Steel, concrete and composite bridges —

Part 5: Code of practice for the design of composite bridges

ICS 93.040

British Standards

Committees responsible for this British Standard

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Contents

Foreword

This part of BS 5400 was prepared by BSI Technical Committee B/525/10, Bridges. It supersedes BS 5400-5:1979 which is withdrawn.

BS 5400 combines 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 of loads*
Part 3:	*Code of practice for design of steel bridges*
Part 4:	*Code of practice for design of concrete bridges*
Part 5:	*Code of practice for design of composite bridges*
Part 6:	*Specification for materials and workmanship, steel*
Part 7:	*Specification for materials and workmanship, concrete, reinforcement and prestressing tendons*
Part 8:	*Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons*
Part 9:	*Bridge bearings*
	Section 9.1 Code of practice for design of bridge bearings
	Section 9.2 Specification for materials, manufacture and installation of bridge bearings
Part 10:	*Code of practice for fatigue*

The BS 5400 series was developed over a period of several years and the previous edition of Part 5 was published before the series was complete. This edition introduces changes to align it with provisions contained in other parts of the series, and with the Highways Agency design manual BD 16/82, *Design of composite bridges — Use of BS 5400: Part 5: 1979.*

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iii, a blank page, pages 1 to 48, an inside back cover and a back cover.

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1 Scope

This part of BS 5400 gives recommendations for the design of bridges that use rolled or fabricated steel sections, cased or uncased, and 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 design of the concrete element cover normal and lightweight aggregate, cast in situ and precast concrete. Recommendations are also made in respect of prestressing and the use of permanent formwork designed to act compositely with in situ concrete.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

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

BS 4395-1, *Specification for high strength friction grip bolts and associated nuts and washers for structural engineering — Part 1: General grade.*

BS 4604, *Specification for the use of high strength friction grip bolts in structural steelwork. Metric series — Part 1: General grade.*

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

BS 5400-2:2005[1)], *Steel, concrete and composite bridges — Part 2: Specification for loads.*

BS 5400-3:2000, *Steel, concrete and composite bridges — Part 3: Code of practice for design of steel bridges.*

BS 5400-4:1990, *Steel, concrete and composite bridges — Part 4: Code of practice for design of concrete bridges.*

BS 5400-7, *Steel, concrete and composite bridges — Part 7: Specification for materials and workmanship, concrete, reinforcement and prestressing tendons.*

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

BS EN 1994-1-1:2004, *Eurocode 4. Design of composite steel and concrete structures — General rules and rules for buildings.*

BS EN 10025-1, *Hot rolled products of structural steels — General technical delivery conditions.*

NOTE Where reference is made in this British Standard to "Part 1", "Part 3", etc. it should be taken as a reference to the respective part of BS 5400.

3 Definitions

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

3.1
cased composite beam
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

3.2
uncased composite beam
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

1) In preparation.

3.3
composite box beam
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.

3.4
composite column
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

3.5
composite plate
in situ concrete slab cast upon, and acting compositely with, a structural steel plate

3.6
concrete slab
structural concrete slab that forms part of the deck of the bridge and acts compositely with the steel beams

NOTE The slab may be of precast, cast in situ or composite construction.

3.7
composite slab
in situ concrete slab that acts compositely with structurally participating permanent formwork

3.8
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

3.9
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

3.10
filler beam construction
rolled or built-up steel sections that act in conjunction with a concrete slab and which are contained within the slab

3.11
interaction

3.11.1
complete interaction
interface between the steel and the concrete slab or encasement such that no significant slip occurs between them

3.11.2
partial interaction
interface between the steel and the concrete slab or encasement such that slip occurs between them and a discontinuity in strain occurs

3.12
shear connector
mechanical device to ensure interaction between concrete and steel

3.13
connector modulus
elastic shear stiffness of a shear connector

3.14 Symbols

The symbols used in this part of this standard 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
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
l_w	Length of wheel patch
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
R^*	Design resistance as defined in Part 1
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
v_1	Ultimate longitudinal shear stress of concrete
x	Neutral axis depth, coordinate, distance
y	Coordinate
Z_{pe}	Plastic section modulus of beams as defined in Part 3, **9.7.1**
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 the steel, taken as 0.4 f_{cu}/0.95 f_y
σ	Stress of steel member (with appropriate subscripts) as defined in Part 3
Φ	Bar size
ϕ	Creep coefficient
ϕ_c	Creep reduction factor
χ	Non-dimensional coordinate
ψ	Effective breadth ratio, coefficient

4 General design principles

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, **2.3.1**.

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^* in accordance with Part 1.

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.

4.1.4 *Verification of structural adequacy*

For a satisfactory design the following relation should be satisfied:

$$R^* > S^* \qquad [1]$$

where R^* is the design resistance.

$$\text{i.e. function } (f_k / \gamma_m) \geq \gamma_{f3} \text{ (effects of } \gamma_{fL}.Q_k) \qquad [2a]$$

where f_k, γ_m, γ_{fL} and Q_k are defined in Part 1.

When the resistance function is linear, and a single value of γ_m is involved, this relation may be rearranged as:

$$(1/\gamma_{f3}.\gamma_m) \text{ function } (f_k) \geq \text{ (effects of } \gamma_{fL}.Q_k) \qquad [2b]$$

It should be noted that the format of equation 2a is used in Part 4 whereas the format given in equation 2b is used in Part 3. Therefore when using this part in conjunction with either Part 3 or Part 4 care has to be taken to ensure that γ_{f3} is applied correctly.

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 limit state. It should be noted that the stress limitations given in Part 4 allow for γ_m. For shear connectors, the appropriate values of γ_m are explicitly given in the expressions for design resistance in this part. For the longitudinal shear resistance of reinforced concrete slabs over steel beams, the appropriate values of γ_m for the concrete and reinforcement are already incorporated in the expressions given in this part.

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*

All structural steelwork in composite beams should be checked for conformity to the recommendations of Part 3 in relation to both limit states. Where recommended in Part 3 the effects of creep, shrinkage and temperature should be calculated in accordance with the recommendations of this part, for the relevant limit state.

The concrete and reinforcement in concrete slabs should satisfy the limit state recommendations made in Part 4 including the serviceability limit state stress limitations given in Part 4, **4.1.1.3**. Where they are part of a composite beam section they should also satisfy the limit state recommendations made in this part. The method of calculating crack widths at the serviceability limit state should follow the recommendations of this part.

Shear connectors should be designed to meet the recommendations of the serviceability limit state given in this part and, where specified in this part, the requirements of the ultimate limit state.

Both shear connectors and structural steelwork should satisfy the fatigue recommendations given in Part 10.

4.3.2 *Serviceability limit state*

A serviceability limit state is reached when any of the following conditions occur:

a) the stress or deformation in structural steel reaches the levels indicated in Part 3;

b) the stress in concrete or reinforcement 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*

At each internal support in continuous beams, the apparent tensile stress in the concrete at the top surface of the slab, due to the greatest 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}, 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 $40f_{tc}/f_{cu}$ % 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*

The stresses in composite sections should be determined in accordance with **5.2.2**, **5.2.3**, **5.2.4** and **5.2.5**. Crack widths should be checked in accordance with **5.2.6**.

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.

When the cross-section of a beam and the applied loading increase by stages, a check for adequacy should be made for each stage of construction in accordance with **6.2.4**, treating all sections as non-compact. The bending stresses should not exceed the appropriate limits given in **6.2.4** using the appropriate values of γ_m and γ_{f3} for the serviceability limit state, except that the limiting tensile stress in the reinforcement should be replaced by $0.75f_{ry}/\gamma_{f3}$.

5.2.3 *Effective breadth of concrete flange*

5.2.3.1 *General*

In calculating the stresses in a flange, and in the absence of rigorous analysis, the effect of in-plane shear flexibility (i.e. shear lag) should be allowed for by assuming an effective breadth of flange in accordance with **5.2.3.2**, **5.2.3.3** and Part 3.

