creep in the design of the shear connectors at the ultimate limit state but the longitudinal shear forces arising from these effects should be considered in the design of the transverse reinforcement (see **6.3.1** and **6.3.3**).

6.2 Analysis of sections

6.2.1 *General.* The strength of composite sections should be assessed by inelastic or elastic analysis, as appropriate, in accordance with **6.2.2** to **6.2.4**.

6.2.2 *Definitions*

6.2.2.1 *Compact cross sections.* Cross sections may be considered as compact when the web and compression flange possess sufficient stiffness to enable full plasticity and adequate rotation to be developed without loss of strength due to local buckling. This may be considered to have been achieved when the slenderness of all steel plates or sections that contribute to the strength of the web and compression flange is less than the relevant limiting values for compact sections as given in Part 3

6.2.2.2 *Slender cross sections.* Slender cross sections are those which do not satisfy the definition of compact given in **6.2.2.1**.

6.2.3 *Analysis of compact cross sections*

6.2.3.1 *Ultimate moment of resistance.* The ultimate moment of resistance of a compact cross section in both positive (sagging) or negative (hogging) bending may be determined by simple plastic theory making the following assumptions:

a) the effective breadth of the concrete flange b_e, on either side of a web, may be taken as l/6 but not greater than the actual breadth b, where l is the span of the composite beam;

b) subject to the provision of e) the whole of the area of the steel member and of the longitudinal reinforcement within the effective breadth of the concrete flange is stressed to the design yield strength in tension or compression, i.e. nominal yield strength/γ_m;

c) the strength of concrete on the tension side of the plastic neutral axis should be neglected;

d) the area of concrete on the compression side of the plastic neutral axis is stressed uniformly to its design compressive strength, which may be taken as 0.4 times the characteristic cube strength;

e) where necessary, allowance should be made in negative (hogging) moment regions for the influence of vertical shear on the ultimate moment of resistance as given in **6.2.3.3**.

6.2.3.2 *Vertical shear.* The design ultimate shear strength of a compact composite section in the absence of bending moment should be calculated in accordance with Part 3 neglecting any composite action in vertical shear.

6.2.3.3 *Influence of vertical shear on ultimate moment of resistance.* No reduction for the effects of vertical shear need be made in calculating the ultimate moment of resistance of a compact composite section:

a) if the vertical shear at the ultimate limit state is less than 30 % of the design ultimate shear strength as given in **6.2.3.2**,

b) in negative (hogging) moment regions, if the vertical shear does not exceed the design ultimate shear strength as given in **6.2.3.2**, and the cross-sectional area of the longitudinal reinforcement within the effective breadth of the slab exceeds 0.15 times the total cross-sectional area of the steel member and the design yield stress for this reinforcement is not less than that for the steel member.

Where neither condition is satisfied, it may be assumed in calculating the ultimate moment of resistance that the longitudinal stresses in the steel section are distributed in any convenient manner that, in combination with the shear stresses, gives equivalent stresses that nowhere exceed the yield stress as defined in Part 3.

6.2.4 ***Analysis of slender cross sections***

6.2.4.1 *Ultimate moment of resistance*

a) The ultimate moment of resistance of slender cross sections (as defined in **6.2.2.2**) may be calculated by elastic theory using an effective breadth of flange, determined in accordance with **5.2.3**, and the partial safety factors on material strength for the ultimate limit state. The elastic moduli of steel and concrete should be those recommended for the serviceability limit state.

The tensile strength of the concrete should normally be neglected but consideration should be given to the possibility of premature buckling of a steel compression flange if concrete in a tension flange remains uncracked.

Except where permitted by b) below, the stresses in structural steelwork should nowhere exceed those specified for the ultimate limit state in Part 3.

The stresses in reinforcement bars should not exceed the design yield strength at the ultimate limit state given in Part 4.

The compressive stress in the concrete due to flexure should nowhere exceed 0.53 times the characteristic cube strength.

b) In negative moment regions of unpropped continuous composite beams yielding of the steel tension flange may be permitted provided:

1) the ultimate strength of the cross section is determined from an elastic/plastic analysis with equilibrium maintained between the internal forces and external loads under each combination of loads;

2) redistribution of stresses is carried out in accordance with the recommendations of Part 3;

3) the shear connectors and transverse reinforcement are designed in accordance with **6.3** for the corresponding longitudinal shear;

4) the amount of redistribution of bending moments at the ultimate limit state does not exceed the amount given in Table 10 for slender cross sections;

5) temperature and shrinkage effects are considered at both the serviceability and ultimate limit states and the requirements of **4.3.2** are satisfied;

6) account is taken of the method of construction.

c) In simply supported beams the ultimate moment of resistance of the composite section may be determined by the method given in **6.2.3.1** provided the plastic neutral axis of the composite section lies within the concrete slab or the compression flange of the steel section. The effects of temperature, shrinkage and creep need not then be considered in calculating the ultimate bending strength but account should be taken of the longitudinal shear force due to these effects as required by **6.1.5**.

6.2.4.2 *Vertical shear.* Vertical shear should be assumed to be resisted by the steel section alone. Nowhere should the stress exceed the stress limitations for the ultimate limit state given in Part 3.

6.3 Longitudinal shear

6.3.1 ***General.*** Longitudinal shear per unit length of the composite beam q should be determined in accordance with **5.3.1** but using the design loadings appropriate to the ultimate limit state.

6.3.2 ***Deck slab.*** The deck slab and its reinforcement should be designed to resist the forces imposed on it by the shear connectors without excessive slip or separation and without longitudinal splitting, local crushing or bursting. Particular care should be taken where there is a free concrete surface adjacent to a connector, e.g. at an end or a side of a slab or in a haunch.

NOTE Designs in accordance with **6.3.1** to **6.3.3** satisfy these recommendations for the ultimate limit state and may be deemed to satisfy the fatigue and serviceability recommendations for transverse reinforcement. Designs which satisfy the fatigue and serviceability recommendations for shear connectors given in **5.3** may be deemed to satisfy the recommendations for the shear connectors for the ultimate limit state. Special consideration should be given to details which are not in accordance with **5.3** and **6.3.1** to **6.3.3**.

6.3.2.1 *Haunches.* Where concrete haunches are used between the steel flange and the soffit of the concrete slab the sides of the haunch should lie outside a line drawn at 45 ° from the outside edge of the connectors as shown in Figure 3. The recommendations of **5.3** and **6.3** to **6.3.3.7** inclusive should also apply.

6.3.3 ***Transverse reinforcement***

6.3.3.1 *Definitions and general requirements*

a) The design method given in **6.3.3.2** to **6.3.3.5** is applicable to haunched and unhaunched composite beams of normal density concrete or lightweight aggregate concrete. The method takes account of interaction between longitudinal shear and transverse bending of the slab.

Attention is drawn to the difference between the meaning of the symbols q and q_p:

q is the total longitudinal shear force per unit length of composite beam at the steel/concrete interface, determined in accordance with **6.3.1**.

q_p is the design longitudinal shear force per unit length of beam on the particular shear plane considered. It may be equal to or different from q, depending on the shear plane

b) Only reinforcement transverse to the steel beam that is fully anchored on both sides of a possible plane of longitudinal shear failure (shear plane) should be included in the definitions given below. Cross-sectional areas of transverse reinforcement per unit length of beam are defined thus:

A_t is reinforcement placed near the top of the slab forming the flange of the composite beam and may include that provided for flexure

A_b is reinforcement placed in the bottom of the slab or haunch at a clear distance not greater than 50 mm from the nearest surface of the steel beam, and at a clear distance of not less than 40 mm below that surface of each shear connector that resists uplift forces, including that bottom reinforcement provided for flexure

A_{bs} is other reinforcement in the bottom of the slab placed at a clear distance greater than 50 mm from the nearest surface of the steel beam

A_{bv} is reinforcement placed in the bottom of the slab or haunch, but excluding that provided for flexure, which complies in all other respects with the definition of A_b above

NOTE Where the depth of a haunch does not exceed 50 mm, reinforcement in the bottom of a slab may be included in the definitions of A_b and A_{bv} provided that it is placed at a clear distance of not less than 40 mm below that surface of each shear connector which resists uplift forces and at a clear distance not greater than 80 mm from the nearest surface of the steel beam.

Examples of five types of shear plane are given in Figure 6 with typical arrangements of reinforcement that satisfy the definitions of A_b, A_t and A_{bs} given above.

A_e is the reinforcement crossing a shear plane that is assumed to be effective in resisting shear failure along that plane.

For planes in unhaunched beams that do not cross the whole thickness of the slab (plane 2–2 in Figure 6) $A_e = 2A_b$.

For planes that cross the whole depth of the slab (shear plane type 1–1 in Figure 6) A_e is the total area of fully anchored reinforcement intersected by that plane, including reinforcement provided for flexure, e.g. in shear plane 1–1 in Figure 6(a) $A_e = A_t + A_b$.

For planes in haunched beams that do not cross the whole depth of the slab (shear plane types 3–3, 4–4, or 5–5 in Figure 6) A_e is the total area of fully anchored reinforcement intersected by that plane, which is placed at a clear distance of not less than 40 mm below that surface of each shear connector that resists uplift forces and may include the area of the hoop in a bar and hoop connector where appropriate.

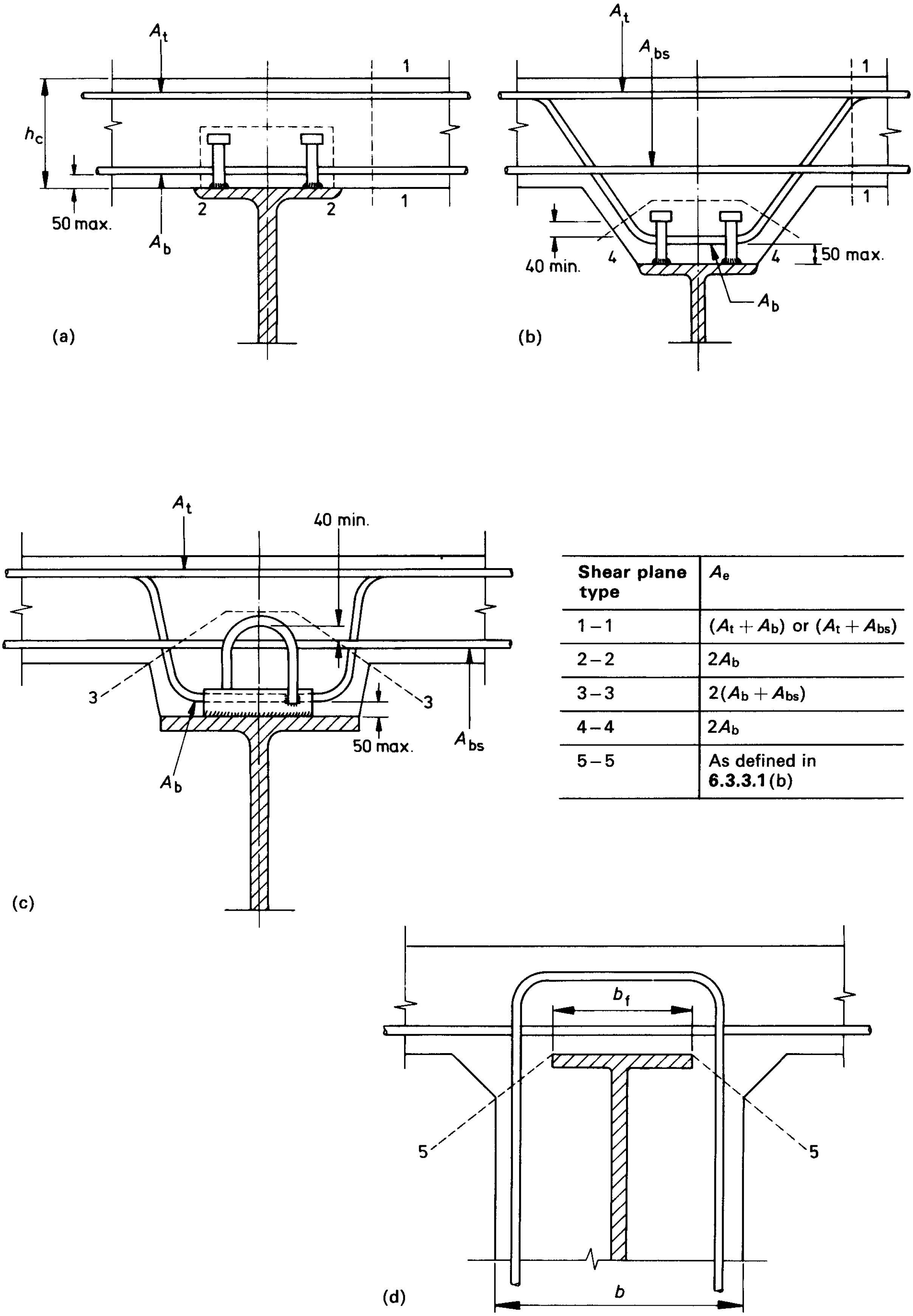
For planes of type 5–5 [see Figure 6(d)] in cased beams A_e is the total cross-sectional area of stirrups (both legs) crossing the shear plane (see **8.5.2** and **8.8**).

