Precast Concrete Floors in Steel Framed Buildings

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FOREWORD

Hot-rolled steel frames with precast floors are a common structural solution for a range of multi-storey building types. These include commercial, retail, residential, schools, hospitals and car parks. Structural design guidance of this effective structural combination is covered by previously published three SCI publications, *Design of composite beams using precast concrete slabs* (P287), *Design of asymmetric Slimflor beams with precast concrete slabs* (P342) and *Design of multi-storey braced frames* (P334). Corus provided the funding for the background research relating to composite construction using precast floors in steel framed buildings; Corus Construction and Industrial provided funding for P342 and P344 and Corus Construction Centre for P287.

The scope of this publication is to provide guidance on the design and construction aspects of using precast floors in steel framed buildings which are not covered by other design guides and where there is more recent guidance. Where appropriate guidance is provided elsewhere, the reader is referred to these sources. Particular topics that are addressed in this publication include; designing to avoid disproportionate collapse, floor diaphragm action, construction best practice, provision of temporary support and detailing recommendations for sound insulation. These topics are demonstrated in four worked examples provided for different building types.

Product specific data is available from precast unit manufacturers but generic guidance as provided in this publication has, until now, not been readily available.

Contents

SUMMARY

This publication provides guidance on precast concrete floors in steel framed buildings. It covers:

- designing to avoid disproportionate collapse

- floor diaphragm action

- construction procedures

- provision of temporary support

- detailing recommendations for compliance with building regulations.

The design and detailing of hollow core, solid and lattice slab precast floor units and precast stair units are included. The range of available precast products and situations in which they are best used with hot-rolled steel frames are described and illustrated by six case studies. Three worked examples of precast unit floors in different building types are also included.

Structural design guidance provided in previously published three SCI publications, *Design of composite beams using precast concrete slabs* (P287), *Design of asymmetric Slimflor beams with precast concrete slabs* (P342):and: *Design of multi-storey braced frames* (P334), is not repeated.

Planchers préfabriqués en béton pour immeubles à ossature en acier

Résumé

Cette publication est destinée à servir de guide pour l'utilisation de planchers préfabriqués en béton dans les immeubles à ossature en acier.

Il couvre :

- *La robustesse du dimensionnement pour éviter des ruptures disproportionnées;*

- *L'effet diaphragme du plancher;*

- *Les procédures de constructions;*

- *L'utilisation de supports temporaires;*

- *Les détails recommandés pour satisfaire aux exigences des codes de construction.*

Le dimensionnement et les détails de réalisation sont donnés pour des unités de planchers préfabriqués de type plancher creux, plancher plein, plancher en treillis et pour des unités d'escaliers préfabriqués. La gamme de produits disponibles est fournie et des exemples de bonnes utilisations, avec des cadres réalisés avec des poutrelles en acier, sont décrits et illustrés par six exemples. Trois exemples de planchers préfabriqués, pour différents types d'immeubles, sont également décrits.

La guidance pour le dimensionnement, qui a fait l'objet de trois publications SCI, Design of Composite Beams using Precast Concrete Slabs (P287), Design of Asymmetric Slimflor Beams with Precast Concrete Slabs (P342) et Design of Multi-Storey Braced Frames (P334), n'est pas reprise dans cette publication.

Betonfertigteildecken in Stahlbauten

Zusammenfassung

Diese Publikation bietet eine Anleitung zu Betonfertigteildecken in Stahlbauten. Sie behandelt:

- *Berechnung zur Vermeidung des Bauteilversagens*

- *Scheibenwirkung*

- *Bauverfahren*

- *Montageunterstützung*

- *ausführliche Empfehlungen zur Übereinstimmung mit den Bauvorschriften*

Die Berechnung und Konstruktion von Hohlkörper-, Massiv- und Gitterträgerfertigteildecken sowie Fertigtreppen ist eingeschlossen. Die Bandbreite verfügbarer Fertigprodukte und Situationen in denen sie am besten mit warm gewalzten Stahltragwerken eingesetzt werden, geht aus sechs Fallstudien hervor. Drei Berechnungsbeispiele von Fertigteildecken in verschiedenen Gebäudetypen sind ebenso enthalten.

Berechnungsempfehlungen aus den drei vorhergehenden Publikationen, Berechnung von Verbundträgern mit Betonfertigteildecken (P287), Berechnung asymmetrischer Flachdeckenträger mit Betonfertigteildecken (P342) und Berechnung mehrgeschossiger unverschieblicher Tragwerke (P334), werden nicht wiederholt.

Solai prefabbricati in calcestruzzo per sistemi intelaiati in acciaio

Sommario

Questa pubblicazione costituisce una guida per l'uso dei solai in calcestruzzo nelle strutture intelaiate in acciaio ed affronta le seguenti tematiche:

- *progettazione per evitare il collasso strutturale;*

- *azione membranale di piano;*

- *procedure costruttive;*

- *prestazioni di vicoli provvisionali (puntellazione);*

- *disposizione costruttive per il soddisfacimento delle regolamentazioni edilizie.*

Vengono trattate la progettazione e le disposizioni costruttive per moduli prefabbricati di solette alleggerite, piene e a graticcio, comprendendo anche i moduli specifici per la realizzazione delle scale. Viene descritta, con la presentazione di 6 casi significativi, la variabilità dei prodotti prefabbricati disponibili e le situazioni di impiego ottimale con riferimento a telai in acciaio realizzati con profili laminati a caldo. Nella pubblicazione sono riportati 3 esempi pratici relativi a solai con moduli prefabbricati per differenti tipologie di edifici.

Non vengono replicate le regole progettuali riportate nelle precedenti pubblicazioni dello Steel Construction Institute (SCI): Progettazione di travi composte con solette prefabbricate in calcestruzzo (P 287), Progettazione di travi Slimflor asimmettriche con solette prefabbricate in calcestruzzo (P342) e Progettazione di telai controventati multipiano (P334).

Forjados de hormigón prefabricado en edificios de estructura de acero.

Resumen

Esta publicación es una guía para el uso de forjados de hormigón prefabricado en edificios de estructura de acero y cubre los siguientes aspectos:

- *cálculo para evitar el colapso desproporcionado*

- *acción del forjado como diafragma*

- *procesos constructivos*

- *uso de apoyos temporales*

- *recomendaciones de detalles para el cumplimiento de la normativa de edificación.*

Se incluyen el cálculo y detalles de forjados de losas de hormigón aligeradas, macizas y de celosía, así como unidades de escaleras prefabricadas. La gama de los productos prefabricados disponibles y las situaciones en las que su uso en edificios de estructura de acero con perfiles laminados en caliente es más recomendable se ilustran a través de seis casos prácticos. También se incluyen tres ejemplos resueltos sobre forjados prefabricados en diferentes tipos de edificaciones.

Las recomendaciones de diseño dadas de previas publicaciones del SCI (Proyecto de vigas mixtas mediante losas prefabricadas de hormigón (P287), Proyecto de vigas Slimflor asimétricas con losas de hormigón prefabricado (P342) y Proyecto de estructuras arriostradas de varias plantas (P334)) no están duplicadas en esta publicación.

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1   INTRODUCTION

1.1   About this publication

This publication provides best practice information on detailing, construction methods, and how to satisfy building regulations for the use of precast floor units in hot-rolled steel framed buildings. The publication has been sponsored by Bison Concrete Products Limited but the information presented is generic.

1.2   Introduction to precast units

Precast concrete floor units are widely used in all types of building. It is estimated that 50% of multi-storey steel framed buildings use precast concrete floors. The precast concrete floor unit is used to span between supporting steel beams as an alternative to in-situ concrete and metal decking. The joints between the units are grouted with in-situ concrete and a concrete structural topping is provided for some applications.

Precast concrete floor units are prestressed concrete slabs and are available as solid elements or with longitudinal hollow cores. The units are available in different depths in order to satisfy the various performance needs for span and loading. The units are normally 1200 mm wide and can be up to 20 m long. The different types of precast concrete unit are described below.

1.2.1   Hollow core floor units

Hollow core floor units are used in all building types. The section profile incorporates hollow cores (Figure 1.1) to reduce the self-weight without significant reduction in section stiffness. Hollow core units typically range in depth from 150 mm to 450 mm. The majority of manufacturers produce units with a nominal width of 1200 mm. However, both wider and narrower units are manufactured. Reinforcement is provided by high tensile prestressing strand or wire that has an ultimate strength of more than three times that of conventional high tensile reinforcement. The structural performance that results from the combination of these features produces a slab that is highly efficient and economic for a wide range of load/span situations. Hollow core units can be used with or without structural topping. Load span tables for preliminary sizing of hollow core units are provided in Section 3.