5.2.3.2 *Effective breadth of cracked flange*

For a concrete flange in tension that is assumed to be cracked, the mean effective breadth ratio ψ obtained from Part 3 should be modified by adding $(1 - \psi)/3$.

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.

5.2.4 *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.

Where these effects co-exist, they should be combined in accordance with Part 4, **4.8**.

5.2.4.2 *Serviceability requirements*

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 *Co-existent stresses*

In calculating coexistent stresses in a deck slab, which also forms the flange of a composite beam, the global longitudinal bending stress across the deck width may be calculated in accordance with Part 3, **A.6**.

5.2.5 *Steel section*

5.2.5.1 *General*

The serviceability limit state should be checked in accordance with Part 3; in performing such checks consideration should be given to the effects noted in **5.2.5.2**, **5.2.5.3** and **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 Part 3 and the relevant design procedures.

NOTE For the purpose of estimating the effective breadth of the flange, where the complete span has not been concreted, the effective span 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*

For the crack width limitations given in **5.2.6.3**, load combination 1 only of Part 2 should be considered. Highway live loading should generally comprise type HA only. However, for transverse cantilever slabs, transversely and two-way spanning slabs and central reserves, the loading should be in accordance with BS 5400-2:2005, **6.4.3**[2)], except that only 30 units of HB should be considered in any notional lane.

[2)] This reference is specific to the 2005 edition and is not correct for the 1978 edition.

5.2.6.3 *Limiting crack width*

The engineer should be satisfied 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 appropriate method given in Part 4, **5.8.8.2**. 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 Part 3, **A.6**.

Where it is expected that the concrete may be subject to abnormally high shrinkage strains (>0.0006), consideration should be given to the increased tensile strain in the concrete slab. In the absence of a rigorous analysis, the value of longitudinal strain at the level where the crack width is being considered should be increased by adding 50 % of the expected shrinkage strain.

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 in both sagging and hogging moment regions. 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 Part 3.

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. Where an abrupt change in section leads to a concentrated force according to this analysis, this force may be spread over a length not exceeding twice the section depth.

5.3.2 *Shear connectors*

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

a) Static strengths

Table 1 gives the nominal static strengths of commonly used types of connectors, which are illustrated in Figure 1, in relation to the specified characteristic cube strengths of the normal grades of concrete. The nominal strengths given in Table 1 may be used where the slab is haunched provided that the haunch conforms to **6.3.2**. 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 1

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 1, 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 conforms to **6.3.2**.

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 1. Static strengths of other sizes of stud and of other types of connectors should be determined experimentally by push-out tests performed 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 conform to **6.3.2**, 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*

Testing of shear connectors should be in accordance with BS EN 1994-1-1, **B.2**, or with the following procedure.

a) Nominal strength

The nominal static strength of a shear connector may be determined by push-out tests. No fewer 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 specimens are given in Figure 2. 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 2 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.

5.3.2.5 *Design resistance of shear connectors*

The design resistance of shear connectors at the serviceability limit state should be taken as P_u/γ_m where P_u is the nominal static strength defined in **5.3.2.1**, **5.3.2.2** or **5.3.2.3** as appropriate, and $\gamma_m = 1.85$.

Table 1 — Nominal static strengths of shear connectors for different concrete strengths

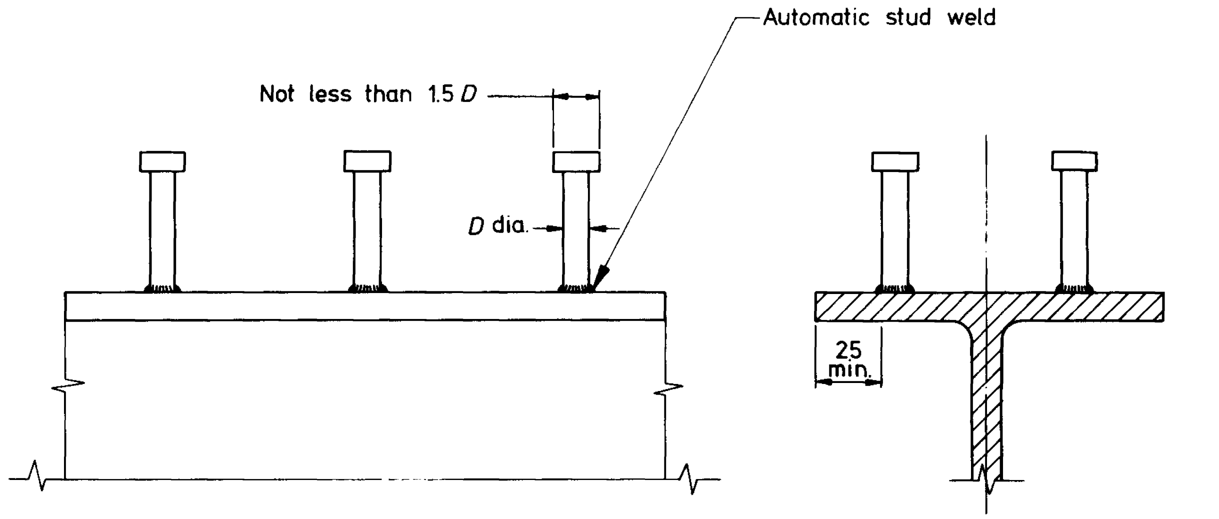
Type of connector		Connector material	Nominal static strengths in kN per connector for concrete strengths f_{cu}, N/mm^2			
			20	30	40	50
Headed studs (see Figure 1a)		Material with a characteristic yield stress of 385 N/mm^2, minimum elongation of 18 % and a characteristic tensile strength of 495 N/mm^2				
Diameter mm	Overall height 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 1b and Figure 1c)		Grade S275 of BS EN 10025				
50 mm × 40 mm × 200 mm bar			697	830	963	1096
25 mm × 25 mm × 200 mm bar			348	415	482	548
Channels (see Figure 1d)		Grade S275 of BS EN 10025				
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 4395-1	see Clause **10**			

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

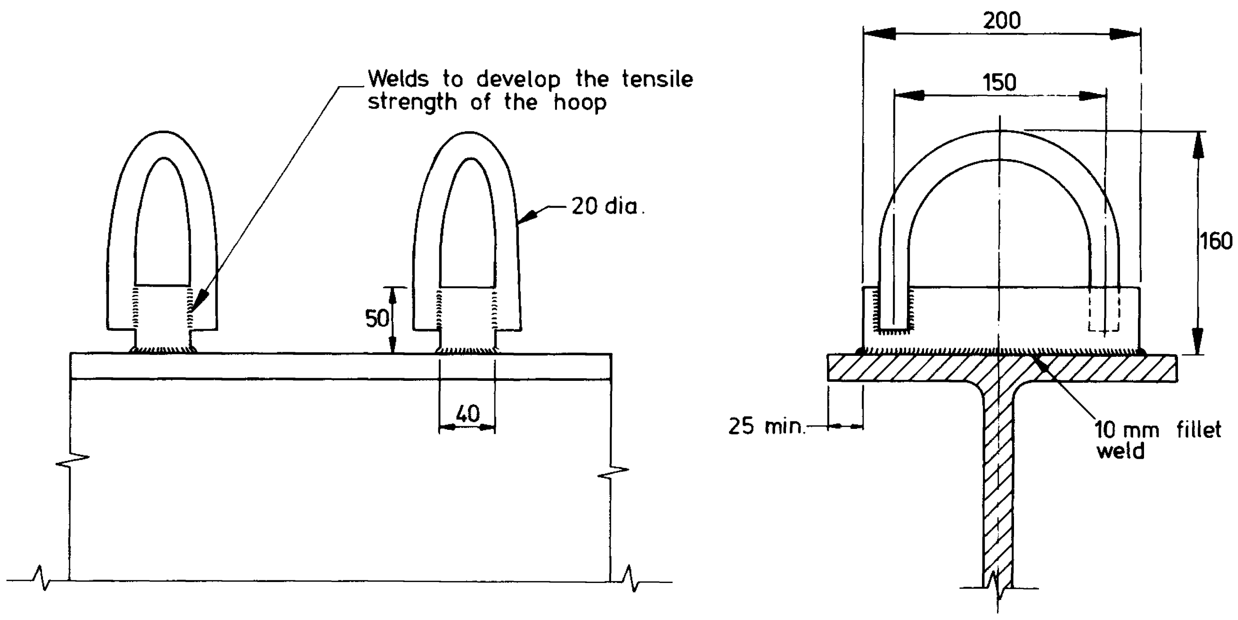
NOTE 2 Strengths for concrete of intermediate grade may be obtained by linear interpolation.

