COMPOSITE DESIGN OF STEEL FRAMED BUILDINGS
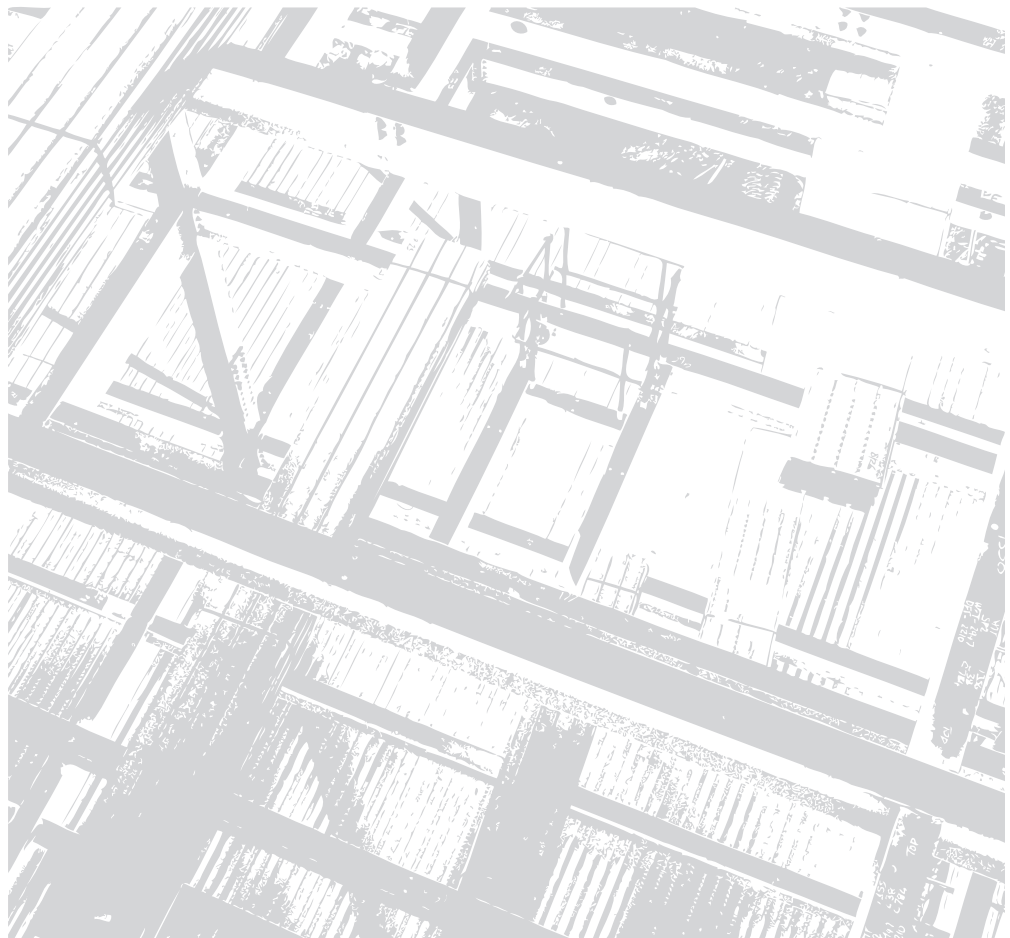
COMPOSITE DESIGN OF STEEL FRAMED BUILDINGS

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In accordance with Eurocodes and the UK National Annexes

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FOREWORD

This publication is aimed at structural engineers designing buildings in the UK in accordance with the Eurocodes and is intended to provide guidance equivalent to that of previous SCI publications covering design in accordance with BS 5950. The general advice provided in the earlier publications is not invalidated by the change of design standard and is repeated here where appropriate.

The Eurocodes, in particular BS EN 1994-1-1, *Eurocode 4. General rules and rules for buildings*, introduce significant differences in the detailed design of composite beams and slabs, as well as numerous changes to nomenclature. Comparisons with traditional UK practice have been made where relevant and important differences are remarked upon.

For buildings in the UK, the Nationally Determined Parameters (NDP) given in the relevant UK National Annexes are used. For design of buildings in other countries, different NDP may apply – the designer must consult the national annexes for the country in which the building is constructed.

This guide complements a range of other SCI publications related to building design to the Eurocodes.

The text of this document has been prepared by Dr Ian Simms and Mr Alastair Hughes with assistance from Ms Mary Brettle. Mr John Lucey has produced the worked example.

CONTENTS

SUMMARY

This publication presents a design methodology for simply supported composite beams and composite slabs in accordance with *Eurocode 4: Design of steel and concrete composite structures* and its UK National Annex. The guide covers composite slabs formed on profiled steel sheeting and I section steel beams that are made to act compositely with the slab by means of shear connectors.

Loading at the construction stage is discussed and guidance is given on the design of the sheeting and the bare steel beam. For the normal stage, guidance is given on determining the bending resistance and for verifying the adequacy of the shear connection. Serviceability criteria are also discussed.

Simple guidance is given on the fire resistance of composite slabs and on the critical temperatures for composite beams (which would allow the level of protection needed to be determined).

Design examples are appended to illustrate typical current practice.

INTRODUCTION

In *Steel building design: Medium rise braced frames* (P365)[1], general guidance is given on a range of floor systems suitable for steel framed buildings. Many of those systems involve use of a composite floor slab – concrete acting compositely with profiled steel sheeting – and most use steel beams acting compositely with the floor slab.

Composite construction takes advantage of the particular qualities of concrete and steel. The steel beams and profiled steel sheeting act as permanent falsework and formwork for the wet concrete. Once cured, the concrete can provide all or most of the compression force needed for bending resistance of the floor slab or beams. The steel beam provides all the tension force for bending resistance and, due to the stabilizing effect of its connection to the concrete slab, as much compression force as is called for.

1.1   Benefits of composite construction

A composite floor represents an outstandingly efficient use of materials, providing quick, cost effective and sustainable construction. Composite construction is robust and does not require tight tolerances, making the system quick to construct.

A composite floor system has a low self weight, which has a direct impact on the vertical structure and foundation size.

For urban multi-storey buildings in the UK, steel frames with composite floors have come to dominate the market over the past 25 years.

1.2   Scope of this publication

This publication provides guidance on the design of composite floor slabs and composite floor beams, in accordance with the Eurocodes, principally with Eurocode 4. The guidance covers floor slabs and beams that are part of a multi-storey braced frame building in which the floor beams are designed as simply supported. Guidance on frame design is covered separately in *Steel building design: Medium rise braced frames* (P365) and connection design is covered in *Joints in steel construction: Simple joints to Eurocode 3* (P358)[2].

The composite slabs covered in this guide are those cast in situ on profiled steel sheeting, with profile heights in the range 45 to 100 mm, creating an overall slab

thickness of 100 to 200 mm. Mesh is incorporated within the slab to serve a number of functions, including:

- Reducing and containing cracking of the concrete at the supports due to flexural tension and shrinkage of the concrete.
- Strengthening the edges at openings.
- Providing negative bending resistance over the internal supporting beams in fire conditions.
- Acting as transverse reinforcement for the composite beams.
- Ensuring adequate performance of the shear connectors.

Design of the slab is usually based on information published by profile steel sheeting suppliers (in the form of load/span and fire resistance tables) for their proprietary profile shapes. The design resistances given by the rules discussed in Section 5 are usually determined by the manufacturer based on test results, rather than purely by calculation.

The design of thicker composite slabs using deep steel sheeting, as employed in *Slimflor®* solutions, is outside the scope of the publication. Guidance on the design of *Slimdek* in accordance with the Eurocodes is published in the *Design of Asymmetric Slimflor® Beams to Eurocodes*[3].

This publication covers primarily the use of hot rolled I sections forming secondary and primary composite beams. The composite slab sits on top of the steel beam and composite action is achieved by means of shear connectors on the top flange. The guidance also covers the use of fabricated beams, although these are less commonly used. Cellular beams and other beams with large web openings are outside the scope of the publication.

The design of composite columns is also outside the scope of the publication, but some general remarks about this form of construction are given in Section 8.

1.3 Design to the Eurocodes

The Eurocodes are a set of standards that provide principles and rules for the design of all common types of structures. For a general introduction to those standards, see *Steel building design: Introduction to the Eurocodes* (P361)[4].

In the UK, the Eurocodes are published by BSI in a number of separate Parts; each Part has a National Annex that gives the national choices (where permitted) for design of structures in the UK. Designers will need to refer to BS EN 1990 for the design basis and BS EN 1991 (Eurocode 1) for actions (loads); for verification of composite structures, designers will need to refer to BS EN 1994 (Eurocode 4), BS EN 1993 (Eurocode 3) and BS EN 1992 (Eurocode 2).

In addition to the Eurocode Parts and National Annexes, reference may also be made to 'non-contradictory complementary information' (NCCI). Such information ranges from text book material to new documents produced in response to issues raised

by the Eurocodes. Of particular interest to the designer of composite beams is NCCI related to resistance of stud connectors that is available on www.steel-ncci.co.uk.

For brevity, references to clauses, figures and tables in the Eurocode Parts are given as, for example, 4-1-1/6.3, meaning Clause 6.3 of BS EN 1994-1-1. Clauses in the National Annexes are distinguished by the prefix NA.

Terminology in the Eurocodes differs in some cases from that traditionally used in the UK – notable examples are the use of 'actions' as a general term for loads and imposed displacements and 'execution' for the whole construction process, from the supply of products and fabrication to erection and in situ construction. In relation to the steel component in composite floor slabs, Eurocode 4 refers to 'profiled steel sheeting', rather than to 'decking'. SCI design guides have generally adopted Eurocode terminology, rather than traditional terminology, to encourage familiarity and to avoid potential confusion with the Eurocode requirements. Consequently, for the composite slab, this publication therefore refers only to 'profiled steel sheeting', even though it is recognized that suppliers often refer to their products as 'decking'.

1.4   Combination of actions

BS EN 1990 sets out the basis of structural design to the Eurocodes and establishes the combinations of actions that should be considered for design. The following combinations are considered for the normal stage design, when the structural elements behave compositely. Reference should be made to Section 3 for the combinations to consider for design at the construction stage and to Section 7 for combinations to use in fire design.

1.4.1   Ultimate limit state

The combinations of actions that are to be considered at the ultimate limit state are given in BS EN 1990, 6.4. For the resistance of the structure and ground limit states, the design value of actions may be determined from expression 6.10 or from the less favourable of expressions 6.10a and 6.10b.

The first option is to express the combination of actions as:

$$\sum_{j\geq1}\gamma_{G,j}G_{k,j}\;"+"\;\gamma_P P\;"+"\;\gamma_{Q,1}Q_{k,1}\;"+"\;\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad (6.10)$$

The second option is to express the combination of actions as the less favourable of the following two expressions:

$$\sum_{j\geq1}\gamma_{G,j}G_{k,j}\;"+"\;\gamma_P P\;"+"\;\gamma_{Q,1}\psi_{0,1}Q_{k,1}\;"+"\;\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad (6.10a)$$

$$\sum_{j\geq1}\xi_j\gamma_{G,j}G_{k,j}\;"+"\;\gamma_P P\;"+"\;\gamma_{Q,1}Q_{k,1}\;"+"\;\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad (6.10b)$$

where

ξ is a reduction factor applied to unfavourable permanent actions (in 6.10b).

The National Annex for the country in which the building is to be constructed must be consulted for guidance on which option to use. In the UK, the National Annex allows either approach to be used. However, in almost all situations in the UK, the use of the second option (the use of expressions 6.10a and 6.10b) will produce lower design values of the effects of actions (and for buildings, 6.10b usually gives the more onerous value). For the construction stage, expression 6.10a governs.

1.4.2   Serviceability limit state

The expressions for the combinations of actions for serviceability limit state design are given in BS EN 1990, 6.5.3 for three possible combinations:

Characteristic combination:

$$\sum_{j\geq1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{6.14b}$$

Frequent combination:

$$\sum_{j\geq1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{6.15b}$$

Quasi-permanent combination:

$$\sum_{j\geq1} G_{k,j} + P + \sum_{i\geq1} \psi_{2,i} Q_{k,i} \tag{6.16b}$$

The choice of combination depends on the effect being considered (see Section 6).

MATERIALS AND STRUCTURAL COMPONENTS

This Section discusses the materials and structural components that are used in composite construction: structural steel, concrete, reinforcement, steel sheeting and shear connectors.

2.1 Structural steel

2.1.1 Steel material

Although the design rules given in BS EN 1994-1-1[5] cover structural steels with nominal yield strengths up to and including 460 N/mm² (S460), grade S460 is not in regular use in the UK. Experience suggests that stiffness limitations would rarely allow significant advantage of grades higher than S355; in some cases there is little advantage in using grades higher than S275.

If grade S420 or S460 is used, it should be noted that in some cases, depending on the position of the plastic neutral axis, the plastic moment resistance of the section is subject to a reduction factor, as given by 4-1-1/6.2.1.2(2) and 4-1-1/Figure 6.3.

BS EN 1994-1-1 refers to BS EN 1993-1-1 for the determination of the nominal yield and ultimate strength. 3-1-1/3.2.1 allows either the use of 3-1-1/Table 3.1 or the nominal values given in the product standards. The UK National Annex specifies the use of the product standards instead of Table 3.1. Nominal yield and ultimate strengths for steels used for rolled or plated open sections are given in Table 7 of BS EN 10025-2[6]; values for S275 and S355 are reproduced here in Table 2.1 The values given for the ultimate strength are the lower values of the range given in BS EN 10025-2.

	YIELD STRENGTH (f_y) N/mm² FOR NOMINAL THICKNESS (mm)			ULTIMATE STRENGTH (f_u) N/mm² FOR NOMINAL THICKNESS (mm)	
	≤ 16	> 16 ≤ 40	> 40 ≤ 63	< 3	≥ 3 ≤ 100
S275	275	265	255	430	410
S355	355	345	335	510	470

Table 2.1 Nominal values of yield strength and ultimate strength for steel

Eurocode 4 uses the design yield strength to determine design resistance, which is given by:

$$f_{yd} = \frac{f_y}{\gamma_{Mi}}$$

where

γ_{Mi} is the appropriate partial factor for structural steel described. For the resistance of cross sections, $\gamma_{M0} = 1.0$; for member stability, $\gamma_{M1} = 1.0$; and for the tensile resistance to facture, $\gamma_{M2} = 1.25$ (all values from 3-1-1/NA.2.15).

2.1.2   Steel beams

This guide is principally aimed at construction using UKB and UKC rolled steel sections but the principles given for the calculation of bending and shear resistance can be readily applied to sections fabricated from rolled sections or plates.

For plastic design of composite beams in accordance with 4-1-1/6.2.1, a Class 1 or 2 section is required to avoid issues with local buckling in the compression elements of the steel section. The flange of the steel section needs to be sufficiently wide to allow the sheeting to be butt jointed if required. In order to satisfy requirements for minimum bearing lengths, a minimum flange width of 150 mm is recommended.

For headed shear studs welded through the steel sheeting profile, a minimum flange thickness of 0.4 times the stud diameter is required. With a 19 mm stud, the thickness required is therefore 7.6 mm. All but two UKB sections meet this requirement.

Where beams are fabricated from rolled sections or plates it is possible to introduce asymmetry into the section, saving weight from the top flange where the tensile resistance of the steel is least effective.

Cellular and castellated beams can also be used compositely. While much of this publication is relevant, these alternatives are covered in detail in *Design of composite beams with large web openings* (P355)[7].

2.2   Concrete

Either normal weight or lightweight concrete may be used in composite floors. Lightweight concrete offers the advantage of extra slab spanning capability due to the weight saving. However, this must be balanced against the effect of a reduced modulus of elasticity, which leads to greater deflections and to lower stud resistances.

The recommendations given in Eurocode 4 are limited to concrete strength classes between C20/25 and C60/75 for normal weight concrete, and LC20/22 and LC60/66 for lightweight concrete.

The material properties for concrete are given by BS EN 1992-1-1[8], either by Table 3.1 (normal weight concrete) or by Table 11.3.1 (lightweight concrete). Values for f_{ck} and E_{cm} are given below in Table 2.2.

*Table 2.2
Properties for
some common
concrete classes
(from 2-1-1/Table 3.1
and Table 11.3.1)*

	C25/30	C30/37	LC25/28 1800 kg/m³	LC30/33 1800 kg/m³
Characteristic cylinder compressive strength (f_{ck} **or** f_{ckl} (N/mm²))	25	30	25	30
Secant modulus of elasticity (E_{cm} **or** E_{cml} (GPa))	31	33	21	22

2-1-1/3.1.6(1)P defines the design value of compressive strength of concrete (f_{cd}) as follows:

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_C}$$

where

α_{cc} is the coefficient taking account of long term effects on the compressive strength and the unfavourable effects resulting from the way load is applied

γ_C is the partial factor for concrete; $\gamma_C = 1.5$ according to 2-1-1/Table NA.1.

On the basis of extensive calibration studies, a value of α_{cc} of 1.0 is appropriate for use with the expressions for determining member resistance of composite sections. Therefore, in 4-1-1/2.4.1.2(2)P, the design value of compressive strength of concrete (f_{cd}) is defined without reference to α_{cc} as follows:

$$f_{cd} = \frac{f_{ck}}{\gamma_C}$$

When used with the simple rectangular stress block model assumed in Eurocode 4, this value of f_{cd} is multiplied by the coefficient 0.85 for beams, slabs and most columns.

Ordinary Portland cement is an energy-intensive product which can and should be replaced by alternatives as far as practical. Commonly available cement replacements are ground granulated blast furnace slag and pulverized-fuel ash. Up to 50% replacement is practical with ground granulated blast furnace slag; rather less with pulverized-fuel ash. The designer should consider the effect of such substitution on the strength gain of the concrete and any negative effect this may have on the structure, for example in determining creep effects.

In most building structures, it will not be necessary or cost-effective to specify concrete stronger than C30/37. No structural advantage will be obtained with stronger classes, and in practice the durability of C25/30 is adequate for the internal environment of most composite floors. Class C30/37 may be preferred if a more severe risk of carbonation induced corrosion is anticipated, or if the slab is to be a wearing surface.

2.2.1 Shrinkage

Shrinkage in concrete occurs during curing (autogenous shrinkage) and as the concrete dries out (drying shrinkage). In accordance with 4-1-1/3.1(4), the effects of

autogenous shrinkage may be neglected when determining stresses and deflections. However, the strains due to drying shrinkage are more significant and will need to be considered in structural calculations when the span to depth ratio is greater than 20.

2.2.2 Creep

Concrete can also develop significant time dependant strains due to creep effects, which need to be considered in composite construction.

2.3 Reinforcement

4-1-1/3.2 refers the designer to 2-1-1/3.2 to obtain the properties of reinforcing steel. However, for composite construction 4-1-1/3.2(2) states that the design value of the modulus of elasticity for reinforcing steel (E_s) may be taken as equal to that of structural steel given in 3-1-1/3.2.6(1). Thus, for composite design:

$$E_s = E = 210\ 000 \text{ N/mm}^2$$

The yield strength and ductility of ribbed weldable reinforcing steel material in either bar or fabric should be specified in accordance with the requirements of BS EN 10080[9]. The characteristic yield strength of reinforcement to BS EN 10080 will be between 400 N/mm² and 600 N/mm², depending on the national market. As noted in 2-1-1/5.6.3, to ensure that the reinforcement has sufficient ductility for plastic analysis, Class B or Class C should be specified.

The reinforcement industry in the UK has decided to standardise on grade 500C reinforcing steel, which has a characteristic yield strength, f_{yk}, of 500 N/mm² and the superior ductility that composite applications demand. The design strength of reinforcement is given by 2-1-1/3.2.7 as:

$$f_{yd} = \frac{f_{yk}}{\gamma_s}$$

where

f_{yk} is the yield strength (0.2% proof stress)

γ_s is the partial factor for reinforcing steel, $\gamma_s = 1.15$.

In accordance with 2-1-1/Table 2.1N and Table NA.1, the partial factor for reinforcing steel is taken as 1.15. In Eurocode 4, the design strength of reinforcement is denoted as f_{sd}.

National standards for the specification of reinforcement will be revised in order to align with BS EN 10080 and retained as non-contradictory complementary information (NCCI), as a common range of steel grades has not been agreed for BS EN 10080. In the UK, bar reinforcement and mesh reinforcement for design to Eurocode 4 should be specified in accordance with BS 4449[10] and BS 4483[11] respectively.

Mesh reinforcement is generally used in the top of a composite slab for crack control. Typically A series mesh in accordance with BS 4483 is specified. This mesh type has a 200 mm bar spacing in both directions and is commonly available as fabric references A142, A193, A252 and A393. The number in the reference is the wire area per metre (mm²/m) in each direction.
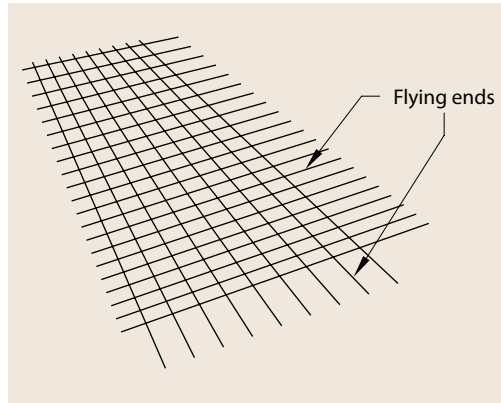

Figure 2.1
Mesh with flying ends

Typically, sheets of mesh reinforcement are 4.8 m by 2.4 m and therefore must be lapped to achieve continuity of the reinforcement. Sufficient lap lengths must therefore be specified and adequate site control put in place to ensure that such details are implemented on site. Lap lengths for mesh reinforcement can be calculated using the methods given in 2-1-1/8.7.5. Table 2.3 shows the calculated lap lengths for typical wire sizes and concrete grades, based on the nominal yield strength of 500 N/mm² (as a lap could coincide with a beam), and have cover of at least three diameters. In practice, a minimum lap length of 250 mm is used for mesh reinforcement. Ideally, mesh should be specified with 'flying ends', as shown in Figure 2.1, to eliminate build up of bars at laps. It will often be economic to order 'ready fit fabric', to reduce wastage.

Standard rectangular (200 × 100 mm) mesh sizes B283 and B385 could be useful if a greater area is required in one direction for the fire condition[12].

Small diameter bar reinforcement may be provided within the ribs when required for fire design or to resist the effects of concentrated loads.

| FABRIC REFERENCE	BAR SIZE (mm)	MINIMUM LAP LENGTH (mm) FOR CONCRETE CLASS		
		C25/30	LC28/35	C30/37 LC30/33
A142	6	195	180	175
B283	6 (long'l bars) 7 (trans bars)	195 230	180 210	175 200
A193, B385	7	230	210	200
A252	8	260	240	230
A393	10	360 (25 cover) 325 (30 cover)	335 (25 cover) 300 (30 cover)	320 (25 cover) 290 (30 cover)

Table 2.3
Minimum lap lengths for mesh

Note: Lapping bars to be in contact or no more than 4 diameters apart. Increase lap lengths by 100 mm (or half the bare spacing, if other than 200 mm) if this will not be controlled. For bars 8 mm and below, lap length is valid for cover not less than 3 diameters.
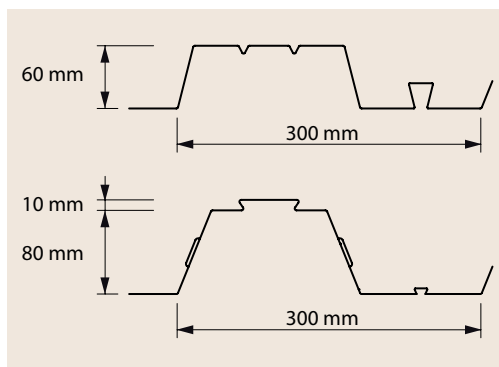
2.4　Profiled steel sheeting

Steel sheeting manufacturers in the UK produce sheets with either a re-entrant or a trapezoidal profile. Generally, profiles have been optimized for a 3 m span. Deeper trapezoidal profiles are produced for larger spans. Profiled sheeting is formed using zinc coated steel coil, which is available in grades S280 to S450. Grade S350 is most commonly used. Typical profiles are shown in Figure 2.2 and Figure 2.3.

Figure 2.2
Examples of re-entrant
sheeting profiles

Figure 2.3
Examples of trapezoidal
sheeting profiles
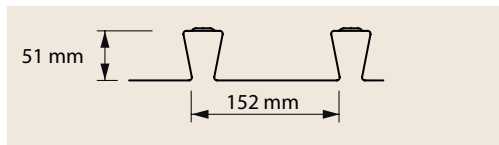
Some trapezoidal profiles include a single stiffener in the centre of each rib, whereas others have two stiffeners. The advantage of two stiffeners is that a stud can be placed centrally in the rib, whereas a single stiffener means that single studs have to be placed off-centre.

The height of the steel sheeting h_p is defined as the height to the shoulder of the profile, even if the profile has a re-entrant detail on the top flange. So for the 80 mm deep profile, shown in Figure 2.3, $h_p = 80$ mm. The overall height of the profile, h_d including the top dovetail only needs to be considered when calculating the depth of the concrete cover to the sheeting, h_c. For steel sheeting profiles that do not have re-entrant details of the top flange, such as the 60 mm profile shown in Figure 2.3, $h_d = h_p$.

4-1-1/NA.2.7 gives the minimum value of nominal bare metal thickness as 0.7 mm, which is in accordance with the recommended value in 4-1-1/3.5(2).

3-1-1/4.2(3) states that a zinc coating of 275 g/m^2 (total for both sides) is suitable for most non-aggressive interior conditions. For exterior or aggressive interior conditions, additional protection is required. Where protective paint is specified, the method and timing of its application needs to be considered, as painted steel sheeting is not suitable for the process of through-sheet welding of studs.

2.5　Shear connectors

The most common type of shear connection used in composite beams in buildings is the 19 mm diameter headed shear stud. This is the only diameter of shear stud that can be practically used for through-sheet welding.

BS EN ISO 13918: 1998[13] gives the specifications for welded studs.

The length of the stud after welding will depend on the initial stud length and the method of welding used. Eurocode 4 covers the use of shear studs welded through the

steel sheeting and shop welded shear studs. For design, it is the finished length of the stud after welding, h_{sc}, that is used to calculate stud resistance.

The commonly available heights of shear studs stocked in the UK are 105 mm, 130 mm, 155 mm and 180 mm, which result in nominal heights of 100 mm, 125 mm, 150 mm and 175 mm when welded directly to a steel section. For through-sheet welding, the height of the stud after welding, h_{sc} will generally be 5 mm less than the nominal height.

CONSTRUCTION STAGE

3.1   Design situation

For composite floors, the most onerous design situation during construction is during the concreting operation when the weight of the wet concrete, personnel and equipment have to be supported by the bare steel structure. For this design situation, the resistance of the bare steel beams is verified according to BS EN 1993-1-1 and the steel sheeting is verified in accordance with BS EN 1993-1-3.

It is possible to prop the steel sheeting and/or the beams during construction, but this is usually not preferred as it slows the construction process. Most steel sheeting profiles are optimised to ensure they achieve adequate spanning capability in the construction stage, without propping.

3.2   Actions

The actions to be considered during the construction stage are given in BS EN 1991-1-6[14]. Recommendations for design actions during concreting for beams and sheeting in composite floors are given below.

3.2.1   Actions on steel sheeting

Permanent actions during concreting

The self weight of the steel sheeting and the reinforcement are the permanent actions that need to be considered during concreting.

The self weight of the sheeting can be readily obtained from manufacturers' literature.

It is considered that the additional 1 kN/m^3 allowance for reinforcement given in 1-1-1/Table A.1 is appropriate for reinforced concrete but not for composite floors. It is recommended that the allowance for the light mesh reinforcement should be calculated on a case-by-case basis.

Variable actions during concreting

There are three types of variable action which need to be considered during concreting operations as described below.
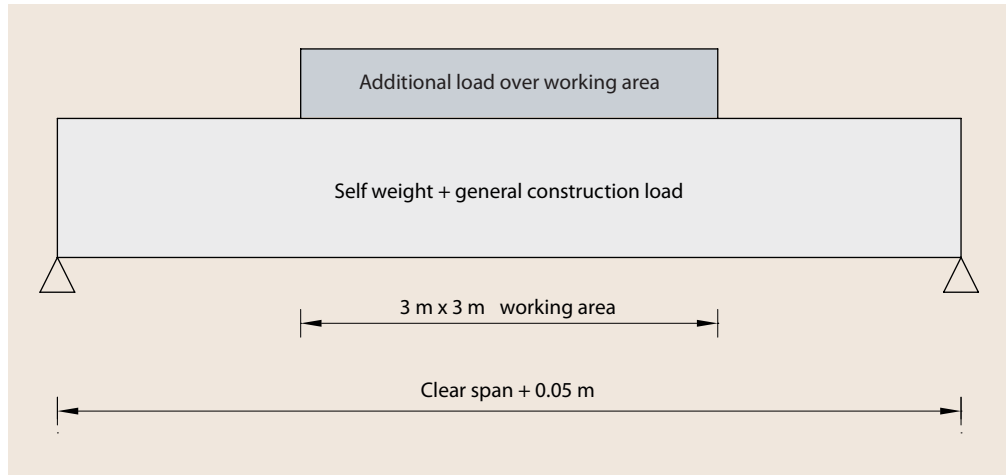
Figure 3.1
Construction loads on
steel sheeting

- The weight of wet concrete applied across the full area, including the additional load due to ponding where appropriate.
- A general construction loading allowance of 0.75 kN/m² acting over all the steel sheeting.
- An additional load of 10% of the slab self weight or 0.75 kN/m², whichever is greater, over a 3 m × 3 m 'working area'. This area should be treated as a moveable patch load that should be applied to cause maximum effect.

Wet and dry concrete densities

The wet and dry densities of unreinforced concrete are given in 1-1-1/Table A.1. The following values for concrete are recommended:

- 24 kN/m³ for dry normal weight concrete and 19 kN/m³ for dry lightweight aggregate concrete.
- 25 kN/m³ for wet normal weight concrete and 20 kN/m³ for wet lightweight aggregate concrete.

If the density of the lightweight aggregate is known then a more exact value of lightweight concrete density may be used. An additional 1.0 kN/m³ should be added to the dry density to allow for the density of free water in unset concrete.

Ponding

Consideration of the effects of ponding of wet concrete during execution is required by 4-1-1/9.3.2. If the deflection of the steel sheeting is greater the 1/10 of the slab depth, the effects of ponding should be considered; if the deflection is less, the effects of ponding may be ignored.

When an allowance for ponding is required, the depth of the concrete should be increased by $0.7w_1$ where w_1 is the maximum vertical deflection of the steel sheeting at the wet concrete stage. This additional weight should also be treated as a variable action.

It should be noted that where laser levelling techniques are employed, the slab depth will be greatly influenced by the deflection of the beams and sheeting. The method

to be used in construction should be considered in the design for the construction stage. Assumptions made regarding the construction methods should be stated in the specification for concreting operations. Further information on levelling techniques is given in AD344[15].