Figure 1.1 *Hollow core floor units*

Hollow core floor units generally have fire resistances of either 1 hour or 2 hours depending on the depth and mass of the unit.

The edges of hollow core units are profiled to provide an effective shear key so that when the joints between units are grouted (Figure 1.2) the individual units behave as a system acting together, similar to a monolithic slab. The grout is commonly a C20/25 or C25/30 concrete with 10 mm aggregate.
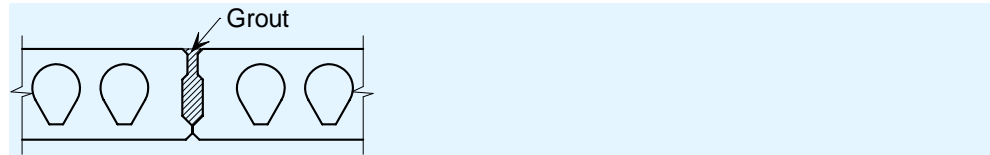
Figure 1.2 *Grouted joint between hollow core units*

The shape of the cores varies according to the manufacturer and the depth of the unit. Core profiles can be circular, square, elongated circles and bulb shaped. Some typical cross-sections of hollow core units are shown in Figure 1.3. Figure 1.4 shows the typical cross-section of a 150 mm 'sound slab'. The 'sound slab' has smaller hollow cores than a standard 150 mm hollow core unit so that its density, and hence its sound insulation performance, is increased. Guidance on sound insulation is provided in Section 0.
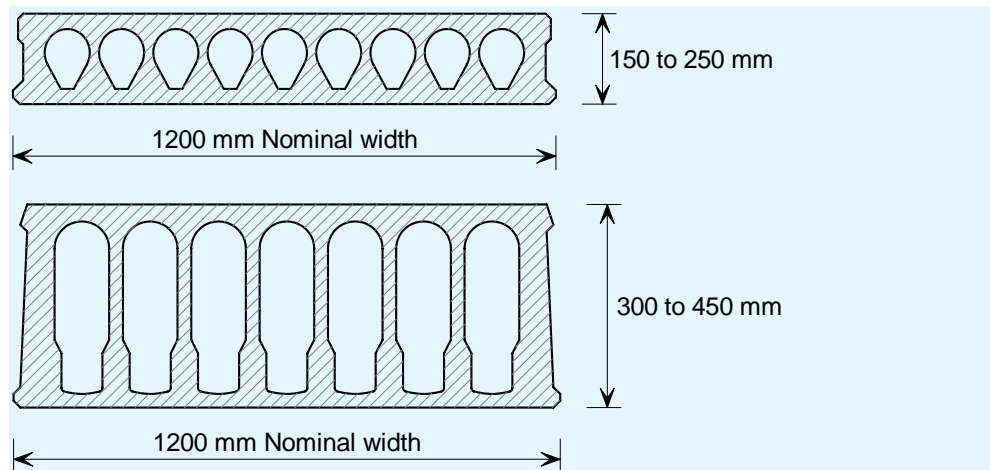
Figure 1.3 *Typical cross sections for hollow core units*
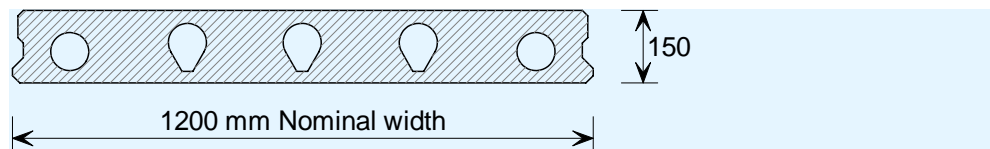
Figure 1.4 *Typical 'sound slab' hollow core floor unit*

1.2.2 Solid precast floor units

Solid precast floor units, as shown in Figure 1.5, are normally 75 mm to 100 mm thick and are generally used in conjunction with a structural in-situ concrete topping. The solid precast units are prestressed using high tensile reinforcement as used for hollow core units. The overall structural depth (including in-situ concrete topping) typically ranges from 150 mm to 200 mm. This is dependent on the floor span and the loading for which the floor is designed. Solid units are generally used where only short spans are required (a 100 mm solid unit with a structural topping can span up to 5 m without propping).

As is the case for hollow core units, the nominal width is usually 1200 mm. The cross-section of a typical solid unit is shown in Figure 1.6.

Figure 1.5 *Solid precast floor units*

75 to 100 mm

1200 mm Nominal width
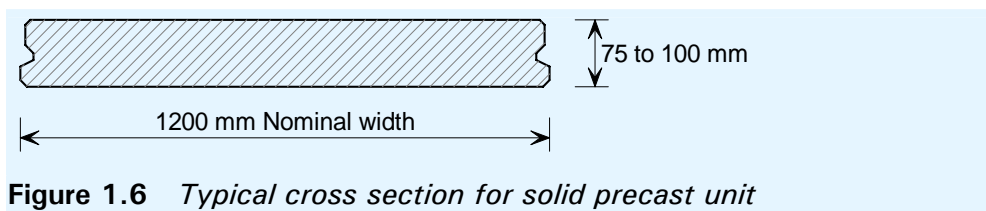
Figure 1.6 *Typical cross section for solid precast unit*

1.2.3 Precast lattice slab floor units

Precast lattice slab floor units are typically 50 to 100 mm thick, reinforced with steel bars forming a lattice girder that is only partially embedded in the precast unit. Figure 1.7 shows typical precast lattice slab floor units. The lattice girders are fabricated from steel reinforcement bars typically 8 mm to 14 mm in diameter and are spaced at 300 to 600 mm centres. Lattice slabs are manufactured in widths up to 2.5 m. With an overall slab depth of up to 400 mm, spans of up to 11 m can be achieved.

The combination of the solid slab and the integral lattice reinforcement forms permanent formwork. The protruding lattice ensures composite action between

the precast unit and the in-situ concrete in the permanent condition. The supporting steel beams can be designed as non-composite or composite.

Lattice slabs can be used in all applications. They have the advantage that they provide a high quality smooth soffit (because they are wet cast in steel moulds), the finished slab can have continuity over supports, has good durability and can be designed for tying in two directions. However, lattice slabs normally require propping during construction (typically 2.5 m centres), which can make them a more expensive option than hollow core units.

The detail design of floors constructed using precast lattice slabs is not covered by the scope of this publication but Appendix C highlights some of the principal design considerations for precast lattice slab floors.

Figure 1.7 *Precast lattice slab floor units*

1.2.4 Precast concrete staircases

Precast concrete staircases, as shown in Figure 1.8, are an economic and effective means of providing stairs in any structure. Well designed stairs are a major element in any successful construction programme. Manufacturers are able to provide bespoke configurations of precast stairs to suit the requirements of clients. Each precast flight of stairs uses approximately 1 m^3 of concrete.

Modern precasting factories are equipped with a range of standard but fully adjustable moulds that will accommodate most stair layouts. The use of adjustable moulds enables staircases to be produced efficiently. Lifting points are cast in at the factory, which makes on site placing, safe, quick and easy.

Figure 1.8   *Precast concrete staircase*

1.3   Advantages of precast units with steel frame

Precast floors used in conjunction with a structural steel frame offer many benefits over alternative structural solutions, such as in-situ concrete, masonry and timber.

1.3.1   Advantages during construction

Speed of erection – The time-consuming activities such as propping, shuttering and concrete pouring are virtually eliminated.   Construction time is further reduced by the utilisation of cast-in lifting hooks, rather than under-slung chains.   The lifting hooks enable the units to be landed in exactly the correct position without the need to adjust their position after the chains are removed. For composite construction, shear studs can be factory welded rather than site welded (which would be necessary for a metal deck option) hence this operation is removed from the site critical path.   Fewer steel beams are needed with precast units compared to conventional metal decking (see Section 1.3.2); this leads to quicker erection of the steelwork.

More secure working platform – Propping is generally not required with hollow core floors.   Therefore, after a precast floor is erected, it is immediately available as a flat working platform.   (Note: the floor should not be used for heavy construction plant e.g. motorised mobile access platforms until the joints

have been grouted and cured.) Steel deck systems often require propping and their load carrying capacity is limited until the in-situ concrete has cured.

Minimum in-situ concrete – Using a precast floor reduces the volume of in-situ concrete that is required, particularly if there is no concrete topping. Placing of in-situ concrete can be a complex and time consuming site operation.

Preformed features – Precast floors can be provided with factory formed service holes, thus avoiding the need for laborious setting out and shuttering on site.

1.3.2 Advantages for design

Structural efficiency – Hollow core units offer an ideal structural section by reducing the deadweight whilst providing the maximum structural efficiency within the slab depth. The structural efficiency is enhanced when hollow core floors are designed to act compositely with steel beam sections.