NOTE 3 For bars (see Figure 1b and Figure 1c), and channels (see Figure 1d) 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 for 100 mm high connectors, unless the static strength is determined from push-out tests in accordance with **5.3.2.4**.



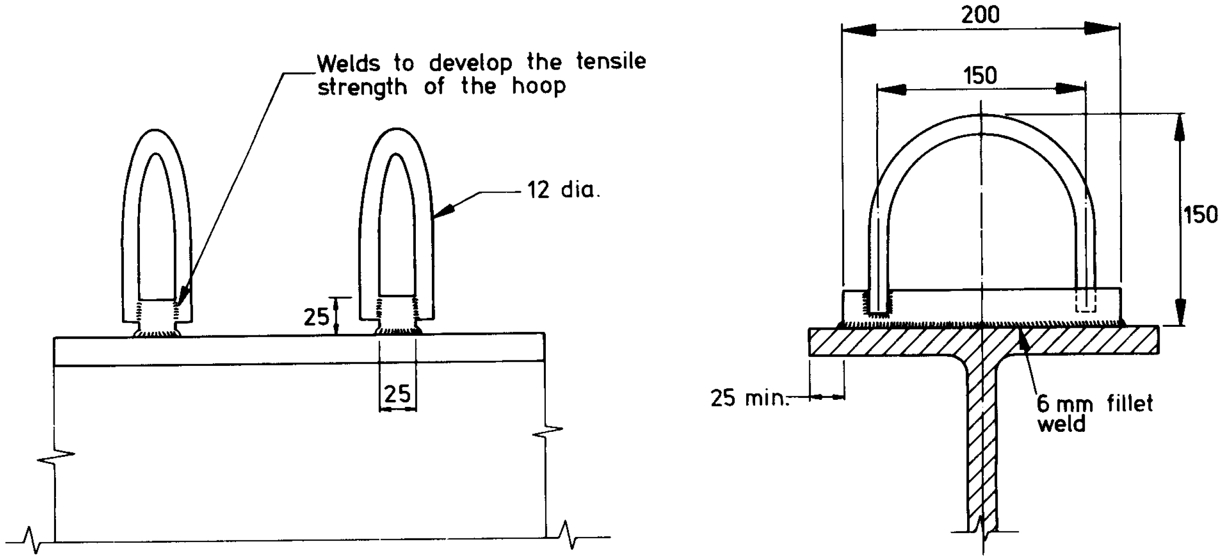
(a) Stud connector



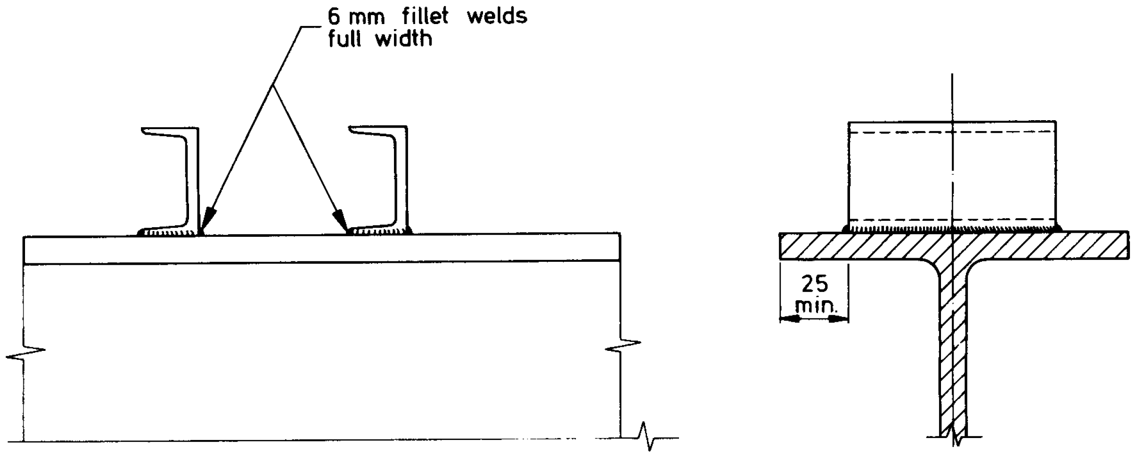
(b) 50 mm × 40 mm bar connector

All dimensions are in millimetres.

Figure 1 — Shear connectors



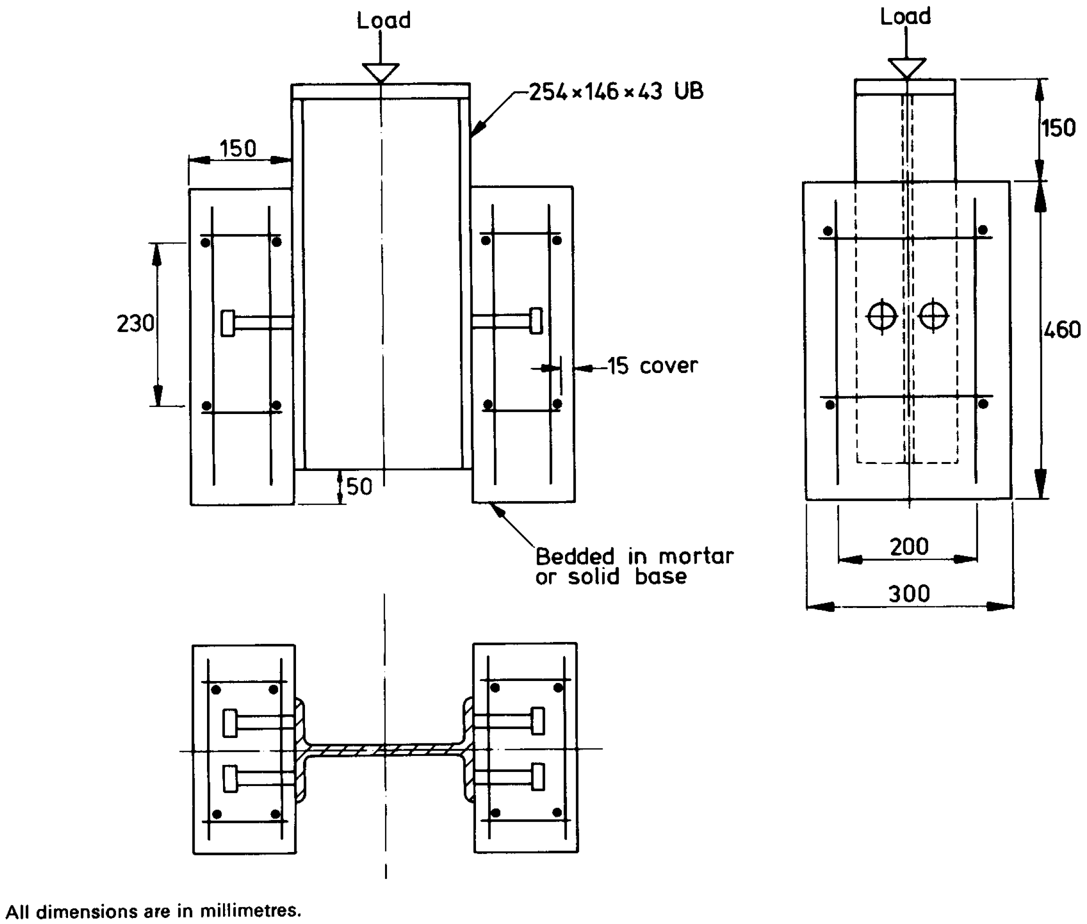
(c) 25 mm × 25 mm bar connector



(d) Channel connector

All dimensions are in millimetres.