Combination of actions

Ultimate limit state

The expression for the combination of actions at the construction stage is determined from consideration of expressions 6.10, 6.10a and 6.10b given in BS EN 1990. The most onerous expression is 6.10a.

Recommended values for the combination factors for use when determining the combination of actions for the construction stage (see Section 1.4.1) are given in 1-1-6/A.1.1(1). The UK NA, 1-1-6/NA.2.18 adopts the recommended values ($\psi_0 = 1.0$), therefore 6.10a simplifies to the following expression:

$$1.35G_{k,1a,sup} + 1.5Q_{k,1a} + 1.5Q_{k,1b} + 1.5Q_{k,1c}$$

where

$Q_{k,1a}$ is the construction load for personnel and heaping of concrete in the 3 m × 3 m working area (at least 0.75 kN/m², as recommended above). (This construction loading covers the action defined in 1-1-6/4.11 as Q_{ca}, which is 'personnel and hand tools', and Q_{cf}, which is defined as 'loads from parts of a structure in a temporary state'.)

$Q_{k,1b}$ is the construction load across the full area (0.75 kN/m²). (This general load is also stated in 1-1-6/4.11 as covering Q_{ca}.)

$Q_{k,1c}$ is the weight of the wet concrete, applied across the full area, including additional concrete from ponding (where applicable). (This general load is stated in 1-1-6/4.11 as covering Q_{cc}, 'Non-permanent equipment' and Q_{cf}, 'Loads from part of a structure in a temporary state'.)

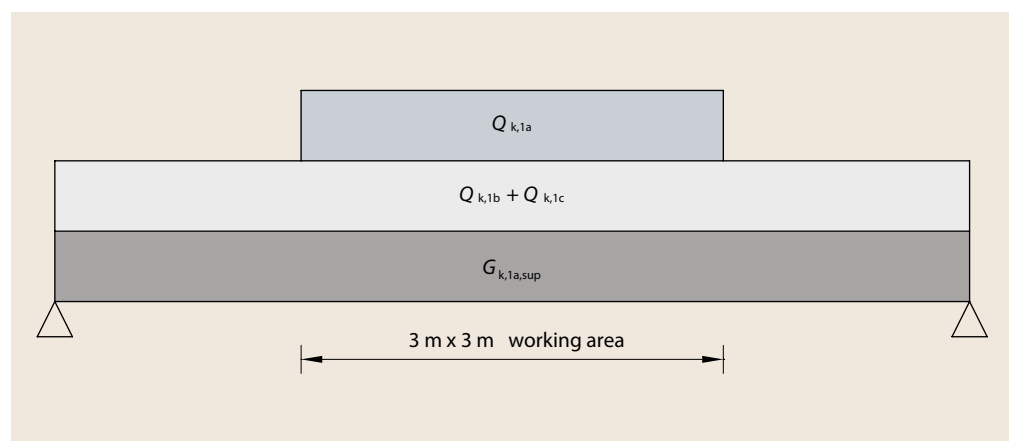$G_{k,1a,sup}$ is the self weight of the sheeting and reinforcement.

Figure 3.2
Actions at ultimate limit state

Serviceability limit state

For the serviceability limit state, 1-1-6/A1.2 specifies the use of the characteristic and quasi-permanent combinations of actions, expressions 6.14b and 6.16b of BS EN 1990.

It is recommended in AD346[16] that the characteristic combination of actions (6.14b) is used to verify both the deflection in the span and the deformation of the cross section of the steel sheeting for the serviceability limit state. When considering deflection, the general expression for the characteristic combination of actions is:

$$\Sigma G_{k,j} + Q_{k,1} + \Sigma \psi_0\, Q_{k,i} \qquad\qquad j \geq 1,\, i > 1$$

As the NA to 1-1-6 gives $\psi_0 = 1.0$, the above simplifies to:

$$\Sigma G_{k,j} + \Sigma Q_{k,i} \qquad\qquad j \geq 1,\, i \geq 1$$

Deformation of the cross section

There is no requirement to consider the deformation of the cross section of the profiled steel sheeting when it acts as formwork during the casting of concrete. However, the serviceability checks in 3-1-3/7.2 limit the deformation of the cross section by ensuring that when plastic global analysis is used at ULS, the combined effect of support moment and support reaction at the internal support does not exceed 0.9 times the combined resistance.

In this case, it is considered appropriate to include the construction load within the 3 m × 3 m working area together with the wet concrete and self weight loads. Thus using the characteristic values defined above, the combination of actions becomes:

$$G_{k,1a,sup} + Q_{k,1a} + Q_{k,1b} + Q_{k,1c}$$

It is possible that the verification of the resistance at the supports in accordance with 3-1-3/7.2 under this combination of action may govern for multi-span profiled sheeting.

3.2.2   Actions on steel beams

Construction loads during the casting of concrete

The construction loads on the beam would include the three components $Q_{k,1a}$, $Q_{k,1b}$ and $Q_{k,1c}$ in accordance with 1-1-6/4.11. As it is unlikely that the construction load for personnel of 0.75 kN/m² ($Q_{k,1b}$) will be present over the whole of the area supported by the beam during the casting of concrete, it is suggested that, with good site control, the load due to the 3 m × 3 m working area ($Q_{k,1a}$) could be neglected for the beam. The designer should make the contractor aware of the assumptions made and the importance of good site practice.

Ponding

Although Eurocode 4 does not call for an allowance for ponding to be included in the actions for steel beams, it is recommended that where an allowance for ponding is made for the sheeting, a similar allowance should be made for the beam.

As noted for the design of sheeting in Section 3.5.2, where laser 'mass flood' levelling techniques are employed, the slab thickness will be greatly influenced by the deflection of the beams. The slab thickness for the design of secondary beams is increased by up to 70% of the beam deflection, plus 70% of the deflection of the sheeting and up to 100% of the deflection of primary beams. For the design of primary beams, the increase is 70% of the combined deflections of the sheeting, primary and secondary beams.

However, if the levelling technique is known to be based on constant thickness rather than constant level, then it is considered that the effect of ponding is negligible.

Combination of actions

Ultimate limit state

Considering BS EN 1990, 6.10, 6.10a and 6.10b, the most onerous case is given by 6.10a. Based on this fundamental combination of actions and the partial factor values given in the UK national annexes, the combination of actions to be considered for the beam during the casting of concrete is:

$$1.35G_{k,1a,sup} + 1.35G_{k,1b,sup} + 1.5Q_{k,1b} + 1.5Q_{k,1c}$$

where

$G_{k,1b,sup}$ is the self weight of the beam section.

Serviceability limit state

The vertical deflection of the steel beams during the wet concrete stage should be considered. The use of the characteristic combination for determining the vertical deflection is specified in 3-1-1/N.A.2.23. Thus, where the beam needs to be verified against excessive vertical deflection, the following combination of actions is recommended:

$$G_{k,1a,sup} + G_{k,1b,sup} + Q_{k,1c}$$

3.3   Design resistance of sheeting

The requirements given in BS EN 1993-1-3[17] should be verified for the profiled steel sheeting when it acts as formwork during the casting of the concrete slab.

For structural efficiency, steel sheets are normally continuous over one or more supports, although in some cases the floor geometry makes a single span unavoidable.

Properties of steel sheeting are usually given by manufacturers. These properties are usually based on the results of structural tests carried out in accordance with 3-1-3/Annex A. Using reliability analysis in accordance with BS EN 1990, characteristic and design values of moment resistance, crushing resistance and second moment of area may be determined. Properties based on testing are generally less conservative than equivalent values determined by calculation.

3.3.1 Bending resistance

The design moment resistance of shallow profiles in steel sheeting is established in 3-1-3 using an effective width model to account of the thin steel elements in compression. This approach is relatively conservative, due to the behaviour of the sheeting being very complex as a result of local buckling.

As an alternative to the calculation of design resistance, the performance of the steel sheeting may be determined from tests carried out and assessed in accordance with 3-1-3/Annex A. As the calculation model for design resistance tends to be conservative, manufacturers prefer to determine design resistances from testing. These test results form the basis of the design tables and software provided by the manufacturers to assist and support the designer.

Bending resistance is usually expressed as a value per metre width of the steel sheeting.

3.3.2 Shear resistance of the cross section

The shear resistance of the sheeting is calculated according to 3-1-3/6.1.5. The design shear resistance of a single web, $V_{b,Rd}$, is given by 3-1-3/(6.8) as follows:

$$V_{b,Rd} = \frac{\dfrac{h_w}{\sin\phi}tf_{bv}}{\gamma_{M0}}$$
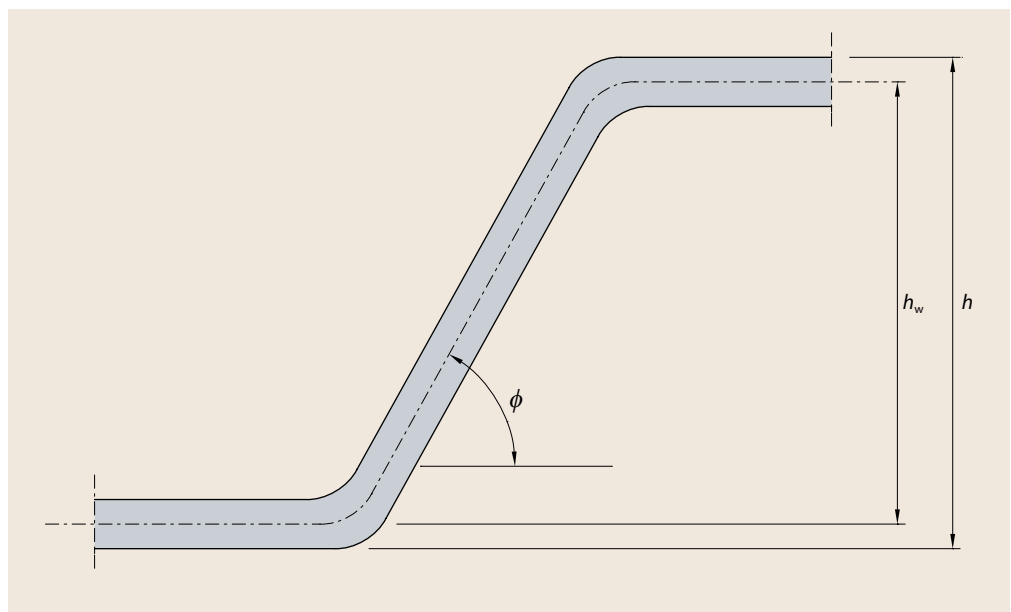

*Figure 3.3
Dimensions of steel
sheeting profile*

where

f_{bv} is the shear strength considering buckling according to 3-1-3/Table 6.1.

h_w is the web height between the midlines of the flanges (See Figure 3.3)

ϕ is the slope of the web relative to the flanges (See Figure 3.3)

γ_{M0} is the partial factor for steel sheeting, taken as 1.0, in accordance with 3-1-3/2(3) and the UK NA.

The shear resistance is normally expressed as a resistance per metre width.

Combined bending and shear

3-1-3/6.1.10 provides a criterion for interaction of shear force, axial force and bending moment, which is taken into account in verification of cross sections subject to high shear. When $V_{Ed} \leq 0.5 V_{w,Rd}$ the modification for shear may be omitted from the expression shown below. Otherwise the criterion for the combined shear force and bending moment is calculated according to 3-1-3/(6.27) as follows:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(\frac{2V_{Ed}}{V_{w,Rd}} - 1\right)^2 \leq 1.0$$

where

N_{Rd} is the design resistance of the cross section to axial force, either in tension or in compression

$M_{y,Rd}$ is the design moment resistance of the cross section

$V_{w,Rd}$ is the design shear resistance of the web (given in 3-1-3/(6.8) and above as $V_{b,Rd}$)

$M_{f,Rd}$ is the moment resistance of a cross section consisting of the effective area of the flanges only

$M_{pl,Rd}$ is the plastic moment resistance of the cross section.

In most designs for steel sheeting there is no axial force and the first term of 6.27) can be ignored leaving an expression for combined bending and shear.

Note: *It is assumed that the resistance values will be translated into a design resistance per unit width before combining the resistances.*

3.3.3   Local resistance

The resistance of the sheeting web to transverse forces during the casting of concrete should be verified against the rules in 3-1-3/6.1.7. The resistance of the web of the profile is affected by the size of the support. Support areas should be verified against manufacturers' recommendations.

The criterion for the crushing, crippling and buckling resistance is given in 3-1-3/6.1.7.3. The local transverse resistance of a single web $R_{w,Ed}$, is determined using expression 3-1-3/(2.18), as follows.

$$R_{w,Rd} = \alpha t^2 \sqrt{f_{yb} E} \left(1 - 0.1\sqrt{r/t}\right)\left(0.5 + \sqrt{0.02 l_a/t}\right)\left(2.4 + (\phi/90)^2\right)/\gamma_{M1}$$

where

t is the design core thickness of the steel sheeting excluding coatings

f_{yb} is the basic yield strength (0.2% proof stress)

r is the internal radius at the corners

ϕ is the angle of the web relative to the flanges (degrees)

l_a is the effective bearing length for the relevant category

α is a coefficient to reflect the loading configuration

γ_{M1} is the partial factor for buckling resistance of steel sheeting (= 1.0).

For Category 1 (distance of local load or reaction $< 1.5h_w$ from a free end), $\alpha = 0.075$ and $l_a = 10$ mm.

For Category 2 (other situations), $\alpha = 0.15$ and the bearing length depends of the ratio of shear forces β_V.

for $\beta_V \leq 0.2$: $l_a = s_s$

for $\beta_V \geq 0.3$: $l_a = 10$ mm

for $0.2 < \beta_V < 0.3$: Interpolate between values for l_a for $\beta_V = 0.2$ and 0.3

where

$$\beta_V = \frac{|V_{Ed,1}| - |V_{Ed,2}|}{|V_{Ed,1}| + |V_{Ed,2}|}$$

In which $|V_{Ed,1}|$ and $|V_{Ed,2}|$ are the absolute values of the transverse shear forces on each side of the local load or support reaction, $|V_{Ed,1}| \geq |V_{Ed,2}|$ and s_s is the length of stiff bearing.

Where test results are available for a steel sheeting product, expression 6.18 can be simplified to:

$$R_{w,Rd} = k \times coeff \times \left(0.5 + \sqrt{0.02 l_a/t}\right)/\gamma_{M1}$$

The value of *coeff* is determined from test results. At an internal support, $k = 1.0$ or for a simply supported profile, k is equal to 1.0 for $c > 1.5h_w$ or k is equal to 0.5 for $c \leq 1.5h_w$.

Combined web crushing and bending moment

The criterion in 3-1-3/6.1.11 for cross-sections subject to the combined effects of a bending moment M_{Ed} and a support reaction F_{Ed} is:

$$\left(\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}}\right) \leq 1.25$$

where

$M_{c,Rd}$ is the moment resistance of the cross section given in 4-1-1/6.1.4.1(1) or determined from tests

$R_{w,Rd}$ is the appropriate value of the local transverse resistance of the web determined from 4-1-1/6.1.7 or from tests.

Each term in the above expression must not exceed 1.0.

3.4 Design resistance of steel beams

Eurocode 3 gives rules for the verification of the resistance of steel sections and members. The rules are presented in terms of cross sectional resistance and member buckling resistance.

Where the steel sheeting spans perpendicularly to the beam and is attached to its top flange, the beam may be considered as restrained along its length. For this case, only the cross sectional requirements of 3-1-1/6.2 need to be verified.

Where the steel sheeting spans parallel to the beam, it is considered that the beam will not be restrained along its length. The beam will only be restrained at its ends and at beam-to-beam connections. Therefore, the buckling resistance will need to be verified based on the length between points of restraint in addition to verifying the resistance of the cross section.

3.4.1 Buckling resistance

The buckling resistance of a steel beam depends on the resistance of the cross section and a reduction factor χ_{LT} for lateral torsional buckling. The factor χ_{LT} depends on the non-dimensional slenderness $\bar{\lambda}_{LT}$, which in turn depends on the elastic critical moment for lateral torsional buckling, M_{cr}. Expressions for M_{cr} can be found in P362[18] or SN002[19].

These publications also provide conservative methods of directly calculating the non-dimensional slenderness. For hot rolled doubly symmetric I and H sections with lateral restraints to the compression flange at the ends of the segment being considered, the expression below gives a conservative approach for determining the non-dimensional slenderness, $\bar{\lambda}_{LT}$.

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UV \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w} = \frac{1}{\sqrt{C_1}} UV \bar{\lambda}_z \sqrt{\beta_w}$$

where

C_1 is a parameter dependent on the shape of the bending moment diagram and, conservatively, may be taken as 1.0

U is a parameter dependent on the section geometry and, conservatively, may be taken as 0.9

V is a parameter related to the slenderness and, provided the loading is not destabilising, may be conservatively taken as 1.0 for sections that are symmetric about the major axis

$$\lambda_z \qquad = \frac{k\,L}{i_z}$$

L is the distance between points of restraint to the compression flange

k is the effective length parameter and should be taken as 1.0

$$\lambda_1 \qquad = \pi \sqrt{\frac{E}{f_y}} = 93.9\varepsilon$$

$$\beta_w \qquad = \frac{W_y}{W_{pl,y}}$$

W_y is the modulus used to calculate $M_{b,Rd}$.

For Class 1 and 2 sections $W_y = W_{pl,y}$

For Class 3 sections $W_y = W_{el,y}$

Alternatively, the software *LTBeam*[20] may be used to determine M_{cr}.

Design values of cross sectional and buckling resistances for hot rolled UKB and UKC sections can be found in P363[21].

3.4.2   Torsional effects

For beams of normal flange width in regular composite construction, torsion effects due to temporary imbalance of construction loads during casting of concrete are negligible.

Simple joints, as described in P358[2], provide a degree of nominal torsion resistance, sufficient to cater for the modest effects at the construction stage.

3.5   Deflection limits

At the construction stage, there is an important link between the levelling techniques used in construction, the deflection of the bare steel structure and the consequent volume and weight of the concrete.

3.5.1   Steel beams

Where laser levelling techniques are employed, the flexibility and deflection of the beams can lead to a significant increase in the weight and volume of concrete that needs to be supported. The designer needs to be aware of this as an issue and it is recommended that the vertical deflections of each steel beam should be no more than 25 mm[22] if flood pouring is to be used.

3.5.2   Profiled steel sheeting

It may be necessary to limit the deflection of the steel sheeting profile to limit the effects of ponding of concrete during construction and the increase in dead weight as a result. According to 4-1-1/9.3.2(2), if the central deflection of the sheeting is greater than a tenth of the slab thickness, ponding should be considered. In this situation,

the nominal thickness of concrete over the complete span may be assumed to be increased by 0.7 times the calculated mid span deflection.

The UK National Annex 4/NA2.15 recommends the following limits, taken from BS 5950-4, for use in Eurocode design.

Where additional loads due to ponding of wet concrete are ignored when determining the loading, a more onerous deflection limit is used to check the steel sheeting at the SLS. In these cases, the mid span deflection should be limited to the lesser of:

$$\frac{\text{effective span}}{180} \text{ and } 20 \text{ mm}$$

Where additional loads due to ponding of wet concrete are included, the mid span deflection should be limited to the lesser of:

$$\frac{\text{effective span}}{130} \text{ and } 30 \text{ mm}$$

COMPOSITE BEAMS AT NORMAL STAGE

The guidance in this Section relates to solid web composite beams formed with a composite slab on top of a rolled section UKB or UKC or a fabricated plate girder. For guidance on the design of beams with large web openings, refer to P355[7] and for guidance on the design of beams with precast concrete floor slabs refer to P351[23].

4.1 Actions

The actions to be considered in the verification of the beams are:

Permanent actions:

- self weight of steel section
- self weight of composite slab, based on the dry density of concrete, and an allowance for sheeting and reinforcement, plus self weight due to ponding of concrete during construction (if appropriate).
- finishes
- services

Variable actions:

- allowance for occupancy loads depending on building usage
- allowance for movable partitions

Thermal, wind and accidental actions do not normally need to be considered.

4.1.1 Combination of actions

Expression 6.10b will usually result in the more onerous combination for the normal stage than expression 6.10a of BS EN 1990 (see comment in Section 1.4).

4.1.2 Design effects

In accordance with 4-1-1/5.4.1, the effects of actions may be determined using elastic global analysis, even when the resistance of the cross section is based on its plastic or non-linear resistance. Elastic global analysis may also be used for the serviceability limit state, making suitable allowance for concrete cracking, creep and shrinkage. At ULS, the effects of creep and shrinkage can be ignored when the composite cross section is Class 1 or 2, and when no allowance for lateral torsional buckling is required. For SLS, the effects of shrinkage can be ignored if the span to depth ratio is less than or equal to 20.

4.2 Bending resistance

In the absence of pre-stressing due to tendons, 4-1-1/6.2.1.1(1)P allows rigid plastic theory to be used when determining the bending resistance for a Class 1 or 2 composite section. Elastic analysis and non linear theory is permitted for all classes of composite beam.

When considering the bending resistance of the composite section, the tensile resistance of the concrete is neglected (4-1-1/6.2.1.1(4)P). For buildings, the profiled steel sheeting must be ignored when it is in compression (4-1-1/6.2.1.2(4)P).

With the concrete in compression and the steel beam in tension, the composite cross section is Class 1. The flange class of all UKB sections and all but the lightest UKC sections is Class 1 so that, where relative size of the steel beam is such that the plastic neutral axis lies below the top flange, the composite beam will still be Class 1. The bending resistance of the composite beam is therefore normally taken as its plastic bending resistance; elastic bending resistance is not considered.

Development of the full plastic resistance moment $M_{pl,Rd}$ requires sufficient shear connection between the slab and the beam. Where sufficient connection exists, it is referred to as full shear connection. The requirements for shear connectors are discussed in Section 4.4.

4.2.1 Effective section

The first step when defining a composite cross section is to assess the width of flange available to act compositely with the steel section. Although the actual effective width varies along the length of the beam, for design it is convenient to consider a constant effective width of slab in the sagging region.

The effective widths given in 4-1-1/5.4.1.2 are expressed in relation to the span of the beam and different values apply at different points along the beam. When elastic global analysis is used a constant effective width may be assumed over the whole of each span, as permitted by 4-1-1/5.4.1.2(4) and 6.1.2(2). This constant value of effective width is taken as $L_e/4$.
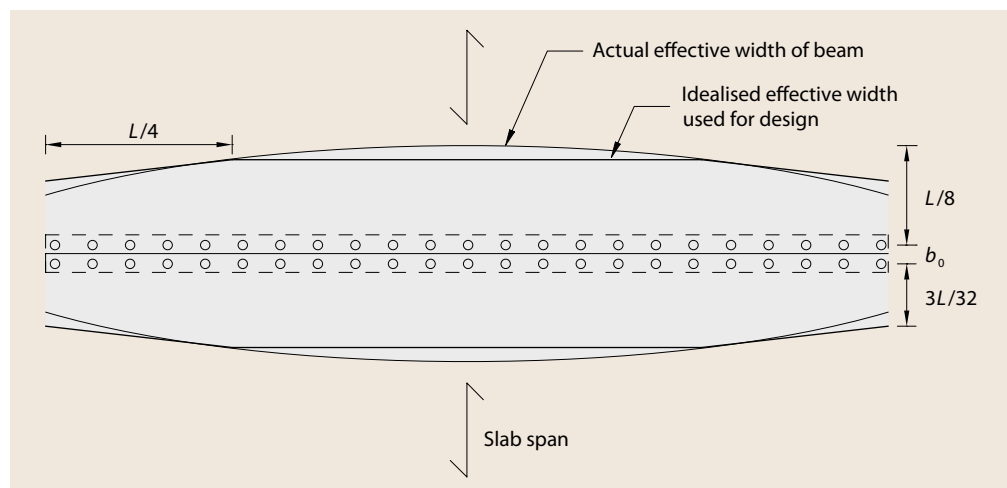

*Figure 4.1
Variation of
effective width of
concrete flange*

For verification of the cross section, the distribution of effective width between the supports and mid span regions may be taken into account in accordance with 4-1-1/6.1.2(1). From 4-1-1/5.4.1.2(5) the effective width at mid span may be taken as:

$$b_{\text{eff}} = b_0 + \sum b_{ei}$$

where

b_0 is the distance between the centres of the rows of shear connectors, (when $n_r = 2$)

b_{ei} is the effective width at midspan of the concrete flange on each side of the steel section, taken as $L_e/8$

L_e is the effective length of the span, which is taken equal to the system length for a simply supported beam.

The effective width can be assumed to reduce over the last quarter of the span to the support. The effective width at the support is given by 4-1-1/5.4.1.2(6).

$$b_{\text{eff}} = b_0 + \sum \beta_i \, b_{ei}$$

where

β_i $= (0.55 + 0.025 L_e/b_{ei}) \leq 1.0$

Holes in the slab for service penetrations should ideally occur outside the effective width of the composite beam. If this is not possible, then the width of concrete flange of the composite beam must be modified to take account of service penetrations. Further guidance can be found in P300[22].

4.2.2 Plastic resistance with full shear connection

Typical plastic stress distributions for composite beams with full shear connection are shown in Figure 4.2. Concrete in compression may be assumed to resist a stress equal to $0.85 f_{cd}$ over the full depth from the plastic neutral axis to the most compressed fibre, in accordance with 4-1-1/6.2.1.2. The contribution of the steel sheeting in compression should be ignored. Generally, the compressive and tensile resistance of reinforcement and steel sheeting to the bending resistance is small and is commonly neglected.

The amount of concrete available to resist the compressive force due to bending is limited by the effective width (b_{eff}) of the concrete flange (see Section 4.2.1) and the depth of concrete cover to the steel sheeting. For secondary beams, where the sheeting is transverse to the beam, the depth of concrete is taken as that between the top surface and the top of the sheeting profile (to the top of the dovetail stiffener, if there is one); for primary beams, with the profile parallel to the beam, it is common to ignore the concrete in the ribs and simply to take the depth to the shoulder of the profile.

For a typical secondary beam, more concrete compression resistance is available than can be employed and the plastic neutral axis lies above the sheeting profile, as shown in Figure 4.2a).
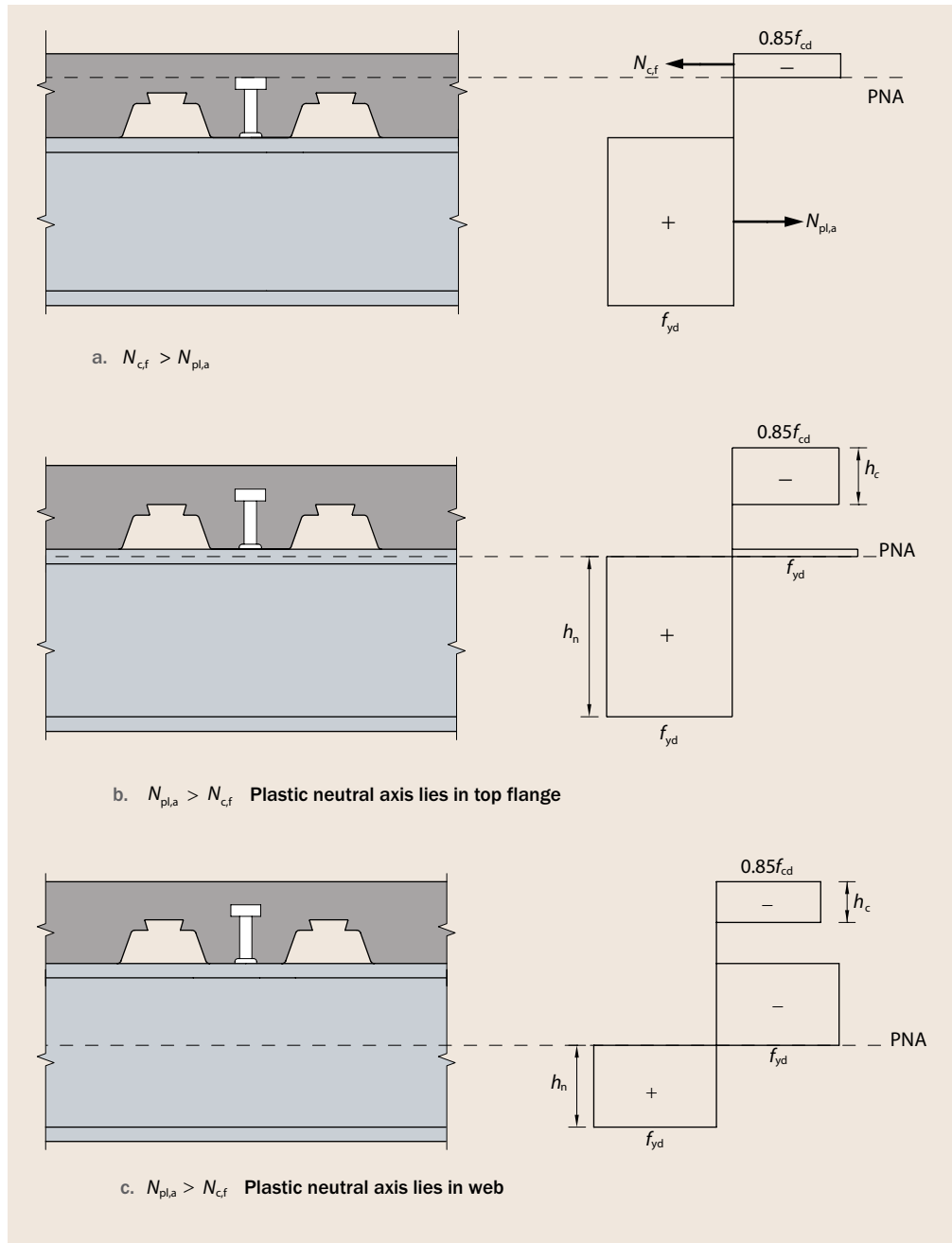
*Figure 4.2
Plastic stress
distributions for full
shear connection*

a. $N_{c,f} > N_{pl,a}$

b. $N_{pl,a} > N_{c,f}$ **Plastic neutral axis lies in top flange**

c. $N_{pl,a} > N_{c,f}$ **Plastic neutral axis lies in web**

For a typical primary beam, it may be found that the steel section offers more tension resistance than the concrete flange can match in compression resistance. The plastic neutral axis is then in either the top flange of the steel section (Figure 4.2b)), or, occasionally, in the web (Figure 4.2c)).