Steel economy - The spanning capabilities of the hollow core units are 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). Typically, hollow core units can achieve bay centres of 7.5 m and greater, which far exceeds that provided by most metal deck solutions.

Composite steel beam design – Composite steel beam design incorporating hollow core or solid units provides a structural and cost efficient solution for steel frames. The total tonnage of steel beams required can be reduced by up to 40% compared with the beams required to support a metal deck floor.

Optimised shear stud design - The use of hollow core and solid units for composite steel beam design allows the optimum number of shear studs to be provided on the steel section because, unlike composite beams with metal decking, the stud spacing is not restricted by the pitch of the deck troughs.

Reduced deflections – Precast units have a natural pre-camber which offsets the imposed load deflections.

Diaphragm action – Precast unit floors can provide a floor with full diaphragm action to the building. The joints between the units must be structurally grouted but a structural concrete topping is not necessarily required.

Acoustic performance – Construction details that satisfy the acoustic requirements for attached dwellings in Part E of the Building Regulations are available for precast floors in steel framed buildings. Manufacturers are currently in the process of obtaining Robust Detail status for these solutions.

Factory-produced quality – Precast floor units are factory-produced which means they are manufactured in an environment which is more controlled than a building site. Therefore the quality of precast products can be assured (see Section 1.4.3).

Flat soffits – A flat soffit is created, either between discrete downstand beams or between asymmetric beams incorporated into the depth of the precast floor. The precast units are manufactured on high quality steel beds and in appropriate cases the undersides are suitable for direct decoration. The provision of units

suitable for direct decoration needs to be requested to ensure correct handling and stacking is carried out to avoid damage.

1.3.3 Maximising the advantages

Precast floor units can be used economically in many types of steel framed structures. However, there are certain choices that can be made and processes followed which will maximise the benefits achieved in practice.

The steel frame and floor beam arrangement can influence the efficiency of the floor construction. Precast floor units are best suited to floor layouts with repeated rectangular grids because this minimises the amount of non-standard units and trimmer beams that need to be used. Precast units can be produced to fit floors with non-rectangular grids but this adds to the manufacturing process and it will often mean that additional trimmer beams or steel hanger brackets will be required.

At an early stage in the design process, the designer should consult with the precast unit manufacturer. The design office of the manufacturer will be able to provide guidance on how to optimise the beam layout for a precast unit floor, advise on floor unit layout to minimise non-standard units and specify minimum beam flange widths to ensure adequate bearing distance (see Section 3.3).

The manufacturer will be able to provide guidance on detailing for tying to avoid disproportionate collapse (see Section 4.2) and ensuring floor diaphragm action (see Section 2.6). However, the responsibility for the design remains with the designer of the frame who needs to be satisfied that the details proposed by the precast unit manufacturer are appropriate.

1.4 Manufacturing process

Precast floor units can be manufactured by slip forming, extrusion or wet casting. The slip form and extrusion methods of production involve the floor units being cast on a long line system of metal casting beds with anchor blocks at each end. The prestressing wires are tensioned between the anchor blocks and the concrete is formed around the wire by a casting machine. In the wet cast method of production a mould is used to form the precast unit. The slip form method of production is described in more detail below.

1.4.1 Slip form production

The slip form method of production can be divided into the following phases of manufacture:

- Concrete mix design

- Wire management and stressing

- Slip forming of units

- Plotting and formation of features

- Installation of lifting hooks

- Curing, cutting, stripping and storage of units.

Concrete mix design

The concrete mix normally has a target minimum cube strength of a 55 N/mm^2 at 28 days. The concrete may contain small controlled quantities of recycled concrete aggregate, recycled slurry water, PFA and air entraining agent all designed to produce a workable mix with a low water to cement ratio to ensure full hydration of the cement. The workability of the mix is vital to ensure that the slip form process can be achieved at the speed of the machine. Workability is monitored to ensure consistency of the mix.

Wire management and stressing

Wire used for prestressing can be indented wire or multiple wire strands. The wire patterns vary depending on the hollow core unit depth. The wires are cut to a marked length, transferred to the casting bed and a multi-headed hydraulic ram pulls the wires to a predetermined length to induce the correct force in the wires (Figure 1.9). The wires are then fixed in the elongated condition against an anchor plate. The simultaneous stressing of several wires reduces production time and decreases the health and safety risks from wire failures compared to an individual wire stressing system.

Figure 1.9 *Wires being stressed*

Slip forming of units

Precast units are slip formed using interchangeable cassettes containing core profiles. The concrete, which has a low water to cement ratio, is forced around the cassette profile by the head of concrete combined with a high degree of vibration. Figure 1.10 shows the slip forming process in a modern manufacturing plant. In traditional slip forming plants the casting machine moves along the stationary casting bed. However, in very modern slip forming plants the casting machine is stationary and the casting beds move, which results in more efficient production.

The slip forming process typically manufactures units at 2.5 to 3 metres per minute. (The extrusion process can only achieve about 1.5 metres per minute.)
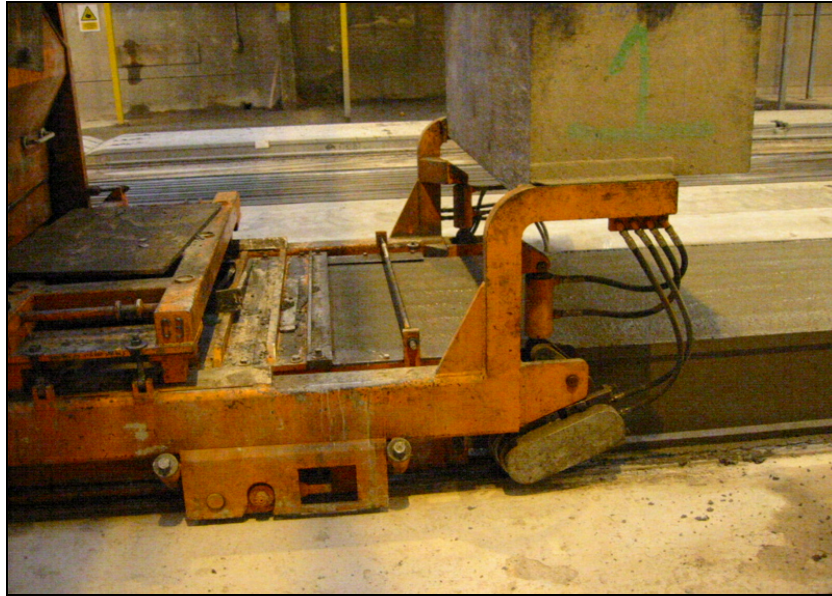
Figure 1.10 *Slip forming of units*

Plotting and formation of features

There are a variety of features that can be incorporated in precast units, as described in Section 1.5. The details (size and location) of features need to be transferred onto the wet concrete of individual units and then the excess concrete removed as necessary. In older manufacturing plants the marking out and removal of unwanted concrete is carried out manually.

In modern plants, a plotting machine travels over the unit, marking the position and size of features on the wet concrete (Figure 1.11). The design system provides this data to the plotter electronically. Other data can also be marked on to the unit at this stage e.g. contract number, item number, length and weight of the unit. The features are created by a machine that follows the plotter. Chamfered ends are created by pressing the wet concrete down, open cores and holes are created by sucking the concrete out of the marked areas (Figure 1.12).

