BRITISH STANDARD

BS 5950-3.1:
1990

Structural use of steelwork in building —

Part 3: Design in composite construction —

Section 3.1 Code of practice for design of simple and continuous composite beams

UDC [624.014.2:693.814+624.072.2]:669.14.018.291

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Civil Engineering and Building Structures Standards Policy Committee (CSB/-) to Technical Committee CSB/27, upon which the following bodies were represented:

British Constructional Steelwork Association Ltd.
British Railways Board
British Steel Industry
Department of the Environment (Building Research Establishment)
Department of the Environment (Housing and Construction Industries)
Department of the Environment (Property Services Agency)
Health and Safety Executive
Institution of Civil Engineers
Institution of Structural Engineer
Royal Institute of British Architects
Steel Construction Institute
Welding Institute

The following bodies were also represented in the drafting of the standard, through subcommittees and panels:

Concrete Society
Society of Engineers Incorporated

This British Standard, having been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee, was published under the authority of the Board of BSI and comes into effect on 31 July 1990

The following BSI references relate to the work on this standard:
Committee reference CSB/27
Draft for comment 88/15145 DC

ISBN 0 580 18342 4

Amendments issued since publication

Amd. No.	Date of issue	Comments

Contents

Foreword

This Section of BS 5950 has been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee. BS 5950 is a document combining codes of practice to cover the design, construction and fire resistance of steel structures and specifications for materials, workmanship and erection.

It comprises the following Parts:

— *Part 1: Code of practice for design in simple and continuous construction: hot rolled sections;*

— *Part 2: Specification for materials, fabrication and erection: hot rolled sections;*

— *Part 3: Design in composite construction;*

— *Section 3.1: Code of practice for design of simple and continuous composite beams*;

— *Section 3.2:* [1]*Code of practice for design of composite columns and frames;*

— *Part 4: Code of practice for design of floors with profiled steel sheeting;*

— *Part 5: Code of practice for design of cold formed sections;*

— *Part 6:* [1]*Code of practice for design of light gauge sheeting, decking and cladding;*

— *Part 7:* [1]*Specification for materials and workmanship: cold formed sections;*

— *Part 8: Code of practice for fire resistant design;*

— *Part 9:* [1]*Code of practice for stressed skin design.*

For the present, Part 3 has been subdivided. Section 3.1 gives recommendations for the design of simple and continuous composite beams. It supersedes CP 117-1 which is withdrawn. Section 3.2 covers the design of composite columns and frames.

The full list of organizations who have taken part in the work of the Technical Committee is given on the back cover. The Chairman of the Committee is Mr P R Brett and the following people have made a particular contribution in the drafting of the code.

Mr P A Rutter, Chairman of Drafting Committee

Mr H Gulvanessian

Mr G T Harding

Mr J H R Haswell

Mr J I Hardwick

Prof. R P Johnson

Dr R M Lawson

Dr D B Moore

Mr J Morrison

Dr G W Owens

Mr J Rushton

Mr P R Salter

Mr J Seifert

Mr J C Taylor

Mr A D Weller

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people and that construction and supervision should be carried out by capable and experienced organizations.

[1] In preparation

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 36, an inside back cover and a back cover.

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

Section 1. General

1.0 Introduction

1.0.1 Aims of economical structural design

The aim of structural design is to provide, with due regard to economy, a structure capable of fulfilling its intended function and sustaining the design loads for its intended life. The design should facilitate fabrication, erection and future maintenance.

The structure should behave as one three-dimensional entity. The layout of its constituent parts, such as foundations, steelwork, connections and other structural components, should constitute a robust and stable structure under normal loading to ensure that, in the event of misuse or accident, damage will not be disproportionate to the cause.

To achieve this it is necessary to define clearly the basic structural anatomy by which the loads are transmitted to the foundations. Any features of the structure which have a critical influence on its overall stability can then be identified and taken account of in design.

Each part of the structure should be sufficiently robust and insensitive to the effects of minor incidental loads applied during service that the safety of other parts is not prejudiced.

While the ultimate strength recommendations within this standard are to be regarded as limiting values, the purpose in design should be to reach these limits in as many parts of the structure as possible, to adopt a layout such that maximum structural efficiency is attained and to rationalize the steel member sizes and details in order to obtain the optimum combination of material and fabrication.

1.0.2 Overall stability

The designer responsible for the overall stability of the structure should ensure the compatibility of design and details of parts and components. There should be no doubt of this responsibility for overall stability when some or all of the design and details are not made by the same designer.

1.0.3 Accuracy of calculation

For the purpose of deciding whether a particular recommendation is complied with, the final value, observed or calculated, expressing the result of a test or analysis should be rounded off. The number of significant places retained in the rounded off value should be the same as for the value given in the recommendation.

1.1 Scope

This Section of BS 5950 gives recommendations for the design of simply supported and continuous composite beams, comprising hot rolled steel sections, plate girders and hollow sections acting compositely with reinforced concrete slab, or with a composite slab complying with BS 5950-4.

This Section of BS 5950 does not cover the design of composite columns or composite frames, for which reference should be made to Section 3.2.[2)]

NOTE The publications referred to in this standard are listed on the inside back cover.

1.2 Definitions

For the purposes of this Part of BS 5950, the definitions given in BS 5950-1 and BS 8110 apply, together with the following.

1.2.1
composite action

the structural interaction which occurs when two elements are inter-connected along their length, so as to modify the behaviour of the individual elements. The inter-connection may be continuous or at discrete points along the member

1.2.2
composite section

a steel beam which acts compositely with a concrete flange

1.2.3
composite slab

a slab consisting of profiled steel sheets, a concrete slab and reinforcement where necessary. The design of the slab may be composite or non-composite with the profiled sheeting

1.2.4
concrete flange

the structural concrete slab forming part of a floor or roof of the structure and acting compositely with the steel beam. The slab may be of precast, in-situ or composite construction

1.2.5
full shear connection

shear inter-connection between steel and concrete elements sufficient to produce full composite action, permitting only negligible slip at the interface and developing the full moment capacity of the composite cross section

[2)] In preparation.

1.2.6
global analysis

analysis of the structure to determine the internal forces and moments in the members

1.2.7
negative moment

bending moment causing "hogging", i.e. moment causing compression at the bottom of a beam

1.2.8
partial shear connection

shear inter-connection between steel and concrete elements producing less composite action than full shear connection, permitting some limited slip at the interface and developing a reduced moment capacity, less than the full moment capacity of the composite cross section

1.2.9 positive moment

bending moment causing "sagging", i.e. moment causing tension at the bottom of a beam

1.2.10
profiled steel sheeting

cold formed steel sheet profiled to increase its second moment of area in one direction

1.2.11
resistance

limit of force which an element of a member can withstand

1.2.12
shear connector

a mechanical device producing interaction between steel and concrete

1.2.13
solid slab

a concrete flange which, at least in the zone surrounding the shear connectors, has a flat soffit and no internal voids

1.3 Major symbols

For the purposes of this Part of BS 5950, the following symbols are used.

A	Area of steel section
A_c	Area of concrete
A_{cv}	Area of concrete shear surface per unit length of beam
A_{sv}	Area of transverse reinforcement per unit length of beam
B_e	Total effective breadth of concrete flange
b_e	Effective breadth of concrete flange, one side of steel beam
b_r	Breadth of the concrete rib
D	Depth of steel section
D_p	Overall depth of profiled steel sheet
D_s	Overall depth of slab
d	Clear depth of web *or* Nominal shank diameter of a stud shear connector
f_{cu}	Characteristic cube strength of concrete (in N/mm^2)
f_y	Characteristic strength of reinforcement
h	Overall height of stud
I_g	Second moment of area of uncracked composite section
I_n	Second moment of area of cracked section for negative moments
I_p	Second moment of area of cracked section for positive moments
I_x	Second moment of area of steel beam about major axis
k	Reduction factor depending on profile shape
L	Length of span
L_z	Distance between points of zero moment
M	Moment
M_c	Moment capacity of composite section
M_s	Moment capacity of steel beam
N	Number of shear connectors in a group
N_a	Actual number of shear connectors between intermediate point and the adjacent support
N_i	Number of shear connectors required between intermediate point and the adjacent support
N_n	Number of shear connectors for negative moments
N_p	Number of shear connectors for positive moments
p_y	Design strength of structural steel (in N/mm^2)
Q_k	Characteristic resistance of shear connector
Q_n	Capacity of shear connector in negative moment regions
Q_p	Capacity of shear connector in positive moment regions

R_c	Resistance of concrete flange
R_f	Resistance of steel flange
R_n	Resistance of slender steel beam
R_o	Resistance of slender web
R_q	Resistance of shear connection
R_r	Resistance of reinforcement
R_s	Resistance of steel beam
R_v	Resistance of clear web depth
R_w	Resistance of overall web depth
r	Ratio of mean longitudinal stress in the web to p_y
s	Longitudinal spacing centre-to-centre of groups of shear connectors
t	Web thickness
v	Longitudinal shear per unit length
v_p	Contribution of profiled steel sheeting per unit length
v_r	Shear resistance of concrete flange per unit length
α_e	Modular ratio
δ	Deflection
ε	Constant, equal to $(275/p_y)^{½}$

1.4 Design procedure

The overall design procedure for steel structures incorporating composite construction should be in accordance with BS 5950-1, except as modified and supplemented by the recommendations of Part 3.

The detailed design of the structural steel components should be as recommended in BS 5950-1, modified as recommended in Part 3 when acting compositely.

In the design of composite construction in accordance with this Part of BS 5950, the recommendations of BS 5950-1 and of BS 8110 also apply, unless modified by this Part.

Reference should also be made to BS 5950-8 for recommendations concerning fire protection.

The requirements of BS 5950-2 should also be considered as applying equally to steelwork designed to act compositely as recommended in this Part. In addition guidance on additional aspects of construction is given in Appendix A. Particular attention should be paid to the need to provide temporary lateral restraint to the structure during construction, prior to the development of composite action.

Composite slabs used compositely with steel beams should be designed and constructed in accordance with BS 5950-4.

Reinforced concrete slabs, including precast slabs, used compositely with steel beams should be designed and constructed as recommended in BS 8110.

Section 2. Limit state design

2.1 General principles and design methods

2.1.1 Limit state concept

Structures should be designed by considering the limit states at which they would become unfit for their intended use, by applying appropriate factors for the ultimate limit state and the serviceability limit state.

All limit states covered in BS 5950-1 or in BS 8110 should be considered.

The recommendations given in this Part should be followed in respect of the ultimate limit states of strength and stability and the serviceability limit states of deflection, cracking and vibration.

2.1.2 Methods of design

2.1.2.1 *General.* The design of any structure or its parts should be carried out by one of the methods given in **2.1.2.2** to **2.1.2.5**.

In all cases, the details of members and connections should be such as to realize the assumptions made in design without adversely affecting any other parts of the structure.

2.1.2.2 *Simple design.* This method is described in **2.1.2.2** of BS 5950-1:1985.

2.1.2.3 *Rigid design.* This method is described in **2.1.2.3** of BS 5950-1:1985.

2.1.2.4 *Semi-rigid design.* This method is described in **2.1.2.4** of BS 5950-1:1985.

2.1.2.5 *Experimental verification.* The recommendations given in **2.1.2.5** of BS 5950-1:1985 should also be considered as applicable to composite beams. However, although the test procedures for steel structures given in section seven of BS 5950-1:1985 are generally applicable, further consideration should also be given to the special features of composite construction.

2.1.3 Methods of analysis

2.1.3.1 *General.* A distinction is made between the global analysis, by which the moments and forces in the structure are determined, and the procedures for member design.

2.1.3.2 *Global analysis.* The moments and forces in the members of any structure should be determined by elastic or plastic global analysis. Elastic global analysis may be used without restriction but plastic global analysis should only be used in structures where the members satisfy the necessary criteria (see **5.2.4** and **D.3**).

2.1.3.3 *Member design.* The plastic moment capacity (see **4.4.2**) should be used for member design provided that the compression flange is class 1 plastic or class 2 compact (see **4.5.3**), otherwise the elastic moment capacity (see **4.4.3**) should be used.

NOTE Plastic design of members is recommended irrespective of whether the global analysis is elastic or plastic.

2.2 Loading

2.2.1 General

All relevant loads should be considered separately and in such realistic combinations as to cause the most critical effects on the elements and the structure as a whole.

The following should be taken into account.

a) The magnitude and frequency of fluctuating loads should be considered.

b) Loading conditions during erection should receive particular attention.

c) The possible adverse effects of settlement of supports may need to be taken into account.

d) When the values of modular ratio recommended in **4.1** are adopted, it is not necessary to give further consideration to the effects of creep.

e) It is not necessary to consider stresses due to shrinkage.

f) When the procedures recommended in **6.1** are adopted, it is not necessary to consider the effects of shrinkage on deflection.

g) For internal steelwork, it is not necessary to consider temperature effects within the range quoted in BS 5950-1.

2.2.2 Dead, imposed and wind loading

Reference should be made to BS 6399-1, CP 3:Chapter V-2, and BS 6399-3 for the determination of the dead, imposed and wind loads.

2.2.3 Construction loads and temporary storage loads

Construction loads should be considered in addition to the nominal weight of the wet concrete slab (see **2.3.2**).

The construction load on the area supported by the beam should be taken as not less than 0.5 kN/m^2. An alternative construction load comprising a moveable point load of not less than 4 kN should also be considered. Local effects due to this point load need not be considered.

NOTE 1 See BS 5950-4 for construction loads to be assumed in the design of composite slabs.

NOTE 2 Allowance is made within these values for construction operatives, impact and heaping of concrete during placing, hand tools, small items of equipment and materials for immediate use. The minimum values quoted are intended for use in general-purpose working areas, but will not necessarily be sufficient for excessive impact or excessive heaping of concrete, or pipeline or pumping loads.

Where excessive loads are expected, reference should be made to BS 5975.