c) L_s is the length of the shear plane under consideration

f_{ry} is the characteristic yield strength of the transverse reinforcement but not greater than 460 N/mm^2

f_{cu} is the characteristic cube strength of concrete or the cube strength used in the design of the slab, if account is taken of loading at ages other than 28 days, but not greater than 45 N/mm^2

s is a constant stress of 1 N/mm^2 re-expressed where necessary in units consistent with those used for the other quantities



Shear plane type	A_e
1 – 1	$(A_t + A_b)$ or $(A_t + A_{bs})$
2 – 2	$2A_b$
3 – 3	$2(A_b + A_{bs})$
4 – 4	$2A_b$
5 – 5	As defined in **6.3.3.1** (b)

NOTE For shear plane type 5–5 L_s = total length of shear plane minus one third b_f.

Figure 6 — Shear planes and transverse reinforcement

d) the size and spacing of transverse reinforcement at each end of each span should be not less than that required for the maximum loading considered. This size and spacing should be maintained for at least 10 % of the length of each span. Elsewhere, the size and spacing may be kept constant over any length where, under the maximum loading considered, the maximum shear force per unit length does not exceed the design value over that length by more than 10 %.

6.3.3.2 *Longitudinal shear.* The longitudinal shear force per unit length q_p on any shear plane through the concrete should not exceed either:

a) $k_1 s L_s + 0.7 A_e f_{ry}$ or

b) $k_2 L_s f_{cu}$

where

k_1 is a constant equal to 0.9 for normal density concrete and 0.7 for lightweight aggregate concrete

k_2 is a constant equal to 0.15 for normal density concrete and 0.12 for lightweight aggregate concrete

If f_{cu} is taken to be less than 20 N/mm^2, the term $k_1 s L_s$ in a) should be replaced by $k_3 f_{cu} L_s$ where k_3 is a constant equal to 0.04 for normal density concrete and 0.03 for lightweight aggregate concrete.

In haunched beams, not less than half the reinforcement required to satisfy a) above in respect of shear planes through the haunch (planes 3-3 and 4–4 in Figure 6), should be bottom reinforcement that complies with the definition of A_{bv} in **6.3.3.1** b).

6.3.3.3 *Interaction between longitudinal shear and transverse bending*

a) *Beams with shear planes passing through the full depth of slab.* Where the shear plane passes through the full depth of the slab no account need be taken of the interaction between longitudinal shear and transverse bending.

b) *Unhaunched beams with shear planes passing round the connectors.* In unhaunched beams where the design loading at the ultimate limit state causes transverse tension in the slab in the region of the shear connectors, account should be taken of the effect of this on the strength of shear planes that do not cross the whole depth of the slab (plane 2–2 in Figure 6) by replacing **6.3.3.2** a) by

$q_p \leqslant k_1 s L_s + 1.4 A_{bv} f_{ry}$

Where the design loads at the ultimate limit state can cause transverse compression in the slab in the region of the shear connectors account may be taken of the beneficial effect of this on the strength of shear planes that do not cross the whole depth of the slab (shear plane type 2–2 in Figure 6), by replacing **6.3.2** a) by

$q_p \leqslant k_1 s L_s + 0.7 A_e f_{ry} + 1.6 F_T$

where

F_T is the minimum tensile force per unit length of beam in the transverse reinforcement in the top of the slab due to transverse bending of the slab. Only loading that is of a permanent nature should be considered when calculating F_T.

NOTE For remaining symbols see **6.3.3.1** a), b) and c).

c) *Haunched beams.* In haunched beams, where the design loading at the ultimate limit state causes transverse tension in the slab in the vicinity of the shear connectors, no account of this need be taken, provided the reinforcement required to satisfy **6.3.3.3** a) is reinforcement that satisfies the definition of A_{bv} and the haunch dimensions satisfy the recommendations of **6.3.2.1**.

Where the design loading at the ultimate limit state causes transverse compression in the region of the shear connectors, no account need be taken providing the recommendations of **6.3.3.2** are satisfied.

6.3.3.4 *Minimum transverse reinforcement.* The cross-sectional area, per unit length of beam, of reinforcement in the slab transverse to the steel beam should be not less than

$0.8\, s h_c / f_{ry}$

where

h_c is the thickness of the concrete slab forming the flange of the composite beam

Not less than 50 % of this area of reinforcement should be placed near the bottom of the slab so that it satisfies the definition of A_{bv} given in **6.3.3.1** b).

Where the length of a possible plane of shear failure around the connectors (shear plane 2–2 in Figure 6) is less than or equal to twice the thickness of the slab h_c reinforcement in addition to that required for flexure should be provided in the bottom of the slab transverse to the steel beam to prevent longitudinal splitting around the connectors. The cross-sectional area of this additional reinforcement, per unit length of beam, A_{bv} should be not less than $0.8\, s h_c / f_{ry}$. This additional reinforcement need not be provided if the minimum compressive force per unit length of beam, acting normal to and over the surface of the shear plane, is greater than $1.4\, s h_c$.

6.3.3.5 *Minimum transverse reinforcement in haunched beams.* The cross-sectional area of transverse reinforcement in a haunch per unit length of beam A_{bv} as defined in **6.3.3.1** b) should be not less than

$0.4\ sL_s/f_{ry}$

where

L_s is the length of a possible plane of shear failure around the connectors (see shear plane type 3–3 or 4–4 in Figure 6).

6.3.3.6 *Curtailment of transverse reinforcement.* The transverse reinforcement provided to resist longitudinal shear may be curtailed provided the recommendations of **6.3.3** are satisfied in all respects for the shear planes through the slab of type 1–1 in Figure 6. For this purpose the longitudinal shear force per unit length q_p for such a plane, may be assumed to vary linearly from the calculated maximum force on the relevant plane, which is adjacent to the shear connectors, to zero mid-way between the centre-line of the beam and that of an adjacent beam or to zero at an adjacent free edge.

6.3.3.7 *Detailing of transverse reinforcement.* The spacing of bottom transverse reinforcing bars, if provided to satisfy the recommendations of **6.3.3**, should be not greater than four times the projection of the connectors (including any hoop which is an integral part of the connector) above the bars nor greater than 600 mm.

6.3.4 ***Shear connectors.*** The design of the shear connectors need not be considered at the ultimate limit state except as directed in **5.3.3.4**, **6.1.3**, and **6.2.4.1** b) 3). Then the size and spacing of shear connectors should be determined in accordance with **5.3.3.6** except that longitudinal shear per unit length should be determined in accordance with **6.3.1** and the design static strength, per connector at the ultimate limit state, should be taken as

$0.8\ P_u/\gamma_m$

where

P_u is the nominal strength (as defined in **5.3.2.1**) and

$\gamma_m = 1.10$ or

$0.8\ P_u'/\gamma_m$ (for design in accordance with **5.3.3.4**).

7 Composite box girders

7.1 General. The design of composite box girders should satisfy the relevant recommendations for steel box girders given in Part 3 together with the recommendations for uncased beams given in this Part of this British Standard and also those given in this clause.

7.2 Effective span. The effective spans for bending of longitudinal or transverse box girders should be as defined in Part 1.

7.3 Effective breadth. The effective breadth of concrete flange should be determined in accordance with clauses **5** or **6** as appropriate. For closed box girders when the steel top flange, which is continuous between webs, acts compositely with the concrete deck slab the effective breadth of the composite plate may also be determined in accordance with clauses **5** or **6** as appropriate.

7.4 Distribution of bending moments and vertical shear forces. In the absence of more exact analysis the distribution of longitudinal bending moments and vertical shear forces may be calculated in accordance with **5.1.1** or **6.1** as appropriate.

7.5 Longitudinal shear

7.5.1 ***Spacing of shear connectors.*** The concrete slab should be positively tied down to the top steel flange plate in accordance with the requirements of **5.3.3.3** and **5.3.3.4**.

In closed box girders, shear connectors should be provided over the whole area of the top flange plate at spacings longitudinally and transversely not greater than 600 mm or three times the thickness of the concrete slab or four times the height of the connector (including any hoop which is an integral part of the connector), whichever is the least. The longitudinal spacing of these shear connectors should not exceed twenty-five times, and the transverse spacing should not exceed forty times, the thickness of the top flange plate.

Alternatively, connectors may be placed in groups with the spacing greater than that specified for individual connectors, provided consideration is given to the non-uniform flow of longitudinal shear and to the greater possibility of slip and vertical separation between the slab and the steel member and of buckling of the flange plate between connectors.

The distance from the edge of the top flange plate to the near edge of the nearest row of shear connectors should not exceed twelve times the thickness of the plate.

7.5.2 *Design of shear connectors.* The shear connectors in box girders should be designed in accordance with clause **5** for the serviceability limit state, except that in closed box girders the number of shear connectors required to satisfy **5.3.3.5** and **5.3.3.6** and their distribution over the breadth of the steel flange plate should be determined as follows.

NOTE 1 The connectors at any cross section are assumed to be all of the same type and size. The design of the shear connectors between each steel web and its associated concrete flange should be considered for each web separately.

The longitudinal shear force Q_x on a connector at distance x from the web centre line should be determined from

$$Q_x = \frac{q}{n}\left[K\left(1-\frac{x}{b_w}\right)^2 + 0.15\right]$$

where

q is the design longitudinal shear due to global and local loadings per unit length of girder at the serviceability limit state for the web considered, calculated assuming full interaction between the steel plate and the concrete slab (in accordance with **5.3.1**).

K is a coefficient determined from Figure 7

b_w is equal to half the distance between the centre lines of adjacent webs, or, for portions projecting beyond an outer web, the distance from the centre line of the web to the free edge of the steel flange.

NOTE 2 The force on any connector due to coexistent global and local loadings should not exceed its design strength at the serviceability limit state determined from clause **5**.

NOTE 3 In Figure 7 n' is the number of connectors per unit length placed within 200 mm of the centre line of the web considered and n is the total number of connectors per unit length of girder within breadth b_w, including any provided in accordance with **7.5.1** or **7.7** a).

If the connector density (number of shear connectors per unit area of steel flange) in any area outside the effective breadth of the steel flange exceeds the least density within the effective breadth at the cross section considered, the connectors additional to those that would give equal densities should be omitted when calculating n in this design method.

NOTE This method is not applicable when connectors are placed in groups or when the number of connectors in any transverse row across the flange is small.