Figure 1.11 *Plotting positions of features*

The automatic forming of features by machines can be controlled by software using transponder cards attached to the units during the plotting stage. An example of a transponder card and portable reader are shown in Figure 1.13. In addition to the dimensional properties and features of the slab, the transponder cards can also contain details of the floor design, construction drawings and specific CDM requirements.
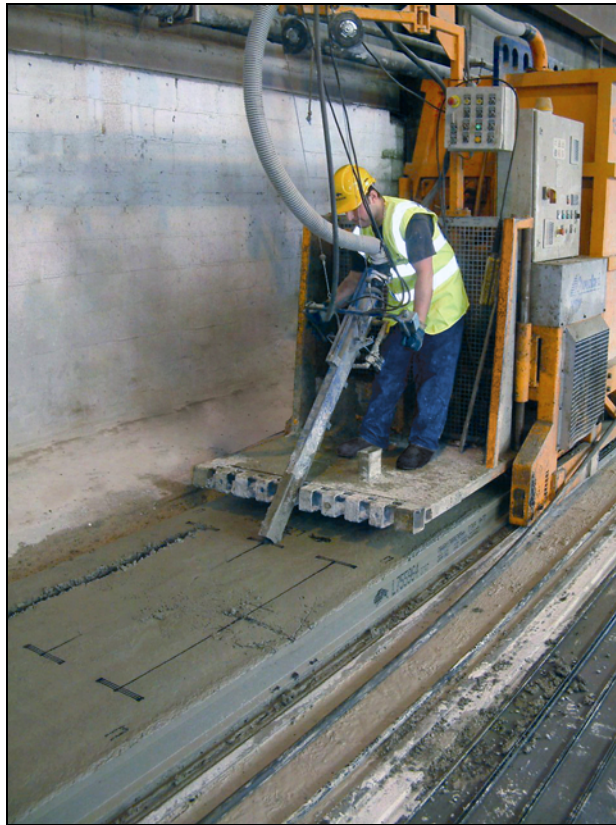
Figure 1.12 *Creating features*
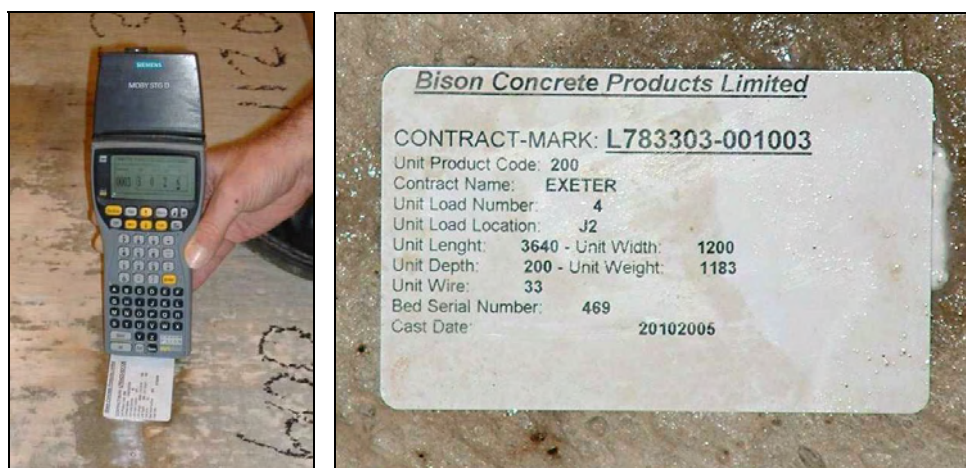
Figure 1.13 *Transponder card and portable reader*

Lifting hooks

Lifting hooks are specially shaped steel bars recessed into the precast unit that aid rapid construction. Lifting hooks are inserted into units while the concrete is still wet, using a machine which also places a specified amount of additional concrete around the hook to secure it in position. Hollow core units with integrated lifting hooks are not available from all manufacturers.

Curing, cutting, stripping and storage of units
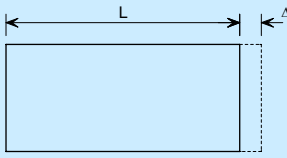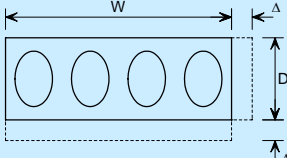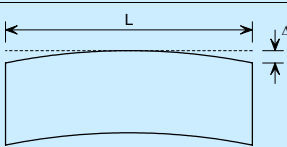
The concrete is carefully cured to achieve the required design strength, at which point the prestressing wires can be released from the anchor blocks. Methods used to shorten the curing time include heated beds and steam curing. Minimum concrete strengths of 30 N/mm^2 can be achieved within 16 hours. The cured concrete bonds to the wires which, when released from the anchor blocks, induce compression into the precast unit.

After releasing the prestressing wire, the units are cut to length at the positions marked by the plotter and transferred to the stockyard. The units are transferred to skeletal pallets for delivery to site in the order required for construction.

1.4.2 Manufacturing tolerances

Clause 6.2.8.3 of BS 8110-1: 1997[1] sets out manufacturing tolerances for precast units. However, normal manufacturing tolerances for typical precast units (i.e. over 6 m long) are generally smaller than those given in BS 8110; typical values are presented in Table 1.1.

Table 1.1   *Typical manufacturing tolerances*

Feature		Dimensions	Deviation (Δ)
Length		All lengths (without plotters): All lengths (with plotters):	± 20 mm ± 6 mm
Cross section		Depth: Width:	± 6 mm ± 6 mm
Straightness or bow		L < 3 m: 3 m < L < 6 m long: 6 m < L < 12 m long: For each subsequent 6 m:	6 mm 9 mm 12 mm 6 mm

Prestressed precast units have a natural upward camber due to the prestressing force. The magnitude of the camber will depend on the length and depth of the unit and the prestress applied. A typical camber is approximately equal to the span divided by 300. When specifying screeds, finishes and partitions etc. due allowance should be made for the upward camber together with the potential differential camber between adjacent units. Differential camber between adjacent is usually less than 6 mm for precast units less than 4.5 m long and less than 9 mm for longer units.

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1.4.3 Quality assurance

Most manufacturers of precast units are either registered to the BSI Quality Management System (QMS) and have been awarded Certificates of Assessed Capability for the design and manufacture of precast floors in accordance with BS EN ISO 9000[2], or operate a similar QMS programme. However, the credentials of manufacturers should be checked by customers.

Membership of the BSI QMS ensures that all procedures and disciplines relevant to the design and manufacturing processes are subject to the independent approval and periodic review by the British Standards Institution.

The quality management system is applied to the whole process and covers estimating to erection. For the manufacturing element of the QMS, 10% of the precast units are checked. These checks include length, depth, width, squareness, wire position and camber. In addition, all products are inspected for bond slip of stressing wires and visual correctness of the plotted features.

1.5 Factory formed features

Features such as holes, cut outs, chamfered ends, and open cores, in the precast units, can be formed in the factory. Modifications to precast units on-site should not be performed without the prior agreement of the manufacturer.

1.5.1 Holes, cut outs and openings

Holes in precast floor units are often required for services. Generally, holes larger than the hollow core unit void width should be factory formed. Units with holes are designed taking into account the holes size and the development length* for the stressing wires. Precast units with holes are checked at the end of the hole, at the end of the development length of the stressing wires and at points in between. As a guide, 300 mm square holes may be formed in the first quarter of the span. Initial parameters can be assessed at the beginning of the project for the manufacturer to give specific guidance. For practical reasons, the largest hole that can be formed in a 1200 mm wide unit is normally approximately 600 mm. It is preferable where possible to form holes by having two cut away units side by side (as shown in Figure 1.14) instead of one unit with a hole.

Note: *The development length is the length required to transmit the prestressing force in the wires to the concrete.
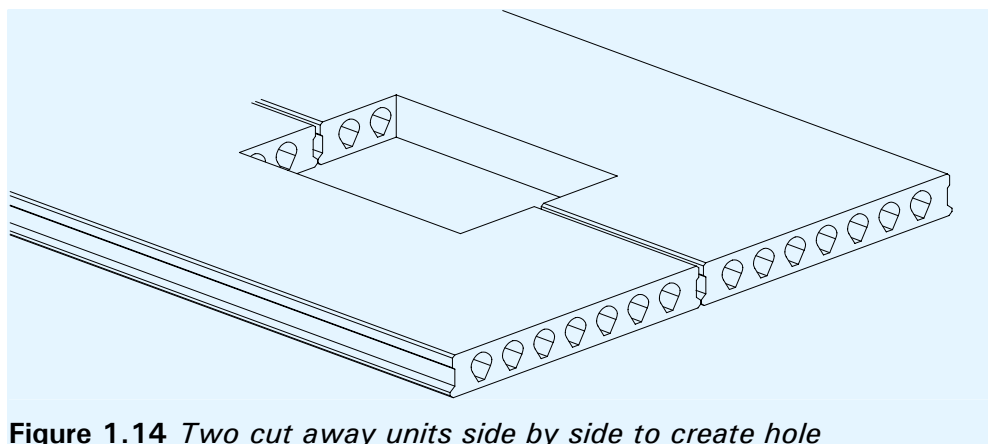
Figure 1.14 *Two cut away units side by side to create hole*

Cut outs are often required to accommodate columns and to create side pockets to enable the unit to be secured to the structural frame in Class 2B buildings (see Section 4.2.3).