Where materials to be stored on the structure during erection (or on a recently formed slab before it has attained sufficient strength to act compositely with the steel beam) may produce loads in excess of the construction loads, provision should be made in design for the additional loading (see **2.3.2**).

2.3 Ultimate limit states

2.3.1 General

In checking the strength and stability of the structure the loads should be multiplied by the relevant γ_f factors given in BS 5950-1. The factored loads should be applied in the most unfavourable realistic combination for the part or effect under consideration.

The load capacity of each member and its connections, as determined by the relevant provisions of this standard, should be such that the factored loads would not cause failure.

2.3.2 Construction stage

Account should be taken of probable variations in dead load during construction or other temporary conditions. For the purpose of assigning γ_f factors, construction loads and temporary storage loads should be treated as imposed loads.

Strength and stability should also be checked for the construction stage where the steel beam acts non-compositely to support the dead load of the formwork and wet concrete plus construction loads or temporary storage loads (see **2.2.3**).

Profiled steel sheeting which is fixed to a beam as recommended in **A.2.3** should be assumed to give lateral restraint to the flange to which it is fixed, provided that the ribs run perpendicular to the beam or at an angle of at least 45° to the beam. Profiled steel sheeting with ribs running parallel to a beam, or at an angle of less than 45° to it, should not be assumed to give it lateral restraint.

Where the profiled steel sheeting is also assumed to act as a stressed skin diaphragm to provide structural stability at the construction stage, reference should be made to BS 5950-9.[3)]

NOTE **A.2** gives design considerations related to construction.

2.4 Serviceability limit states

2.4.1 General

The design recommendations for serviceability given in section 6 should be followed.

The serviceability loads should be taken as the unfactored values. Dead and imposed loads should be considered as recommended in the appropriate clause. The construction loads should not be included in the serviceability loads.

2.4.2 Deflection

The deflection under serviceability loads of a building or part should not impair the strength or efficiency of the structure or its components or cause damage to the finishings.

Reference should be made to BS 5950-1 for recommendations on limiting values for deflections under unfactored imposed loads.

Deflections at the construction stage due to the dead load of the concrete flange and the steel beam may also need to be considered when unpropped construction is used.

NOTE If necessary, precambering can be used to counteract deflections due to dead loads.

2.4.3 Irreversible deformation

To prevent gross deformations under normal service conditions, irreversible deformations should be avoided in simply supported beams and cantilevers, and in the mid-span regions of continuous beams.

The stresses based on the elastic properties of the section should be calculated under serviceability loading (see **6.2**). When unpropped construction is used, the stresses due to the dead load of the concrete flange and the steel beam should be based on the properties of the steel beam.

In simply supported beams and cantilevers, and in the mid-span regions of continuous beams, the stress in the extreme fibre of the steel beam should not exceed the design strength p_y and the stress in the concrete flange should not exceed 0.50 f_{cu}.

It is not necessary to limit the stresses over the supports of continuous beams, provided that the recommendations given in **6.1.3** and **6.2** are followed.

2.4.4 Cracking of concrete

Cracking of concrete should be controlled by attention to detail as recommended in **6.3**.

2.4.5 Durability

The relevant recommendations in BS 5950-1 and in BS 8110 should be followed as appropriate.

2.4.6 Vibration

To control vibration, the recommendations given in **6.4** should be followed.

3) In preparation.

Section 3. Materials

3.1 Structural steel

Structural steel complying with grades 43, 50 or WR50 of BS 4360 may be used. If other steels are used, due allowance should be made for variations in properties, including ductility.

The design strength p_y should be obtained by reference to BS 5950-1.

However, in the design of composite beams, p_y should not be taken as being more than 355 N/mm^2.

NOTE The limit of 355 N/mm^2 is due to lack of test evidence using higher-strength steels.

3.2 Concrete

Concrete should be in accordance with the recommendations given in BS 8110.

The nominal maximum size of aggregate should not exceed 20 mm.

For normal weight concrete, the grade specified should be in the range C30 to C50.

The dry density of lightweight aggregate structural concrete should normally be not less than 1 750 kg/m^3. The grade specified should be in the range C25 to C40.

Other densities can be used, but all references to lightweight concrete elsewhere in this Part of BS 5950 assume a dry density of at least 1 750 kg/m^3. Where lightweight concrete of less than 1 750 kg/m^3 dry density is used, due allowance should be made for variations in properties of concrete and their effect on the resistances of shear connectors.

NOTE 1 Information on the elastic modulus, creep coefficient, shrinkage coefficient and coefficient of thermal expansion for concrete may be obtained from BS 8110-2.

NOTE 2 The minimum grades are generally in line with the minimum grades recommended in BS 8110. They do not apply to existing structures.

3.3 Reinforcement

Reinforcement should comply with BS 4449, BS 4482 or BS 4483. Different types of reinforcement may be used in the same structural member.

To simplify calculation, the modulus of elasticity of reinforcement and the modulus of elasticity of structural steel may both be taken as 205 kN/mm^2.

3.4 Shear connectors

3.4.1 Stud shear connectors

The studs should be headed, with a minimum head diameter of 1.5d and a minimum depth of head of 0.4d, where d is the nominal shank diameter of the stud.

The stud material should be mild steel with minimum properties (in the cold drawn condition), when tested in accordance with BS 18, as follows:

ultimate tensile strength: 450 N/mm^2

elongation (on $5.65\sqrt{S_o}$ gauge length, as given in BS 18): 15 %.

NOTE These material properties relate to the steel from which the studs are manufactured. It is not possible at this time to give recommendations for the properties to be expected when testing finished studs.

3.4.2 Other types of shear connector

Where other types of shear connector are used, structural steel used for fabricated shear connectors should be in accordance with the recommendations given in **3.1**.

Friction grip bolts used as shear connectors should be in accordance with BS 4395-1.

Other materials may also be used for shear connectors provided that they can be demonstrated to produce shear connections possessing sufficient deformation capacity (see **5.4.2**).

3.5 Profiled steel sheets

Profiled steel sheets used to form composite slabs should be in accordance with BS 5950-4.

3.6 Concrete flange

3.6.1 General

A slab acting compositely with a steel member may be designed assuming it is supported by that member and capable of free rotation about it.

Any longitudinal reinforcement required in negative moment regions of a composite beam should be provided in addition to any required to reinforce the slab for moments due to loading acting directly on it.

3.6.2 Reinforced concrete slabs

A reinforced concrete slab should be designed for the effects of loading acting directly on it in accordance with BS 8110.

3.6.3 Composite slabs

A composite slab should be designed for the effects of loading acting directly on it in accordance with BS 5950-4.

3.6.4 Precast concrete slabs

A slab constructed using precast concrete units should be designed for the effects of loading acting directly on it in accordance with BS 8110. Where precast units are designed to act compositely with in-situ concrete, the interface of the precast and in-situ components should be designed to resist horizontal shear in both the longitudinal and the transverse direction.

Sufficient clearance should be provided to facilitate the placing and compaction of in-situ concrete around the shear connectors.

Where the effective continuity of transverse reinforcement is provided by means of overlapping horizontal U-bars, these U-bars should pass around the shear connectors.

Section 4. Section Properties

4.1 Modular ratio

The elastic section properties of composite members may be expressed in terms of an equivalent steel section by dividing the contributions of the concrete components by the effective modular ratio α_e.

The effective modular ratio α_e to be used in design should be determined from the proportions of the loading which are considered to be long term and short term, using the expression:

$$\alpha_e = \alpha_s + \rho l(\alpha_l - \alpha_s)$$

where

α_l is the modular ratio for long-term loading;
α_s is the modular ratio for short-term loading;
ρ_l is the proportion of the total loading which is long term.

For the purpose of determining the modular ratio, it should be assumed that all spans are fully loaded. Imposed loads on floors should be assumed to be two-thirds short term and one-third long term in buildings of normal usage. Storage loads and loads which are permanent in nature should be taken as long term. Imposed roof loads, wind loads and snow loads should be treated as short term.

The values of short-term and long-term modular ratios given in Table 1 may be used for all grades of concrete.

Table 1 — Modular ratio

Type of concrete	Modular ratio for short-term loading α_s	Modular ratio for long-term loading α_l
Normal weight	6	18
Lightweight (see **3.2**)	10	25

4.2 Second moment of area

4.2.1 General

For composite beams three possible values of the second moment of area should be distinguished as follows:

I_g the gross value for the uncracked section;

I_n the cracked section value for negative moments;

I_p the cracked section value for positive moments.

The appropriate value of the second moment of area should be determined as follows:

a) for deflection calculations (see **6.1.3.5**) using I_g;

b) for elastic global analysis (see **5.2.3**) using:

1) for the gross uncracked section method (see **5.2.3.1**), I_g;

2) for the cracked section method (see **D.2**), I_g and I_n;

c) for the elastic section modulus (see **4.3**) using I_g, I_p or I_n as appropriate.

4.2.2 Gross uncracked section

The gross value of the second moment area of the uncracked composite section I_g should be calculated using the mid-span effective breadth of the concrete flange (see **4.6**). The concrete flange should be assumed to be uncracked, but unreinforced. The full area of concrete within the effective breadth of the concrete flange should normally be included in the effective section. Alternatively, for simplicity, the concrete within the depth of the ribs may conservatively be neglected. Any concrete beam casing should normally be neglected.

NOTE **B.3.1** gives a formula for the gross value of the second moment of area of a composite beam in which the steel beam has equal flanges and any concrete within the depth of the ribs is neglected.

4.2.3 Cracked section, negative moments

For negative moments, the second moment of area of the cracked composite section should be calculated using a section comprising the steel member together with the effectively anchored reinforcement located within the effective breadth of the concrete flange (see **4.6**) at the support.

NOTE A formula for the negative moment value of the second moment of area of the cracked composite section in which the steel beam has equal flanges is given in **B.3.2**.

4.2.4 Cracked section, positive moments

For positive moments, the second moment of area of the cracked composite section should be calculated using the mid-span effective breadth of the concrete flange (see **4.6**) but neglecting any concrete in tension.

NOTE A formula for the positive moment value of the second moment of area of a cracked composite section in which the steel beam has equal flanges is given in **B.3.3**.

4.3 Elastic section modulus

The elastic section modulus of a composite section, required to calculate stresses at the serviceability limit state (see **6.2**), should be determined from the appropriate value of the second moment of area (see **4.2**).

For positive moments, the gross value I_g (see **4.2.2**) should be used if the elastic neutral axis is in the steel section and the cracked section value for positive moments I_p (see **4.2.4**) if the elastic neutral axis is in the concrete flange.

For negative moments, the cracked section value for negative moments I_n (see **4.2.3**) should be used.

4.4 Moment capacity

4.4.1 Effective cross section

The moment capacity of a composite beam should be based on the following effective cross section.

a) The effective breadth B_e of the concrete flange (see **4.6**) should be used.

b) The effective section of a composite slab which spans onto a beam (and thus has its ribs running perpendicular to the beam) should be taken as the concrete above the top of the ribs only. The concrete within the depth of the ribs should be neglected.

The effective section of a composite slab with its ribs running parallel to the beam should normally be taken as the full cross section of the concrete.

The effective section of a composite slab with its ribs running at an angle θ to the beam should be taken as the full area of the concrete above the top of the ribs plus $\cos^2 \theta$ times the area of the concrete within the depth of the ribs.

Alternatively, for simplicity, the concrete within the depth of the ribs may conservatively be neglected.

c) Profiled steel sheets should not be included in the effective section.

d) Concrete in tension should be neglected.

e) Reinforcement in compression should be neglected, unless it is restrained by being "contained" by links in accordance with BS 8110.

f) All welded mesh reinforcement and any bar reinforcement which is less than 10 mm in diameter should be treated as nominal reinforcement and should not be included in the effective section.

4.4.2 Plastic moment capacity

The plastic moment capacity of a composite cross section should be calculated on the following basis.

a) Concrete should be assumed to be stressed to a uniform compression of $0.45f_{cu}$ over the full depth of concrete on the compression side of the plastic neutral axis.

b) The structural steel member should be assumed to be stressed to its design strength p_y either in tension or in compression. For sections with a semi-compact or slender web, the effective section described in **4.5.3** should be used.

c) Longitudinal reinforcement should be assumed to be stressed to its design strength $0.87f_y$ where it is in tension.

NOTE Formulae for the plastic moment capacity of a composite beam in which the steel beam has equal flanges are given in **B.2**.

4.4.3 Elastic moment capacity

The elastic moment capacity of a composite cross section should be calculated on the following basis.

a) The strain distribution in the effective cross section should be linear.

b) The stress distribution in the concrete may be assumed to be linear, based on the appropriate value of the modular ratio from **4.1** and limited to a value of $0.50f_{cu}$. Alternatively the parabolic-rectangular stress distribution recommended in BS 8110 may be used, with a limiting compressive strain of 0.0035 and a limiting stress of $0.45f_{cu}$.

c) The stress in the steel beam should be limited to the design strength p_y, reduced as recommended in BS 5950-1 where the section is class 4 slender (see **4.5.2**).

d) The stress in longitudinal reinforcement should be limited to $0.87f_y$.

4.5 Limiting proportions of cross sections

4.5.1 General

The capacities of cross sections may be limited by local buckling of the web or of the steel compression flange.

In the absence of a more refined calculation, account should be taken of these effects in the design of composite beams for the ultimate limit state by using the design methods recommended in section 5.

NOTE These vary according to the classification of the cross section.

All composite cross sections should be classified as recommended in **4.5.2**. In calculations for the construction stage of a composite beam based on the plain steel section, the classification of cross sections should be in accordance with BS 5950-1.

4.5.2 Classification of composite cross sections

To classify a composite cross section, the position of the neutral axis should be based on the effective cross section determined in accordance with **4.4.1**.

When the concrete flange is in tension, the flange reinforcement should be included in the cross section if utilized in the design of the member.