Figure 1 — Shear connectors *(concluded)*



NOTE Reinforcement should be by 10 mm high yield bars, fully anchored by loops or by welding at intersections.

Figure 2 — 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.

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 1).

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.

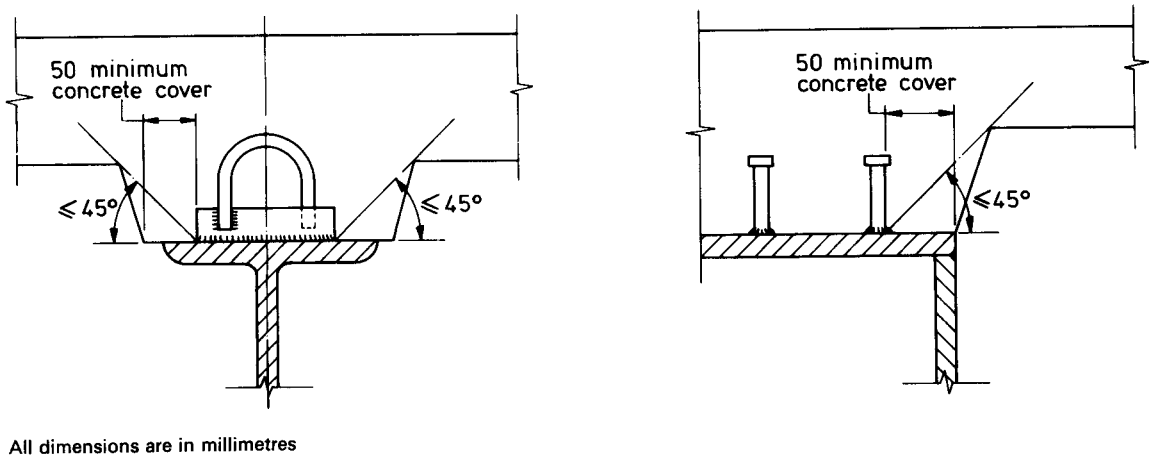


Figure 3 — Dimensions of haunches

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 5) 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 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 *Design procedure: general*

Shear connectors should be designed initially to satisfy the serviceability limit state in accordance with **5.3.3.5**. The initial design should be checked in accordance with Part 10 for fatigue.

Shear connectors need not be checked for static strength at the ultimate limit state except when required by **5.3.3.6** or **6.1.3**, or when redistribution of stresses from the tension flange has been made in accordance with Part 3.

5.3.3.5 *Design procedure: spacing and design resistance*

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 resistance/unit length 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, as defined in **5.3.2.5**.

5.3.3.6 *Uplift on shear connectors*

Where the shear connectors are subject to significant 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, or from the forces generated at the corners when the slab acts as part of "U" frame,

additional ties, suitably anchored, should be provided to resist these forces.

Where stud connectors are used and are subject to both shear Q and tension due to uplift T_u, the equivalent shear Q_{max} to be used in checking the connectors for static strength and fatigue should be taken as:

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

In addition the stud connectors should be also checked at the ultimate limit state in accordance with **6.3.4**, using the appropriate value of Q_{max}.

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 should be considered at the serviceability limit state for the beam section in accordance with the recommendations of Part 3. Serviceability checks are essential for shear connectors. In such checks account should be taken of the longitudinal shear forces arising from these effects. 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 reaction 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 of α_e appropriate to short term loading. No account need be taken of shear lag. Concrete should be assumed to be uncracked, except that, for calculating longitudinal bending stresses due to the secondary effects discussed in c), the concrete in tension may be ignored.

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 β_L 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

$$l_s = 2\sqrt{KQ/\Delta f} \qquad [4]$$

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_t, 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 2.

The values in Table 2 should only be used where the concrete specification conforms to the limits given in Figure 4. For situations outside the scope of Figure 4 and Table 2 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 Part 4, Appendix C.

The value of ϕ_c should then be taken as

$$\phi_c = \frac{1}{1+\phi} \qquad [5]$$

Table 2 — 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**.

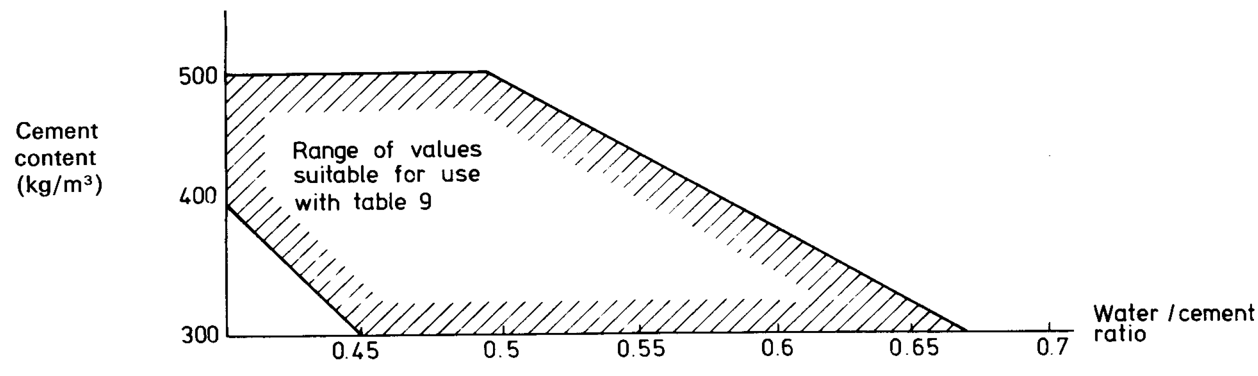


Figure 4 — Range of concretes for which Table 2 can be used

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. 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 in accordance with Part 3.

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 Part 4, Appendix C or as ϕ_c times the short term modulus where ϕ_c is given in Table 2 for concrete conforming to Figure 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**, 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 Part 1, **7.2** and should only be undertaken where they can be shown to model adequately the combined effects of local and global loads due to combinations 1–5 as given in Part 2.

6.1.2 *Deck slabs forming the flanges of composite beams*

The deck slab should be designed to resist separately 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, subject to the provisions of **6.1.1**.

The resistance to global effects should be determined in accordance with **6.2**. For local effects, the design of the slab cross-section should be in accordance with Part 4. The combined effects of global bending and local wheel loading should be taken into account 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, for a beam built in stages, the entire load is assumed to act on the final cross-section in accordance with Part 3, **9.9.5**, or where tensile stresses are redistributed from the tension flange in accordance with Part 3, **9.5.5.2**, the shear connectors and transverse reinforcement should be designed for the corresponding longitudinal shear in accordance with **6.3**.

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 that 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*

If the concrete is assumed uncracked over the whole length, up to 10 % of the support moments may be redistributed to the span provided that equilibrium between the internal forces and external loads is maintained under each appropriate combination of ultimate loads.

6.1.5 *Temperature effects and shrinkage modified by creep*

The effects of temperature and shrinkage modified by creep on the longitudinal stresses in the composite section should be considered at the ultimate limit state where required by Part 3. 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 in accordance with Part 3 and in accordance with the provisions of **6.2.2**, **6.2.3** and **6.2.4**.

6.2.2 ***Plastic moment of resistance of sections***

The plastic moment of resistance of the effective cross-section should be determined in accordance with Part 3, **9.7.1**.