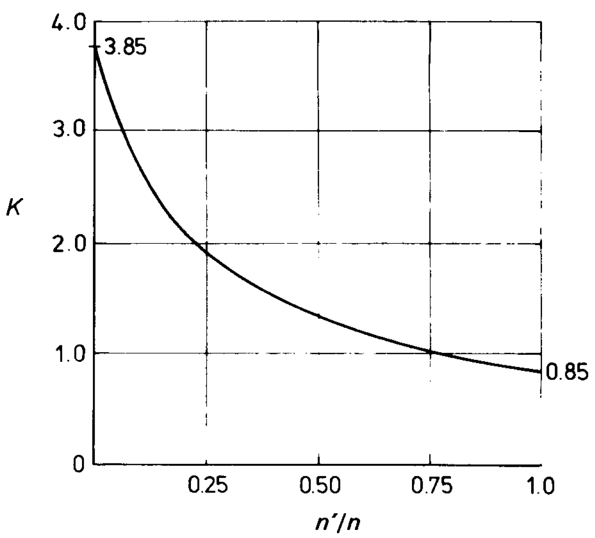


Figure 7 — Coefficient *K*

7.6 Torsion. In open box girders with no steel top flange continuous between webs consideration should be given to the effect of cracking of the concrete flange in negative (hogging) moment regions on the torsional rigidity of the box girder and on the distribution of torsional shear forces.

7.7 Composite plate. Where the concrete deck slab is cast on the top steel flange plate of a closed box girder the plate and the concrete slab, including the reinforcement, may be considered as acting compositely in resisting longitudinal and transverse effects of loading on the deck, provided that:

a) adequate shear connectors are provided to transmit the resulting shear force at the interface, ignoring the effect of bond;

b) adequate ties are provided in accordance with **5.3.3.3** and **5.3.3.4** to prevent separation of the two elements;

c) the combination of coexistent effects is taken into consideration, as required by **5.2.4.1** and **6.1.2**, together with the effects caused by the weight of wet concrete acting on the steel flange plate alone during construction. Consideration should be given to the effects of temporary construction loading in accordance with **9.4**.

Where these conditions are not satisfied the deck slab and the steel top flange plate should be designed as non-composite elements in accordance with Part 3 or Part 4 as appropriate. Proper account should be taken of the additional shear forces due to transverse bending of the deck and the effects of local wheel loading that may be imposed on the shear connectors provided to resist longitudinal shear in accordance with **7.5**.

8 Cased beams and filler beam construction

8.1 Scope. This clause applies to simply supported filler beam decks, with or without the soffit of the tension flange of the steel member exposed, and to simply supported or continuous cased beams. The recommendations apply only where the encasement or filling is of normal density concrete (2 300 kg/m^3 or greater).

8.2 Limit state requirements. Except where special requirements are given in the following clauses cased beams and filler beams decks should be designed for the serviceability and ultimate limit states in accordance with clauses **4**, **5** and **6**.

8.3 Analysis of structure. The distributions of bending moments and vertical shear forces, due to the design loadings at the serviceability and ultimate limit states, should be determined by an elastic analysis in accordance with **5.1** and **6.1**. Redistribution of moments at the ultimate limit state (see **6.1.4.2**) should not be permitted in cased beams.

In simply supported filler beam decks transverse bending moments may be determined by a distribution analysis of the the deck as an orthotropic plate or by the method given in **8.3.1**.

8.3.1 ***Transverse moments in filler beam decks (approximate method).*** This method is applicable to filler beam decks subject to standard highway loading type HA and/or up to 45 units of type HB loading where the following conditions are satisfied:

a) the construction consists of simply supported steel beams solidly encased in normal density concrete;

b) the span in the direction of the beams is not less than 6 m and not greater than 18 m and the angle of skew does not exceed 20°;

c) the clear spacing between the tips of the flanges of the steel beams does not exceed two-thirds of their depth;

d) the overall breadth of the deck does not exceed 14 m;

e) the amount of transverse reinforcement provided in the top of the slab is not less than 300 mm^2/m if mild steel is used or 200 mm^2/m if high yield steel is used.

The maximum design transverse sagging moment per unit length of deck M_y due to either HA or HB loading, at any point not less than 2 m from a free edge, may be taken as

$$M_y = (0.95 - 0.04l)\, M_x \alpha_L$$

where

M_x is the longitudinal bending moment per unit width of deck at the point considered due to the design HA loading for the limit state considered

l is the span in metres

α_L is the ratio of the product of the partial safety factors

$\gamma_{fL}\ \gamma_{f3}$ for HB loading to the corresponding product for HA loading for the limit state being considered

Longitudinal bending moments per unit width of deck due to HA loading may be found by analysis of the deck as a set of separate longitudinal strips each of width not exceeding the width of one traffic lane.

It may be assumed that there is a linear reduction in M_y from the value at 2 m from the free edge of the deck to zero at the edge.

The transverse hogging moment at any point may be taken as 0.1 M_y per unit length of deck.

8.4 Analysis of sections

8.4.1 ***Serviceability limit state.*** Longitudinal bending stresses in the steel and concrete should be determined by an elastic analysis in accordance with **5.2** except that the effects of shear lag may be neglected in calculating the stresses or deflections in filler beam decks. Vertical shear should be assumed to be resisted by the steel section alone.

8.4.2 ***Ultimate limit state.*** Longitudinal bending stresses in the steel and concrete at the ultimate limit state should be determined by an elastic analysis in accordance with **6.2.4.1**.

The effects of shear lag may be neglected in filler beam decks.

8.5 Longitudinal shear

8.5.1 ***Serviceability limit state.*** The longitudinal shear force per unit length between the concrete and steel beam should be calculated by elastic theory, in accordance with **5.3.1** except that, in positive (sagging) moment regions of cased beams and in filler beams, concrete in tension should be neglected. Shear lag effects may be neglected in filler beam decks. The shear force to be transferred should be that appropriate to the area of concrete and steel reinforcement in compression.

For highway bridges and footbridges the longitudinal shear force, other than that due to temperature and shrinkage effects, may be assumed to be resisted by bond between the steel and concrete provided the local bond stress nowhere exceeds 0.5 N/mm^2 in cased beams or 0.7 N/mm^2 in filler beams. The bond may be assumed to be developed uniformly only over both sides of the web and the upper surfaces of the top and bottom flanges of the steel beam, where there is complete encasement, and over both sides of the web and the upper surface of the top flange of the steel beam where the beam soffit is exposed. Where the local bond stress, calculated in the manner described, exceeds 0.5 N/mm^2 in cased beams or 0.7 N/mm^2 in filler beams the bond should be ignored entirely and shear connectors provided, in accordance with **5.3.2** and **5.3.3**, to transmit the whole of the longitudinal shear.

8.5.2 ***Ultimate limit state.*** The longitudinal shear force per unit length of beam should be calculated in accordance with **8.5.1** but for the design loading at the ultimate limit state. In cased beams other than filler beams, where shear connectors are not provided to transmit the longitudinal shear force due to vertical loading (see **8.5.1**), particular attention should be given to shear planes of type 5–5 [Figure 6(d)]. The total cross-sectional area per unit length of beam of fully anchored reinforcement intersecting the shear surface A_e, should be not less than

$$(q_p - k_1 s L_s)/0.7 f_{ry}$$

where

q_p is the longitudinal shear force per unit length at the ultimate limit state acting on that shear plane

L_s is the total length of shear plane minus one-third b_f

NOTE The remaining terms are as defined in **6.3.3**.

8.6 Temperature and shrinkage effects

8.6.1 ***General.*** Temperature and shrinkage effects need not be considered in filler beam construction. In cased beams, other than filler beams, consideration should be given to the effects of temperature and shrinkage at the serviceability limit state. In the absence of more precise information the effects of temperature in cased beams should be determined using the temperature effects given in Part 2 for a similar reinforced concrete structure. The effects of shrinkage as modified by creep should be assessed using the values of free shrinkage strain ϵ_{cs} and the reduction factor for creep ϕ_c as given in **5.4.3**.

8.6.2 ***Longitudinal stresses and strains.*** Longitudinal stresses and strains due to temperature effects and shrinkage modified by creep should be calculated in accordance with **5.4.2** and **5.4.3**.

8.6.3 ***Longitudinal shear.*** Shear connectors should be provided at the ends of cased beams, to transmit the longitudinal shear force Q, due to temperature effects and shrinkage modified by creep as described in **5.4.2.3** and **5.4.3**. The longitudinal shear force to be transmitted by the connectors should be the change in net longitudinal force in the steel beam due to temperature and shrinkage effects calculated on an elastic basis assuming full interaction. It may be assumed to be distributed at the ends of the beam in the manner described in **5.4.2.3**. The concrete should be assumed to be uncracked. The effective breadth of the concrete flange should be determined in accordance with **5.4.2.1** (see Table 8).

8.7 Control of cracking

8.7.1 ***General.*** Subject to the recommendations of **8.7.2** and **8.7.3**, the methods given in **5.2.6** may be used to ensure that cracking is not excessive at the serviceability limit state. Tensile reinforcement, provided to satisfy the recommendations of this clause, may be assumed to contribute to the section properties of the composite beam.

8.7.2 ***Cased beams***

Longitudinal bars placed in the side face of beams to control flexural cracking should be of a diameter Φ such that:

$$\Phi \nless \sqrt{s_b\, s \frac{b}{f_{ry}}}$$

s_b is the spacing of bars in the side face of the beam,

b is the breadth of the section at the point where the crack width is being considered,

s is a constant stress of 1 N/mm^2, re-expressed where necessary in units consistent with those used for other quantities,

f_{ry} is the characteristic yield stress of the reinforcement.

Where the overall depth of a cased beam exceeds 750 mm longitudinal bars at 250 mm spacing or closer should be provided in the side faces of the beam over a distance of two-thirds of the overall depth measured from the tension face, unless the calculation of crack widths (see Appendix B) shows that a greater spacing is acceptable.

The clear distance from the corner of a beam in the tension zone to the nearest longitudinal bar that is enclosed by a stirrup should not exceed half the clear distance between bars determined from Table 5 or Table 6 as appropriate.

8.7.3 ***Filler beams.*** The widths of cracks due to transverse bending of a filler beam deck should be determined in accordance with Part 4, as for a reinforced concrete slab, neglecting any contribution from the steel beams to the control of cracking.

8.8 Design and construction. The concrete cover to the steel beam should nowhere be less than 50 mm except that the underside of the bottom flanges of filler beams may be exposed. The soffit and upper surface of exposed flanges of filler beams should be protected against corrosion.

In cased beams, other than filler beams, stirrups formed by reinforcing bars should enclose the steel beam and the reinforcement provided for control of cracking of the beam encasement. The spacing of stirrups in cased beams should not exceed 600 mm. The total cross-sectional area of stirrups (both legs) crossing a possible plane of shear failure of type 5α5 [Figure 6(d)], should be not less than

$0.8\ sL_s/f_{ry}$ per unit length of beam

where

L_s is as defined in Figure 6(d)

s is defined in **6.3.3.1**.

NOTE Alternatively, mesh of equivalent area may be used.

Concrete cover to reinforcement should be in accordance with the recommendations of Part 4.

9 Permanent formwork

9.1 General. The recommendations of this clause apply to formwork for in situ concrete, generally supported from the steelwork, which becomes part of the permanent construction. Where the steel plate forming the top flange of a closed box girder acts as permanent formwork to the concrete deck slab separate recommendations are given in **7.7**.

Special attention should be given to the provision of a suitable seal between the steelwork and the permanent formwork to minimize the possibility of corrosion throughout the life of the bridge. This seal should be placed along the edges of steelwork which have been previously painted.

9.2 Materials. Materials suitable for use as permanent formwork are as follows:

a) reinforced or prestressed precast concrete;

b) precast concrete acting compositely with a steel girder or lattice which is eventually embedded in the overlying in situ concrete;

c) profiled steel sheeting;

d) reinforced plastic or asbestos cement sheeting or similar.

Particular care should be exercised by construction staff and operatives to prevent accidents from occurring when materials of a fragile nature [e.g. those given in d)] are used as permanent formwork.

9.3 Structural participation. Permanent formwork may be considered as either:

a) structurally participating with the overlying in situ concrete slab under the action of loading imposed upon the slab after casting; or

b) structurally non-participating.