Classification of cross sections of composite beams should generally be in accordance with BS 5950-1, except for the following.

a) The limiting width to thickness ratios, d/t, for webs should be obtained from Table 2.

b) A steel compression flange restrained by effective attachment to a solid concrete flange by shear connectors in accordance with **5.4** may be assumed to comply with class 1 plastic.

c) Where a steel compression flange is restrained by effective attachment by shear connectors in accordance with **5.4**, to a composite slab in which either:

the ribs run at an angle of at least 45° to the axis of the beam; or

the breadth b_r (as defined in **5.4.7.2**, measured perpendicular to the axis of the rib) of the rib located directly over the beam is not less than half the breadth of the beam flange;

then the classification of the compression flange may be assumed to be:

class 1 plastic if its classification in accordance with Part 1 is class 2 compact; or

class 2 compact if its classification in accordance with Part 1 is class 3 semi-compact.

In Table 2, r is the ratio of the mean longitudinal stress in the web to the design strength p_y, compressive stresses being taken as positive and tensile stresses as negative (see Figure 1).

When the compression flange is class 1 plastic or class 2 compact, a plastic stress distribution on the gross cross section should be assumed when calculating r [see Figure 1 a)]. In the case of partial shear connection, reference should also be made to **5.5.2**.

When the compression flange is class 3 semi-compact or class 4 slender, the elastic stresses calculated on the gross cross section at the ultimate limit state should be assumed when calculating r [see Figure 1 b)].

NOTE Formulae for the calculation of r are given in Appendix C.

4.5.3 Sections with class 1 or class 2 flanges

When the compression flange is class 1 plastic or class 2 compact, the plastic moment capacity (see **4.4.2**) should be used, provided that the web is not class 4 slender.

The reduced plastic moment capacity of a section with a class 3 semi-compact web should be determined using the effective section shown in Figure 2. The depth of web taken as effective in resisting compression should be limited to $19t\epsilon$ adjacent to the compression flange and $19t\epsilon$ adjacent to the plastic neutral axis. The remainder of the web on the compression side of the plastic neutral axis should be neglected.

NOTE Formulae for the reduced plastic moment capacity of composite sections in which the steel beam has equal flanges are given in Appendix B.

4.5.4 Sections with class 3 or class 4 flanges

When the compression flange is class 3 semi-compact or the web or the compression flange is class 4 slender, the elastic moment capacity (see **4.4.3**) should be used. If the web or the compression flange is class 4 slender, the elastic moment capacity should be reduced as recommended in BS 5950-1.

4.6 Effective breadth of concrete flange

Allowance should be made for the in-plane shear flexibility (shear lag) of a concrete flange by using an effective breadth.

The total effective breadth B_e of concrete flange acting compositely with a steel beam should be taken as the sum of the effective breadths b_e of the portions of flange each side of the centreline of the steel beam.

In the absence of any more accurate determination, the effective breadth of each portion should be taken as follows.

a) For a slab spanning perpendicular to the beam, $b_e = L_z/8$ but not greater than b

b) For a slab spanning parallel to the beam, $b_e = L_z/8$ but not greater than $0.8b$

where L_z is the distance between points of zero moment. For a simply supported beam L_z is equal to the effective span L (see **5.1.1**). For a continuous beam L_z may be obtained from Figure 3.

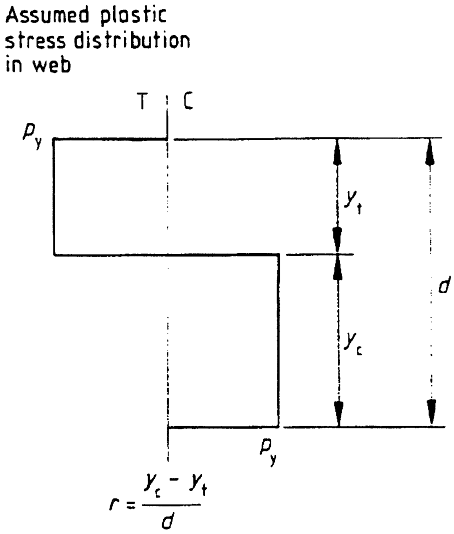
If a separate allowance is made for the co-existing effects of slab bending, the limit of $0.8b$ need not be applied.

The actual breadth b of each portion should be taken as half the distance to the adjacent beam, measured to the centreline of the web, except that at a free edge the actual breadth is the distance from the beam to the free edge.

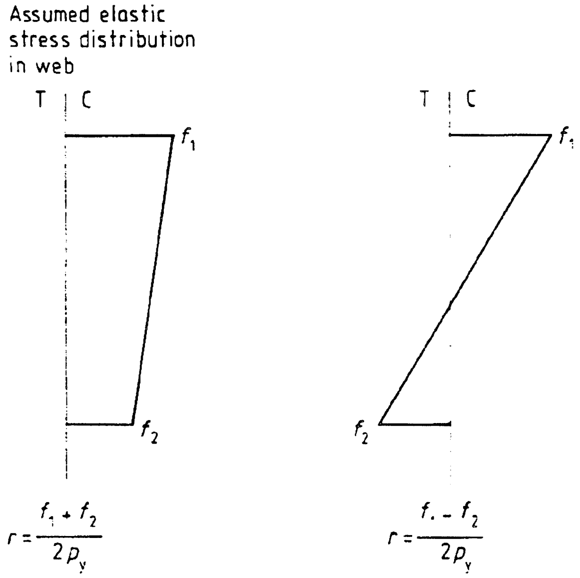
Table 2 — Limiting width to thickness ratios for webs
(Elements which exceed these limits are to be taken as class 4 slender)

Type of element	Class of section		
	Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
Web, with neutral axis at mid-depth	$\frac{d}{t} \leq 64\epsilon$	$\frac{d}{t} \leq 76\epsilon$	$\frac{d}{t} \leq 114\epsilon$
Web, generally	$\frac{d}{t} \leq \frac{64\epsilon}{1+r}$	$\frac{d}{t} \leq \frac{76\epsilon}{1+r}$	when r ≥ 0.66: for rolled sections: $\frac{d}{t} \leq \frac{114\epsilon}{1+2r}$ for welded sections: $\frac{d}{t} \leq \left(\frac{41}{r} - 13\right)\epsilon$ when $0.66 > r \geq 0$: $\frac{d}{t} \leq \frac{114\epsilon}{1+2r}$ when r < 0: $\frac{d}{t} \leq \frac{114\epsilon(1+r)}{(1+2r)^{\frac{3}{2}}}$

NOTE 1 These ratios apply to the composite section. During construction the classification in BS 5950-1 applies.
NOTE 2 Check webs for shear buckling in accordance with BS 5950-1 when $d/t \geq 63\varepsilon$.
NOTE 3 The values in this table do not apply to T-sections.
NOTE 4 $\epsilon = [275/p_y]^{½}$

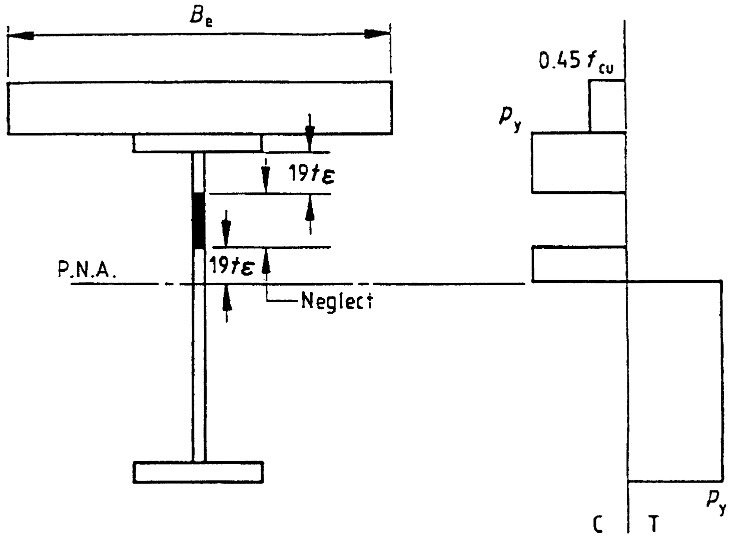


(a) Section with class 1 plastic or class 2 compact compression flange

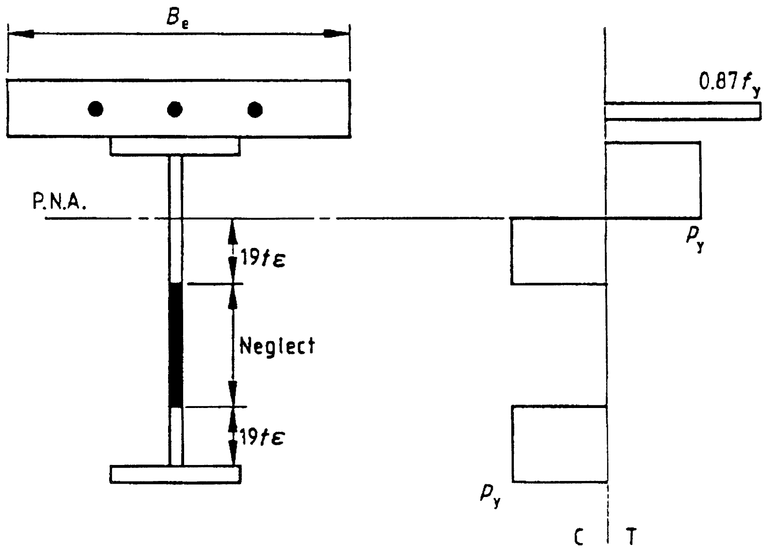


(b) Section with class 3 semi-compact or class 4 slender compression flange

Figure 1 — Determination of web stress ratio *r*



(a) Positive moment



(b) Negative moment

NOTE. P.N.A. denotes plastic neutral axis of effective section.

Figure 2 — Plastic moment capacity of effective section

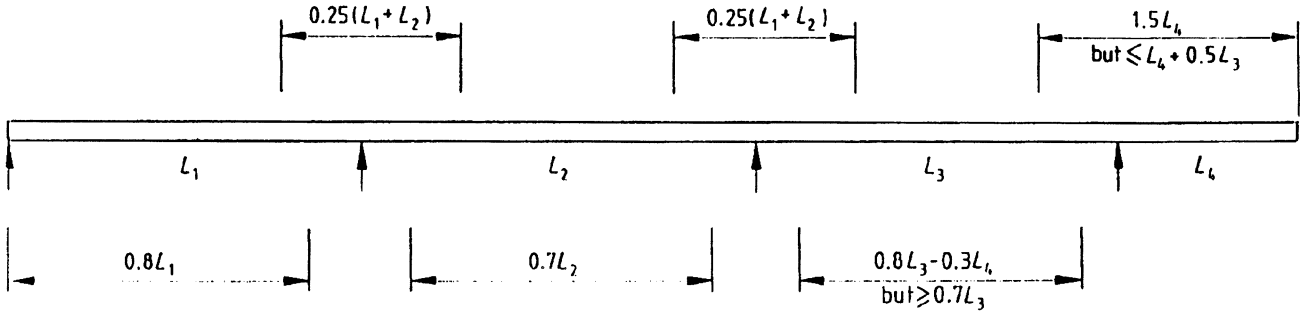


Figure 3 — Values of L_z for continuous beams

Section 5. Composite beams: ultimate limit state

5.1 General

5.1.1 Effective span

The effective span of a beam should generally be taken as the distance between the centres of the supports, but need not be taken as greater than the clear distance between the supports plus the depth of the steel member.

The effective length of a cantilever should generally be taken from the centre of the support, but need not be taken as greater than the projecting length from the face of the support plus half the depth of the steel member.

5.1.2 Unpropped construction

Beams with class 1 plastic or class 2 compact compression flanges throughout may be designed assuming that at the ultimate limit state the whole of the loading acts on the composite member, provided that the longitudinal shear is calculated accordingly.

Beams which have class 3 semi-compact or class 4 slender compression flanges at any point should be designed allowing for the separate effects of loading applied to the steel beam or the composite member as appropriate.

5.1.3 Propped construction

Where propped construction is used, all beams may be designed assuming that at the ultimate limit state the whole of the loading acts on the composite member.

5.1.4 Vertical shear force

The steel member should be designed in accordance with BS 5950-1, to resist the whole of the vertical shear force. The reduction of moment capacity due to high shear load should be determined in accordance with **5.3.4**.

5.2 Moments in continuous beams

5.2.1 General

5.2.1.1 *Methods.* The methods given in **5.2** apply to beams which are effectively continuous at all internal supports. In these methods all supports should be assumed to be simple supports.

The moments in such continuous composite beams may be determined using any of the following methods, provided that the beam complies with the relevant conditions.

a) Simplified method (see **5.2.2**).

b) Elastic analysis (see **5.2.3**).

c) Plastic analysis (see **5.2.4**).

In each case, the shear forces should be in equilibrium with the moments and the applied loads.

5.2.1.2 *Non-reinforced class 1 plastic sections.* The recommendations given for non-reinforced class 1 plastic sections apply exclusively to cross sections with only nominal tension reinforcement [see **4.4.1** f)] in negative moment regions.

The nominal tension reinforcement should be neglected when calculating the plastic moment capacity (see **4.4.2**).

The classification of both the web and the compression flange should be class 1 plastic, in accordance with **4.5.2**.

5.2.2 Simplified method

The moments in continuous composite beams may be determined using the coefficients given in Table 3, provided that the following conditions are satisfied.

a) The steel beam should be of uniform section with equal flanges and without any haunches.

b) The steel beam should be of the same section in each span.

c) The loading should be uniformly distributed.

d) The unfactored imposed load should not exceed 2.5 times the unfactored dead load.

e) No span should be less than 75 % of the longest.

f) End spans should not exceed 115 % of the length of the adjacent span.

g) There should not be any cantilevers.

The coefficients in Table 3 should be multiplied by the free bending moment $WL/8$, where W is the total factored load on the span L. Where the spans each side of a support differ, the mean of the values of $WL/8$ for the two adjacent spans should be used to calculate the support moment.

The values in Table 3 already allow for pattern loads and for redistribution. No further redistribution should be carried out when using this method.