6.2.3 ***Moment of resistance of non-compact cross sections***

The moment of resistance of a non-compact cross-section if lateral torsional buckling is prevented should be determined in accordance with Part 3, **9.8**, except that M_{ult} should also not exceed $Z_{\mathrm{xr}} \times (0.87 f_{\mathrm{ry}} \gamma_{\mathrm{m}})$ or $Z_{\mathrm{xs}} \times (0.5 f_{\mathrm{cu}} \gamma_{\mathrm{m}})$, as appropriate,

where

Z_{xr} is the elastic section modulus of the transformed section with respect to the extreme reinforcement, for a section where the concrete is in tension;

Z_{xs} is the elastic section modulus of the transformed section with respect to the extreme fibre of the concrete, where the concrete is in compression;

NOTE In determining the section moduli of a composite section, the transformed area of concrete in compression should be obtained using either the short term or the long term modular ratio of the concrete, as appropriate to the type of loading.

f_{cu} is the characteristic concrete cube strength;

f_{ry} is the characteristic yield strength of the reinforcement;

γ_{f3} is the partial safety factor in accordance with Part 3;

γ_{m} is the partial material factor for steel in accordance with Part 3.

6.2.4 ***Construction in stages***

Composite sections should be considered to be built in stages. A check for adequacy should be made for each stage of construction in accordance with Part 3, **9.9.5**.

In addition to the limitations stated in Part 3, **9.9.5.4**, a) and b), the total accumulated stress should not exceed:

a) $\dfrac{0.5 f_{\mathrm{cu}}}{\gamma_{\mathrm{f3}}}$ for concrete in compression; or

b) $\dfrac{0.87 f_{\mathrm{ry}}}{\gamma_{\mathrm{f3}}}$ for reinforcement in tension.

In the application of Part 3, **9.9.5.5** and **9.9.5.6**, where the fibre being considered is either concrete in compression or reinforcement in tension, the value of M, $M_{\mathrm{x,\,max}}$, M_{Dx}, $M_{\mathrm{y,\,max}}$ and $M_{\mathrm{D,\,max}}$ should be based on Z_{xr}, Z_{xs} and M_{ult}, as defined in **6.2.3**.

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 and disregarding the effects of shear lag.

6.3.2 ***Deck slab and haunches***

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**, **6.3.2** and **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. Where separate ultimate limit state checks are necessary for shear connectors, reference should be made to **6.3.4**.

Special consideration should be given to details that are not in accordance with **5.3**, **6.3.1**, **6.3.2** and **6.3.3**.

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**, **6.3.1** and **6.3.3** 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**, **6.3.3.3**, **6.3.3.4** and **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 conforms in all other respects with the definition of A_b;

NOTE 1 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 the surface of each shear connector that 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 5 including typical arrangements of reinforcement that satisfy the definitions of A_b, A_t and A_{bs}.

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

NOTE 2 For planes in unhaunched beams that do not cross the whole thickness of the slab (plane type 2-2 in Figure 5), $A_e = 2A_b$.

For planes that cross the whole depth of the slab (shear plane type 1-1 in Figure 5), A_e is the total area of fully anchored reinforcement intersected by that plane, including reinforcement provided for flexure, e.g. in shear plane type 1-1 in Figure 5a), $A_e = A_t + A_b$.

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 5), 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 the 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 5d)) 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) The definitions of L_s, f_{ry}, f_{cu} and s are as follows:

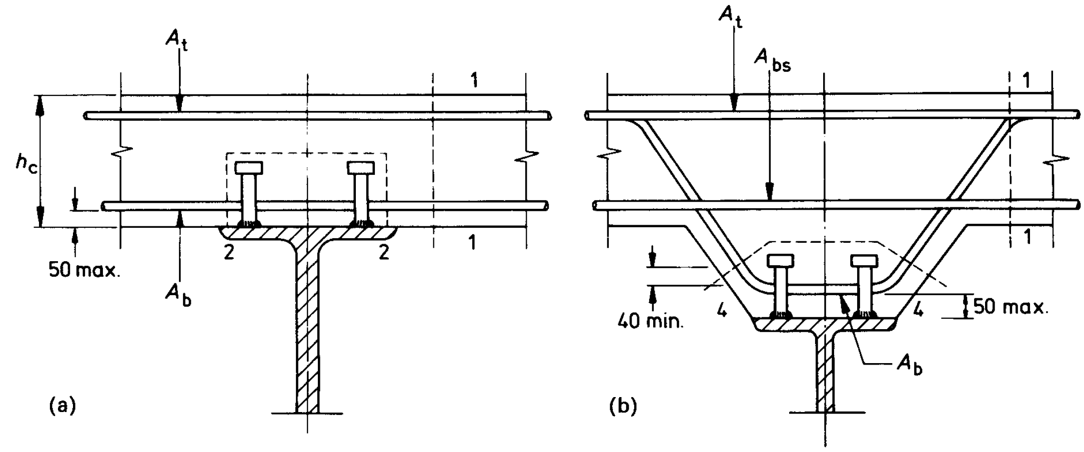
L_s is the length of the shear plane under consideration;

f_{ry} is the characteristic yield strength of the transverse reinforcement but not greater than 500 N/mm^2;

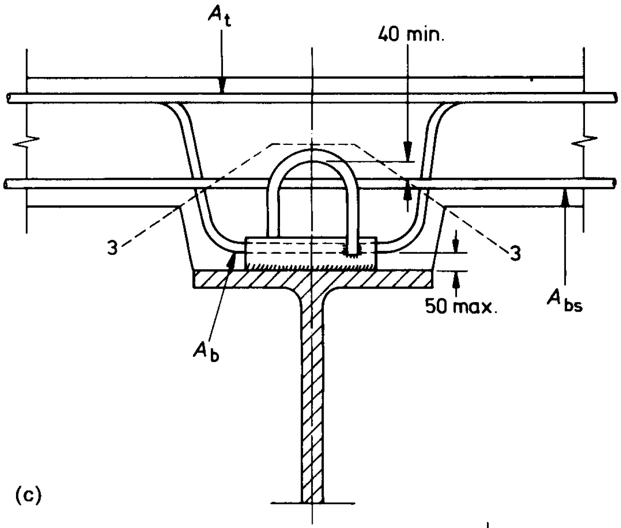
f_{cu} is the characteristic cube strength of concrete or the cube strength used in the design of the slab, if account is taken of loading at ages other than 28 days, but not greater than 45 N/mm^2;

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

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 %.

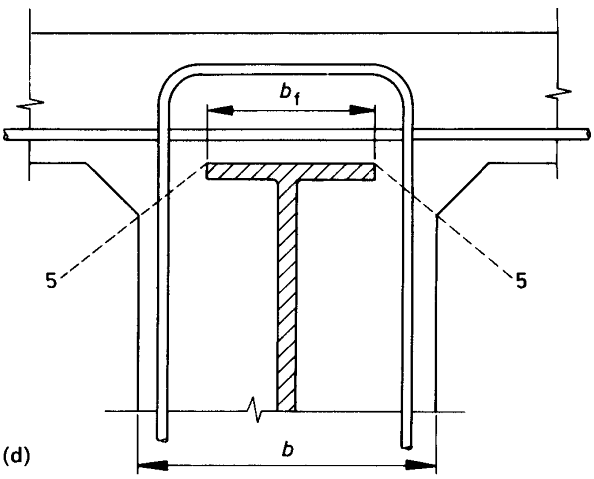


(a) (b)



(c)

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)



(d)

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

Figure 5 — Shear planes and transverse reinforcement

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 the lesser of the following:

a) $k_1 f_{cu} L_s$

b) $v_1 L_s + 0.7 A_e f_{ry}$

where

k_1 is a constant equal to 0.15 for normal density concrete and 0.12 for lightweight aggregate concrete;

v_1 is the ultimate longitudinal shear stress in the concrete for the shear plane under consideration, to be taken as 0.9 N/mm^2 for normal density concrete and 0.7 N/mm^2 for lightweight aggregate concrete.

