Design of Composite Beams Using Precast Concrete Slabs

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FOREWORD

Pre-cast concrete floors are widely used in building construction, but there is little detailed design guidance on their application in steel framed buildings. It is estimated that close to 50% of floors used in steel framed buildings in the UK use hollow core or solid plank slabs. Most of these applications are in regular steel construction in which the precast slabs sit on the top flange of the beams, but there is an increasing number of composite frames and slim floor constructions where the precast slabs are designed to interact structurally with the steel frame. Composite action can be developed by welded shear connectors attached to the steel beams and by transverse reinforcement, but this form of construction is currently outside the provisions of BS 5950-3: 1990 and little design guidance currently exists. Due to the fact that this type of construction is not properly covered by the Codes of Practice, this publication presents design guidance on the interaction and detailing of precast slabs (of hollow core or solid plank section) that are supported by composite beams or slim floor beams.

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SUMMARY

This publication provides guidance on the design of steel beams acting compositely with precast concrete slabs in multi-storey buildings. The use of hollow core or solid plank precast units offer benefits in terms of cost (the long spanning capabilities of the precast slabs lead to fewer secondary beams) as well as the advantages offered by composite construction. The design basis is generally in accordance with BS 5950-3, supplemented by recommendations from Eurocode 4 and data from tests. Particular issues affecting the use of precast concrete concern the requirements of effective shear connection and transverse reinforcement. Small-scale push-out tests, and earlier composite beam tests, have established reduction factors for the design resistance of welded shear connectors as a function of the gap between the ends of the precast concrete units, and the amount of transverse reinforcement provided.

The guidance also emphasises the importance of the design of the steel beam in the non-composite construction stage, where out-of-balance loads can occur during installation of the precast concrete units. The guidance applies to hollow core units of 150 to 260 mm depth, and to solid precast planks.

A step-by-step design procedure is given for composite beams using various forms of precast concrete units, with or without a concrete topping. This is supplemented by a fully worked design example for a composite beam in a 15.8 m × 7.2 m grid, and a series of design tables for concept design.

Dimensionnement de poutres composites utilisant des dalles en béton préfabriquées

Résumé

Cette publication est destinée à servir de guide de dimensionnement de poutres en acier agissant, de manière composite, avec des dalles en béton préfabriquées dans les immeubles multi-étagés. L'utilisation de dalles creuses ou pleines est intéressante tant en terme de coût (la possibilité de grandes portées conduit à diminuer le nombre de poutres secondaires), que pour son action composite. Le dimensionnement est généralement conforme à la BS 5950-3 tout en utilisant des informations complémentaires provenant de l'Eurocode 4 et d'essais.

Deux points particuliers doivent être pris en compte, à savoir la réalisation d'assemblages efficaces en cisaillement ainsi que les armatures transversales. Des essais à petite échelle ainsi que les résultats obtenus précédemment sur des poutres composites ont permis d'établir des coefficients de réduction de la résistance de dimensionnement des connecteurs soudés en fonction de la distance entre les extrémités des éléments en béton et la quantité d'armatures transversales à mettre en œuvre.

Le guide attire l'attention sur l'importance des phases de construction où des déséquilibres peuvent survenir durant la poste des éléments préfabriqués. Cette publication s'applique aux éléments préfabriqués creux de 150 à 260 mm d'épaisseur ainsi qu'aux éléments pleins.

Une procédure de dimensionnement, procédant d'étape en étape, est exposée pour les poutres composites utilisant diverses formes d'éléments préfabriqués, avec ou sans utilisation d'une chape en béton. Un exemple complet de dimensionnement d'une poutre

composite avec une grille de poutres de 15,8 m x 7,2 m et une série de tableaux de dimensionnement complètent la publication.

Berechnung von Verbundträgern unter Verwendung von Fertigteilplatten

Zusammenfassung

Diese Publikation bietet eine Anleitung zur Berechnung von Stahlträgern die im Verbund mit Betonfertigteilplatten in mehrgeschossigen Gebäuden wirken. Die Verwendung von Hohlplatten oder massiven Fertigteilelementen bietet sowohl Kostenvorteile (die längeren Spannweiten der Fertigteildecken führen zu weniger Sekundärträgern) als auch Vorteile durch die Verbundkonstruktion. Die Berechnungsgrundlage stimmt im allgemeinen mit BS 5950-3 überein, ergänzt durch Empfehlungen aus dem Eurocode 4 und Daten aus Versuchen. Spezielle Probleme, die die Verwendung von Fertigteilen betreffen, behandeln die Anforderungen einer wirksamen Schubverbindung und der Querbewehrung. Push-out Versuche und frühere Versuche an Verbundträgern haben zu Reduktionsfaktoren für den Bemessungswiderstand von geschweißten Schubverbindern geführt, als Funktion des Abstands zwischen den Enden der Betonfertigteile und dem Grad der Querbewehrung.

Die Anleitung hebt auch die Bedeutung der Berechnung des Stahlträgers im Bauzustand ohne Verbund hervor, bei dem Belastungen während der Montage der Betonfertigteile auftreten können. Die Anleitung betrifft Hohlplatten von 150 bis 260 mm Querschnittshöhe und massive Fertigteilplatten.

Ein schrittweises Berechnungsverfahren für Verbundträger mit verschiedenen Arten von Betonfertigteilen, mit oder ohne Aufbeton, wird vorgestellt. Es wird ergänzt durch ein Berechnungsbeispiel eines Verbundträgers in einem 15.8 m x 7.2 m Raster und einer Anzahl von Tafeln für die Vorbemessung.

Proyecto de vigas mixtas mediante losas prefabricadas de hormigón

Resumen

Esta publicación sirve de guía para el proyecto de vigas de acero actuando como estructura mixta con losas prefabricadas de hormigón en edificios de varias plantas. El uso de núcleos huecos o macizos ofrece ventajas en términos de coste (la capacidad de las losas prefabricadas para cubrir vanos largos reduce el número de viguetas) así como en el aprovechamiento de los inherentes a la construcción mixta.

Generalmente las bases de proyecto están de acuerdo con la BS 5950-3 complementadas con recomendaciones del Eurocódigo 4 y con datos de ensayos. Algunos temas especiales que afectan el uso del hormigón prefabricado se refieren a los requisitos de conexiones efectivas antes esfuerzos cortantes y el armado transversal. El uso de ensayos de empuje progresivo (push-out) a pequeña escala y otros ensayos previos de vigas mixtas han permitido el establecimiento de factores reductores de la resistencia a cortante del proyecto de conectores soldados en función del ancho de la junta entre los extremos de unidades prefabricadas y la cantidad de armado que se coloca.

La guía también remarca la importancia del proyecto de la viga de acero en la etapa constructiva como sola estructura resistente donde pueden producirse cargas desequilibradas durante la colocación de las unidades prefabricadas de hormigón. La guía es aplicable a unidades con núcleos huecos o macizos y cantos entre 150 y 260 mm.

Se presenta un método paso a paso de proyecto de vigas mixtas usando varias formas de unidades prefabricadas de hormigón con o sin capa de reparto de hormigón. Ello se completa con un ejemplo del proyecto, totalmente desarrollado, de una viga en una malla de 15'8 x 7'2 m y una serie de tablas útiles en la etapa de anteproyecto.

Progettazione di travi composte con solette prefabbricate in calcestruzzo

Sommario

Questa pubblicazione fornisce una guida alla progettazione di travi in acciaio collaboranti con solette composte prefabbricate in edifici multipiano. L'uso di moduli prefabbicati con solette alveolate o piane implica non trascurabili benefici economici (la possibilità di coprire luci notevoli permette l'eliminazione di travi secondarie), unitamente a vantaggi associati alla costruzione composta in acciaio e calcestruzzo.

La progettazione di base viene generalmente effettuata in accordo alla BS 5930-3, integrata dalle raccomandazioni dell'Eurocodice 4 e dai risultati della sperimentazione. Particolari indicazioni sull'uso dei moduli in calcestruzzo prefabbricati riguardano le specifiche della connessione a taglio e dell'armatura trasversale.

Sulla base di prove di resistenza del piolo e di precedenti prove sulla trave composta sono proposti i fattori di riduzione da utilizzarsi nella progettazione per le definizione della resistenza di progetto di connettori a taglio saldati, in funzione della distanza tra le estremità dei moduli prefabbricati e del quantitativo di armatura trasversale presente.

Nel volume viene sottolineata anche l'importanza della progettazione della trave in acciaio nella fase di costruzione, quando il calcestruzzo non è collaborante e possono agire azioni flettenti non bilanciate dovute al posizionamento dei moduli. Le indicazioni riportate sono applicabili per moduli alveolati di altezza variabile da 150 mm a 260 mm e a solette piene prefabbricate.

Viene fornita una procedura di tipo passo-a-passo per la progettazione della trave composta con differenti tipologie di moduli prefabbricati, considerando la presenza, ovvero l'assenza, del getto di completamento superiore in calcestruzzo. In aggiunta si riporta un'applicazione progettuale completa per la trave composta di una maglia strutturale di dimensioni 15.8m x 7.2 m e sono fornite alcune utili tabelle progettuali.

Utformning av samverkansbalkar genom att använda prefabricerade betongelement

Sammanfattning

Denna publikation tillhandahåller vägledning vid utformning av stålbalkar i samverkan med prefabricerade betongelement i flervåningsbyggnader. Användningen av håldäckselement eller prefabricerade betongplattor innebär kostnadsfördelar (den långa spännvidden med prefabricerade element möjliggör färre sekundärbalkar) liksom de andra fördelar som man får ut av samverkanskonstruktioner. Utformningen är, generellt sett, i överensstämmelse med BS 5950-3, tillsammans med rekommendationer från Eurocode 4 och resultat från olika försök.

Svårigheter som påverkar användningen av prefabricerad betong är kopplat till de krav som finns på effektiv skjuvförbindning och tvärgående armering. Småskaliga utdragstester och tidigare tester på samverkansbalkar har utgjort grunden för de reduktionsfaktorer som används vid bestämning av bärförmåga för svetsade skjuvförband.

Reduktionsfaktorerna tar hänsyn till gapet mellan betongelementens ändar samt mängden tvärgående armering.

Denna anvisning understryker också vikten av att dimensionera stålbalkarna i det byggnadsskede då inte samverkan med betongen uppnåtts, då montagelaster kan inträffa då betongelementen monteras. Anvisningen är praktiskt tillämpbar för håldäckselement med elementtjocklekar på 150 och 260 mm, och för prefabricerade betongplattor.

*Ett steg för stegförfarande av dimensioneringen presenteras för samverkandbalkar som har varierande form av prefabricerade betongelement, med eller utan betong på ytan. Detta kompletteras av ett fullständigt beräkningsexempel för en samverkansbalk i ett 15.8 m * 7.2 m rutnät, och serier av dimensioneringstabeller för konceptuell utformning.*

1 INTRODUCTION

1.1 Background

Steel construction has achieved a high market share in building construction, and is often used in conjunction with various types of precast concrete floors. It is estimated that 50% of multi-storey steel frames use precast concrete floors, and in many building sectors (such as hotels, residential buildings and car parks), the percentage is much higher.

Precast slabs can be used with steel beams either in the traditional 'downstand beam' arrangement (slab on top of beams) or with *Slimflor* beams (slab within the depth of the steel beams). In both cases, the precast units provide a flat soffit and achieve long spans between the supporting beams; with *Slimflor* construction, the soffit is flat over the whole floor area.

1.2 Benefits of composite beams using precast concrete units

The synergy between the use of precast concrete units and steel structures is that they both come from a manufacturing technology rather than a site-based activity, and share the quality control, accuracy and reliability of factory production.

The particular advantages of using these two components in composite applications are:

- The weight and depth of the steel section can be reduced relative to non-composite applications, leading to savings in both steel cost and building height.

- The span of the hollow core slabs is such that the number of secondary beams can be reduced compared to traditional composite beams (where the secondary beam spacing is dictated by the spanning capabilities of the composite deck-slab), leading to fewer beams, and therefore quicker erection of the steelwork.

- A wide range of precast concrete products and steel beam sizes is available.

- A flat soffit is created between discrete downstand beams (which can be aligned with walls).

- Precast concrete units may be preferred in semi-exposed applications, such as car parks, where enhanced durability is required.

- The construction system is most efficient for column grids of approximately 9 m × 9 m, where the spanning capabilities of the precast concrete units can be maximised, and the beam size provides adequate bearing length for the units.

- Shear connectors can be shop-welded before delivery to site (i.e., fewer site operations).

- The optimum number of shear connectors may be provided on the steel section (unlike traditional composite beams, where the pitch of the troughs within the profiled steel decking dictates the stud spacing).

- The precast units have a natural pre-camber which offsets imposed load deflections. The steel beams can also be delivered with a pre-camber for long-span applications.

- 'Dry construction' may be used if there is no topping.

1.3   Design considerations

The combined use of structural steel and precast concrete requires careful attention at the design stage. The following should be taken into consideration:

- The different industries from which the components are sourced.

- The different design standards (or absence of standards in some areas) for their use in combination.

- The responsibilities for design and installation may not be clearly defined at the preliminary design stage.

- The stability of the beams during installation of the precast units must be ensured by temporary or permanent restraints, which should be properly designed.

- Building Regulation requirements for robustness and other issues must be addressed.

- The interaction between the steel support beams and hollow core slabs may give rise to secondary stresses in the slabs.

- The compatibility of fire resistance requirements of the supporting steel structure and the precast concrete flooring.

- The provision for openings and secondary attachments to the slab may influence the design of the slab and its support structure.

- The CDM Regulations require the designer and contractor to cooperate to ensure safety during construction, and provide information for the building owner.

It is timely to prepare design guidance on new uses of precast concrete floors, particularly those relying on composite action between the steel and concrete.

1.4   Scope of this publication

This publication covers the design of composite beams using precast concrete units of hollow core or solid plank cross-section, in accordance with the principles of BS 5950-3:1990[1] and also with Eurocode 4[2] (DD ENV 1994-1-1:1994). The forms of composite beams considered are described in Section 2.

Solid plank units are normally used with an *in situ* topping, which enables composite action of both primary and secondary steel beams to be achieved. Conversely, when hollow core units are used, only composite action with the long-span secondary beams directly supporting the units is possible (due to the orientation of the cores).

Particular issues addressed in this guidance are:

- Effective width of the slab for composite action.

- Shear connection, and minimum degree of shear connection.

- Transverse reinforcement (site placed reinforcement perpendicular to the longitudinal axis of the beam).

- Constructional issues (e.g., bearing length and gap between the units).

- Temporary stability of the beams during installation.

- Fire resistance requirements.

- Serviceability performance.

- Steelwork connections (which affect stability during construction).

- Design tables for common design cases.

- Slim floor construction (as influenced by composite action).

- Temporary propping of beams during construction.

1.5   Design basis

This publication follows the design recommendations given in BS 5950-3: 1990[1] and, where necessary, includes recommendations given in Eurocode 4: Part 1.1[2] (DD ENV 1994-1-1:1994).   For cases where the guidance in these codes of practice is unavailable, or incomplete, design equations based on the principles of BS 5950 have been developed from test information.   As a result, this publication is intended as a supplement for designing composite beams in accordance with BS 5950-3: 1990.   Although the principles presented here may be adapted for DD ENV 1994-1-1:1994, it is not the intention of this publication to offer particular design guidance for this Eurocode.

Design of the precast concrete units should be carried out in accordance with BS 8110-1:1997[3] and BS 8110-2:1985[4], or with Eurocode 2.   Currently, reference may be made to DD ENV 1992-1-1[5], DD ENV 1992-1-2:1996[6] and DD ENV 1992-1-3:1996[7].   These CEN pre-standards will be replaced in due course by EN 1992-1-1 and EN 1992-1-2.

Design to both BS 5950[1] and BS 8110[3][4] is based on limit state principles in which:

- partial factors of 1.6 (1.05 in the case of accidental damage) and 1.4 are applied to imposed and dead loads respectively; and

- partial factors of 1.0 are used in serviceability calculations.

The approach in the Eurocodes is subtly different because partial factors are applied to both loads and materials, and reduced partial factors are applied at the serviceability limit state.

2   FORMS OF CONSTRUCTION

In this Section, the different types of floor system using steel beams and precast concrete units are described. A general overview of the range of precast slabs that are available, and their impact on the sizing of the supporting beams is also presented.

2.1   Generic forms

Four generic forms of steel construction using precast concrete slabs may be identified:

- Non-composite steel beams supporting precast slabs on their top flange.

- Non-composite slim floor beams supporting precast slabs on their lower flange, or shelf angle beams supporting the slab on an angle connected to the web of the beam.

- Composite steel beams supporting precast hollowcore or solid planks in which the two act compositely, due to shear connectors welded to the top flange.

- Composite slim floor beams supporting precast slabs on their lower flange, using an *in situ* concrete topping and welded shear connectors.

Some examples of precast concrete units used in composite applications are illustrated in Figure 2.1.

Composite beam with square-ended
hollowcore slabs

Composite beam with chamfered-ended
hollowcore slabs

Composite beam with precast planks

Composite slim floor beam
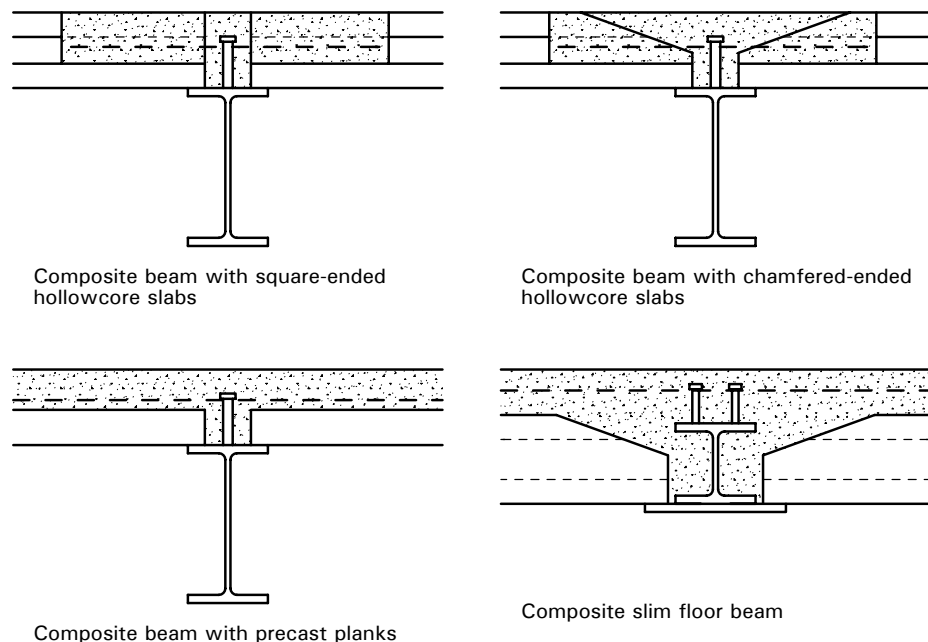
Figure 2.1   *Different composite applications of steel and precast
concrete*

A typical example of composite floor construction using hollow core units is shown in Figure 2.2; the line of shear connectors indicates the position of the composite beam.

Figure 2.2 *A typical example of composite floor construction with precast concrete hollow core units, showing transverse reinforcement bars being placed within the open cores*

2.2 Types of precast slab

The most common types of precast concrete slabs used in conjunction with steel beams are:

- Hollow core units, of 150 to 260 mm depth, with continuous circular or elongated openings along their length (see Figure 2.3).

- Solid planks, of 75 to 100 mm depth, which are intended for use with an *in situ* concrete topping.

The guidance in this document applies only to the use of precast units within the above ranges of size.

Hollow core units do not usually require any structural topping, except possibly when: used with slim floor beams (see Figure 2.1), diaphragm action is required for taller buildings, or for fire safety reasons.
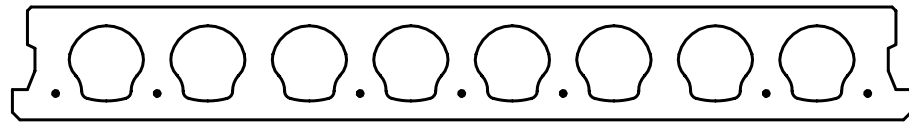
(a)   Non-circular cores
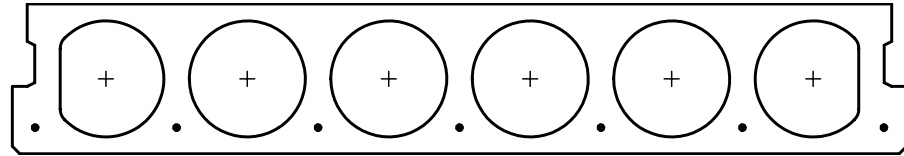
(b)   Circular cores
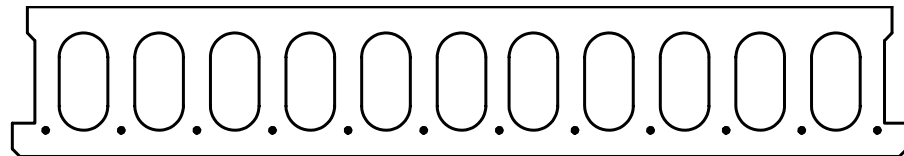
(c)   Elongated cores

Figure 2.3   *Examples of hollow core slabs*

A wide range of precast slab products is available from various manufacturers. Most precast concrete slabs are produced in a process in which wires or strands are pre-tensioned and high strength concrete is cast around them in a factory controlled process, often involving over 100 m of continuous casting. Two methods of casting are used: slipforming; and extruding. The ends of the units can be formed with a chamfer during manufacture.

When the concrete has reached its specified strength (often after a few hours), the wires or strands are released from their anchorages, and the units are cut to the required length. The pre-stressing force causes compression in the concrete section, which increases its bending resistance and stiffness. The design of the precast concrete units is highly complex and detailed design for particular applications is normally carried out by the manufacturer.

2.3   Downstand beams

When downstand beams are used, the steel beam should be at least 180 mm wide (see Section 3.1) in order to support the precast units and to allow space for the concrete encasement around the shear connectors. Therefore, a 406×178 UB section is normally the minimum beam size that can be used, unless the steel section is fabricated, or made from separate Tee-sections.

2.4   *Slimflor*[†] **beams**

Precast slabs can be used in conjunction with *Slimflor*® Fabricated Beams (SFB) and Rectangular Hollow Section *Slimflor*® Edge Beams (RHSFB). The precast units are supported on a bottom flange plate, which is welded to a UC or RHS section. The bottom flange plate should be sufficiently wide to extend a minimum of 100 mm from each flange tip, to allow for sufficient end bearing of

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[†] *Slimflor*® is a Registered Trademark of Corus.

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the precast units and for effective placement of the concrete around the UC section (see Section 8). Composite action with an *in situ* topping can be achieved by the provision of short welded studs attached to the top flange of the UC. Specific guidance on the design of RHSFB and SFB may be found in two SCI publications[8],[9]. For non-composite applications, Asymmetric *Slimflor®* Beams (ASB)[10] may also be used.

2.5 Materials

Structural steel should be supplied in accordance with BS EN 10025:1993[11]. Two strength grades are used within the UK, grade S275 and S355.

The design resistance of headed stud shear connectors is defined in BS 5950-3:1990[1]. Two common diameters are used: 19 mm (for site or factory welding) and 22 mm (usually only for factory welding). For use with hollow core units, they are usually supplied in 125 mm height (120 mm as-welded height). However, other height studs may be used.

According to BS 8110-1:1997[3], concrete strength is defined by its cube strength. For the precast units, the cube strength of the concrete is typically between 50 and 60 N/mm². The minimum specified cube strength of the *in situ* concrete should be at least 30 N/mm², and its maximum aggregate size is normally specified as 10 mm (to facilitate placement of concrete between the units).

Steel reinforcement bars should conform to BS 4449:1997[12]. The 1997 edition of BS 4449 was revised considerably compared to its earlier versions, to bring it in line with DD ENV 1992-1-1:1992[5]. Reinforcement steel is classified as follows:

* High (class H) or normal (class N), according to ductility characteristics.