5.2.3 Elastic analysis

5.2.3.1 *General.* Elastic global analysis of continuous beams may be carried out using the section properties of the gross uncracked section described in **4.2.2** throughout. The resulting negative moment at any support may be reduced (except adjacent to cantilevers) by an amount not exceeding the appropriate maximum percentage given in Table 4. Corresponding increases should then be made to the positive moments in the adjacent spans to maintain equilibrium with the applied loads. The shear forces should also be adjusted, if necessary, to maintain equilibrium.

Alternatively the cracked section method given in **D.2** may be used.

Table 3 — Simplified table of moment coefficients (to be multiplied by *WL*/8)

Location	Number of spans	Classification of compression flange in negative moment region				
		Class 4	Class 3	Class 2	Class 1 Plastic	
		Slender	Semi-compact	compact	Generally	Non-reinforced (see 5.2.1.2)
Middle of end span	2	0.71	0.71	0.71	0.75	0.79
	3 or more	0.80	0.80	0.80	0.80	0.82
First internal support	2	0.91	0.81	0.71	0.61	0.50
	3 or more	0.86	0.76	0.67	0.57	0.48
Middle of internal spans	3	0.51	0.51	0.52	0.56	0.63
	4 or more	0.65	0.65	0.65	0.65	0.67
Internal supports except the first	4 or more	0.75	0.67	0.58	0.50	0.42

Table 4 — Maximum redistribution of support moments for elastic global analysis, using properties of gross uncracked section

Classification of compression flange at support				
Class 4	Class 3	Class 2	Class 1 Plastic	
Slender	Semi-compact	Compact	Generally	Non-reinforced (see 5.2.1.2)
%	%	%	%	%
10	20	30	40	50

5.2.3.2 *Pattern loads.* Imposed loads should be arranged in the most unfavourable realistic pattern for each case. Dead load γ_f factors need not be varied when considering such pattern loading.

For continuous beams subject to uniformly distributed imposed load, only the following arrangements of imposed load need be considered.

a) Alternate spans loaded

b) Two adjacent spans loaded

5.2.4 Plastic analysis

Plastic global analysis may be used to determine the moments in continuous beams with non-reinforced class 1 plastic sections (see **5.2.1.2**) at internal supports and with class 1 plastic sections at mid-span, provided that conditions a) to d) given in **5.2.2** for the simplified method are also satisfied.

Alternatively the general plastic method given in **D.3** may be used.

5.2.5 Stability of bottom flange

The stability of the bottom flange should be checked for each span in turn. The span being checked should be assumed to be loaded with factored dead load only and the negative moments at each internal support should be assumed to be equal to the relevant moment capacity M_c (elastic, plastic or reduced plastic) applicable for design of the cross section at the support (see Figure 4). However, the support moments need not be taken as more than those obtained from an elastic analysis (using the properties of the gross uncracked section) without redistribution.

5.2.6 Connections in continuous beams

Any moment-resisting bolted connections which occur in the negative moment regions of continuous beams should be designed on the assumption that the support moments are 1.1 times those recommended in **5.2.5**.

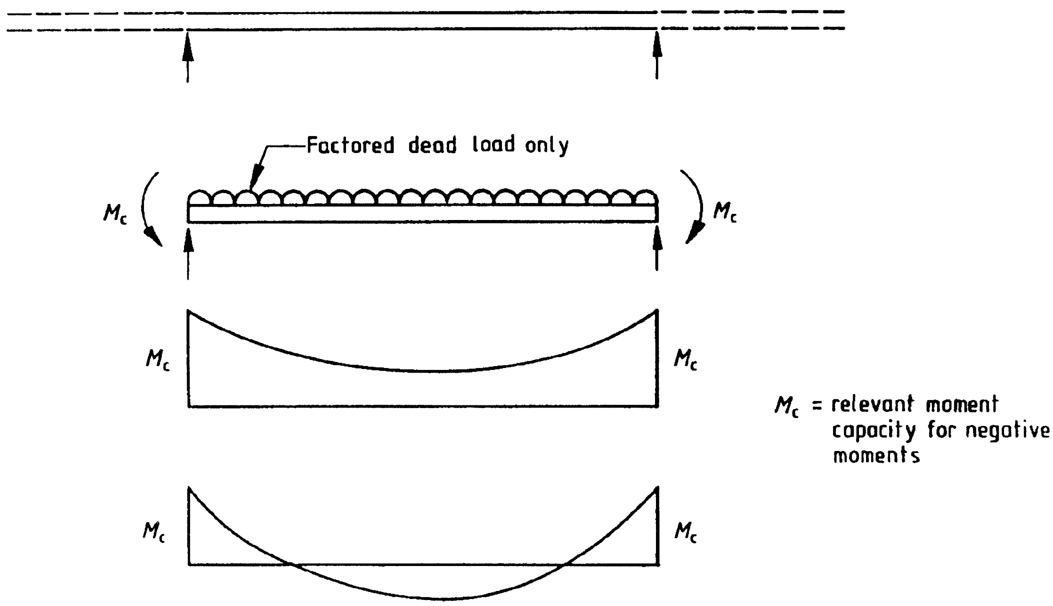


Figure 4 — Check for stability of bottom flange

5.3 Design of members

5.3.1 Simply supported beams

The moment capacity of a simply supported beam with a compression flange which is class 1 plastic or class 2 compact and which is subject to positive moment should be taken as the plastic moment capacity of the composite section, provided that the web is not class 4 slender. If the web is class 1 plastic or class 2 compact, the plastic moment capacity should be calculated on the basis given in **4.4.2**. If the web is class 3 semi-compact, the plastic moment capacity should be determined in accordance with **4.5.3**.

The moment capacity of a simply supported beam with a compression flange which is class 3 semi-compact or a web or a compression flange which is class 4 slender and which is subject to positive moment should be taken as the elastic moment capacity calculated on the basis given in **4.4.3**.

5.3.2 Cantilevers

The moment capacity of a cantilever should be based on the steel section together with any effectively anchored tension reinforcement within the effective breadth of the concrete flange (see **4.6**) but excluding any which is provided to reinforce the slab for moments due to loading acting directly on it.

When the compression flange is class 1 plastic or class 2 compact, the moment capacity should be taken as the plastic moment capacity, provided that the web is not class 4 slender. If the web is class 1 plastic or class 2 compact, the plastic moment capacity should be calculated on the basis in **4.4.2**. If the web is class 3 semi-compact, the plastic moment capacity should be determined in accordance with **4.5.3**.

For cantilevers with a compression flange which is class 3 semi-compact or a web or a compression flange which is class 4 slender, the elastic moment capacity (see **4.4.3**) should be used.

5.3.3 Continuous beams

The positive moment capacity of a continuous beam should be determined as for a simply supported beam (see **5.3.1**) and the negative moment capacity should be determined as for a cantilever (see **5.3.2**).

5.3.4 Moment capacity with high shear load

Where the shear force F_v exceeds 0.5 P_v, the moment capacity should be reduced to allow for the influence of shear. The reduced moment capacity M_{cv} should be determined from the following expression:

$$M_{cv} = M_c - (M_c - M_f)\,(2F_v/P_v - 1)^2$$

where

M_c is the plastic moment capacity of the composite section;

M_f is the plastic moment capacity of that part of the section remaining after deduction of the shear area A_v defined in BS 5950-1;

P_v is the lesser of the shear capacity and the shear buckling resistance, both determined from BS 5950-1.

For sections with a class 3 semi-compact web or class 4 slender web, M_c should be determined in accordance with **4.5.3**.

For sections with a compression flange which is class 3 semi-compact or a web or a compression flange which is class 4 slender, M_{cv} should not be taken as greater than the above or greater than the elastic moment capacity determined in accordance with **4.4.3**, whichever is less.

5.3.5 Stability of compression flange

To prevent lateral-torsional buckling, the compression flange should be laterally restrained as recommended in BS 5950-1.

When checking the lateral stability of the bottom flange in negative moment regions, the methods given in appendix G of BS 5950-1:1985 may be used. Other methods that include allowances for the torsional restraint provided by the concrete slab may also be used. Reference may be made to BS 5400-3 or to specialist literature.

At plastic hinge locations, other than the last hinge to form in each span, the recommendations for plastic hinge locations given in BS 5950-1 should be followed.

Where the reduction of negative moments as described in **5.2.3.1** exceeds 30 % at the supports of beams of uniform section (or 20 % when using the cracked section method given in **D.2**), the points of support should be treated as plastic hinge locations.

In beams of varying section, the locations of the potential negative moment plastic hinges, implied by the redistribution of support moments, should be identified. When the reduction of negative moments at such locations exceeds 30 % (or 20 % when using **D.2**), they should be treated as active plastic hinge locations.

5.4 Shear connection

5.4.1 General

The shear connection should be capable of transmitting the longitudinal shear between the concrete slab and the steel beam due to factored loads, without causing crushing or other damage to the concrete and without allowing excessive slip or separation between the concrete and the steel.

5.4.2 Types of shear connector

Shear connectors commonly take the form of headed studs welded to the steel beam, either directly or through profiled steel sheets. The purpose of the head of the stud is to resist any uplift component of the forces applied to the stud.

The material requirements and proportions of studs should be as given in **3.4.1**.

Other types of shear connector can also be used, provided that they have adequate deformation capacity, i.e. not less than that provided by bar and hoop connectors of the type illustrated in BS 5400-5, and provided that means for resisting uplift is incorporated.

5.4.3 Capacities of shear connectors in solid slabs

In a solid slab, the capacities of shear connectors to resist longitudinal shear should be taken as follows.

a) For positive moments,

$$Q_p = 0.8Q_k$$

b) For negative moments,

$$Q_n = 0.6Q_k$$

where

Q_k is the characteristic resistance of the shear connector.

The characteristic resistance for a headed stud should be obtained by reference to **5.4.6**.

As values are not at present given in this code for types of shear connector other than headed studs, the characteristic resistances of other types of shear connector should be determined from push out tests.

NOTE A specification for push out tests is given in BS 5400-5.

5.4.4 Provision of shear connectors

5.4.4.1 *Positive moments.* For full shear connection, the total number of shear connectors N_p required to develop the positive moment capacity of the section, i.e. the number of shear connectors each side of the point of maximum moment (see **5.4.5.1**), should be determined from the equation:

$$N_p = F_p/Q_p$$

where

Q_p is the capacity of the shear connector in positive moment regions [see **5.4.3** a)];

F_p is the longitudinal compressive force in the concrete slab at the point of maximum positive moment.

Where design is based on the plastic moment capacity of the composite section, F_p should be taken as the lesser of Ap_y or $0.45f_{cu}$ times the area of concrete within the effective cross section (see **4.4.1**).

Where the ribs of composite slabs run parallel to the beam (or at an angle to the beam), the area of concrete in the ribs may be neglected when determining F_p, provided that this area is also neglected when calculating the moment capacity of the cross section.

Where the maximum moment is less than the plastic moment capacity, the number of shear connectors may be reduced provided that the recommendations for partial shear connection in **5.5** are followed.

Where design is based on the elastic moment capacity of the composite section, the force F_p should be determined from the calculated stresses in the concrete slab. However, N_p should not be taken as less than the number of connectors required for partial shear connection (see **5.5**).

5.4.4.2 *Negative moments.* Where the negative moment capacity includes the contribution of reinforcement in the slab, shear connectors should be provided to resist a longitudinal force F_n equal to $0.87f_yA_r$, where f_y is the characteristic strength of the reinforcement and A_r is the area of reinforcement in the effective cross section (see **4.4.1**).

The required number of shear connectors N_n, i.e. the number of shear connectors each side of the point of maximum moment (see **5.4.5.1**), should be determined from the equation:

$$N_n = F_n/Q_n$$

where

Q_n is the capacity of the shear connector in negative moment regions, see **5.4.3** b).

Where partial shear connection is used, the number of shear connectors provided to develop the negative moment capacity should not be reduced below N_n. This also applies where the elastic moment capacity is used.

5.4.5 Spacing of shear connectors

5.4.5.1 *General.* The total number of shear connectors between a point of maximum positive moment and each adjacent support should not be less than the sum of N_p and N_n, obtained from **5.4.4.1** and **5.4.4.2** respectively.

The total number of shear connectors should normally be spaced uniformly along the beam, provided that the recommendations on spacing in **5.4.5.2** to **5.4.5.6** are satisfied. Where variation of the spacing is necessary for any reason, the shear connectors should be spaced uniformly within two or more zones, changing at intermediate points which comply with **5.4.5.5**.

In continuous beams, the shear connectors should be spaced more closely in negative moment regions, where this is necessary, to suit the curtailment of tension reinforcement (see **5.4.5.6**).

In cantilevers, the spacing of the shear connectors should be based on the curtailment of the tension reinforcement.

5.4.5.2 *Additional checks.* Additional checks on the adequacy of the shear connection as recommended in **5.4.5.5** should be made at intermediate points where any of the following apply.

a) A heavy concentrated load occurs within a positive moment region.

b) A sudden change of cross section occurs.

c) The member is tapered (see **5.4.5.3**).

d) The concrete flange is unusually large (see **5.4.5.4**).

In case a), a concentrated load should be considered "heavy" if its free moment M_o exceeds 10 % of the positive moment capacity of the composite section. The free moment M_o is the maximum moment in a simply supported beam of the same span due to the concentrated load acting alone.

5.4.5.3 *Tapered members.* In members which reduce in depth towards their supports, additional checks as recommended in **5.4.5.5** should be made at a series of intermediate points, selected such that the ratio of the greater to the lesser moment capacity for any pair of adjacent intermediate points does not exceed 2.5.

5.4.5.4 *Large concrete flanges.* If the concrete flange is so large that the plastic moment capacity of the composite section exceeds 2.5 times the plastic moment capacity of the steel member alone, additional checks as recommended in **5.4.5.5** should be made at intermediate points approximately mid-way between points of maximum positive moment and each adjacent support.