Plastic neutral axis within the concrete slab

When the plastic neutral axis (PNA) lies within the concrete slab, the bending resistance of the composite cross section may be determined from:

$$M_{pl,Rd} = N_{pl,a}\left(\frac{h_a}{2} + h_s - \left[\frac{N_{pl,a}}{N_{c,f}} \frac{h_c}{2} \right] \right)$$

where

$N_{pl,a}$ is the axial resistance of the steel section

$N_{c,f}$ is the resistance of the effective area of the concrete flange acting compositely with the steel section $= (0.85 f_{cd} b_{eff} x_c)$

h_a is the depth of the steel section (see Figure 4.3)

h_s is the depth of the composite slab

h_c is the depth of the concrete above the steel sheeting profile $(= h_s - h_d)$. (see Figure 4.3)

h_p is the depth of the steel sheeting profile measured to the shoulder of the profile (see Figure 4.3)

h_d is the overall depth of the steel sheeting profile, including the height of the top dovetail stiffener, if present. If the deck has no top stiffener $h_d = h_p$. (see Section 2.4).
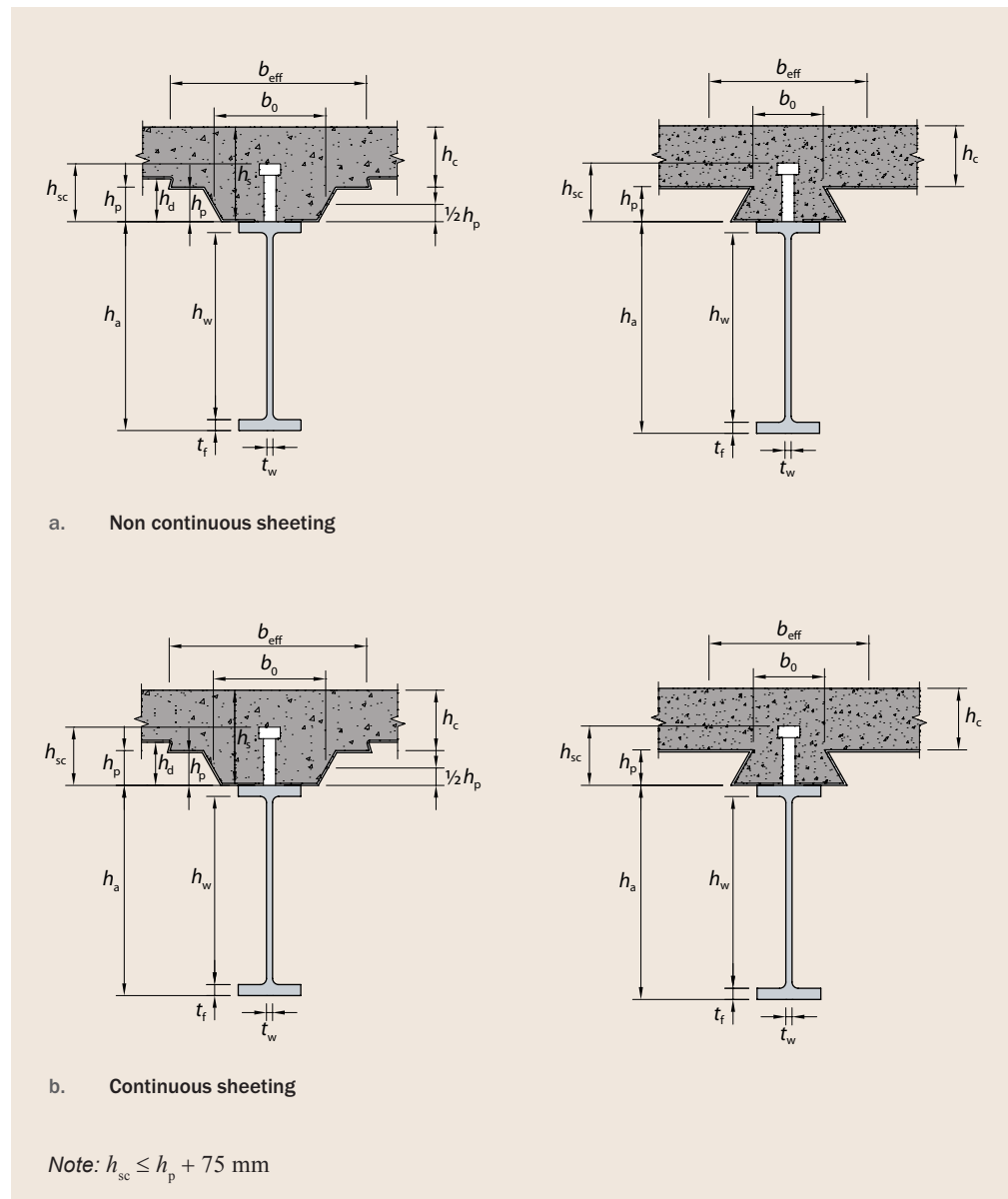

a.   Non continuous sheeting

b.   Continuous sheeting

Figure 4.3
Composite beam and
steel sheeting
dimensions

Note: $h_{sc} \leq h_p + 75$ mm

Plastic neutral axis within the top flange

Neglecting the contribution from the part of the top flange that is in compression, the plastic bending resistance of the composite beam may conservatively be determined from:

$$M_{pl,Rd} = N_{pl,a}\frac{h_a}{2} + N_{c,f}\left(\frac{h_c}{2} + h_d\right)$$

Plastic neutral axis within the web

The plastic bending resistance of the composite beam may be determined from:

$$M_{pl,Rd} = M_{pl,a,Rd} + N_{c,f}\left(\frac{h_c + 2h_d + h_a}{2}\right) - \left(\frac{N_{c,f}^{\;2}}{N_w}\frac{h_a}{4}\right)$$

where

$M_{pl,a,Rd}$ is the design bending resistance of the steel section ($W_{pl}f_{yd}/\gamma_{M0}$)

N_w $= 0.95f_{yd}t_w h_w$

h_w $= h_a - 2t_f$

4.2.3 Plastic bending resistance with partial shear connection

When the full compression resistance of the concrete flange ($N_{c,f}$) is not required for the bending resistance of the composite beam, the shear connectors are not required to transfer a force equal to $N_{c,f}$. For this situation, the composite beam may be designed with partial shear connection.

To develop the resistance moment with partial shear connection, the shear connectors must be sufficiently ductile (see Section 4.4.3).

A plastic stress distribution for a beam with partial shear connection is given in Figure 4.4.

The simplest method of determining the moment resistance of a composite section is the 'linear-interaction' approach, covered by 4-1-1/6.2.1.3. The bending resistance for partial interaction is given by:

$$M_{Rd} = M_{pl,a,Rd} + \left(M_{pl,Rd} - M_{pl,a,Rd}\right)\frac{N_c}{N_{c,f}}$$
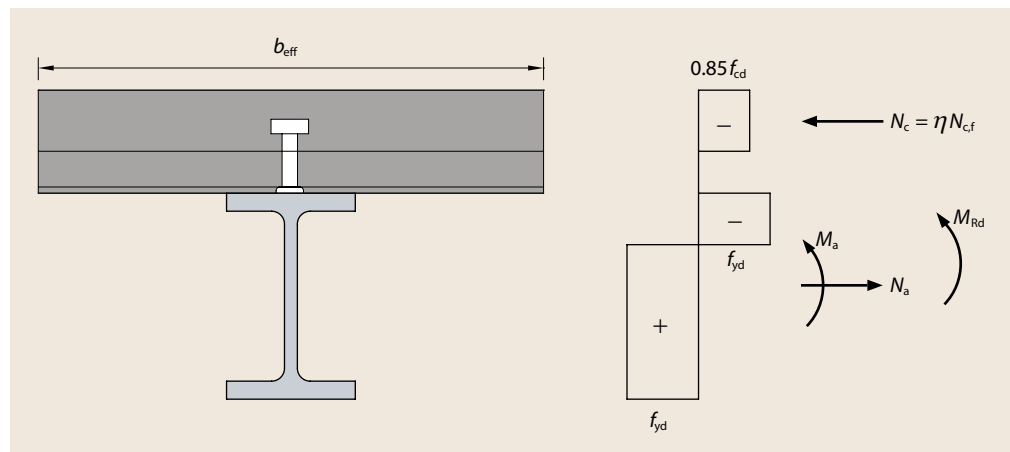
Figure 4.4
Plastic stress distribution for partial shear connection
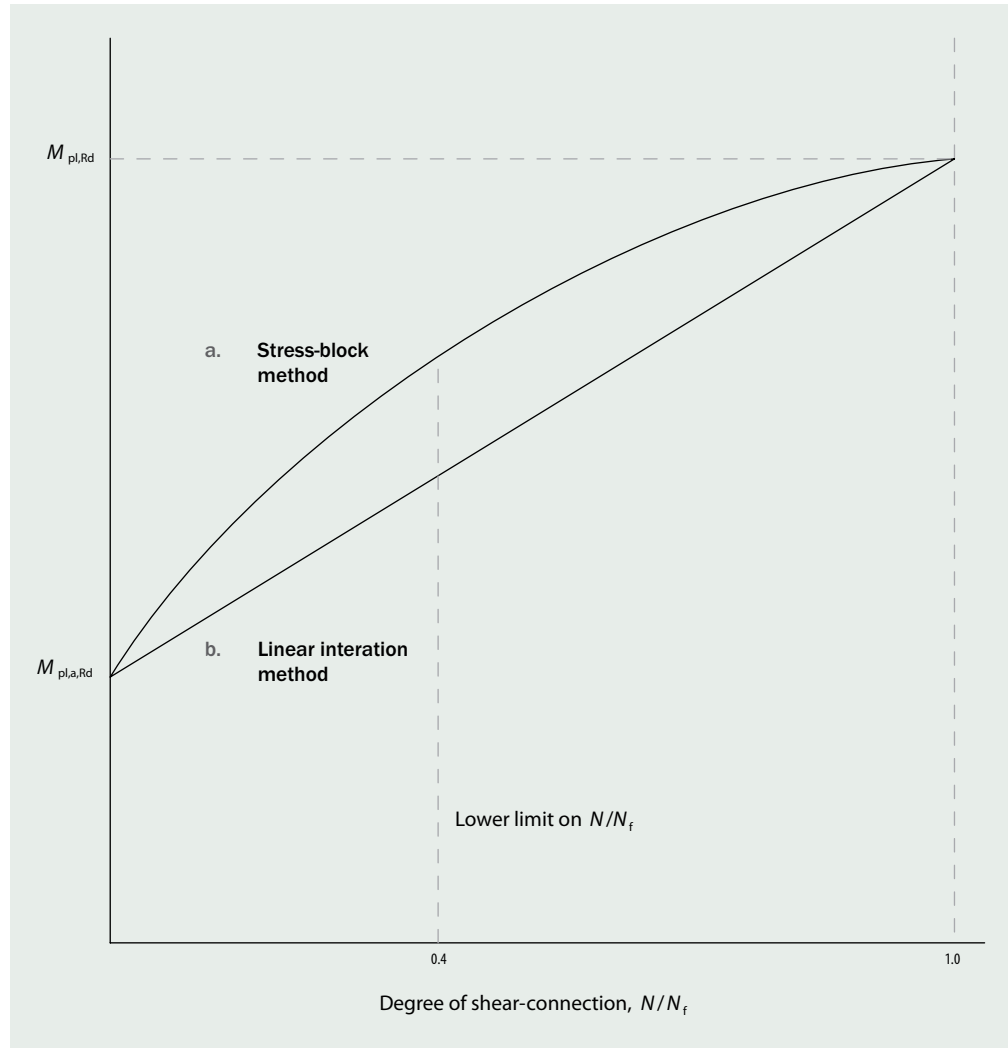
where

$M_{\mathrm{pl,Rd}}$ is the moment resistance of the composite section with full shear connection

$M_{\mathrm{pl,a,Rd}}$ is the plastic moment resistance of the steel section.

The linear interaction method is conservative with respect to the stress block method described above, as illustrated in Figure 4.5.

The stress block method is presented in 4-1-1/6.2.1.3. It is a more complex method in that the equilibrium of the section is achieved by equating the compression force in the concrete slab to the longitudinal shear force transferred by the shear connectors. The moment resistance calculated using the stress-block method is less conservative than that calculated using the linear interaction method.

4.3   Vertical shear resistance

The resistance of the composite beam to vertical shear is normally taken as the shear resistance of the steel section; any contribution from the slab is ignored, unless a value for this contribution has been established experimentally.

The proportions of UKB and UKC sections are such that shear buckling does not need to be considered, so the plastic shear resistance may be used in design. The design plastic shear resistance, $V_{pl,Rd}$, may be determined from 3-1-1/6.2.6, which gives the following expression.

$$V_{pl,Rd} = \frac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$$

where

A_v is the shear area.

For a rolled section, the shear area is given as:

$$A_v = A - 2bt_f + (t_w + 2r)t_f \qquad \text{but not less than } \eta h_w t_w$$

where

A is the cross sectional area of the rolled section

b is the width of the flange

t_f is the flange thickness

h_w is the depth of the web

t_w is the web thickness

r is the root radius

η given by 3-1-5/NA.2.4 as 1.0.

For a fabricated section, the shear area is given as:

$$A_v = \eta(h_w t_w)$$

For fabricated sections, it may also be necessary to determine the shear buckling resistance $V_{b,Rd}$, in accordance with 3-1-5/5, if the web is slender.

4.3.1 Combined bending with vertical shear

Where the vertical shear force is greater than half the vertical shear resistance, an allowance for its effect on the resistance moment is required. For cross-sections in Class 1 or 2, the influence of vertical shear on the bending resistance is taken into account by a reduced steel design strength $(1 - \rho)f_{yd}$ for the shear area. The parameter ρ is calculated as follows:

$$\rho = (2V_{Ed}/V_{Rd} - 1)^2$$

For Class 3 and 4 cross sections, 3-1-5/7.1 is applicable.

4.4 Shear connection

4.4.1 General

The design rules for determining the resistance of headed studs used as shear connectors with profiled steel sheeting are given in 4-1-1/6.6.4. Rules for sheeting spanning parallel to the supporting beam are given in 6.6.4.1, while 6.6.4.2 covers sheeting transverse to the supporting beam. The resistance of a headed stud within profiled sheeting is determined by multiplying the design resistance for a headed stud connector in a solid concrete slab (P_{Rd}) by a reduction factor (k_ℓ for parallel sheeting and k_t for transverse sheeting).

Shear connectors should be spaced along the beam in accordance with an appropriate longitudinal shear force distribution. Consideration must also be given to the need to prevent separation between the steel and the concrete. To prevent uplift, the shear connector should have a tensile resistance equal to at least a tenth of its shear resistance; headed shear studs can satisfy this requirement. When ductile shear connectors are used, the studs may be spaced uniformly along the beam, which simplifies design and construction.

4.4.2 Design resistance of a headed stud connector

Solid concrete slabs

The expressions presented in 4-1-1/6.6.3.1 are used to determine the resistance of a headed stud connector in a solid slab. The resistance is taken as the lesser of the values determined from expressions 4-1-1/(6.18) and (6.19). Those expressions use a partial factor of γ_V, for which a value of 1.25 is adopted by 4-1-1/NA.2.3.

The design resistance of a headed stud shear connector in a solid slab is the smaller of:

$$P_{Rd} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V} = 0.16 f_u \pi d^2 \qquad \text{and}$$

$$P_{Rd} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} = 0.232 \alpha d^2 \sqrt{f_{ck} E_{cm}}$$

where

f_u is the ultimate tensile strength of the headed stud, but not more than 500 N/mm² for sheeting spanning parallel to the supporting beam and not more than 450 N/mm² for sheeting spanning transversely to the supporting beam (for studs type SD1 to BS EN ISO 13918 f_u = 450 N/mm²).

d is the diameter of the shank of the headed stud (16 mm $\leq d \leq$ 25 mm).

f_{ck} is the characteristic cylinder strength of the concrete of density not less than 1750 kg/m³ (given in 2-1-1/Table 3.1).

E_{cm} is the secant elastic modulus of concrete (given in 2-1-1/Table 3.1).

$$\alpha \qquad = 0.2\left(\frac{h_{sc}}{d}+1\right) \qquad \text{for } 3 \le \frac{h_{sc}}{d} \le 4$$

$$\alpha \qquad = 1.0 \qquad\qquad \text{for } 4 < \frac{h_{sc}}{d}$$

h_{sc} is the as welded height of the headed stud (see Figure 4.3, Figure 4.7 and Figure 4.6).

Profiled steel sheeting spanning parallel to the supporting beam

Based on 4-1-1/6.6.4.1(2), the design shear resistance of a single headed stud connector in a rib that is parallel to the supporting beam in sheeting that is continuous across the beam is determined from:

$$k_\ell P_{Rd}$$

where

P_{Rd} is the design resistance of a headed stud connector in a solid slab

$$k_\ell \qquad = 0.6\frac{b_o}{h_p}\left(\frac{h_{sc}}{h_p}-1\right) \qquad \text{but, } k_\ell \le 1.0$$

b_o is the width of a trapezoidal rib at mid height of the profile (h_p), see Figure 4.6 or the minimum width of the rib for re-entrant profiles, see Figure 4.7

h_p is the height of the steel sheeting measured to the shoulder of the profile

h_{sc} is the as welded height of the stud, but not greater than $h_p + 75$ mm.

Where the sheeting is discontinuous across the supporting beam, the method for continuous sheeting may be used provided that the steel sheeting is anchored appropriately to the supporting beam. The purpose of appropriate anchorage is to ensure that the rib formed by the steel sheeting has adequate tensile reinforcement to prevent the rib being split by the forces from the shear studs. Eurocode 4 does not define what constitutes appropriate anchorage but it seems reasonable to assume that fixings with a shear resistance equivalent to the force required to unfold the profile if it were subject to transverse tension would provide suitable resistance. Typically, appropriate anchorage to discontinuous steel sheeting may be provided by fixing the edges of the steel sheeting to the beam with self-drilling self-tapping screws (Tek 4.8 × 20 mm or equivalent) or shot fired pins (Hilti ENP2 or equivalent) at 250 mm centres, see NCCI PN003a-GB[24].

Profiled steel sheeting spanning transverse to the supporting beam

Where the following criteria are met, the reduction factor (k_t) given in 4-1-1/6.6.4.2(1) may be applied to P_{Rd} to determine the resistance of headed stud connectors in a rib of profiled steel sheeting spanning transversely to the supporting beam.

- The studs are placed in ribs with $h_p \le 85$ mm and $b_o \ge h_p$.
- For shear studs welded through the steel sheeting, $d \le 20$ mm.
- For holes provided in the sheeting, $d \le 22$ mm.

$$k_t = \frac{0.7}{\sqrt{n_r}} \frac{b_o}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right), \qquad \text{but } k_t \le k_{max}$$

where

n_r is the number of stud connectors in one rib at a beam intersection. This must not be greater than 2.

h_p, b_o and h_{sc} are defined above.

Values for k_{max} are given in 4-1-1/Table 6.2 and reproduced here in Table 4.1.

NUMBER OF STUD CONNECTORS PER RIB	THICKNESS OF SHEETING (mm)	k_{max}	
		STUDS WELDED THROUGH SHEETING, $d \le 20$ mm	SHEETING WITH HOLES, $d = 19$ mm or 22 mm
1	≤ 1.0	0.85	0.75
1	> 1.0	1.0	0.75
2	≤ 1.0	0.7	0.6
2	> 1.0	0.8	0.6

Table 4.1 Values of k_{max}

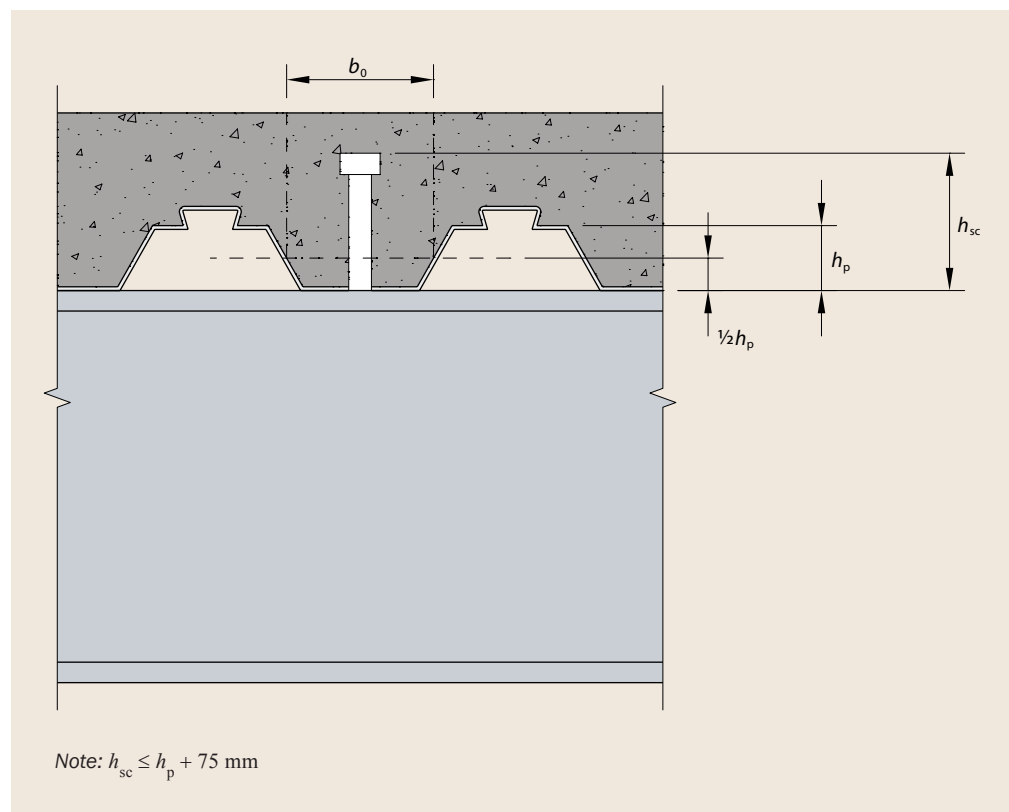*Figure 4.6 Dimensions for headed stud connectors in trapezoidal steel sheeting spanning transverse to the supporting beam*
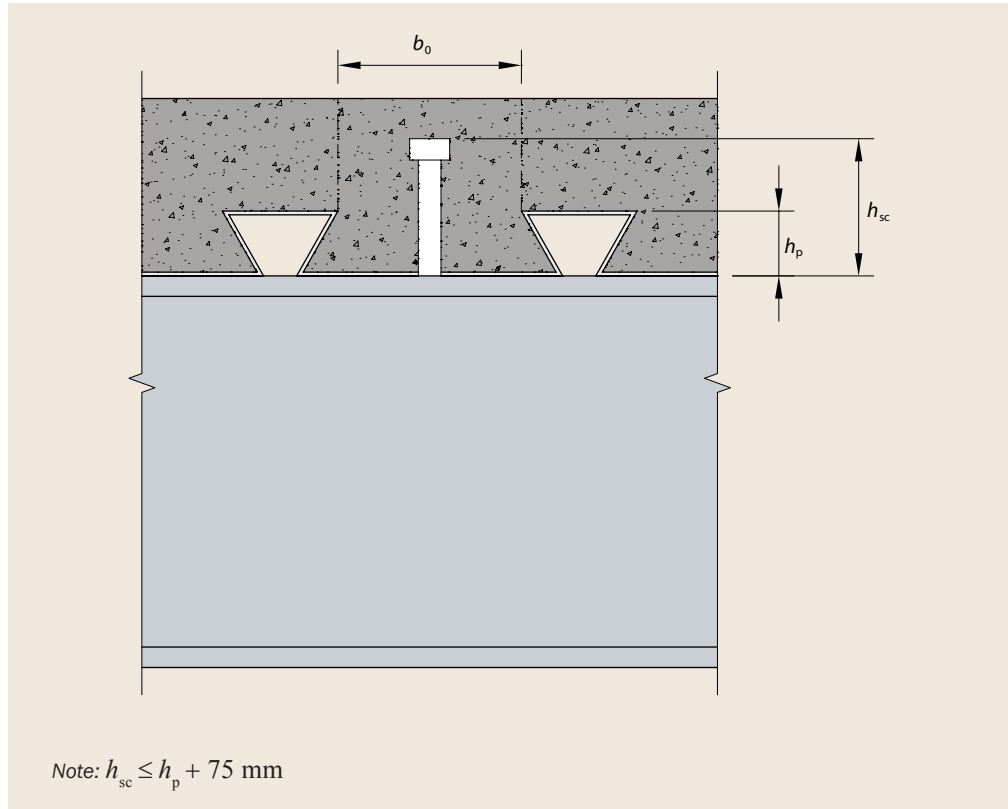
Note: $h_{sc} \le h_p + 75$ mm

*Figure 4.7
Dimensions for
headed stud
connectors in
re-entrant steel
sheeting spanning
transverse to the
supporting beam*

Note: $h_{sc} \leq h_p + 75$ mm

Steel sheeting with a trapezoidal profile

The test results which form the basis of the design methods given in Eurocode 4 have been available for some time. However, the steel sheeting profiles in use when the experimental work was carried out differ significant from the steel sheeting profiles currently used in composite construction in the UK. Therefore, between 2006 and 2008, further experimental work was conducted on more modern steel sheeting profiles. The experimental programme included tests on full scale composite beam specimens in additional to companion push tests on small scale samples. The work has resulted in modified shear stud resistances.

As reported in NCCI PN001a-GB[25], the resistance for shear studs in transverse sheeting may be based on the resistance in a solid slab modified by the reduction factor (k_t) given in 4-1-1/6.6.4.2 and an additional reduction factor (k_{mod}), i.e. the value for a solid slab is multiplied by $k_t\, k_{mod}$.

*Table 4.2
k_{mod} values for
trapezoidal
sheeting spanning
transversely to the
supporting beam*

NUMBER OF STUDS PER RIB	POSITION OF MESH	k_{mod}
1	Above the heads of the studs	1.0
1	At least 10 mm below the heads of the studs	1.0
2	Above the heads of the studs	0.7
2	At least 10 mm below the heads of the studs	0.8

Note: Positioning the mesh below the heads of the studs may have practical implications

The value of k_{mod} is given in Table 4.2, depending on the number of studs welded in each rib and the position of the mesh relative to the head of the stud.

The modified design values of resistance should be determined using $\gamma_V = 1.25$.

This guidance may be used when the following criteria are met:

- The height of the steel sheeting profile measured to the shoulder, as described below, is not less than 35 mm nor greater than 80 mm.
- The height of the steel sheeting may be calculated excluding any small re-entrant stiffener on the crest of the profile, provided that the width of the crest of the profile is not less than 110 mm and the stiffener does not exceed 15 mm in height and 55 mm in width.
- The mean width of the ribs of the sheeting is not less than 100 mm.
- The number of stud connectors in one rib at a beam intersection is not more than 2.
- The nominal diameter of the studs is 19 mm, with an as-welded height, h_{sc} of at least 95 mm.
- The ultimate strength of the studs f_u is not to be taken as greater than 450 N/mm².
- The as-welded height of the studs is at least 35 mm greater than the height of the trapezoidal profile, measured to the shoulder.
- The nominal thickness of the steel sheeting is not less than 0.9 mm (bare metal thickness 0.86 mm).
- Where there is a single stud connector per rib, it should be placed in the central position. If this is not possible, studs should be placed in the favourable position (see Figure 4.8). The resistance of studs in the favourable position may be assumed to be the same as that in the central position.

Support   Force   mid span

a.   **Shear connector fixed through profiled steel sheeting in the central position**

Support   Force   Force applied to slab   mid span

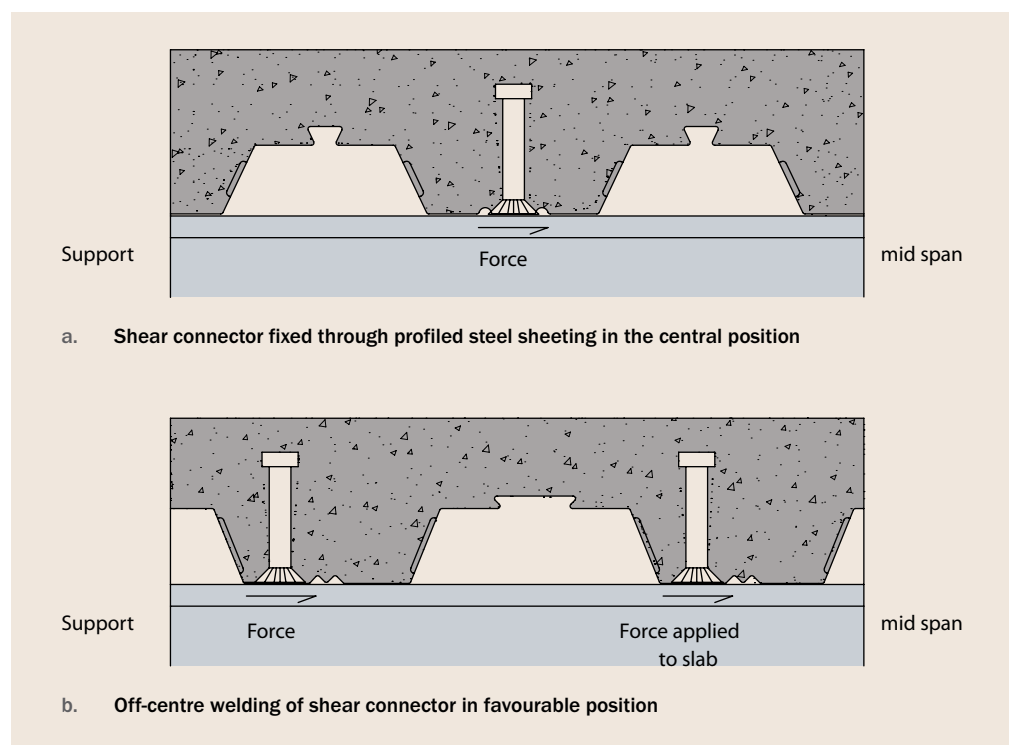b.   **Off-centre welding of shear connector in favourable position**

Figure 4.8
Steel sheeting with a trapezoidal profile showing studs in the favourable and central positions

- Where there are two studs per rib, they should both be placed in the central position, or in the favourable position.

Steel sheeting with a re-entrant profile

Recent experimental work on steel sheeting profiles currently used in composite construction has shown that EC4 stud resistances in re-entrant steel sheeting profiles need no modification.

4.4.3 Minimum degree of shear connection in buildings

The minimum degree of shear connection introduced in 4-1-1/6.6.1.2 ensures that the shear studs have adequate deformation capacity, based on a characteristic slip capacity of 6 mm. In principle, the use of rigid-plastic theory imposes greater deformations on the shear connectors at failure than the linear interaction method (see Section 4.2.3). Therefore, these limits are more conservative when using the linear interaction method. (Limits for shear connectors with greater values of slip capacity have been determined from tests in the UK, and are given in NCCI PN002a-GB[26].)

The degree of shear connection is defined in 4-1-1/6.6.1.2 as:

$$\eta = \frac{n}{n_{\mathrm{f}}}$$

where

n is the number of shear connectors provided in the length L_{e}

n_{f} is the number of shear connectors required for full shear connection in the length L_{e}

L_{e} is the distance (m) between points of zero bending moment (beam span for simply supported beams).

For steel sections with equal flanges, the general limit on the minimum degree of shear connection is defined in 4-1-1/6.6.1.2.