Where holes or cut-outs within a unit cannot be made sufficiently large, openings can be formed by using one or more special units that are shorter than the span. The ends of these short units are then supported on a steel hanger bracket which is held up by bearing on the adjacent units, as shown in Figure 1.15.
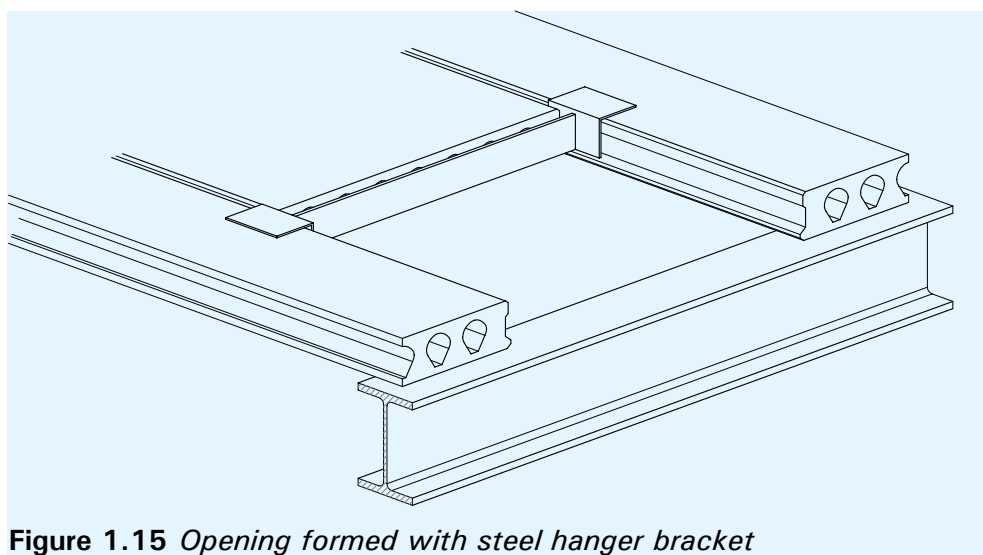
Figure 1.15 *Opening formed with steel hanger bracket*

1.5.2   Square ends

The standard finish for units is a square end that includes a nib of approximately 5 mm, as shown in Figure 1.16. The nib is usually ignored for detailing and dimensioning purposes as there is at least a 25 mm gap detailed between the ends of precast units that are supported on steel beams.
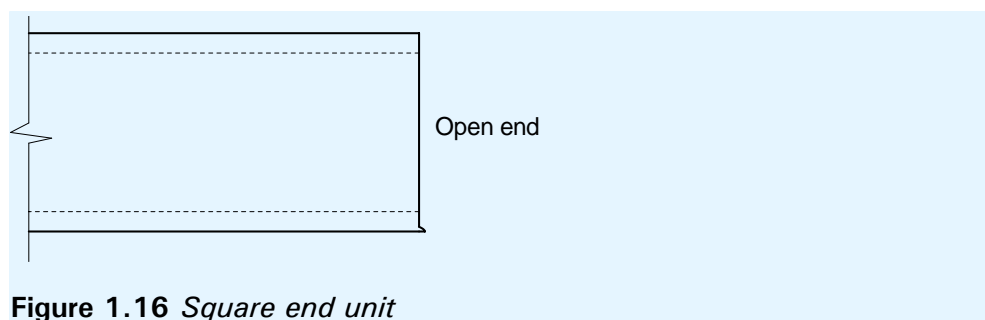
Figure 1.16 *Square end unit*

1.5.3   Solid ends

Hollow core unit manufactures are sometimes requested to supply units with solid ends i.e. approximately the first 100 mm of the core filled with concrete. There is not usually any structural justification for specifying solid ends. Hollow core units have sufficient shear capacity to support even heavy walls (e.g. masonry) built on the end of the unit.

To meet the air tightness requirements for buildings, it may be necessary to block the ends of the hollow core units at floor edges. Where this is required, manufacturers can supply end seal devices; two varieties are shown in Figure 1.17. End seal devices are usually only required for office building

applications where the use of raised floors and air permeable ceilings mean the structural floor becomes part of the air tightness system. In other applications (e.g. residential buildings) the use of airtight plasterboard ceilings and walls means that sealing the hollow core ends is not generally required.

Figure 1.17 *Hollow core end-sealing devices*

1.5.4   Notched and chamfered ends

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

The formation of the taper should be carefully controlled to ensure that there is sufficient depth of unit 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.

For composite construction, the shear connectors do not need to project above the chamfered ends, but sufficient transverse reinforcement must be placed below the level of the heads of the shear connectors (see Figure 2.2).

There are a range of notch and chamfer details that are regularly used. Reducing the unit depth by a maximum of 85 mm and chamfering over a horizontal length of 235 mm is typical for units less than 300mm deep (see Figure 1.18a).   For deep units, square notches are often used (see Figure 1.18b).   Bungs (as shown in Figure 1.18b) are not required for cores with chamfered ends because the forming process (by pressing down the wet concrete) closes the opening to the core.

a) Chamfer on unit < 300mm deep

b) Square Notch on unit > 300 mm deep
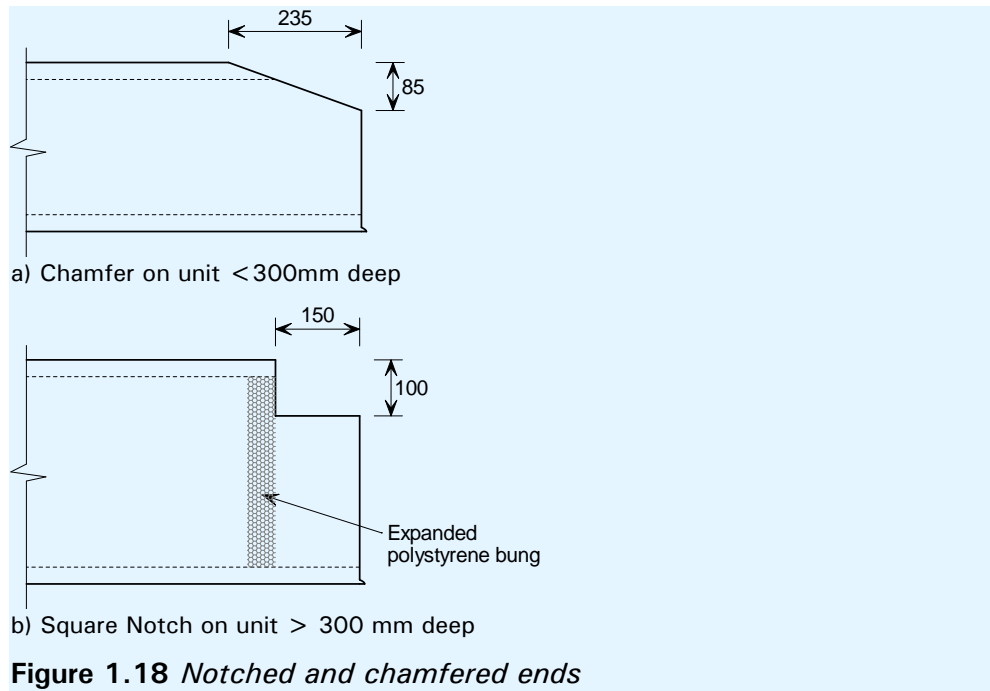
Figure 1.18 *Notched and chamfered ends*

1.5.5 Opened hollow cores

One of the advantages of hollow core units is that some cores can be opened out to receive transverse reinforcement. The tops of a specified number of hollow cores (usually two, three or four per unit end, as shown in Figure 1.19) may be opened up. Typically, this opening up operation is carried out during manufacture. Transverse reinforcement is required for composite design (see Section 2.1). Cores may also be opened so that reinforcement can be placed and concreted in to satisfy the tying requirements to avoid disproportionate collapse (see Section 4.2.3). Slots are typically 500 mm long.
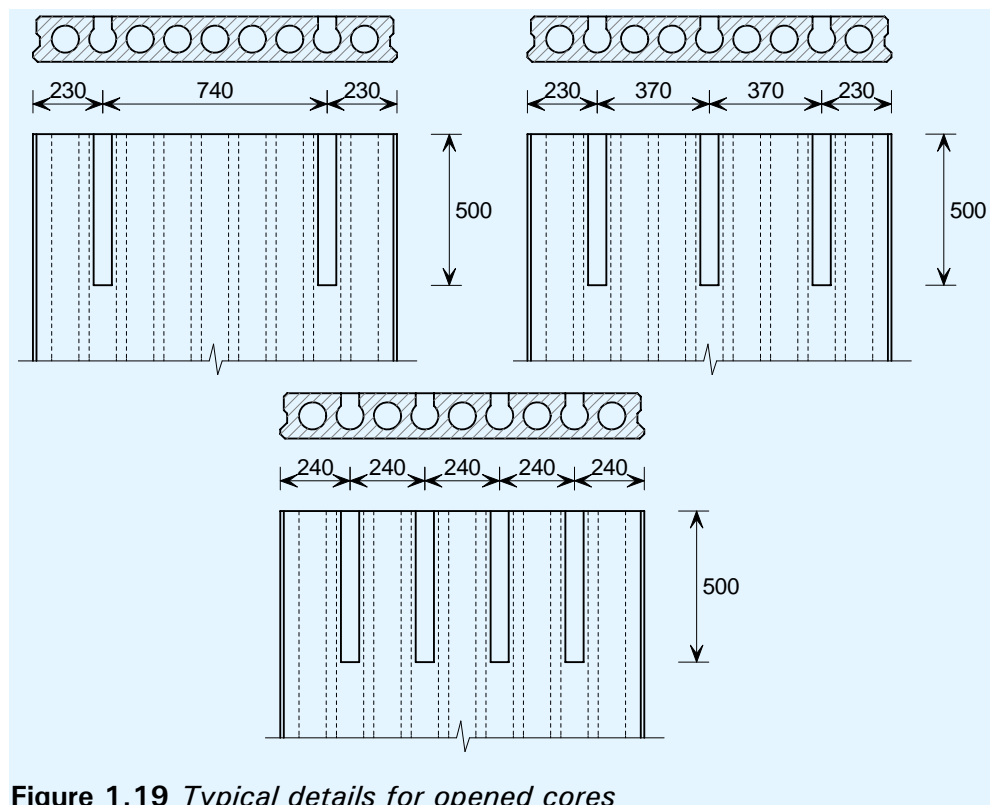
Figure 1.19 *Typical details for opened cores*

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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 could be 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 reinforcement bars.