5.4.5.5 *Adequacy of shear connection.* The adequacy of the shear connection should be checked at all intermediate points where the spacing of shear connectors changes (see **5.4.5.1**) and at the intermediate points described in **5.4.5.2**.

The total number of shear connectors between any such intermediate point and the adjacent support should be not less than N_i determined from the following expressions:

for positive moments

$$N_i = N_p (M - M_s)/(M_c - M_s) + N_n \text{ but } N_i \geq N_n$$

for negative moments

$$N_i = N_n (M_c - M)/(M_c - M_s) \text{ but } N_i \leq N_n$$

where

M is the moment at the intermediate point;

M_c is the positive or negative moment capacity of the composite section, as appropriate;

M_s is the moment capacity of the steel member.

Alternatively, for positive moments, the adequacy of the shear connection may be demonstrated by checking the plastic moment capacity at the intermediate point, assuming that the compressive force F_c in the concrete flange equals $(N_a - N_n)Q_p$ where N_a is the actual number of shear connectors between the intermediate point and the adjacent support. In this check, the classification of the web (see **4.5.2**) should also be based on the value of the ratio r determined using the above value of F_c.

5.4.5.6 *Curtailment of reinforcement.* Where tension reinforcement is used in negative moment regions, every bar should extend beyond the point at which it is no longer required to assist in resisting the negative moment, by a distance not less than 12 times the bar size. In addition the lengths of the bars should comply with the recommendations given in BS 8110 for anchorage of bars in a tension zone.

The longest bars should extend beyond the zone containing the N_n shear connectors required to transfer the longitudinal force F_n, by a distance not less than the longitudinal spacing of the shear connectors. If necessary, the lengths of the bars should be increased to achieve this. Alternatively, the shear connectors should be spaced more closely in this region to avoid increasing the lengths of the bars.

5.4.6 Headed studs in solid slabs

The characteristic resistance Q_k of a headed shear stud with the dimensions and properties given in **3.4.1** embedded in a solid slab of normal weight concrete should be taken from Table 5.

For lightweight aggregate concrete (see **3.2**), the characteristic resistance should be taken as 90 % of the value given in Table 5.

5.4.7 Headed studs in composite slabs

5.4.7.1 *General.* The recommendations of this clause apply to headed shear studs with the dimensions and properties given in **3.4.1**, embedded in slabs comprising profiled steel sheets and concrete.

These recommendations apply only when the following conditions are satisfied.

a) The overall depth of the profiled sheet should be not less than 35 mm nor greater than 80 mm.

b) The mean width of the troughs of the profiled sheet should be not less than 50 mm.

c) The nominal diameter of the studs should not exceed 19 mm.

d) The height of the studs should be at least 35 mm greater than the overall depth of the profiled sheets.

e) The recommendations of BS 5950-4 concerning the thickness of concrete cover above the profiled steel sheet should be satisfied.

5.4.7.2 *Ribs perpendicular to the beam.* The capacity of headed studs in composite slabs with the ribs running perpendicular to the beam should be taken as their capacity in a solid slab (see **5.4.3**) multiplied by the reduction factor k given by the following expressions:

for one stud per rib

$$k = 0.85 (b_r/D_p)\{(h/D_p) - 1\} \text{ but } k \leq 1$$

for two studs per rib

$$k = 0.6 (b_r/D_p)\{h/D_p) - 1\} \text{ but } k \leq 0.8$$

for three or more studs per rib.

$$k = 0.85 (b_r/D_p)\{(h/D_p) - 1\} \text{ but } k \leq 0.6$$

where

b_r is the breadth of the concrete rib;

D_p is the overall depth of the profiled steel sheet;

h is the overall height of the stud.

h should not be taken as more than $2D_p$ or D_p + 75 mm, whichever is less, although studs of greater height may be used.

Provided that the studs are located centrally in the rib, b_r should be taken as equal to the mean width of the trough b_a for open profile sheets but equal to the minimum width of the trough b_b for re-entrant profile sheets (see Figure 5).

Where it is necessary for the studs to be located noncentrally in the rib, the studs should preferably be placed in the favourable location such that the zone of concrete in compression in front of the stud is maximized. Where it is necessary for the studs to be placed in the unfavourable location, b_r should be reduced to $2e$, where e is the distance to the nearer side of the rib (see Figure 5). The distance e should be not less than 25 mm.

Where the studs are placed in pairs but in an off-set pattern alternately on the favourable and unfavourable sides (subject to the minimum spacings given in **5.4.8.4**), b_r should be determined as for centrally located studs.

5.4.7.3 *Ribs parallel to the beam.* The capacity of headed studs in composite slabs with the ribs running parallel to the beam should be taken as their capacity in a solid slab (see **5.4.3**) multiplied by the reduction factor k given by the following expressions:

when $b_r/D_p \geq 1.5$

$k = 1$

when $b_r/D_p < 1.5$

$k = 0.6\ (b_r/D_p)\{(h/D_p) - 1\}$ but $k \leq 1.0$

where

b_r, D_p and h are as in **5.4.7.2**.

Where there is more than one longitudinal line of studs in a concrete rib, the mean width b_a of the trough of the profiled steel sheet should be at least 50 mm greater than the transverse spacing of the lines of studs.

Optionally, the trough of the profiled sheet may be split longitudinally and separated to form a wider concrete rib over the flange of the steel beam and, in this case, b_r should also be increased accordingly in the expression for k given above.

5.4.7.4 *Ribs at other angles.* Where the ribs run at an angle θ to the beam, the reduction factor k should be determined from the expression:

$$k = k_1 \sin^2 \theta + k_2 \cos^2 \theta$$

where

k_1 is the value of k from **5.4.7.2**;

k_2 is the value of k from **5.4.7.3**.

Alternatively the smaller value of k may be used.

5.4.8 Dimensional details

5.4.8.1 *Maximum spacing.* The longitudinal spacing of shear connectors should not normally exceed 600 mm or $4D_s$, whichever is less, where D_s is the overall depth of the slab.

Shear connectors may be arranged in groups, with a mean spacing as above and a maximum spacing of $8D_s$, provided that due account is taken of the resulting non-uniform flow of longitudinal shear and of the greater possibility of vertical separation between the concrete flange and the steel beam.

Where the stability of either the steel beam or the concrete flange depends on the shear connectors, the maximum spacing should be limited accordingly and appropriate resistance to uplift should be provided.

5.4.8.2 *Edge distance.* The clear distance between a shear connector and the edge of the steel flange should be not less than 20 mm [see Figure 6 a)].

5.4.8.3 *Haunches.* Except where profiled steel sheets are used, the sides of a concrete haunch between the steel beam and the soffit of the slab should lie outside a line drawn at 45° from the outside edge of the shear connectors, and the concrete cover to the shear connectors should be not less than 50 mm [see Figure 6 b)].

5.4.8.4 *Stud shear connectors*

5.4.8.4.1 *Minimum spacing.* The minimum centre-to-centre spacing of stud shear connectors should be $5d$ along the beam and $4d$ between adjacent studs, where d is the nominal shank diameter. Where rows of studs are staggered, the minimum transverse spacing of longitudinal lines of studs should be $3d$.

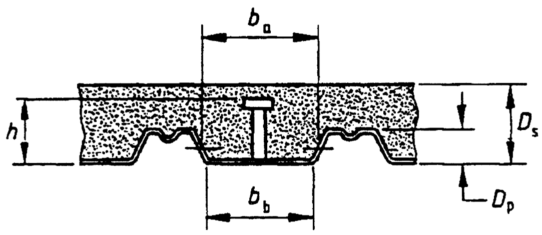
5.4.8.4.2 *Maximum diameter.* Unless located directly over the web, the nominal diameter of a stud shear connector should not exceed 2.5 times the thickness of the flange to which it is welded.

5.4.8.5 *Other types of shear connectors.* The dimensional details and minimum spacing of other types of shear connector should be within the ranges demonstrated as satisfactory by push out tests.

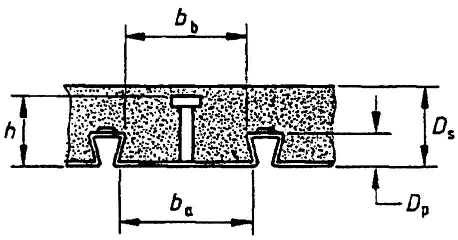
Table 5 — Characteristic resistance Q_k of headed studs in normal weight concrete

Dimensions of stud shear connectors		Characteristic strength of concrete				
Nominal shank diameter	Nominal height	As-welded height	N/mm² 25	N/mm² 30	N/mm² 35	N/mm² 40
mm	mm	mm	kN	kN	kN	kN
25	100	95	146	154	161	168
22	100	95	119	126	132	139
19	100	95	95	100	104	109
19	75	70	82	87	91	96
16	75	70	70	74	78	82
13	65	60	44	47	49	52

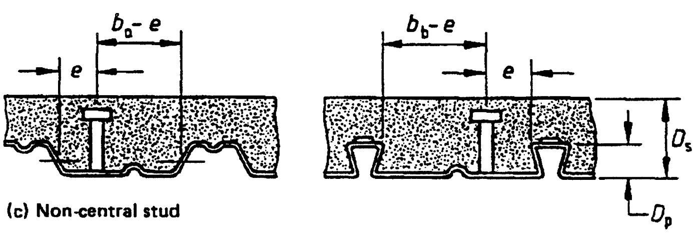
NOTE 1 For concrete of characteristic strength greater than 40 N/mm² use the values for 40 N/mm².
NOTE 2 For connectors of heights greater than tabulated use the values for the greatest height tabulated.



(a) Open trough profile $b_r = b_a$



(b) Re-entrant trough profile $b_r = b_b$



(c) Non-central stud

Figure 5 — Breadth of concrete rib b_r

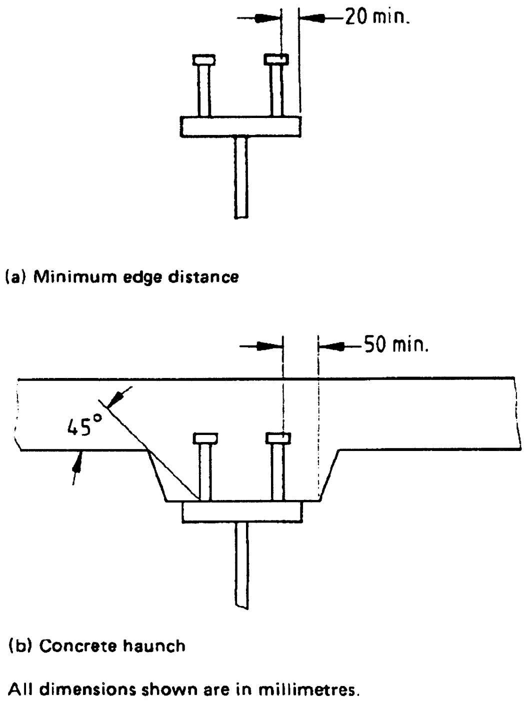


Figure 6 — Minimum dimensions

5.5 Partial shear connection

5.5.1 Conditions

This method should be used only when the shear connectors are headed studs with the dimensions and properties given in **3.4.1** or other types of shear connectors which have at least the same deformation capacity as headed studs.

The spacing of the shear connectors should satisfy the recommendations given in **5.4.5**.

5.5.2 Number of shear connectors

Where the maximum positive moment in a span is less than the plastic moment capacity of the composite section, calculated on the basis given in **4.4.2** or **4.5.3** as appropriate, the actual number of shear connectors N_a may be reduced below N_p, the number required for full shear connection, as given in **5.4.4.1**.

For spans up to 10 m, the actual number of shear connectors N_a supplied to develop the reduced plastic moment capacity should be not less than 0.40 N_p. When the span exceeds 16 m, N_a should not be reduced below Np. For spans between 10 m and 16 m, the following relationship should be satisfied:

$$N_a/N_p \geq (L - 6)/10 \text{ but } N_a/N_p \geq 0.40$$

where

L is the span in metres.

No reduction should be made in the number of shear connectors N_n required for full shear connection for negative moments.

The reduced plastic moment capacity of the composite section should be calculated assuming a reduced value of the compressive force F_c in the concrete flange equal to the resistance of the shear connection R_q given by the expression:

$$R_q = N_a Q_p$$

The depth of the concrete flange in compression should be determined by assuming a uniform stress in the compression zone equal to $0.45 f_{cu}$, where f_{cu} is the characteristic strength of the concrete.

The classification of the web (see **4.5.2**) should be based on the value of the ratio r determined assuming that the compressive force F_c in the concrete flange equals R_q.

5.6 Transverse reinforcement

5.6.1 General

Transverse reinforcement refers to the reinforcement in the concrete flange running transverse to the span of the beam. Where profiled steel sheets are used they may also act as transverse reinforcement (see **5.6.4**).

Sufficient transverse reinforcement should be used to enable the concrete flange to resist the longitudinal shear transmitted by the shear connectors, both immediately adjacent to the shear connectors and elsewhere within its effective breadth.

5.6.2 Shear to be resisted

The total longitudinal shear force per unit length v to be resisted at any point in the span of the beam should be determined from the spacing of the shear connectors by the following expression:

$$v = NQ/s$$

where

N is the number of shear connectors in a group;

s is the longitudinal spacing centre-to-centre of groups of shear connectors;

Q is either Q_p or Q_n for shear connectors resisting positive or negative moments respectively (see **5.4.3**).

For positive moments, the shear on any particular surface of potential shear failure should be determined taking account of the proportion of the effective breadth of the concrete flange lying beyond the surface under consideration.

For negative moments, the shear on any particular surface of potential shear failure should be determined taking account of the arrangement of the effective longitudinal reinforcement.