Permanent formwork made from the materials given in **9.2** d) should be considered as structurally non-participating.

9.4 Temporary construction loading. The design loads due to temporary construction loading should be determined in accordance with Part 2. Consideration should be given to the mounding of concrete that may occur during casting as well as the loads from construction plant and personnel.

9.5 Design

9.5.1 ***General.*** The permanent formwork should be capable of carrying the design loads due to temporary construction loading without failure or excessive deflection. The design should satisfy the relevant limit states given in Part 3 or Part 4 as appropriate.

9.5.2 ***Non-participating formwork.*** Where the permanent formwork is structurally non-participating account should be taken of any effects of differential shrinkage or composite action that may adversely affect the structure. Requirements for cover to reinforcement and crack control applicable to the in situ slab should be satisfied ignoring the presence of the formwork. Connection between the permanent formwork and the in situ concrete should be the minimum to prevent separation.

9.5.3 ***Participating formwork.*** Where composite action between the permanent formwork and in situ slab is relied upon the design of the composite slab should satisfy all relevant recommendations of this Part of this British Standard.

9.6 Precast concrete or composite precast concrete permanent formwork

9.6.1 ***Design.*** Precast concrete units should comply with the relevant clauses given in Part 4. Continuity between units may be provided by lapping reinforcement projecting from units by post-tensioning or by using high-strength bolts.

9.6.2 ***Welding of reinforcement.*** Welding of reinforcement should only be permitted when the effects of repeated loading can be shown not to be detrimental to the permanent structure. Design and construction in accordance with Parts 10 and 7 may be deemed to satisfy this recommendation.

9.6.3 ***Interfaces.*** Interfaces between precast and in situ concrete should develop sufficient shear resistance to ensure composite action in both the transverse and longitudinal directions.

9.6.4 ***Cover to reinforcement.*** The clear distance between a precast unit and reinforcement to be embedded in the in situ concrete slab should exceed the maximum nominal size of aggregate used in the in situ concrete by not less than 5 mm.

10 The use of friction grip bolts as shear connectors in composite beams

10.1 General. High strength friction grip bolts may be used to provide the shear connection between the steel member and the concrete slab forming the flange of the composite beam. The following method may be used for the design of the connection where general grade bolts complying with the requirements of BS 4395-1 or BS 3139-1 are used. The use of higher grades of bolts is not excluded where adequate tests have been carried out to determine the design criteria.

10.2 Design requirements: static loading

10.2.1 ***Serviceability limit state.*** The longitudinal shear resistance per unit length developed by friction between the concrete flange and steel beam should be not less than the longitudinal shear force per unit length at the serviceability limit state calculated in accordance with clause **5**. The design frictional resistance developed by each bolt at the interface should be taken as:

$$\frac{\mu \times \text{net tensile force in the bolt}}{1.2}$$

where

μ, the coefficient of friction at first slip, may be taken as 0.45 provided the recommendations of **10.4** are satisfied.

Where the concrete flange is cast in situ on the steel beam the value of μ may be increased to 0.50 at the discretion of the engineer. The nominal initial tensile force in the bolt may be taken as the proof load as given in BS 4604-1 or BS 3294, as appropriate, provided the method of tightening complies with the requirements of these British Standards. In determining the net tensile force in the bolt account should be taken of the loss of bolt tension due to shrinkage of the concrete and creep of the steel and concrete.

Where the connectors are subject to external tensile forces in addition to shear, e.g. where loads are suspended from the steelwork, account should be taken of the reduction in effective clamping force in the bolt.

10.2.2 ***Ultimate limit state.*** Designs in accordance with **10.2.1** may be deemed to satisfy the recommendations for the shear connectors at the ultimate limit state.

10.3 Fatigue. For connections subject only to shear in the plane of the friction interface no account need be taken of the effects of repeated loading.

10.4 Other considerations. The design of the connection should ensure that there is a uniform bearing surface between the steel beam and the concrete flange. Where the slab is precast it may be necessary to provide suitable bedding material between the slab and the steel beam. Except as required by **9.1** the interface should be free of paint or other applied finishes, oil, dirt, loose rust, loose mill scale, burrs and other defects which would prevent a uniform seating between the two elements or would interfere with the development of friction between them. Tight mill scale is not detrimental.

Adequate reinforcement, usually in the form of spirals, should be provided to ensure that the load is transferred from the bolt to the interface without local splitting or crushing of the concrete.

11 Composite columns

11.1 General

11.1.1 ***Scope.*** This clause gives a design method for concrete encased steel sections and concrete filled circular and rectangular hollow steel sections which takes account of the composite action between the various elements forming the cross section. Bending about the two principal axes of the column is considered separately for each axis. A method is given in **11.3.6** for determining the effect of interaction when bending about both axes occurs simultaneously. The column may be either statically determinate or rigidly connected to other members at one or both ends in which case the loads and moments depend on the relative stiffnesses of adjoining members and cannot be obtained by statics alone. Members may be assumed to be rigidly connected where, for example, the connection possesses the full rigidity that can be made possible by welding or by the use of high-strength friction grip bolts.

11.1.2 ***Materials***

11.1.2.1 *Steel.* In columns formed from concrete encased steel sections the structural steel section should be either:

a) a rolled steel joist or universal section of grade 43 or 50 steel which complies with the requirements of BS 4-1; or

b) a symmetrical I-section fabricated from grade 43 or 50 steel.

Concrete filled hollow steel sections may be either rectangular or circular and should either:

1) be a symmetrical box section fabricated from grade 43 or 50 steel; or

2) comply with BS 4-2 or BS 4848-2, as appropriate; and

3) have a wall thickness of not less than:

$b_s \sqrt{f_y / 3E_s}$	for each wall in a rectangular hollow section (RHS), or
$D_e \sqrt{f_y / 8E_s}$	for circular hollow sections (CHS)

where

b_s	is the external dimension of the wall of the RHS
D_e	is the outside diameter of the CHS
E_s	is Young's modulus of elasticity of steel
f_y	is the nominal yield strength of the steel

The surface of the steel member in contact with the concrete filling or encasement should be unpainted and free from deposits of oil, grease and loose scale or rust.

11.1.2.2 *Concrete.* The concrete should be of normal density (not less than 2 300 kg/m^3) with a characteristic 28 day cube strength of not less than 20 N/mm^2 for concrete filled tubes nor less than 25 N/mm^2 for concrete encased sections and a nominal maximum size of aggregate not exceeding 20 mm.

11.1.2.3 *Reinforcement.* Steel reinforcement should comply with the relevant clauses on strength of materials given in Part 4.

11.1.3 ***Shear connection.*** Provision should be made for loads applied to the composite column to be distributed between the steel and concrete elements in such proportions that the shear stresses at the steel/concrete interface are nowhere excessive. It is recommended that shear connectors should be provided where this shear stress, due to the design ultimate loads, would otherwise exceed 0.6 N/mm^2 for cased sections or 0.4 N/mm^2 for concrete filled hollow steel sections.

11.1.4 ***Concrete contribution factor.*** The method of analysis in **11.3** is restricted to composite cross sections where the concrete contribution factor α_c, as given below, lies between the following limits:

for concrete encased steel sections	$0.15 < \alpha_c < 0.8$
for concrete filled hollow steel sections	$0.1 < \alpha_c < 0.8$

where

$$\alpha_c = \frac{0.45\, A_c\, f_{cu}}{N_u}$$

and the squash load N_u is given by:

$$N_u = 0.91\, A_s f_y + 0.87\, A_r f_{ry} + 0.45\, A_c f_{cu}$$

except that for concrete filled circular hollow steel sections α_c and N_u should be determined in accordance with **11.3.7**.

A_s	is the cross-sectional area of the rolled or fabricated structural steel section
A_r	is the cross-sectional area of reinforcement
A_c	is the area of concrete in the cross section
f_y	is the nominal yield strength of the structural steel
f_{ry}	is the characteristic yield strength of the reinforcement
f_{cu}	is the characteristic 28 day cube strength of the concrete.

11.1.5 ***Limits on slenderness.*** The ratio of the effective length, determined in accordance with **11.2.2.4** to the least lateral dimension of the composite column, should not exceed:

a) 30 for concrete encased sections; or

b) 55 for concrete filled circular hollow sections; or

c) 65 for concrete filled rectangular hollow sections.

11.2 Moments and forces in columns

11.2.1 ***General.*** The loads and moments acting in the two principal planes of the column, due to loading at the ultimate limit state, should be determined by an appropriate analysis in which the actual length of the column is taken as the distance between the centres of end restraints. Proper account should be taken of the rotational and directional restraint afforded by adjoining members and the reduction in member stiffness due to inelasticity and axial compression. Alternatively, the method given in **11.2.2** may be used.

11.2.2 ***Semi-empirical design method for restrained composite columns***

11.2.2.1 *Scope.* The semi-empirical method of analysis given in **11.2.2.2** to **11.2.2.6** is only applicable to isolated columns or columns forming part of a single storey frame provided that the restraining members attached to the ends of the column remain elastic under their design ultimate load; otherwise the stiffness of the restraining members should be appropriately reduced in calculating the effective length of the column and the end moments.

11.2.2.2 *Moments and forces on the restrained column.* End moments and forces acting in the two principal planes of the column should be determined either by statics, where appropriate, or by an elastic analysis neglecting the effect of axial loads both on member stiffness and on changes in the geometry of the structure as it deflects under load. The relative stiffness of members (I/l) should be based on the gross (concrete assumed uncracked) transformed composite cross section using an appropriate modulus of elasticity determined from Part 4, with/taken as the distance between centres of end restraints.

11.2.2.3 *Equivalent pin-ended column.* The actual column should be replaced by an equivalent pin-ended column of length equal to the effective length of the restrained column in the plane of bending and subjected to the same end loads and end moments as the restrained column, except that where the column is free to sway, the equivalent pin-ended column should always be considered to be in single curvature bending with the smaller end moment in a particular plane taken as the calculated value or three-quarters of the larger end moment, whichever is greater. The strength of the equivalent pin-ended column should then be determined in accordance with **11.3**.

11.2.2.4 *Effective length.* For isolated columns with simple forms of end restraint the effective length may be determined from Table 11.

Table 11 — Effective length of columns

End restraint condition	Effective length
Effectively restrained against rotation and translation at both ends	0.7 l[a]
Effectively restrained against translation at both ends and restrained against rotation at one end	0.85 l[a]
Effectively restrained against translation at both ends but not restrained against end rotation	1.0 l[a]
Effectively restrained against translation and rotation at one end and at the other end imperfectly restrained against rotation and not restrained against translation	1.5 l[a]
Effectively restrained against rotation and translation at one end but not restrained against either rotation or translation at the other	2.0 l[a]

[a] l is the actual length of the column between centres of end restraints.

Where columns form part of a single storey frame account should be taken of the flexural stiffness of the restraining members framing into the ends of the column when calculating its effective length.

11.2.2.5 *Transverse loads.* Transverse loads should be included in the elastic analysis of the restrained column if this results in a more severe loading condition. In a braced frame (or column) when the maximum resultant moment within the length of the column M_{max}, due to the whole of the design ultimate loads, is greater than half the modulus of the algebraic sum of the end moments the alternative loading condition of single curvature bending should also be considered with the end moments equal to M_{max}. Single curvature bending is here assumed to produce end moments of the same sign at each end of the column.

11.2.2.6 *Column self weight.* The axial component of self weight may be considered as an additional end load acting concentrically on the column. In raking columns, account should also be taken of the bending moments in the column due to the normal component of its self weight.

11.3 Analysis of column cross section

11.3.1 ***General.*** For these calculations, the actual column should be replaced by a pin-ended column, of length equal to the effective length of the actual column in the plane of bending, using the methods given in **11.2**.