* Plain smooth or, ribbed bars, according to surface characteristics.

Note that DD ENV 10080:1996[13] which is currently at the draft for development stage, will eventually replace BS 4449.

The strength and mechanical properties of reinforcing steels to these two Standards are given in Table 2.1.

Table 2.1 *Mechanical properties of reinforcing steel*

Reinforcing steel standard	BS 4449: 1997[12]		DD ENV 10080: 1996[13]	
Name	460A (class N)	460B (class H)	B500A (class N)	B500B (class H)
Minimum yield strength, f_y	460 N/mm²		500 N/mm²	
Total elongation at maximum force, A_{gt}	2.5%	5.0%	2.5%	5.0%
Elongation at fracture	12%	14%	-	-
Modulus of elasticity, E_s	210000 N/mm²		210000 N/mm²†	

† According to DD ENV 1994-1-1: 1994[2]

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As can be seen from Table 2.1, apart from the obvious difference in the minimum yield strength f_y, BS 4449 also specifies a minimum elongation at fracture; thereby guaranteeing the length of the plastic deformation plateau. (And it may also be noted that the minimum elongation at fracture for 460B steel, is higher than the 12% requirement for this yield strength, given in earlier versions of this British Standard). A graphical representation of the elongation requirements for these two standards is shown in the stress-strain curve in Figure 2.4.
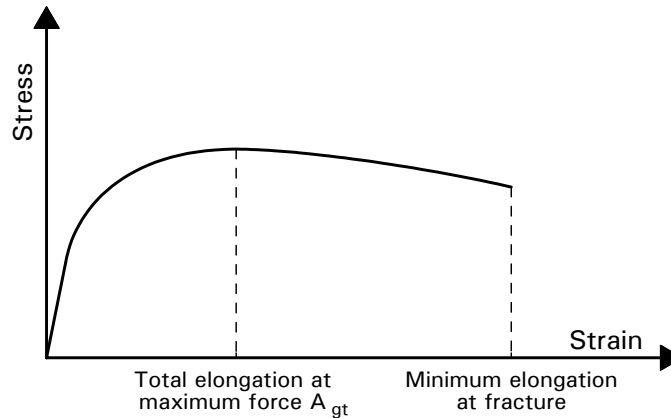
Figure 2.4 *Elongation limits for steel reinforcement bars*

Class H bars of 12 or 16 mm diameter are recommended for composite construction. For cases when partial shear connection is employed, 16 mm diameter Class H bars should be provided.

3   PRACTICAL CONSIDERATIONS

Prior to embarking on a detailed design of a composite beam using hollow core units (or solid planks with a concrete topping), the following practical issues should first be considered by the engineer:

- Shop-welding or site-welding of shear connectors.

- Minimum beam width.

- End conditions of hollow core units.

- Positioning of transverse reinforcement.

- Detailing of edge beams.

- Temporary stability during installation of concrete units.

- 'Robustness' against explosions, etc.

Due to the orientation of the cores, hollow core units can be designed to act compositely only with the supporting long-span secondary beams. Because solid plank units are normally used with an *in situ* topping, both the secondary and primary beams may be assumed to act compositely with the slab.

3.1   Minimum beam width

The minimum beam width required depends on: the type of slab; whether the shear connectors are shop-welded or site-welded; and whether the beam is an internal or edge beam. The width chosen must also take account of the allowances for site tolerances.

3.1.1 Tolerances

There are four factors that affect the size of the actual end bearing of precast units on a steel beam:

- The nominal bearing, as defined on the drawings.

- Variations in the size and position of the steelwork.

- Length variations in the manufacture of the units.

- The accuracy with which the units can be positioned on site.

BS 8110-1:1997[3] recommends a minimum bearing width of 40 mm, except where wider bearings are necessary to control local stresses on concrete supports. However, it is not necessary to increase the net bearing width specifically to control stresses for precast concrete flooring supported on steelwork. A simple summation of the dimensional variations in the above list is permitted by the various standards for the components, but these would lead to a value for a nominal bearing width that could be below the minimum net bearing width. BS 8110 recognises that a global view of variations can be taken to minimise the possibility of unacceptably small bearing widths, as discussed below.

The National Structural Steelwork Specification[14] (sometimes referred to as the 'Black Book') specifies a maximum deviation of ± 20 mm in the spacing of the

beams on plan, but this is easily achieved in practice, and a more likely deviation is ±10mm. BS 8110 defines a tolerance in the manufactured length of the precast concrete units of ±12 mm for units up to 6 m length and ±18 mm for units up to 12 m length. It may be expected that the units will be placed on site within the tolerance of their manufacture.

It is assumed that the units can be positioned in such a way that the amount of bearing can be equalised on both supports, giving typical 'on site' variations of approximately ±15 mm per support for long-span (> 6 m) units (typically hollow core units) and ±10 mm for short span units.

BS 8110 Clause 5.2.3.6 recommends that the nominal bearing (as specified on the drawings) should take account of the effects of spalling and constructional inaccuracies, which include deviations in the setting out, the construction work on site as well as the manufacture and erection of prefabricated components. For prestressed units where the tendons are exposed in the end face of the unit (as is normally the case) and supported on steel beams, it is assumed that the support and the supported component will not be subject to spalling (BS 8110 Tables 5.1 and 5.2). Clause 5.2.4 recommends that under normal circumstances, a 9 m span unit supported on steelwork should have bearing widths that allow for a constructional inaccuracy of 27 mm (3 mm/m × 9 m), or 13.5 mm per bearing.

It is assumed that where a combination of dimensional variations results in an increase in bearing width, the units will be trimmed on site, as necessary, to ensure the minimum gap between unit ends, or to ensure clearance to the studs is maintained.

Table 3.1 presents recommended nominal bearing widths for units of various spans and depths, such that the minimum bearing width (calculated in accordance with the above), will not be less than 40 mm. Corresponding maximum bearing widths (based on nominal bearing width + 10 mm tolerances) are also given.

Table 3.1 *Bearing widths of precast units used in composite construction*

Span of unit	Depth of unit (typical)	Recommended nominal bearing width (mm)	Maximum bearing width (Nominal + Tolerance)* (mm)
3.75	75 mm	50	60
5 m	100 mm	55	65
6 m	150 mm	55	65
7.5 m	200 mm	55	65
10 m	≤ 260 mm	60	70

Minimum bearing width (taking account of all negative tolerances) is 40 mm.

* Subject to manufacturers' tolerances.

3.1.2 Hollow core slabs

For practical applications, the minimum gap between the ends of the units, as measured on site, should not be less than:

- 65 mm for site-welded shear connectors to allow sufficient space for the welding tool; and

- 50 mm for shop-welded shear connectors to allow for concrete placement around the shear connectors.

The minimum beam width should be equal to the minimum gap between the units, plus the <u>maximum</u> bearing widths, taking account of all positive tolerances (see Table 3.1). The <u>nominal</u> gap for detailing purposes is: the beam width minus twice the nominal bearing width.

These cases are illustrated in Figure 3.1 and Figure 3.2 for a hollow core slab of 150 mm depth. It follows that the <u>minimum</u> beam widths should be as given in Table 3.2. For the longest spanning hollow core units, the minimum beam sizes are therefore:

- 406 × 178 UB for beams with chamfered-ended units and shop-welded shear connectors (allowing for 2 mm differences).

- 457 × 191 UB for beams with square-ended units and shop-welded shear connectors.

- 533 × 210 UB for beams with site welded shear connectors, or where the decision on welding is not known at the design stage.

Beams with a narrower flange are not permitted, unless closer tolerances are specified, and are known to be achievable. It is not normally practicable to place shear connectors in pairs, except on wide flange beams used for long-span applications.

Table 3.2 *Minimum beam widths for different depths of hollow core concrete units*

Span of unit	Depth of unit (typical)	Minimum beam width (+) for use with:		
		Shop-welded studs		Site-welded studs
		Chamfered-ended units	Square-ended units	All cases
6 m	150 mm	180	190	195
7.5 m	≤ 200 mm	180	190	195
10 m	≤ 260 mm	190	200	205

(+) Minimum beam width is based on maximum bearing widths given in Table 3.1

70   Nominal gap

50   Minimum gap

120
(after welding)

Shear studs welded in factory
Tolerances ± 10 mm
10 mm maximum coarse
aggregate in situ concrete

40   Minimum bearing

55   Nominal bearing

65   Nominal bearing
+ tolerances

Minimum flange width = 190 mm

a) Pre-welded shear connectors

85   Nominal gap

65   Minimum gap

120
(after welding)

Shear studs welded on site
Tolerances ± 10 mm
10 mm maximum coarse
aggregate in situ concrete

40   Minimum bearing

55   Nominal bearing

65   Nominal bearing
+ tolerances

Minimum flange width = 195 mm

b) Site welded shear connectors

Figure 3.1 *End bearing and geometrical limitations for square-ended hollow core slabs with a maximum span of 6 m (see Table 3.1)*

70   Nominal gap

50   Minimum gap

85

Shear studs welded in factory
Tolerances ± 10 mm
10 mm maximum coarse
aggregate in situ concrete

120
(after welding)

235

40   Minimum bearing

55   Nominal bearing

65   Nominal bearing
      + tolerances

Minimum flange width = 180 mm

a) Pre-welded shear connectors

85   Nominal gap

65   Minimum gap

85

Shear studs welded on site
Tolerances ± 10 mm
10 mm maximum coarse
aggregate in situ concrete

120
(after welding)

235

40   Minimum bearing

55   Nominal bearing

65   Nominal bearing
      + tolerances

Minimum flange width = 195 mm

b) Site welded shear connectors

Figure 3.2 *End bearing and geometrical limitations for chamfered-ended hollow core slabs with a maximum span of 6 m (see Table 3.1)*

3.1.3   Solid planks

When solid planks are employed, the same dimensional limitations apply, depending on whether the shear connectors are shop-welded or site welded.

The gap between the units influences the efficiency of the shear connection because it is not possible to place transverse reinforcement low in the depth of the slab. The minimum beam flange width is therefore equal to the nominal gap plus twice the nominal bearing length, as illustrated in Figure 3.3.

It follows that the minimum beam widths should be as given in Table 3.3. For the longest spanning solid planks, the minimum beam sizes are therefore:
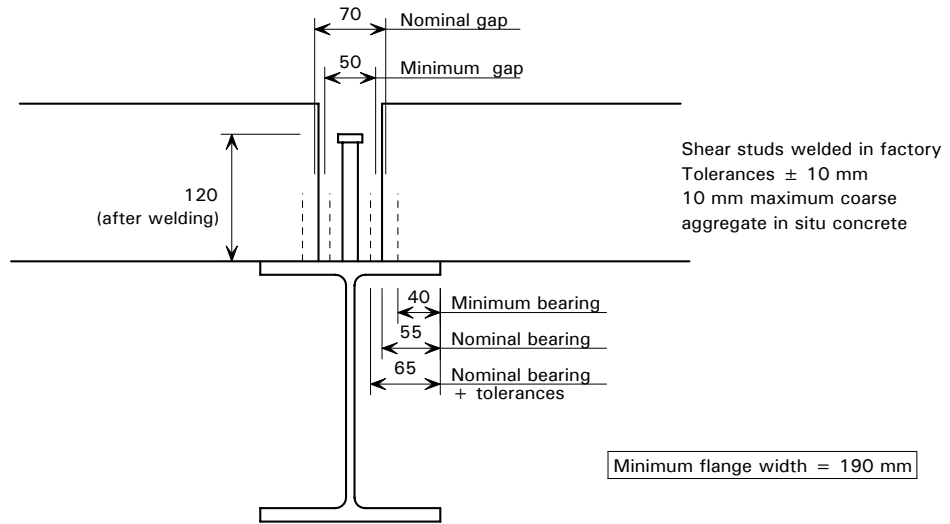
- 406 × 178 UB for beams with chamfered-ended units and shop-welded shear connectors (allowing for 2 mm differences).

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- 457 × 191 UB for beams with square-ended units and shop-welded shear connectors.

- 533 × 210 UB for beams with site welded shear connectors, or where the decision on welding is not known at the design stage.

Beams with a narrower flange are not permitted, unless closer tolerances are specified, and are known to be achievable. For deep solid planks, the ends to the units can be chamfered locally and the reinforcement cranked in the gap.

Table 3.3   *Minimum beam widths for different depths of solid plank concrete units*

Maximum span of unit	Depth of unit (typical)	Minimum beam width (+) for use with:	
		Shop-welded studs	Site-welded studs
3.75 m	75 mm	180	195
5 m	100 mm	190	205

(+) minimum beam width includes allowance for site tolerances
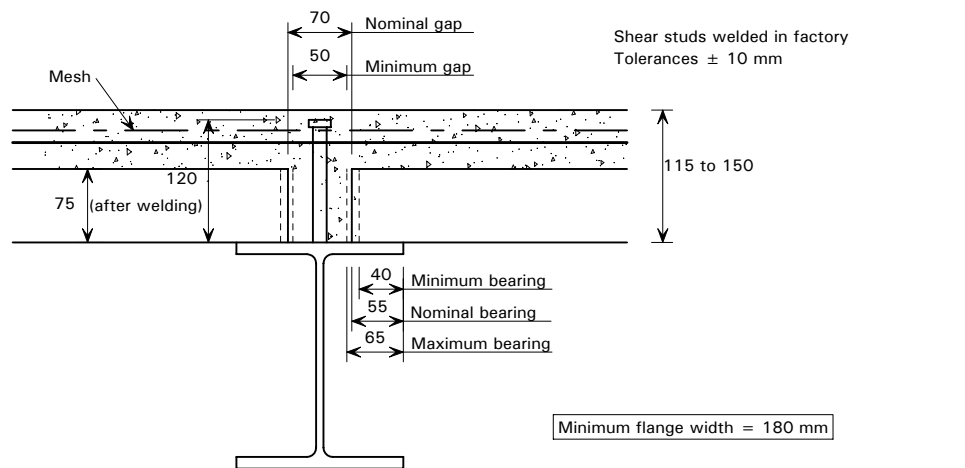
a) Geometrical limitations for shallow slabs using solid planks
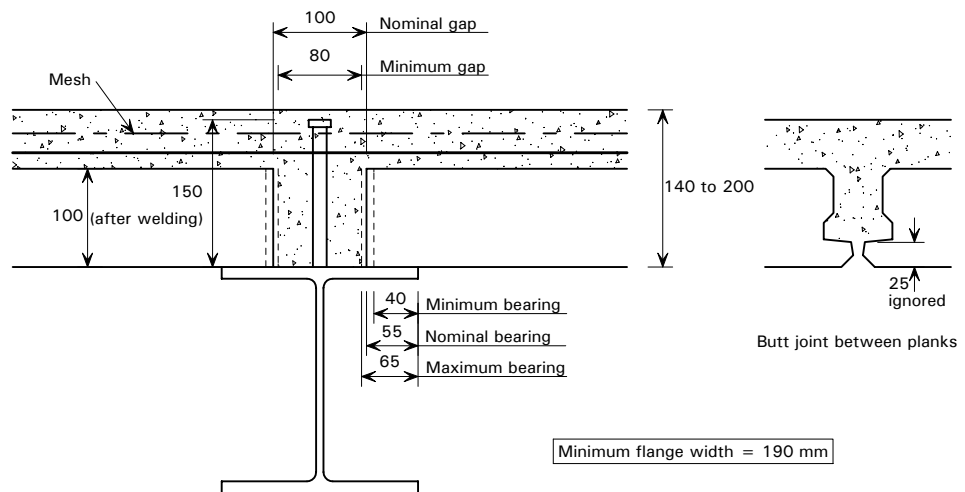
b) Geometrical limitations for deep slabs using solid planks

Figure 3.3   *End bearing and geometrical limitations for solid planks*

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3.2   Welding of shear connectors

Headed stud shear connectors of 19 mm diameter may be welded on site or in the factory, but larger diameter (22 or 25 mm) studs are only normally welded in the factory, because of the high electrical power input that is required.   In practice, most shear connectors are welded in the factory.

Shear connectors must be of sufficient height to project above the planks, or the reinforcement in the hollow core units, and so develop composite action with the *in situ* concrete.   There must also be sufficient space around the studs to allow for effective placement of the concrete, as their shear resistance is influenced by the gap between the ends of the precast units (see Section 4.4.3). The position of the studs can vary by 10 mm (relative to the steel beam) in any direction from the position shown on the drawings or given within the specification.

For welding on site, a generator is used with a local control unit.   A minimum gap of 65 mm is required in order to fit the welding gun between the concrete units.   In the case of welding on site, the top flange of the beam must be unpainted and free of moisture, dirt and mill-scale.

Welding in the factory is preferred, especially where the beam is to be galvanized or painted before delivery to site.   It is not necessary to remove the galvanized or paint coating from the shear connectors, although the top flange of the beam should be free of all coatings when the shear connectors are welded.

3.3   Factory preparation of the ends of hollow core units

3.3.1   Square-ended units

No special factory preparation is required for square-ended units.   In these circumstances, sawn-ended units may be used.

3.3.2   Chamfered-ended units

The ends of the hollow core units can be chamfered to achieve a smaller gap between the units.   This is normally carried out during the manufacturing process.

A chamfer removing a maximum of 85 mm from the top of the slab, over a horizontal length of 235 mm, is typical.   The formation of the taper should be carefully controlled to ensure that there is sufficient depth of slab left at the support to resist vertical shear forces that may be applied during construction, including those due to the weight of any *in situ* topping.

The shear connectors do not need to project above the chamfered ends, but sufficient transverse reinforcement (see Section 3.4) must be placed below the level of the heads of the shear connectors.

3.3.3   Opened hollow cores

The tops of a specified number of hollow cores (usually three or four per unit end) should be opened up so that transverse reinforcement may be placed within them.   Typically, this opening up operation is carried out during manufacture.

The opening of two adjacent cores should be avoided, as it is difficult to preserve the integrity of the chamfered rib between them. It is advisable not to open the outer core for a similar reason. Also, the outer rib is liable to slump, thereby making it vulnerable to damage during handling and erection.

The void at the back of each opened core is blocked with concrete during manufacture; the other cores are normally blocked using a polystyrene bung. For shallow, chamfered-ended units, the ends of the other cores may be blocked with concrete during the formation of the chamfered ends.

The layout of the units should be planned to ensure that the opened cores are reasonably aligned, in order to allow correct placing of the transverse reinforcement bars.

3.4   Placing of transverse reinforcement

Reinforcing bars are placed in the opened hollow cores, perpendicular to the longitudinal axis of the beam. For good composite action, they must be located at least 15 mm below the head of the shear connectors (BS 5950-3: 1990 Clause 5.6.5) which, for general applications, are usually 125 mm long. Since the base of a core is normally between 30 and 40 mm above the soffit, this requirement is not critical for hollow core units.

The minimum recommended bar sizes, for transverse reinforcement, are shown in Table 3.4.

Table 3.4   *Recommended bar sizes for transverse reinforcement*

Slab depth	Bar sizes
Solid Planks	T10 @ 300 mm centres plus A142 mesh reinforcement
Hollow Core Units (up to 200 mm deep)	T12 @ 200 to 350 mm centres*
Hollow Core Units (up to 260 mm deep)	T16 @ 200 to 350 mm centres

* 16 mm diameter bars should be provided if partial shear connection is used.

The spacing of the bars may be adjusted to suit the hollow cores, so that bars can be placed in alternate cores. The spacing between the bars should not exceed 350 mm. The shear connectors are often placed at 120 to 225 mm centres along the beam, and so do not align directly with these bars (see Figure 3.4). Lacer bars are often required to support the bars at the correct height (above the base of the core, allowing space for infill concrete). The length of the transverse reinforcement should be at least 1000 mm, plus the gap width, so that it provides sufficient anchorage in the filled hollow cores (see Figure 3.5). However longer straight bars, or L-bars may be necessary for fire conditions (See Section 6).

For deep solid planks, bars may be bent down below the head of the shear connectors, and may be detailed to coincide with the stud spacing. For shallow solid precast planks, mesh reinforcement may be used in addition to the bar reinforcement if needed.
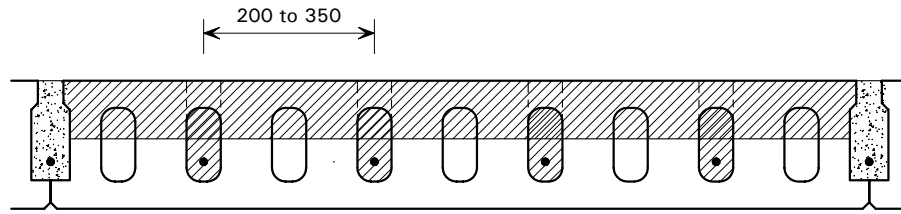
a) Longitudinal view of filled hollow cores
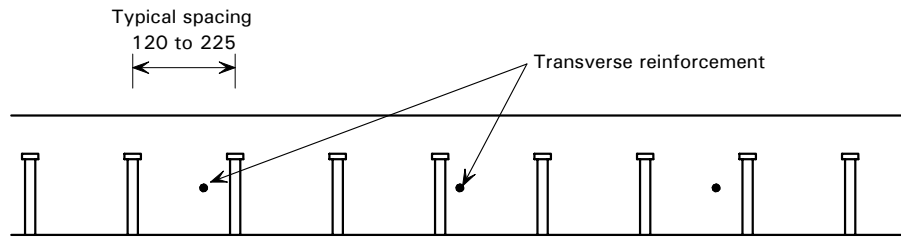
b) Longitudinal view of shear connectors

Figure 3.4 *Longitudinal view of transverse reinforcement*

<u>Note</u>: The above recommendations on bar size and spacing of transverse reinforcement are based on test specimens using hollow core units with cores at a particular range of pitches (see Appendix A) It may be possible to increase the spacing of the transverse reinforcement if justified by tests that demonstrate adequate resistance and deformation capacity.
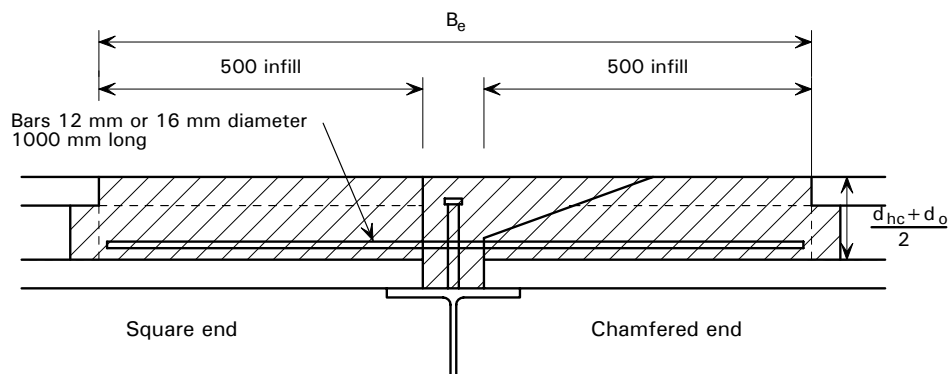
a) Cross-section at filled hollow core
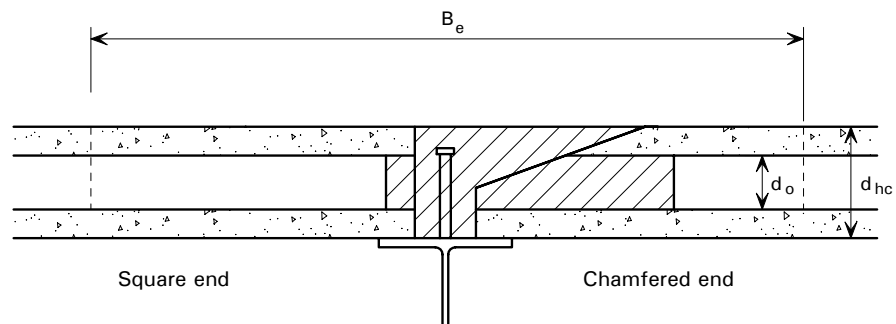
b) Cross-section at unfilled hollow core

Figure 3.5 *Cross-section through the hollow cores (shaded area indicates extent of concrete infill)*

3.5 Detailing of edge beams

Edge beams require special consideration because:

- They are normally required to act as peripheral ties.