5.6.3 Resistance of concrete flange

For any surface of potential shear failure in the concrete flange, the longitudinal shear force per unit length should not exceed the shear resistance v_r given by the following relationship:

$$v_r = 0.7A_{sv}f_y + 0.03\eta A_{cv}f_{cu} + v_p$$

$$\text{but } (v_r \leq 0.8\eta A_{cv}\sqrt{f_{cu}} + v_p)$$

where

f_{cu} is the characteristic cube strength of the concrete in N/mm^2, but not more than40 N/mm^2, although concrete of higher strengths may be used;

η = 1.0 for normal weight concrete;

η = 0.8 for lightweight concrete (see **3.2**);

A_{cv} is the mean cross-sectional area, per unit length of the beam, of the concrete shear surface under consideration;

A_{sv} is the cross-sectional area per unit length of the beam, of the combined top and bottom reinforcement crossing the shear surface (see Figure 7);

v_p is the contribution of the profiled steel sheeting, if applicable (see **5.6.4**).

Only reinforcement which is fully anchored should be included in A_{sv}. Where U-bars are used, they should be looped around the shear connectors.

The length of the shear surface *b–b* shown in Figure 7 should be taken as equal to $2h$ plus the head diameter for a single row of stud shear connectors or staggered stud connectors, or as equal to $2h + s_t$ plus the head diameter for stud shear connectors arranged in pairs, where h is the height of the studs and s_t is the transverse spacing centre-to-centre of the studs.

Where profiled steel sheeting is used it is not necessary to consider shear surfaces of type *b–b*, provided that the capacities of the studs are determined using the appropriate reduction factor k as recommended in **5.4.7**

5.6.4 Contribution of profiled steel sheeting

Profiled steel sheeting may be assumed to contribute to the transverse reinforcement provided that it is either continuous across the top flange of the steel beam or alternatively that it is welded to the steel beam by stud shear connectors.

The resistance of the concrete flange v_r given in **5.6.3** should be modified to allow for profiled steel sheeting as follows.

a) Where the profiled steel sheets are continuous across the top flange of the steel beam, the contribution of profiled steel sheeting v_p with ribs running perpendicular to the span of the beam should be determined from the expression:

$$v_p = t_p\, p_{yp}$$

per unit length of the beam for each intersection of the shear surface by the sheeting, where t_p is the thickness of the profiled steel sheeting and p_{yp} is its design strength, given (as p_y) in BS 5950-4.

b) Where the profiled steel sheeting is discontinuous across the top flange of the steel beam, and stud shear connectors are welded to the steel beam directly through the profiled steel sheets, the contribution of the profiled steel sheets v_p should be determined from the relationship:

$$v_p = (N/s)\,(ndt_p p_{yp}) \text{ but } v_p \leq t_p\, p_{yp}$$

where

d is the nominal shank diameter of the stud;

N and s are as given in **5.6.2**;

n is taken as 4 unless a higher value is justified by tests.

In the case of a beam with separate spans of profiled steel sheeting on each side, the studs should be staggered or arranged in pairs, so that each span of sheeting is properly anchored.

c) The area of concrete shear surface A_{cv} should be determined taking account of the effects of the ribs. Where the ribs run perpendicular to the span of the beam, the concrete within the depth of the ribs should be included in the value of A_{cv}.

d) Where the ribs of the profiled steel sheets run parallel to the span of the beam, the potential shear failure surfaces at lap joints between the sheets should also be checked.

e) Where the ribs of the profiled steel sheeting run at an angle θ to the span of the beam, the effective resistance should be determined from the expression:

$$v_r = v_1 \sin^2\theta + v_2 \cos^2\theta$$

where

v_1 is the value of v_r for ribs perpendicular to the span;

v_2 is the value of v_r for ribs parallel to the span.

5.6.5 Longitudinal splitting

To prevent longitudinal splitting of the concrete flange caused by the shear connectors, the following recommendations should be applied in all composite beams where the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300 mm.

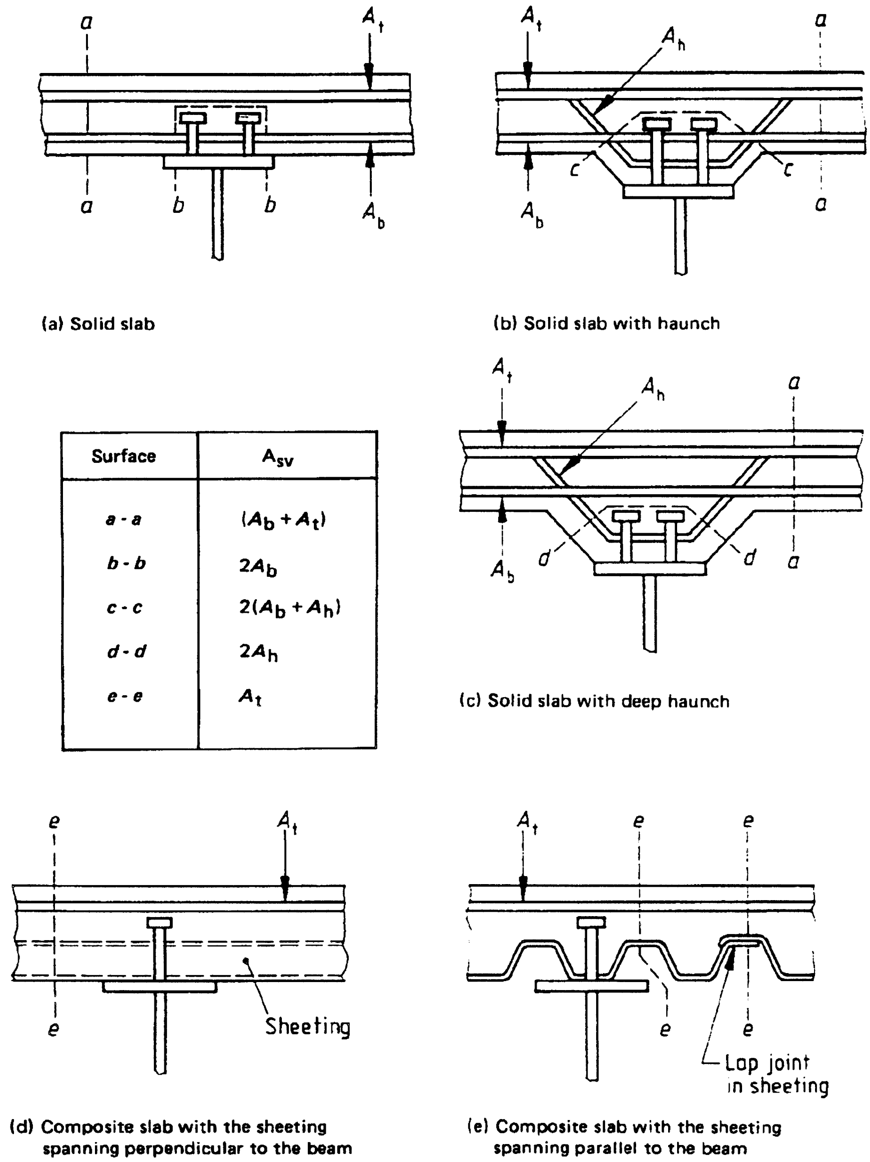
a) Transverse reinforcement should be supplied by U-bars passing around the shear connectors. These U-bars should be located at least 15 mm below the top of the shear connectors (see Figure 8).

b) Where headed studs are used as shear connectors, the distance from the edge of the concrete flange to the centre of the nearest stud should be not less than 6*d*, where *d* is the nominal diameter of the stud, and the U-bars should be not less than 0.5*d* in diameter and detailed as shown in Figure 8.

c) The nominal bottom cover to the U-bars should be the minimum permitted by the design requirements for the concrete flange.

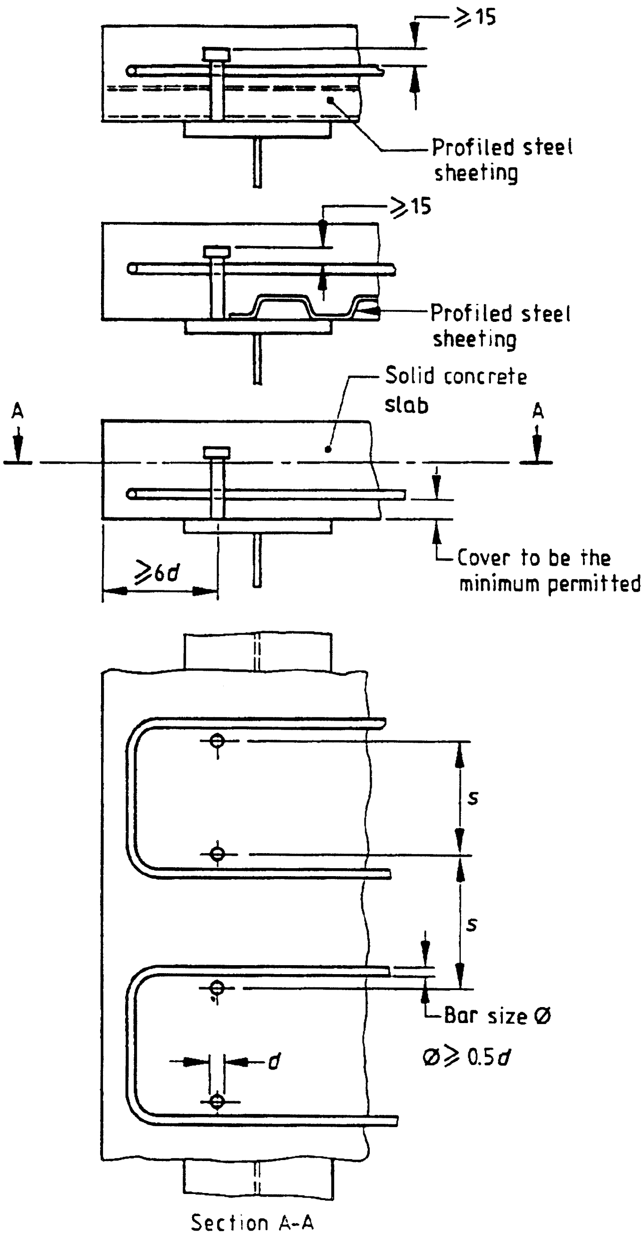
In addition, the recommendations given in **5.6.3** should be met.

NOTE These conditions apply to edge beams and also to beams adjacent to large openings.



Surface	A_{sv}
a - a	$(A_b + A_t)$
b - b	$2A_b$
c - c	$2(A_b + A_h)$
d - d	$2A_h$
e - e	A_t

Figure 7 — Transverse shear surfaces



All dimensions shown are in millimetres.

Figure 8 — Edge beam details

Section 6. Composite beams: serviceability

6.1 Deflections

6.1.1 General

Deflections should be determined under serviceability loads (see **2.4.1**).

For unpropped construction the imposed load deflection should be based on the properties of the composite section but the dead load deflection, due to the self weight of the steel beam and the concrete flange, should be based on the properties of the steel beam.

For propped construction all deflections should be based on the properties of the composite section.

When calculating deflections, the behaviour of composite beams should be taken as linear elastic, except for the redistribution of moments recommended in **6.1.3** and the increased deflections for partial shear connection recommended in **6.1.4**.

6.1.2 Simply supported beams

Deflections of simply supported composite beams should be calculated using the properties of the gross uncracked section described in **4.2.2**.

NOTE For steel beams with equal flanges, the second moment of area of the gross uncracked composite section may be calculated from the formula given in **B.3.1**.

6.1.3 Continuous beams

6.1.3.1 *General.* For continuous beams, the imposed load deflections should allow for the effects of pattern loading. Where design at the ultimate limit state is based on plastic global analysis or on an analysis involving significant redistribution of support moments, the effects of shakedown on deflections should also be included in the imposed load deflections.

As an alternative to rigorous analysis, the methods given in **6.1.3.2** to **6.1.3.5** may be used to allow for the effects of pattern loading and shakedown by modifying the initial support moments.

6.1.3.2 *Allowance for pattern loading.* The initial moments at each support should be determined for the case of unfactored imposed load on all spans. Reductions should then be made to these initial support moments (except adjacent to cantilevers) to allow for pattern loading, as follows:

for normal loading: 30%

for storage loading: 50%

6.1.3.3 *Allowance for shakedown effects.* Allowance should be made for the effects of shakedown if the beam has been designed for the ultimate limit state using:

plastic global analysis (see **5.2.4** and **D.3**);

elastic global analysis, using the properties of the gross uncracked section (see **5.2.3.1**) with redistribution exceeding 40 %;

elastic global analysis, using the properties of the cracked section (see **D.2**) with redistribution exceeding 20 %.

The support moments should be determined, without any redistribution, for the following combination of unfactored loads:

for normal loading: dead load plus 80 % of imposed load;

for storage loading: dead load plus 100 % of imposed load.

Where these support moments exceed the plastic moment capacity of the section for negative moments, the excess moments should be taken as the moments due to shakedown.

The deflections produced by these shakedown moments should be added to the imposed load deflections. This should be done by further reducing the calculated support moments due to imposed loading, by values equal to the shakedown moments, in addition to the reductions for the effects of pattern loading given in **6.1.3.2**.

6.1.3.4 *Calculation of moments.* The support moments required in **6.1.3.2** and **6.1.3.3** should be based on an analysis using the properties of the gross uncracked section throughout. Alternatively, provided the conditions given in **5.2.2** for the simplified method are satisfied, the support moments may be taken as follows:

two-span beam: $WL/8$;

first support in a multi-span beam: $WL/10$

other internal supports: $WL/14$.

In these expressions, W is the appropriate unfactored load on the span L. Where the spans each side of a support differ, the mean of the values of WL. for the two adjacent spans should be used.

6.1.3.5 *Calculation of deflections.* The imposed load deflection in each span should be based on the loads applied to the span and the support moments for that span, modified as recommended to allow for pattern loading and shakedown effects. Provided that the steel beam is of uniform section without any haunches, the properties of the gross uncracked composite section should be used throughout.

The dead load deflections should be based on an elastic analysis of the beam. For unpropped construction, the properties of the steel beam should be used. For propped construction, the properties of the gross uncracked composite section should be used.