For $L_{\mathrm{e}} \leq 25$ $\eta \geq 1 - \left(\dfrac{355}{f_{\mathrm{y}}}\right)(0.75 - 0.03L_{\mathrm{e}}),$ $\eta \geq 0.4$

For $L_{\mathrm{e}} > 25$ $\eta \geq 1.0$

The influence of the steel strength, f_{y} is introduced because of the higher strains, and hence deformation demands, in plastic design using higher strength steels. The variation of minimum degree of shear connection with L_{e} is shown in Appendix A, Figure A.1.

A relaxation of the degree of shear connection is permitted when all the following conditions are met:

- the studs have an overall length after welding not less than 76 mm and a nominal shank diameter of 19 mm

- the steel section is a rolled or welded I or H section with equal flanges
- the concrete slab is composite with profiled steel sheeting that spans perpendicular to the beam and the concrete ribs are continuous across the beam
- there is one stud per rib
- the proportions of the rib of the slab are $b_o/h_p \geq 2$ and $h_p \leq 60$ mm
- the linear interaction method, described in 6.2.1.3(3) and 4-1-1/Figure 6.5 is used.

When all these conditions are met, the following limits on the minimum degree of shear connection apply:

$$L_e \leq 25: \qquad \eta \geq 1 - \left(\frac{355}{f_y}\right)(1 - 0.04L_e), \qquad \eta \geq 0.4$$

$$L_e > 25: \qquad \eta \geq 1.0$$

The variation of the minimum degree of shear connection with L_e in this case is shown in Appendix A, Figure A.2.

General limits for unpropped members

An unpropped member is defined in 4-1-1/1.5.2.10 as a 'member in which the weight of concrete elements is applied to steel elements which are unsupported in the span'.

Although Eurocode 4 gives no specific rules for the minimum degree of shear connection required for unpropped members, such rules would be less onerous than the equivalent rules for propped construction. Alternative limits for the minimum degree of shear connection that may be used in buildings have been determined and are published as NCCI. Limits for the minimum degree of shear connection to be used for unpropped sections can be found in Section 2.1 of PN002a-GB and are shown in Figure A.3 of this publication. These rules may be employed when the following criteria are satisfied:

- The diameter of the headed studs is not less than 16 mm and not greater than 25 mm.
- The overall height of the headed stud connector after welding is not less than 4 times the diameter.
- The design uniformly distributed imposed floor load ($\gamma_Q q_k$) is not greater than 9 kN/m².

For steel sections with equal flanges:

$$\eta \geq 1 - \left(\frac{355}{f_y}\right)(0.802 - 0.029L_e), \qquad \eta \geq 0.4$$

Figure A.3 shows the variation of minimum degree of shear connection limit with beam span.

Steel sheeting with a trapezoidal profile spanning transversely to the supported beam

Under certain conditions, shear connectors welded through steel sheeting that is transverse to the beam span have been shown to be more ductile than the minimum requirement of Eurocode 4. In such cases, less onerous limits for the minimum degree of shear connection may be used. Guidance on such alternative limits is given below for trapezoidal sheeting spanning transversely to the supporting beam, where:

- The shear connection is achieved using headed studs that are welded to the section through the sheeting.
- The overall height of the headed studs after welding is not less than 95 mm.
- The diameter of the headed studs is 19 mm.

Unpropped members

The limits for the minimum degree of shear connection given for unpropped beams supporting trapezoidal sheeting spanning transversely to the beam and with design imposed floor load ($\gamma_Q q_k$) not greater than 9 kN/m², are given below for symmetric and asymmetric sections.

For steel sections with equal flanges or with an area of the top flange greater than the area of the bottom flange, the limit on the minimum degree of shear connection is:

$$\eta \geq 1 - \left(\frac{355}{f_y}\right)(2.019 - 0.070L_e), \qquad \eta \geq 0.4$$

For steel sections with unequal flanges where the area of the bottom flange is up to 3 times the area of the top flange, the limit on the minimum degree of shear connection is:

$$\eta \geq 1 - \left(\frac{355}{f_y}\right)(0.434 - 0.011L_e), \qquad \eta \geq 0.4$$

The variation of the minimum degree of shear connection with span for unpropped beams is shown in Figure A.4.

Propped members

For a propped member, with transverse steel sheeting and equal flanges the following limitation on the degree of shear connection should be adopted:

$$\eta \geq 1 - \left(\frac{355}{f_y}\right)(1.433 - 0.054L_e), \qquad \eta \geq 0.4$$

The variation of the minimum degree of shear connection with span for propped beams is shown in Figure A.5. For steel sections with unequal flanges, the limits given in 4-1-1/6.6.1.2 should be used.

4.4.4 Detailing of the shear connection

Rules for the detailing of the shear connection are given in 4-1-1/6.6.5.

Headed shear connectors

Rules for the dimensions and spacing of headed stud shear connectors are given in 4-1-1/6.6.5.7. For buildings, additional rules are given in 4-1-1/6.6.5.7 and 6.6.5.8. The principal limits are shown in Figure 4.9 and detailed best practice advice is given in P300[22].

When verifying the construction against these detailing rules the nominal height of the stud may be used as defined in Section 2.5. In a solid slab, there is no additional resistance to be gained by increasing stud height above 80 mm, but with profiled sheeting it is frequently advantageous to use the highest studs that will fit within the slab. In the controlled environment of a building, durability will not be an issue, and zero cover to the top of the stud may be accepted. There is no further benefit in k-factor from a stud which projects more than 75 mm above the steel sheeting.

Steel flange

The dimensions of the flange of the steel section should comply with the requirements of 4-1-1/6.6.5.6; these limits are shown in Figure 4.9.
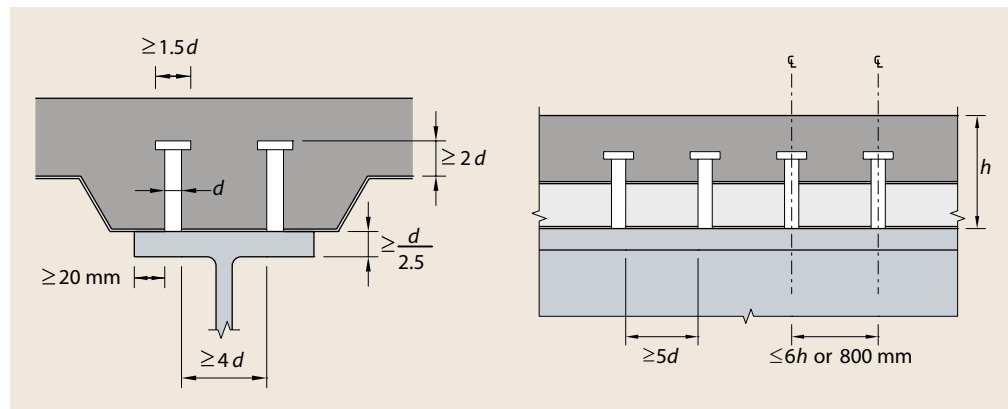

*Figure 4.9
Detailing for
headed shear studs
in buildings*

4.5 Longitudinal shear

It is necessary to ensure that the concrete flange can resist the longitudinal shear force transmitted to it by the shear connectors. The total shear force per metre is the stud resistance times the number of studs per metre.

The rules given in 2-1-1/6.2.4 should be used to determine the design resistance to longitudinal shear for the relevant shear failure surfaces given in 4-1-1/Figures 6.15 and 6.16. The failure surfaces for concrete slabs with sheeting are shown here in Figure 4.10. The model given in BS EN 1992-1-1 is based on considering the flange to act like a system of compressive struts (angled on plan) combined with a system of ties in the form of the transverse reinforcement. The values for the effective transverse

reinforcement per unit length A_{sf}/s_f are given in Table 4.3 where A_t and A_b are the areas of the top and bottom reinforcement per unit length respectively. Values of A_{sf}/s_f are determined using expression 2-1-1/(6.21), as shown in Section 4.5.1.

For profiled steel sheeting transverse to the beam, as shown in Figure 4.10(a), it is not necessary to consider shear surfaces of type 'b' when the stud resistance is reduced by the factor k_t. With parallel or discontinuous transverse sheeting, a failure surface passing round the studs (type 'b') should be considered, as in Figure 4.10 (b). Alternatively, a shorter failure surface that starts and finishes at the truncation points of the top of the ribs (type 'c') may be more critical. (Note that the calculation of these areas depends on rather precise advance knowledge of the sheeting geometry and placement.) The practical solution is to make the studs high enough for type 'a' or 'd' to govern.

In a typical primary beam, which cannot rely on sheeting for transverse reinforcement, the mesh reinforcement will often be inadequate and additional bar reinforcement may be required to achieve sufficient resistance to longitudinal shear.
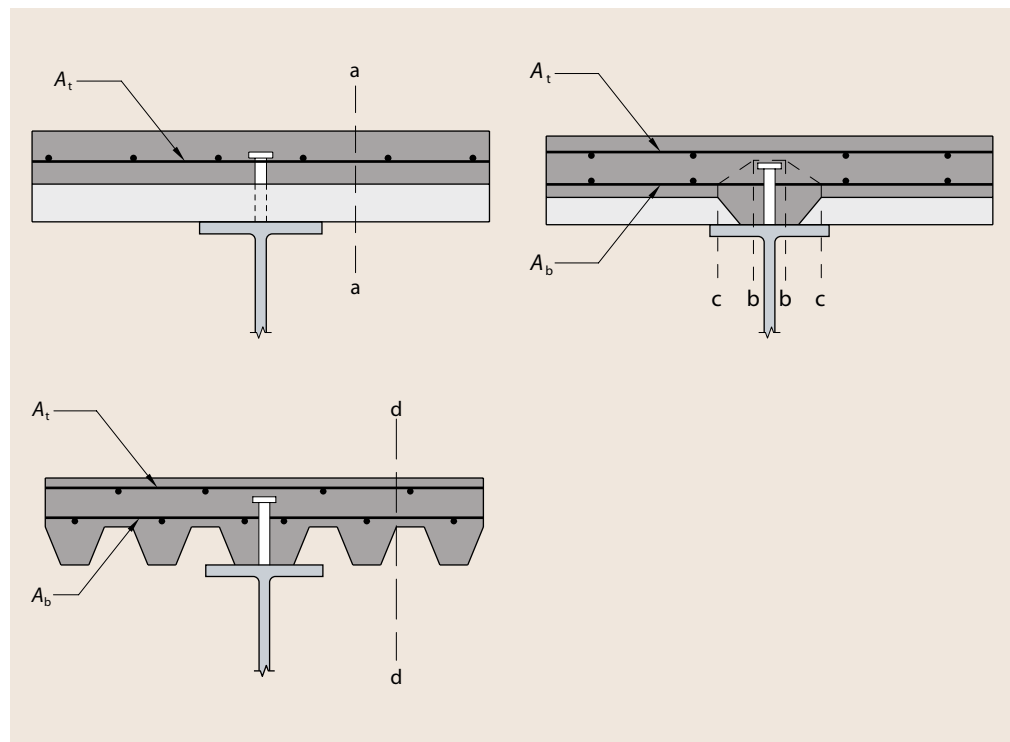
Figure 4.10 Potential surfaces of shear failure for concrete slabs where sheeting is used

FAILURE TYPE	A_{sf}/S_f
a-a	A_t
b-b	$2A_b$
c-c	$2A_b$
d-d	$A_t + A_b$

Table 4.3 A_{sf}/s_f values for slabs with profiled metal sheeting

Note: A_t and A_b are areas per unit length

The shear force will not necessarily be equally divided between the two sides. If the flange is unsymmetrical, because of an edge or an opening, the side with the larger flange area must resist a proportionately higher share of the shear force. The force at any failure surface is proportional to the area outside it, and it may be worth quantifying this if the available concrete resistance is likely to govern in the shear calculation.

4.5.1 Transverse reinforcement

Transverse reinforcement is required to ensure that the longitudinal shear is transferred into the concrete, without failure of the concrete flange. The design shear strength of the concrete flange should be determined in accordance with 2-1-1/6.2.4 using dimensions given in 4-1-1/6.6.6.2. The approach in 2-1-1 is to consider a reinforced concrete T section. In this model, the area of transverse reinforcement per unit length should satisfy:

$$\frac{A_{sf} f_{yd}}{s_f} \geq v_{Ed} \frac{h_f}{\cot\theta_f}$$

where

$\dfrac{A_{sf}}{s_f}$ is the effective reinforcement per unit length for the failure surfaces, as shown in Figure 4.10, taken from 4-1-1/Figure 6.16 and given in Table 4.3.

s_f is the spacing of the reinforcement bars

A_{sf} is the area of reinforcement per unit length

f_{yd} is the design yield strength of the reinforcement (f_{sd} in 4-1-1/1.6)

v_{Ed} is the design value of the transverse shear force

h_f is the depth of the flange, as defined in 4-1-1/6.6.6.2 and 6.6.6.4.
       (= h_c for a composite slab)

θ_f is the angle of dispersion of the force from the shear connector, taken as, $26.3° \leq \theta_f \leq 45°$ for compression flanges.

The minimum area of transverse reinforcement is determined in accordance with 2-1-1/9.2.2(5), which gives the minimum area of reinforcement as a proportion of the concrete area. The ratio is given by expression 2-1-1/(9.5N) as follows.

$$\rho_{w,min} = \left(0.08\sqrt{f_{ck}}\right)/f_{yk}$$

By substituting expression (9.5N) into expression (9.4), the following expression for the minimum area of transverse reinforcement is obtained:

$$A_{sw} = \frac{0.08\sqrt{f_{ck}}\, sh_f \sin\alpha}{f_{yk}}$$

where

A_{sw} is the area of transverse reinforcement within length s.

f_{ck} is the characteristic compressive cylinder strength of the concrete at 28 days.

f_{yk} is the characteristic yield strength of the reinforcement.

s is the spacing of the transverse reinforcement measured along the longitudinal axis of the beam.

h_f is the depth of the concrete flange.

α is the angle between the transverse reinforcement and the longitudinal axis, where $45 \leq \alpha \leq 90°$.

Steel sheeting that is continuous and transverse to the beam can also contribute to the transverse reinforcement. In this case, expression 2-1-1/(6.21) is replaced by expression (6.25) in 4-1-1/6.6.6.4(4), as follows.

$$\left(A_{sf} f_{yd} / s_f \right) + A_{pe} \, f_{yp,d} > v_{Ed} \, h_f / \cot\theta_f$$

where

A_{pe} is the effective cross sectional area of the profiled steel sheeting per unit length of beam neglecting embossments, in accordance with 4-1-1/9.7.2(3).

Profiled steel sheeting that is transverse to the beam but discontinuous over the top flange may still contribute to the transverse reinforcement, provided it is fixed to the beam by shear studs welded through the steel sheeting. The contribution to the transverse reinforcement then becomes a function of the design bearing resistance of the shear stud welded through the sheeting, $P_{pb,Rd}$, as shown in 4-1-1/6.6.6.4(5), expression (6.26). $P_{pb,Rd}$ is calculated in accordance with 4-1-1/9.7.4.

Primary beams must generally rely on supplementary rebar and mesh. Any contribution from longitudinal sheeting is neglected, even if studs are welded through the sheet, because side laps are likely to occur within the width of the effective concrete flange, causing a discontinuity in the sheeting resistance. Side lap fasteners do not provide meaningful resistance to longitudinal shear. Even without joints, a parallel profiled sheet cannot be considered to provide the transverse tensile resistance.

If the mesh and (if available) the sheeting do not provide the resistance required, additional transverse rebar must be provided. This need not extend over the full effective width; the shear force per unit length may be assumed to diminish linearly (in a constant thickness slab) to zero at the edge of the effective width. The supplementary bars can be curtailed at the point where no longer required, so long as tension anchorage is available on both sides of the critical section (at the potential failure surface).

	BAR SIZE			
Anchorage length	8 mm	10 mm	12 mm	16 mm
	230 mm	320 mm	410 mm	600 mm

Table 4.4 Tension anchorage for rebar

Assumptions: Concrete C25/30 (or stronger); cover 25 mm (or more); bond conditions 'good'; design strength $f_{yd} = 434$ N/mm^2

4.6   Edge beams

4.6.1   Composite edge beams

In an edge beam condition, there is only half the normal effective width with which to develop composite action, which may result in a reduced bending resistance. Deflection limits will often control the design of an edge beam; composite action is very beneficial for controlling deflections. The studs are also close to the edge of the slab and steps must be taken to prevent the concrete bursting out along the edge of the slab due to the compressive forces applied by the studs. If the projection of the slab beyond the line of the studs is less than 300 mm, additional reinforcement in the form of U-shaped bars should be provided.

The horizontal U-bars are not just for longitudinal shear. They also prevent longitudinal splitting of a slab with studs near the edge. 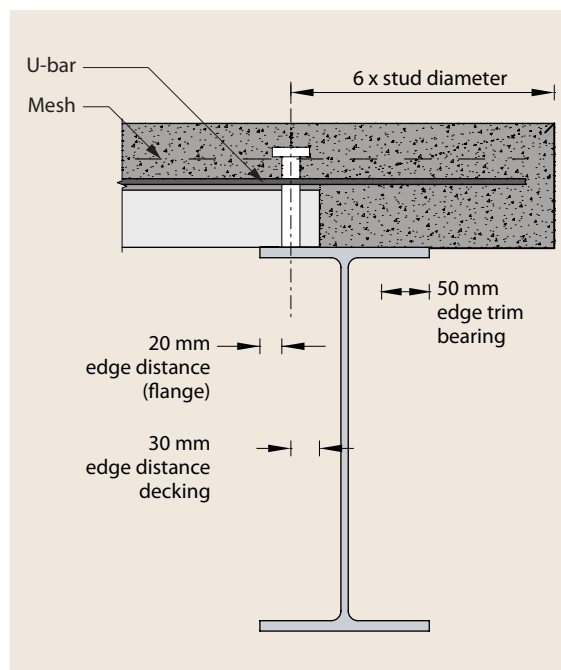A U-bar should pass around each stud or pair of studs, but need not be in direct contact; rather, it should be set close to the edge trim so as to maximize anchorage. U-shaped bars with a minimum diameter of 10 mm (min) are recommended if the stud line is less than 300 mm from the edge. If the projection is 300 mm or more, proper anchorage of the mesh can be achieved.

The shear connectors must be positioned at a distance of at least 6 times the stud diameter (114 mm for 19 mm diameter headed shear studs) from the edge of the slab (see Figure 4.11).
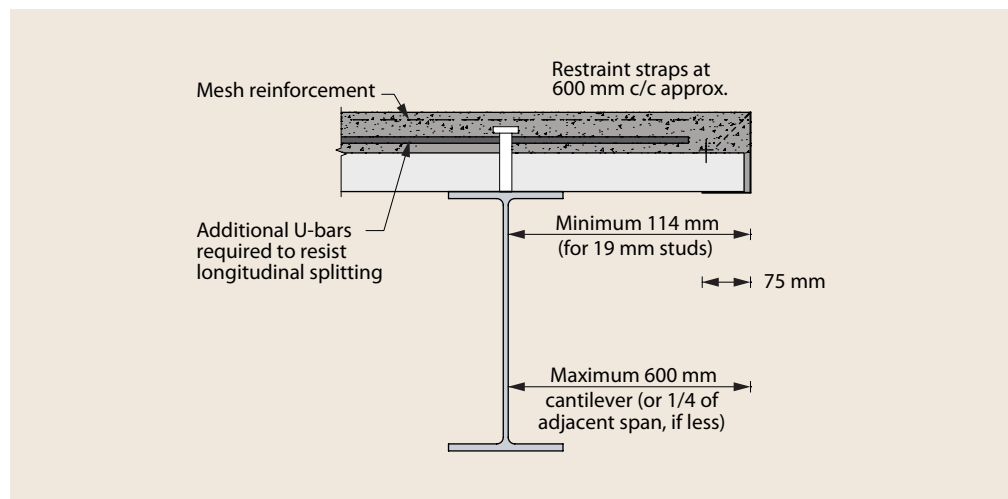

Figure 4.11 Stud position in composite edge beams

Figure 4.12 Typical end cantilever edge beam configuration (ribs transverse to beam)
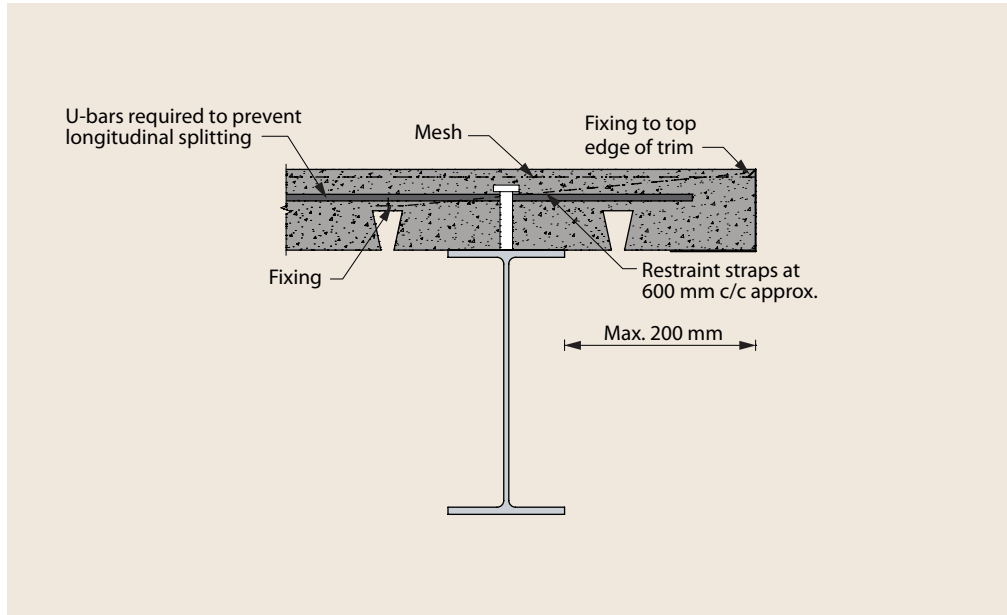
Figure 4.13
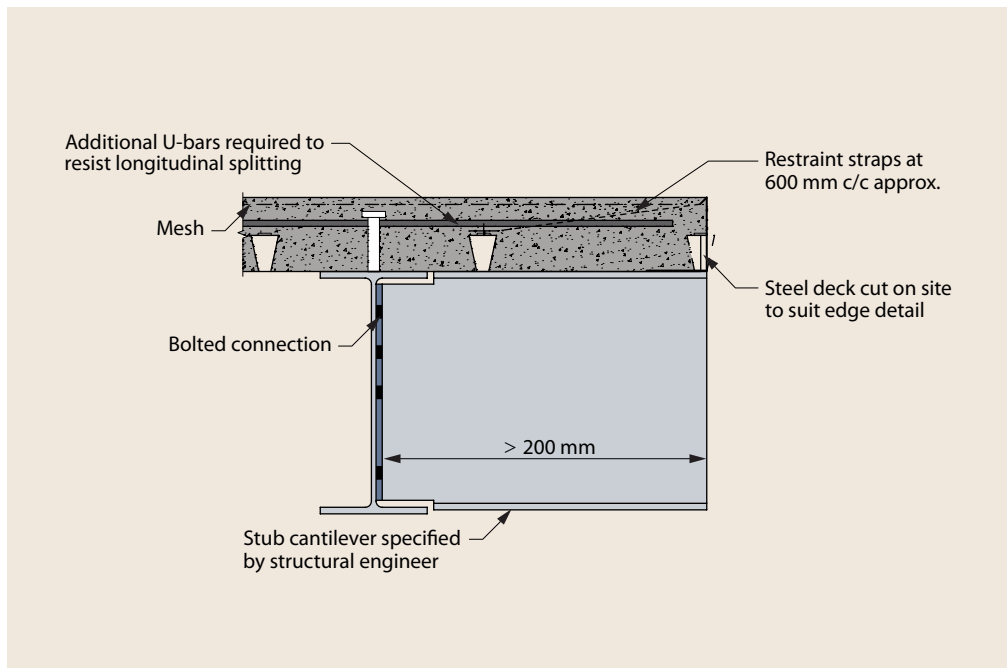*Typical edge beam*
configuration
(decking ribs parallel
to beam)

Figure 4.14
Typical edge beam
configuration with
sheeting supported
on a cantilever beam
(decking ribs parallel
to beam)

Where the profiled steel sheeting is transverse to the edge beam, the slab can cantilever up to 600 mm from the stud position, as shown in Figure 4.12. For primary edge beams, where the profiled steel sheeting spans parallel to the beam (see Figure 4.13), slab cantilevers of more than 200 mm from the flange tip will require stub cantilever beams for support, as shown in Figure 4.14.

4.6.2   Non-composite edge beams

In many composite floors, the edge beams are designed as non-composite beams even though they may have a nominal provision of studs. Removing the additional reinforcement required for composite construction from the slab is beneficial where façade connections are also to be embedded in the edge of the slab, as shown in Figure 4.15.
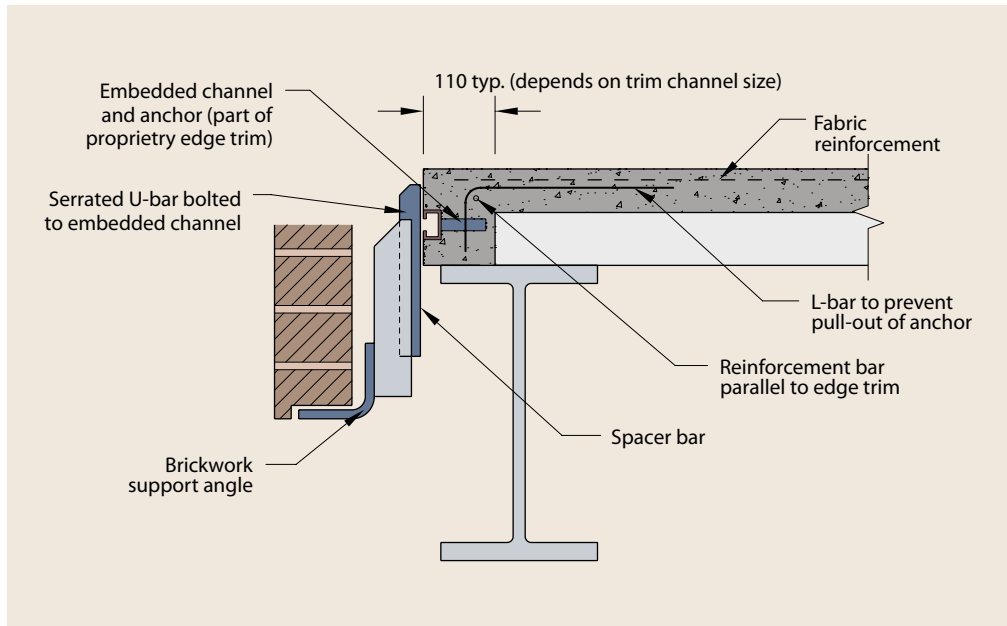
Figure 4.15
A typical cast-in cladding attachment in a composite slab supported on a non-composite edge beam

Edge beams support loads from the floor that are a little over half the load supported by interior beams, plus a line load from the façade. A non-composite edge beam can often be of identical size to the typical interior beam, allowing consistent detailing. With studs added, serviceability performance will compare well with the minimum size composite beam that might otherwise have been selected.

COMPOSITE SLABS AT ULS

The design of composite slabs is usually based on information published by sheeting suppliers (in the form of load/span and fire resistance tables). This is appropriate for a proprietary product, as it saves repetitive effort, avoids error and allows for the superior-to-calculated performance that can be justified by physical testing of the product. The designer needs to be confident in the reliability of the published values.

In composite slabs, the profiled steel sheeting is generally capable of providing all the necessary tension resistance for composite bending resistance. Supplementary rebar is sometimes placed in the ribs, with appropriate cover, to provide tensile resistance in the fire condition.

The design of the profiled steel sheeting is governed by the requirements of the construction stage. Most sheeting profiles are optimised to suit beam centres of at least 3 m while spanning unpropped, to support the wet concrete and construction loading.

When verifying the resistance of a composite slab, 4-1-1/9.4.2(5) allows the slab to be considered as if it were simply supported, even though the finished composite slab is usually continuous over a number of supports. For ULS, the composite action between the concrete and the profiled steel sheeting gives sufficient load carrying capacity as simple spans, without needing to rely on continuity. Mesh reinforcement for crack control is generally provided as blanket coverage over the full slab area, and this is available to act structurally in fire conditions.

5.1 Resistance

5.1.1 Composite action

Once the concrete has hardened, there is composite action between the steel sheeting and the concrete. Local buckling no longer limits the effective section of the sheeting, as it is stabilized by the concrete. The effective area A_{pe} of the steel sheeting is used in the calculation of the flexural resistance. The effective area is calculated ignoring the width of embossments and indentations in the sheeting.

Partial shear connection

A composite slab under test will probably fail to achieve the bending resistance predicted by assuming full shear connection in accordance with 4-1-1/9.7.2, unless

its span is relatively long. At modest (3 – 3.6 m) spans, shear bond is likely to be the critical failure mechanism. The bond between the metal and the concrete reaches its maximum value of resistance, usually between the point of load application and the supports at the ends of the span, and the concrete slips relative to the steel sheeting. Indentations and embossments may be rolled into the profiled steel sheeting to enhance the bond and dovetail features may be included to restrict separation between the two materials.

The concept of partial shear connection in a composite slab has something in common with partial shear connection between beams and slabs, but the two should not be confused. In design, partial interaction is allowed for either directly or indirectly by testing. It is only by testing that a design longitudinal shear resistance can reliably be derived for use in design. 4-1-1/9.7.3(2) limits the use of the method to composite slabs 'with a ductile longitudinal shear behaviour'.

The longitudinal shear behaviour is considered as ductile if the failure load in a test exceeds the load required to produce a slip of 0.1 mm by more than 10%.

Resistance with full shear connection

In the case of full shear connection, the bending resistance per unit width M_{Rd} can be determined by plastic theory. When the neutral axis is located in the slab, above the steel sheeting profile, as shown in Figure 5.1, the compressive force in the concrete $N_{c,f}$ is equal to the tensile force in the profiled steel sheeting, N_p. Thus:

$$N_{c,f} = N_p = A_{pe} f_{yp,d}$$

where

A_{pe} is the effective area of the profiled steel sheeting, per unit width of slab, neglecting embossments and indentations, as required by 4-1-1/9.7.2(3).

$f_{yp,d}$ is the design yield strength of the profiled steel sheeting.