- They often transfer diaphragm forces into vertical bracing.

- Cladding attachments can cause eccentricity of loadings.

- Deflection limits are often stricter than for internal beams.

For practical purposes, edge beams are normally designed as non-composite, in order that a similar section size to that used for the (composite) internal beams may be employed. However, in these cases, sufficient tying action must still be provided in order for these members to act as peripheral ties and to transfer in-plane forces.

Should a composite design of an edge beam be desired, in the absence of experimental data, comparison with conventional composite applications suggests that a minimum edge distance of the shear connectors, and sufficient transverse reinforcement must be provided. BS 5950-3:1990[1] requires an axis distance (distance from the centroid of the stud to the free edge of the slab) of at least six times the diameter of the shear connectors in order to ensure effective composite action (see Figure 3.6). This corresponds to 115 mm for 19 mm diameter studs. Therefore, when the flange of the steel beam is flush with the edge of the slab, and 19 mm diameter shop-welded stud connectors are used, the minimum beam width is 210 mm (i.e., the nominal bearing + the gap + the axis distance of 115 mm); this corresponds to a 533 × 210 UB. The beam size may be reduced if the slab projects over the edge of beam (although this requires shuttering).

U-bars are placed around the studs to provide effective transverse reinforcement and tying action (see Figure 3.7). These U-bars should be of 12 mm minimum diameter and should be anchored in each filled hollow core.

Edge beams of the configuration shown in Figure 3.6(a) must be laterally restrained during construction[15]. In these circumstances, it may be necessary to consider the effects of torsion on the design of the edge beams (see the guidance given in Section 4.1). However, in the normal condition, composite action with the slab ensures that torsional effects do not increase when the composite beam is later subjected to imposed loading. Cladding loads generally counteract the torsional effect. Attachments for the cladding should generally be made to the steel beam rather than to the *in situ* concrete.

For the special case of a non-composite beam in which the precast unit is supported by the entire width of the steel flange (Figure 3.6(b)), full lateral restraint is provided, and torsional effects may be ignored [15]. A 300 mm section of the slab is broken-out to facilitate the shear connectors and placing of U-bars.
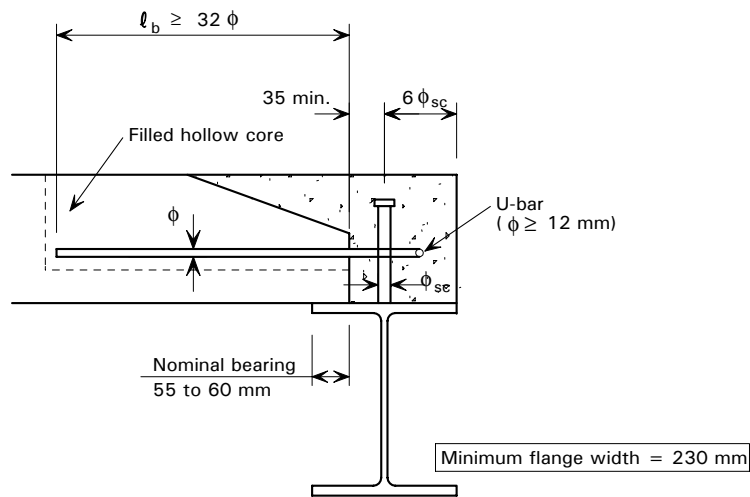
a) Composite edge beam
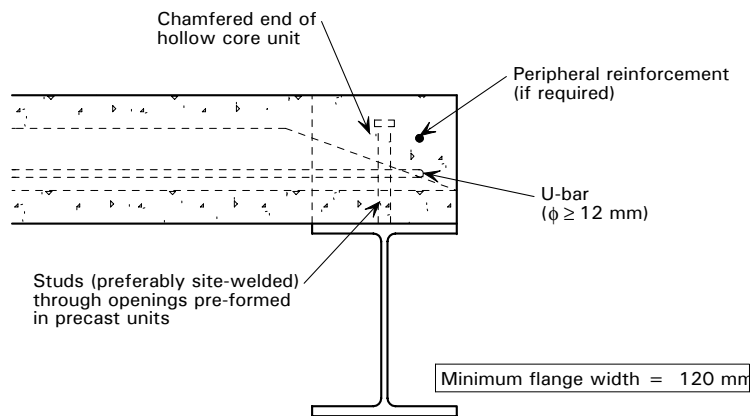
b) Non-composite edge beam
   (as peripheral ties)

Figure 3.6 *Detailing of composite and non-composite edge beams*

Figure 3.7 *U-bars placed around the studs to an edge beam*

3.6 Temporary stability

The stability of the steel beams during the erection of the floor units, and the placement of the structural topping, must be considered. The designer should take due account of the floor erection process (which will usually require erection in 'bays', to avoid excessive re-siting of the crane). Should a particular sequence of erection or temporary support be necessary, this should be noted in the specification and on the drawings.

The placement of the precast concrete units should be carefully controlled in order that out-of-balance loads are kept within the limits assumed in the beam design (see Section 4.1). Edge beams and beams around openings, with the configuration shown in Figure 3.6(a), should be designed for combined bending and torsion at the construction stage.

The treatment of combined bending and torsion at the construction stage is discussed in Section 4.1.

Temporary stability may be achieved by placing ties between the compression flanges of the beams at a minimum spacing of 40 $\times$ beam flange width for UBs (normally at approximately 6 m). Ties between the tension flanges are insufficient to prevent torsion unless combined with a U-frame or other measures (see Figure 3.8).

When loads on either side of the beam are 'balanced', it may be assumed that the beam is fully laterally restrained for spans less than, or equal to, 160 $\times$ precast unit bearing width[15], because of the restoring effect of the slab and the friction between the slab and beam (as illustrated in Figure 3.9). Furthermore, for cases when the width of the top flange to the beam is such that a large gap between the ends of the hollow core units occurs (particularly in cases where shear connectors are to be site-welded), it is recommended that the joints along the sides of the units be grouted after each unit has been correctly positioned; this is to ensure that the possibility of accidental damage arising from the installation of the adjacent unit is minimised.

a) Lateral restraint to top flange   b) Lateral restraint to both flanges by U-frame action
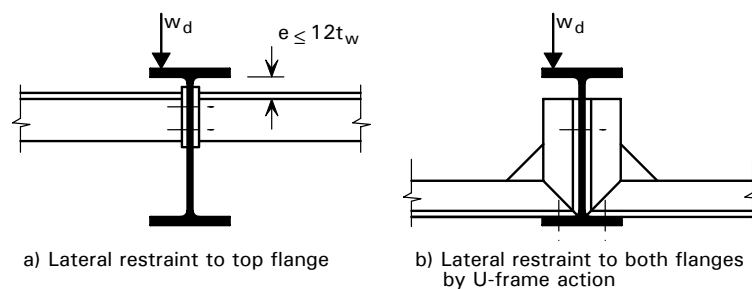
Figure 3.8 *Lateral restraint to beams during construction*
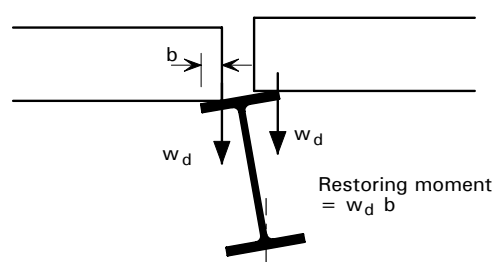
Figure 3.9 *Restoring moment due to balanced loading*

4   DESIGN OF COMPOSITE BEAMS

The design of composite beams should commence by checking the bare steel sections for the torsional and bending moments developed during the construction condition. Having established the adequacy of the steel section for this stage, the composite design should follow the general procedure presented in BS 5950-3:1990[1], taking account of:

- Effective width of the slab.

- Plastic bending resistance.

- Shear connection (and minimum degree of shear connection).

- Transverse reinforcement.

- Serviceability requirements.

4.1   Construction condition

The steel beams are designed for two distinct stages in the construction condition:

- Out-of-balance loads acting on the beam due to the sequence of installation of the precast units. This stage considers the self-weight of the units.

- Balanced loads, when all the precast units are installed (assuming they are of equal span on either side of the beam). This stage considers the construction load, together with the self-weight of the units and the weight of the concrete topping (if used).

4.1.1   Combined bending and torsion

For the case when the sequence of installation produces an out-of-balance load (see Figure 4.1), the supporting beam will be subject to combined bending and torsion. A graphical representation of the internal forces in an I-beam, arising from the out-of-plane loading, is shown in Figure 4.2.

In these circumstances, the ultimate limit state design criteria for the bare steel section are:

(i)   resistance to buckling,

(ii) local capacity, and

(iii) resistance to shear stresses from torsion and warping.
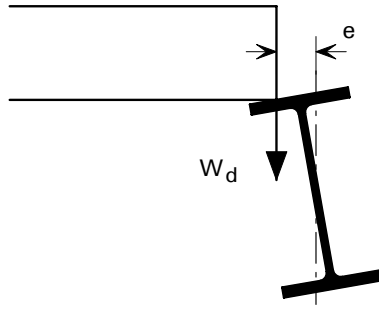
Figure 4.1 *Out-of-balance loading from the installation of a precast unit on one side of the beam flange*

(a) Rotation of cross-section   (b) Torsion and warping stresses
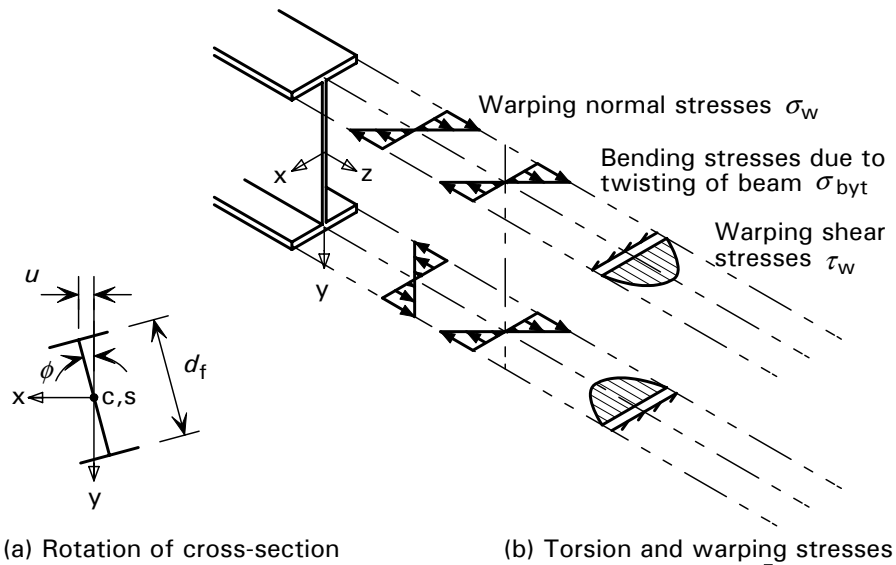
Figure 4.2 *Warping stresses in an I-beam*

For the buckling check, the following criterion should be satisfied[16]:

$$\frac{\overline{M}_x}{M_b} + \frac{(\sigma_{byt} + \sigma_w)}{p_y}\left[1 + 0.5\frac{\overline{M}_x}{M_b}\right] \leq 1.0 \tag{1}$$

where:

$\overline{M}_x$ is the equivalent uniform moment given by: $\overline{M}_x = m_{LT}M_x$, in which m_{LT} is as given by Clause 4.3.6.6 to BS 5950-1

M_x is the applied major-axis bending moment

M_b is the buckling resistance moment of the beam between restraints

σ_{byt} is the bending stress in the flange tips given by: $\sigma_{byt} = M_{yt}/Z_y$

M_{yt} is the minor-axis bending arising from torsional deformations, given by: $M_{yt} = \phi M_x$

Z_y is the elastic modulus about the minor-axis of the steel section

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ϕ is the total angle of twist at the transverse section of the beam (in radians)

σ_w is the warping normal stress given by: $\sigma_w = -EW_{n0}\phi''$

E is the Young's modulus of elasticity for steel

W_{n0} is the normalised warping function at the flange tips which, for symmetrical I-sections, is given by: $W_{n0} = hB/4$

h is the depth of the I-beam between the centres of the flanges ($h = D - T$).

ϕ'' is the second derivative of ϕ with respect to z.

z is the distance from the left-hand support to the section under consideration.

p_y is the yield stress of the steel.

The local capacity of the section should be checked, using the following criterion:

$$\sigma_{bx} + \sigma_{byt} + \sigma_w \leq p_y \qquad\qquad (2)$$

where:

σ_{bx} is the major-axis bending stress given by $\sigma_{bx} = M_x / Z_x$

Z_x is the elastic modulus about the major-axis of the steel section.

Although seldom critical in symmetrical I-beams, the shear resistance of the steel section should also be checked for the shear stresses due to bending, plus the additional shear stresses from torsion and warping. In these circumstances, the following criterion should be satisfied:

$$\tau_b + (\tau_t + \tau_w)(1 + 0.5\overline{M}_x / M_b) \leq 0.6 p_y \qquad\qquad (3)$$

where:

τ_b is the shear stress due to plain bending

τ_t is the shear stress due to pure torsion

τ_w is the shear stress due to warping (see Figure 4.2).

In the majority of cases, for composite beams using precast units, the steel section will be subjected to a uniform torque (due to the units being installed on one side of the beam), with the beam-ends torsion fixed, warping free. For this special case, the angle of twist and its derivatives may be evaluated using the methodology given below, which is based on the SCI publication P057 *Design of members subject to combined bending and torsion*[16]. In the rare occurrences where the applied torque is not uniform (e.g. point loads occurring on a primary beam due to the installation of solid precast planks on secondary beams), or when other boundary conditions are imposed on the beam-ends, guidance may be found from the above publication[16].

The torsional bending constant is given by:

$$a = \left(\frac{EH}{GJ} \right)^{0.5}$$

(4)

where:

H is the warping constant

J is the torsional constant.

The angle of twist and its derivatives may be found from Table 4.1, according to the ratio of the clear span to the torsional bending constant L/a.

Table 4.1   *Torsional functions for beams with a uniform torque and ends torsion fixed, warping free*

L/a	$\dfrac{\phi GJ}{T_q a}$	$-\dfrac{\phi'' GJa}{T_q}$
0	0	0
0.5	0.002	0.061
1	0.012	0.113
1.5	0.036	0.152
2	0.074	0.176
2.5	0.124	0.188
3	0.183	0.192
3.5	0.248	0.189
4	0.316	0.184
4.5	0.387	0.176
5	0.458	0.167
5.5	0.529	0.159
6	0.600	0.150
8	0.880	0.120
10	1.151	0.099
12	1.417	0.083
14	1.679	0.071
16	1.938	0.062
18	2.194	0.056
20	2.450	0.050
22	2.705	0.045
24	2.958	0.042
26	3.212	0.038

For intermediate values of *L/a*, linear interpolation is permitted.

This design case is usually more critical than the balanced (symmetrical) load case for a standard UB. For heavily perforated sections, the removal of a large proportion of the web material will significantly reduce the torsional constant *J*, thereby increasing the torsional bending constant *a*. In these circumstances, additional temporary restraints may be necessary to reduce the effective length of the member.

4.1.2 Buckling resistance

For the final part of the sequence of installation, providing the spacing of the beams is equal, the load from the precast units on each side of the beam flange (together with the load from the topping and imposed construction loads) produces a balanced load. In these circumstances, the ultimate limit state design criteria for the bare steel section is:

(i)   lateral torsional buckling resistance; and

(ii) moment capacity.

The treatment of lateral torsional buckling is subtly different in BS 5950-1: 2000[17] from that presented in the 1990 version of BS 5950. The effective slenderness of a beam for lateral torsional buckling is now given by:

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$ (5)

In which

$$\lambda = L_E / r_y$$

where:

L_E is the effective length for lateral torsional buckling

r_y is the radius of gyration about the minor axis

u is the buckling parameter

β_W is a ratio that depends on the section classification and may be taken as:
    =   1.0   for Class 1 and 2 sections
    = Z_x/S_x for Class 3 sections.

The slenderness factor, v, may be determined from Table 19 of BS 5950-1: 2000 as a function of λ/x and η or is given for mono-symmetric I-sections, by:

$$v = \left[\left(4\eta(1-\eta) + 0.05(\lambda/x)^2 + \psi^2 \right)^{0.5} + \psi \right]^{-0.5}$$

where:

η = $I_{yc}/(I_{yc} + I_{yt})$

ψ is the monosymmetry index

I_{yc} is the second moment of area of the compression flange about the minor axis

I_{yt} is the second moment of area of the tension flange about the minor axis

x is the torsional index, which may be approximated by the ratio of beam depth to flange thickness, D/t.

The effective slenderness λ_{LT} is used to determine the bending strength of the beam, p_b, as in Table 16 of BS 5950-1:2000.

The buckling resistance moment is given by:

Class 1 or 2 sections: $\qquad M_b = p_b S_x$

Class 3 sections: $\qquad M_b = p_b Z_x$

where S_x and Z_x are the plastic and elastic moduli of the section, as defined earlier.

For cases when the design is governed by lateral torsional buckling of the steel beam, the maximum span can be increased by following the logic that, as the beam buckles, a restoring moment develops from the couple between the precast unit reactions (see Figure 3.9). In these circumstances, it may be assumed that the beam is fully laterally restrained for spans less than, or equal to, $160 \times$ precast unit bearing width [15] (typically about 8 m).

As well as lateral torsional buckling considerations, the moment capacity M_c of the bare steel section should also be checked.

For cases of high shear ($F_v \geq 0.6 P_v$ i.e., beams subjected to point loads), the moment capacity should be taken as the smaller of:

Class 1 or 2 sections: $\qquad M_c = p_y \left(S_x - \rho S_v \right)$

Class 3 sections: $\qquad M_c = p_y \left(Z_x - \rho S_v / 1.5 \right)$

where:

$\quad S_x \quad$ is the plastic modulus of the section, as defined earlier

$\quad Z_x \quad$ is the elastic modulus of the section, as defined earlier

$\quad S_v \quad$ is the plastic modulus of the shear area A_v (for sections with equal flanges)

$\quad \rho \quad = \left[2 \left(F_v / P_v \right) - 1 \right]^2$

$\quad F_v \quad$ is the applied shear force

$\quad P_v \quad$ is the shear capacity, given by $P_v = 0.6 p_y A_v$

$\quad A_v \quad$ is the shear area (taken as tD for UB and UC sections, with the applied load parallel to the web).

4.1.3 Non-uniform moment

The design moment, for consideration of buckling between positions of adjacent lateral restraints, is the maximum moment within this part of the span, M_{max}, multiplied by the equivalent uniform moment factor, m_{LT}. The value of m_{LT} may be calculated by considering the moment at equidistant points between these restraints. Expressed as a function of the maximum moment, M_{max}, the moment variation factor is given in Table 18 of BS 5950-1:2000[17] by a modification of Simpson's Rule, as follows:

$$m_{LT} = 0.2 + \frac{0.15 M_2 + 0.5 M_3 + 0.15 M_4}{M_{max}} \qquad (6)$$

and $1.0 \geq m_{LT} \geq 0.44$

The location of these moments is illustrated in Figure 4.3. The design moment for buckling checks is given by:

$$\overline{M} = m_{LT} M_{max} \tag{7}$$

For an acceptable design in terms of lateral torsional buckling:

$$\overline{M} \le M_b$$

where M_b is determined for a uniform moment, as presented in Section 4.1.2.
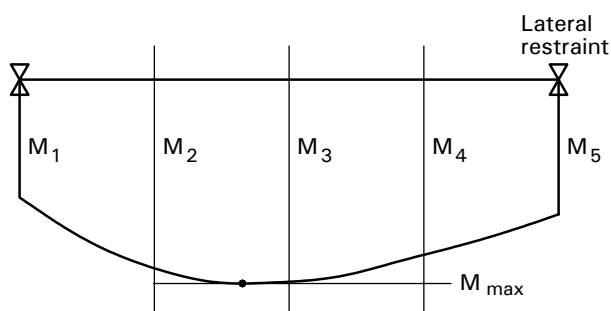
Figure 4.3 *General case for calculation of moment variation factor*

4.1.4 Serviceability conditions

As well as calculating the vertical deflection of the beam under unfactored balanced loading, for later consideration in the total deflection check (see Section 4.6.2), the angle of twist of the beam under out-of-balance loading should also be calculated.

It is recommended[16] that the following criterion should be satisfied for the angle of twist under out-of-balance working loads:

$$\phi \le 2^{\circ} \ (0.035 \ \text{radians}) \tag{8}$$

4.2 Effective slab width for composite action

In most composite applications, the effective width of a solid slab is taken as span/4 (but not exceeding the beam spacing). However, due to the fact that the presence of the hollow core units no longer makes the slab construction monolithic, the effective width will be smaller than for slabs using *in situ* concrete. The design implications for the effective width, when using hollow core units, or solid planks, are discussed below.

4.2.1 Hollow core slabs

For hollow core units, the strength of the *in situ* concrete, and the amount of transverse reinforcement bars provided, will strongly affect the effective width of the slab that may be considered in the composite beam design. From the recent research work[18] the following equation for the effective width of the slab should be used for composite beams using hollow core units:

$$B_e = \frac{L}{8} \ \text{but not greater than the total width of the concrete infill} + g \tag{9}$$

where:

   L is the clear span of the composite beam.

g is the gap width between the ends of the hollow core units.

The effective width is reduced by half for edge beams, or beams around openings.

4.2.2 Solid planks

Due to the fact that more *in situ* concrete is used with solid planks, the effective width may be calculated in the same way as for a composite beam using solid concrete slabs:

$$B_e = \frac{span}{4} \quad \text{but not greater than } b \tag{10}$$

where b is the beam spacing, as defined earlier.

The solid plank may be considered to act in compression when the edge details permit transfer of compression through the *in situ* concrete. In this case, a minimum of 25 mm of the slab depth should be deducted because of the lack of concrete at the interface between the planks (see Figure 3.3(b)).

If there is a butt joint detail between the planks, this does not achieve effective compression transfer, and the depth of the slab should be taken as equal to the depth of the concrete topping.

4.3   Plastic bending resistance

In accordance with BS 5950-3:1990, the plastic moment resistance for I-sections with equal flanges is expressed in terms of the resistance of various elements of the composite beam, as follows:

Resistance of steel section, $R_s = A p_y$

Resistance of steel flange, $R_f = B T p_y$

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

Resistance of clear web depth, $R_v = d t p_y$

Resistance of slender web, $R_o = 38 \varepsilon t^2 p_y$

Resistance of slender steel beam, $R_u = R_s - R_v + R_o$

Resistance of shear connection, $R_q = N_a Q_p$

Resistance of concrete flange, $R_c = 0.45 f_{cu} B_e D_s$

where:

   A is the area of the steel beam

   B is the width of the steel flange

   B_e is the effective width of the concrete flange (from Section 4.2.1 or 4.2.2)

D	overall depth of steel beam
D_s	is the overall depth of the concrete flange (including the structural topping, if used). For solid planks, a minimum of 25 mm should be deducted from the slab depth (see Section 4.2.2)
d	is the clear depth of the steel web
f_{cu}	is the characteristic strength of the *in situ* concrete infill
M_s	plastic moment resistance of the steel section
N_a	is the actual number of shear connectors within the positive moment region (minimum number, one side of the point of maximum moment)
p_y	is the design strength of the steel beam
Q_p	is the design capacity of the shear connectors within the positive moment region (taken as 80% of the characteristic resistance Q_k, from Table 4.2, multiplied by the reduction factor k as appropriate)
T	is the thickness of the steel flange
t	is the thickness of the steel web
ε	$= (275/p_y)^{0.5}$

The plastic bending resistance of the composite beam is calculated from the position of the plastic neutral axis (PNA), which may lie within:

a)   the concrete slab,

b)   the steel flange, or

c)   the steel web.