The x axis, also called the major axis, should be chosen so that the slenderness function λ_x is not greater than λ_y where in general:

$$\lambda = \frac{l_e}{l_E} \text{ and}$$

$$l_E = \pi \left[\frac{E_c I_c + E_s I_s + E_r I_r}{N_u} \right]^{\frac{1}{2}}$$

l_E is the length of column for which the Euler Load equals the squash load

l_e is the effective length of the actual column in the plane of bending considered; the suffices x and y denote values calculated for the major and minor axes respectively

E_c is the modulus of elasticity of concrete which, for the purpose of this clause, should be taken as 450 f_{cu}, where f_{cu} is the characteristic cube strength of the concrete

E_s, E_r are Young's moduli of elasticity for the structural steel and reinforcement, respectively

I_c, I_s, I_r are the second moments of area of the uncracked concrete cross section, the steel section, and the reinforcement respectively about the axis of the composition column, used with an additional subscript x or y to designate the appropriate plane of bending

N_u is the squash load obtained from **11.1.4** or **11.3.7**, as appropriate.

11.3.2 ***Axially loaded columns.*** In an axially loaded column, failure occurs by buckling about the minor axis due to initial imperfections in straightness of the steel member. In practice, end moments due solely to the load acting at an eccentricity may arise from construction tolerances. The design methods given in **11.3.2.1** to **11.3.7** for axially loaded columns therefore include an allowance for an eccentricity about the minor axis not exceeding 0.03 times the least lateral dimension of the composite column. Where this is inappropriate it may be increased at the discretion of the engineer and the failure load calculated in accordance with **11.3.3**.

11.3.2.1 *Short columns.* Where both the ratios l_x/h and l_y/b do not exceed 12 the axial load at the ultimate limit state N should not exceed the axial load at failure N_{ay} given by:

$$N_{ay} = 0.85\, K_{1y}\, N_u$$

where

K_{1y} is determined from **C.1** using the parameters appropriate to the minor axis

N_u is the squash load, obtained from **11.1.4** or **11.3.7**

h and b are the greatest and least lateral dimensions of concrete in the cross section of the composite column

The factor 0.85 is a reduction factor to allow for the moments due to construction tolerances, as given in **11.3.2**.

11.3.2.2 *Slender columns.* Where either of the ratios l_x/h or l_y/b exceeds 12 account should be taken of the eccentricity due to construction tolerances by considering the column in uniaxial bending about the minor axis. The load acting on the column N should be not greater than the load N_y calculated from **11.3.3** with the moment acting about the minor axis M_y taken as the moment produced by the applied load N, acting at an eccentricity of 0.03 b, where b is the least lateral dimension of the column.

11.3.3 ***Columns under uniaxial bending about the minor axis.*** Where the end moments about the major axis are nominally zero failure occurs by uniaxial bending about the minor axis. The column should be designed so that:

a) the design ultimate moment of resistance of the composite section about the minor axis M_{uy}, calculated in accordance with **C.4**, should be not less than the maximum applied design moment acting about the minor axis M_y. To allow for construction tolerances M_y should never be taken as less than the moment produced by the design load N acting at a constant eccentricity of 0.03b, where b is the least lateral dimension of the column.

b) the design load acting on the column N is not greater than N_y, which is given by:

$$N_y = N_u \left[K_{1y} - (K_{1y} - K_{2y} - 4K_3) \frac{M_y}{M_{uy}} - 4K_3 \left(\frac{M_y}{M_{uy}} \right)^2 \right]$$

where

N_y is the design failure load of a column subjected to a constant design moment M_y

K_{1y} and K_{2y} are determined from **C.1** and **C.2**, using the parameters appropriate to the minor axis.

K_3 is determined from **C.3**.

11.3.4 ***Columns under uniaxial bending about the major axis restrained from failure about the minor axis.*** Where the column is restrained from failure about the minor axis the column should be designed so that:

a) the design ultimate moment of resistance of the composite section about the major axis M_{ux}, determined in accordance with **C.4**, should be not less than the maximum applied design moment acting about the major axis M_x. M_x should be taken as not less than the moment produced by the design load N acting at a constant eccentricity of 0.03b, where b is the least lateral dimension of the column;

b) the design load acting on the column N is not greater than N_x, which is given by:

$$N_x = N_u \left[K_{1x} - (K_{1x} - K_{2x} - 4K_3)\frac{M_x}{M_{ux}} - 4K_3\left(\frac{M_x}{M_{ux}}\right)^2 \right]$$

where

N_x is the design failure load of a column subjected to a constant design moment M_x and the remaining notation is as in **11.3.3** except that the parameters should be calculated for the major axis.

NOTE Values of K_{1x} and K_{2x} are determined from **C.1** and **C.2**.

11.3.5 ***Columns under uniaxial bending about the major axis unrestrained against failure about the minor axis.*** Where the end moments about the minor axis are nominally zero and the column is unrestrained against failure about the minor axis the column is likely to fail in a biaxial mode unless the axial load is very small. The column should be designed so that:

a) the requirements of **11.3.4** a) are satisfied; and

b) the design load acting on the column N is not greater than the strength of the column in biaxial bending N_{xy} calculated from the equation given in **11.3.6** b) except that N_y should be calculated from **11.3.3** b) taking M_y as equal to $0.03Nb$ to allow for construction tolerances, where b is the least lateral dimension of the column.

11.3.6 ***Columns under biaxial bending.*** Where the end moments about both axes are non-zero failure occurs in a biaxial mode.

The column should be designed so that:

a) the maximum moment due to design loads at the ultimate limit state acting on each axis M_x or M_y is not greater than the design ultimate moment of resistance of the composite section about the major axis or the minor axis respectively; and

b) the design load acting on the column N is not greater than the ultimate strength of the column in biaxial bending N_{xy}, which is given by

$$\frac{1}{N_{xy}} = \frac{1}{N_x} + \frac{1}{N_y} - \frac{1}{N_{ax}}$$

where

N_x is determined in accordance with **11.3.4** b)

N_y is determined in accordance with **11.3.3** b) and

$N_{ax} = K_{1x}N_u$, where K_{1x} is determined from **C.1** using the parameters appropriate to the major axis of the column.

11.3.7 ***Ultimate strength of axially loaded concrete filled circular hollow sections.*** In axially loaded columns formed from concrete filled circular hollow steel sections account may be taken of the enhanced strength of triaxially contained concrete in the method given in **11.1.4** and **11.3.2** by replacing the expressions for α_c and N_u given in **11.1.4** by the following:

$$\alpha_c = \frac{0.45 f_{cc} A_c}{N_u}$$

$$N_u = 0.91 f'_y A_s + 0.45 f_{cc} A_c$$

where

f_{cc} is an enhanced characteristic strength of triaxially contained concrete under axial load, given by:

$$f_{cc} = f_{cu} + C_1 \frac{t}{D_e} f_y, \text{ and}$$

f'_y is a reduced nominal yield strength of the steel casing, given by:

$$f'_y = C_2 f_y$$

where

C_1 and C_2 are constants given in Table 12

D_e is the outside diameter of the tube

t is the wall thickness of the steel casing and the remaining symbols are defined as in **11.1** and **11.2.2.4**

Table 12 — Values of constants C_1 and C_2 for axially loaded concrete filled circular hollow sections

$\frac{l_e}{D_e}$	C_1	C_2
0	9.47	0.76
5	6.40	0.80
10	3.81	0.85
15	1.80	0.90
20	0.48	0.95
25	0	1.0

11.3.8 ***Tensile cracking of concrete.*** No check for crack control need be made in the following:

a) concrete filled hollow steel sections, or

b) concrete encased steel sections provided the design axial load at the ultimate limit state is greater than $0.2 f_{cu} A_c$, where the symbols are as defined in **11.1.4**.

Where the design axial load in concrete encased steel sections is less than the value given in b) and tensile stresses due to bending can occur in one or more faces of the composite section, the column should be considered as a beam for the purpose of crack control. Reinforcement should be provided in accordance with **5.2.6** using the bending moments appropriate to the serviceability limit state.

11.3.9 ***Design details.*** To prevent local spalling of the concrete, reinforcement should be provided in concrete encased sections. Stirrups of an appropriate diameter should be provided throughout the length of the column at a spacing not exceeding 200 mm with the provision of at least four longitudinal bars which are capable of supporting the reinforcing cage during concreting.

The concrete cover to the nearest surface of the steel member should be not less than 50 mm. Adequate clearance should be provided between the steel elements to ensure proper compaction of the concrete.

12 Influence of method of construction on design

12.1 Sequence of construction. The sequence of construction should be considered as an integral part of the design process, for example, when calculating the stresses or deflections in a composite section. The engineer should describe, on the drawings, the particular sequence or method of construction on which his design is based, including the position of any construction joints.

Where a partially cast slab is assumed to act compositely the shear connection should be designed for this condition as well as for the final condition.

Consideration should be given to the speed and sequence of concreting to prevent damage occurring to partly matured concrete as a result of limited composite action due to deformation of the steel beams under subsequent concreting operations.

It is recommended that, whenever possible, loading of the composite section should be delayed until the concrete has attained a cube strength of not less than 20 N/mm^2.

Where the composite section is loaded before the concrete has attained its 28 day characteristic cube strength the elastic properties and limiting compressive stresses of the concrete and the nominal strengths of shear connectors should be based upon f_c, the cube strength of the concrete at the time considered, except that no reduction in stiffness of the concrete need be made if

$$0.75\, f_{cu} < f_c < f_{cu}$$

Where the cube strength of the concrete at the time considered f_c, is not less than 20 N/mm^2, the nominal strengths of shear connectors may be determined by linear interpolation of the values given in Table 7.

12.2 Permanent formwork. Recommendations for temporary construction loading, which should be assumed in the design of permanent formwork, are given in **9.4**.

13 Prestressing in composite construction

13.1 General. Prestressing can reduce, or in some circumstances prevent, the cracking of concrete under service loading so increasing stiffness and improving the protection of steel from corrosion.

13.2 Methods of prestressing. Among the methods by which prestressing may be achieved are the following:

a) a system whereby a moment is applied to the steel section in the same direction as it will act in the structure. The tension flange is then encased in concrete and the moment relaxed when the concrete has adequate strength;

b) the use of jacking to alter the relative levels of the supports of a continuous member after part or the whole of the concrete deck has been cast and matured;

c) prestressing the concrete slab or sections of the slab by tendons or jacking whilst it is independent of the steel section and subsequently connecting them;

d) prestressing the steel beam by tendons prior to concreting. The tendons may or may not be released after the concrete has matured;

e) prestressing the composite sections by tendons or jacking.

Special consideration should be given to composite beams which are prestressed by an external system or by tendons not directly bonded to the concrete. In these circumstances, the calculation of prestressing forces should take account of the deformation of the whole structure.

13.3 Limit state requirements. Composite members which are prestressed should be designed for the serviceability and ultimate limit states in accordance with the general recommendations of this and other Parts of this British Standard.

13.4 Prestressing the steel beam. Consideration should be given to the stability of the steel beam during prestressing. The stresses in the steelwork should not exceed the limiting stresses given in Part 3.

13.5 Stress limitations in concrete at transfer. Stresses in the concrete at transfer should be calculated in accordance with **5.2**.

Where the concrete is precompressed by the release of a temporary prestress in the steel beam the compressive stress in the concrete at transfer, before losses, should, in general, not exceed 0.5 f_{ci}, where f_{ci} is the cube strength at transfer, but may be increased to 0.6 f_{ci} when the strain in the prestressing steel before transfer does not exceed 0.25 %.

Where the concrete slab or a section of the slab is permanently prestressed before it acts compositely with the steel beam the stresses in the concrete at transfer, in tension or compression should not exceed the limitations given in Part 4 for prestressed concrete.

Where the composite section is permanently prestressed the stresses in the concrete at transfer, in tension or compression should not exceed the limitations given in Part 4 for prestressed concrete.

13.6 Loss of prestress. The loss of prestress and the effects of shrinkage in non-composite prestressed concrete members should be calculated in accordance with the recommendations of Part 4.