2   STRUCTURAL DESIGN

2.1   Structural Options

This Section concentrates on hollow core floor units because this is the predominant type of precast floor unit that is used in steel framed buildings.

The range of hollow core floor units is described in Section 1.3.2 and there is a variety of steel beam options for supporting these units. Precast concrete floor units may be supported on the top flange of a steel beam, on shelf angles attached to the beam web, or when a slim floor system is adopted to minimise the structural floor depth, the units may be supported on a wide bottom flange, or a wide plate welded to the bottom flange of a standard H section beam. To aid erection shelf angles are generally required to extend 50 mm past the tip of the top flange. Typical details employing precast units are shown in Figure 2.1.

a) Downstand beam

b) Shelf angle beam

c) *Slimflor* beam (UC with welded plate)

d) ASB (Asymmetric *Slimflor* Beam)
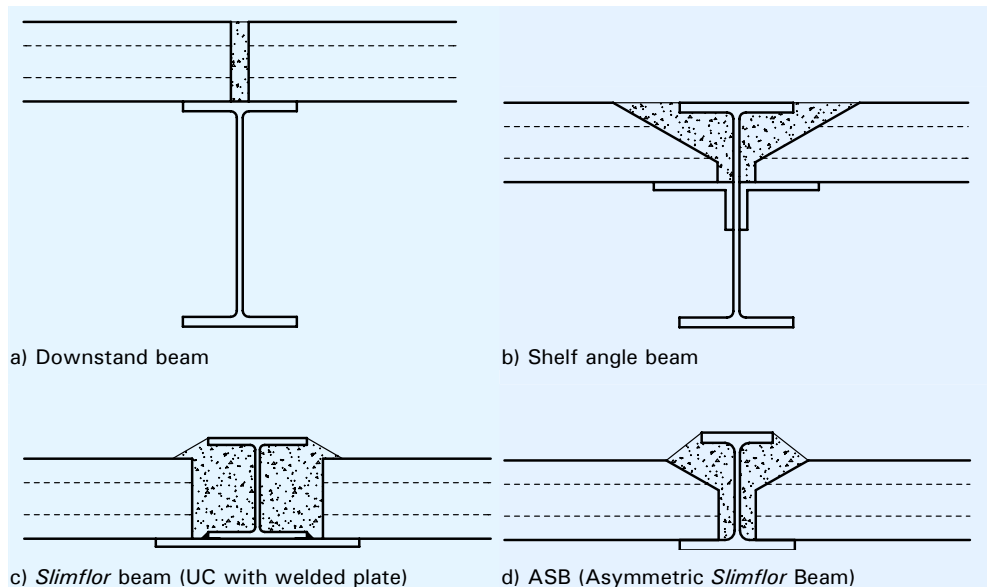
Figure 2.1   *Non-composite beam options for precast floor units*

It is possible to combine the different options shown in Figure 2.1 in the same building to maximise the advantages they offer. For example, option c) can be used for internal beams to minimise the construction depth but option a) could be used for edge beams, where the downstand beam can be incorporated into the walls.

The detailing requirements that must be followed (e.g. minimum bearing length) are described in Section 4.

Beams can be designed as either composite or non-composite. Composite action between the steel beam and the floor is achieved by fixing a line of shear connectors along the centre line of the beam and casting concrete around reinforcing bar, the shear connectors and the precast units (see Figure 2.2). For the composite slim floor beam shown in Figure 2.2, only the area of concrete above the top of the precast unit can be considered in the composite beam design.

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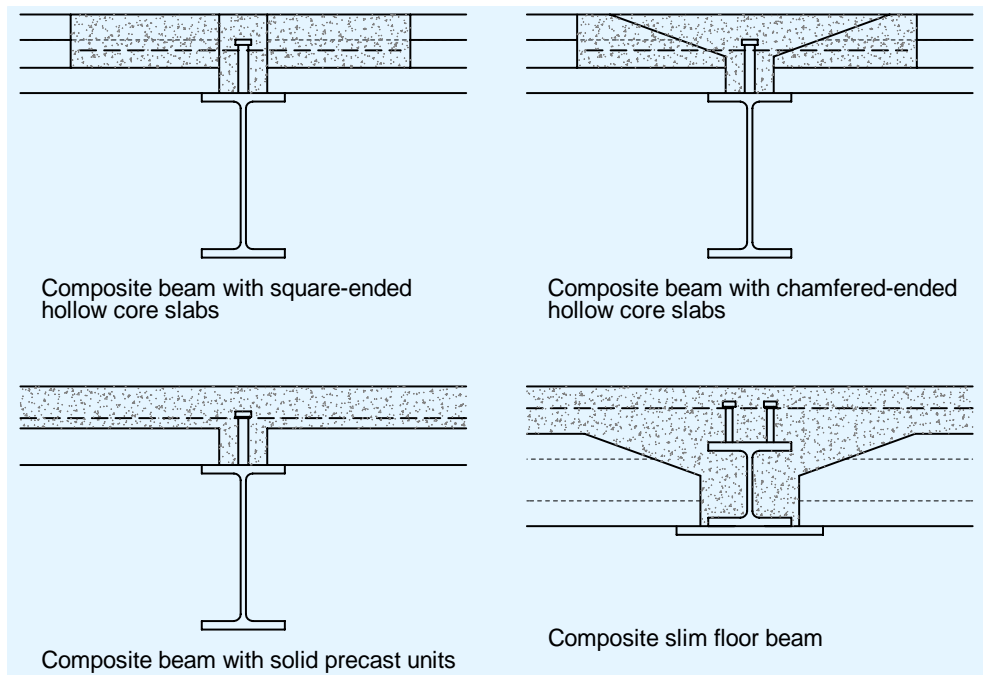
Figure 2.2 *Composite beam options with precast floor units*

Floors with precast units can be designed and constructed with or without a structural topping.   A structural topping is normally only required for: composite action with slim floor beams, enhanced diaphragm action in tall buildings, vibration control of sensitive floors, durability in car parks and certain fire safety requirements.   Where precast unit cantilever balconies are used, a reinforced structural topping is required to provide support for the precast balcony by means of the tension capacity of the reinforcement.

If a structural topping is used, the composite action between the topping and the precast floor units increases the load carrying capacity of the floor.   Typically a 20% to 60% increase is achieved with composite design.

A practical solution that is often adopted is to design the internal supporting beams as composite and the edge beams as non-composite.   This approach avoids the need to follow the detailing that is required for the edge beams to act compositely with the precast units.   Non-composite edge beams are usually deeper than a beam which is composite but the increased stiffness (and reduced deflection) that a deeper beam provides can be beneficial for glazing and claddings.

2.2   Beam arrangements

The usual arrangement of beams in a steel framed building with precast concrete floors is shown in Figure 2.3.   Precast floor units span onto supporting beams that are supported by columns.   Tie beams are connected to the columns in the direction of the span of the floor units and therefore do not in theory carry any vertical load except their own dead weight.   However, these members are usually important for frame stability during construction and as structural ties for avoidance of disproportionate collapse (see Section 4.2).   Inverted tee sections are sometimes used as tie beams, with the stem of the tee placed between two precast units.   However, this arrangement breaks the floor slab into sections, which can adversely affect the sensitivity of the floor to vibration.

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Also, the tee section provides minimal flexural stiffness during construction. Circular hollow sections within the depth of the floor slab are not recommended as tie beams because it is difficult to place grout around the CHS and again the floor slab is broken into sections. A preferred solution is to place the tie member below the precast units to avoid breaking the floor slab into sections. Tie members may be hollow sections or open sections.