If f_{cu} is taken to be less than 20 N/mm^2, the term $v_1 L_s$ in b) should be replaced by $k_2 f_{cu} L_s$ where k_2 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 b) in respect of shear planes through the haunch (planes 3-3 and 4-4 in Figure 5), should be bottom reinforcement that conforms to 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 5). This should be done by checking in accordance with **6.3.3.2**b) replacing expression b) with:

$v_1 L_s + 1.4\ A_{bv} f_{ry}$

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 5), by replacing **6.3.3.2**b) by

$q_p < v_1 L_s + 0.7\ A_e f_{ry} + 1.6\ F_T$

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 that the reinforcement required to satisfy **6.3.3.2** is reinforcement that satisfies the definition of A_{bv} and the haunch dimensions satisfy the recommendations of **6.3.2**.

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 5) 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 sh_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 $1.4sh_c$.

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 5).

6.3.3.6 *Curtailment of transverse reinforcement*

The transverse reinforcement provided to resist longitudinal shear may be curtailed provided that the recommendations of **6.3.3** are satisfied in all respects for the shear planes through the slab of type 1-1 shown in Figure 5. 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 reinforcement 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 **5.3.3.6**, **6.1.3** or where redistribution of stresses from the tension flange is carried out in accordance with Part 3. Then the size and spacing of shear connectors should be determined in accordance with **5.3.3.5** 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

$$P_u / \gamma_m$$

where

P_u is the nominal strength as defined in **5.3.2.1** or **5.3.2.2**; and,

γ_m = 1.40

7 Composite box girders

7.1 General

In addition to the recommendations made in this part, the design of composite box girders should satisfy the relevant recommendations for steel box girders given in Part 3.

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 for serviceability limit state calculations should be determined in accordance with Part 3. 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 Part 3.

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 **5.3.3.3** and **5.3.3.6**.

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.

In open-top box girders the spacing of shear connectors should be as given in **5.3.3.1** for composite I beams.

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** in respect of the serviceability limit state, except that in closed box girders the number of shear connectors needed in order to satisfy **5.3.3.4** 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

$$Q_x = \frac{q}{n}\left[K\left(1-\frac{x}{b_w}\right)^2 + 0.15\right] \qquad [6]$$

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 6;
- 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;
- 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);
- n' is the number of connectors per unit length placed within 200 mm of the centre line of the web considered.

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**.

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 3 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.

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.

In addition to its effect on the global distribution of moments and shear forces, the cracking may also need to be taken into account when assessing the torsional resistance of the particular section.

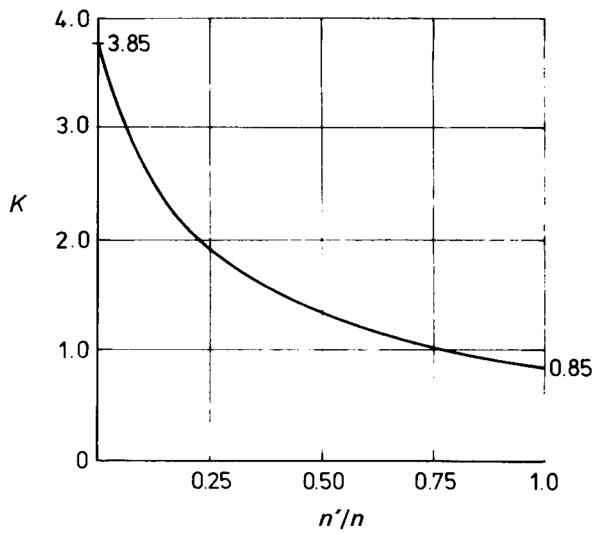


Figure 6 — Coefficient *K*

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 **5.3.3.6** 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 considerations do not apply, 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. Full 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**.

The longitudinal shear forces due to local wheel loads in the regions of a composite plate supported by cross-members may be determined by considering the plate as an equivalent simply supported beam spanning these cross-frames; the width of the equivalent beam, b, supporting the wheel load should be taken as:

$$b = \frac{4}{3}x + l_w \tag{7}$$

where

x is the distance from centroid of wheel patch to the nearest cross-frame;

l_w is the length of wheel patch which is parallel to cross-frame.

8 Cased beams and filler beam construction

8.1 General

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 beam decks should be designed for the serviceability and ultimate limit states in accordance with Clauses **4**, **5** and **6**.

8.3 Analysis of structure

8.3.1 *General*

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 deck as an orthotropic plate or by the method given in **8.3.2**.

8.3.2 *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 200 mm^2 in the top of the slab.

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

$$M_y = (0.95 - 0.04l)\, M_x \alpha_L \tag{8}$$

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 of the beams, 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

The moments of resistance of cased and filler beams should be checked in accordance with **5.2** and **6.2** at the serviceability and ultimate limit states respectively. For this purpose a beam should be considered as compact provided that any part of the steel section not encased in concrete satisfies the criteria given in Part 3. Vertical shear should be assumed to be resisted by the steel section alone and the effects of shear lag in filler beam decks may be disregarded at the serviceability limit state.

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 disregarded. Shear lag effects may be disregarded 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 that the local bond stress nowhere exceeds 0.5 N/mm^2 in cased beams of 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 5d)). 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

$(q_p - v_1, L_s) / 0.7\, f_{ry}$

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 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**.

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:

$$\Phi \geq \sqrt{s_b s \frac{b}{f_{ry}}} \qquad [9]$$

where

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 **5.2.6**) shows that a greater spacing is acceptable.

8.7.3 *Filler beam*

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, disregarding 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 5-5 (Figure 5d)), should be not less than:

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

where

L_s is as defined in Figure 5);

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 that have previously been 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 listed 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. Connection between the permanent formwork and the in situ concrete should be adequate to prevent separation during the life of the bridge.

9.5.3 ***Steel 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. Particular attention should be paid to the following aspects of any design:

a) fatigue behaviour;

b) durability;

c) bond between permanent formwork and concrete slab both under long term and under impact loading;

d) corrosion protection.

9.6 Special provisions for precast concrete or composite precast concrete permanent formwork

9.6.1 ***Design***

Precast concrete units should conform to the recommendations made in the relevant clauses of 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 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 conforming to the requirements of BS 4395-1 are used in accordance with BS 4604-1. 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 criteria: static loading

10.2.1 ***Serviceability limit state***

The longitudinal shear resistance per unit length developed by friction between the concrete slab and the 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 that 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, provided that the method of tightening conforms to the requirements of that British Standard. 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. When checking for the possibility of longitudinal shear failure through the depth of the slab, in accordance with **6.3.3**, it should be noted that the presence of pockets for the bolts reduce the length of the effective shear plane.

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 slab and that suitable washers or bearing plates are provided to spread the loads from the bolts in order to prevent the concrete underneath being crushed. Where the slab is precast it may be necessary to provide suitable bedding material between the slab and the steel beam. Except in respect of the recommendations made in **9.1**, the interface should be free of paint or other applied finishes, oil, dirt, loose rust, loose mill scale, burrs and other defects that would prevent a uniform seating between the two elements or would interfere with the development of friction between them.

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 slab, although this can be difficult to achieve while at the same time maintaining adequate cover around the bolt hole. The detail around the bolt hole needs careful attention to ensure that local crushing forces on the concrete are not increased by loads being directly transmitted via the bolt head. Care also has to be taken to ensure that forces and moments can be adequately transmitted across the joints between adjacent pre-cast units, and that no gaps are left, or may occur in the course of time, where corrosion could take place between the flange and the concrete slab.

11 Composite columns

11.1 General

11.1.1 *Principles*

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 S275 or S355 steel that conforms to the requirements of BS 4-1 and BS EN 10025; or

b) a symmetrical I-section fabricated from grade S275 or S355 steel conforming to BS EN 10025.

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

1) be a symmetrical box section fabricated from grade S275 or S355 steel complying with BS EN 10025; or

2) conform to BS EN 10210; 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 the 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

$$\alpha_c = \frac{0.45 A_c f_{cu}}{N_u} \qquad [10]$$

and the squash load N_u is given by:

$$N_u = 0.95\, A_s f_y + 0.87\, A_r f_{ry} + 0.45\, A_c f, \qquad [11]$$

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

In the previous expressions,

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. Full 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 *General*

The semi-empirical method of analysis given in **11.2.2.2**, **11.2.2.3**, **11.2.2.4**, **11.2.2.5** and **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 l 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 in the plane of bending of the restrained column. This equivalent column should normally be subjected to the same end loads and end moments as the restrained column. However, where the column is free to sway, it should always be considered to be in single curvature bending. The smaller end moment in a particular plane should be 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 Part 4, Table 11.