For continuous beams under uniform load or symmetric point loads, the deflection δc at mid-span may be determined from the expression:

$$\delta_c = \delta_o((1 - 0.6)(M_1 + M_2)/M_0)$$

where

δ_o is the deflection of a simply supported beam for the same loading;

M_o is the maximum moment in the simply supported beam;

M_1 and M_2 are the moments at the adjacent supports (modified as appropriate).

6.1.4 Partial shear connection

The increased deflection under serviceability loads (see **2.4.1**) arising from partial shear connection should be determined from the following expressions:

for propped construction

$$\delta = \delta_c + 0.5(1 - N_a/N_p)(\delta_s - \delta_c)$$

for unpropped construction

$$\delta = \delta_c + 0.3(1 - N_a/N_p)(\delta_s - \delta_c)$$

where

δ_s is the deflection for the steel beam acting alone;

δ_c is the deflection of a composite beam with full shear connection for the same loading.

For continuous beams, the same formulae apply, but δ_s and δ_c refer to the deflection of the continuous beam, δ_c being calculated as recommended in **6.1.3**.

6.2 Irreversible deformation

In continuous beams, the sagging moments in each span should be increased as necessary to maintain equilibrium with the applied loads, allowing for the reductions in the support moments recommended in **6.1** to allow for the effects of pattern loading and shakedown.

Stresses in simply supported beams and cantilevers and the mid-span regions of continuous beams, under the serviceability loads given in **2.4.1**, should not exceed the limits given in **2.4.3**. It is not necessary to modify the elastic section modulus to take into account partial shear connection at the serviceability limit state.

NOTE For composite beams in which the steel beam has equal flanges, the section modulus of the composite section may be calculated from the formulae given in **B.4**.

6.3 Cracking

Where it is required to limit the crack width, reference should be made to BS 8110.

Where environmental conditions will not give rise to corrosion, such as in heated office buildings, it is not normally necessary to check crack widths, even where the composite beams are designed as simply supported, provided that the concrete flange slab is reinforced as recommended in BS 5950-4 or BS 8110 as appropriate.

NOTE In such cases crack widths may be outside the limits given in BS 8110, but experience has shown that no durability problems arise.

In cases of exposure to adverse environmental conditions (such as floors in car-parking structures or roofs generally) additional reinforcement in the concrete flange over the beam supports may be required to control cracking and the relevant clauses in BS 8110 should be referred to.

To avoid visible cracks where hard finishes are used, the use of crack control joints in the finishes should be considered.

6.4 Vibrations

Where vibration may cause discomfort to the occupants of a building or damage to its contents, the response of long-span composite floors should be considered. If necessary reference should be made to specialist literature.

NOTE For further guidance, reference may be made to SCI Publication 076 "Design Guide on the Vibration of Floors".[4)]

[4)] Available from The Steel Construction Institute, Silwood Part, Buckhurst Road, Ascot, Berks, SL5 7QN.

Appendix A Guidance on additional aspects of construction

A.1 General

A.1.1 The construction of the steel frame should be as specified in BS 5950-2. The construction of composite slabs with profiled steel sheets should be as recommended in BS 5950-4. The construction of reinforced and precast concrete slabs should be in accordance with BS 8110.

A.1.2 This appendix gives guidance on additional aspects of construction which arise when composite construction is used.

A.1.3 A.2 considers aspects of construction which may affect the design process. Information or guidance on these aspects may also need to be sent to site.

A.1.4 A.3 considers additional aspects of construction on which it may be necessary to send information or guidance to site.

A.2 Design requirements

A.2.1 *Construction loads*

The values of construction and storage loads assumed in design should be clearly indicated on the relevant drawings sent to site.

A.2.2 *Sequence of construction*

The sequence of construction should be considered as an integral part of the design process and should be clearly described in the information sent to site.

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

When the composite member carries load before the concrete has attained its characteristic cube strength f_{cu}, the resistance of the shear connectors and the elastic properties and limiting compressive stresses in the concrete should be based upon the cube strength at the time considered f_c. However, no reduction need be made in the modulus of elasticity of the concrete E_c provided that $f_c \geq 0.75f_{cu}$.

For values of $f_c < 25$ N/mm^2, but ≥ 10 N/mm^2, the resistance of a shear connector should be taken as (f_c/25) times its resistance for $f_{cu} = 25$ N/mm^2. No account should be taken of composite action when $f_c < 10$ N/mm^2.

The stage at which props can be removed should be indicated on the relevant drawings, in terms of either cube strength or elapsed time.

A.2.3 *Stability during construction*

The stability of the steel beam during construction should be checked, particularly before the members act compositely.

Profiled steel sheeting spanning onto steel may be assumed to provide restraint to the beam flanges to which it is connected. It should be fixed using any of the following:

- shot fired fixings; or
- self tapping screws; or
- welding (including stud shear connectors welded through the sheeting); or
- bolting.

The spacing of fasteners should be not greater than 500 mm at the ends of sheets, nor greater than 1 000 mm where the sheets are continuous.

The design of the fixings should be in accordance with BS 5950-6.

The stiffness of other types of shuttering or formwork is generally not sufficient to provide the necessary lateral restraint, unless specifically designed to do so.

A.3 Construction procedures

A.3.1 *Construction loads*

Those responsible for controlling the work on site should ensure that the construction and storage loads shown on the relevant drawings are not exceeded.

A.3.2 *Shear connectors*

A.3.2.1 *Stud shear connectors.* The welding procedure, including the required current and other settings of the welding equipment, should be based on the recommendations of the manufacturer of the welding equipment.

Before stud welding is commenced, this procedure should be checked by trials under site conditions, supplemented by appropriate tests. These trials should cover the full range of flange thicknesses and stud sizes to be used. Where welding through profiled steel sheets is required, this should be included in the welding trials.

In these trials a minimum of two test studs should be welded for each case. These studs should then be cold bent manually through an angle of 30° by means of a steel tube placed over the stud. If failure occurs in the weld zone of either stud, the welding equipment should be adjusted, and the tests repeated.

The proper operation of the welding equipment should be rechecked after it has been moved and at the commencement of each shift or other period of work.

All areas to be welded should be free from loose rust, mill scale and grease. Prior to stud welding, prefabrication primer and other paints or coatings should not be used in areas to be welded.

The steel should be kept at a temperature of at least 5 °C during welding. Any areas where moisture is present should be thoroughly dried by the application of heat immediately before welding.

The quality of the stud welding should be checked by visual inspection. If any studs do not show full fusion or a full 360° weld "flash" (fillet), each stud should be cold bent manually through an angle of 15° towards the nearer end of the beam, by means of a steel tube placed over the stud. Provided that further visual inspection does not indicate any crack in the welding, the studs should be accepted and left in the bent position. Any defective studs should be replaced. It is recommended that replacement studs be welded in a new position.

Where the initial visual inspection is satisfactory, a minimum of 1 % of the studs, selected at random, should be tested by cold bending through 15° as above.

A.3.2.2 *Welding through profiled steel sheets.* Where stud shear connectors are to be welded through profiled steel sheets to the supporting beams, any paint or plastic coating on the top of the beams or the underside of the sheets should be removed. The sheets should be in close contact with the steel beam at the time of welding.

Welding through two galvanized profiled steel sheets is not recommended. However, with care, it is possible to weld through a profiled steel sheet overlapping an edge trim. The sheets should be in close contact and the total thickness of the sheeting should not normally exceed 1.25 mm if galvanized or 1.5 mm if not galvanized. The maximum thickness of galvanizing should not normally exceed 30 µm on each sheet face.

A.3.2.3 *Other types of welded shear connectors.* The welding of other types of welded shear connectors should be in accordance with BS 5135.

A.3.2.4 *Non-welded shear connectors.* Where non-welded shear connectors are to be fixed directly to the supporting beam, or fixed to the supporting beam through profiled steel sheets, the method of installation (including the maximum thickness of profiled steel sheet, the minimum required flange thickness and the method of testing) should be based on the recommendations of the manufacturer.

A.3.3 ***Differential deflection of beams and shuttering***

The design of the shuttering and its supports should be such that they can follow the deflected pattern of the steel beams during casting and setting of the concrete.

When unpropped construction is used, measures should be taken to limit any additional thickness of the concrete due to deflection of the steel beams.

A.3.4 ***Compaction of concrete***

All concrete should be compacted as recommended in BS 8110. Special attention should be paid to the critical areas around shear connectors.

A.3.5 ***Propped construction***

Where props are used, they should be kept in place until the in-situ concrete reaches the stage indicated on the relevant drawings, in terms of either cube strength or elapsed time.

A.3.6 ***Spacers***

Spacers should be provided to ensure that the positioning of any reinforcement in a slab is in accordance with the recommendations of BS 8110 for a reinforced concrete slab, or BS 5950-4 for a composite slab.

Appendix B Formulae for calculating section properties

B.1 Introduction

This appendix provides formulae for calculating section properties of composite beams in which the steel beam is a symmetric I or H section with equal flanges. The concrete flange is assumed to be either a solid concrete slab or a composite slab with the profiled steel sheets running perpendicular to the beam. The formulae are conservative in the case of a composite slab with the profiled steel sheets running parallel to the beam.

The following are covered.

a) Plastic moment capacity (see **B.2**).

b) Second moment of area (see **B.3**).

c) Elastic section modulus (see **B.4**).

B.2 Plastic moment capacity

B.2.1 ***Resistances***

The plastic moment capacity is expressed in terms of the resistance of various elements of the beam as follows.

Resistance of concrete flange,	$R_c = 0.45 f_{cu} B_e (D_s - D_p)$
Resistance of steel flange,	$R_f = BTp_y$
Resistance of slender steel beam,	$R_n = R_s - R_v + R_o$
Resistance of slender web,	$R_o = 38 \epsilon t^2 p_y$
Resistance of shear connection,	$R_q = NQ$
Resistance of reinforcement,	$R_r = 0.87 f_y A_r$

Resistance of steel beam, $R_s = Ap_y$

Resistance of clear web depth, $R_v = dtp_y$

Resistance of overall web depth, $R_w = R_s - 2R_f$

where

A is the area of the steel beam;

A_r is the area of the reinforcement in the effective cross section;

B is the breadth of the steel flange;

B_e is the effective breadth of the concrete flange;

D_p is the depth of the profiled steel sheet;

D_s is the overall depth of the concrete flange;

d is the clear depth of the web;

f_{cu} is the characteristic strength of the concrete;

f_y is the characteristic strength of the reinforcement;

N is the actual number of shear connectors for positive or negative moments as relevant (minimum number, one side of the point of maximum moment);

p_y is the design strength of structural steel (in N/mm^2);

Q is the capacity of the shear connectors for positive or negative moments as relevant;

T is the thickness of steel flange;

t is the web thickness;

ϵ is a constant $(275/p_y)^{½}$.

B.2.2 *Positive moments, full shear connection*

Full shear connection applies when R_q is greater than (or equal to) the lesser of R_c and R_s.

In a composite section with full shear connection, where the steel beam has equal flanges the plastic moment capacity M_c for positive moments is given by the following:

Case 1: $R_c < R_w$ (plastic neutral axis in web)

a) $\frac{d}{t} \leqslant 76\epsilon$ or $\frac{d}{t} \leqslant \frac{76\epsilon}{1 - R_c/R_v}$ (web compact)

$$M_c = M_s + R_c \frac{(D + D_s + D_p)}{2} - \frac{R_c^2}{R_v} \frac{d}{4}$$

where

D is the overall depth of the steel beam;

M_s is the plastic moment capacity of the steel beam.

b) $\frac{d}{t} > \frac{76\epsilon}{1 - R_c/R_v}$ (web not compact)

$$M_c = M_s + R_c \frac{(D + D_s + D_p)}{2} - \frac{R_c^2 + (R_v - R_c)(R_v - R_c - 2R_o)}{R_v} \frac{d}{4}$$

Case 2: $R_c \geq R_w$ (plastic neutral axis in flange)

a) $R_s > R_c$ (plastic neutral axis in steel flange)

$$M_c = R_s \frac{D}{2} + R_c \frac{(D_s + D_p)}{2} - \frac{(R_s - R_c)^2}{R_f} \frac{T}{4}$$

NOTE The last term in this expression is generally small.

b) $R_s \leq R_c$ (plastic neutral axis in concrete flange).

$$M_c = R_s \left\{ \frac{D}{2} + D_s - \frac{R_s}{R_c} \frac{(D_s - D_p)}{2} \right\}$$

B.2.3 *Positive moments, partial shear connection*

Partial shear connection applies when R_q is less than both R_c and R_s.

In a composite section with partial shear connection, where the steel beam has equal flanges the plastic moment capacity M_c for positive moments is given by the following:

Case 3: $R_q < R_w$ (plastic neutral axis in web)

a) $\frac{d}{t} \leqslant 76\epsilon$ or $\frac{d}{t} \leqslant \frac{76\epsilon}{1 - R_q/R_v}$ (web compact)

$$M_c = M_s + R_q \left\{ \frac{D}{2} + D_s - \frac{R_q}{R_c} \frac{(D_s - D_p)}{2} \right\} - \frac{R_q^2}{R_v} \frac{d}{4}$$

b) $\frac{d}{t} > \frac{76\epsilon}{1 - R_q/R_v}$ (web not compact)

$$M_c = M_s + R_q \left\{ \frac{D}{2} + D_s - \frac{R_q}{R_c} \frac{(D_s - D_p)}{2} \right\} - \frac{R_q^2 + (R_v - R_q)(R_v - R_q - 2R_o)}{R_v} \frac{d}{4}$$

Case 4: $R_q \geq R_w$ (plastic neutral axis in flange)

$$M_c = R_s \frac{D}{2} + R_q \left\{ D_s - \frac{R_q}{R_c} \frac{(D_s - D_p)}{2} \right\} - \frac{(R_s - R_q)^2}{R_f} \frac{T}{4}$$

B.2.4 ***Negative moments***

In a composite section where the steel beam has equal flanges, the plastic moment capacity M_c for negative moments is given by the following:

Case 5: Plastic neutral axis in web

a) $\frac{d}{t} \leqslant 38\epsilon$ or $\frac{d}{t} \leqslant \frac{76\epsilon}{1 + R_r/R_v}$ (web compact)

$R_r < R_w$

$$M_c = M_s + R_r \left(\frac{D}{2} + D_r \right) - \frac{R_r^2}{R_v} \frac{d}{4}$$

where

D_r is the distance from the top of the steel beam to the centroid of the reinforcement.

b) $\frac{d}{t} > 38\epsilon$ and $\frac{d}{t} > \frac{76\epsilon}{1 + R_r/R_v}$ (web not compact)

i.e

$R_r < R_o$

$$M_c = M_s + R_r \left(\frac{D}{2} + D_r \right) - \frac{R_r^2 + (R_v + R_r)(R_v + R_r - 2R_o)}{R_v} \frac{d}{4}$$

Case 6: Plastic neutral axis in flange

a) $\frac{d}{t} \leq 38\epsilon$ (web compact)

i.e.