Where the concrete acts compositely with the steel section account should be taken of the reduction in prestress and the effect on the stresses in the composite section due to elastic deformation, shrinkage and creep of the concrete and relaxation in the prestressing steel or tendon.

Appendix A Calculation of effective breadth ratios ψ

A.1 General

The methods given in this appendix may be used to determine the effective breadth ratios ψ for conditions not specifically covered by **5.2.3** 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 appendix is the same as that used in **5.2.3** except that l should be taken as the distance between adjacent points at which the bending moment is zero. Table 2 should be used to determine all effective breadth ratios ψ in conjunction with the methods given in this appendix.

A.2 Equivalent simply supported spans

In structures other than simply supported beams **A.3** and **A.4** may be used to determine the effective breadth ratios by treating each portion of a continuous beam between adjacent points of zero moment as an equivalent simply supported span. 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 not at midspan

For point loads at positions other than midspan the effective breadth ratio ψ,under the point of application of the load, may be determined from:

$$\psi = \frac{1}{3}\left(2\psi^{p}{}_{x} + \psi^{p}{}_{(l-x)}\right)$$

where

$\psi^{p}{}_{x}$ is the value of ψ from table for a point load at midspan with $l = 2x$

$\psi^{p}(l - x)$ is the value of ψ from Table 2 for a point load at midspan 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^{p} from Table 2 for a point load at midspan.

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^{p} given in Table 2 at quarterspan. Within a distance $l/4$ of the point load the effective breadth ratio ψ 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.4 Combination of loads

Under combinations of distributed and/or point loads the values of ψ may be derived from:

$$\psi = \frac{\Sigma M}{\frac{M_1}{\psi_1} + \frac{M_2}{\psi_2} \dots + \frac{M_n}{\psi_n}}\,{}^{*}$$

where

$M_1 \dots M_n$ are the bending moments at the cross section considered due to each component of load

M is the total bending moment at the same section due to load components 1 to n

$\psi_1 \dots \psi_n$ are the effective breadth ratios for the same section appropriate to each load component.

* In calculating the value of ψ due account should be taken of the algebraic sign of the bending moments.

Appendix B Calculation of crack widths in composite members

B.1 General

The bar spacing rules given in **5.2.6.4** ensure that cracking is not serious in the worst practical situation. It will almost invariably be found that wider bar spacings can be used if the crack widths are checked explicitly, particularly in shallow members.

The width of flexural cracks at a particular point on the surface of a member depends primarily on three factors:

a) the distance between the point being considered to the nearest reinforcing bar placed perpendicular to the plane of the crack;

b) the distance from the point being considered to the neutral axis;

c) the average surface strain at the point considered.

NOTE The formula in **B.2** gives acceptably accurate results in most normal design circumstances for members in flexure.

B.2 Formula for estimating crack widths due to flexure

Provided the stress in the tension reinforcement, calculated neglecting the tensile strength of the concrete, does not exceed 0.8 f_{ry}, the surface crack width, which should not exceed the appropriate limiting crack width in **5.2.6.3**, may be calculated from the following equation:

$$\text{surface crack width} = \frac{3\,a_{cr}\,\epsilon_m}{1 + 2\left[\dfrac{a_{cr} - c_{min}}{h - x}\right]}$$

where

c_{min} is the minimum cover to the tension reinforcement,

a_{cr} is the distance from the point considered to the surface of the nearest longitudinal reinforcing bar,

h is the overall depth of the composite member,

x is the depth to the neutral axis of the composite section from the extreme compression fibre, used in the analysis to determine ϵ_1

ϵ_m is the average longitudinal strain at the level where the crack width is being considered, calculated allowing for the stiffening effect of concrete in the tension zone from the following equation,

$$\epsilon_m = \epsilon_1 - \left[\frac{1.2\,b_t\,h_t\,s\ \ (a' - x)}{(A_t + A_{st})\,f_{ry}\,(h\text{-}x)}\right] 10^{-3}$$

where

b_t is the effective breadth of the composite section at the level of the tension reinforcement

a' is the distance from the compression face to the point at which the crack width is calculated

h_t is the overall depth of the composite member except that in composite beams, where the concrete flange is in tension, it should be taken as the depth of the concrete flange

ϵ_1 is the longitudinal strain at the level considered, calculated from a normal elastic analysis of the composite section, assuming the concrete in the tension zone to have zero tensile strength.

NOTE The modular ratio for transient or permanent loading should be used as appropriate. Tensile reinforcement provided to satisfy the requirements of this clause may be included in calculating the section properties of the composite member.

A_t is the area of tension reinforcement

A_{st} is the area of the encased tension flange of the structural steel member, where appropriate

f_{ry} is the characteristic yield stress of the reinforcing bars, but not greater than 460 N/mm^2

s is a constant stress of 1 N/mm^2, re-expressed where necessary in units consistent with those used for other quantities.

A negative value of ϵ_m indicates that the section is uncracked.

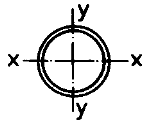
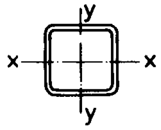
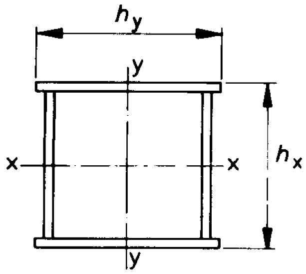
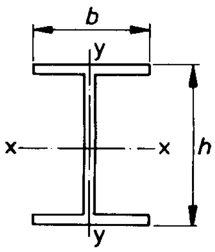
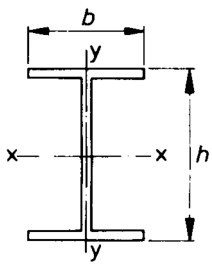
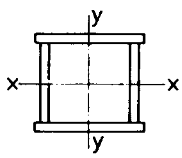
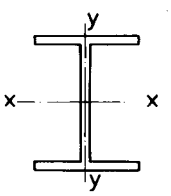
Where it is expected that the concrete may be subject to abnormally high shrinkage strains (> 0.006) consideration should be given to the increased tensile strain in the concrete slab. In the absence of a rigorous analysis, the value of should be increased by adding 50 % of the expected ϵ_m shrinkage strain.

Appendix C Formulae and tables for the design of composite columns

C.1 Coefficient K_1

Values of coefficient K_1 used with the additional subscripts x or y to describe the plane of bending may be determined from Table 13.1 to Table 13.3; the appropriate table to be used being selected from Table 13, depending on the type of steel section used and the axis of bending.

Table 13 — Strut curve selection chart

Shape of steel section		Curve	Table
 	rolled tubes welded tubes (hot finished)	a	Table 13.1
welded box sections x–x, h_x y–y, h_y		b	Table 13.2
I and H rolled sections:[a] 	Buckling about the x–x axis $h/b > 1.2$ $h/b \leqslant 1.2$	a b	Table 13.1 Table 13.2
	Buckling about y–y axis $h/b > 1.2$ $h/b \leqslant 1.2$	b c	Table 13.2 Table 13.3
I and H welded sections 	Buckling about x–x axis a) flame cut flanges b) rolled flanges	b	Table 13.2
	Buckling about y–y axis a) flame cut flanges b) rolled flanges	b c	Table 13.2 Table 13.3
Box sections stress relieved by heat treatment		a	Table 13.1
I and H sections, stress relieved by heat treatment Buckling about x–x axis Buckling about y–y axis		a b	Table 13.1 Table 13.2

[a] Not applicable to sections where the flange thickness exceeds 40 mm.

Table 13.1 — Values of coefficient K_1 for column curve a

λ	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	λ
0.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0
0.1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.1
0.2	1.000	0.998	0.996	0.994	0.992	0.990	0.988	0.985	0.983	0.981	0.2
0.3	0.978	0.977	0.973	0.971	0.968	0.966	0.963	0.961	0.958	0.956	0.3
0.4	0.954	0.953	0.948	0.945	0.942	0.939	0.936	0.933	0.930	0.926	0.4
0.5	0.923	0.919	0.916	0.912	0.908	0.904	0.900	0.896	0.892	0.889	0.5
0.6	0.884	0.881	0.877	0.873	0.869	0.866	0.861	0.857	0.854	0.849	0.6
0.7	0.845	0.842	0.836	0.831	0.826	0.821	0.816	0.812	0.807	0.802	0.7
0.8	0.796	0.791	0.786	0,781	0.775	0.769	0.763	0.758	0.752	0.746	0.8
0.9	0.739	0.734	0.727	0.721	0.714	0.708	0.701	0.695	0.688	0.681	0.9
1.0	0.675	0.668	0.661	0.654	0.647	0.640	0.634	0.629	0.619	0.613	1.0
1.1	0.606	0.599	0.593	0.585	0.579	0.573	0.565	0.559	0.553	0.547	1.1
1.2	0.542	0.533	0.527	0.521	0.515	0.509	0.503	0.497	0.491	0.485	1.2
1.3	0.480	0.474	0.469	0.463	0.456	0.453	0.447	0.442	0.437	0.432	1.3
1.4	0.427	0.422	0.417	0.412	0.408	0.403	0.398	0.394	0.389	0.386	1.4
1.5	0.381	0.375	0.372	0.368	0.364	0.360	0.356	0.352	0.348	0.344	1.5
1.6	0.341	0.337	0.333	0.330	0.326	0.323	0.319	0.316	0.312	0.309	1.6
1.7	0.306	0.303	0.300	0.298	0.294	0.291	0.288	0.285	0.282	0.280	1.7
1.8	0.277	0.274	0.271	0.269	0.266	0.264	0.261	0.258	0.256	0.253	1.8
1.9	0.251	0.248	0.246	0.243	0.242	0.239	0.236	0.234	0.232	0.230	1.9
2.0	0.228	0.226	0.224	0.222	0.219	0.217	0.215	0.213	0.211	0.209	2.0
2.1	0.208	0.206	0.204	0.202	0.201	0.199	0.197	0.196	0.194	0.192	2.1
2.2	0.191	0.189	0.187	0.186	0.184	0.183	0.181	0.180	0.179	0.177	2.2
2.3	0.175	0.174	0.172	0.170	0.158	0.167	0.166	0.165	0.164	0.163	2.3
2.4	0.162	0.160	0.159	0.158	0.156	0.155	0.154	0.153	0.152	0.150	2.4
2.5	0.149	—	—	—	—	—	—	—	—	—	2.5

Table 13.2 — Values of coefficient K_1 for column curve b

λ	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	λ
0.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.0
0.1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.1
0.2	1.000	0.997	0.993	0.989	0.986	0.983	0.980	0.977	0.972	0.969	0.2
0.3	0.965	0.961	0.957	0.953	0.950	0.945	0.941	0.937	0.933	0.929	0.3
0.4	0.925	0.921	0.917	0.913	0.909	0.905	0.901	0.897	0.893	0.889	0.4
0.5	0.885	0.881	0.876	0.872	0.867	0.862	0.858	0.853	0.849	0.843	0.5
0.6	0.838	0.833	0.828	0.823	0.817	0.812	0.807	0.802	0.796	0.791	0.6
0.7	0.785	0.780	0.774	0.768	0.762	0.757	0.751	0.745	0.739	0.733	0.7
0.8	0.727	0.721	0.715	0.709	0.702	0.695	0.690	0.683	0.677	0.670	0.8
0.9	0.663	0.656	0.650	0.643	0.636	0.631	0.624	0.618	0.611	0.605	0.9
1.0	0.599	0.592	0.586	0.580	0.574	0.568	0.562	0.555	0.549	0.544	1.0
1.1	0.537	0.531	0.526	0.521	0.515	0.509	0.503	0.497	0.491	0.486	1.1
1.2	0.480	0.475	0.470	0.465	0.459	0.454	0.449	0.444	0.439	0.434	1.2
1.3	0.429	0.424	0.419	0.415	0.410	0.405	0.401	0.396	0.392	0.387	1.3
1.4	0.383	0.379	0.375	0.370	0.366	0.362	0.358	0.354	0.350	0.346	1.4
1.5	0.343	0.339	0.335	0.332	0.328	0.324	0.321	0.317	0.314	0.311	1.5
1.6	0.307	0.304	0.301	0.298	0.295	0.292	0.289	0.286	0.283	0.279	1.6
1.7	0.277	0.274	0.271	0.268	0.265	0.263	0.260	0.258	0.255	0.253	1.7
1.8	0.250	0.248	0.246	0.243	0.241	0.239	0.236	0.234	0.232	0.230	1.8
1.9	0.227	0.225	0.224	0.221	0.219	0.217	0.215	0.213	0.211	0.209	1.9
2.0	0.207	0.205	0.203	0.202	0.200	0.198	0.197	0.195	0.193	0.191	2.0
2.1	0.190	0.188	0.186	0.185	0.183	0.182	0.180	0.179	0.178	0.176	2.1
2.2	0.175	0.173	0.172	0.170	0.169	0.168	0.166	0.165	0.164	0.162	2.2
2.3	0.161	0.160	0.159	0.157	0.156	0.154	0.153	0.152	0.151	0.149	2.3
2.4	0.148	0.147	0.146	0.145	0.144	0.143	0.142	0.141	0.140	0.139	2.4
2.5	0.138	—	—	—	—	—	—	—	—	—	2.5