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:

$$\lambda = \frac{l_e}{L_e} \text{ and} \qquad [12]$$

$$l_E = \pi \left[\frac{0.45 E_c I_c + 0.95 E_s I_s + 0.87 E_r I_r}{N_u} \right]^{0.5} \qquad [13]$$

I_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 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 denote 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***

11.3.2.1 *General*

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 **11.3.2.2** 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**.

11.3.2.2 *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:

$$N_{ay} = 0.85\, K_{1y} N_u \qquad [14]$$

where

K_{ly} 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 **11.3.2.1**.

11.3.2.3 *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 $0.03b$, where b is the least lateral dimension of the column.

11.3.3 *Column 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 **A.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:

$$N_y = N_u \left[K_{1y} - \left(K_{1y} - K_{2y} - 4K_3 \right) \frac{M_y}{M_{uy}} - 4K_3 \left(\frac{M_y}{M_{uy}} \right)^2 \right] \qquad [15]$$

where

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

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

K_3 is determined from **A.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 **A.4**, is 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:

$$N_x = N_u \left[K_{1x} - (K_{1x} - K_{2x} - 4K_3)\frac{M_x}{M_{ux}} - 4K_3\left(\frac{M_x}{M_{ux}}\right)^2 \right] \qquad [16]$$

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 **A.1** and **A.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 equation [17] (**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_y, which is given by:

$$\frac{1}{N_{xy}} = \frac{1}{N_x} + \frac{1}{N_y} - \frac{1}{N_{ax}} \qquad [17]$$

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 **A.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:

$$\alpha_c = \frac{0.45 f_{cc} A_c}{N_u} \qquad [18]$$

$$N_u = 0.95\, f'_y A_s + 0.45 f_{cc} A_c \qquad [19]$$

where

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

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

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

$f'y = C_2\, f_y$

C_1 and C_2 are constants given in Table 3;

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 **11.1.4**.

Table 3 — Values of constants C_1 and C_2 for axially loaded concrete filled circular hollow sections

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 of deflections in a composite section. The engineer should describe, on the drawings, the particular sequence or method of construction on which the 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 1.

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 that 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 $0.5f_{ci}$, 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 made in 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.

Annex A (normative) Formulae and tables for the design of composite columns

A.1 Coefficient K_1

Values of the coefficient K_l used with the additional subscripts x or y to describe the plane of bending may be determined from Part 3, **10.6**. The value of K_l should be taken as the value of σ_c/σ_y determined in accordance with Part 3, Figure 37 for a value of the slenderness parameter

$\frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}}$ given by 75.5λ, where λ is defined in **11.3.1** of this part.

A.2 Coefficient K_2

A.2.1 *General*

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 **A.2.2** or **A.2.3**, as appropriate, between the following limits:

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

$$K_{20} \le 0.75$$

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

A.2.2 *Concrete-filled circular hollow sections*

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

$K_{20} = 0.9\alpha_c^2 + 0.2$ 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 appropriately denoted by λ_x, λ_y respectively.

A.2.3 ***Concrete-encased steel sections and concrete-filled rectangular hollow sections***

$$\frac{K_2}{K_{20}} = \left[\frac{90 - 25(2\beta - 1)(1.8 - \alpha_c) - C_4\lambda}{30(2.5 - \beta)}\right] \quad [21]$$

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;

the appropriate curve being selected in accordance with Part 3, Figure 37 and the remaining terms are as given in **A.2.2**.

A.3 Coefficient K_3

A.3.1 ***Concrete-filled circular hollow sections***

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

$$K_3 = K_{30} + \frac{\left[(0.5\beta + 0.4)\left(\alpha_c^{\,2} - 0.5\right) + 0.15\right]C_5\lambda}{1 + \left(C_5\lambda\right)^3} \quad [22]$$

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 **A.2.2**;

C_5 is a constant which may be taken as 1.0.

A.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 might 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:

$K_3 = 0.425 - 0.075\beta - 0.005\ C_4\lambda$ 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 **A.2.2** and **A.2.3**.

A.4 Ultimate moment of resistance, M_u, of composite columns

A.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 / γ_m;

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 stressed 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 **A.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 disregarded in calculating the compressive force.

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

A.4.2 *Equations for calculating* M$_u$ *for concrete-encased steel sections and concrete-filled rectangular hollow sections*

A.4.2.1 *General*

The ultimate moment of resistance may be calculated from the equations given in **A.4.2.2**, **A.4.2.3**, **A.4.2.4**, **A.4.2.5** and **A.4.2.6**, 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 $0.4f_{cu}/0.95f_y$;

f_{cu} is the characteristic 28-day cube strength of concrete;

f_y is the nominal yield strength of steel.

A.4.2.2 *Concrete-encased steel sections: plastic neutral axis outside the steel section*

(see Figure A.1a)).

This condition arises when:

$\rho b d_s > A_s$ then

$$d_c = A_s / b\rho \qquad [23]$$

$$M_{ux} = 0.95 f_y A_s \frac{(h - d_c)}{2} + 0.87 f_{ry} \frac{Ar}{2} d_r \qquad [24]$$

$$M_{uy} = 0.95 f_y A_s \frac{(b - d_c)}{2} + 0.87 f_{ry} \frac{Ar}{2} d_r \qquad [25]$$

where

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 in **A.4.2.1**.

A.4.2.3 *Cased sections: plastic neutral axis within top flange/major axis bending* (see Figure A.1b)).

This condition arises when:

$\rho b d_s < A_s$ and

$0.95 A_s f_y \leq 0.4 f_{cu} \, [b d_s + t_f(b - b_f)] + 1.82 A_f f_y$ then

$d_c = (A_s + 2 b_f d_s) / (b\rho + 2 b_f)$ and [26]

$$M_u = 0.95 f_y \left[A_s \frac{(h - d_c)}{2} - b_f d_s (d_c - d_s) \right] + 0.87 f_{ry} \frac{Ar}{2} d_r \qquad [27]$$

where

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 **A.4.2.1** and **A.4.2.2**.

A.4.2.4 *Cased sections: plastic neutral axis in web/major axis bending* (see Figure A.1c)).

This condition arises when:

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

$$d_c = \frac{h t_w}{(b\rho + 2 t_w)} \text{ and} \qquad [28]$$

$$M_u = 0.95 f_y \left[A_s \frac{(h - d_c)}{2} - b_f t_f (d_c - d_s) - t_w d_w (d_c - d_w) \right] + 0.87 f_{ry} \frac{A_r}{2} d_r \qquad [29]$$

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 **A.4.2.1**, **A.4.2.2** and **A.4.2.3**.

A.4.2.5 *Cased sections: plastic neutral axis in flanges/minor axis bending* (see Figure A.1d)).

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 [30]

$$M_u = 0.95 f_y \left[A_s \frac{(h - d_c)}{2} - 2 t_f d_s (d_c - d_s) \right] + 0.87 f_{ry} \frac{Ar}{2} d_r \qquad [31]$$

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

A.4.2.6 *Concrete-filled rectangular hollow sections*

(see Figure A.1e)).

$$d_c = (A_s - 2b_f t_f) / (b\rho + 4t_f) \text{ and} \qquad [32]$$

$$M_u = 0.95 f_y \left[A_s \frac{(h - d_c)}{2} + b_f t_f (t_f + d_c) \right] \qquad [33]$$

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

A.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:

$$M_u = 0.95\, S f_y\, (1 + 0.01m) \qquad [34]$$

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 A.2

where

D_e is the outside diameter of the steel section;

t is the wall thickness;

ρ is as defined in **A.4.2.1**.