The concrete is assumed to achieve a compressive stress of $0.85 f_{cd}$ over the full depth between the most compressed fibre and the plastic neutral axis. The depth of
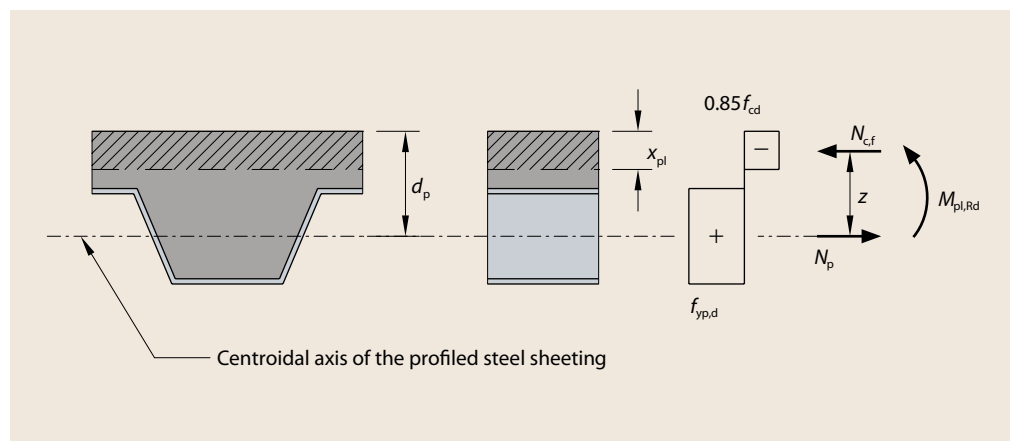

Figure 5.1
Full shear connection with plastic neutral axis in the slab

Centroidal axis of the profiled steel sheeting

the concrete in compression and the position of the plastic neutral axis from the top surface of the slab are given by the following expression:

$$x_{pl} = \frac{N_{c,f}}{0.85 f_{cd} b}$$

The moment resistance is then given by:

$$M_{Rd} = N_{c,f} z$$

where

z is the lever arm, given by:

$$z = h_s - x_{pl}/2 - e$$

h_s is the depth of the slab

e is the height of the centroidal axis of the profiled steel sheeting above the underside of the sheet

b is the unit width of slab.

If the plastic neutral axis is in the profiled steel sheeting, the design moment resistance per unit width is given by:

$$M_{Rd} = N_{c,f} z + M_{pr}$$

where the lever arm is given by:

$$z = h - 0.5 h_c - e_p + \left(e_p - e\right) \frac{N_{c,f}}{A_{pe} f_{yp,d}}$$

and the reduced plastic resistance of the profiled steel sheeting is given by:

$$M_{pr} = 1.25 M_{pa} \left(1 - \frac{N_{c,f}}{A_{pe} f_{yp,d}}\right) \leq M_{pa}$$

in which

e_p is the distance from the underside of the profiled steel sheeting to the plastic neutral axis of the profile

M_{pa} is the bending resistance of the profiled steel sheeting per unit width of slab, allowing for the effect of local buckling in the compressed parts of the sheeting, using effective widths.

Resistance with partial shear connection

The design moment resistance per unit width for partial shear connection is given by the following expression:

$$M_{Rd} = N_c z + M_{pr}$$

The force in the concrete for partial shear connection N_c is given by expression (9.8) in 4-1-1/9.7.3(8) as follows:

$$N_c = \tau_{u,Rd} b L_x \qquad \text{but} \leq N_{c,f}$$

where

$\tau_{u,Rd}$ is the design shear strength ($\tau_{u,Rk}/\gamma_{Vs}$) where $\tau_{u,Rk}$ is obtained from slab tests demonstrating ductile behaviour

b is the unit width of slab

L_x is the distance to the cross section considered from the nearest support.

Note: *If $\tau_{u,Rd} b L_x$ exceeds $N_{c,f}$ then full shear resistance exists.*

The value of partial factor for shear strength is taken as 1.25, in accordance with 4-1-1/2.4.1.2 and NA.2.3.

The lever arm, z, is given by expression (9.9) in 4-1-1/9.7.3(8) as:

$$z = h - 0.5x_{pl} - e_p + \left(e_p - e\right)\frac{N_c}{A_{pe} f_{yp,d}}$$

The force in the concrete for partial shear connection may be enhanced due to the increase in longitudinal shear resistance caused by the support reaction, provided that the design shear strength $\tau_{u,Rd}$ is determined after the effect of the support reaction has been deducted from the measured resistance. The enhanced value is given by:

$$N_c = \tau_{u,Rd} b L_x + \mu R_{Ed} \qquad \text{but} \leq N_{c,f}$$

where

R_{Ed} is the support reaction

μ is a nominal factor, with a recommended value of 0.5.

The presence of additional bottom reinforcement may also be taken into account in the partial shear connection method 4-1-1/9.7.3(10).

Empirical *m* and *k* method

A traditional semi-empirical calculation, permitted by Eurocode 4 as an alternative to its partial shear connection method, is based on two empirical constants *m* and *k*, which may be thought of as representing, respectively, the mechanical interlock and the frictional components of the resistance to interface shear. Both are established by testing, with cyclic loading to break the chemical bond.

The main advantage of the *m* and *k* method is that it can be applied in cases where the longitudinal shear behaviour is non ductile.

If the m and k method is used, the designer should verify that the maximum design vertical shear V_{Ed} for a given width of slab, b, does not exceed the design shear resistance $V_{l,Rd}$ given by the following expression:

$$V_{l,Rd} = \frac{bd_p}{\gamma_{Vs}}\left(\frac{mA_p}{bL_s} + k\right)$$

where

b	is the unit width of the composite slab
d_p	is the distance between the centroid axis of the profiled steel sheeting and the extreme fibre of the composite slab in compression
A_p	is the nominal cross sectional area of the profiled steel sheeting per unit width of slab
m, k	are design values of empirical factors (N/mm²) based on slab tests and available from the profiled sheeting manufacturer
L_s	is the shear span.

For design, L_s should be taken as $L/4$ for uniformly applied loads over the entire span or the distance between the point of load application and the nearest support for two equal and symmetrically placed point loads. For other loading arrangements, an assessment should be made based on test results.

End anchorage

For sheeting that has mechanical and frictional interlock with the concrete, the shear connection may be enhanced by end anchorage in the form of headed shear studs welded through the steel sheeting, or by deformation of the ends of the steel sheet.

In these cases, the design resistance of the slab may be determined using the partial shear connection method, with the force in the concrete N_c increased by the design resistance of the end anchorage, in accordance with 4-1-1/9.7.4.

The design resistance of headed shear studs welded through the profiled steel sheeting and used for anchorage may be determined using the following expression:

$$P_{pb,Rd} = k_\varphi d_{do} t f_{yp,d}$$

with

$$k_\varphi = 1 + a/d_{do} \leq 6.0$$

where

d_{do}	is the diameter of the weld collar, which may be taken as 1.1 times the diameter of the shank of the stud
a	is the distance from the centre of the stud to the end of the sheeting (to be not less than $1.5d_{do}$)
t	is the thickness of the sheeting.

5.1.2 Vertical shear

The vertical shear resistance $V_{v,Rd}$ of a composite slab over a width equal to the distance between the centres of the ribs should be determined in accordance with 2-1-1/6.2.2. It should be verified that the shear force on the cross section (V_{Ed}) is less than or equal to the design shear resistance of the member without shear reinforcement ($V_{Rd,c}$).

The resistance of the cross section is dependant on the area of tensile reinforcement (A_{sl}) crossing the shear plane. The tensile reinforcement needs to extend beyond the section considered by an appropriate anchorage length ($l_{bd} + d$). For a composite slab subject primarily to uniform loading, the shear resistance need not be checked at a distance less than d from the face of the support, meaning that the reinforcement must extend beyond the face of the support by a distance equal to the design anchorage length (l_{bd}). The design shear resistance of a member without shear reinforcement is given by the following expression:

$$V_{Rd,c} = \left[C_{Rd,c}\, k \left(100\, \rho_1\, f_{ck}\right)^{\frac{1}{3}} + k_1\, \sigma_{cp} \right] b_w d$$

but not less than the value given by the following:

$$V_{Rd,c} = \left(v_{min} + k_1 \sigma_{cp} \right) b_w d$$

where

$k \qquad = 1 + \sqrt{\dfrac{200}{d}} \le 2.0$

$\rho_1 \qquad = \dfrac{A_{sl}}{b_w d} \le 0.02$

$f_{ck} \qquad$ is the characteristic cylinder strength of the concrete

$A_{sl} \qquad$ is the area of the tensile reinforcement, take as A_{pe} for a composite slab

$b_w \qquad$ is the smallest width of the cross-section in the tensile area

$d \qquad$ is the effective depth of the section, taken as the depth from the top surface to the centroid of the profile for a composite slab

$l_{bd} \qquad$ is the design anchorage length

$\sigma_{cp} \qquad = N_{ed}/A_c < 0.2 f_{cd}$

$N_{Ed} \qquad$ is the axial force in the cross section due to loading or prestressing

$A_c \qquad$ is the area of concrete cross section.

The values of the following nationally determined parameters (defined in 2-1-1/6.2.2(1)) are given in 2-1-1/Table NA.1.

$k_1 \qquad = 0.15$

$C_{Rd,c} \qquad = 0.18/\gamma_c$

$v_{min} \qquad = 0.035 k^{\frac{3}{2}} f_{ck}^{\frac{1}{2}}.$

5.1.3 Resistance to concentrated loads

The design of composite slabs to resist concentrated loads should be given careful considered. As composite slabs are one-way spanning elements, their ability to carry concentrated loads is limited. The most onerous effect is generally the concentration of shear, for which the worst location is at twice the effective depth of the slab from the edge of a beam. At such locations, the majority of the load is carried to the adjacent beam via a small number of ribs, perhaps just two. This is likely to be more severe than punching shear, as illustrated in 4-1-1/Figure 9.8.

It is recommended that the resistance to localized shear should be verified in accordance with Eurocode 2 for the reinforced concrete section, with the regular mesh in the top of the slab, increased as necessary, acting as tensile reinforcement. In many typical composite slabs of thickness around 130 mm, there would be severe practical difficulties in accommodating a second layer of mesh at low level. Increasing the first layer is a more realistic option.

4-1-1/9.4.3 gives rules for calculating the effective width of the slab that can be mobilised to support a concentrated load. The effective width is a function of the longitudinal and transverse stiffness of the slab and the rules are based on a combination of experience, experimentation and simplified analysis.

If the characteristic distributed (line) load[1] does not exceed 5 kN/m, the resistance to concentrated loads up to 7.5 kN may be assumed to be adequate without calculation, if there is nominal transverse reinforcement that complies with 4-1-1/9.4.3(5)). For loads of greater magnitude, bending moments in the slab must be calculated and the resistance determined in accordance with Eurocode 2, as required by 4-1-1/9.4.3(6); a method for verifying the adequacy is given in below.

There is an increasing use of composite slabs in storage buildings, where forklift trucks are in frequent use; the slab must then be capable of supporting point loads at any location, rather than have a small number of special positions reinforced locally. The additional reinforcement required for the point loads must be provided over the whole of the floor slab. For heavy concentrated loads, punching shear and local shear transfer must be considered, in addition to normal flexural action.

The punching shear resistance $V_{p,Rd}$ of a composite slab at a concentrated load position should be determined in accordance with 2-1-1/6.4.4, where the critical perimeter is taken as shown in Figure 5.2.

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[1] The limiting value of line loading is incorrectly quoted as a distributed load per unit area in 4-1-1/9.4.3(5). The information given above (i.e. that it is a line load of 5 kN/m was obtained from the *Design manual for composite slabs*, ECCS Publication 87, 1995.
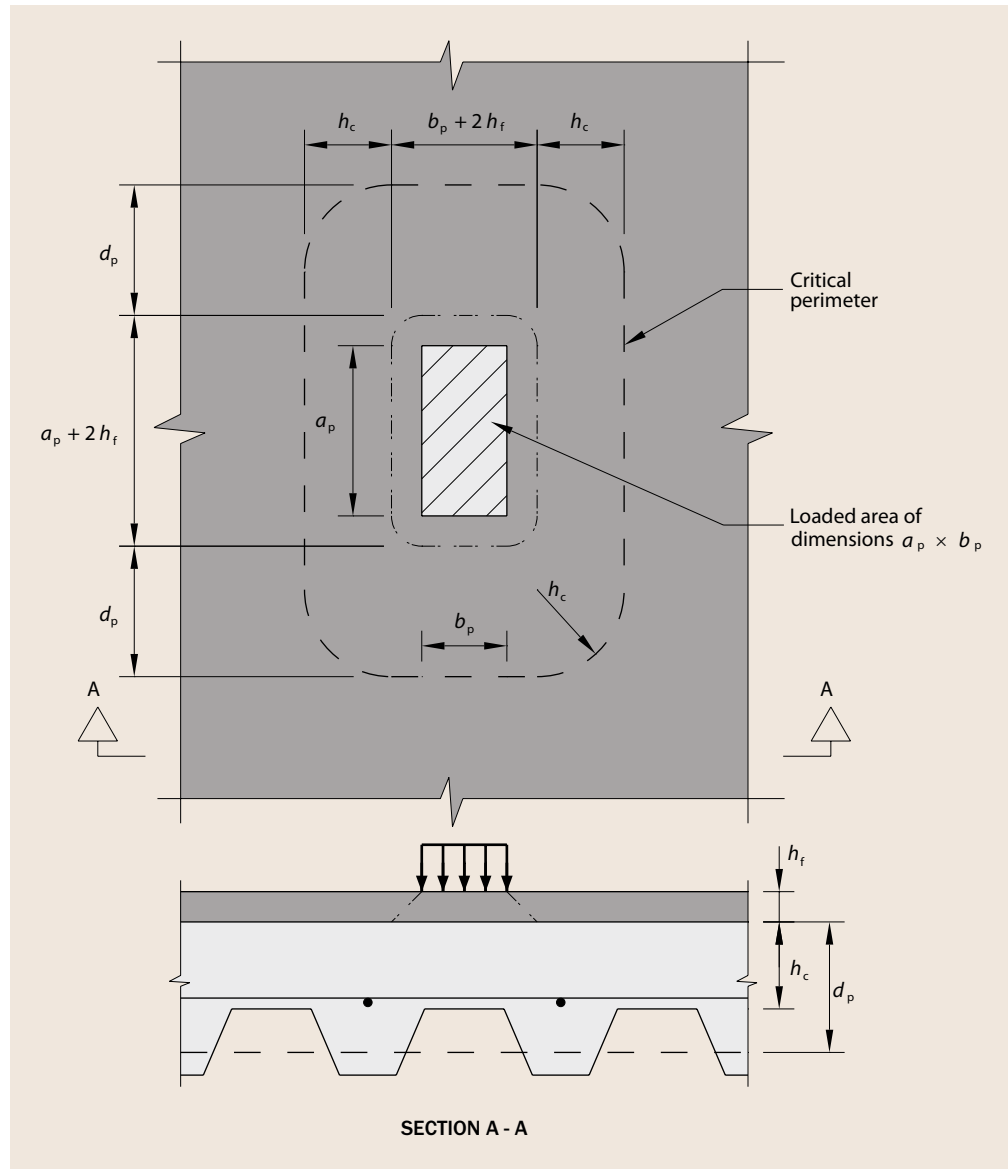
Figure 5.2
Critical perimeter for
punching shear

Verification of adequacy for heavy loading

Where a concentrated (line) load exceeds a characteristic value of 7.5 kN or a distributed load has a characteristic value greater than 5.0 kN/m, adequate transverse reinforcement must be provided in accordance with 4-1-1/9.4.3(6).

A concentrated load, or a line load parallel to the span, may be considered to be distributed over a width b_m given by expression 4-1-1/(9.1) as:

$$b_m = b_p + 2(h_c + h_f)$$

where

b_p is the width of the contact area under the load

h_f is the thickness of finishes

h_c is the depth of slab over the steel sheeting profile ($= h_s - h_p$).

The width of slab effective for resistance in sagging bending may be determined using expressions 4-1-1/(9.2) to (9.4). For a simply supported slab, or the external span of a continuous slab, the effective width is given by expression 4-1-1/(9.2) as:

$$b_e = b_m + 2L_p(1 - L_p/L) \quad \text{but not greater than the width of the slab}$$

where:

L is the span of the slab

L_p is the position of the concentrated load in the span.

While the load is distributed over a width b_m, the resistance in sagging transverse bending is distributed over a width b_e.

The transverse bending moment due to a point load Q_d can be taken as:

$$M_{Ed} = Q_d \frac{b_e - b_m}{8}$$

The transverse bending resistance of the slab is given by:

$$M_{Rd} = A_s f_{sy} (h_s - h_p - x_c)$$

where

A_s is the area of transverse reinforcement over a width b_e

f_{yd} is the design strength of the reinforcement

h_s is the depth of the composite slab

h_p is the height of the steel sheeting measured to the shoulder of the profile

x_c is the depth of concrete in compression.

In accordance with the detailing rules given in 2-1-1/9.3.1.1, the transverse reinforcement in the slab should be a minimum of 20% of the area of the principal reinforcement. In the case of composite slabs, the principal reinforcement is the steel deck. The transverse reinforcement should extend over the effective width b_e of the slab and should have an appropriate anchorage.

5.2   Openings in the slab

Suppliers of profiled steel sheeting also offer suggestions for the treatment of openings. A key principle is that the sheeting should remain intact until the concrete has cured. Typically, openings of up to 700 mm square are blocked out in plywood, with the sheeting cut only after the concrete has hardened. Additional reinforcement is often required to trim the opening and compensate for lost bending resistance in the normal stage. A single isolated opening of 300 mm square or less may be acceptable without trimming reinforcement.

Openings in excess of 700 mm square need to be surrounded with trimming beams that are designed to support the slab spanning onto them. With fully trimmed openings, the sheeting may be cut prior to concreting.

Any opening to be made in a composite slab after concreting must be formed non-percussively to avoid bond damage. Diamond drilling is acceptable.

For further guidance on forming openings in composite slabs, see 'Composite slabs and beams using steel decking. Best practice for design and construction' (P300)[22].

SERVICEABILITY LIMIT STATE

In accordance with BS EN 1990, A1.4.2, serviceability limit states in buildings should take into account criteria related to floor stiffness, expressed in terms of limits for vertical deflections and for vibrations. The serviceability criteria should be specified for each project and agreed with the client.

6.1 Composite beams

6.1.1 General criteria

Composite beams should be verified using the combinations of actions and criteria given for the serviceability limit state in BS EN 1990. The UK National Annex (NA.2.2.6) to that Eurocode recommends the use of the following combinations of actions for the serviceability limit state:

Characteristic combination – for irreversible limit states, which may include impairment of functional performance and damage to structural elements, non-structural elements (e.g. partition walls) and finishes. The verification of deflections should include the permanent and variable actions present following construction of the element or finishes concerned.

Frequent combination – for reversible limit states, such as dynamic effects that influence the comfort of the users.

Quasi-permanent combination – for long term effects such as creep and cases where deflections are only likely to influence the appearance of the structure.

BS EN 1990 gives combination factors for use in these combinations of actions. For typical floors within the scope of this publication, the variable actions that occur may be considered not to be independent of each other. Therefore, only combination factors ψ_1 and ψ_2 are required. See Section 1.4.2 of this publication for expressions for the combinations of actions.

6.1.2 Calculation of deflections

4-1-1/7.3.1 states that elastic analysis should be used to determine the deflection of composite members.

Creep effects

Creep is a time-dependent inelastic strain phenomenon. Its effects are initially rapid but the creep rate decreases with time. The strain resulting from creep can eventually exceed the elastic strain by a factor of two or more. The single most important influence on the magnitude of creep strains is the age of the concrete at first loading.

For beams with one composite flange, creep can be allowed for by using an appropriate modular ratio for the concrete calculated in accordance with expression 4-1-1/(5.6). The use of a modular ratio allows deflections to be calculated in a familiar quasi-elastic way, using standard beam formulae. Creep coefficients for use in expression (5.6) are determined using the rules given in 2-1-1/3.1.4 (normal weight concrete) and 11.3.3 (lightweight concrete).

The modular ratio n_0 for short term loading (with no inelastic effects) is given in 4-1-1/5.4.2.2 as:

$$n_0 = \frac{E_a}{E_{cm}}$$

where

E_a is the modulus of elasticity of structural steel (given as 210 000 N/mm² in 3-1-1/3.2.6)

E_{cm} is the secant modulus of elasticity of concrete for short term loading.

Values for E_{cm} for normal weight concrete are given in 2-1-1/Table 3.1. Alternatively, the following expression may be used to determine E_{cm}:

$$E_{cm} = 22\left(\frac{f_{cm}}{10}\right)^{0.3} \text{ (N/mm}^2)$$

where

f_{cm} is the mean value for the cylinder compressive strength of concrete, taken as $f_{ck} + 8$.

For lightweight concrete, E_{cm} is denoted as E_{lcm}. Based on the expressions given in 2-1-1/Table 11.3.1, E_{lcm} may be determined using:

$$E_{lcm} = E_{cm}\left(\frac{\rho}{2200}\right)^2 \text{ (kN/mm}^2)$$

where

ρ is the upper limit of the density for the relevant class of concrete, as given in 2-1-1/Table 11.1.

For the consideration of the effects of long term loading, creep of the concrete must be considered. Creep will depend on the relative humidity within the building and on the age at first loading. For typical internal environments within buildings the relative humidity can be taken as 40%. For unpropped construction, the age at first loading t_0,

Table 6.1
Properties for some
common concrete
classes (from 2-1-1/
Table 3.1 and
Table 11.3.1)

	C25/30	C30/37	LC25/28 1800 kg/m³	LC30/33 1800 kg/m³
Characteristic cylinder compressive strength f_{ck} (N/mm²)	25	30	25	30
f_{cm} (N/mm²)	33	38	33	38
Secant modulus of elasticity E_{cm} (GPa)	31	33	21	22

may be taken as 1 day. For structural elements propped during construction, the age at first loading can be considered to be when the props are removed. Consideration should also be given to the use of cement replacements in this context. While the 28 day strength of a concrete with cement class R will be similar to an equivalent concrete grade with cement class N, the graph of strength gain with time may be very different resulting in higher creep strains when cement replacement is used. This effect is included in design by modifying the age of the concrete when load is applied, t_0.

The long term modular ratio may be calculated using 4-1-1/Equation (5.6) reproduced below:

$$n_L = n_0(1 + \psi_L \varphi_t)$$

where

n_L Modular ratio appropriate to the type of loading

φ_t Creep coefficient $\varphi(t,t_0)$ from 2-1-1/3.1.4 or 11.3.3

t is the age of the concrete in days at the time considered in the design

t_0 is the age of the concrete in days at loading. May be modified to take account of different cement classes, see 2-1-1/B.9

ψ_L is the creep multiplier, which, depending on the type of loading, is taken as:

         $\psi_L = 1.1$ for permanent loads

         $\psi_L = 0.55$ for primary and secondary effects of shrinkage.

When calculating deflections due to variable occupancy loads, the modular ratio may be calculated assuming two thirds short term loading and one third long term loading. The short term modulus can be determined easily from the secant modulus of elasticity of the concrete. For long term loading, the modular ratio for internal environments is at least 3 times the short term value.

Second moment of area

Deflection may be calculated using elastic properties. In positive bending (sagging), the concrete may be assumed to be uncracked. An approximation for the second moment of area of the uncracked transformed section is:

$$I \approx \frac{A_a \left(h_c + 2h_d + h_a \right)^2}{4(1 + nr)} + \frac{b_{eff} h_c^3}{12n} + I_a$$

where

A_a is the area of the steel section

I_a is the second moment of area of the steel section

h_c is the depth of the concrete above the steel sheeting profile $(h_s - h_d)$

h_a is the depth of the steel section

h_d is the overall height of the steel sheeting profile

n is the modular ratio for the design situation considered. For deflection due to occupancy loads, the modular ratio may be taken as a proportion of the long term and short term ratios, as noted above

r is the ratio of the cross sectional area of the steel section relative to the concrete section, $\dfrac{A_a}{b_{eff}\, h_c}$

b_{eff} is the effective width.

Where the neutral axis is within the concrete slab, concrete below that level is neglected during resistance verifications. However, that concrete will contribute some stiffness so should be included when determining the second moment of area.

Influence of partial shear connection

The effects of partial shear connection on the deflection of the beam may be neglected when the following criteria in 4-1-1/7.3.1(4) are satisfied:

- the design of the shear connection is in accordance with Eurocode 4
- either no less shear connectors are used than half the number for full shear connection, or the forces resulting from an elastic behaviour and which act on the shear connectors in the serviceability limit state do not exceed P_{Rd} and
- in the case of a ribbed slab with ribs transverse to the beam, the height of the ribs does not exceed 80 mm.

If these criteria are not satisfied, the influence of shear connection on the deflection is considered significant but Eurocode 4 provides no guidance for calculating this deflection.

Where the guidance in NCCI PN002a-GB has been followed for the minimum degree of partial shear connection, the above criteria will not be met. In such cases, the deflection of the beam should be calculated using:

Propped construction $\delta = \delta_c + 0.5\,(1 - \eta)\,(\delta_s - \delta_c)$

Unpropped construction $\delta = \delta_c + 0.3\,(1 - \eta)\,(\delta_s - \delta_c)$

where

δ_s is the deflection of the steel beam acting alone

δ_c is the deflection of a composite beam acting with full shear connection for the same total loading used to determine δ_s

η is the degree of shear connection given in NCCI PN002a-GB (see Section 4.4.3 of this publication).

Shrinkage effects

Shrinkage is the other inelastic phenomenon that is liable to increase the deflection of a composite structure. 4-1-1/5.4.2.2(1) requires appropriate allowance for the effects of shrinkage. For beams with one composite flange, shrinkage can be allowed for using an appropriate modular ratio for the concrete calculated in accordance with expression 4-1-1/(5.6) (this modular ratio is different from that for long term imposed loads). The use of a modular ratio allows deflections to be calculated in a familiar quasi-elastic way, using standard beam formulae.

The curvature, K_s, due to a free shrinkage strain, ε_s, is:

$$K_s = \frac{\varepsilon_s \left(h_c + 2h_p + h_a \right) A_a}{2\left(1 + nr\right) I_c}$$

where

n	is the modular ratio appropriate for shrinkage calculations
r	is the ratio of the cross sectional area of the steel section relative to the concrete section
A_a	is the area of the steel section
I_c	is the second moment of area of the composite section
h_a	is the height of the steel section.

For dry environments within buildings, 4-1-1/Annex C states that the total final free shrinkage strain may be taken as:

$$\varepsilon_s = 325 \times 10^{-6} \qquad \text{for normal weight concrete}$$

$$\varepsilon_s = 500 \times 10^{-6} \qquad \text{for lightweight concrete}$$

The deflection due to shrinkage induced curvature is calculated from:

$$\delta_s = \frac{K_s L^2}{8}$$

where

L	is the span of the composite beam.

6.1.3   Serviceability stress verification

Serviceability stress verifications are not prescribed by Eurocode 4. However, following the guidance given in BS EN 1990, A1.4.2, stress checks may be included as part of the serviceability criteria where there is a risk of inelastic deflection under serviceability loading. As deflections are based on elastic analysis, it seems prudent to have some verification that this assumption is valid.

6.2 Composite slabs

6.2.1 Crack control

Clause 2-1-1/7.4.1(3) states that the adoption of minimum reinforcement (7.4.2 and 7.4.3) will limit crack width to what is 'acceptable'. Moreover, it is not necessary to adopt minimum reinforcement if the beams are simply supported and control of crack width is not critical for design, as will often be the case for floors within the scope of the present publication.

For unpropped construction, the requirement is 0.2% of the area of concrete, and for propped construction this is doubled to 0.4%. Strictly, this reinforcement is not required at midspan, but in practice it takes the form of blanket coverage of mesh. It applies equally to composite slabs and to composite beams.

Table 6.2 Minimum mesh sizes for unpropped construction

OVERALL SLAB DEPTH (mm)	MINIMUM MESH SIZE FOR PROFILED SHEETING		
	CF51	CF60	CF80
125	A193		
130	A193	A142	
135	A193	A142	
140	A252	A142	A142
145	A252	A142	A142
150	A252	A193	A142
155	A252	A193	A142
160	A252	A193	A142
165	A252	A193	A142
170	A252	A252	A193
175	A252	A252	A193

Note: These sizes are based on providing a minimum area of reinforcement equal to 0.2% of the concrete area above the profile. The minimum area of reinforcement increases to 0.4% for propped construction. Mesh may also need to be increased to cater for the fire condition or concentrated loads.

The reinforcement area is calculated as a percentage of the concrete area above the profile, using the depth used in beam design. The reinforcement is placed with minimum cover to the top of the slab; with the thinnest composite slabs this means roughly in the middle of the limited depth available. In such cases the use of mesh with flying ends is recommended to reduce congestion in the slab at laps and to ensure minimum cover is preserved.

6.2.2 Deflection

After the concrete has hardened, imposed loads will act on the much stiffer composite section. Just how much stiffer in the long term is dependant on creep, and therefore on load history. A useful contribution comes from the mesh extending full span in the top of the slab. The mesh reinforcement will increase the stiffness and continuity at internal supports.

For composite slabs where both the following criteria are satisfied, 4-1-1/9.8.2(4) states that calculation of the deflections may be omitted:

▪ Span / effective depth ratio does not exceed the limits given in 2-1-1/7.4 for lightly stressed concrete.

▪ The effect of end slip can be ignored, which is the case for most composite slab construction in the UK.

Based on 2-1-1/Table 7.4N, limits for span / depth ratios are given in Table 6.3.

Table 6.3
Limiting span /depth ratios for lightly stressed concrete slabs (values taken from P300, Table 4.5)

STRUCTURAL SYSTEM	SPAN / DEPTH FOR LIGHTLY STRESSED CONCRETE	
	NORMAL WEIGHT CONCRETE	LIGHTWEIGHT CONCRETE
Single spans	20	18.8
End spans	26	24.5
Internal spans	30	28.3

In normal (non-storage) building practice, the thickness of a composite slab is rarely controlled by deflection.

Shrinkage induced deflections

Shrinkage of the concrete slab will contribute a non reversible deflection as the slab dries out, which is in addition to the deflection predicted by the elastic calculation, but the effect is normally relatively small and to some extent relieved by cracking and creep.

Where the above limits on span to depth ratio are observed and the slab is nominally continuous, the effects of shrinkage do not need to be allowed for when determining its deflection using elastic analysis (4-1-1/9.8.2(3)). However, in long span situations where these span to depth ratios are exceeded, the effect of shrinkage should be included.

For dry environments within buildings, 4-1-1/Annex C states that the free shrinkage strain may be taken as:

ε_s $= 325 \times 10^{-6}$ for normal weight concrete

ε_s $= 500 \times 10^{-6}$ for lightweight concrete

Shrinkage-induced deflections of composite slabs may be calculated from a simplified formula:

$$\delta_{sh} = \frac{\varepsilon_s h_c (y_e - 0.5h_c)}{8n_L I_c}$$

where

h_c is the depth of concrete topping

y_e is the depth of the elastic neutral axis of the composite slab (as for I_c)

I_c is the second moment of area of the composite slab using n_L

n_L is the modular ratio for long term loads.

