

Steel, concrete and composite bridges —

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

ICS 93.040

Committees responsible for this British Standard

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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 It supersedes BS 5400-5:1979 which is withdrawn.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 5400-2:2005¹⁾, *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 γ_{fL} γ_{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
$\gamma_{f1}, \gamma_{f2}, \gamma_{f3}$	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:

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

where R^* is the design resistance.

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

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:

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

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, 6.4.3²⁾, 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 $1\,400\text{ kg/m}^3$ 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 $1\,400\text{ kg/m}^3$ 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 $\pm 20\%$. 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 $\gamma_m = 1.85$.