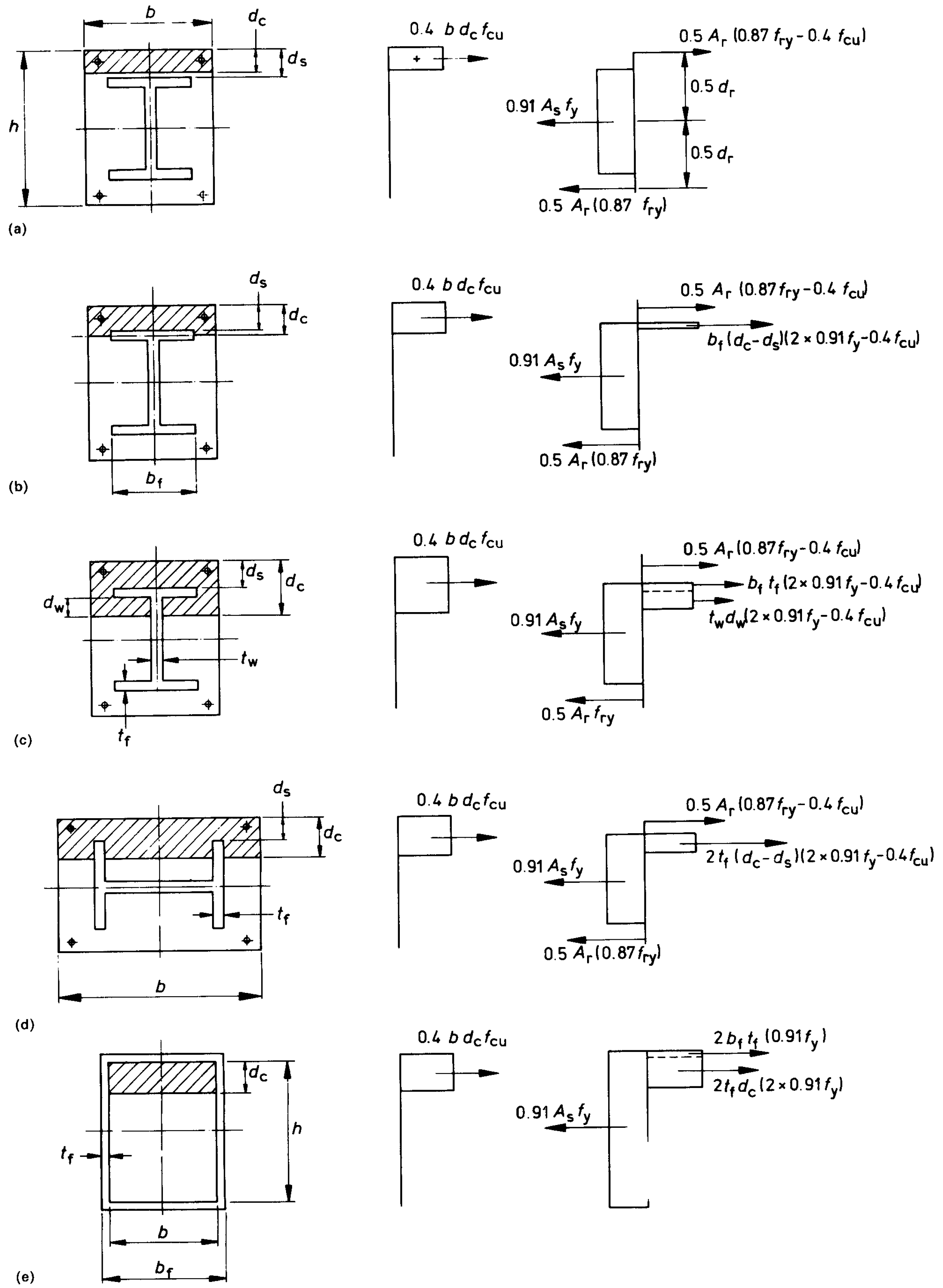


Figure A.1 — Force diagrams for calculating M_u

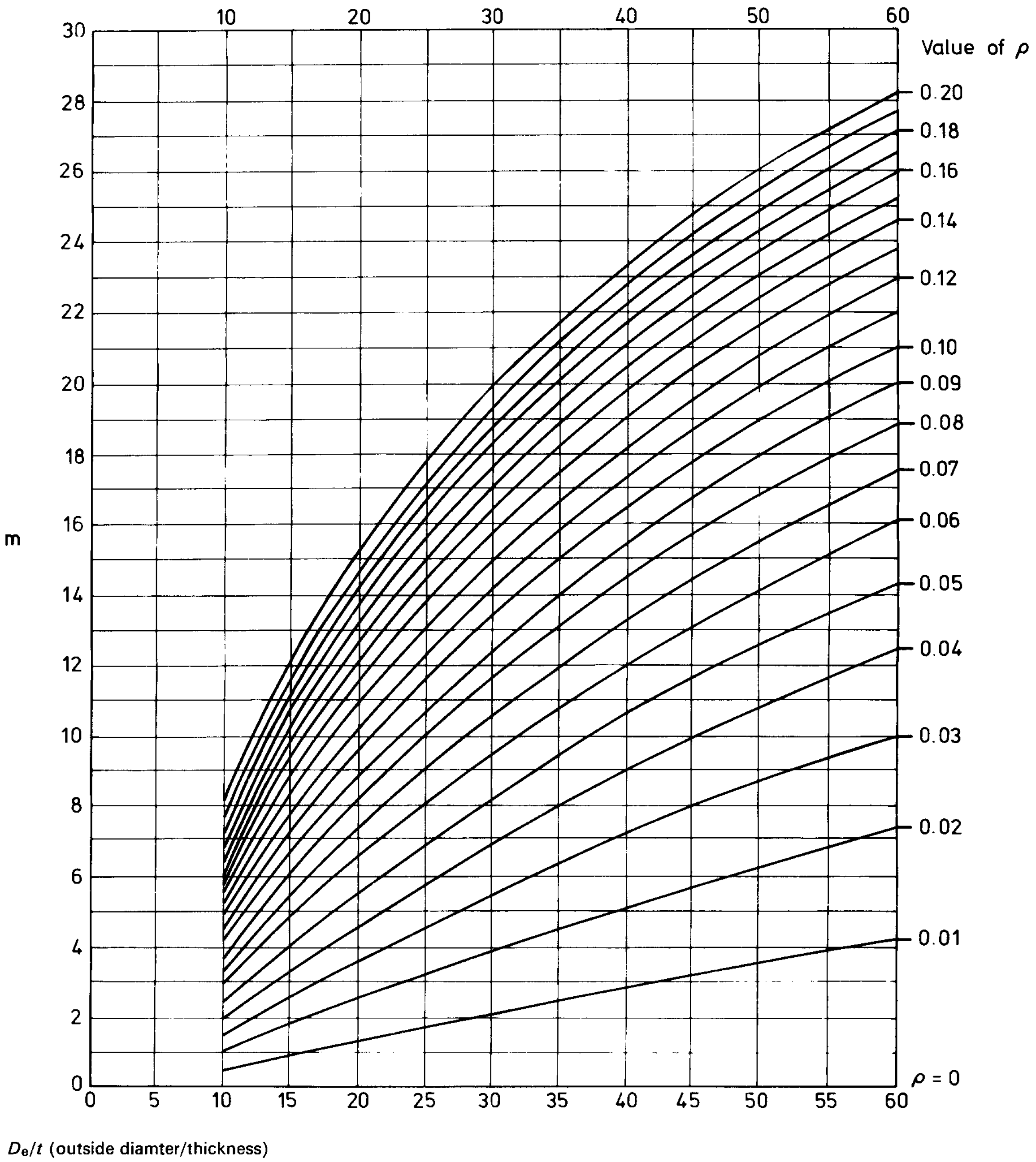


Figure A.2 — Chart for evaluating M_u of concrete filled circular hollow section

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