6.3 Vibration

In the UK, the traditional approach used to determine the sensitivity of a floor to vibrations has been to determine the natural frequencies of the primary and secondary supporting beams. If the natural frequencies were found to be greater than 4 Hz the floor was considered acceptable for normal use e.g. offices. A new approach has been developed and presented in SCI publication P354[27].
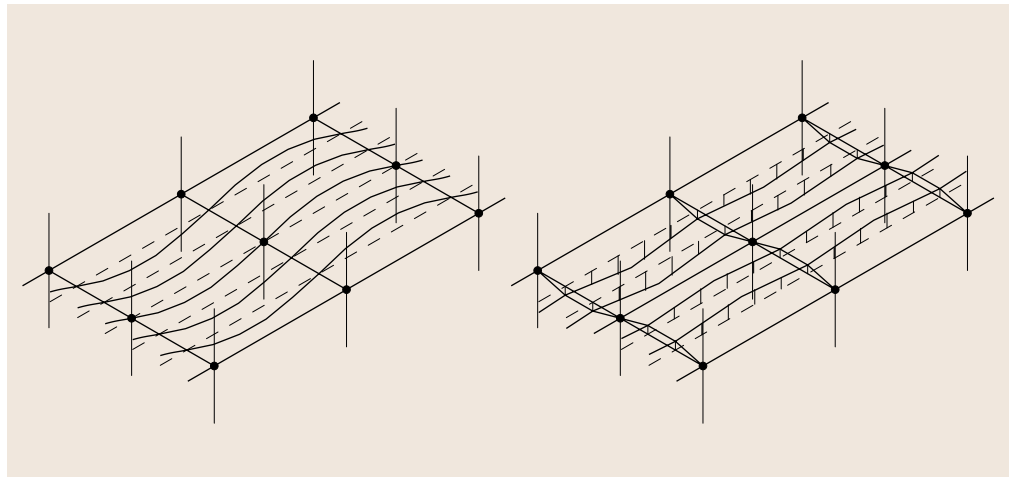
Figure 6.1
Floor vibration modes
A and B

There are typically two modes to consider. In Mode A, alternate secondary spans may be deflecting up and down (effectively simply supported) with participation of the slab (as fixed ended) but not the primary beams. In Mode B the primary beams may be deflecting in the same manner, but in this case the secondary beams and the slab, which are effectively fixed ended, contribute extra deflection (but are effectively fixed ended). For this case (Mode B), δ is the sum of three contributions.

The lower of the two natural frequencies calculated is the fundamental frequency. For composite floors, the fundamental frequency should be at least 3 Hz to ensure that walking activities will be outside the frequency range which could cause resonance.

The design procedures for determining the dynamic performance of a composite floor include the following steps:

▪ Determine the natural frequency.
▪ Determine the modal mass for the floor.
▪ Evaluate the response of the floor.
▪ Verify the response of the floor against the requirements.

Detailed guidance on the above steps is given in P354.

For hospitals and other particularly sensitive occupancies, a more detailed analysis should be undertaken.

6.4 Deflection limits

Eurocode 4 does not specify deflection limits for composite beams. Therefore, the deflection limits for composite beams should be specified for each project depending on the sensitivity of the finishes, visual appearance, etc. to meet the client's needs.

For composite slabs, 4-1-1/9.8.2(4) allows a span/depth ratio criterion to be substituted for a deflection calculation. A limit of $L/350$ or 20 mm is recommended for the deflection of a composite slab due to imposed loads. The deflection due to the total load less the deflection due to the self weight of the slab should be limited to $L/250$. For propped construction, this deflection should also include the deflection due to prop removal.

BEAM TYPE	LOAD CASE	LIMIT	ABSOLUTE LIMIT (mm)
Internal beams	Imposed load	Span/360	To suit finishes
	Total load	Span/200	To suit finishes
	Dead load at construction stage	—	25 mm[†]
Edges beams supporting floor and cladding	Imposed load	Span/500	To suit cladding
	Imposed load plus cladding	Span/360	To suit finishes
	Total load	Span/250	To suit cladding
Edge beams supporting cladding only	Cladding self weight	Span/500	To suit cladding

Table 6.4 Recommended limiting values for vertical deflection of composite beams[22]

[†]   This is not a serviceability criterion but is intended to limit the additional load due to ponding of the concrete.

FIRE DESIGN OF COMPOSITE BEAMS AND SLABS

Fire safety is one of the essential requirements in the Eurocode regulatory framework and the Eurocodes aim to provide a reference document for the purpose of achieving a fire safe design.

The main requirements for fire safety are given in national legislation. While these documents will differ in detail between member states, the principles are common. In the UK separate legislative documents exist for England & Wales[28], Scotland[29] and Northern Ireland[30]. Guidance on the application of the legislation is given in Approved Document B[31], Technical Standards[32] and Technical Booklet E[33] in each of the UK administrative regions.

BS EN 1990, 1.5.2.5, refers to fire design in the definition of accidental design situations. Therefore the combination of actions given in BS EN 1990, (6.11) is used for fire design. Fire design is concerned with the ultimate limit state, therefore the structural resistance must remain adequate for a specified period of time.

Fire exposure may be determined from BS EN 1991-1-2. In most cases a nominal fire exposure would be used, of which the standard time-temperature curve shown in Figure 7.1 is most common. The Eurocodes give suitable thermal and mechanical models to allow the assessment of steel concrete composite structures.

7.1   Design basis

Clause 4-1-2/2.1.2 defines the failure criteria for nominal fire exposure under three headings:

- Mechanical resistance, or load bearing capacity (R).
- Integrity (E).
- Insulation (I).

A structural element must maintain adequate mechanical resistance for the required period in fire conditions and, if it is also a separating element, then insulation and integrity criteria must also be fulfilled.

Composite floor slabs are often designed to be separating elements and must therefore behave adequately in accordance with each of these performance criteria; composite beams are only required to meet criterion R.

In the UK, periods of fire resistance given in Approved Document B to the Building Regulations range between 30 and 120 minutes in increments of 30 minutes. Typically for a low to medium rise commercial building, 60 minutes or 90 minutes fire resistance is sufficient. This would cover the majority of UK construction. High rise buildings whose highest floor is above 30 m would require 120 minutes.

The fire resistance periods correspond to the standard time-temperature curve given in 1-1-2/3-2-1 and shown in Figure 7.1. The expression that describes the standard time-temperature curve is as follows:

$$\theta_t = 20 + 345 \, \text{Log}(8t + 1)$$

Provision of an effective sprinkler system will control or extinguish fires that may occur in the building. Most of the UK technical guidance documents allow a reduction in the required period of fire resistance if sprinklers are installed to an appropriate standard. For example, in accordance with Approved Document B, the required fire resistance period for a five storey office where the top floor is no more than 18 m above ground level may be reduced by 30 minutes, if a sprinkler system is provided.
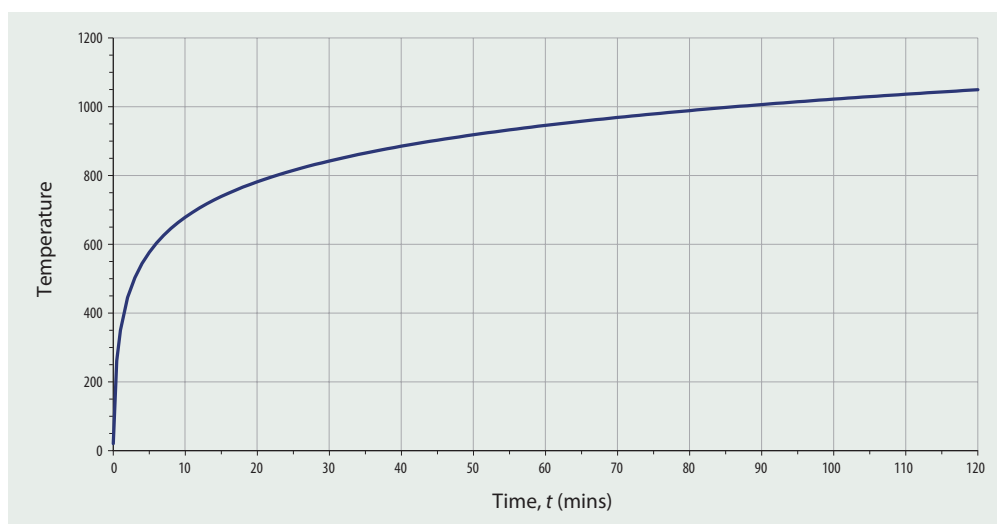
Figure 7.1
Standard temperature-time curve from 1-1-2/3.2.1

7.2 Fire resistance of composite slabs

The structural resistance of composite slabs in fire conditions can be determined from the design methods given in BS EN 1994-1-2. Fire design for composite slabs may be based on the plastic resistance of the slab, allowing for continuity over the supports in accordance with 4-1-2/4.3.1, provided that rotational capacity can be assured. Unlike design at room temperature, the profiled steel sheeting offers little tensile resistance and the reinforcement in the slab therefore becomes the principal reinforcement. BS EN 1994-1-2 expects reinforcing bars to be placed in the ribs of the slab but test evidence in the UK shows that adequate performance can also be achieved using mesh reinforcement without bottom bars, provided that the slab is continuous over at least one support.

7.2.1 Unprotected slabs

4-1-2/4.3.2 provides design guidance for unprotected simply supported or continuous composite slabs. For 30 minutes fire resistance, slabs designed in accordance with Eurocode 4 will be adequate when evaluated against the load bearing criterion, R. The slab thickness will have to be adequate for insulation performance. For composite slabs the integrity criterion may be assumed to be satisfied provided that the rotational capacity of the slab is considered when calculating the bending resistance of the slab.

BS EN 1994-1-2 refers to its Annex D for calculation of the insulation and load bearing capacity of composite slabs. However, the use of Annex D is not recommended by the UK National Annex to BS EN 1994-1-2. The guidance given below on the insulation thickness required for composite slabs on re-entrant and trapezoidal sheeting profiles has been recognised as appropriate for buildings in the UK.

Insulation

For trapezoidal sheeting profiles, the insulation requirements are met by providing a minimum thickness of concrete cover over the steel sheeting. The minimum thickness is between 60 mm and 90 mm for normal weight concrete. Testing has shown that the deeper 80 mm trapezoidal profiles require less concrete cover to achieve the same insulation performance as a 60 mm profile.

CONCRETE TYPE	MINIMUM THICKNESS OF CONCRETE ($h_s - h_p$) FOR A FIRE RESISTANCE PERIOD (MINS) OF:					
	30	60	90	120	180	240
NC ($h_p \geq 80$ mm and average rib depth > 0.4 pitch)	60	60	70	80	115	130
NC (All other cases)	60	70	80	90	115	130
LC (All cases)	50	60	70	80	100	115

Table 7.1 Minimum thickness of concrete over a trapezoidal profile ($h_s - h_p$) for insulation criteria (I)

Note: For structural reasons, a minimum of 50 mm of concrete cover to the steel sheeting is required.

For re-entrant sheeting the insulation thickness is defined in terms of overall slab thickness, h_s.

CONCRETE TYPE	MINIMUM THICKNESS OF CONCRETE FOR A FIRE RESISTANCE PERIOD (MINS) OF:					
	30	60	90	120	180	240
NC	90	90	110	125	150	170
LC	90	90	105	115	135	150

Table 7.2 Minimum slab thickness h_s for re-entrant sheeting profiles for insulation criteria (I)

Note: For structural reasons, a minimum of 50 mm of concrete cover to the steel sheeting is required.

The load bearing resistance of composite slabs may be determined from fire testing or by calculation using 4-1-2/4.3.1, provided that the rotational capacity of the slab is considered in accordance with 2-1-1/5.6.3. In the UK, most manufacturers of sheeting profiles have fire tested their products and are able to offer design tables based on the calculation methods given above for slab solutions which only contain mesh reinforcement.

7.2.2 Protected slabs

Although the use of fire protected slabs is covered by 4-1-2/4.3.3, the use of fire protected composite slabs is not very common in the UK. Usually, this solution is only considered as a remedial measure if an error has been made in the design or construction of a composite slab.

The performance of a fire protection system used in this application should be evaluated in accordance with BS EN 13381-5[34], based on suitable evidence from loaded fire resistance tests.

4-1-2/4.3.3(4) states that the load bearing criterion is fulfilled as long as the temperature of the steel sheet does not exceed 350°C when heated from below by the standard fire. This is the most straightforward critical temperature to use in evaluation of the performance of the fire protection material. However, with suitable test evidence, other critical temperatures could be evaluated and may be demonstrated to be appropriate for some applications.

7.3 Fire resistance of beams

Most composite beams will be conventional downstand beams and the effects of fire on the steel section are little different from those on a non-composite beam. Therefore, in most cases, beams will be fire protected using intumescent coatings, sprayed non-reactive coatings or boarding. In order to determine the correct thickness of fire protection to apply to the structure, the fire resistance period required for the building must be known, and for the member to be protected the critical temperature and section factor must be known also. The critical temperature depends on the combination of actions which the beam is subject to and the variation of the bending resistance with temperature.

Fire design is concerned with the ultimate limit state and for a beam this means the resistance of the beam is the only performance that needs to be evaluated.

The variation of temperature in the steel section may be calculated using the method given in 4-1-2/4.3.4.2.2 for an unprotected or fire protected steel section. The problem in applying this method arises with the protected section, as, when the structure is designed, the materials to be used as fire protection may not be known and it is also unlikely from a commercial point of view that the thermal characteristic of these materials will be in the public domain. It is therefore more desirable to evaluate the

variation of resistance values with steel temperature. If the load to be supported by the beam in fire conditions is known, the critical temperature may be determined as the temperature at which the resistance is equal to the design load. The required thickness of fire protection may then be evaluated based on how long it will take the steel section to heat up to this temperature.

The design load considered to be acting at the time of the fire can, at its simplest, be approximated as 65% of the design load for normal design situations (70% if the floor is used for storage). Lower percentages may be justifiable for a high value of the variable to permanent load ratio and low $\psi_{1,1}$ (see 4-1-2/Figure 2.1).

7.3.1 Critical Temperature

4-1-2/4.3.4.2.3 provides a simple design method that allows the critical temperature of steel-concrete composite sections to be calculated for steel sections up to 500 mm deep supporting concrete slabs not less than 120 mm deep. Using the load level for the composite beam in fire conditions, the critical temperature is determined by the reduction in steel strength according to the following relationships:

For R30 $$0.9\,\eta_{\mathrm{fi,t}} = f_{\mathrm{ay},\theta_{\mathrm{cr}}}/f_{\mathrm{ay}}$$

For R60 and above $$1.0\,\eta_{\mathrm{fi,t}} = f_{\mathrm{ay},\theta_{\mathrm{cr}}}/f_{\mathrm{ay}}$$

where

$\eta_{\mathrm{fi,t}}$ is the load level for fire design

$f_{\mathrm{ay},\theta_{\mathrm{cr}}}$ is the yield strength at the critical temperature

f_{ay} is the yield strength at room temperature.

BS EN 1994-1-2 also provides a method of calculating the bending resistance of composite beams in fire conditions. The method is based on calculating the plastic moment resistance of the cross section in a similar way to that used for room temperature design, taking account of the reduction in material properties with temperature. The shear connection between the steel section and the concrete slab is also calculated using the resistance equations given in BS EN 1994-1-1, taking account of the reduced strength of the shear studs and concrete. To simplify the analysis, BS EN 1994-1-2 permits the designer to assume that the design temperature of the shear studs is 80% of the top flange temperature and that the design temperature of the concrete is 40% of the top flange temperature of the steel section.

This model has been used to produce a table of critical temperatures for composite beams. These critical temperatures are reproduced in Table 7.3 and are presented in terms of load level and degree of shear connection for room temperature design.

DEGREE OF SHEAR CONNECTION	CRITICAL TEMPERATURES (°C) FOR A LOAD LEVEL OF					
	0.7	0.6	0.5	0.4	0.3	0.2
40%	558	588	628	666	698	763
60%	556	586	619	655	691	752
80%	545	577	608	647	684	744
100%	536	567	598	639	679	738

Table 7.3
Critical temperatures
for composite beams
in bending

7.4   Fire protection

Fire protection materials are tested and assessed in accordance with BS EN 13381. For sprays and boards, referred to as non-reactive fire protection, the assessment is undertaken in accordance with the recommendations of BS EN 13381-4[35]. For intumescent coatings, which are classed as reactive fire protection, assessment is in accordance with BS EN 13381-8[36].

The assessment is expressed by manufacturers as tables of minimum thickness, according to section factor, period of fire resistance and steel temperature, known as the assessment temperature. Multi-temperature assessments are usually provided, meaning that protection thicknesses are derived for a range of steel temperatures rather than a single value.

When specifying fire protection, the following information should be included in the specification.

- section factor of the structural member
- period of fire resistance specified by Approved Document B
- critical temperature of the structural member.

Further information on fire protection materials can be found in *Structural fire safety: A handbook for architects and engineers*, P197[37].

Steel beams may be fire protected with intumescent coatings applied off-site prior to frame erection. The application of intumescent coating in workshop conditions prior to delivery to site is now a mature technology, with many coatings being developed specifically for this application. Further information on off-site application of intumescents can be found in *Code of Practice for Off-site Applied Thin Film Intumescent Coatings*[38].

The top surface (where studs are to be welded) must be left uncoated. The heat generated by the stud welding process may cause a little local damage or discoloration of the intumescent coating on the underside of the top flange. However, this will not adversely affect the performance of the intumescent coating and may be ignored, unless repair is required for aesthetic reasons.

7.4.1 Unfilled voids above steel beams

The steel temperatures calculated using the simple thermal analysis tool in 4-1-2/4.3.4.2.2 may be used for beams where at least 85% of the top flange is in contact with the composite slab or the voids are filled with non-combustible material. With re-entrant profiles, the tiny amount of additional exposure of the top flange below the re-entrant voids to fire can be ignored. Trapezoidal profiles will require voids to be filled unless evidence is available to demonstrate satisfactory performance with the voids unfilled.

COMPOSITE COLUMNS

This section briefly introduces composite columns but detailed guidance is outside the scope of this publication.

Figure 8.1
Concrete filled hollow
section composite
columns at the
Wellcome Trust

Design rules for composite columns and composite compression members in structural frames are presented in 4-1-1/6.7. Rules are provided for composite H sections, either fully or 'partially encased' (web infill only), and for concrete filled hollow sections. Typical cross sections are shown in Figure 8.2. Composite columns requiring formwork during execution tend not to be viewed as cost-effective in the UK.

Concrete filled hollow section compression members need no formwork and they can use material more efficiently than an equivalent H section. Concrete infill adds compression resistance but the gain in fire resistance may be at least as valuable, especially if it permits the column to be left unprotected or only lightly protected. Infill concrete retains free water which in other situations would dry out; its latent heat of evaporation significantly delays temperature rise.
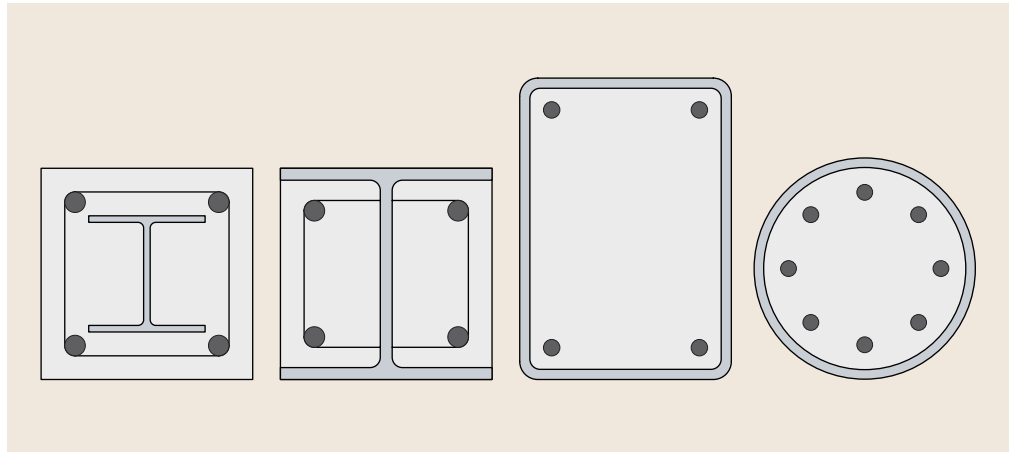
*Figure 8.2
Typical composite
compression member
cross sections*

Rectangular and circular hollow sections can be used. Rectangular sections have the advantage of a flat face for end plate beam-to-column connections (using Flowdrill or Hollo-bolt connections), though ordinary fin plates can be employed with either shape.

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CREDITS

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	81	Gibbs Building Wellcome Trust Photograph: Andrew Orton
	88	Courtesy of SMD

APPENDIX A - MINIMUM DEGREE OF SHEAR CONNECTION

The Figures in the Appendix provide a graphical representation of the various minimum degree of shear connection rules given in Eurocode 4 and UK NCCI material.

Figure A.1 and Figure A.2 are reproduced from 4-1-1/6.6.1.2(1) and 4-1-1/6.6.1.2(3) respectively.

Figure A.3, Figure A.4 and Figure A.5, are based on additional rules for minimum degree of shear connection given in UK NCCI as discussed in Section 4.4.3.
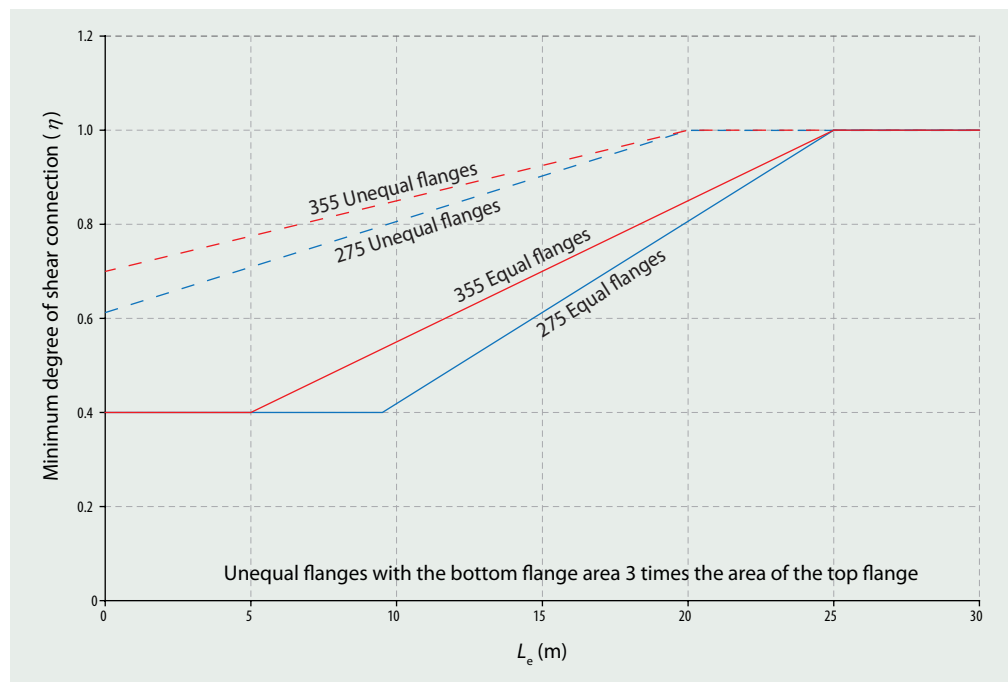
Figure A.1 Minimum degree of shear connection using 4-1-1/6.6.1.2(1)
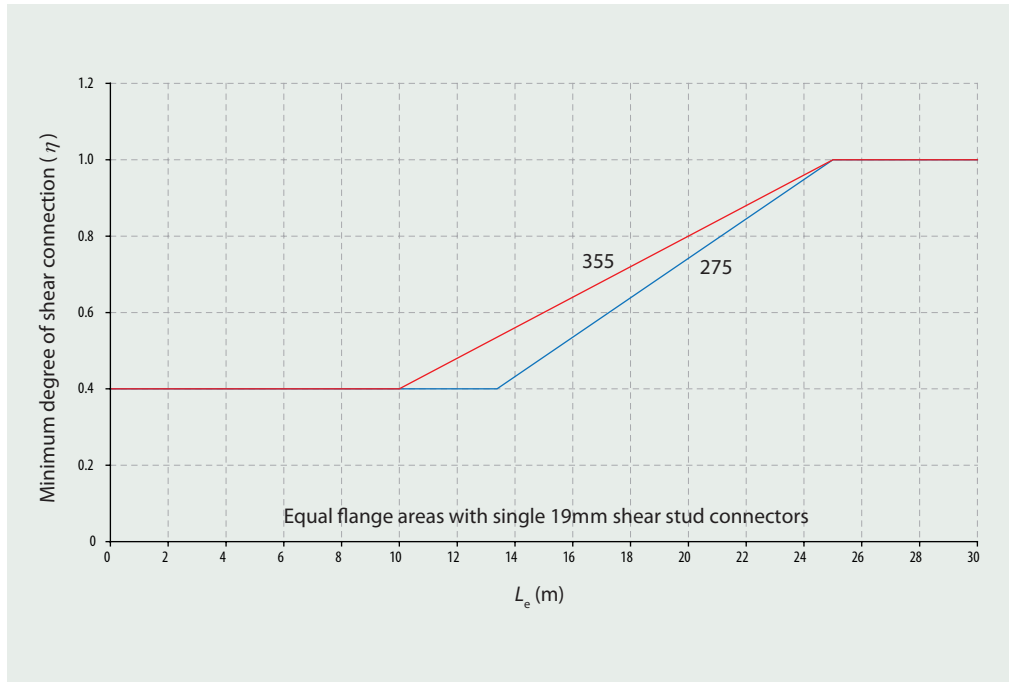
Figure A.2
Minimum degree of
shear connection
using 4-1-1.6.6.1.2(3)

Figure A.2 chart:

Y-axis: Minimum degree of shear connection (η), 0 to 1.2

X-axis: L_e (m), 0 to 30

Curves labelled 355 and 275

Equal flange areas with single 19mm shear stud connectors
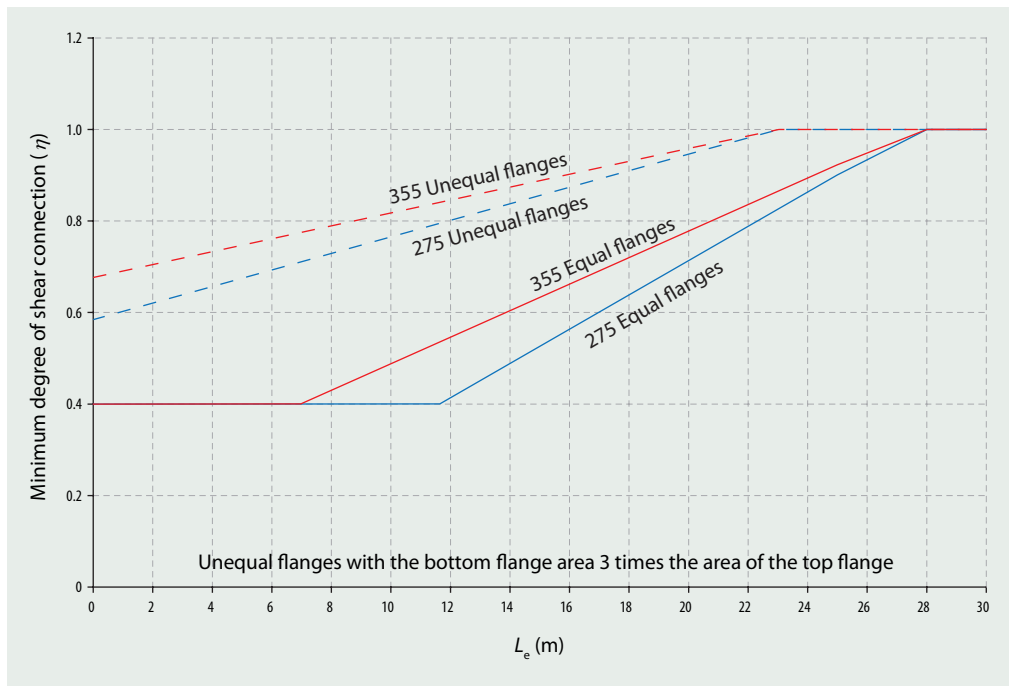
Figure A.3
Minimum degree of
shear connection
using the UK NCCI
General limits for
beams unpropped
during execution

Figure A.3 chart:

Y-axis: Minimum degree of shear connection (η), 0 to 1.2

X-axis: L_e (m), 0 to 30

Curves labelled: 355 Unequal flanges, 275 Unequal flanges, 355 Equal flanges, 275 Equal flanges

Unequal flanges with the bottom flange area 3 times the area of the top flange

Figure A.4 Minimum degree of shear connection using the UK NCCI Trapezoidal steel sheeting spanning transverse to supporting beam unpropped during execution
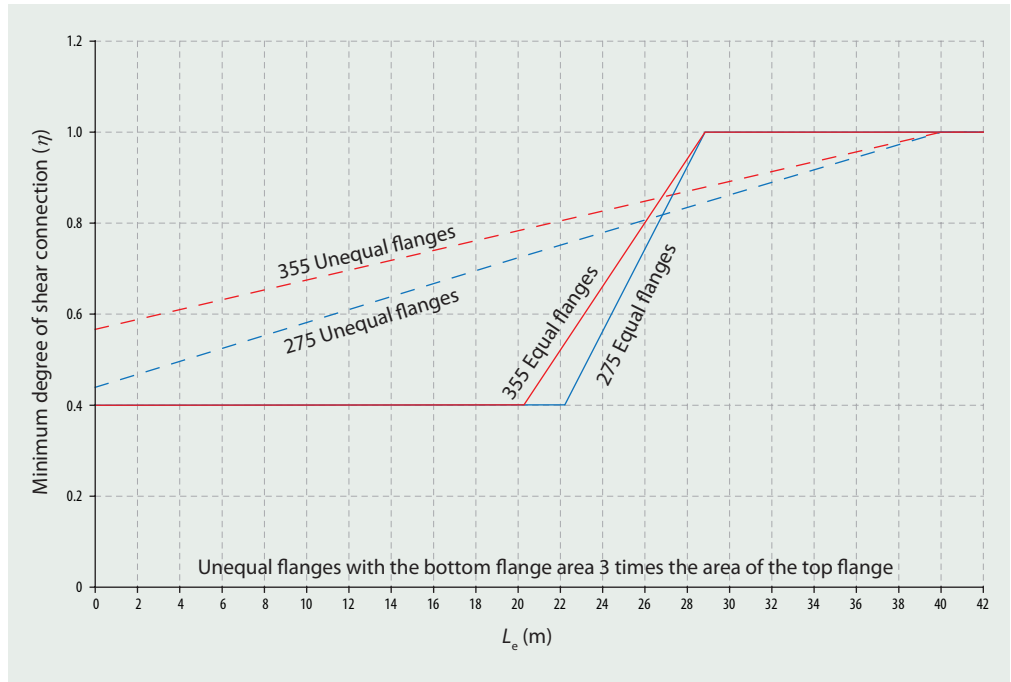
Figure A.5 Minimum degree of shear connection using the UK NCCI Trapezoidal steel sheeting spanning transverse to supporting beam propped during execution
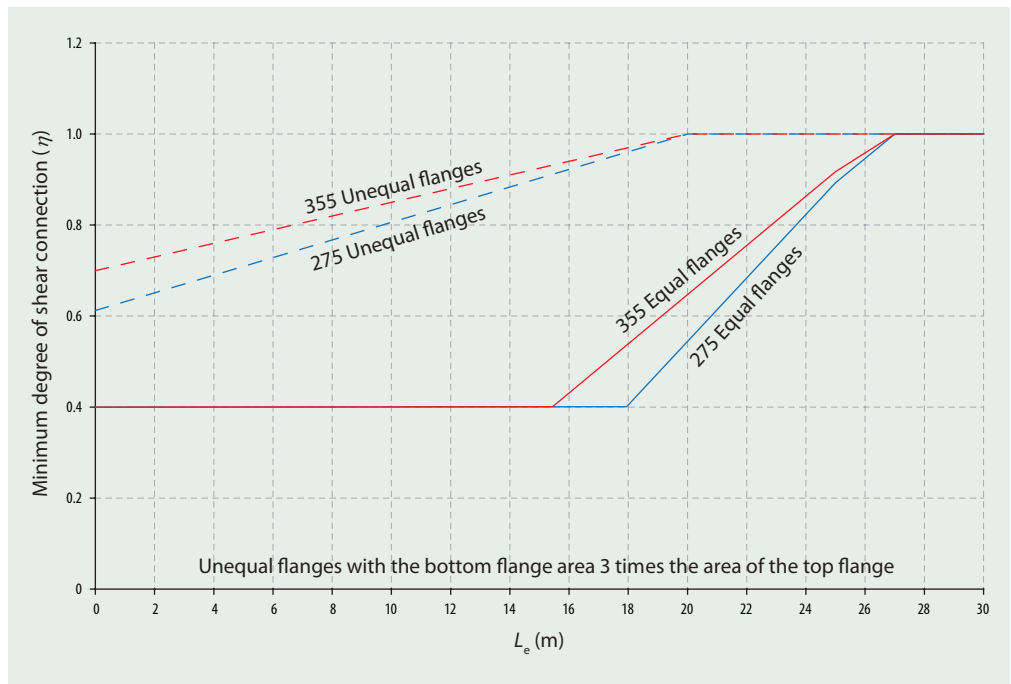
APPENDIX B - WORKED EXAMPLE

The worked example shows the design of the composite beams supporting a composite floor slab. The composite floor slab has been chosen to suit a span of 3 m and to achieve a 60 minute fire resistance. The fire design of the beams is not covered in the example: it is assumed that the beams will be protected.