Table 13.3 — Values of coefficient K_1 for column curve c

λ	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	λ
0.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.0
0.1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.1
0.2	1.000	0.995	0.990	0.985	0.980	0.975	0.970	0.965	0.960	0.955	0.2
0.3	0.951	0.946	0.941	0.936	0.931	0.926	0.921	0.915	0.910	0.905	0.3
0.4	0.900	0.895	0.890	0.884	0.878	0.873	0.867	0.861	0.856	0.850	0.4
0.5	0.844	0.838	0.832	0.826	0.820	0.814	0.808	0.802	0.795	0.789	0.5
0.6	0.783	0.776	0.770	0.764	0.757	0.753	0.744	0.738	0.731	0.726	0.6
0.7	0.719	0.712	0.706	0.700	0.693	0.687	0.680	0.674	0.667	0.661	0.7
0.8	0.654	0.647	0.642	0.635	0.629	0.623	0.617	0.611	0.605	0.599	0.8
0.9	0.593	0.587	0.581	0.575	0.570	0.565	0.559	0.553	0.547	0.542	0.9
1.0	0.537	0.532	0.526	0.521	0.517	0.511	0.506	0.501	0.496	0.491	1.0
1.1	0.486	0.481	0.476	0.471	0.466	0.461	0.457	0.452	0.447	0.443	1.1
1.2	0.438	0.434	0.429	0.425	0.421	0.416	0.412	0.408	0.403	0.399	1.2
1.3	0.395	0.391	0.387	0.383	0.379	0.375	0.372	0.368	0.364	0.360	1.3
1.4	0.357	0.353	0.350	0.346	0.343	0.339	0.336	0.333	0.329	0.326	1.4
1.5	0.323	0.320	0.318	0.314	0.311	0.308	0.305	0.302	0.299	0.296	1.5
1.6	0.293	0.290	0.287	0.284	0.281	0.277	0.275	0.273	0.270	0.268	1.6
1.7	0.265	0.263	0.261	0.258	0.256	0.253	0.250	0.248	0.245	0.243	1.7
1.8	0.241	0.238	0.236	0.234	0.232	0.230	0.228	0.226	0.224	0.222	1.8
1.9	0.220	0.218	0.217	0.215	0.213	0.212	0.210	0.208	0.206	0.204	1.9
2.0	0.202	0.201	0.199	0.197	0.196	0.194	0.192	0.191	0.189	0.187	2.0
2.1	0.186	0.185	0.184	0.182	0.181	0.179	0.177	0.176	0.175	0.173	2.1
2.2	0.172	0.170	0.169	0.167	0.166	0.165	0.164	0.162	0.161	0.160	2.2
2.3	0.159	0.157	0.156	0.155	0.154	0.152	0.151	0.150	0.149	0.148	2.3
2.4	0.147	0.146	0.145	0.144	0.142	0.141	0.140	0.139	0.139	0.138	2.4
2.5	0.137	—	—	—	—	—	—	—	—	—	2.5

Alternatively, values of K_1 may be calculated from the following equations:

$$K_1 = \tfrac{1}{2}\left[1 + \frac{(1+\eta)}{\lambda^2}\right] - \sqrt{\tfrac{1}{4}\left[1 + \frac{(1+\eta)}{\lambda^2}\right]^2 - \frac{1}{\lambda^2}}$$

where the imperfection constant η is given by

$\eta = \psi\lambda_E\,(\lambda - 0.2) \leqslant 0$

λ is determined from clause **11.3.1** using the parameters for the x or y axis; as appropriate

$$\lambda_E = \pi\sqrt{\frac{1.1E_s}{f_y}}$$

and ψ is determined from Table 14

Table 14 — Values of coefficient ψ

Type of steel section	Axis of buckling	Value of ψ
Rolled *I* (UB, Joist etc.)	x–x y–y	0.0020 0.0035
Rolled H (UC etc.) flanges up to 40 mm	x–x y–y	0.0035 0.0055
Rolled H (UC etc.) flanges over 40 mm	x–x y–y	0.0055 0.0080
Welded I or H flanges up to 40 mm	x–x y–y	0.0035 0.0055
Welded I or H flanges over 40 mm	x–x y–y	0.0035 0.0080
Hot-rolled structural hollow sections	any	0.0020

C.2 Coefficient K_2

Values of the coefficient K_2, used with the additional subscripts x or y, to describe the plane of bending, may be calculated from the equations given in **C.2.1** or **C.2.2**, as appropriate, between the following limits:

$$0 \leqslant \frac{K_2}{K_{20}} \leqslant 1 \text{ and}$$

$$K_{20} \leqslant 0.75$$

except that if the calculated value of K_2/K_{20} is negative then K_2 should be taken as zero.

C.2.1 *Concrete filled circular hollow sections*

$$\frac{K_2}{K_{20}} = \left[\frac{115 - 30(2\beta - 1)(1.8 - \alpha_c) - C_3\lambda}{50(2.1 - \beta)}\right] \text{ and}$$

$$K_{20} = 0.9\,\alpha_c{}^2 + 0.2$$

where

β is the ratio of the smaller to the larger of the two end moments acting about each axis, used with the additional subscripts x or y to denote the plane of bending considered, the sign convention being such that β is positive for single curvature bending

α_c is the concrete contribution factor, calculated from **11.1.4** or **11.3.7** as appropriate

C_3 is a constant, which may be taken as 100

λ is the slenderness function, calculated from **11.3.1**, for the major or minor axis as appropriate denoted λ_x, λ_y respectively.

C.2.2 *Concrete encased steel sections and concrete filled rectangular hollow sections*

$$\frac{K_2}{K_{20}} = \left[\frac{90 - 25(2\beta - 1)(1.8 - \alpha_c) - C_4\lambda}{30(2.5 - \beta)}\right]$$

where

C_4 is taken as:

100 for columns designed on the basis of curve a

120 for columns designed on the basis of curve b

140 for columns designed on the basis of curve c

and the remaining terms are as given in **C.2.1**.

C.3 Coefficient K_3

C.3.1 *Concrete filled circular hollow sections.*

The value of K_3 should be calculated from the following equation:

$$K_3 = K_{30} + \frac{[(0.5\beta + 0.4)(\alpha_c{}^2 - 0.5) + 0.15]\,C_5\lambda}{1 + (C_5\lambda)^3}$$

where

$K_{30} = 0.04 - (\alpha_c/15)$ except that K_{30} should not be taken less than zero

α_c and β are as defined in **C.2.1**

C_5 is a constant which may be taken as 1.0

C.3.2 *Values of the coefficient K_3 for encased sections and concrete filled rectangular hollow sections*

a) for bending about the strong axis of the steel section (which may not be the x–x axis as defined in **11.3.1**), K_3 should be taken as zero;

b) for bending about the weak axis of the steel section, K_3 should be calculated from;

$K_3 = 0.425 - 0725\beta - 0.005\ C_4\lambda$ for encased sections

but should be taken as not less than $-0.03(1 + \beta)$ nor more than $(0.2 - 0.25\alpha_c)$. For minor-axis bending of concrete filled rectangular hollow sections the value of K_3 may be taken as zero. The symbols are as defined in **C.2.1** and **C.2.2**.

C.4 Ultimate moment of resistance M_u of composite columns

C.4.1 *General*

The ultimate moment of resistance in pure bending of a composite column formed from a concrete encased steel section or concrete filled hollow steel section may be calculated using the following assumptions:

a) the whole of the area of steel, including the reinforcement (if any), is stressed to its design yield strength in tension or compression, i.e. nominal yield strength/γm;

b) the strength of concrete on the tension side of the plastic neutral axis is neglected;

c) the area of concrete on the compression side of the plastic neutral axis is tressed uniformly to its design compressive strength, which should be taken as 0.4 f_{cu};

d) the flanges of the steel section are of constant thickness and fillets are ignored.

Alternatively, for concrete encased steel sections and concrete filled rectangular hollow steel sections the ultimate moment of resistance may be calculated from the equations given in **C.4.2**, which are based on the foregoing assumptions, and on the assumption that:

1) the area of reinforcement in the cross section is small with equal amounts in tension and compression;

2) the concrete displaced by the steel section in an encased column is neglected in calculating the compressive force.

NOTE The ultimate moment of resistance of concrete filled circular hollow sections may either be obtained from **C.4.3** or calculated using assumptions a) to d) above.

C.4.2 ***Equations for calculating M_u for concrete encased steel sections and concrete filled rectangular hollow sections***

C.4.2.1 The ultimate moment of resistance may be calculated from the equations given in **C.4.2.2** to **C.4.2.5** where:

$$\rho = \frac{0.4\, f_{cu}}{0.91\, f_y}$$

where

ρ	is the ratio of the average compressive stress in the concrete at failure to the design yield strength of the steel taken as $0.4\, f_{cu}/0.91\, f_y$
f_{cu}	is the characteristic 28 day cube strength of concrete
f_y	is the nominal yield strength of steel

C.4.2.2 *Concrete encased steel sections: plastic neutral axis outside the steel section.* [See Figure 8(a)]. This condition arises when:

$\rho\, b\, d_s \geqslant A_s$ then

$d_c = \dfrac{A_s}{b\rho}$ and

$$M_{ux} = 0.91\, f_y\, A_s \frac{(h - d_c)}{2} + 0.87\, f_{ry} \frac{A_r}{2} d_r$$

$$M_{uy} = 0.91\, f_y\, A_s \left(\frac{b - d_c}{2}\right) + 0.87\, f_{ry} \frac{A_r}{2}\, d_r$$

b	is the breadth of concrete in cross section
d_s	is the thickness of concrete cover to encased steel section
A_s	is the area of rolled or fabricated steel section
d_c	is the distance of neutral axis from the most compressed face of concrete
M_{ux} or M_{uy}	is the design ultimate moment of resistance about the x and y axes respectively, in the absence of axial load
h	is the depth of concrete cross section or depth of concrete in filled rectangular hollow sections
f_{ry}	is the characteristic yield strength of reinforcement
A_r	is the area of reinforcement in the cross section
d_r	is the distance between symmetrically placed reinforcing bars measured perpendicular to the axis of bending

NOTE The remaining symbols are as defined **C.4.2.1**.

C.4.2.3 *Cased sections: plastic neutral axis within top flange/major axis bending.* [See Figure 8(b)]. This condition arises when

$\rho\, b d_s < A_s$ and

$0.91\, A_s f_y \leqslant 0.4\, f_{cu}\, [b d_s + t_f\, (b - b_f)] + 1.82\, A_f f_y$ then

$d_c = (A_s + 2 b_f\, d_s)/(b\rho + 2 b_f)$ and

$$M_u = 0.91\, f_y \left[A_s \frac{(h - d_c)}{2} - b_f\, d_s\, (d_c - d_s)\right] + 0.87 f_{ry} \frac{A_r}{2} d_r$$

A_f	is the area of the top flange of the steel section
t_f	is the average thickness of the flange of a steel section
b_f	is the breadth of steel flange of I section or the external dimension of a rectangular hollow section.