Common structural grids used for steel framed buildings with precast floors are discussed in Sections 2.2.1 to 2.2.3.
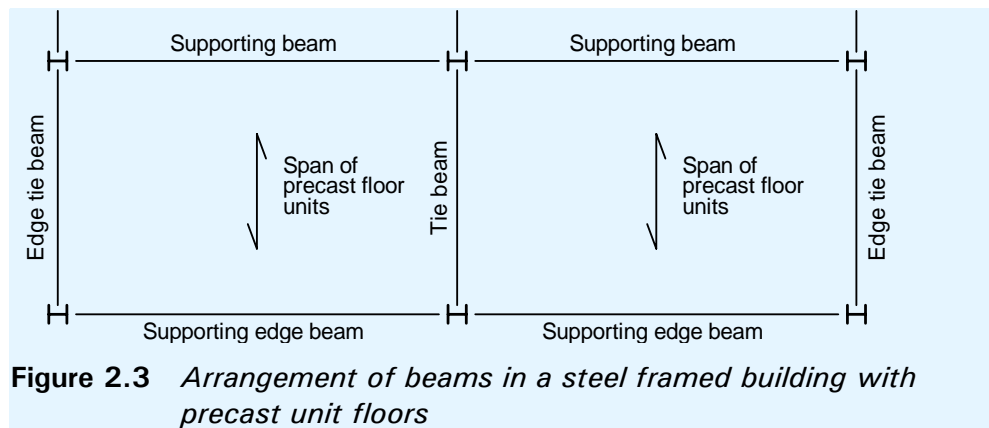
Figure 2.3 *Arrangement of beams in a steel framed building with precast unit floors*

Precast infill pieces

Wherever possible, floor units are arranged so that full width units (nominally 1200 mm) can be utilised. Where this is not possible there are two options a) to use part width units or b) to use precast infill pieces or PIPs (see Figure 2.4). Precast infill pieces are inserted between floor units and are secured into position as the joints are grouted. The infill pieces are generally between 75 mm and 150 mm wide and available in 25 mm increments.
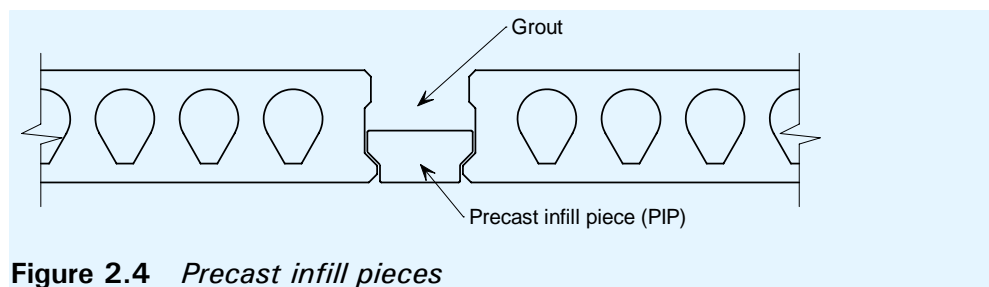
Figure 2.4 *Precast infill pieces*

2.2.1   Commercial floors

A typical floor plan of a commercial building is shown in Figure 2.5a). The bay is 8.5 m × 6 m with 150 mm deep hollow core precast units spanning 6 m. The supporting beams could be composite beams with welded shear studs and the precast units supported on the top flange (see Figure 2.2a and b) or non-composite shelf angle beams (see Figure 2.1b). The overall structural depth of these two options will be about 550 mm.

Another typical floor plan of a commercial building is shown in Figure 2.5b. The bay is 6 m × 7.5 m with 200 mm deep hollow core precast units spanning 7.5 m. For this floor plan the supporting beams could be non-composite UC sections with a plate welded to the bottom flange to support the precast floor units (see Figure 2.1c). The overall structural depth will be approximately 280 mm.

A further example of a floor plan for a commercial building is shown in Figure 2.6. The bay is 6 m × 13.5 m with 350 mm deep hollow core precast units spanning 13.5 m. For this floor plan the supporting beams could be composite or non-composite.

a) 6 m span

b) 7.5 m span
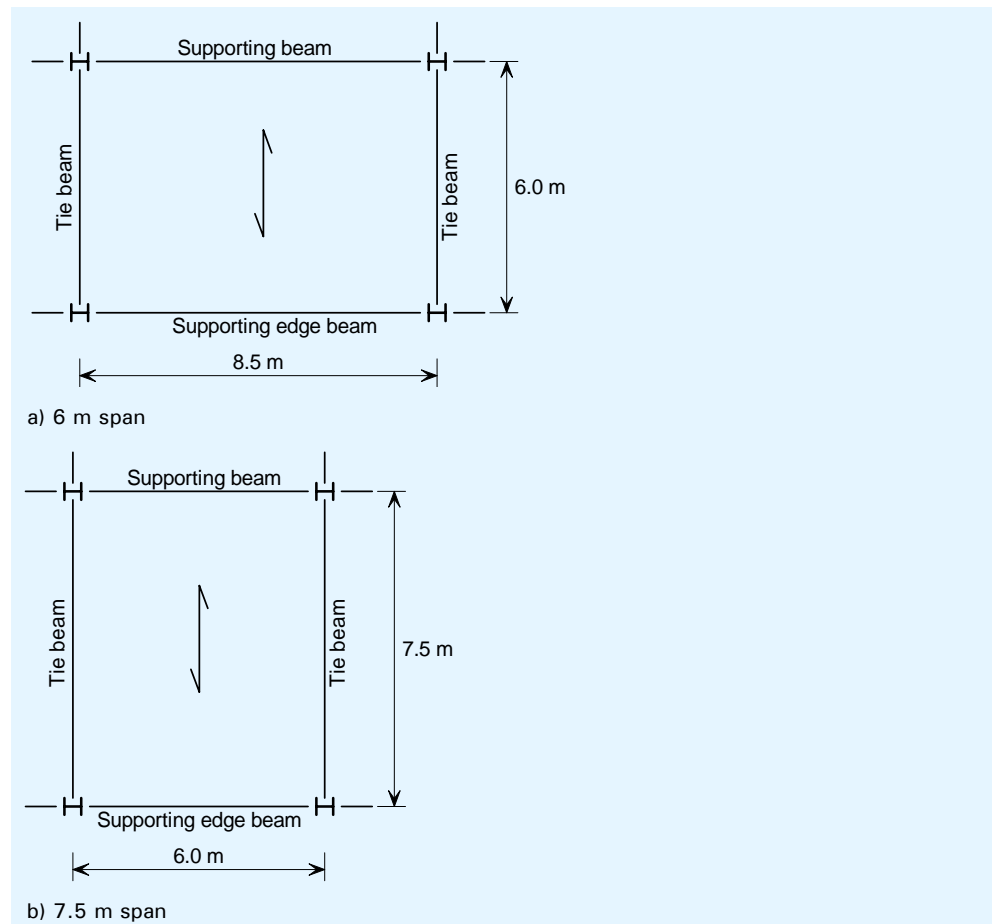
Figure 2.5 *Typical floor plans for a commercial building*

Figure 2.6 *Long span floor plan for a commercial building*
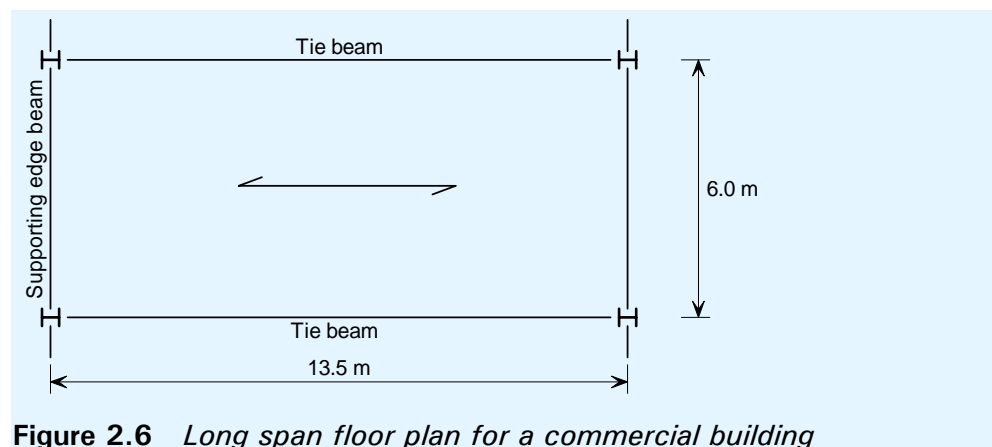
2.2.2　Residential floors