The design aspects covered in this example are:

B.1 Configuration & dimensions

PLAN

Composite beam
Composite beam
Composite beam
Composite beam

3 m
3 m
3 m

9 m

A

A

SECTION A-A

Detail 1

9000 mm

DETAIL 1

170 mm
130 mm
A 193 MESH

70 mm
55 mm
15 mm
60 mm

180 mm
120 mm

Tata Steel
CF60 profile
metal deck

Figure B.1
Slab details

B.2   Floor details

Beam Span	L	= 9.0 m
Beam spacing	b	= 3.0 m
Slab depth	h_s	= 130.0 mm
Tata Steel data sheet Profiled steel sheeting		Tata Steel CF60 (0.9 mm thick)
Sheeting profile height to shoulder	h_p	= 60.0 mm
h_c based on overall Overall height of sheeting profile	h_d	= 75.0 mm
height of profile Depth of concrete above profile	h_c	= 55.0 mm

B.2.1   Shear connectors

Connector diameter	d	= 19 mm
Overall (as-welded) height	h_{sc}	= 95 mm
Ultimate tensile strength	f_u	= 450 N/mm²

B.2.2   Concrete

Normal weight concrete grade, C25/30

2-1-1/Table 3.1 Characteristic cylinder strength	f_{ck}	= 25 N/mm²
Characteristic cube strength	$f_{ck,cube}$	= 30 N/mm²
Secant modulus of elasticity of concrete	E_{cm}	= 31 kN/mm²
Tata Steel data sheet Concrete volume		= 0.097 m³/m²

B.2.3   Reinforcement

Reinforcement bar diameter		= 7 mm
Spacing of bars		= 200 mm
Area of steel reinforcement per unit width		= 193 mm²/m
Self-weight of the mesh per unit area		= 3.03 kg/m²
Yield strength	f_{sd}	= 500 N/mm²

B.3   Actions

B.3.1   Construction stage

Permanent actions

Sheeting self-weight	$g_{k,1}$	= 0.10 kN/m²
Allowance for beam self-weight	$g_{k,2}$	= 1.0 kN/m

3.2   *Note: The allowance for self-weight of reinforcement of 1 kN/m³ given in 1-1-1/, Table A.1 is appropriate for reinforced concrete but not for composite floors, which have a relatively light mesh.*

Allowance for mesh $= 3.03 \times 9.81 \times 10^{-3} = 0.03$ kN/m²

Variable actions

Note: The weight of fresh concrete is to be treated as a variable action, which means that the partial factor γ_Q is applied, rather than γ_G.

Slab (0.097 m³/m²) @ 25 kN/m³ [24 + 1 (wet concrete)]

$$q_{k,1} = 2.43 \text{ kN/m}^2$$

Construction loads

3.2 *Note:* As discussed in Section 3.2, SCI recommends that designers take advantage of 1-1-6/N.A.2.13 to use "values of Q_{ca} and Q_{cc} determined for the individual project". Thus $Q_{k,1a} = 0$ and $q_{k,1b} = 0.75 \text{ kN/m}^2$ for the design of beams.
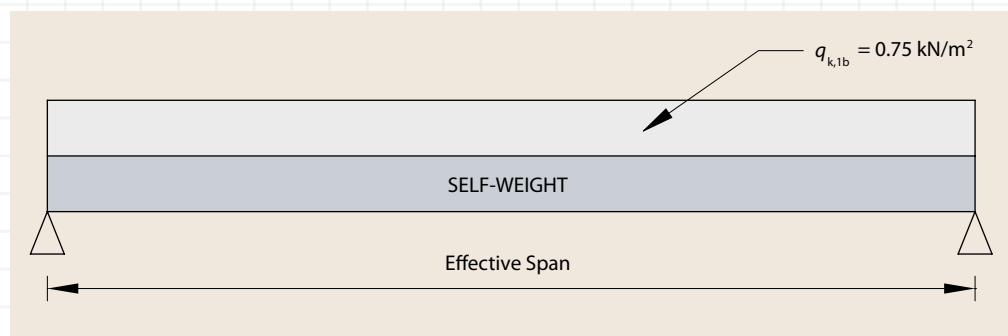
Figure B.2 Variable action for construction

4.1 ## B.3.2 Composite stage

Permanent actions

1-1-1/Table A.1	Slab (0.097 m³/m² @ 24 kN/m³) dry concrete	= 2.33 kN/m²
	Sheeting self-weight	= 0.10 kN/m²
	Allowance for mesh	= 0.03 kN/m²
	Total $g_{k,1}$	= 2.46 kN/m²
	Allowance for beam self-weight $g_{k,2}$	= 1.0 kN/m
	Ceiling and services $g_{k,3}$	= 0.50 kN/m²
	Raised floor $g_{k,4}$	= 0.35 kN/m²

Variable actions

1-1-1/6.3.1.2(8) As the composite floor allows a lateral distribution of loads, a uniformly distributed load can be added to the imposed variable floor load to allow for movable partitions.

For movable partitions with:

Self-weight > 1 kN/m and ≤ 2.0 kN/m, allowance	= 0.8 kN/m²
Imposed floor load (office occupancy)	= 4.0 kN/m²
Total imposed occupancy floor partition loads $q_{k,1}$	= 4.8 kN/m²

B.3.3 Partial factors for actions

BS 1990 Table NA.A1.2(B)

Partial factor for permanent actions	γ_G	= 1.35
Partial factor for variable actions	γ_Q	= 1.50

Reduction factor $\qquad \xi \qquad = 0.925$

1-1-6/A.1.1(1) $\quad \psi_{0,1} = 1.0$ (construction stage) $\qquad \psi_{0,1} = 0.7$ (composite stage)

B.4 Design values for combined actions

Use the more onerous of:

BS EN 1990
Eq. (6.10a)
$$\sum_{j\geq1}\gamma_{G,j}G_{k,g} + \gamma_p P + \gamma_{Q,1}\psi_{0,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

BS EN 1990
Eq. (6.10b)
$$\sum_{j\geq1}\xi_j\gamma_{G,j}G_{k,g} + \gamma_p P + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

since $\gamma_p P, \gamma_{Q,1}\psi_{0,1}Q_{k,1}, \gamma_{Q,1}Q_{k,1}$ and $\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$ are all equal to zero.

Eq. (6.10a) reduces to:

$$\sum_{j\geq1}\gamma_{G,j}G_{k,g} + \gamma_{Q,1}\psi_{0,1}Q_{k,1} = 1.35G_{k,j} + (1.5)(1.0)(Q_{k,1})$$

Eq. (6.10b) reduces to:

$$\sum_{j\geq1}\xi\gamma_{G,j}G_{k,g} + \gamma_{Q,1}Q_{k,1} = 0.925(1.35)G_{k,j} + 1.5Q_{k,1}$$

Eq. (6.10a) will result in a higher design load for construction stage.

Eq. (6.10b) will result in a higher design load for composite stage, as $\psi_{0,1} = 0.7$.

B.4.1 Construction stage at ULS

$$
\begin{aligned}
F_{d,1} \quad &= 1.35 \times 1.0 + (1.35(0.1 + 0.03) + 1.5(2.43 + 0.75)) \times 3 \\
&= 16.19 \text{ kN/m}
\end{aligned}
$$

B.4.2 Composite stage at ULS

$$
\begin{aligned}
F_d \quad &= \xi\gamma_G g_{k,2} + \left[\xi\gamma_G\left(g_{k,1} + g_{k,3} + g_{k,4}\right) + \gamma_Q q_{k,1}\right] \times b \\
&= 0.925\times(1.35)(1.0) + \left[0.925\times1.35\times(2.46+0.5+0.35) + (1.5)(4.8)\right]\times3 \\
&= 35.25 \text{ kN/m}
\end{aligned}
$$

6.1 ## B.4.3 Composite stage at SLS

The characteristic load combination will be used. Therefore the actions for calculation of deflections are:

Permanent actions applied to steel beam:

self weight of the slab + mesh + sheeting + steel section

$$g_1 \quad = (2.33 + 0.1 + 0.03) \times 3 + 1.0 = 8.38 \text{ kN/m}$$

Permanent actions applied to composite beam:

ceiling and services + raised floor

$g_2 \qquad = (0.5 + 0.35\ ) \times 3 = 2.55 \text{ kN/m}$

Variable actions applied to composite beam:

$q_1 \qquad = q_{k,1} = 4.8 \times 3 = 14.4 \text{ kN/m}$

B.5 Design bending moments & shear forces
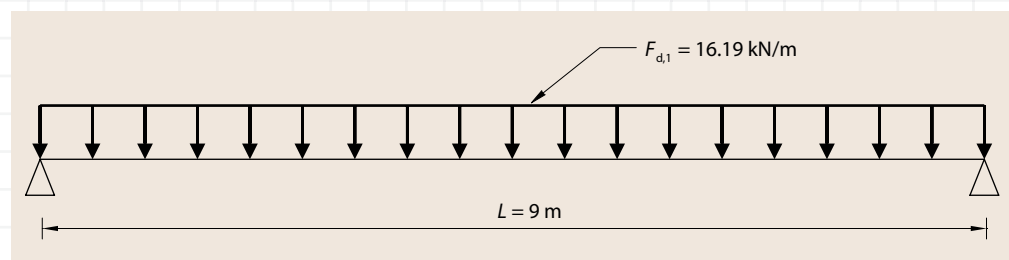
B.5.1 Construction stage

Figure B.3

$$M_{\text{Ed}} \quad = \frac{F_{\text{d,1}}L^2}{8} = \frac{(16.19)(9)^2}{8} = 164 \text{ kNm}$$

$$V_{\text{Ed}} \quad = \frac{F_{\text{d,1}}L}{2} = \frac{(16.19)(9)}{2} = 72.9 \text{ kN}$$
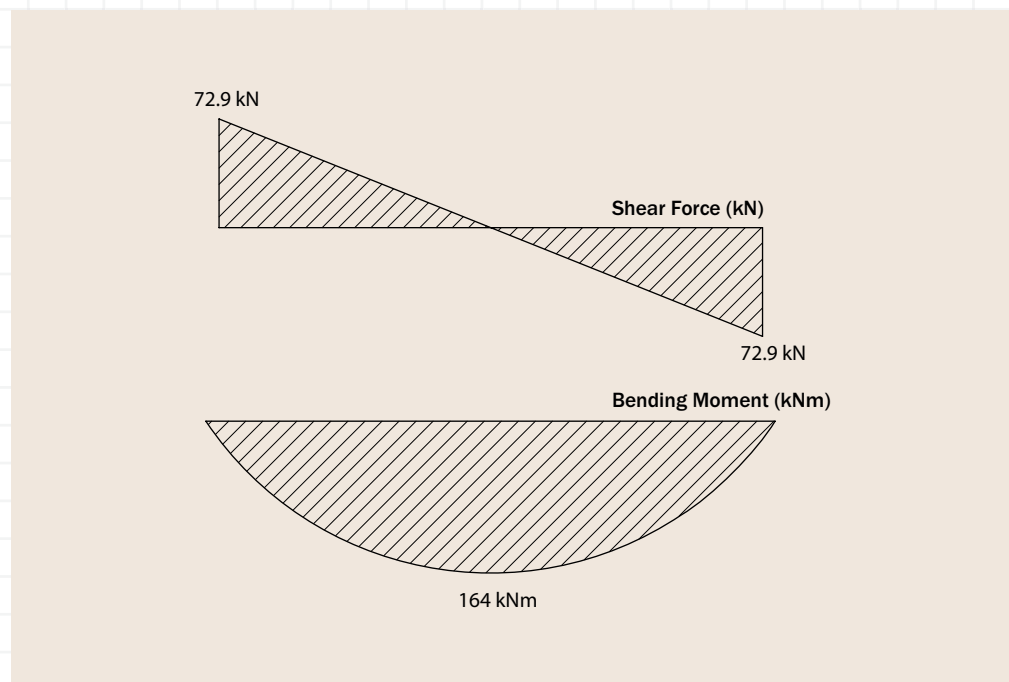
*Figure B.4
Effect of
actions at the
construction stage*

B.5.2 Composite stage

Maximum design bending moment is:

$$M_{Ed,comp} = \frac{F_d L^2}{8} = \frac{(35.25)(9)^2}{8} = 357 \text{ kNm}$$

Maximum design shear force is:

$$V_{Ed,comp} = \frac{F_d L}{2} = \frac{(35.25)(9)}{2} = 159 \text{ kN}$$
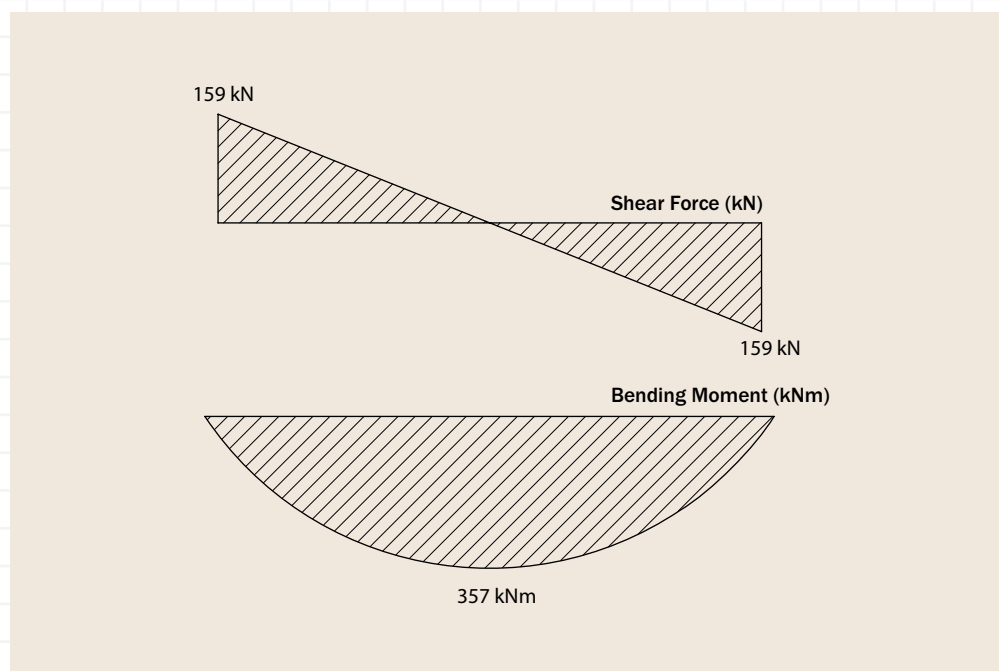
Figure B.5
Effect of actions at
composite stage

B.6 Section properties

For a 406 × 140 × 46 UKB in S275 steel

From section property tables:

Depth	h_a	= 403.2 mm
Width	b	= 142.2 mm
Web thickness	t_w	= 6.8 mm
Flange thickness	t_f	= 11.2 mm
Root radius	r	= 10.2 mm
Depth between fillets	d	= 360.4 mm
Second moment of area y axis	I_y	= 15,700 cm⁴
Elastic modulus y axis	$W_{el,y}$	= 778 cm³
Plastic modulus y axis	$W_{pl,y}$	= 888 cm³
Area	A_a	= 58.6 cm²
3-1-1/3.2.6(1) Modulus of elasticity	E	= 210 kN/mm²

For buildings that will be built in the UK, the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given the lowest nominal value should be used.

For S275 steel and $t \leq 16$ mm

$$f_y \qquad = 275 \text{ N/mm}^2$$

B.7   Cross section classification

$\varepsilon \qquad = \sqrt{\dfrac{235}{f_y}} = \sqrt{\dfrac{235}{275}} = 0.92$

Outstand of compression flange

$$c \qquad = \frac{b - t_w - 2r}{2} = \frac{142.2 - 6.8 - 2(10.2)}{2} = 57.5 \text{ mm}$$

$$\frac{c}{t_f} \qquad = \frac{57.5}{11.2} = 5.13$$

The limiting value for Class 1 is $\dfrac{c}{t_f} \leq 9\varepsilon = 9 \times 0.92 = 8.32$

5.13 $\qquad < 8.32$

Therefore the flange in compression is Class 1

Web subject to bending

$$c \qquad = d = 360.4 \text{ mm}$$

$$\frac{c}{t_w} \qquad = \frac{360.4}{6.8} = 53$$

The limiting value for Class 1 is $\dfrac{c}{t_w} \leq 72\varepsilon$

53 $\qquad \leq 72 \times 0.92$

53 $\qquad \leq 66.2$

Therefore the web in bending is Class 1.

Therefore the cross section in bending at the construction stage is Class 1. At the composite stage, the cross section will also be Class 1.

B.8 Partial factors for resistance

3-1-1/NA.2.15 **Steel section**

$$\gamma_{M0} = 1.00$$
$$\gamma_{M1} = 1.00$$
$$\gamma_{M2} = 1.10$$

Shear connector

For the resistance of a shear connector, the UK NA to 4-1-1/2.3 adopts the recommended value of $\gamma_v = 1.25$ given in 4-1-1/6.6.3.1:

> *"unless stud resistances given in non-contradictory complementary information would justify the use of an alternative value"*

4-1-1/6.6.3.1 Here, take $\gamma_v = 1.25$

Concrete

2-1-1/Table NA.1 For persistent and transient design situations

$$\gamma_c = 1.5$$

Reinforcement

2-1-1/Table NA.1 For persistent and transient design situations

$$\gamma_s = 1.15$$

B.9 Design resistance for the construction stage

3.4 ## B.9.1 Cross-sectional resistance of the steel beam

Shear buckling

4-1-1/6.2.2.3(1) The shear buckling resistance of an un-encased web should be verified using Section 5 of 3-1-5 if:

3-1-5/5.1(2)
$$\frac{h_w}{t_w} > \frac{72\varepsilon}{\eta}$$

3-1-5/NA.2.4 $\eta = 1.0$

$$h_w = h_a - 2t_f = 403.2 - 2(11.2) = 380.8 \text{ mm}$$

$$\frac{h_w}{t_w} = \frac{380.8}{6.8} = 56\varepsilon = \sqrt{\frac{235}{f_y}}$$

$$72\frac{\varepsilon}{\eta} = 72 \times \frac{0.92}{1.0} = 66.2$$

$$56 < 66.2$$

Therefore shear buckling of the web does not need to be verified.

Vertical shear resistance

Verify that:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0$$

For plastic design, $V_{c,Rd}$ is the design plastic shear resistance ($V_{pl,Rd}$)

$$V_{c,Rd} = V_{pl,a,Rd} = \frac{A_v(f_y / \sqrt{3})}{\gamma_{M0}}$$

A_v is the shear area and is determined as follows for rolled I and H sections with the load applied parallel to the web.

$$
\begin{aligned}
A_v &= A - 2bt_f + t_f(t_w + 2r) \text{ but not less than } \eta h_w t_w \\
&= 58.6 \times 10^2 - (2 \times 142.2 \times 11.2) + 11.2(6.8 + 2(10.2)) \\
&= 2979 \text{ mm}^2
\end{aligned}
$$

$$\eta h_w t_w = 1.0 \times ((380.8) \times 6.8) = 2589 \text{ mm}^2 < 2979 \text{ mm}^2$$

Therefore,

$$A_v = 2979 \text{ mm}^2$$

Plastic shear resistance

$$V_{pl,a,Rd} = \frac{A_v(f_y / \sqrt{3})}{\gamma_{M0}} = \frac{(2979) \times \left(\dfrac{275}{\sqrt{3}}\right) \times 10^{-3}}{1.0} = 473 \text{ kN}$$

Maximum design shear for the construction stage is $V_{Ed} = 72.9$ kN

$$\frac{V_{Ed}}{V_{pl,a,Rd}} = \frac{72.9}{473} = 0.15 < 1.0$$

Therefore the shear resistance of the cross-section is adequate.

B.9.2 Bending resistance

Verify that:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0$$

As $\dfrac{V_{pl,a,Rd}}{2} = \dfrac{473}{2} = 237.5 \text{ kN} > V_{Ed}$ (72.9 kN).

No reduction in the bending moment resistance of the steel section need be accounted for at any point along the beam.

The design resistance to bending moment for Class 1 and 2 cross sections is:

$$M_{c,Rd} = M_{pl,a,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{888 \times 10^3 (275)}{1.0} \times 10^{-6} = 244 \text{ kNm}$$

$$\frac{M_{y,Ed}}{M_{c,Rd}} = \frac{164}{244} = 0.67 < 1.0$$

Therefore the bending moment resistance is adequate.

B.9.3 Buckling resistance

The steel sheeting provides continuous restraint to the top flange of the steel beam, so the beam is not susceptible to lateral torsional buckling.

4.4 B.10 Shear connection

B.10.1 Design resistance of shear connectors

Shear connector in a solid slab

The design resistance of a single headed shear connector in a solid concrete slab, automatically welded in accordance with BS EN 14555, should be determined as the smaller of:

$$P_{Rd} = \frac{0.8 \times f_u \times \pi \times \frac{d^2}{4}}{\gamma_V}$$

$$P_{Rd} = \frac{0.29 \times \alpha \times d^2 \sqrt{f_{ck} \times E_{cm}}}{\gamma_V}$$

where

$$\alpha = 1.0 \quad \text{as} \quad \frac{h_{sc}}{d} = \frac{95}{19} > 4$$

$$P_{Rd} = \frac{0.8 \times 450 \times \pi \times \frac{19^2}{4}}{1.25} = 81.7 \text{ kN}$$

$$P_{Rd} = \frac{0.29 \times 1.0 \times 19^2 \times \sqrt{25 \times 31 \times 10^3}}{1.25} \times 10^{-3} = 73.7 \text{ kN}$$

4.4.2 Shear connectors in profiled steel sheeting

For profiled sheeting with ribs running transverse to the supporting beams $P_{Rd,solid}$ should be multiplied by the following reduction factor.

$$k_t = \frac{0.7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right)$$

Note: As discussed in Section 4.4.2,the method outlined in NCCI: Resistance of headed stud shear connectors in transverse sheeting PN001a-GB will be followed, i.e. the application of a k_{mod} factor to the resistance.

For a single stud per trough with the mesh positioned above the head of the studs.

k_{mod} = 1.0 (Table 2.1 NCCI PN001a-GB)

Figure B.6 Dimensions that influence calculation of stud resistance
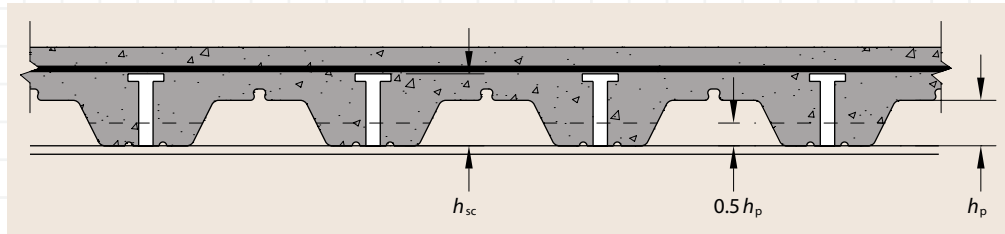
b_0 = 145 mm

h_{sc} = 95 mm

h_p = 60 mm

n_r = 1, for one shear connector per rib

k_t = $\dfrac{0.7}{\sqrt{1.0}} \times \dfrac{145}{60} \times \left(\dfrac{95}{60} - 1 \right) = 0.99$

But k_t should not be taken greater than the appropriate value $k_{t,max}$ given in 4-1-1/Table 6.2.

For shear connectors welded through the profiled sheeting, $t \leq 1.0$ mm and $n_r = 1$.

$k_{t,max}$ = 0.85

Therefore,

k_t = 0.85

Hence, the design resistance per shear connector in a rib where there is one connecter per rib is:

P_{Rd} = $k_t P_{Rd,solid}$ = 0.85 × 73.7 = 62.6 kN

And the design resistance per rib is:

$n_r P_{Rd}$ = 1 × 62.6 = 62.6 kN

For illustration purposes, this example also considers the case of two shear connectors per rib $n_r = 2$

k_{mod} = 0.7 (Mesh above the heads of studs)

Eq (6.23) k_t $= \dfrac{0.7}{\sqrt{1.0}} \times \dfrac{145}{60} \times \left(\dfrac{95}{60} - 1\right) = 0.70$

Table 6.2 For shear connectors welded through the profiled sheeting, $t \leq 1.0$ mm and $n_r = 2$.

$k_{t,max}$ $= 0.7$

As for 1 stud earlier:

P_{Rd} $= k_{mod} \times k_t \times P_{Rd,solid} = 0.7 \times 0.7 \times 73.7 = 36.1$ kN per stud

The design resistance per rib is:

$n_r P_{Rd}$ $= 2 \times 36.1 = 72.2$ kN

4.4.3 *Degree of shear connection*

For composite beams in buildings, the headed shear connectors may be considered as ductile when the minimum degree of shear connection given in 4-1-1/6.6.1.2 is achieved.

4-1-1/6.6.1.2(1) For headed shear connectors with:

h_{sc} $\geq 4d$ and 16 mm $\leq d \leq 25$ mm

The degree of shear connection may be determined from:

η $= \dfrac{N_c}{N_{c,f}}$

where

N_c is the reduced value of the compressive force in the concrete flange (i.e. the force transferred by the shear connectors)

$N_{c,f}$ is the compressive force in the concrete flange at full shear connection (i.e. the minimum of the axial resistance of the concrete and the axial resistance of the steel).

For steel sections with equal flanges and $L_e < 25$ m

4-1-1/Eq (6.12) η $\geq 1 - \left(\dfrac{355}{f_y}\right)(0.75 - 0.03 L_e)$, $\eta \geq 0.4$

where

L_e is the distance between points of zero bending moment.

Therefore for a simply supported beam:

L_e $= L = 9$ m

$$\eta \geq 1 - \left(\frac{355}{275}\right)(0.75 - 0.03 \times 9.0) = 0.38 \qquad \text{Therefore,} \quad \eta = 0.40$$

Degree of shear connection present

To determine the degree of shear connection present in the beam first the axial resistances of the steel and concrete are required ($N_{pl,a}$ and $N_{c,f}$ respectively).
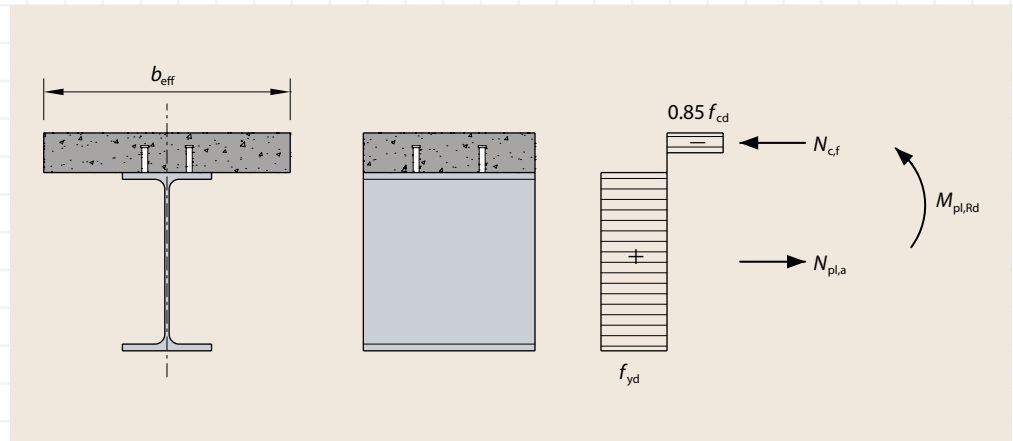
4.2 Determine the effective width of the concrete flange

At the mid-span the effective width of the concrete flange is determined from:

$$b_{eff} = b_0 + \sum b_{ei}$$

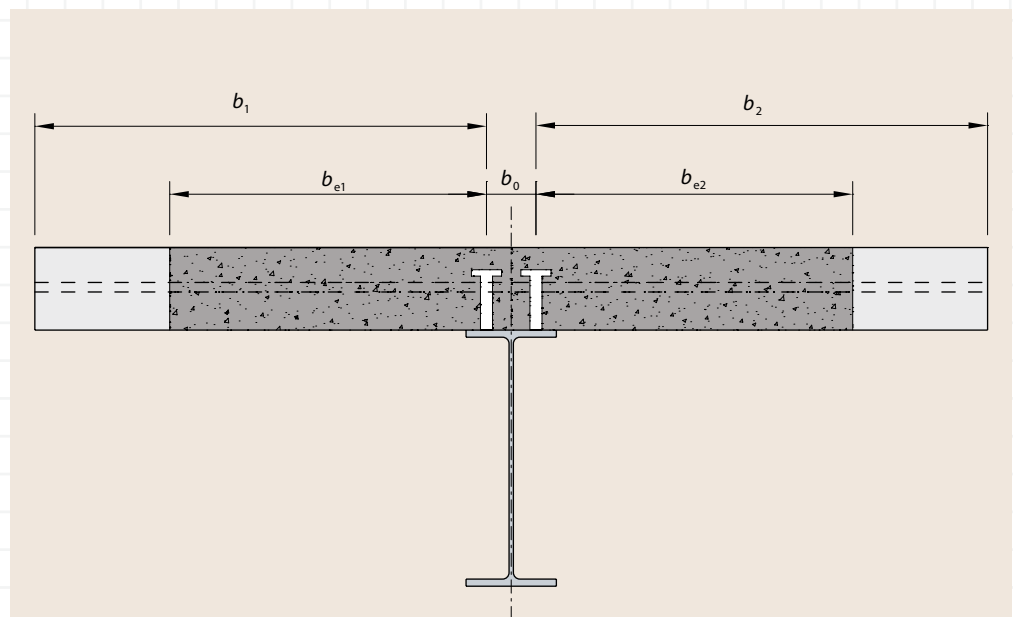For $n_r = 1$, $b_0 = 0$ mm, $n_r = 2$, $b_0 = 80$ mm (assumed shear stud spacing)

$$b_{ei} = \frac{L_e}{8}, \text{ but not greater than } b_i$$

where

L_e is the distance between points of zero bending moment.