NOTE The remaining symbols are as defined in **C.4.2.1** and **C.4.2.2**.

C.4.2.4 *Cased sections: plastic neutral axis in web/major axis bending.* [See Figure 8(c)]. This condition arises when:

$(A_s - 2\, b_f\, t_f) > \rho\, [b d_s + t_f (b - b_f)]$

$d_c = \dfrac{h t_w}{(b\rho + 2 t_w)}$ and

$$M_u = 0.91\, f_y \left[A_s \frac{(h - d_c)}{2} - b_f t_f\, (d_s - d_w) - t_w d_w (d_c - d_w)\right]$$

$$+ 0.87\, f_{ry} \frac{A_r}{2} d_r$$

where

t_w is the thickness of web of steel section

d_w is the depth of steel web in compression zone

NOTE The remaining symbols are as defined in **C.4.2.1**, **C.4.2.2** and **C.4.2.3**.

C.4.2.5 *Cased sections: plastic neutral axis in flanges/minor axis bending.* [See Figure 8(d)]. This condition arises when:

$\rho b d_s < A_s$ then

$d_c = (A_s + 4 t_f\, d_s)/(b\rho + 4 t_f)$ and

$$M_u = 0.91\, f_y \left[A_s \frac{(h - d_c)}{2} - 2 t_f\, d_s\, (d_c - d_s)\right] + 0.87 f_{ry} \frac{A_r}{2} d_r$$

NOTE For definition of symbols see **C.4.2.1**, **C.4.2.2** and **C.4.2.3**.

C.4.2.5 *Concrete filled rectangular hollow sections* [See Figure 8(e)]

$$d_c = \frac{(A_s - 2b_f t_f)}{(b\rho + 4t_f)} \text{ and}$$

$$M_u = 0.91 f_y \left[A_s \frac{(h - d_c)}{2} + b_f t_f (t_f + d_c) \right]$$

NOTE For definition of symbols see **C.4.2.1**, **C.4.2.2** and **C.4.2.3**.

C.4.3 ***Equations for calculating M_u for concrete filled circular hollow steel sections.*** The ultimate moment of resistance M_u of a concrete filled circular hollow steel section without reinforcement may be calculated from the following equation:

$$M_u = 0.91\ Sf_y\ (1 + 0.01\ m)$$

where the plastic section modulus of the steel section S, is given by:

$$S = t_3\left(\frac{D_e}{t} - 1\right)^2$$

m is determined from Figure 9

where

D_e is the outside diameter of the steel section

t is the wall thickness

ρ is as defined in **C.4.2.1**.

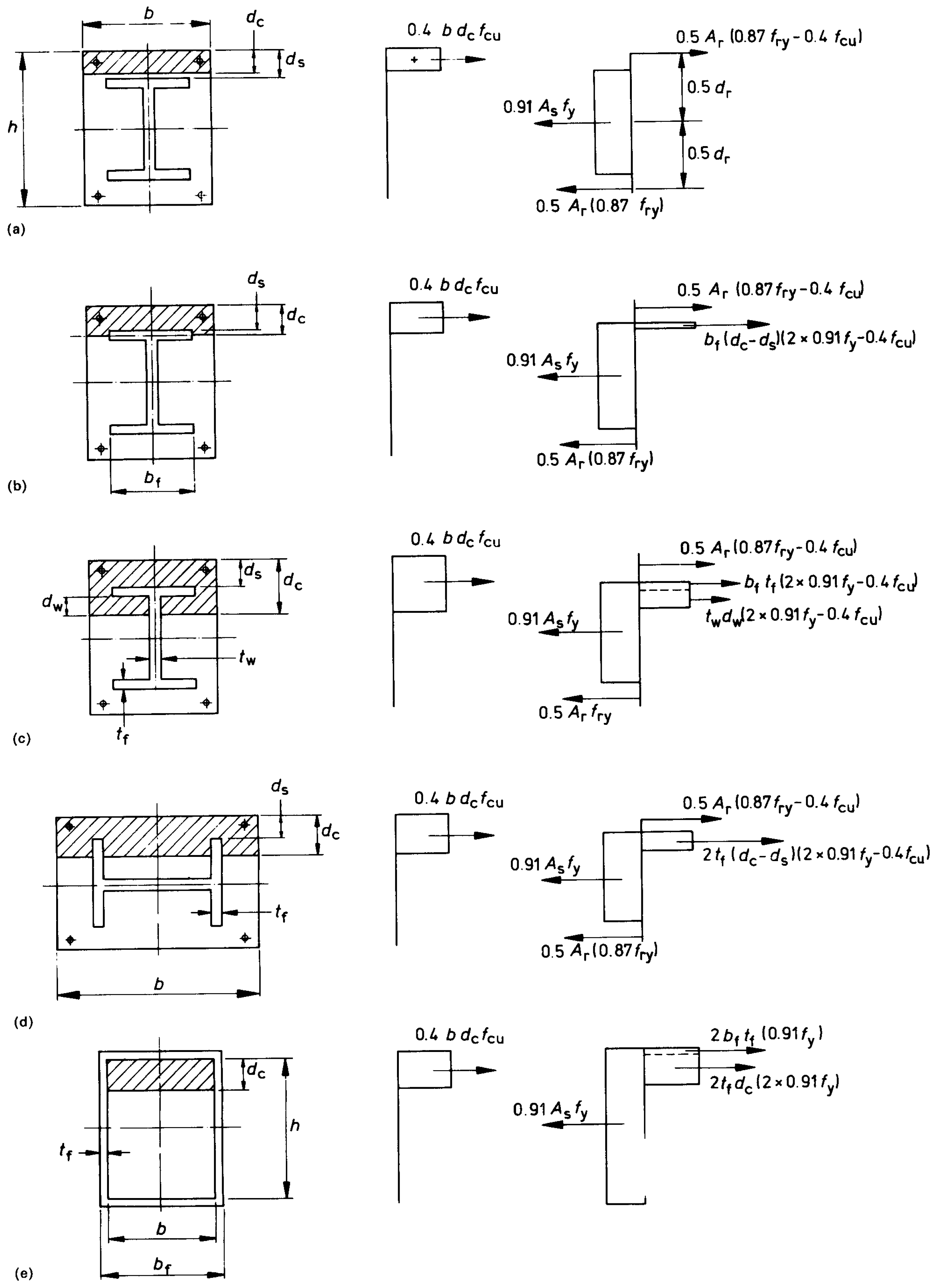


Figure 8 — Force diagrams for calculating M_u

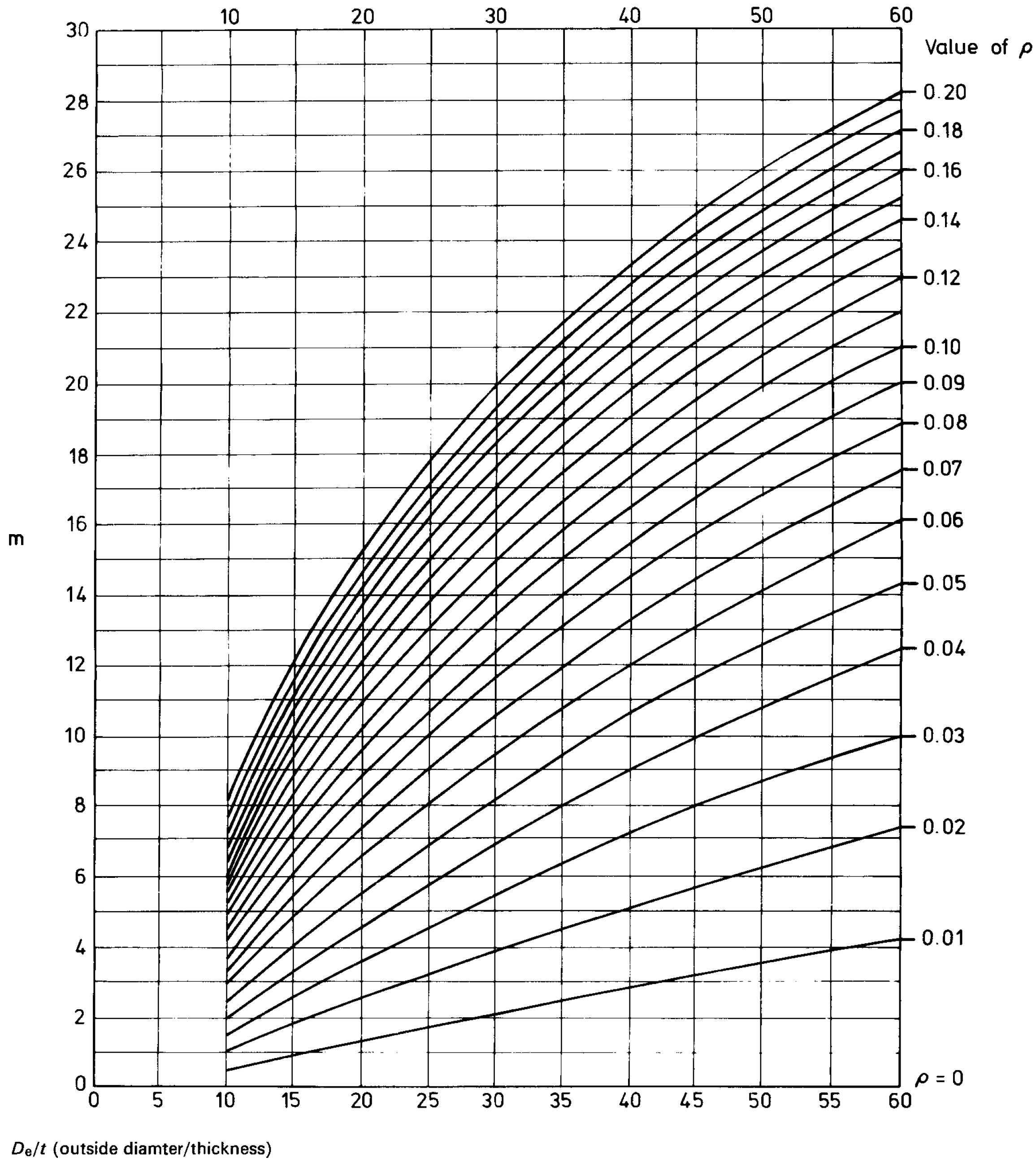


Figure 9 — Chart for evaluating M_u of concrete filled circular hollow sections

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Publications referred to

BS 4, *Structural steel sections.*
BS 4-1, *Hot-rolled sections.*
BS 4-2, *Hot-rolled hollow sections.*
BS 3139, *High strength friction grip bolts for structural engineering.*
BS 3139-1, *General grade bolts.*
BS 3294, *The use of high strength friction grip bolts in structural steelwork.*
BS 4360, *Weldable structural steels.*
BS 4395, *High strength friction grip bolts and associated nuts and washers for structural engineering.*
BS 4395-1, *General grade.*
BS 4848, *Hot-rolled structural steel sections.*
BS 4848-2, *Hollow sections.*
BS 5400, *Steel, concrete and composite bridges.*
BS 5400-1, *General statement.*
BS 5400-2, *Specification for loads.*
BS 5400-3, *Code of practice for design of steel bridges*[2)].
BS 5400-4, *Code of practice for design of concrete bridges.*
BS 5400-6, *Specification for materials and workmanship, steel.*
BS 5400-7, *Specification for materials and workmanship, concrete, reinforcement and prestressing tendons.*
BS 5400-8, *Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons.*
BS 5400-9, *Code of practice for bearings*[2)].
BS 5400-10, *Code of practice for fatigue.*
CP 110, *The structural use of concrete.*
CP 110-1, *Design, materials and workmanship.*

[2)] In course of preparation.

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