The position depends on the relative magnitudes of the tensile resistance of the steel beam R_s and the compressive resistance of the concrete slab R_c. For full shear connection, these cases are illustrated in Figure 4.4.
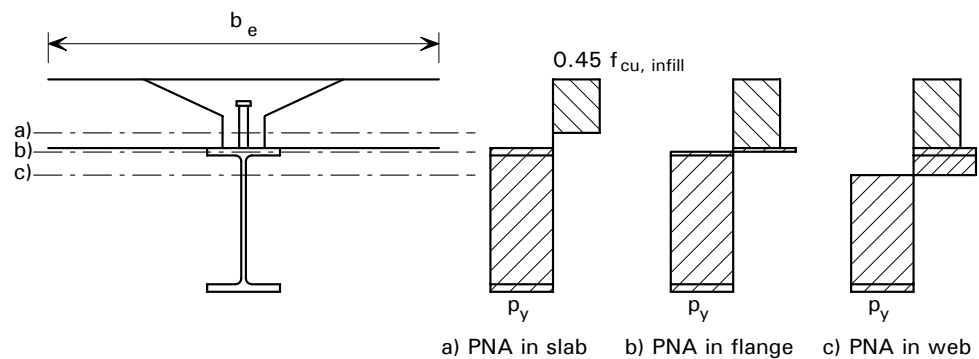
Figure 4.4   *Plastic analysis of composite section*

4.3.2 Full shear connection

Full shear connection applies when R_q is greater than, or equal to, the lesser of R_c and R_s. For cases when the steel section has *equal* flanges, the plastic moment capacity M_c is given by the following:

Case (a) $R_c \geq R_w$ *and* $R_s \leq R_c$ *(plastic neutral axis in concrete slab)*

$$M_c = R_s \left\{ \frac{D}{2} + D_s - \frac{R_s}{R_c} \frac{D_s}{2} \right\} \tag{11}$$

This case is *not* permitted when hollow core units are used. In these circumstances, it is necessary to increase the size of the steel beam to cause the plastic neutral axis to fall within the steel section. If this is not practical, an alternative solution may be to provide partial shear connection (see Section 4.3.3).

Case (b) $R_c \geq R_w$ *and* $R_s > R_c$ *(plastic neutral axis in flange of steel beam)*

$$M_c = R_s \frac{D}{2} + R_c \frac{D_s}{2} - \frac{(R_s - R_c)^2}{R_f} \frac{T}{4} \tag{12}$$

Case (c) $R_c < R_w$ *(plastic neutral axis within web)*

(i) $\quad \dfrac{d}{t} \leq 76\varepsilon \text{ or } \dfrac{d}{t} \leq \dfrac{76\varepsilon}{1 - R_c / R_v} \quad$ (web Class 1 or Class 2)

$$M_c = M_s + R_c \left(\frac{D + D_s}{2} \right) - \frac{R_c^2}{R_v} \frac{d}{4} \tag{13}$$

(ii) $\quad \dfrac{d}{t} > \dfrac{76\varepsilon}{1 - R_c / R_v} \quad$ (web Class 3)

$$M_c = M_s + R_c \left(\frac{D + D_s}{2} \right) - \left(\frac{R_c^2 + (R_v - R_c)(R_v - R_c - 2R_o)}{R_v} \right) \frac{d}{4} \tag{14}$$

4.3.3 Partial shear connection

Partial shear connection applies when R_q is less than both R_c and R_s. For cases when the steel section has equal flanges, the plastic moment capacity M_c is given by the following:

Case (d) $R_q \geq R_w$ *(plastic neutral axis in flange of steel beam)*

$$M_c = R_s \frac{D}{2} + R_q D_s \left(1 - \frac{R_q}{2R_c} \right) - \frac{(R_s - R_q)^2}{R_f} \frac{T}{4} \tag{15}$$

Case (e) $R_q < R_w$ (plastic neutral axis within web)

(i) $\dfrac{d}{t} \le 76\,\varepsilon$ or $\dfrac{d}{t} \le \dfrac{76\,\varepsilon}{1 - R_q / R_v}$ (web Class 1 or Class 2)

$$M_c = M_s + R_q \left\{ \frac{D}{2} + D_s \left(1 - \frac{R_q}{2R_c} \right) \right\} - \frac{R_q{}^2}{R_v} \frac{d}{4} \tag{16}$$

(ii) $\dfrac{d}{t} > \dfrac{76\,\varepsilon}{1 - R_q / R_v}$

$$M_c = M_s + R_q \left\{ \frac{D}{2} + D_s \left(1 - \frac{R_q}{2R_c} \right) \right\}$$
$$- \frac{R_q{}^2 + \left(R_v - R_q \right)\left(R_v - R_q - 2R_o \right)}{R_v} \frac{d}{4} \tag{17}$$

Additionally, the shear connection should satisfy the limits given in (21) below and, where hollow core units are used, the degree of shear connection should satisfy the limits given by Section 4.4. Also, when hollow core units are used, the minimum diameter of the transverse reinforcement bars should be 16 mm (Section A.1.2).

4.4   Shear connection

4.4.1   Shear resistance

The resistance of the shear connection, in sagging moment regions, is given by:

$$R_q = N_a Q_p = 0.8 N_a Q_k k \tag{18}$$

where:

N_a is the actual number of shear connectors provided from the support to the point of maximum moment.

Q_k is the characteristic resistance of the shear connectors based on the strength of the *in situ* concrete infill (see Table 4.2).

k is a reduction factor due to the geometry of the precast units and, for hollow core units, the amount of transverse reinforcement provided (see Section 4.4.3).

Table 4.2 *Characteristic resistance of headed stud shear connectors Q_k in normal weight concrete (taken from Table 5 of BS 5950-3: 1990)*

Diameter of shear connectors	Nominal height of shear connectors	Characteristic cube strength of concrete (N/mm²)			
		25	30	35	40
mm	mm	kN	kN	kN	kN
19*	100	95	100	104	109
19*	75	82	87	91	96
22	100	119	126	132	139
25	100	146	154	161	168

* Recommended diameters for site welded shear connectors

For connectors of heights greater than tabulated, use the values of the greatest height tabulated

For concrete of characteristic strength greater than 40 N/mm², use the values for 40 N/mm².

The factor of 0.8 takes account of the non-uniform shear flow along the beam.

The reduction factor on the design resistance of the shear connectors, k takes account of the influence of:

- The gap width between the ends of the precast units.

- The level of confinement to the concrete around the shear connectors, from the transverse reinforcement provided in the opened cores of the hollow core units.

- The variation of force through wide hollow core units.

Guidance on evaluating k is given in Section 4.4.3.

4.4.2 Partial shear connection

Partial shear connection exists when the resistance of the shear connection provided R_q is less than R_c and R_s. In these circumstances, the degree of shear connection is given by:

$$K = \frac{R_c}{R_q} \text{ for } R_c < R_s \tag{19}$$

$$K = \frac{R_s}{R_q} \text{ for } R_s < R_c \tag{20}$$

For a steel section with equal flanges, BS 5950-3:1990[1] places a strict limit on the degree of shear connection as a function of the span of the beam, according to:

$$K \geq \frac{L-6}{10} \text{ and } 1.0 \geq K \geq 0.4 \tag{21}$$

where: L is the beam span (m).

This same formula is taken to apply to the use of precast units, as to solid or composite slabs, provided that due account is taken on the influence of the

transverse reinforcement and the geometry of the studs relative to the precast units (see Section 4.4.3). Furthermore, to ensure that the shear connection has adequate deformation capacity, 16 mm diameter high tensile transverse reinforcement bars should be used when hollow core units are employed.

For cases when solid planks are employed, no special consideration need be made on the size of the transverse reinforcement bars. However, it is recommended that the minimum sizes given in Table 3.4 should be observed.

The limits in (21) are based on parametric studies on composite beams using symmetrical steel sections[19]. When asymmetric steel sections are employed, the above equation can become unconservative, particularly for beams with low degrees of shear connection. This effect is recognised in Eurocode 4[2], which presents different shear connection limits as a function of the ratio of the steel section's bottom flange area to its top flange area.

4.4.3 Reduction factor

Hollow core units

For hollow core units, the reduction factor, k, to take account of the influence of the confinement of the shear connectors from the transverse reinforcement and the geometry of the connectors relative to the hollow core units, is based on empirical results of push-out tests by Lam *et al.*[20] (see Section A.1.2). The reduction factor is given by:

$$k = \beta \varepsilon \sqrt{\omega} \leq 1.0 \tag{22}$$

in which:

β is the gap width factor, which is given by:

$$\beta = \frac{g + 70}{140} \text{ for } 70 \geq g \geq 50 \text{ mm}$$

ε is the stud confinement factor, which is given by:

$$\varepsilon = \frac{\phi + 20}{40} \text{ for } 20 \geq \phi \geq 8 \text{ mm}$$

ω is the transverse joint factor, which is given by:

$$\omega = \frac{w + 600}{1200} \text{ for } 1200 \geq w \geq 600 \text{ mm}$$

where:

g is the gap width

ϕ is the transverse reinforcement diameter

w is the width of the hollow core unit.

Solid planks

From comparisons with push tests results[21], Equation (22) may also be used to calculate the reduction factor for studs embedded in slabs using solid planks. However, only the effect of the gap between the solid planks need be considered, by calculating the gap width factor, β (see Section A.1.3). Therefore, the reduction factor is given by:

$$k = \beta \ngtr 1.0 \tag{23}$$

4.4.4 Minimum spacing of shear connectors

The minimum longitudinal spacing of shear connectors is $5d$ (where d is the diameter of the shear connectors) i.e. 95 mm for 19 mm diameter shear connectors.

For wide flange beams, it may be possible to place shear connectors in pairs, in which case, their minimum transverse spacing is $4d$ (or approximately 80 mm for 19 mm diameter studs). The minimum beam width then becomes 260 mm for use of shop-welded shear connectors (see Section 3.1.2). For solid planks with shop-welded shear connectors, the minimum width becomes 260 mm for thin planks, and 270 mm for planks of 100 mm depth.

4.5 Transverse reinforcement

Transverse reinforcement should be placed within the depth of the shear connectors so that longitudinal splitting of the concrete is controlled.

For full shear connection, the longitudinal shear force is given by:

$$v = R_s / s \text{ or } R_c / s \text{ whichever is the lesser} \tag{24}$$

where s is the longitudinal spacing of the shear connectors

For partial shear connection, the longitudinal shear force is given by:

$$v = N_a Q_p / s \tag{25}$$

For any surface of potential shear failure within the concrete flange, the longitudinal shear force per unit length v, should not exceed the shear resistance v_r, given by the semi-empirical design formula in BS 5950-3: 1990[1] as follows:

$$v_r = 0.03 A_{cv} f_{cu} + 0.7 A_{sv} f_y \text{ but } v_r \leq 0.8 A_{cv} \sqrt{f_{cu}} \tag{26}$$

where:

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

A_{sv} is the cross-sectional area of the transverse reinforcement crossing the shear surface, per unit length

f_{cu} is the characteristic cube strength of the *in situ* concrete infill.

f_y is the design strength of the reinforcement

For an internal beam, two possible shear surfaces exist: a-a and b-b (see Figure 4.5). For edge beams, which have been designed compositely, only surface a-a need be considered. Placing sufficient transverse reinforcement should prevent splitting, so that v_r exceeds the longitudinal shear force per unit length.
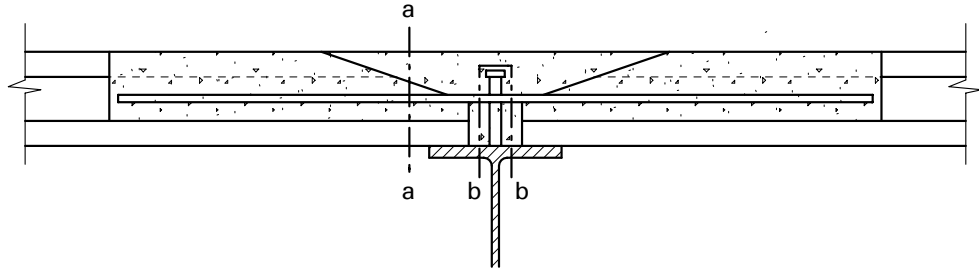
Figure 4.5 *Potential shear failure surfaces for transverse reinforcement considerations*

The length of the transverse reinforcement bars should extend over the effective width of the slab, B_e. For edge beams, U-bars are placed around the shear connectors to give proper end anchorage.

The suggested minimum bar sizes are given in Section 3.4. It is not necessary for the bars to align with the shear connectors. For cases when solid planks are employed, mesh reinforcement within the concrete topping is only effective if it is placed at least 15 mm below the head of the shear connectors. For deep solid planks, 'bent-down' transverse reinforcement bars may be necessary to meet this requirement.

4.6   Serviceability conditions

There are four design criteria at the serviceability limit state:

- A limit on deflection due to imposed load.

- A limit on the total deflection (which may be off-set by precambering).

- Avoidance of irreversible deformation.

- Avoidance of excessive vibrations.

Elastic section properties should be used in all serviceability calculations.

4.6.1   Elastic properties of composite section

The second moment of area of the uncracked composite section is established by transforming the cross-sectional area of concrete into an equivalent area of steel, by dividing by the modular ratio, α_e. The composite section may be considered to be uncracked when the neutral axis lies in the steel beam, which occurs when:

$$A > \frac{D_s^{\,2} B_e}{D \alpha_e}$$ (27)

where:

A is the steel cross-sectional area

D_s is the overall depth of the concrete flange (including the structural topping, if used). For solid planks, a minimum of 25 mm should be deducted from the slab depth (see Section 4.2.2)

B_e is the effective width of the concrete slab (see Section 4.2)

D is the beam depth.

α_e is the modular ratio, which depends on the duration of loading (see Table 4.3)

In these circumstances, the position of the elastic neutral axis from the top surface of the concrete flange is:

$$y_g = \frac{A\alpha_e (D + 2D_s) + B_e D_s^2}{2(A\alpha_e + B_e D_s)} \tag{28}$$

The uncracked second moment of area, in steel units, is:

$$I_g = I_x + \frac{B_e D_s^3}{12\alpha_e} + \frac{AB_e D_s (D + D_s)^2}{4(A\alpha_e + B_e D_s)} \tag{29}$$

where I_x is the steel second moment of area about the major axis.

If the neutral axis lies within the concrete flange, concrete on the tensile side of the neutral axis should be ignored. In these circumstances, the section properties may be calculated by an iterative process.

Beams may be propped during construction; in which case, the deflection after removal of props should be calculated using the long-term value of the modular ratio, α_e. Props should not be removed until the infill concrete has gained its specified design strength. In these circumstances, the effect of prop removal on the shear resistance of the hollow core units should be considered (see Section 5.2).

Table 4.3   *Modular ratio α_e of steel to concrete*

Loading Duration	Normal Weight Concrete
Short-term	6
Long-term	18
Imposed loading	10
Dynamic	5.4

4.6.2 Deflection limits

The recommended limits on deflection are given in Table 4.4. Generally, it is the total deflection that controls the design of long-span beams, and pre-cambering or propping during construction is often used to counteract this effect.

Table 4.4   *Recommended deflection limits*

Type of Loading	Beam Type	
	Internal	Edge
Imposed loading	L/360	L/500
Total deflection	L/200	L/350
Absolute deflection	60 mm	35 mm

When partial shear connection is provided, the composite beam is more flexible because of slip. For unpropped construction, the increased deflection under serviceability loads, δ, should be determined using the following expression:

$$\delta = \delta_c + 0.3(1 - N_a / N_p)(\delta_s - \delta_c) \qquad (30)$$

where:

δ_s is the deflection of the steel beam acting alone

δ_c is the deflection of the composite beam, assuming full shear connection, for the same loading.

N_a is the actual number of shear connectors provided from the support to the point of maximum moment.

N_p is the number of shear connectors, from the support to the point of maximum moment, required for full shear connection.

For propped construction, the factor of 0.3 may be increased to 0.5.

4.6.3 Irreversible deformation

This check is required to ensure that yielding of the section does not occur at serviceability loads, so that the basic assumption that the beam remains linearly-elastic (made in calculating the deflections) is validated. For unpropped beams, the stresses in the bare steel section (arising from the self-weight loads in the construction condition) should be added to the subsequent stresses in the final composite condition (arising from the imposed loads, and any superimposed dead loads). However, for propped beams, the stresses from the dead and imposed loads (from prop removal) should be calculated by considering the composite cross-section.

In accordance with BS 5950-3: 1990[1], the total steel stress should be less than, or equal to, p_y. The concrete compression flange is limited to a stress of $0.5 f_{cu}$.

4.6.4 Dynamic considerations

Natural frequency

When individual structural components are inter-connected to form a complete floor system and this floor system vibrates, the whole floor structure moves up and down in a particular form, known as a *mode shape*. Although, each floor frequency has a particular mode shape associated with it, it is generally the lowest (1st mode) or *fundamental frequency* that is of particular interest in design, due to the fact that the largest acceleration response is normally found when this mode is excited to resonance. The fundamental frequency of the floor system is lower than the frequency of any of the components.

In conventional composite floor systems, the fundamental frequency may be estimated by using engineering judgement on the likely deflected shape of the floor (mode shape), and considering how the supports and boundary conditions will affect the behaviour of the individual structural components. For example, on a simple floor comprising a slab using solid precast planks continuous over a number of secondary beams that are, in turn, supported by stiff primary beams, there are two possible mode shapes that may be sensibly considered:

1.   Secondary beam mode

The primary beams form nodal lines (i.e. they have zero deflection), about which the secondary beams vibrate as simply-supported members (see Figure 4.6(a)). In this case, the slab flexibility is affected by the approximately equal deflections of the supports. As a result of this, the slab frequency is assessed on the basis that fixed-ended boundary conditions exist.

2.   Primary beam mode

The primary beams vibrate about the columns as simply-supported members (see Figure 4.6 (b)) and the primary beams at each end of any secondary beam are in phase. Thus, the secondary beams each have approximately equal deflections at their supports; the secondary beams and their frequency (like the slab) are assessed on the basis that fixed-ended boundary conditions exist.

For composite beams that use hollow core units, the beams are usually supported directly by columns. In these circumstances, only the secondary beam mode need be considered.
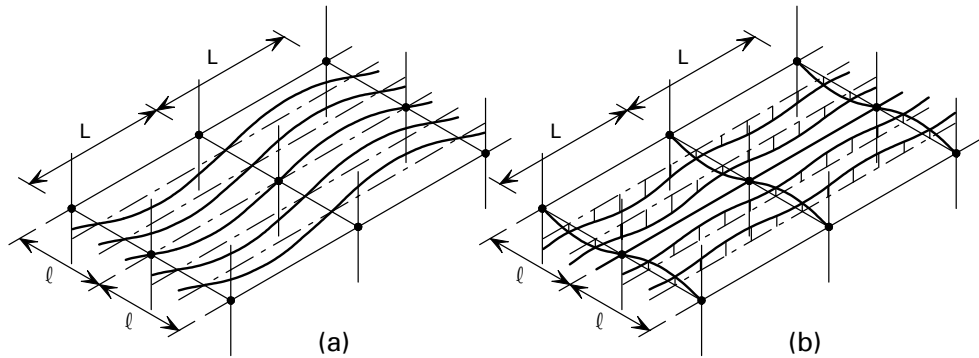
Figure 4.6 *Typical fundamental mode shapes for composite floor systems (a) governed by secondary beam flexibility (b) governed by primary beam flexibility*

As composite construction is essentially an overlay of one-way spanning elements, the frequency of the whole floor system can be calculated for each mode shape, by summing the deflexion calculated from each of the above components, and placing this value within Equation (31). The lowest frequency value determined by consideration of these two cases defines the fundamental frequency of the floor f_0 (and its corresponding mode shape).

$$f_n = 18 / \sqrt{\delta_{sw}} \quad \text{Hz} \tag{31}$$

where δ_{sw} is the instantaneous deflection (in mm), due to reapplication of the self weight and other permanent loads acting on the beam, plus 10% of the imposed load.

The uncracked second moment of area of the composite beam (Equation (29)), using the dynamic modular ratio, should be used in calculating δ_{sw}.

Alternatively, it can sometimes be convenient to use these component frequencies directly, to evaluate the fundamental frequency of the floor f_0 by

Dunkerly's approximation shown in Equation (32) below; both methods give identical results.

$$\frac{1}{f_0{}^2} = \frac{1}{f_1{}^2} + \frac{1}{f_2{}^2} + \frac{1}{f_3{}^2} \tag{32}$$

where f_1, f_2 and f_3 are the component frequencies (Hz) of the composite slab, secondary beams and primary beams respectively, with their appropriate boundary conditions, as defined above.

For floors that are to be subjected to walking traffic, it is recommended[22] that the fundamental frequency, f_0, should be at least 3.55 Hz. For steel-framed car parks, this limit is traditionally relaxed to a value of 3.0 Hz. For floors that are to be subjected to synchronised crowd movement (such as aerobics areas, gymnasia, etc.), the effect of the dynamic loading on the ultimate limit state criteria should be considered. In accordance with BS 6399-1:1996[23], resonant effects may be ignored if the fundamental frequency of the floor f_0 is greater than 8.4 Hz (based only on the self weight and other permanent loads). If this frequency limit cannot be satisfied, the dynamic loads should be calculated directly using the method given in Annex A of BS 6399-1:1996. In these circumstances, a partial factor of 1.0 should be applied to the dynamic loads and a partial factor of 1.4 to the dead loads.

Acceptability of floors

Like conventional steel framed construction, the acceptability of floors that are subjected to walking traffic should be assessed by calculating the response factor in accordance with the SCI publication P076 *Design guide on the vibration of floors*[24]. For cases when the precast slabs do not act compositely with the steel beams, the effective width of floor participating in the vibration *S* (which is used to calculate the response factor), should correspond to the width of the slab *b*. The value of *b* should be taken as half the distance to each of the adjacent beams, measured to the centre-line of the web (unless continuity is provided for, over the supporting beams, through suitably detailed transverse reinforcement bars within the infill concrete).

4.7   Special cases

Precast hollow core units may also be used in special applications, such as:

- Changes of slab orientation internally.

- Slabs with large openings.

- Beams with web openings.

- Beams with local point loads.

- Cantilever beams.

These cases are not covered by the current guidance, but the following qualitative statements can be made:

- Openings up to 50% of the depth of the beam can be ignored, in terms of their effect on local stresses in the hollow core units.

- There is no restriction on the use of precast slabs using solid planks, which are insensitive to local bending effects.

4.8   Steelwork connections

Connections take two generic forms:

- Beam-to-column connections.

- Beam-to-beam connections.

Beam-to-column connections should be designed as non-composite connections using full depth end plate connections, in order to:

- Resist out-of-balance forces on the beam.

- Reduce deflections at the construction stage, as a result of the effective stiffness of the joint.

Beam-to-beam connections can only be detailed as full depth end plates, if the top flange of the primary beam projects above the secondary beam. In other cases, partial depth end plates should be used.

a) Full depth end plate
(beam - column)

b) Partial depth connection
(beam - beam)
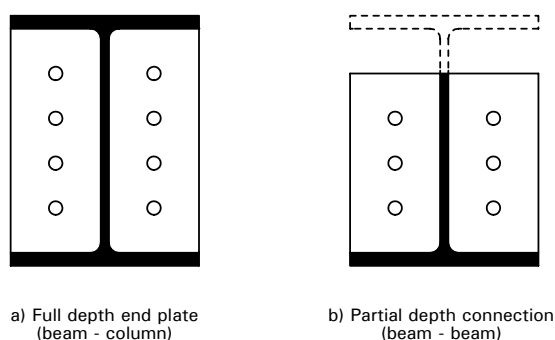
Figure 4.7 *End plate connections*

In future, it may be possible to design composite connections by providing suitably anchored reinforcement, positioned parallel to the longitudinal axis of the beams, around the column locations.   However, no test information presently exists on the performance of the shear connection to composite beams with hollow core units in negative moment regions. Also, the additional weight of the structural topping (needed to embed the longitudinal reinforcement bars) may offset the savings in steel weight offered by the effects of continuity.