Therefore for a simply supported beam:

L_e = 9 m

b_i is the distance from the outside shear connector to a point mid-way between adjacent webs, therefore:

For $n_r = 1$, $b_1 = b_2$ = 1.5 m
For $n_r = 2$, $b_1 = b_2$ = 1.46 m

$$b_{e1} = b_{e2} = \frac{L_e}{8} = \frac{9}{8} = 1.125 \text{ m}$$

Therefore,

For $n_r = 1$, $b_{e1} = b_{e2}$ = 1.125 m
For $n_r = 2$, $b_{e1} = b_{e2}$ = 1.125 m

Hence at the mid-span the effective width of the concrete flange is:

For $n_r = 1$, $b_{eff} = b_0 + b_{e1} + b_{e2}$ $= 0 + (2 \times 1.125)$ = 2.25 m
For $n_r = 2$, $b_{eff} = b_0 + b_{e1} + b_{e2}$ $= 0.08 + (2 \times 1.125)$ = 2.33 m

Compressive resistance of the concrete flange

4-1-1/2.4.1.2(2)P   The design strength of the concrete is

$$f_{cd} = \frac{f_{ck}}{\gamma_c}$$

For persistent and transient design situations the design compressive strength of the concrete is:

$$f_{cd} = \frac{25}{1.5} = 16.7 \text{ N/mm}^2$$

For compressive resistance of the concrete flange the depth of concrete considered is that above the top of the re-entrant top flange stiffener present. The CF60 profile has a 15 mm deep re-entrant stiffener above the top flange making the overall profile depth, $h_d = 75$ mm.

Compressive resistance of the concrete flange is:

For $n_r = 1$,

$$N_{c,f} = 0.85 f_{cd} b_{eff} h_c = 0.85 \times 16.7 \times 2250 \times 55 \times 10^{-3} = 1757 \text{ kN}$$

For $n_r = 2$,

$$N_{c,f} = 0.85 f_{cd} b_{eff} h_c = 0.85 \times 16.7 \times 2330 \times 55 \times 10^{-3} = 1819 \text{ kN}$$

Tensile resistance of the steel member

$$N_{pl,a} = f_y A_a = 275 \times 58.6 \times 10^2 \times 10^{-3} = 1612 \text{ kN}$$

Compressive force in the concrete flange

The compressive force in the concrete at full shear connection is the lesser of $N_{c,f}$ and $N_{pl,a}$. Hence:

For $n_r = 1$ $N_c = 1612$ kN
For $n_r = 2$ $N_c = 1612$ kN

Resistance of the shear connectors

n is the number of shear connectors present to the point of maximum bending moment.

In this example there are 15 ribs available for positioning shear connectors, per half span (i.e. $9/(2 \times 0.3)$).

For $n_r = 1$, $n = 15$
For $n_r = 2$, $n = 30$

Where there is less than full shear connection, the reduced value of the compressive force in the concrete flange, N_c, is the combined resistance of the shear connectors in each half-span. Thus,

For $n_r = 1$, $N_c = n \times P_{Rd} = 15 \times 62.6 = 939$ kN
For $n_r = 2$, $N_c = n \times P_{Rd} = 30 \times 36.1 = 1083$ kN

Shear connection present

The degree of shear connection, η, is the ratio of the reduced value of the compressive force, N_c, to the concrete compressive force at full shear connection, $N_{c,f}$.

For $n_r = 1$, $\eta = \dfrac{N_c}{N_{c,f}} = \dfrac{939}{1612} = 0.58 > 0.40$

For $n_r = 2$, $\eta = \dfrac{N_c}{N_{c,f}} = \dfrac{1083}{1612} = 0.67 > 0.40$

B.11 Design resistances of the cross-section for the composite stage

As noted for the construction stage, the top flange is restrained laterally and therefore only cross-sectional resistances need to be verified.

4.3 ## B.11.1 Vertical shear resistance

Shear buckling

As noted for the construction stage, the shear buckling resistance of the web does not need to be verified.

Plastic resistance to vertical shear

4-1-1/6.2.2.2(1) The resistance to vertical shear ($V_{pl,Rd}$) should be taken as the resistance of the structural steel section ($V_{pl,a,Rd}$).

Appendix B.9.1 $V_{pl,a,Rd}$ = 473 kN

Appendix B.5.2 Maximum design shear for the composite stage is V_{Ed} = 159 kN.

$$\frac{V_{Ed}}{V_{pl,a,Rd}} = \frac{159}{473} = 0.34 \ < 1.0$$

Therefore the vertical shear resistance of the section is adequate.

4.2.3 ## B.11.2 Resistance to bending

4-1-1/6.2.2.4(1) As $\dfrac{V_{pl,a,Rd}}{2} = \dfrac{473}{2} = 237$ kN $> V_{Ed}$ (159 kN).

No reduction in the bending resistance of the steel section need be accounted for at any point along the beam.

Resistance with one shear connector per trough ($n_r = 1$)

For one connector per trough, rigid plastic theory from 4-1-1/(6.2.1.2) may be used.

With partial shear connection, the axial force in the concrete flange N_c is less than $N_{pl,a}$ (939 kN < 1612 kN). Therefore, the plastic neutral axis lies within the steel section.

Assuming that the plastic neutral axis lies a distance x_{pl} below the top of the top flange of the section, where

$$x_{pl} = \frac{\left(N_{pl,a} - N_c\right)}{2 f_y b} = \frac{(1612 - 939)}{2 \times 275 \times 142.2} \times 10^3$$

$$= 8.61 \text{ mm} < t_f = 11.2 \text{ mm}$$

∴ PNA lies in the top flange.

For the PNA in the top flange, the forces are as shown below.
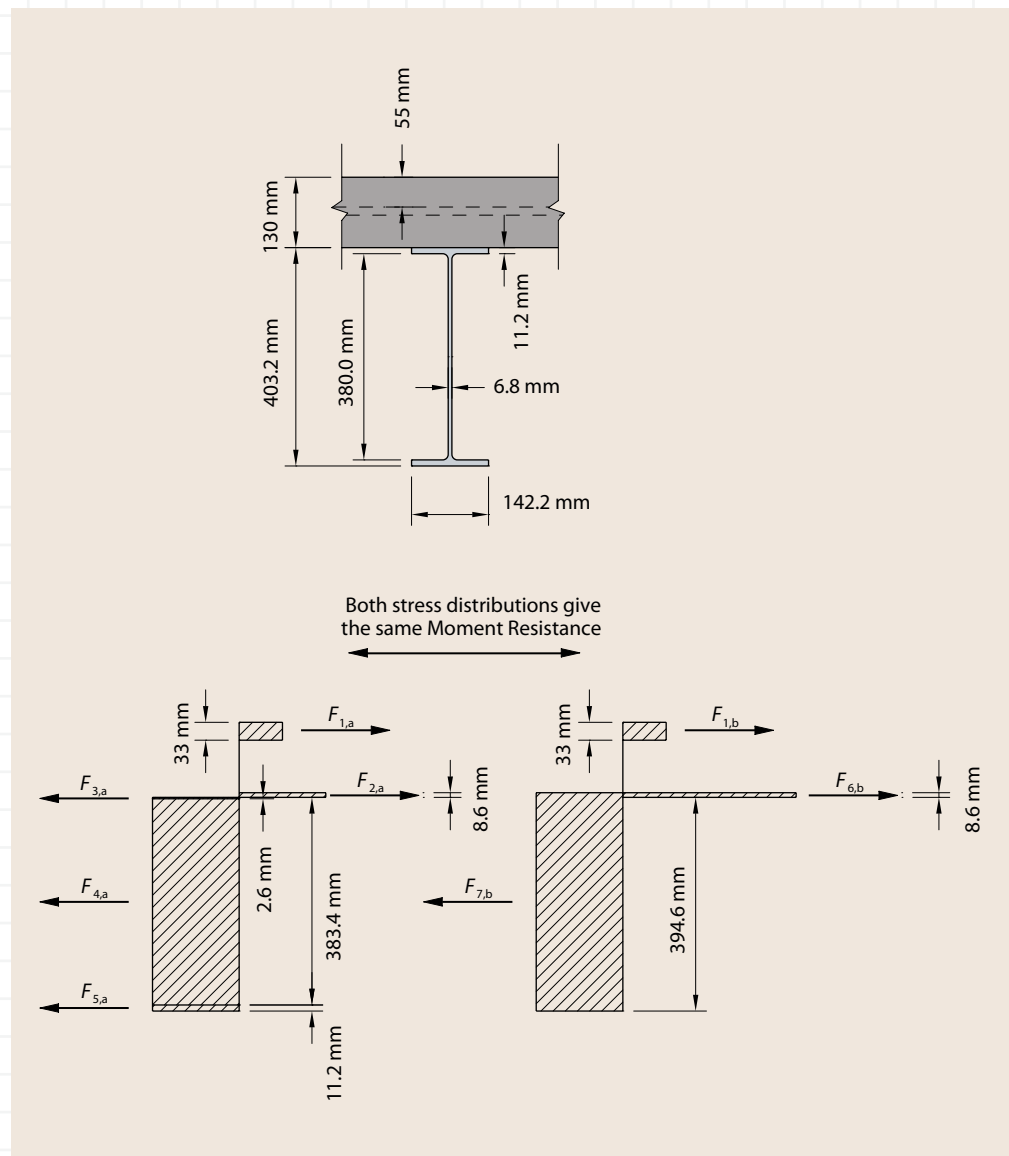

Figure B.9
Position of plastic
neutral axis for
the composite
section with partial
shear connection
(one stud per rib)

Using the right-hand stress distribution:

$$F_{1,a} \quad = F_{2,a} \quad = N_c \quad = 939 \text{ kN}$$

$$F_{6,b} \quad = 2 \times F_{2,a} \quad = 2f_y b x_{pl} \quad = 2 \times 275 \times 142.2 \times 8.6 \times 10^{-3} \quad = 673 \text{ kN}$$

$$F_{7,b} \quad = N_{pl,a} \quad = 1612 \text{ kN}$$

$$M_A \quad = -F_{1,b}\left(\frac{0.0294}{2}\right) - F_{6,b}\left(0.130 + \frac{0.0086}{2}\right) + F_{7,b}\left(0.130 + \frac{0.4032}{2}\right)$$

$$= -939(0.0147) - 673(0.1343) + 1612(0.3316)$$

$$= -13.803 - 90.384 - 534.54$$

$$M_{Rd} \quad = M_A \quad = 430 \text{ kNm}$$

$$M_{Ed} \quad = 357 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{Rd}} = \frac{357}{430} = 0.83 < 1.0$$

Design bending resistance is adequate.

Resistance with two shear connectors per trough ($n_r = 2$)

Similar to the case of one shear connector per trough rigid plastic theory from 4-1-1/(6.2.1.2) may be used.

PNA lies in the flange of the steel beam, as before:

$$x_{pl} = \frac{\left(N_{pl,a} - N_c\right)}{2f_y b}$$

$$N_{pl,a} = 1612 \text{ kN}$$

$$N_c = 1083 \text{ kN}$$

$$x_{pl} = \frac{(1612 - 1083)}{2(275)(142.2)} \times 10^3$$

$$x_{pl} = 6.76 \text{ mm}$$

Taking $x_{pl} = 6.76$ mm from top of flange 6.76 mm $\leq$ 11.2 mm (t_f)

For this case the forces are as shown to the right.

Using the right-hand stress distribution:

$$F_{1,a} = F_{2,a} = N_c = 1083 \text{ kN}$$

$$F_{6,b} = 2 \times F_{2,a} = 2f_y b x_{pl} = 2 \times 275 \times 142.2 \times 6.76 \times 10^{-3} = 529 \text{ kN}$$

$$F_{7,b} = N_{pl,a} = 1612 \text{ kN}$$

$$M_A = -F_{1,b}\left(\frac{0.033}{2}\right) - F_{6,b}\left(0.130 + \frac{0.00676}{2}\right) + F_{7,b}\left(0.130 + \frac{0.4032}{2}\right)$$

$$= -1083(0.0165) - 529(0.1334) + 1612(0.3316)$$

$$= -17.87 - 70.57 - 534.54$$

$$M_{Rd} = M_A = 446 \text{ kNm}$$

$$M_{Ed} = 357 \text{ kNm}$$

$$\frac{M_{Ed}}{M_{Rd}} = \frac{357}{446} = 0.80 < 1.0$$

Design bending resistance is adequate.

There is very little benefit in providing 2 studs per rib.

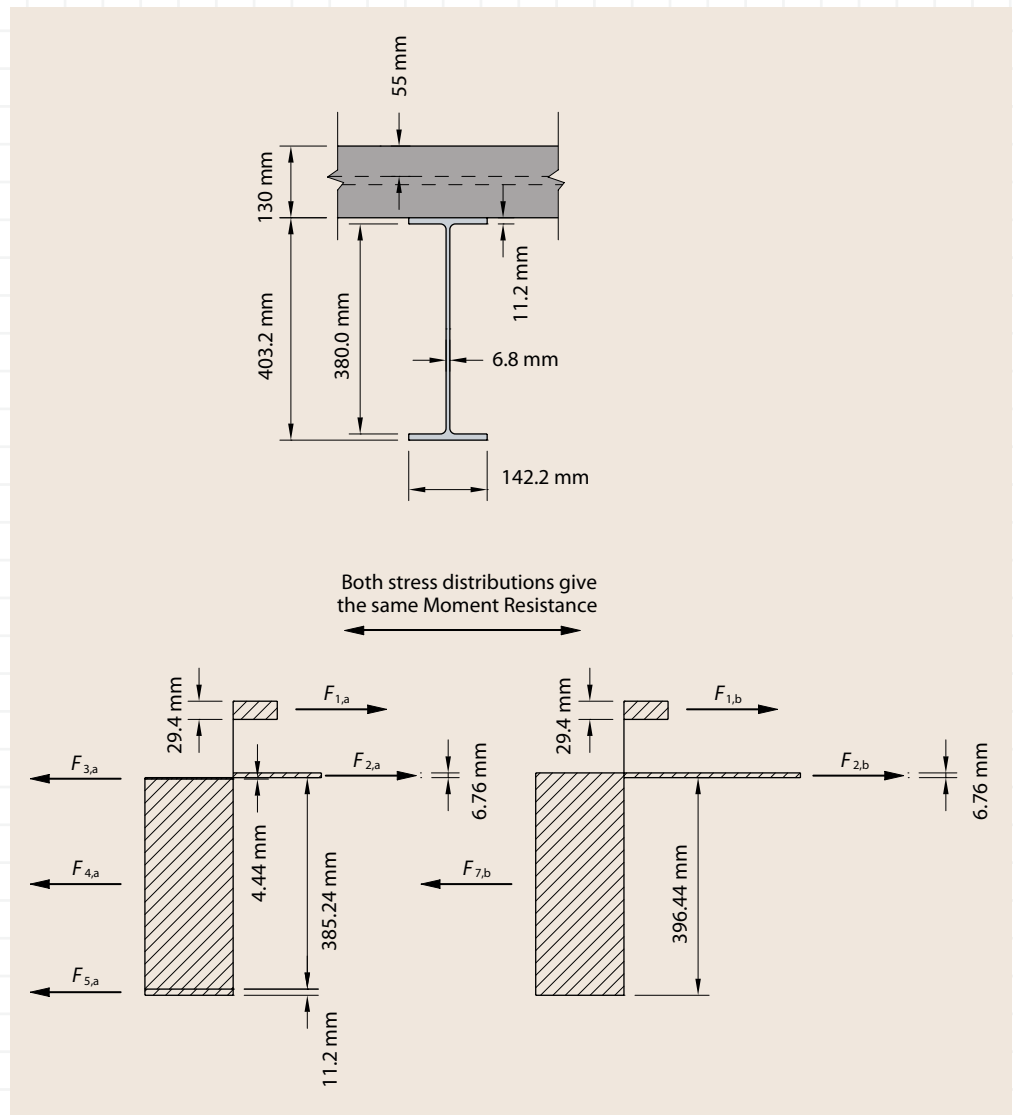The remaining check will be carried out for one stud only.

*Figure B.10
Plastic stress
blocks and plastic
neutral axis position
for the composite
section with partial
shear connection
(two studs per rib)*

B.11.3 Longitudinal shear resistance of the slab

Transverse reinforcement

As the profiled steel sheeting has its ribs transverse to the beam, is continuous over the beam and has mechanical interlocking, its contribution to the transverse reinforcement for the shear surface shown above may be allowed for by replacing expression (6.21) in 2-1-1/6.2.4(4) by:

4-1-1/Eq (6.25)
$$\left(\frac{A_{sf} f_{sd}}{s_f} \right) + \left(A_{pe} \, f_{yp,d} \right) > \frac{v_{Ed} h_f}{\cot \theta}$$

However, in practice it is usual to neglect the contribution of the steel sheeting. Therefore, verify that:

2-1-1/Eq (6.21)
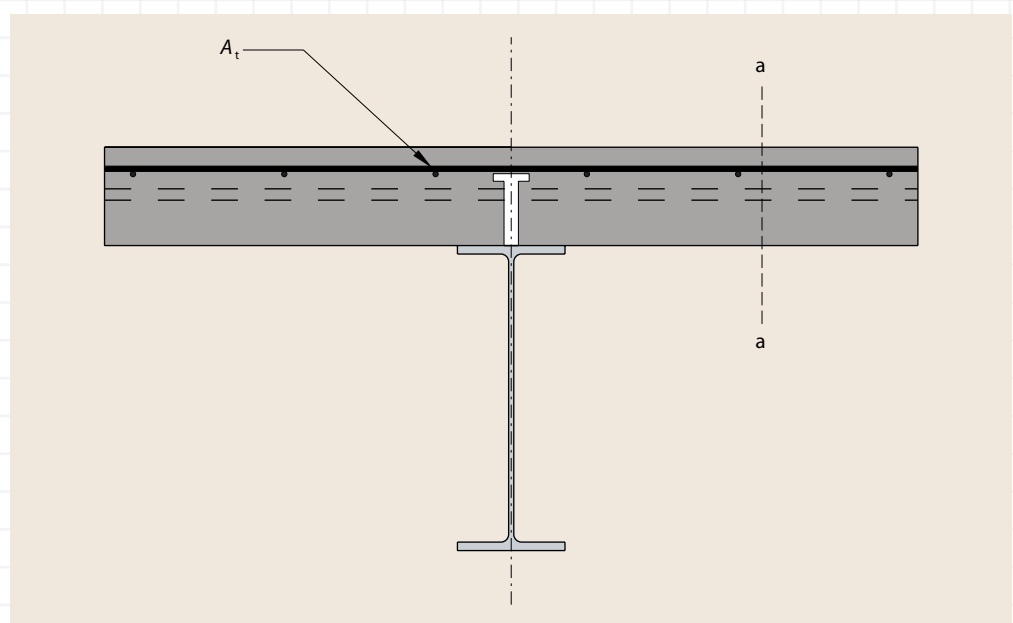$$\frac{A_{sf} f_{sd}}{s_f} > \frac{v_{Ed} h_f}{\cot \theta}$$

where

v_{Ed}	is the design longitudinal shear stress in the concrete slab
f_{sd}	is the design yield strength of the reinforcing mesh
f_{sd}	$= \dfrac{f_y}{1.15} = \dfrac{500}{1.15} = 434.8$ N/mm^2
h_f	is the depth of concrete above the profiled sheeting
h_f	$= h_c = 70$ mm
θ	$= \theta_f$ given in BS EN 1992-1-1 as the angle of failure.

2-1-1/6.2.2(6) $\quad 45° \leq \theta_f \leq 26.5°$

To minimise the amount of reinforcement, try:

$\theta_f \qquad = 26.5°$

4-1-1/Figure 6.16 $\quad \left(\dfrac{A_{sf}}{s_f}\right) = A_t$ (for the failure plane shown in Figure B.11 as section a-a).

$A_t \qquad$ is the cross-sectional area of transverse reinforcement (mm²/m)

Therefore, the verification becomes:

$$A_t f_{yd} \quad > \quad \frac{v_{Ed} h_f}{\cot \theta}$$

And the required area of tensile reinforcement (A_t) must satisfy the following

$$A_t \quad > \quad \frac{v_{Ed} h_f}{f_{yd} \cot \theta}$$

2-1-1/6.2.4(3) The longitudinal shear stress is given by:

$$v_{Ed} = \frac{\Delta F_d}{h_f \Delta x}$$

where

Δx is the critical length under consideration, which for this example is the distance between the maximum bending moment and the support.

$$\Delta x = \frac{L}{2} = \frac{9}{2} = 4.5 \text{ m}$$

$$\Delta F_d = \frac{N_c}{2}$$

For $n_r = 1$, $\quad \Delta F_d = 939/2 \quad = 469.5$ kN

$$h_f = 70 \text{ mm}$$

For $n_r = 1$, $\quad v_{Ed} = \frac{\Delta F_d}{h_f \Delta x} = \frac{(469.5) \times 10^3}{(70)(4500)} = 1.49 \text{ N/mm}^2$

For $n_r = 1$, $\quad \dfrac{v_{Ed} h_f}{f_{yd} \cot \theta} = \dfrac{(1.49) \times 70}{434.8 \times \cot(26.5°)} = 0.120 \text{ mm}^2/\text{mm}$

Therefore, the area of tensile reinforcement required is:

For $n_r = 1$, $\quad A_t \geq 120 \text{ mm}^2/\text{m}$

The reinforcement provided is A193 mesh, for which:

$$A_t = 193 \text{ mm}^2/\text{m} > 137 \text{ mm}^2/\text{m}$$

Therefore an A193 mesh is adequate.

Crushing of the concrete flange

Verify that:

$$v_{Ed} \leq v f_{cd} \sin\theta_f \cos\theta_f$$

where

$\quad v = 0.6\left[1 - \dfrac{f_{ck}}{250}\right] = 0.6 \times \left[1 - \dfrac{25}{250}\right] = 0.54$

$\theta_f = 26.5°$

$$v f_c d \sin\theta_f \cos\theta_f = 0.54 \times 16.7 \times \sin(26.5°) \times \cos(26.5°) = 3.60 \text{ N/mm}^2$$

$$v_{Ed} = 1.71 \text{ N/mm}^2 < 3.60 \text{ N/mm}^2$$

Therefore the crushing resistance of the concrete is adequate.

<div style="text-align:right">6.1</div>

B.12 Verification at SLS

<div style="text-align:right">6.1.2</div>

B.12.1 Modular ratios

For short term loading, the secant modulus of elasticity should be used. From Appendix B.2.2, $E_{cm} = 31$ kN/mm². This corresponds to a modular ratio of:

$$n_0 = \frac{E_a}{E_{cm}} = \frac{210}{31} = 6.77$$

For long term loading, the modular ratio should be calculated from:

$$n_L = n_0(1 + \psi_L \varphi_t)$$

where

ψ_L is the creep multiplier, taken as 1.1 for permanent loads

φ_t is the creep coefficient, taken as 3 in this case

n_L $= 6.77 \times (1 + 1.1 \times 3) = 29.11$

When calculating deflections due to variable actions the modular ratio is taken as follows:

$$n = \frac{1}{3}n_L + \frac{2}{3}n_0 = 14.22$$

For dynamic conditions (i.e. natural frequency calculations), the value of E_c should be determined according to SCI publication P354, which gives $E_c = 38$ kN/mm², and so the dynamic modular ratio is:

$$n_d = \frac{E_a}{E_c} = \frac{210}{38} = 5.53$$

B.12.2 Second moment of area of composite section

For the case $n_r = 1$

b_{eff} $= 2.25$ m

n_0 $= 6.77$, $I_c = 57{,}819$ cm⁴   ($z_{el} = 438$ mm from bottom flange)

n_L $= 29.11$, $I_c = 40{,}619$ cm⁴   ($z_{el} = 344$ mm from bottom flange)

n_d $= 5.53$, $I_c = 59{,}620$ cm⁴   ($z_{el} = 448$ mm from bottom flange)

n $= 14.22$, $I_c = 49{,}848$ cm⁴   ($z_{el} = 395$ mm from bottom flange)

<div style="text-align:right">3.5</div>

B.12.3 Beam deflection

Deflection due to actions on the steel section at the construction stage

$$\delta_{G1} = \frac{5g_1 L^4}{384 EI} = \frac{(5) \times (8.38 \times 10^3) \times 9^4}{(384) \times (210 \times 10^9) \times (15{,}700 \times 10^{-8})} \times 10^3 = 21.7 \text{ mm}$$

Deflection due to permanent actions on the composite beam

$$\delta_{G2} = \frac{5g_2 L^4}{384EI} = \frac{5 \times 2.55 \times 10^3 \times (9)^4}{384 \times 210 \times 10^9 \times 40,619 \times 10^{-8}} \times 10^3 = 2.55 \text{ mm}$$

Deflection due to variable actions on the composite beam

$$\delta_{Q1} = \frac{5q_1 L^4}{384EI} = \frac{5 \times 14.4 \times 10^3 \times (9)^4}{384 \times 210 \times 10^9 \times 49,848 \times 10^{-8}} \times 10^3 = 11.75 \text{ mm}$$

3-1-1/NA.2.23 $\quad \delta_{TOT} = \delta_{G1} + \delta_{G2} + \delta_{Q1} = 21.7 + 2.55 + 11.75 = 36 \text{ mm} < \dfrac{L}{200} = 45 \text{ mm}$

Table 6.4 Deflection due to variable actions is $11.75 \text{ mm} < \dfrac{L}{360} = 25 \text{ mm}$ OK

6.1.3 B.12.4 SLS stress verification

To validate the assumptions used to calculate the deflections, the stress in the steel and concrete should be calculated to ensure that neither material exceeds its limit at SLS.

Stresses in the steel section due to actions on the steel section at the construction stage

$$\sigma_{G1,a} = \frac{g_1 L^2 z_{el}}{8I} = \frac{(8.38 \times 10^3)(9)^2 (201.6 \times 10^{-3})}{8 \times 15,700 \times 10^{-8}} \times 10^{-6} = 141 \text{ N/mm}^2$$

Stresses in the steel section due to permanent actions on the composite beam

$$\sigma_{G2,a} = \frac{g_2 L^2 z_{el}}{8I} = \frac{(2.55 \times 10^3)(9)^2 (344 \times 10^{-3})}{(8)(40,619 \times 10^{-8})} \times 10^{-6} = 23 \text{ N/mm}^2$$

Stresses in the steel section due to variable actions on the composite beam

$$\sigma_{Q1,a} = \frac{q_1 L^2 z_{el}}{8I} = \frac{(14.4 \times 10^3)(9)^2 (395 \times 10^{-3})}{(8)(49,848 \times 10^{-8})} \times 10^{-6} = 116 \text{ N/mm}^2$$

Maximum stress in the steel section

$$\sigma_a = 141 + 23 + 116 = 280 \text{ N/mm}^2$$

Although the extreme fibre stress slightly exceeds the yield strength of the steel section (275 N/mm²), for practical purposes this will have very little effect of the deflection; the deflection will still be below the limiting value. Increasing the steel grade or section size simply to reduce this stress would be unnecessarily onerous.

Stresses in the concrete flange due to permanent actions on the composite beam

$$\sigma_{G2,c} = \frac{q_2 L^2 z_{el}}{8I n_L} = \frac{(2.55 \times 10^3)(9^2)(403.2 + 130 - 344) \times 10^{-3}}{(8)(57,819 \times 10^{-8}) \times 29.11} \times 10^{-6} = 0.67 \text{ N/mm}^2$$

Stresses in the concrete flange due to variable actions on the composite beam

$$\sigma_{Q1,c} = \frac{q_1 L^2 z_{el}}{8 I n_0} = \frac{\left(14.4 \times 10^3\right)\ (9)^2\ (403.2 + 130 - 395) \times 10^{-3}}{(8)\ (49,848 \times 10^{-8}) \times 14.22} \times 10^{-6} = 2.84\ \text{N/mm}^2$$

Maximum stress in the concrete

$$\sigma_c \qquad = 0.67 + 2.84 = 3.51\ \text{N/mm}^2 < f_{cd} = 16.7\ \text{N/mm}^2$$

6.3   B.12.5 Natural frequency

For the calculation of the natural frequency, SLS values of the full permanent load and 10% of the variable actions are assumed to be present in service.

From Appendix B.4.3. this total is:

$$8.38 + 2.55 + (4.8 \times 3 \times 0.1) = 12.4\ \text{kN/m}$$

The deflection under this load is:

$$\delta_{G1} \qquad = \frac{5 \omega L^4}{384 EI} = \frac{(5)\left(12.37 \times 10^3\right)(9)^4}{(384)\ (210 \times 10^9)(59,620 \times 10^{-8})} \times 10^3 = 8.45\ \text{mm}$$

The natural frequency of the beam is therefore:

P354 $f \qquad = \dfrac{18}{\sqrt{\delta}} = \dfrac{18}{\sqrt{8.45}} = 6.19\ \text{Hz}$

P354   Thus $f > 4$ Hz. Therefore the beam is OK for initial calculation purposes. However, the dynamic performance of the entire floor should be verified using a method such as the one in P354.

COMPOSITE DESIGN OF STEEL FRAMED BUILDINGS

Steel-concrete composite floor construction makes efficient use of structural materials to achieve an economic solution, making it a preferred form of construction for steel framed buildings. Concrete slabs are cast on profiled steel sheeting; the two materials act structurally together once the concrete has hardened. The slab is supported on steel beams and is made to act structurally with the beams by means of welded shear connectors. In service, the steel and concrete act together to offer a light and efficient floor system. This design guide provides a comprehensive design methodology for composite slabs and composite beams, for design in accordance with the Eurocodes.

The guidance is complemented by a full numerical worked example.

Complementary titles
A suite of publications support building design to the Eurocodes in the UK, including:

P365
Steel Building Design: Medium Rise Braced Frames

P355
Design of Composite Beams with Large Web Openings

P363
Steel Building Design: Design Data (The Blue Book)