$R_r \geq R_w$

i) $R_r < R_s$ (plastic neutral axis in steel flange)

$$M_c = R_s \frac{D}{2} + R_r D_r - \frac{(R_s - R_r)^2}{R_f} \frac{T}{4}$$

ii) $R_r \geq R_s$ (plastic neutral axis outside steel beam)

$$M_c = R_s \left(\frac{D}{2} + D_r \right)$$

b) $\frac{d}{t} \leq 38\varepsilon$ (web not compact)

i.e.

$R_r \geq R_o$

i) $R_r < R_n$ (plastic neutral axis in steel flange)

$$M_c = R_n \frac{D}{2} + R_r D_r - \frac{(R_n - R_r)^2}{R_f} \frac{T}{4}$$

ii) $R_r \geq R_n$ (plastic neutral axis outside steel beam)

$$M_c = R_n \left(\frac{D}{2} + D_r \right)$$

B.3 Second moment of area

B.3.1 ***Uncracked section***

For a composite section in which the steel beam has equal flanges, the gross value of second moment of area I_g of the uncracked section is given by the expression:

$$I_g = I_x + \frac{B_e (D_s - D_p)^3}{12 \alpha_e} + \frac{A B_e (D_s - D_p)(D + D_s + D_p)^2}{4 \{ A \alpha_e + B_e (D_s - D_p) \}}$$

B.3.2 ***Cracked section, negative moments***

For a composite section in which the steel beam has equal flanges, the second moment of area I_n of the cracked section for negative moment is given by the expression:

$$I_n = I_x + \frac{A A_r (D + 2D_r)^2}{4(A + A_r)}$$

B.3.3 ***Cracked section, positive moment***

For a composite section in which the steel beam has equal flanges, the second moment of area I_p of the cracked section for positive moments is given by the expression:

$$I_p = I_x + \frac{B_e y_e^3}{3 \alpha_e} + A \left(\frac{D}{2} + D_s - y_e \right)^2$$

in which y_e is the depth of the elastic neutral axis below the top of the concrete flange given by the expression:

$$y_e = \frac{(D + 2D_s)}{1 + \left\{ 1 + \frac{B_e}{A \alpha_e} (D + 2D_s) \right\}^{1/2}}$$

B.4 Elastic section modulus

B.4.1 *Positive moments*

For a composite section in which the steel beam has equal flanges, the elastic section moduli for positive moments are given by the following:

Case 1: Elastic neutral axis in concrete flange

This case applies when:

$$A < \frac{(D_s - D_p)^2 B_e}{(D + 2D_p)\alpha_e}$$

In this case concrete on the tension side of the elastic neutral axis is taken as cracked and the properties of the cracked section are used. The elastic section modulus Z_p for the concrete flange is then given by the expression:

$$Z_p = l_p \alpha_e / y_e$$

and the elastic section modulus Z_s for the bottom flange of the steel member is given by the expression:

$$Z_s = l_p/(D + D_s - y_e)$$

where

l_p and y_e are obtained from **B.3.3**.

Case 2: Elastic neutral axis in steel member

This case applies when:

$$A \geqslant \frac{(D_s - D_p)^2 B_e}{(D + 2D_p)\alpha_e}$$

In this case the concrete is uncracked and the gross section properties apply. The depth y_g of the elastic neutral axis below the top of the concrete flange is given by the expression:

$$y_g = \frac{A\alpha_e(D + 2D_s) + B_e(D_s - D_p)^2}{2\left\{A\alpha_e + B_e(D_s - D_p)\right\}}$$

The elastic section modulus for the concrete flange is then given by the expression:

$$Z_g = l_g \alpha_e / y_g$$

and for the bottom flange of the steel member:

$$Z_s = l_g/(D + D_s - y_g)$$

where

l_g is obtained from **B.3.1**.

B.4.2 *Negative moments*

For a composite section in which the steel beam has equal flanges, for negative moments the depth y_r of the elastic neutral axis below the centroid of the reinforcement is given by the expression:

$$y_r = \frac{A(D + 2D_r)}{2(A + A_r)}$$

The elastic section modulus for the stress in the reinforcement is then given by the expressions:

$$Z_r = l_n / y_r$$

and for the bottom flange of the steel member:

$$Z_s = I_n/(D + D_r - y_r)$$

where

l_n is obtained from **B.3.2**.

Appendix C Classification of webs

C.1 Plastic stress distribution

C.1.1 *Steel beam with equal flanges*

For a plastic distribution of stresses on a composite cross section in which the steel beam has equal flanges, the ratio r for use in **4.5.2** is given by the following relationships:

for positive moments

$$r = -F_c/R_v \text{ but } r \geq -1$$

for negative moments

$$r = R_r/R_v \text{ but } r \leq 1$$

where

F_c is the compressive force in the concrete flange;

R_r is the resistance of the reinforcement $0.87 f_y A_r$;

R_v is the resistance of the clear web depth dtp_y;

f_y is the characteristic strength of the reinforcement;

A_r is the area of the reinforcement in the effective cross section;

d is the clear depth of the web;

t is the web thickness;

p_y is the design strength of structural steel.

C.1.2 *Steel beam with unequal flanges*

For a plastic distribution of stresses on a composite cross section in which the steel beam has unequal flanges, the ratio r for use in **4.5.2** is given by the following relationships:

for positive moments

$$r = -(F_c + R_{fc} - R_{ft})/R_v \text{ but } r \geq -1$$

for negative moments

$$r = (R_r + R_{ft} - R_{fc})/R_v \text{ but } (r \leq 1)$$

where

R_{fc} and R_{ft} are the values of the resistance R_f for the compression and tension flanges respectively and $R_f = BT_{Py}$ (see **B.2.1**).

C.1.3 ***Force in concrete flange***

For a plastic distribution of stresses on a composite cross section, the compressive force F_c in the concrete flange is generally equal to the lesser of R_c and R_q, where R_c is the resistance of the concrete flange and R_q is the resistance of the shear connection, as given by the following expressions:

for full shear connection

$$F_c = R_c = 0.45 f_{cu} B_e (D_s - D_p) \qquad \text{(see } \mathbf{B.2.1}\text{)}$$

for partial shear connection

$$F_c = R_q = N_a\, Q_p \qquad \text{(see } \mathbf{5.5.2}\text{)}$$

When checking the moment capacity at an intermediate point:

$$F_c = (N_a - N_n)\, Q_p \qquad \text{(see } \mathbf{5.4.5.5}\text{)}$$

NOTE The force F_c may also be limited by the resistance of the steel beam R_s but in this case the plastic neutral axis will be in the concrete flange and the value of r will not be required.

C.2 Elastic stress distribution

For an elastic stress distribution on a composite cross section in which the steel beam has equal flanges, the ratio r for use in **4.5.2** is given by the following relationships:

for positive moments

$r = -F_c/R_s$ but $r \geq -1$

for negative moments

$r = F_r/R_s$ but $r \leq 1$

where

F_c is the compressive force in the concrete flange;

F_r is the tensile force in the reinforcement;

R_s is the resistance of the steel beam Ap_y.

For composite cross sections in which the steel beam has unequal flanges use the formula given in Figure 1 b).

Appendix D General methods for determining moments in continuous beams

D.1 General

The following methods may be used to determine the moments in continuous beams, as alternatives to those in **5.2**.

a) Elastic analysis, using cracked section properties (see **D.2**).

b) Plastic analysis, general method (see **D.3**).

The shear forces should be compatible with the final bending moment distribution. Pattern loads should be as given in **5.2.3.2**.

D.2 Elastic analysis, using cracked section properties

As an alternative to the method given in **5.2.3** elastic global analysis may be carried out assuming that for a length of 15 % of the span on each side of internal supports, the section properties are those of the cracked section for negative moments (see **4.2.3**). Elsewhere the section properties of the gross uncracked section are used. The resulting moments may be adjusted, as described in **5.2.3.1**, by an amount not exceeding the appropriate maximum percentage given in Table 6.

It is also permissible to iteratively adjust the length of span which is assumed to be cracked on each side of an internal support, to correspond to the points of contraflexure determined from the redistributed moment diagram.

Table 6 — Maximum redistribution of support moments for elastic global analysis, using properties of cracked section

Classification of compression flange at support				
Class 4	Class 3	Class 2	Class 1 Plastic	
Slender	Semi-compact	Compact	Generally	Non-reinforced (see 5.2.1.2)
%	%	%	%	%
0	10	20	30	40

D.3 Plastic analysis, general method

D.3.1 ***General***

As an alternative to the method given in **5.2.4**, plastic global analysis may be used to determine the moments in continuous beams subject to the following conditions.

a) Adjacent spans should not differ by more than 33 % of the larger span.

b) End spans should not exceed 115 % of the length of the adjacent span.

c) In any span in which more than half the total factored load on a span is concentrated within a length of one-fifth of the span, the cross section at each positive moment plastic hinge location should be such that the plastic neutral axis lies within 0.15 $(D + D_s)$ below the top of the concrete flange, where D_s is the depth of the concrete flange. This condition need not be satisfied where it can be shown that the hinge will be the last to form in that span.

d) At plastic hinge locations, both the compression flange and the web should be class 1 plastic.

e) Unless the steel beam is of uniform section without haunches, the conditions given in **D.3.2** should also be satisfied.

D.3.2 ***Beams of non-uniform cross section***

Where the cross section of the steel beam varies along its length, the following additional conditions should also be satisfied.

a) Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance along the beam from the plastic hinge location of at least 2d, where d is the clear depth of the web at the plastic hinge location.

b) Adjacent to plastic hinge locations, the compression flange should be class 1 plastic for a distance along the beam from the plastic hinge location of not less than the greater of:

1) 2d, where d is as defined in a).

2) the distance to the point at which the moment in the beam has fallen to 0.8M_c, where M_c is the plastic moment capacity at the point concerned.

c) Elsewhere the compression flange should be class 1 plastic or class 2 compact and the web should be class 1 plastic, class 2 compact or class 3 semi-compact.

blank

Publications referred to

BS 18, *Method for tensile testing of metals (including aerospace materials).*

BS 4360, *Specification for weldable structural steels.*

BS 4395, *Specification for high strength friction grip bolts and associated nuts and washers for structural engineering.*

BS 4395-1, *General grade.*

BS 4449, *Specification for carbon steel bars for the reinforcement of concrete.*

BS 4482, *Specification for cold reduced steel wire for the reinforcement of concrete.*

BS 4483, *Specification for steel fabric for the reinforcement of concrete.*

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

BS 5400, Steel, , *concrete and composite bridges.*

BS 5400-3, *Code of practice for design of steel bridges.*

BS 5400-5, *Code of practice for design of composite bridges.*

BS 5950, *Structural use of steelwork in building.*

BS 5950-1, *Code of practice for design in simple and continuous construction: hot rolled sections.*

BS 5950-2, *Specification for materials, fabrication and erection: hot rolled sections.*

BS 5950-3.2, *Code for practice for design of composite columns and frames*[5].

BS 5950-4, *Code of practice for design of floors with profiled steel sheeting.*

BS 5950-8, *Code of practice for fire resistant design.*

BS 5950-9, *Code of practice for stressed skin design*[5].

BS 5975, *Code of practice for falsework.*

BS 6399, *Loading for buildings.*

BS 6399-1, *Code of practice for dead and imposed loads.*

BS 6399-3, *Code of practice for imposed roof loads.*

BS 8110, *Structural use of concrete.*

CP 3, *Code of basic data for the design of buildings.*

CP 3:Chapter V, *Loading.*

CP 3-2, *Wind loads.*

[5] In preparation.

BSI — British Standards Institution

BSI is the independent national body responsible for preparing British Standards. It presents the UK view on standards in Europe and at the international level. It is incorporated by Royal Charter.

Revisions

British Standards are updated by amendment or revision. Users of British Standards should make sure that they possess the latest amendments or editions.

It is the constant aim of BSI to improve the quality of our products and services. We would be grateful if anyone finding an inaccuracy or ambiguity while using this British Standard would inform the Secretary of the technical committee responsible, the identity of which can be found on the inside front cover. Tel: 020 8996 9000. Fax: 020 8996 7400.

BSI offers members an individual updating service called PLUS which ensures that subscribers automatically receive the latest editions of standards.

Buying standards

Orders for all BSI, international and foreign standards publications should be addressed to Customer Services. Tel: 020 8996 9001. Fax: 020 8996 7001.

In response to orders for international standards, it is BSI policy to supply the BSI implementation of those that have been published as British Standards, unless otherwise requested.

Information on standards

BSI provides a wide range of information on national, European and international standards through its Library and its Technical Help to Exporters Service. Various BSI electronic information services are also available which give details on all its products and services. Contact the Information Centre. Tel: 020 8996 7111. Fax: 020 8996 7048.

Subscribing members of BSI are kept up to date with standards developments and receive substantial discounts on the purchase price of standards. For details of these and other benefits contact Membership Administration. Tel: 020 8996 7002. Fax: 020 8996 7001.

Copyright

BSI
389 Chiswick High Road
London
W4 4AL