4.9   Robustness

Robustness of structures is taken to relate to the resistance to accidental damage and unusual loadings, such as explosions. There is a statutory requirement for avoidance of "disproportionate collapse" of buildings in Part A of the Building Regulations[25].   Codes cover this requirement by specifying minimum tying forces between the various elements (refer to BS 5950-1: 2000[17] and BS 8110-1: 1997[3]). A steel framed structure achieves tying action by appropriate design of the beam-to-column connections. In general, the following tying action is required:

- Peripheral ties around the perimeter of the building.

- Internal ties between the internal beam and floor slab.

- Internal ties between the columns (may be distributed across the slab).

The measures required for diaphragm action and fire resistance (which are discussed in the next two sections), normally achieve sufficient robustness of the construction.

5   DESIGN OF THE FLOOR SLAB

The following Sections present a summary of the design of hollow core and solid plank precast units. Further guidance may be obtained from the Precast Flooring Federation (PFF).

5.1   Design of precast units

In the majority of cases, the manufacturer will undertake the design of the hollow core units. The main design issues that need to be considered for strength purposes are discussed briefly below.

Unlike conventional reinforced concrete members, hollow core units have no reinforcement other than the longitudinal prestressing tendons anchored by bond. Consequently, whenever possible, tensile stresses in unreinforced zones are normally avoided by designing the floors to be simply supported.

The bending resistance of hollow core units is determined like any prestressed concrete member in that the prestressing force precompresses the concrete in the regions where tensile stresses will develop. As a consequence, when the member is subjected to increments of load, the bending stresses will gradually reduce the built-in compression in those regions; however, once the load is removed, the beam returns to its original state of stress.

As well as the shear resistance check normally used in conventional reinforced concrete design, additional checks in the vicinity of the supports are also required. These checks ensure that there is sufficient resistance to prevent shear tension failure from occurring (which occurs when the principal tensile stress in the web reaches the tensile strength of the concrete), and that there is sufficient anchorage of the prestressing steel. Both of these checks are strongly affected by the length from the support over which the full prestressing force is developed (known as the 'transmission length').

If a structural topping is used, the composite action between the topping and the hollow core units will often make it is possible to increase the resistance of the hollow core units; typically an increase in resistance of between 20 to 60% may be obtained.

5.2   Allowance for non-rigid supports

As discussed above, hollow core units are generally designed as simply-supported elements on rigid supports (see Figure 5.1(a)). However, when these units are supported by beams that deflect under the imposed load (Figure 5.1(b)), shear stresses parallel to the longitudinal axis of the supporting beam are applied across the ends of the hollow core units. Test results and Finite Element analyses[26][27] have shown that these additional stresses are directly related to the vertical shear force due to the imposed load (these stresses are in addition to the stresses within the slab, had rigid supports been provided). The combination of stresses arising from non-rigid supports should be taken account of when the shear resistance of the hollow core units is checked.
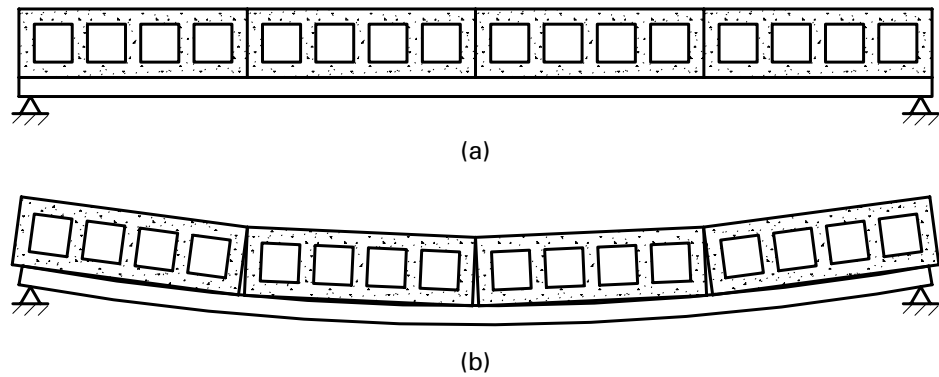
Figure 5.1 *Cross-section of hollow core units on beam with frictionless bearing: (a) rigid supports, and (b) beam deformed due to load*

In most practical applications, where the secondary beams are unpropped during construction, sufficient shear resistance will normally exist within the hollow core units to withstand the additional stresses arising from the effect of the flexible supports. However, when propped construction is used, particular care should be taken as the removal of the props can significantly increase the applied shear stresses within the hollow core units.

The structural resistance of hollow core units on flexible supports can be improved by infilling the ends of the hollow core units to a distance equal to the depth of the unit, or by providing an *in situ* reinforced concrete topping over the units. Alternatively, the stiffness of the supporting beam can be increased by providing a heavier or deeper beam than is required for bending resistance. For composite beams, infilling of at least half of the cores achieves this objective.

For unpropped non-composite beams, the influence of support stiffness need not be considered[28] if the factored shear force that is applied to the slab is less than $0.35V_{Rd}$ (where V_{Rd} is the shear resistance of the hollow core units provided by the manufacturer). For cases when propped construction is used, or when the factored shear force applied to the slab is greater than $0.35V_{Rd}$, advice from the manufacturer of the precast units should be sought.

Pre-cambered beams have no effect on the resistance of the hollow core units, since the beams will become approximately level under the action of the dead load from the slab.

5.3 Diaphragm action

The floor is often required to provide diaphragm action in order to transfer wind loads to braced walls or concrete core walls. This action can be achieved through the following measures:

- Provision of a continuous *in situ* reinforced topping in order to transfer the in-plane forces in both orthogonal directions.

- Ties between the perimeter members and the floor (attached by welded shear connectors and looped bars, for example).

- Ties to the shear walls or reinforced cores.

- Where an *in situ* topping is **not** used, additional internal ties should be provided (a topping is recommended for larger floors or taller buildings). This is achieved by provision of transverse reinforcement in a composite beam.

The same measures are also appropriate to achieve robustness (see Section 4.9).

Steel beams around the perimeter of the building should be tied into the floor plate for diaphragm action, and for torsional resistance (if they support cladding). I-beams may be considered to act as peripheral ties, provided that they are connected mechanically to the slab through shear connectors (see Figure 3.6). A *Slimflor* Fabricated beam (SFB) or a Rectangular Hollow Section Fabricated Beam (RHSFB) may also be considered to act as peripheral.

The location of these ties for composite construction using I-beams, as well as *Slimflor* construction, is illustrated in Figure 5.2.
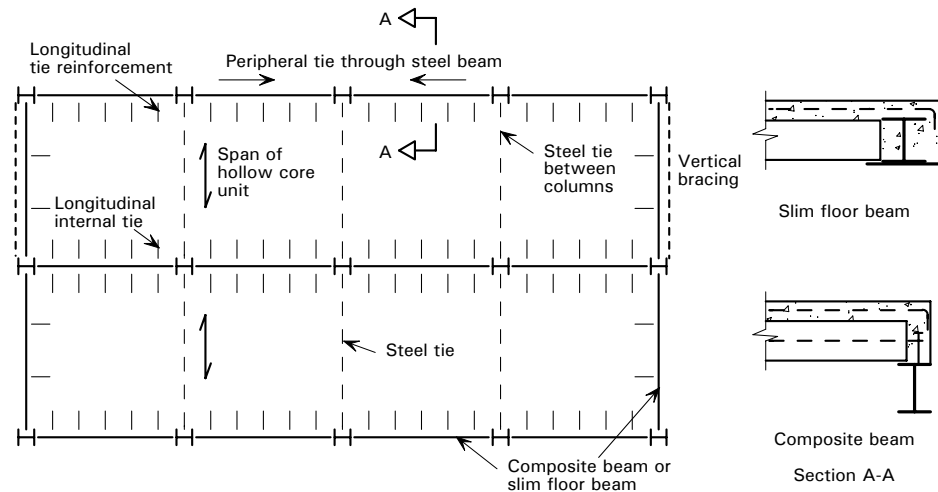
Figure 5.2 *Detailing for diaphragm action of a floor using precast units*

6   FIRE RESISTANCE

Fire resistance is defined in terms of endurance of structural elements in a standard fire test.   Compliance with the Building Regulations requires a resistance of 30, 60, 90 or 120 minutes, depending on the building.   The general requirements for fire resistance are:

- Insulation between compartments, which is achieved by a minimum thickness of concrete slab (possibly requiring an *in situ* topping).

- Integrity: by filling of the joints between the units to prevent passage of flames and hot gases.

- Load resistance: to support the reduced loads acting at the fire limit state (typically 60% of the design ultimate loads).

Clearly, by considering the supporting beams and the hollow core units in isolation, the component with the lowest fire resistance will define the fire resistance of the whole construction.   The following sub-sections give the requirements for normal composite beams.   For *Slimflor* beams, greater care is needed in detailing tie reinforcement for fire conditions.   These measures are described separately in Section 8.3.

6.1   Support beams

The rate of increase in temperature of a steel cross-section depends on the ratio of the exposed surface area to the volume of the member per metre length A_m/V.   This ratio is invariably expressed in units of m^{-1} and is known as the 'section factor'.   In the UK, the section factor is typically represented as the ratio of the heated perimeter to the cross-sectional area, H_p/A.   However, both relationships give the same value and the European terminology A_m/V will become the standard relationship given in the future.   (N.B. Members with low section factors will heat up more slowly than members with high section factors).

For the required fire resistance of the construction, the choice of the type of fire protection that is to be applied to the steel section is established as follows:

Spray coating   This is applied around the profile, and the section factor for determining the thickness of protection uses the perimeter of the profile, excluding the top flange in contact with the slab.

Boards   This is applied as a box around the section, and the section factor for determining the thickness of protection uses twice the beam depth plus the width of the bottom flange.

Intumescent coatings   These coatings are applied around the profile of the section; some of these coatings can be applied off-site.   Thin (0.6 to 1.5 mm) and thick film (> 2 mm) coatings may be used

For protected beams, the amount of protection (by spray, board or intumescent coating) should be such to keep the temperature of the steel section below the limiting temperature for the period of time stated. The limiting temperature is dependant on the level of applied load at the fire limit state. For example, a limiting temperature of 620°C is appropriate[29] for a composite beam supporting hollow core units with a load ratio of 0.6 (load ratio = load at the fire limit state ÷ member resistance at 20°C). The required thickness of fire protection may be obtained from the design tables within the ASFP/SCI publication[30] using the appropriate section factor.

6.2 Hollow core units

The detailing requirements shown in Figure 6.1 may be adopted to ensure satisfactory performance in fire[28].

For downstand composite beams (see Figure 6.1), the transverse reinforcement used to develop composite action is normally sufficient to provide satisfactory performance in fire conditions. These bars should be embedded to a minimum distance of 600 mm from the ends of the units. For 90 and 120 minutes fire resistance, a concrete topping will normally be required[31]. The effect of non-rigid supports (see Section 5.2) need not be considered.
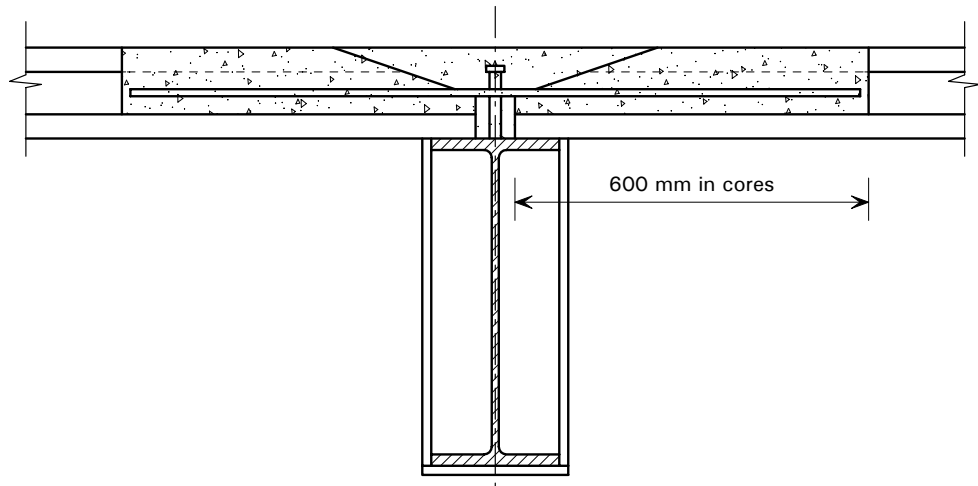
(a) 60 minutes fire resistance
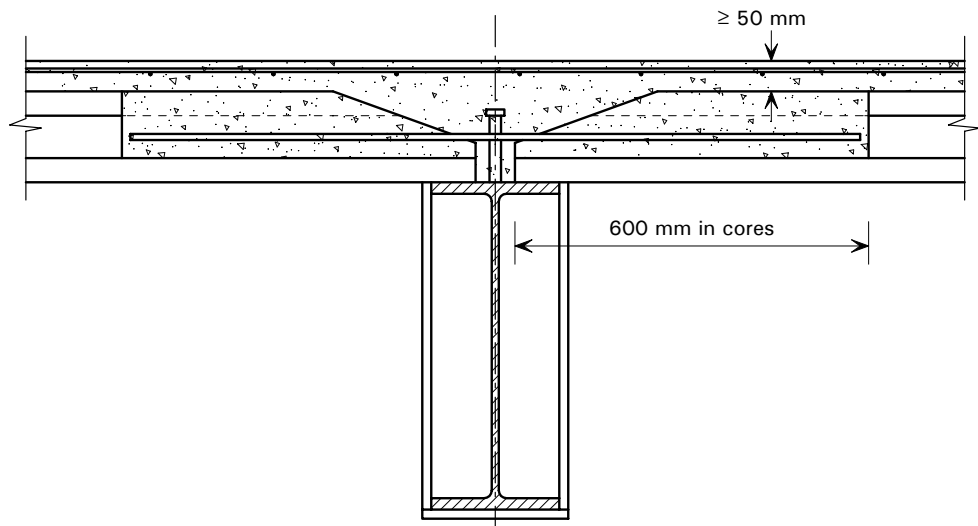
(b) 90 and 120 minutes fire resistance

Figure 6.1 *Detailing measures for hollow core units with downstand steel or composite beams to achieve standard periods of fire resistance*

7 CONSTRUCTION CONSIDERATIONS

As a summary of the constructional aspects discussed in the previous Sections, Table 7.1 gives the main features that should be considered when designing composite steel beams using precast hollow core units (HCUs) and their range of applicability.

Table 7.1 *Practical considerations for composite beams using precast hollow core units*

Feature	Range of Application
Range of slab depths	150 to 260 mm depth for HCUs.
Overall slab depth	≤ 260 mm considered for design purposes (i.e. additional depth ignored).
Range of slab spans	35 to 40 × HCU depth is typical for office loading (3.5 to 5 kN/m^2).
Minimum beam widths - Internal beams	Using shop-welded studs: 180 mm for depth of HCU ≤ 200 mm 190 mm for depth of HCU > 200 mm.
- Edge beams	210 mm in all cases.
Effective slab width	Beam span/8 but not exceeding: Infill of hollow core + Gap between units.
Concrete grade	C30 to C40 for in-situ concrete topping. 10 mm aggregate NWC (C50 to C60 for HCU).
Bar diameter	12 mm minimum at 200 to 350 mm centres. 16 mm for partial shear connection. 12 mm U-bars provided at edge beams and at openings.
Stud details	19 mm × 125 mm - site welded studs 22 mm × 125 mm - factory welded studs at 150 mm centres typically.
Construction condition	Provide temporary torsional restraint at a spacing not exceeding 8 m; or Provide temporary propping to prevent lateral movement of the beam.
Fire resistance	Concrete topping is required for 90 or 120 minutes fire resistance. Correct detailing of the transverse reinforcement is required to achieve the appropriate period of fire resistance.
End connections	Provide full depth bolted end plate to beam for stability during installation of HCUs.
Serviceability	As for normal composite beam design. Consider pre-cambering for spans > 10 m.
Robustness	Achieved by transverse reinforcement in composite design.

8   *SLIMFLOR* CONSTRUCTION

As explained in Section 2.4, precast slabs can be used in conjunction with *Slimflor*® Fabricated Beams (SFB) and Rectangular Hollow Section *Slimflor*® Edge Beams (RHSFB). Composite action may be achieved by providing an *in situ* topping and welding short studs to the top flange of the UC.

This Section provides general detailing requirements for precast slabs used in *Slimflor* construction. Specific guidance on the structural design of the beams themselves may be found in other SCI publications[8][9].

Hollow core concrete units can be used effectively in *Slimflor* construction, although careful attention should be given to the following design and detailing requirements:

- Global requirements for diaphragm action and robustness.

- Additional shear forces caused by non-rigid supports.

- Requirements for fire resistance.

These aspects are discussed in Sections 4.9, 5.2, 5.3 and 8.3, where it is shown that improved structural performance of the hollow core units can be achieved by use of additional tie reinforcement, and/or an *in situ* concrete topping.

8.1   Construction condition

The construction condition needs to consider safety issues during installation of the precast units. The following detailing issues should be addressed in design, and achieved on site:

1.   In composite applications, the nominal bearing length of the precast concrete units on the bottom plate of the *Slimflor* beam should be 60 mm (see Section 3.1.1).   However, in practice, tolerances exist in manufacture and in installation of the units.   The minimum bearing distance should not be less than 40 mm, when measured on site.   The maximum bearing length of 75 mm is dictated by tolerances on installation and by ease of the later concreting operation (see 2, below). The nominal length of the units should therefore be the distance between the tips of the bottom plate plus 120 mm.

     In non-composite applications, the nominal bearing length should be increased to 75 mm, because the units may be treated as isolated i.e., not inter-connected.

2.   It may be necessary to chamfer the ends of deeper precast units in order to facilitate:

     - the installation of the precast units safely

     - placement of concrete around the *Slimflor* section (which is required for fire resistance).

     However, the chamfer should be such as to not affect the shear resistance of the units. The detailing of these chamfers is shown in Figure 8.1.

3. In all cases, the **minimum** gap between the tip of the top flange and the nearest part of the precast unit should be 60 mm, in order to permit installation of the units and proper placement and compaction of the *in situ* concrete around the sections.

4. The beam should be checked for the condition where the precast concrete units are placed first on one side of the beam (which causes out-of-balance forces), leading to combined bending and torsion on the section. In cases where the precast units are very heavy, it may be necessary to organise the installation of the units such that loads on either side of the beam are approximately balanced.

    Due to the eccentricity between the ends of the precast units and the centroid of the SFB, a particular design consideration is presence of a transverse moment in the flange plate. In this situation, if the applied transverse moment is greater than 0.732 times the plastic moment resistance of the flange plate, the bending resistance of the SFB is reduced[8].

5. An in *situ* concrete topping with mesh reinforcement is often placed on the precast units to achieve composite action, in order to increase the spanning capabilities of the slab. The minimum depth of the concrete topping is given by the more onerous of the requirements in Table 8.1.

    In composite applications, the beam is usually shallower than the precast units that it supports.

6. The design strength of the *in situ* concrete should be at least grade C25, so that it contributes to the shear resistance of the slab. The maximum aggregate size should be 10 mm (as used in a pumpable concrete mix).

A non-structural screed is not sufficient on its own, except for low-rise buildings (i.e., not subject to robustness issues, and requiring only 30 minutes fire resistance); see Sections 4.8 and 8.3.

View on solid portion   View on hollow core

Limits : $60 \le b_{hc} \le 90$ mm
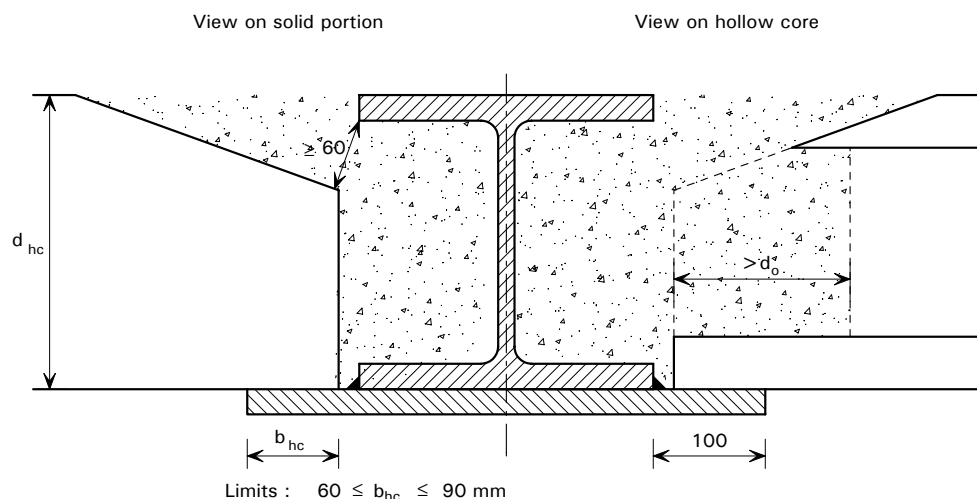
Figure 8.1   *End detailing geometry for hollow core units supported by Slimflor beams*

Table 8.1 *Recommended minimum and maximum depths of in situ concrete in non-composite and composite Slimflor beams*

Depth of Concrete Topping	Minimum Depth (mm)	Maximum Depth (mm)
Depth over precast unit	50	100
Depth over non-composite beam	35	70
Depth over composite beam	85*	120

* for 75 mm welded shear connectors.

8.2 Normal conditions of use

The important design conditions for the *Slimflor* beam supporting the precast concrete units, in their normal conditions of use, are as follows:

1. The steel beam may be designed either:

    -   Non-compositely for general applications, but including the concrete encasement for stiffness purposes[8].

    -   Compositely using an *in situ* concrete topping and welded shear connectors[8]. Sufficient transverse reinforcement is required to transfer the force from the shear connectors into the slab.

2. The shear resistance of hollow core units is particularly affected by the support flexibility of the *Slimflor* beam (see Section 5.2).

8.3 Fire resistance of *Slimflor* beams with precast units

8.3.1 Support beams

For the required fire resistance of the construction, the choice of whether to fire protect the steel section is established as follows[28]:

30 minutes fire resistance:	No applied fire protection is required for *Slimflor* Fabricated beams without an *in situ* topping (see Figure 8.2(a)).
60 minutes fire resistance:	For this period of fire resistance, the load resistance must be checked using appropriate software. In most applications, no applied fire protection is required for *Slimflor* Fabricated beams <u>with</u> an *in situ* topping (see Figure 8.2(a)).
90 or 120 minutes fire resistance:	Protect the bottom flange of the steel section.

For protected beams, the amount of protection (through spray, board or intumescent coatings) should be such to maintain the temperature of the steel support to the hollow core units below 650°C for the period of time stated.

The required thickness of fire protection may be obtained from the design tables within the ASFP/SCI publication[30] using a section factor of $(1/t_f)$, where t_f is the thickness of the bottom flange in metres. The thickness obtained in this way may be conservative, and more detailed information may be obtained from the

fire protection manufacturers. Often, only a nominal thickness of protection is required because the section factor A_m/V is relatively low (<50 m^{-1}) in comparison to that of conventional steel members.

Most protection materials are assessed at a limiting temperature of 550°C. As *Slimflor* sections reach higher limiting temperatures, the thickness of protection may be reduced.

8.3.2   Hollow core units

Provided that the applied shear force acting at the ends of the hollow core units in fire conditions (using the reduced load factors in fire) is less than, or equal to, $0.20V_{Rd}$ (where V_{Rd} is the shear resistance of the hollow core units provided by the manufacturer), the detailing requirements shown in Table 8.2 may be adopted[28]:

Table 8.2   *Detailing requirements for hollow core units at the fire limit state*

Fire resistance (mins)	Option	Tie reinforcement	Reinforced concrete topping	Applied shear ≤ 0.2V$_{Rd}$	Comment
		Protected beams† (T flange ≤ 650°C)			
30		No	No	No	Note a.
60	1	No	No	No	Depth ≤ 265 mm & Note (a).
	2	In joints	No	No	Ties in joints or cores.
	3	No	Yes	No	
90		In joints	No	No	Topping is optional.
120		In cores	Yes	No	Suspension reinforcement.
		Unprotected beams (T flange > 650°C)			
30		No	No	No	Note (a).
60	1	No	No	Yes	Depth ≤ 265 mm & Note (a).
	2	In joints	No	No	Ties in joints or cores.
	3	No	Yes	No	
90		In cores	No	Yes	Topping is optional.
120		In cores	Yes	Yes	Suspension reinforcement.

Notes

†   A fire protected steel beam, or concrete encased steel beam, provides an insulated support to the hollow core units.

(a). In this case, effective global tying action should be achieved through the three-dimensional steel structure. If this is not satisfied, tying action should be achieved through the floor slab in Options 2 or 3.

The tie reinforcement should be detailed as shown in Figure 8.2. When specially detailed tie bars are required (in addition to the mesh reinforcement), the minimum number of tie, or suspension, bars should be 2 per unit (but for units wider than 1.2 m, one bar every 600 mm should be used). The minimum diameter for these additional tie reinforcement bars should be 10 mm. The minimum depth of the concrete topping is 50 mm, and the minimum area of mesh reinforcement should be 98 mm²/m, or as required for 'crack control'. For cases when the mesh reinforcement area is greater, or equal to, 252 mm²/m, bending continuity should be considered.

For *Slimflor* beams requiring 30 or 60 minutes fire resistance, the ends of the hollow core units should be filled with concrete to a nominal distance equal to the depth of the cores (Figure 8.2(a)). For protected beams requiring 90 minutes fire resistance, tie bars may be placed in the joints between the units. These bars may be straight or inclined bars, depending on the type of beam (for a *Slimflor* Fabricated beam see Figure 8.2 (b)), and should be embedded a minimum length of 1.2 m from the ends of the units. In addition, the ends of the hollow core units should be filled with concrete to a nominal distance equal to the depth of the cores.

For *Slimflor* beams requiring 120 minutes fire resistance, suspension reinforcement should be bent over the steel section at 45°, and embedded within the cores at a minimum distance of 600 mm from the ends of the units (see Figure 8.2(c)). Alternatively, like conventional steel beams (Figure 8.2(b)), L-bars may be used. However, in this case, the bars need to be passed through holes in the beam web; due to difficulties in alignment with the broken out hollow cores, these holes should be spaced at 300 mm along the beam.

Topping is optional

Topping is required
≥ 85 mm

d_o

30 minutes fire resistance
(non-composite)

$> d_o$

60 minutes fire resistance

(a) 30 and 60 minutes fire resistance

≥ 85 mm

d_o

$> d_o$

1200 mm in joints

(b) 90 minutes fire resistance

≥ 85 mm

600 mm in cores
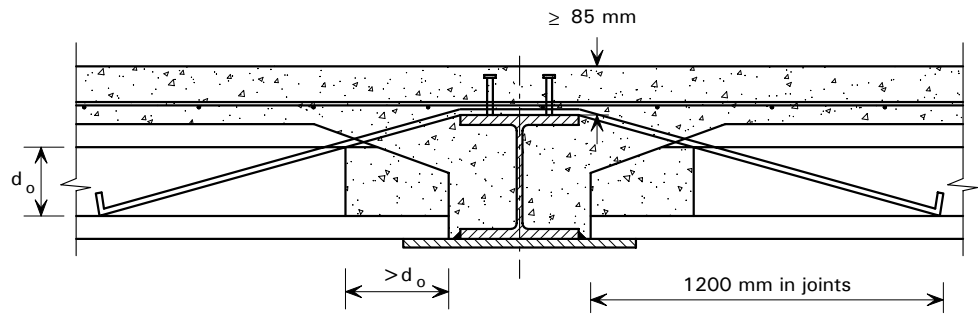
(c) 90 and 120 minutes fire resistance

Figure 8.2 *Detailing measures for hollow core units with Slimflor beams to achieve: (a) 30 and 60 minutes fire resistance; (b) 90 minutes fire resistance; and (c) 90 and 120 minutes fire resistance*

9 LOAD-SPAN TABLES FOR INITIAL SIZING

This Section provides three tables to assist the selection of a suitable size for secondary beams, used with hollow core units, at the initial design stage. The tables give the size of standard UB sections in S275 steel, for a range of beam spans, using beams at spacings chosen to suit three values of imposed load. In all cases, T16 bars are provided as transverse reinforcement.

The three tables relate to the following different construction conditions:

- Internal beams unrestrained during construction.

- Internal beams temporarily restrained at mid-span during construction.

- Internal beams temporarily restrained at mid-span during the construction and where there is a 50 mm-structural topping on the hollow core units.

The spacing of the beams was defined by the spanning capabilities of the hollow core units, which were chosen to suit the imposed load. The beam spacings and hollow core unit thicknesses that were chosen as typical are as follows:

- 2.5 kN/m²   6 m beam spacing   150 mm deep units.

- 3.0 kN/m²   7.5 m beam spacing   200 mm deep units.

- 4.0 +1.0 kN/m²   9.0 m beam spacing   250 mm deep units.

Limits used in establishing the adequacy of the designs are as follows:

a   Construction stage: buckling check from combined bending and torsion due to out-of-balance (one-sided) loads.

b   Construction stage: lateral torsional buckling check from balanced (two-sided) loading.

c   Normal stage: composite bending resistance.

d   Serviceability: 2° angle of twist check at the construction stage from out-of-balance (one-sided) loads.

Design Tables: ***S275 Universal Beam sizes for use with hollow core units***

Table 9.1 *Unrestrained internal beams*

Beam span (m)	Imposed load (kN/m²) / beam spacing (m) / unit thickness (mm)					
	2.5 / 6.0 / 150		3.0 / 7.5 / 200		4.0 + 1.0 / 9.0 / 250	
	UB size	Limit	UB size	Limit	UB size	Limit
6.0	406 × 178 × 67	a	406 × 178 × 67	a	457 × 191 × 74	a
7.5	406 × 178 × 74	a	457 × 191 × 89	a	533 × 210 × 109	a
9.0	457 × 191 × 98	a	533 × 210 × 122	a	610 × 229 × 140	a
10.5	533 × 210 × 122	a	686 × 254 × 152	a	610 × 305 × 179	d
12.0	686 × 254 × 152	b	838 × 292 × 194	a	914 × 305 × 224	a
13.5	610 × 305 × 179	d	838 × 292 × 226	a	1016 × 305 × 272	a
15.0	914 × 305 × 224	a	1016 × 305 × 272	b	1016 × 305 × 314	b

Table 9.2 *Internal beams temporarily restrained at mid-span during the construction condition*

Beam span (m)	Imposed load (kN/m²) / beam spacing (m) / unit thickness (mm)					
	2.5 / 6.0 / 150		3.0 / 7.5 / 200		4.0 + 1.0 / 9.0 / 250	
	UB size	Limit	UB size	Limit	UB size	Limit
6.0	406 × 178 × 67	d	406 × 178 × 67	d	457 × 191 × 74	d
7.5	406 × 178 × 67	d	457 × 191 × 82	d	457 × 191 × 98	c
9.0	457 × 191 × 89	d	533 × 210 × 109	d	533 × 210 × 122	d
10.5	533 × 210 × 109	d	610 × 229 × 140	d	686 × 254 × 170	d
12.0	610 × 229 × 140	d	686 × 254 × 170	d	762 × 267 × 197	d
13.5	686 × 254 × 170	d	1016 × 305 × 222	d	610 × 305 × 238	c
15.0	762 × 267 × 197	d	610 × 305 × 238	d	1016 × 305 × 272	d

Table 9.3 *Internal beams temporarily restrained at mid-span during the construction condition plus a 50 mm structural topping*

Beam span (m)	Imposed load (kN/m²) / beam spacing (m) / unit thickness (mm)					
	2.5 / 6.0 / 150		3.0 / 7.5 / 200		4.0 + 1.0 / 9.0 / 250	
	UB size	Limit	UB size	Limit	UB size	Limit
6.0	406 × 178 × 67	d	406 × 178 × 67	d	457 × 191 × 74	d
7.5	406 × 178 × 67	d	457 × 191 × 82	d	457 × 191 × 98	c
9.0	457 × 191 × 89	d	533 × 210 × 109	d	533 × 210 × 122	d
10.5	533 × 210 × 109	d	610 × 229 × 140	d	686 × 254 × 170	d
12.0	610 × 229 × 140	d	686 × 254 × 170	d	762 × 267 × 197	d
13.5	686 × 254 × 170	d	1016 × 305 × 222	d	1016 × 305 × 249	d
15.0	762 × 267 × 197	d	610 × 305 × 238	d	1016 × 305 × 272	d

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APPENDIX A   Tests on composite beams using hollow core units

Appendix A describes the test information used to develop the design recommendations presented in this publication, in particular, the resistance of the headed stud shear connectors.

A.1   Push tests

A.1.1   Form of test

The property of a shear connector most relevant to normal building design is the relationship between the shear force transmitted and the slip at the interface. This load-slip curve should ideally be found from tests on full-scale composite beams, but in practice a simpler specimen is necessary. Most of the available data on connectors has been obtained from *push-out* or *push* tests.

Typically, the form this test takes is that the flanges of a short length of steel beam are connected to two small concrete slabs by means of shear connectors (see Figure A.1). The slabs are then bedded down onto the floor, or platens of a compression-testing machine, with the load being applied to the upper end of the steel member. Slip between the steel member and the two slabs is measured at specified load or displacement increments. The standard test arrangement specified in DD ENV 1994-1-1:1994 (Eurocode 4)[2] is shown in Figure A.1.
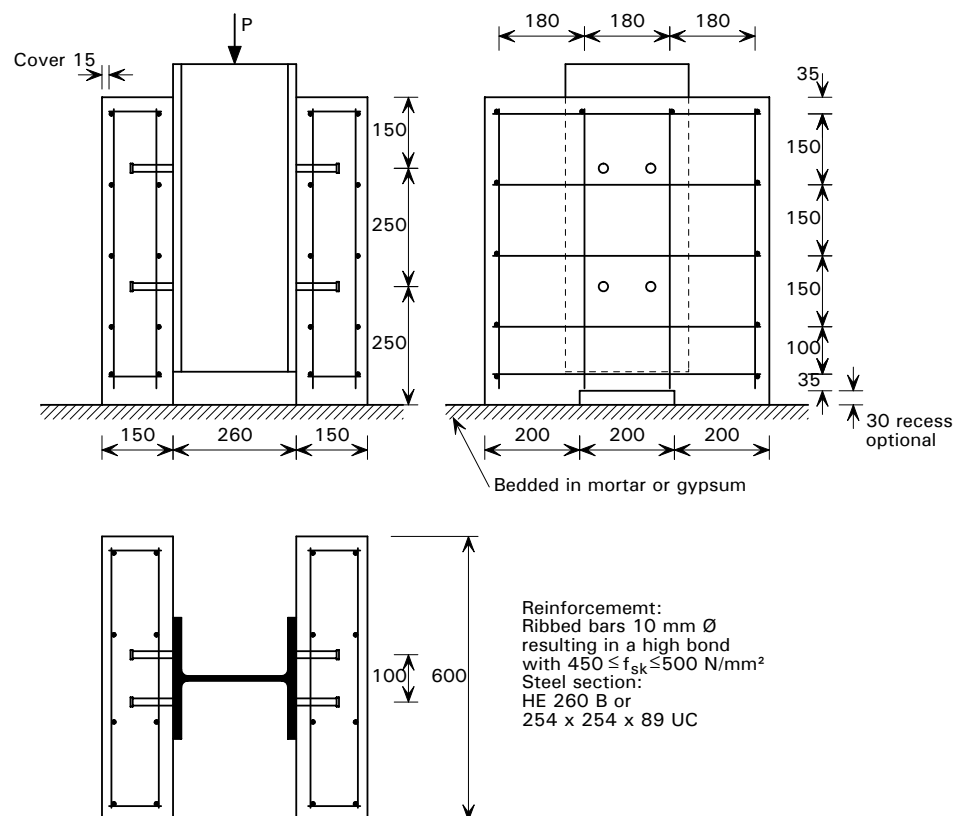
Figure A.1   *Eurocode 4 standard push test*

To ensure that the designers can assume a plastic distribution of force at the shear connection, the studs should also possess adequate ductility; this measure of ductility is normally derived from push tests, and is given in terms of the *slip capacity*.

For composite beams employing partial shear connection, the demand for the shear connectors to have adequate slip capacity is greater than for cases when full shear connection is provided (i.e., R_q is greater, or equal to, the lesser of R_s and R_c); in particular, in long-span beams and beams with a bottom flange area significantly larger than the top flange area. Rather than stating slip capacities directly, both BS 5950-3:1990[1] and Eurocode 4 allow designers to assume a plastic distribution of force at the shear connection by specifying minimum degrees of shear connection in terms of the span of the beam. These code rules are based on parametric studies of composite beams that considered the slip capacity of the shear connection explicitly.

For the partial shear connection rules for BS 5950-3:1990, the slip capacity was defined as the slip at which the shear resisted by the connector fell below 5% of its peak value[32] (see Figure A.2).
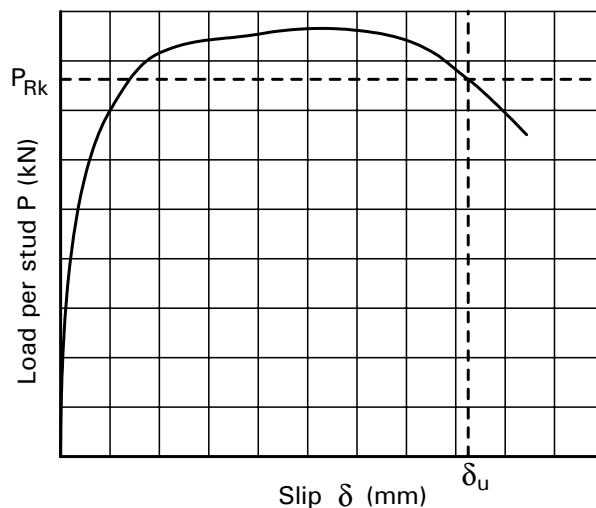
Figure A.2 *Determination of characteristic resistance and slip capacity from push test load-slip curve*

In Eurocode 4, the ductility of the shear connectors is defined in a slightly different way by using the *characteristic slip capacity*. In this case, the characteristic resistance of a shear connector P_{Rk} is defined as 0.9 × the peak load per stud, and the slip capacity δ_u is taken as the slip value where the characteristic resistance of the stud connector intersects the falling branch of the load-slip curve (see Figure A.2). For a small number of results (less than four[33]), the characteristic slip capacity is then taken as $\delta_{uk} = 0.9\delta_u$.

The rules for partial shear connection in Eurocode 4 are applicable for larger spans than are currently considered in BS 5950-3:1990[1]. Eurocode 4 also gives rules for partial shear connection in beams that use steel sections with unequal flanges (where the bottom flange area does not exceed 3 times the area of the top flange). In both cases, these rules are only valid if the shear connectors are 'ductile'. For 19 mm studs, the partial shear connection rules in BS5950-3:1990 assume a slip capacity of 7 mm[32]. For Eurocode 4, 'ductile' connectors

are defined as shear connectors that possess a characteristic slip capacity δ_{uk} of at least 6 mm[34].

A.1.2 Hollow core units

Due to the practicalities of testing shear connectors with hollow core slabs using the standard push specimen shown in Figure A.1, the one-sided arrangement shown in Figure A.3 was devised[17], so that the specimens could be tested in the horizontal position (this type of arrangement has also been successfully used in research on stub-girder assemblies[35]).
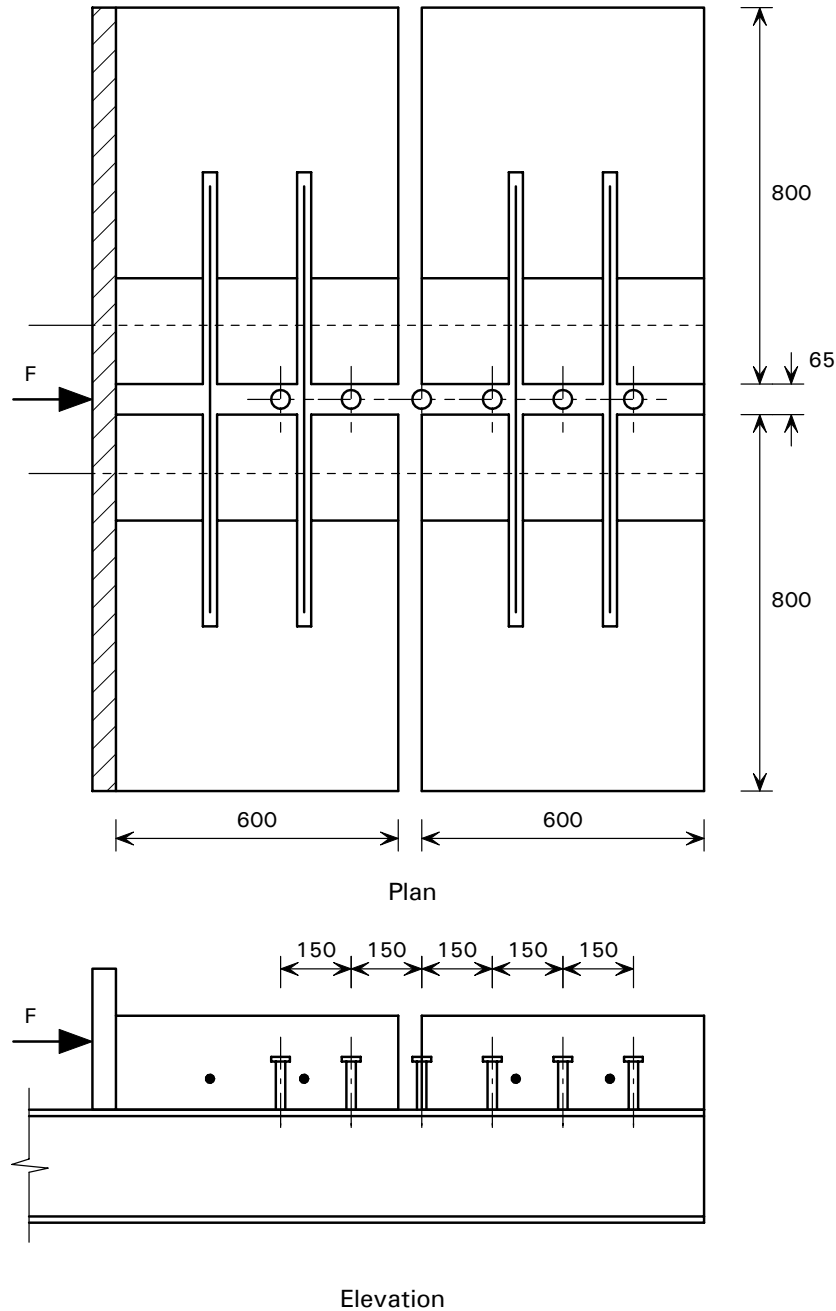
Figure A.3 *Push test used for hollow core specimens*

Chamfered-ended units

Push tests on specimens with headed stud connectors, and chamfered-ended hollow core units, were undertaken by Lam *et al.*[20] at the University of Nottingham.

In total, nine tests were undertaken using hollow core units, and two using solid reinforced concrete slabs (see Table A.1). Each specimen consisted of a S275 356×171×51UB with a single row of 19 mm × 125 mm long headed studs welded along the centre-line of the UB at 150 mm cross-centres. The nine hollow core specimens consisted of two 1200 mm wide, or four 600 mm wide, × 150 mm deep units supplied by Bison Concrete Products Ltd. (the 600 mm slab width was chosen instead of the more common 1200 mm wide units, so that the effect of the edge joint was included within the specimen length). Four milled slots were provided in each specimen to receive the transverse reinforcement bars. Each of these slots was 500 mm long, which meant that the total length of these bars was 1000 mm plus the gap between the ends of the units (see Figure A.3). As shown in Table A.1, these bars varied between 8 mm to 25 mm in diameter. The gap between the ends of the units was also varied from 40 mm to 120 mm (see Table A.1). An *in situ* concrete infill was only provided for these specimens.

In addition, seven test results from a recent investigation by Nip[36] at the University of Leeds are also presented in Table A.1. For these tests, the specimen geometry was identical to that described above.

Table A.1 *Push test results using chamfered-ended hollow core units*

Test	Bar size ϕ (mm)	Infill cube strength f_{cu} (N/mm²)	Gap g (mm)	Width of hollow core unit (mm)	Measured load per stud (kN)	Predicted load per stud‡ (kN)	Model factor†
Nottingham tests[20] – all specimens with C25 infill concrete (except as noted)							
T8 transverse reinforcement, 19 mm studs							
Half width	8	28.6	40	600	56.5	54.2	1.04
Half width	8	23.5	65	600	69.7	60.3	1.16
Full width 1	8	23	65	1200	54.3	72.3	0.75
Full width 2	8	23	65	1200	78	72.3	1.08
Half width	8	24.5	120	600	72.8	65.2	1.12
T16 transverse reinforcement, 19 mm studs							
Full width	16	23	40	1200	88.4	75.7	1.17
Half width	16	24.6	65	600	88.7	81.1	1.09
T25 transverse reinforcement, 19 mm studs							
Full width	25	23	40	1200	97.1	84.1	1.15
Half width	25	25.5	65	600	100.8	92.1	1.09
Nottingham tests[20] – solid slabs, 19 mm studs							
C38 slab	8	37.5	-	1200	72.9	91.3	0.80
C25 slab	16	25	-	1200	97	104.7	0.93
Leeds tests[36] – all specimens with C25 infill concrete							
T12 transverse reinforcement, 19 mm studs							
Half width 1	12	26.62	65	600	69.9	62.4	1.12
Half width 2	12	26.09	65	600	93.5	61.2	1.53
Full width	12	26.85	65	1200	67.0	77.1	0.87
T16 transverse reinforcement, 19 mm studs							
Half width-test 1	16	26.62	65	600	98.2	70.2	1.40
Half width-test 2	16	26.62	65	600	81.2	70.2	1.16
Full width-test 1	16	26.85	65	1200	117.4	86.8	1.35
Full width-test 2	16	26.85	65	1200	83.3	86.8	0.96

† Model factor = experimental value / predicted value

‡ Characteristic stud resistance from BS 5950-1: 1990 multiplied by reduction factor given by Equation (22)

The typical behaviour of the lightly reinforced specimens, with 8 mm transverse bars, was that the initial stiffness was lost relatively early. As a result of this, the maximum load was attained at a smaller deformation: typically, at a slip between 1.5 to 2.0 mm. Beyond this point, it was reported[20] that yielding of the reinforcement was accompanied by very large cracks, both longitudinally and transversely in the face of the concrete slab, and a gradual decrease in resistance.

In contrast, the more heavily reinforced specimens, with 16 and 25 mm transverse bars, had quite a high initial stiffness. The maximum load was typically obtained at slips of between 14 and 17.5 mm. However, beyond this point, due to the stud connectors shearing off in turn, it was reported[20] that the load resistance was lost suddenly in these specimens. It was also reported that the cracks in the face of the concrete slab were much smaller compared to the more lightly reinforced specimens, and the measured strains within the

transverse reinforcement indicated that, at failure, these bars were at less than 50% of their yield strength.

By examining the effect of the gap between the units, the diameter of the transverse reinforcement bars and the width of the hollow core units, the empirical reduction factor given by Equation (22) was developed by Lam et al.[20]. As can be seen in the seventh and eighth column in Table A.1, by multiplying the characteristic stud resistance from BS 5950-3:1990 (see Table 4.2) by the value of k given by Equation (22), the predicted resistance of the stud connectors agrees very well with measured results from the push tests.

As discussed above, as well as having adequate resistance to transmit the longitudinal shear force in a composite beam, the studs should also possess sufficient slip capacity to ensure that the shear connection is ductile. Based on the assumptions that were used to develop the rules in BS 5950-3: 1990[1], the slip capacity for the specimens reported by Lam et al.[20] and Nip[36] are presented in Table A.2; the corresponding characteristic slips, calculated in accordance with Eurocode 4, are also shown for comparison purposes.

Table A.2   *Slip capacities from push tests conducted by Lam et al. [20] and Nip[36] on studs embedded in slabs using chamfered-ended hollow core units*

Slab type	Transverse reinforcement bar size (mm)	BS 5950-3: 1990 slip capacity δ_u (mm)	Eurocode 4 characteristic slip capacity δ_{uk} (mm)
Reinforced concrete	8	2.91	3.22
Hollow core units	8	3.27	3.69
Hollow core units[36]	12	8.15	9.58
Reinforced concrete	16	23.14	21.65
Hollow core units	16	15.53	15.28
Hollow core units	25	14.46	13.50

Although it was reported[20] that in test conditions the hollow core specimens with 16 and 25 mm transverse reinforcement bars failed suddenly, the slip capacities of 15.53 and 14.46 mm shown in Table A.2 mean that, in terms of BS 5950-3:1990, these connectors may be assumed to be 'ductile' (i.e., the measured slip capacity is greater than 7 mm) and the rules for partial shear connection design may be used. A similar conclusion can also be made when the characteristic slip capacity is compared with the Eurocode 4 requirements.

However, for the hollow core specimens with 8 mm transverse reinforcement bars, the characteristic slip of 3.27 mm means that the connectors do not satisfy the BS 5950-3: 1990 requirements for 'ductile' connectors. Furthermore, as can be seen from Table A.2, the slip capacities for the control specimens (with solid reinforced concrete slabs) have broadly similar values to their companion specimens with hollow core units. This observation clearly indicates that the slip capacity is simply affected by the size of the transverse reinforcement bars and not by the gap or the joint in the hollow core specimens.

From these tests, it is concluded that the minimum transverse reinforcement bar size that may be used in composite beams with chamfered-ended hollow core units is 12 mm.

Square-ended units

Push tests on specimens with headed stud connectors and square-ended hollow core units have recently been undertaken by Nip[36] at the University of Leeds. Like the push tests on chamfered-ended units, these specimens were again tested in the horizontal position (see Figure A.3).

Each specimen consisted[37] of a S275 254 × 254 × 73 UC with a single row of 19 mm × 100 mm long headed studs welded along the centre-line of the UC at 150 mm cross-centres. In a similar way as the chamfered-ended tests, these specimens consisted of two 1200 mm wide, or four 600 mm wide, × 150, 200 and 300 mm deep units supplied by Bison Concrete Products Ltd. Four milled slots were provided in each specimen to receive the transverse reinforcement bars (see Figure A.3). As shown in Table A.3 and Table A.4, these bars varied between 10 mm to 20 mm in diameter. The gap between the ends of the units was also varied from 40 mm to 140 mm (see Table A.3 and Table A.4). Only an *in situ* concrete infill was provided for these specimens.

Table A.3 *Push test results using square-ended hollow core units with 10 and 12 mm diameter transverse reinforcement*

Test	Infill conc. grade	Bar size ϕ (mm)	Infill cube strength f_{cu} (N/mm²)	Gap g (mm)	Width of hollow core unit (mm)	Measured load per stud (kN)	Predicted load per stud‡ (kN)	Model factor †
Leeds tests[36]								
T10 transverse reinforcement, 19 mm studs, 150 mm deep slab								
Half width 1	C25	10	23.38	65	600	63.8	51.4	1.24
Half width 2	C25	10	23.38	65	600	68.7	51.4	1.34
Half width 3	C25	10	23.38	65	600	60.8	51.4	1.18
T10 transverse reinforcement, 19 mm studs, 150 mm deep slab								
Half width 1.	C35	10	34.20	80	600	73.6	73.0	1.01
Half width 2	C35	10	40.17	80	600	89.2	76.3	1.17
Half width	C50	10	52.27	80	600	91.9	81.8	1.12
T10 transverse reinforcement, 19 mm studs, 150 mm deep slab								
Half width	C25	10	34.20	100	600	82.1	73.0	1.12
Half width	C25	10	38.80	140	600	83.2	75.6	1.10
T10 transverse reinforcement, 19 mm studs, 150 mm deep slab								
Half width	C35	10	38.80	120	600	83.6	75.6	1.11
Half width	C50	10	53.30	120	600	89.2	81.8	1.09
T12 transverse reinforcement, 19 mm studs, 150 mm deep slab								
Half width	C35	12	40.33	80	600	81.3	81.4	1.00
Half width	C50	12	74.53	80	600	116.8	87.2	1.34
T12 transverse reinforcement, 19 mm studs, 200 mm deep slab								
Half width	C25	12	29.02	80	600	101.4	70.6	1.44
T12 transverse reinforcement, 19 mm studs, 300 mm deep slab								
Full width	C20	12	19.91	100	1200	90.9	59.3	1.53
Full width	C25	12	26.41	100	1200	99.7	78.7	1.27
Full width	C45	12	46.60	100	1200	113.3	104.1	1.09

† Model factor = experimental value / predicted value
‡ Characteristic stud resistance from BS 5950-1: 1990 multiplied by reduction factor given by Equation (22)

Table A.4 *Push test results using square-ended hollow core units with 16 and 20 mm diameter transverse reinforcement*

Test	Infill conc. grade	Bar size ϕ (mm)	Infill cube strength f_{cu} N/mm²	Gap g (mm)	Width of hollow core unit (mm)	Measured load per stud kN	Predicted load per stud‡ kN	Model factor †
Leeds tests[36]								
T16 transverse reinforcement, 19 mm studs, 150 mm deep slab and gap g = 80 mm								
Half width 1	C25	16	18.74	80	600	114.3	64.1	1.78
Half width 2	C25	16	18.74	80	600	97.3	64.1	1.52
Half width 3	C25	16	18.74	80	600	101.9	64.1	1.59
Full width 1	C25	16	19.23	80	1200	115.1	80.6	1.43
Full width 2	C25	16	19.26	80	1200	102.0	80.7	1.26
Full width	C40	16	32.62	80	1200	114.6	112.5	1.02
T16 transverse reinforcement, 19 mm studs and 200 mm deep slab								
Half width	C25	16	22.79	40	600	88.6	61.2	1.45
Half width 1	C25	16	24.71	80	600	99.7	67.6	1.47
Half width 2	C25	16	24.71	80	600	106.1	67.6	1.57
Half width	C25	16	28.49	60	600	92.4	72.4	1.28
Half width	C25	16	28.49	100	600	100.1	77.9	1.28

† Model factor = experimental value / predicted value

‡ Characteristic stud resistance from BS 5950-1: 1990 multiplied by reduction factor given by Equation (22)

As can be seen from the last two columns of Table A.3 and Table A.4, the predicted resistances agree reasonably well with measured results from the push tests, albeit more conservative than the predictions for the chamfered-ended hollow core specimens.

Based on the assumptions that were used to develop the rules in BS5950-3: 1990[32], the slip capacities for the square-ended hollow core specimens are presented in Table A.5. The corresponding characteristic slips, calculated in accordance with Eurocode 4, are also shown for comparison purposes.

As can be seen from Table A.5, for specimens with 16 mm transverse reinforcement bars, the slip capacity of 9.0 mm means that, in terms of BS 5950-3: 1990, the connectors may be classified as 'ductile' (i.e. the slip capacity is greater than 7 mm) and the rules for partial shear connection design may be used. Conversely, the slip capacity of 3.69 and 4.84 mm for the specimens with 10 and 12 mm bars means that the connectors may not be considered as ductile with this level of transverse reinforcement.

Table A.5 *Slip capacities from push tests conducted by Nip[36] on studs embedded in slabs using square-ended hollow core units*

Slab type	Transverse reinforcement bar size (mm)	BS5950-3: 1990 slip capacity δ_u (mm)	Eurocode 4 characteristic slip capacity δ_{uk} (mm)
Hollow core units	10	3.69	5.25
Hollow core units	12	4.84	6.37
Hollow core units	16	9.00	9.83

When considering the Eurocode 4 requirements, the characteristic slip capacities shown in Table A.5 mean that in addition to the specimens with 16 mm bars, specimens with 12 mm transverse reinforcement bars may be classified as 'ductile' (i.e. the characteristic slip is at least 6 mm). However, in a similar way to the above comparisons with the BS 5950-3: 1990 requirements, the characteristic slip capacity of 5.25 mm for the specimens with 10 mm bars means that the connectors may not be considered as ductile with this level of transverse reinforcement.

It is interesting to note that the slip capacities shown in Table A.5 are less than those shown in Table A.2, for specimens with an identical transverse reinforcement bar diameter, but with chamfered-ended hollow core units. However, this may be as a consequence of shorter studs being used in the square-ended tests (100 mm cf. 125 mm for the chamfered-ended specimens).

From the above results, it is therefore concluded that the minimum transverse reinforcement bar size that may be used in composite beams with square-ended hollow core units is 16 mm.

A.1.3 Solid planks

An investigation on the shear resistance of headed stud connectors embedded within slabs using precast solid planks was undertaken at the University of Southampton[21]. In this research the 'standard' push test, having a similar arrangement to that shown in Figure A.1, was used to determine the load-slip characteristics of the stud connectors.

In total, 12 tests were undertaken using solid reinforced concrete slabs (test numbers 1 to 12), and 15 using slabs with solid precast planks (test numbers 13 to 27). Each specimen consisted of two 19 mm diameter studs embedded within 650 mm long × 450 mm wide × 150 mm deep slabs. The studs varied in height, with a nominal as-welded-height of 95 mm or 120 mm. To examine what effect the transverse reinforcement had on the resistance of the headed stud connectors, five different configurations, including different mesh sizes, were evaluated. Tests 13 to 27 employed 65 mm thick solid planks. Also, the gap between the ends of the planks was varied, at 109, 89 and 69 mm. Details of the tests are summarised in Table A.6.

Table A.6 *Push test results using solid planks*

Test	Cube strength f_{cu} N/mm²	Gap mm	Bar size mm	Measured load per stud kN	Predicted load per stud‡ kN	Model factor †
1	68.6	-	-	68.6	109.0	1.15
2	46.9	-	-	46.9	109.0	0.95
3	47.1	-	-	47.1	109.0	1.06
4	34.3	-	-	34.3	103.4	0.92
5	45.4	-	-	45.4	109.0	1.20
6	37.4	-	-	37.4	106.4	1.06
7	28.9	-	-	28.9	98.9	0.98
8	38.9	-	-	38.9	107.9	1.07
9	40.1	-	-	40.1	109.0	1.18
10	36.8	-	-	36.8	105.8	1.07
11	33.9	-	-	33.9	103.1	0.95
12	42.0	-	-	42.0	109.0	1.17
13	31.5	109.3	-	31.5	101.2	1.15
14	59.6	109.3	-	59.6	109.0	1.18
15	56.3	109.3	-	56.3	109.0	1.11
16	42.4	109.3	-	42.4	109.0	0.87
17	57.5	109.3	-	57.5	109.0	1.13
18	39.2	109.3	-	39.2	108.2	1.07
19	59.7	89.3	-	59.7	109.0	1.08
20	55.7	89.3	-	55.7	109.0	1.17
21	54.0	69.3	-	54.0	108.5	0.93
22	54.4	69.3	-	54.4	108.5	1.21
23	41.8	109.3	6	41.8	109.0	1.42
24	39.8	109.3	6 + 7	39.8	108.8	1.29
25	55.1	109.3	6 + 6	55.1	109.0	1.41
26	54.9	109.3	6 + 6	54.9	109.0	1.42
27	45.4	109.3	7 + 7	45.4	109.0	1.50

† Model factor = experimental value / predicted value

‡ Characteristic stud resistance from BS 5950-1: 1990 multiplied by reduction factor given by Equation (23)

As can be seen from the last two columns in Table A.6, the predicted stud resistance compares very well with the measured results from the push tests.

Although the slip capacity from these specimens was not given directly, it was reported by Moy & Taylor[21] that they were similar to those reported elsewhere for headed stud connectors in solid reinforced concrete slab specimens. As a result of this, no special considerations need be made for composite beams using solid precast planks and the rules for partial shear connection given in BS 5950-1: 1990 may be applied.

A.2 Tests on composite beams

Three full-scale composite beam tests with hollow core units were undertaken at the University of Nottingham[17].

The composite beams were simply-supported over a span of 5.6 m, and subjected to two point loads at quarter-points from each support. Each specimen consisted of a S275 grade $356 \times 171 \times 51$ UB with a single row of 19×125 mm long headed studs welded along the centre-line of the UB at 150 mm cross-centres. To form the concrete flange, 150 mm deep $\times$ 1200 mm wide chamfered-ended hollow core units, supplied by Bison Concrete Products Ltd., were employed. A total slab width of 1665 mm was provided for each specimen, which is slightly larger than the effective width that is assumed to exist in normal composite beams using *in situ* concrete. Four milled slots were provided in each unit to receive the transverse reinforcement bars, which varied in size and yield strength (see Table A.7). A gap of 65 mm was provided in each specimen between the ends of the hollow core units. An *in situ* concrete infill was used and, once a compressive cube strength of at least 25 N/mm² was achieved, the tests on the beam specimens were commenced.

The three beam specimens were given the designation of CB1, CB2 and CB3. A summary of the measured results is given in Table A.7.

Table A.7 *Summary of results from tests on full-scale composite beams with slabs using hollow core units*

Test	Transverse reinforcement bar size and measured yield strength	Infill cube strength f_{cu} N/mm²	Measured yield strength of structural steel p_y N/mm²	Maximum applied bending moment kNm	Model factor† for bending failure	Model factor† for failure of the studs	Model factor† for longitudinal splitting of the concrete
CB1	16 mm; 585 N/mm²	32.5	312.1	496	1.05	1.26‡	0.77
CB2	8 mm; 473 N/mm²	25.5	312.3	470	1.06	1.39	1.65‡
CB3	8 mm; 473 N/mm²	28.0	316.3	345	0.78‡*	0.26	0.29

† Model factor = experimental value / predicted value

‡ Relevant failure mode

* Composite action reduced due to presence of pre-cracked joint

The applied moment-deflection curves for the three composite beam tests are shown in Figure A.2. For Test CB1, visible distress to the specimen from cracks in the soffit of the slab was first observed at an applied moment of 396 kNm, just above the working load. At this point, yielding in the bottom flange of the UB commenced. As a result of this, strain gauge readings indicated[17] that the neutral axis had moved approximately 25 mm into the slab which, in turn, caused tension and further cracking to the soffit of the slab. As further load was applied, yielding of the steel section and the extension of the existing cracks caused a gradual reduction in stiffness. As the applied moment reached 496 kNm, sudden shearing of the stud connectors precipitated a rapid reduction in load (see Figure A.2). After this occurrence, the beam reached a new equilibrium position at an applied moment of approximately 350 kNm, where the remaining shear connectors transferred the longitudinal shear force into the slab. After attempting to apply further load, this test was terminated at a mid-span deflection of approximately span/90.
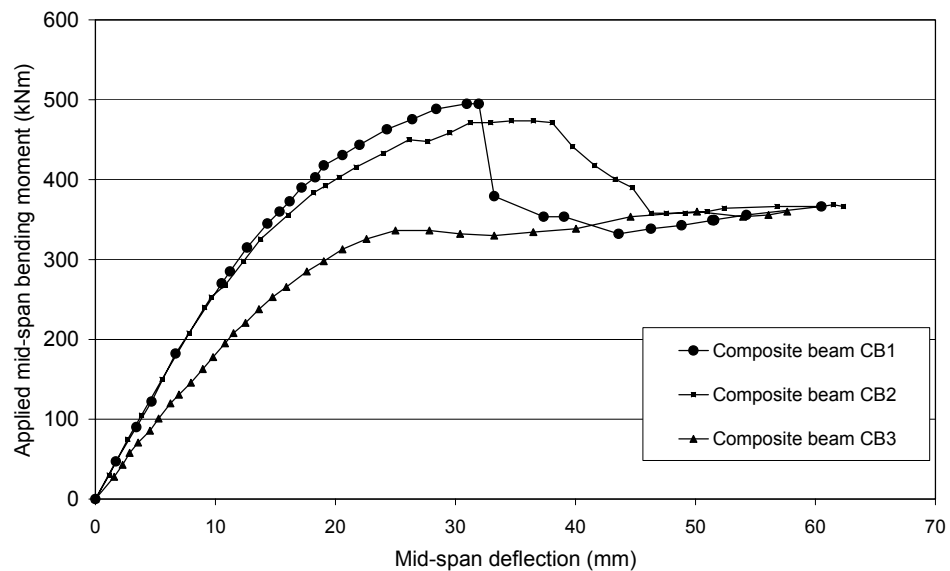
Figure A.2 *Applied moment vs. mid-span deflection from tests on full-scale composite beams with slabs using hollow core units*

The main difference in Test CB2, compared to its predecessor, was that 8 mm rather than 16 mm bars were used for the transverse reinforcement (see Table A.7). Under load, the behaviour of beam CB2 was remarkably similar to CB1, up to an applied moment of 280 kNm (see Figure A.2). Beyond this point, hairline cracks were visible over the ribs of the hollow core units, adjacent to the loading positions. On applying further load, a reduction in stiffness to the beam was observed due to further cracks in the hollow core units and yielding of the UB. As the applied moment reached its maximum value of 470 kNm, yielding of the transverse reinforcement bars was observed[17]. Following this, longitudinal splitting on the surface of the slab caused concrete failure around the shear studs and a gradual reduction to the applied moment. This test was terminated once the mid-span deflection reached span/90.

The only difference in specimen CB3 from CB2 was that a pre-cracked joint was introduced, by providing polythene sheets along the interface between the tapered ends of the hollow core units and the infill concrete. Under load, the

main characteristic that was observed in this specimen was that the position of the neutral axis was much lower than the companion tests[17] (located in the web of the steel beam), thereby indicating a reduction in the composite capabilities of the cross-section.

The behaviour of the specimen was linear up to an applied moment of 145 kNm (see Figure A.2), at which point the neutral axis moved further down into the web of the steel section; fine cracks were observed in the surface of the concrete slab, immediately above the pre-cracked joints. The stiffness for this specimen was 71% of that observed in the earlier tests[17], indicating a significant reduction of the effective slab width as a result of the bond at the interface between the hollow core units and the infill concrete having been destroyed. A gradual reduction in stiffness continued until crushing of the top surface of the slab adjacent to the loading position occurred at an applied moment of 327 kNm.

As the applied moment reached its maximum value of 345 kNm, continuous crushing of the concrete slab, yielding of the bottom flange of the UB and yielding of the transverse reinforcement bars leading to longitudinal splitting of the slab, were observed. Overall, it was reported[17] that the failure was very ductile, and the applied moment at maximum deflection was approximately equal to that found in specimens CB1 and CB2 (see Figure A.2). This test was terminated once the mid-span deflection reached approximately span/100.

In all tests, due to the span of the beams being relatively short, the end slip that developed at the maximum applied moment was quite small. For beams CB1, CB2 and CB3 this was 0.4, 2.6 and 5.9 mm respectively.

The test results from the three beam tests are summarised and compared to the predicted resistance according to three failure modes in Table A.7. The back-analysis method employed in calculating the values shown in this table followed the design procedure given in this publication, using measured material strengths and setting any partial safety strengths to unity. In interpreting the test results, the ratio of the actual failure load to that of the predicted resistance is termed the 'model factor'. Values greater, or equal to, unity for the critical mode of failure indicate satisfactory performance in the design method, while model factors less than unity for the non-critical modes of failure merely indicate that the resistance predicted by the design method was not critical. For example, in test CB1 longitudinal splitting of the concrete flange did not occur.

As can be seen from the critical modes indicated in Table A.7, the model factors of 1.05, 1.65 and 0.78 demonstrate adequate safety in the design methodology, particularly as measured material strengths have been used, and partial safety factors have been set to unity.

APPENDIX B   Worked Example

The following Worked Example considers the design of a 15.8 m span composite beam supporting a 7.2 m span hollow core slab. It is representative of use for a car park or similar application. The worked example considers:

- the design of the steel beam for the construction stage;

- the design of the composite beam and its shear connection; and

- serviceability calculations.

The critical design checks are those of:

- the angle of twist at the serviceability limit state for the out-of-balance loading during construction;

- the bending resistance of the composite beam in the normal condition; and

- the total deflection of the beam.

It is assumed that the perimeter beam provides sufficient tying action for robustness and diaphragm action.

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		Subject	Worked example - Simply Supported Composite Beam Arrangement and specification						
CALCULATION SHEET		Client DETR, Corus, PFF		Made by	SXH		Date	Mar. 2003	
				Checked by	MDH		Date	Mar. 2003	

ANALYSIS OF SIMPLY-SUPPORTED COMPOSITE BEAM IN A CAR PARK

GENERAL ARRANGEMENT

Section A - A
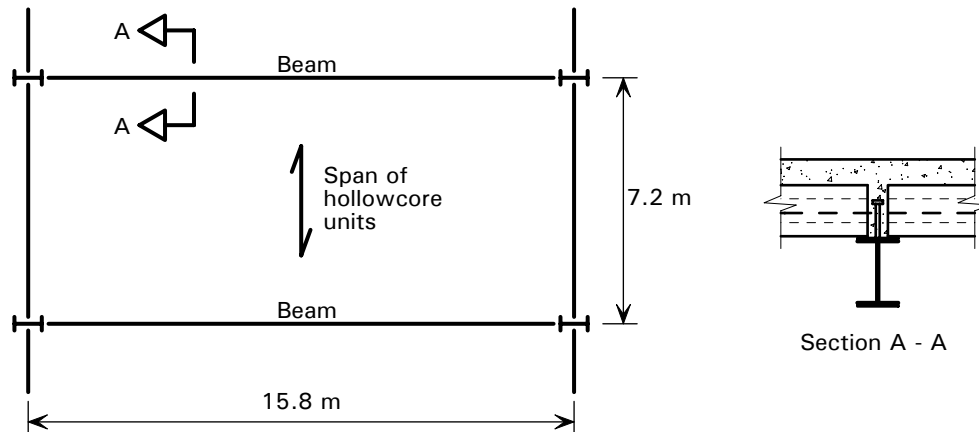
SPECIFICATION

Materials

S275 structural steel
E = 205 kN/mm^2

G = 78.8 kN/mm^2

Normal weight concrete Grade 30 f_{cu} = 30 N/mm^2

Density = 2400 kg/m^3 (wet)

= 2350 kg/m^3 (dry)

Precast Hollow Core Units

Unit depth	150 mm
Unit width w	1200 mm
Number of cores	9
Spacing of cores	133 mm
Self weight	2.4 kN/m^2
Length of concrete infill	500 mm
Thickness of structural topping	50 mm

Shear Connectors

19 mm diameter studs

120 mm as-welded height

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</table>

FLOOR LOADING

Hollow Core Unit and Topping

Weight of topping (wet) $= 2400 \times 9.81 \times 50 \times 10^{-6}$ $= 1.18$ kN/m^2

Hollow core unit weight   2.40

Total   3.58 kN/m^2

Construction Stage

a) Unbalanced loading

Floor (hollow core units)   2.40 kN/m²

Steel beam   0.32

Dead load   2.72 kN/m^2

b) Balanced loading

Floor (hollow core units + topping)   3.58

Steel beam   0.32

Dead load   3.90 kN/m^2

Imposed construction load   0.50 kN/m^2 BS 5950-3 Cl. 2.2.3

Composite Stage

Weight of topping (dry) $= 1.18 \times 2350/2400$ $= 1.16$ kN/m^2

Hollow core unit   2.40

Steel beam   0.32

Dead load   3.88 kN/m^2

Total imposed load   2.50 kN/m^2 BS 6399-1 Table 1

(No BS 6399 load reduction is used)

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</table>

INITIAL SELECTION OF BEAM SIZE

Try $610 \times 305 \times 238$ kg/m UB in S275 steel

Section properties

P202[38]

Depth of section	D	= 635.8 mm
Width of section	B	= 311.4 mm
Web thickness	t	= 18.4 mm
Flange thickness	T	= 31.4 mm
Second moment of area	I_x	= 210000 cm^4
Radius of gyration	r_y	= 7.23 cm
Elastic modulus	Z_x	= 6590 cm^3
	Z_y	= 1020 cm^3
Plastic modulus	S_x	= 7490 cm^3
Buckling parameter	u	= 0.887
Torsional index	x	= 21.3
Warping constant	H	= 14.5 dm^6
Torsional constant	J	= 785 cm^4
Area of section	A	= 303 cm^2

Section classification: class 1 plastic

BS 5950-1
Table 11

$T = 31.4$ mm $\ngtr 40$ mm $\quad \therefore p_y = 265$ N/mm^2

Table 9

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Job Title Design of Composite Beams using Precast Concrete Slabs

Subject Worked example – construction condition

CONSTRUCTION CONDITION – STEEL BEAM DESIGN

Ultimate Limit State

a) Unbalanced loading   (i.e. units on one side of the beam)

Design load $= (1.4 \times 2.72) \times \dfrac{7.2}{2}$ $= 13.71$ kN/m

Design shear F_v $= \dfrac{wL}{2} = \dfrac{13.71 \times 15.8}{2}$ $= 108$ kN

Design moment M_x $= \dfrac{wL^2}{8} = \dfrac{13.71 \times 15.8^2}{8}$ $= 428$ kNm

Shear capacity

$P_v \quad = \quad 0.6\, p_y\, A_v \quad = 1860$ kN > 108 kN   **OK**   P202 p. C-78

Moment capacity

For $610 \times 305 \times 238$ UB S275:

Class 1 plastic section M_{cx} $= 1980$ kNm > 428 kNm   **OK**   P202 p. C-59

Buckling resistance moment

Due to hollow core unit being placed on one side of UB flange, above the shear centre, check for a destabilising load condition.

Assume that a temporary lateral restraint is provided at mid-span during the construction condition.

L_{LT} $= 0.5L$ $=$ 0.5×15.8 $= 7.9$ m

For nominal torsional restraint at the supports, with both flanges free to rotate in plan   BS 5950-1 Table 13

L_E $= 1.2\, L_{LT}$ $= 9.48$ m

m_{LT} $= 1.0$ Cl. 4.3.6.6

λ $= 9480/72.3$ $= 131.1$

v $= \dfrac{1}{(1 + 0.05(\lambda / x)^2)^{0.25}}$

$= \dfrac{1}{(1 + 0.05(131.1 / 21.3)^2)^{0.25}} = 0.77$

λ_{LT} $= uv\lambda\sqrt{\beta_w}$ $= 0.887 \times 0.77 \times 131.1 \sqrt{1.0}$ $= 89.1$ Eqn (5)

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$\therefore p_b \quad = 143 \text{ N/mm}^2 \quad \therefore M_b \quad = P_b\, S_x \quad = 1071 \text{ kNm}$ | BS 5950-1, Table 16

$\overline{M}_x \quad = m_{LT}\, M_x \quad = 1.0 \times 428 \quad = 428 \text{ kNm} > M_b \quad$ **OK** | Eqn (7)

Combined bending and torsion

Taking the worst case of a minimum bearing width of 40 mm, eccentricity of load is:

$e \quad = B/2 - 20 \quad = \dfrac{311.4}{2} - 20 \quad = 135.7 \text{ mm}$ | Table 3.1

$\therefore$ Torsional moment is:

$T_q \quad = F_v e \quad = 108 \times 135.7 \times 10^{-3} \quad = 14.7 \text{ kNm}$

Buckling check

$a \quad = \sqrt{EH/GJ}$ | Eqn (4)

$\quad = \sqrt{205 \times 14.5 \times 10^{12} / 78.8 \times 785 \times 10^{4}}$

$\quad = 2192 \text{ mm}$

$L/a \quad = 15800/2192 \quad = 7.21$

It is assumed that only simple connections are provided, i.e. ends torsion fixed, warping free

From Table 4.1 $\quad \dfrac{\phi\, GJ}{T_q\, a} \quad = 0.769 \quad$ and $\quad -\dfrac{\phi''\, GJa}{T_q} \quad = 0.132$

$\phi \quad = \dfrac{0.769 \times 14.7 \times 10^{6} \times 2192}{78.8 \times 10^{3} \times 785 \times 10^{4}} \quad = 0.040 \text{ radians}$

$M_{yt} \quad = \phi M_x \quad = 0.040 \times 428 \quad = 17.1 \text{ kNm}$

$\therefore \sigma_{byt} \quad = M_{yt}/Z_y = \quad 17.1 \times 10^3/1020 \quad = 17 \text{ N/mm}^2$

$-\phi'' \quad = \dfrac{0.132 \times 14.7 \times 10^{6}}{78.8 \times 10^{3} \times 785 \times 10^{4} \times 2192} \quad = 1.431 \times 10^{-9}$

For a symmetrical I-section:

$W_{n0} \quad = hB/4 \quad = (635.8 - 31.4) \times 311.4 / 4 \quad = 47050 \text{ mm}^2$

$\therefore \sigma_w \quad = -EW_{n0}\phi'' \quad = 205 \times 10^3 \times 47050 \times 1.431 \times 10^{-9} \quad = 14 \text{ N/mm}^2$

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$$\frac{\overline{M}_x}{M_b} + \frac{(\sigma_{byt} + \sigma_w)}{p_y}\left[1 + 0.5\frac{\overline{M}_x}{M_b}\right] \le 1.0$$

Eqn (1)

$$= \quad \frac{428}{1071} + \frac{(17+14)}{265}\left[1 + 0.5 \times \frac{428}{1071}\right] \qquad = 0.54 \ < \ 1.0 \qquad \textbf{OK}$$

Local capacity check

$$\sigma_{bx} + \sigma_{byt} + \sigma_w \le p_y$$

Eqn (2)

$$\sigma_{bx} \quad = \frac{M_x}{Z_x} \qquad = \frac{428 \times 10^3}{6590} \qquad = 65 \text{ N/mm}^2$$

$$65 + 17 + 14 \qquad = 96 \text{ N/mm}^2 \ < \ 265 \text{ N/mm}^2 \qquad \textbf{OK}$$

Shear check

Strictly, the shear stresses due to combined bending and torsion should also be checked, although these are seldom critical for hot rolled sections. For completeness, this check will be demonstrated here.

P057[16]

Shear stresses due to vertical loading

At support

In web $\qquad \tau_{bw} \quad = \frac{F_v Q_w}{I_x t}$

For I-sections, $Q_w \quad = \frac{A}{2}y_w$

where: A is the total area of the cross-section; and y_w is the distance from the neutral axis to the centroid of area above the neutral axis.

For a symmetrical section, $A_{yw} / 2$ is equivalent to:

$$\frac{S_x}{2} \quad = \frac{7490}{2} \qquad = 3745 \text{ cm}^3$$

$$\tau_{bw} \quad = \frac{108 \times 10^3 \times 3745 \times 10^3}{210000 \times 10^4 \times 18.4} \qquad = 11 \text{ N/mm}^2$$

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At the root of the flange $\qquad \tau_{bf} = \dfrac{F_v Q_f}{I_x T}$

For I-sections, $Q_f = A_f y_f$

where: A_f is the area of half the flange; and y_f is the distance from the neutral axis to the centroid of A_f

$$\tau_{bf} = \frac{108 \times 10^3}{210000 \times 10^4 \times 31.4} (31.4(311.4 - 18.4)/2 \times (635.8 - 31.4)/2)$$

$$= 2 \text{ N/mm}^2$$

From Appendix B of P057 (Case 4), at the support:

$\phi' = 8.60 \times 10^{-6}$ and $\phi''' = -6.86 \times 10^{-13}$

Stresses due to pure torsion

In web $\quad \tau_{tw} = Gt\phi' \qquad = 78.8 \times 10^3 \times 18.4 \times 8.60 \times 10^{-6} \qquad = 13 \text{ N/mm}^2$

In flange $\tau_{tf} = GT\phi' \qquad = 78.8 \times 10^3 \times 31.4 \times 8.6 \times 10^{-6} \qquad = 21 \text{ N/mm}^2$

Stresses due to warping

$$\tau_w = -\frac{ES_{w1}\phi'''}{T}$$

where S_{w1} is the warping statical moment over the junction with the web

For symmetrical I-sections, $S_{w1} = hB^2T/16$

$$\therefore \tau_w = -205 \times 10^3 \times (-6.86 \times 10^{-13}) \left(\frac{(635.8 - 31.4) \times 311.4^2}{16} \right) = 0.5 \text{ N/mm}^2$$

Combined stresses

$$\tau_b + (\tau_t + \tau_w) \left[1 + 0.5 \frac{\overline{M}_x}{M_b} \right] \le 0.6 p_y \qquad\qquad \text{Eqn (3)}$$

In web $\quad \pm 11 + (13 + 0) \left[1 + 0.5 \times \dfrac{428}{1071} \right] \qquad = 27 \text{ N/mm}^2$

In flange $\quad \pm 2 + (21 + 0.5) \left[1 + 0.5 \times \dfrac{428}{1071} \right] \qquad = 28 \text{ N/mm}^2$

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Shear strength, p_v $= 0.6\, p_y$ $= 159$ N/mm² > 27 and 28 N/mm²   **OK**

As expected, section adequate in shear.

b) Balanced loading   (i.e. units on both sides)

Design load $= (1.6 \times 0.5 + 1.4 \times 3.90) \times 7.2$ $= 45.1$ kN/m

Design shear force, F_v $= \dfrac{wL}{2}$ $= \dfrac{45.1 \times 15.8}{2}$ $= 356$ kN

Design moment M_x $= \dfrac{wL^2}{8} = \dfrac{45.1 \times 15.8^2}{8}$ $= 1407$ kNm

Shear capacity

From sheet 4, P_v $= 1860$ kN > 356 kN   **OK**

Moment capacity

Since $0.6\, P_v > F_v$ the section is in low shear

$\therefore$ from sheet 4, M_{cx} $= 1980$ kNm > 1407 kNm   **OK**

Buckling resistance moment

Nominal bearing width $= 55$ mm   Table 3.1

Length of span that may be considered to be fully laterally restrained is $160 \times$ bearing width. From sheet 4, since a temporary lateral restraint is provided at mid-span during the construction condition:

$160 \times 55 \times 10^{-3}$ $= 8.8$ m $> L_{LT} = 7.9$ m   Section 4.1.2
$\therefore$ full lateral restraint is provided.

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Serviceability Limit State

a) Unbalanced loading

Angle of twist

$$T_q = \frac{14.7}{1.4} = 10.5 \text{ kNm}$$
Sheet 5

$$\phi = \frac{0.04}{1.4} = 0.029 \text{ radians}$$
Sheet 6

Since $\phi < 0.035$ radians (i.e. 2 degrees)   **OK**

b) Balanced loading

Vertical deflection of beam after construction

$$\delta = \frac{5 \times (3.9 \times 7.2) \times 15800^4}{384 \times 205 \times 1000 \times 210000 \times 10^4} = 53 \text{ mm} \ (= \text{Span}/298)$$

Bending stress in steel section

$$M_x = \frac{(3.9 \times 7.2) \times 15.8^2}{8} = 876 \text{ kNm}$$

$$\text{Bending stress, } \sigma_{bx} = \frac{876 \times 10^3}{6590} = 133 \text{ N/mm}^2$$

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NORMAL CONDITION – COMPOSITE BEAM DESIGN

Ultimate Limit State

Design load $\quad = (1.6 \times 2.5 + 1.4 \times 3.88) \times 7.2 \quad = 67.9$ kN/m

Design shear force, $\quad F_v \quad = 67.9 \times \dfrac{15.8}{2} \quad = 536$ kN

Design moment, $\quad M_A \quad = \dfrac{67.9 \times 15.8^2}{8} \quad = 2119$ kNm

Shear capacity

From sheet 4, $\quad P_v \quad = 1860$ kN > 536 kN $\qquad\qquad$ **OK**

Effective breadth of slab

$B_e \quad = \dfrac{span}{8}$ but not greater than total width of infill + gap $\qquad$ Eqn (9)

$\dfrac{span}{8} \quad = \dfrac{15800}{8} \quad = 1975$ mm

Gap $g \quad = 311.4 - 2 \times 55 \quad = 201.4$ mm

Total infill width + gap $\quad = 2 \times 500 + 201.4 \quad = 1201.4$ mm

$\therefore B_e \quad = 1201.4$ mm

Moment capacity

Since $0.6\, P_v > F_v$ the section is in low shear

Tension resistance of steel beam

$R_s \quad = A\, p_y \qquad = 303 \times 10^2 \times 265 \times 10^{-3} \qquad = 8029.5$ kN $\qquad$ Section 4.3

Compressive resistance of concrete flange

$R_c \quad = 0.45\, f_{cu}\, B_e\, D_s \quad = 0.45 \times 30 \times 1201.4 \times 200 \times 10^{-3} \quad = 3243.8$ kN

As $R_s > R_c$ plastic neutral axis lies within steel section

$R_f \quad = BT\, p_y \qquad = 311.4 \times 31.4 \times 265 \times 10^{-3} \qquad = 2591.2$ kN

$R_w \quad = R_s - 2R_f \quad = 8029.5 - 2 \times 2591.2 \quad = 2847.1$ kN

$\therefore$ As $R_c > R_w$ Case (b) plastic neutral axis within flange of UB

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For full shear connection: $M_c = R_s \dfrac{D}{2} + R_c \dfrac{D_s}{2} - \dfrac{(R_s - R_c)^2}{R_f} \dfrac{T}{4}$ Eqn (12)

$M_c = 8029.5 \times \dfrac{635.8}{2 \times 10^3} + 3243.8 \times \dfrac{200}{2 \times 10^3} - \dfrac{(8029.5 - 3243.8)^2}{2591.2} \times \dfrac{31.4}{4 \times 10^3}$

$ = 2808$ kNm

As $M_c > 2119$ kNm   **OK**

Shear Connection

Characteristic resistance of shear connector:

For 19 mm dia × 120 mm long in Gr 30 NWC

$Q_k = 100$ kN   Table 4.2

Design capacity for positive moments $Q_p = 0.8\, Q_k\, k$

Reduction factor due to presence of hollowcore units

$k = \beta \varepsilon \sqrt{\omega}$ Eqn (22)

Since gap $g = 201.4$ mm > 70 mm, take $\beta = 1.0$

Taking the diameter of transverse reinforcement $\phi = 16$ mm

$\varepsilon = \dfrac{\phi + 20}{40} \qquad = 0.90$

Width of hollowcore unit $= 1200$ mm

$\omega = \dfrac{w + 600}{1200} \qquad = 1.5$

$k = 1.0 \times 0.90 \sqrt{1.5} \quad = 1.1$

But k may not be more than 1.0 $\therefore$ take $k = 1.0$

$\therefore$ Design resistance of one shear connector $Q_p = 0.8 \times 100 \times 1.0 \qquad = 80$ kN

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Number of shear connectors required for full shear connection (i.e. R_q greater, or equal to the lesser of R_s and R_c):

$$\therefore N_p \quad = 3243.8/80 \quad = 41$$

$$\text{Spacing of connectors} \quad = \frac{15800}{2 \times 41} \quad = 190 \text{ mm}$$

$\therefore$ Provide 19 mm dia $\times$ 120 mm long studs @ 190 mm cross-centres

Transverse Reinforcement

Total longitudinal shear force per unit length

$$v \quad = (3243.8/41) \times 10^3/190 \quad = 416 \text{ N/mm} \qquad \text{Eqn (25)}$$

$$\text{Shear resistance } v_r \quad = 0.03\, A_{cv}\, f_{cu} + 0.7\, A_{sv}\, f_y \le 0.8\, A_{cv}\, \sqrt{f_{cu}} \qquad \text{Eqn (26)}$$

For shear planes a-a $\quad v \quad = 416/2 \quad = 208 \text{ N/mm}$ $\qquad$ Fig. 4.5

Assuming that transverse reinforcement is placed in every second core within hollow core unit, spacing $= 267$ mm

$$v_r \quad = 0.03 \times 1.0 \times 200 \times 30 + 0.7 \times \frac{\pi \times 16^2}{4} \times \frac{460}{267} \quad = 423 \text{ N/mm} \qquad \text{Eqn (26)}$$

But $v_r \le 0.8 \times 1.0 \times 200 \sqrt{30} \quad = 876$ N/mm $\qquad$ **OK**

$v_r \quad = 423$ N/mm > 208 N/mm $\qquad$ **OK**

For shear planes b-b $\quad v \quad = 416$ N/mm $\qquad$ Fig. 4.5

$$v_r \quad = 0.03 \times 1.0 \times (2 \times 120 + 19) \times 30 + 0.7 \times \frac{\pi \times 16^2}{4} \times \frac{460}{267} \times 2 \quad = 719 \text{ N/mm} \qquad \text{Eqn (26)}$$

But $v_r \le 0.8 \times 1.0 \times (2 \times 120 + 19)\, \sqrt{30} \quad = 1135$ N/mm $\qquad$ **OK**

$v_r \quad = 719$ N/mm > 416 N/mm $\qquad$ **OK**

Therefore, 16 mm bars @ 267 mm cross-centres is adequate as transverse reinforcement

Support Flexibility

Shear resistance of hollow core units, from manufacturers data $\quad = 160$ kN/unit.

For 1200 mm wide units: $V_{Rd} = 160/1.2 \quad = 133$ kN/m

Factored shear load at ends of the slab

$V_{Sd} \quad = 64.7/2 \quad = 32.4$ kN/m

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Job Title **Design of Composite Beams using Precast Concrete Slabs**

Subject **Worked example – normal condition**

CALCULATION SHEET

Client
DETR, Corus, PFF

Made by **SXH** Date **Dec 2002**

Checked by **MDH** Date **Mar 2003**

Factored shear load due to loads applied <u>after</u> the installation of the units

$V_{Sd,q}$ = 32.4 – (2.4 × 1.4) × 7.2/2 = 20.3 kN/m < 0.35 V_{Rd} **OK**

Fire Resistance

Height of top floor less than 30 m. Car park open-sided

Therefore, 15 minutes fire resistance required

Bldg Reg.s App Doc B

Load factors for fire limit state (FLS)

Dead load γ_f = 1.0 and Imposed load γ_f = 0.8

BS5950-8 Table 2

Load at FLS = (0.8 × 2.5 + 1.0 × 3.55) = 5.55 kN/m²

Applied moment at FLS = $\dfrac{(5.55 \times 7.2) \times 15.8^2}{8}$ = 1247 kNm

Load ratio R = 1247 / 2829.1 = 0.44

For this size of UB, since R < 0.6 the section may be left unprotected

P186[39] Appendix C

Serviceability Limit State

Effective modular ratio $\alpha_e = \alpha_s + \rho_\ell (\alpha_\ell - \alpha_s)$

BS5950-3 Cl. 4.1

From Table 4.3, α_s = 6 α_ℓ = 18

Long term loading: Dead load 3.88 kN/m²

Total loading: 3.88 + 2.50 = 6.38 kN/m²

∴ ρ_ℓ = $\dfrac{3.88}{6.38}$ = 0.61

∴ α_e = 6 + 0.61 (18-6) = 13

Depth of elastic neutral axis below top of concrete flange y_g

y_g = $\dfrac{A\alpha_e (D + 2D_s) + B_e D_s^2}{2(A\alpha_e + B_e D_s)}$

Eqn (28)

= $\dfrac{303 \times 10^2 \times 13 (635.8 + 2 \times 200) + 1201.4 \times 200^2}{2(303 \times 10^2 \times 13 + 1201.4 \times 200)}$ = 359.6 mm

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Second moment of area of composite section

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

Eqn (29)

$$= 210000 \times 10^4 + \frac{1201.4 \times 200^3}{12 \times 13}$$

$$+ \frac{303 \times 10^2 \times 1201.4 \times 200 (635.8 + 200)^2}{4 (303 \times 10^2 \times 13 + 1201.4 \times 200)} \quad = 416700 \text{ cm}^4$$

Check stresses in steel and concrete:

Concrete flange $Z_g = \dfrac{416700 \times 13}{359.6 \times 10^{-1}}$ 150625 cm^3

Bottom flange of steel section

$$Z_s = \frac{416700 \times 10}{(635.8 + 200 - 359.6)} \quad = 8750 \text{ cm}^3$$

Loading on composite section at SLS:

Imposed load   = 2.50 kN/m^2

Design load   = 2.50 × 7.2   = 18 kN/m

Design moment $M = \dfrac{w\ell^2}{8} = \dfrac{18 \times 15.8^2}{8}$ = 562 kNm

Stress in concrete $= \dfrac{M}{Z_g} = \dfrac{562 \times 10^3}{150625}$ = 4 N/mm^2

$< 0.5 f_{cu}$ = 15 N/mm^2 **OK**

Bending stress in steel member:

$= \dfrac{M}{Z_s} = \dfrac{562 \times 10^3}{8750}$ = 64 N/mm^2

<table>
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</table>

Total stress in bottom flange:

Construction stage (sheet 9)   133

Composite stage   <u>64</u>

                        197 Nmm^2

As 197 N/mm^2 < p_y = 265 N/mm^2 **OK**

Deflection due to imposed load:

$$\delta_c = \frac{5\,wL^4}{384\,EI} = \frac{5 \times 18 \times 15800^4}{384 \times 205 \times 1000 \times 416700 \times 10^4}$$

$$= 17 \text{ mm} \ (= \text{Span}/929) \qquad \textbf{OK}$$ Table 4.4

Total deflection

After construction (no precambering)   53 (sheet 9)

Composite stage   <u>17</u>

                       70 mm (Span/226)   **OK**   Table 4.4

Dynamic Considerations

Load considered for dynamic properties:

Dead load   3.88

10% imposed load   <u>0.25</u>

                      4.13 kN/m^2

Design load   = 3.8 × 7.2   = 29.7 kN/m

Use dynamic modular ratio of α = 5.4   Table 4.3

Recalculate I_g (see Calculation sheet 14)

I_g = 539100 cm^4

Deflection of beam due to the above loads:

$$\delta_{sw} = \frac{5\,wL^4}{384\,EI} = \frac{5 \times 29.7 \times 15800^4}{384 \times 205 \times 1000 \times 539100 \times 10^4} = 21.8 \text{ mm}$$

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Natural frequency of beam

$$f_{beam} = \frac{18}{\sqrt{\delta_{sw}}} = 3.9 \text{ Hz}$$

Eqn (31)

Second moment of area of slab $= 90000 \text{ cm}^4/\text{m}$

As the beams are directly framed into the supporting columns, only the secondary beam mode need be considered for determining the fundamental frequency of the floor.

Section 4.6.4

Deflection of slab taking fixed ended boundary conditions

$$\delta_{sw} = \frac{wL^3}{384\,EI} = \frac{29.7 \times 7200^3}{384 \times 205 \times 90000 \times 10^4} = 0.2 \text{ mm}$$

$$f_{slab} = \frac{18}{\sqrt{\delta_{sw}}} = 40.2 \text{ Hz}$$

Using Dunkerly's approximation, the system frequency is:

$$f_0 = (3.9^{-2} + 40.2^{-2})^{-0.5} = 3.9 \text{ Hz} > 3.0 \text{ Hz} \quad \textbf{OK}$$

Eqn (32)

This is acceptable for a car park.