A typical floor plan for a residential building is shown in Figure 2.7. The bay is 7.5 m × 7.5 m with 200 mm deep hollow core precast units spanning 7.5 m. The supporting beams could be non-composite downstand beams with the precast units supported on the top flange (see Figure 2.1a). In which case, the downstand beam positions can be designed to align with wall positions so the floor to ceiling height is not restricted by the beam depth.
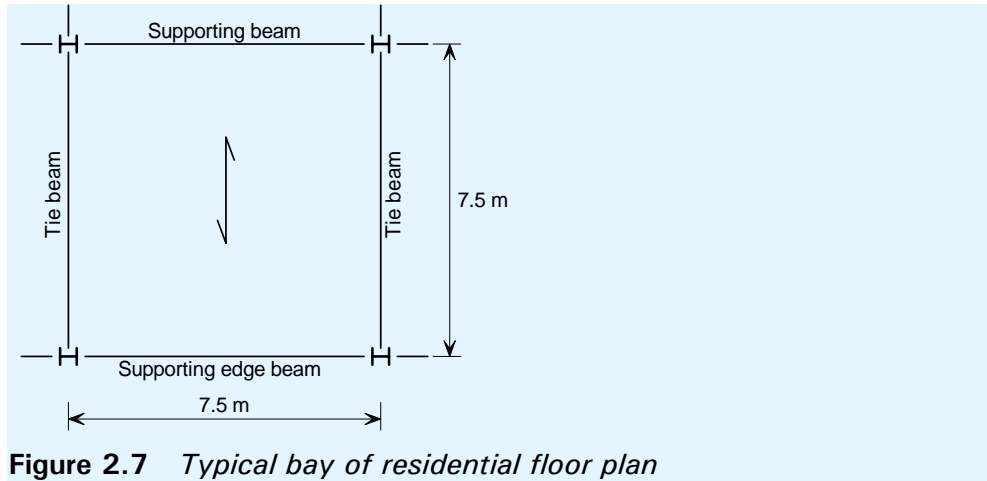
Figure 2.7　*Typical bay of residential floor plan*

Alternatively, ASB or *Slimflor* beams could be used with the precast units supported on the bottom flange of an asymmetric beam (see Figure 2.1c and d). In such cases, the construction depth of the floor is kept to about 300 mm.

2.2.3　Car parks floors

A typical bay of a car park floor plan is shown in Figure 2.8. The bay is 15.9 m × 7.2 m with a 150 mm deep hollow core precast unit plus an 80 mm structural topping, spanning 7.2 m. The span of 7.2 m is the width of three parking bays and the width of 15.9 m is the length of two parking bays with an aisle between them.
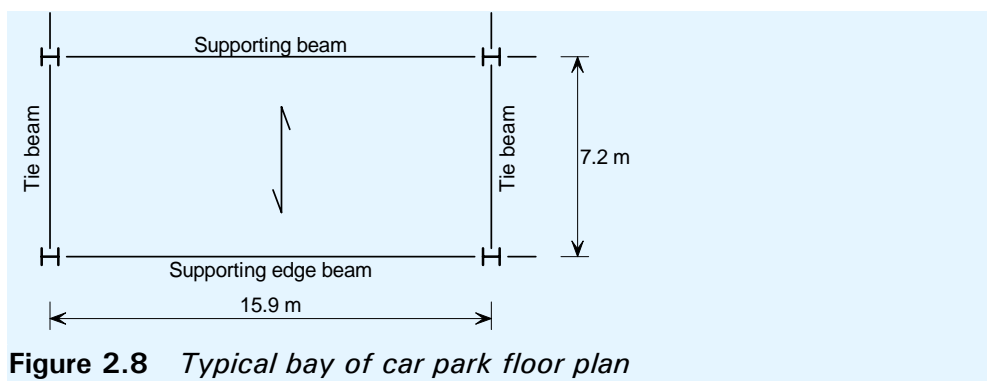
Figure 2.8　*Typical bay of car park floor plan*

The supporting beam for the floor plan shown in Figure 2.8 would typically be a composite downstand beam. The overall construction depth including the 150 mm precast unit and 80 mm structural topping would be approximately 850 mm.

Alternatively, the precast units could span the long direction and the column spacing could be increased to 9.6 m (the width of four parking bays) as shown in Figure 2.9. The precast units would be 400 mm deep with a structural topping.
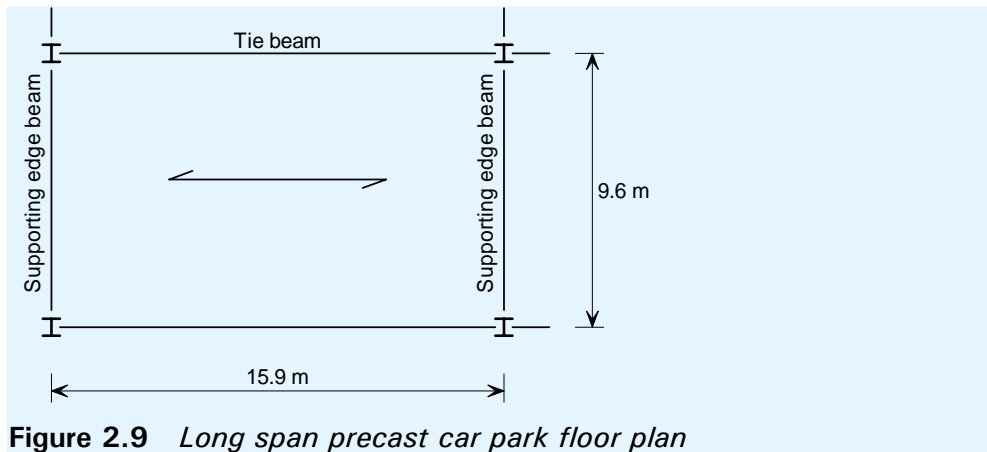
Figure 2.9   *Long span precast car park floor plan*

2.3   Beam design

Design guidance of steel beams acting compositely with precast concrete units is provided in SCI publication P287[3]. SCI publication P342[4] provides design guidance for Asymmetric *Slimflor* Beams supporting precast floor.

2.3.1   Supporting beams

Construction condition

The beams supporting precast floor units need to be checked at the construction stage, without assuming any composite action between the steel beams and the precast units or between the precast units and the structural topping (if specified). Guidance regarding the temporary stability of beams during construction is provided in Section 5.3.2. The following situations should be considered:

1. Unbalanced ULS loads acting on the beam due to the sequence of installation of the precast units. Verify the strength under the self-weight of the floor units. The beam is subjected to combined bending and torsion. Beams with unbalanced loading during the construction stage are not restrained by the precast units and must be designed as laterally unrestrained.

2. Balanced ULS loads when all the precast units are installed (assuming they are of equal span on either side of the beam). Verify the strength under the construction load, together with the self-weight of the units and the weight of the concrete topping (if used). Beams with balanced loads during the construction stage should not be assumed to be laterally restrained. Guidance given in Section 5.3 should be used to determine the level of lateral restraint provided and the requirements for lateral restraint and stability during construction.

3. Unbalanced SLS loads. Verify the acceptability of the angle of twist under the self weight of the units. Two degrees of twist is normally acceptable.

4. Balanced SLS loads. Verify the acceptability of the vertical deflection under the construction load, together with the self-weight of the units and the weight of the concrete topping (if used).

Note: Situations 2 and 4 will not be critical if the beams are non-composite in the completed condition.

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In the normal condition composite action may be assumed, if the appropriate shear connection has been provided. The following situations should be considered:

1. Balanced ULS loads. Verify the acceptability of the composite or non-composite beam design, whichever is appropriate.

2. Unbalanced ULS loads. Verify the acceptability beam subject to bending and torsion due to pattern loading of imposed load. Other unbalanced loading conditions to consider are unequal spans and edge beams.

3. Robustness. The tying capacities of the steelwork connections and anchorage of the precast units should satisfy the requirements described in Section 4.2.

4. Fire resistance. Verify that the beams and floor have the required period of fire resistance using the guidance given in Section 2.8.

5. Verify acceptability of vertical deflections and dynamic response at the serviceability limit state.

2.3.2 Beam to column connections

1. Design connections for shear only due to balanced loading.

2. Design connections for shear and torsion due to unbalanced loading.

See Section 2.7 for guidance on connection design and detailing.

2.3.3 Tie beams

Tie beams should be designed to ensure that the structural integrity requirements of BS 5950-1 are satisfied (see Section 4.2). In theory, tie beams do not carry any vertical load except their own dead weight. However, for sizing, tie beams can be assumed to carry a 1 m strip of floor load.

2.4   Precast unit design

The design of precast units is a traditional prestressed concrete analysis with some well established considerations appropriate to its geometrical profile and manufacturing method. The design is carried out by the precast manufacturer in consultation with the frame designer. The frame designer will be required to specify the design floor loading.

The only reinforcement in hollow core units is the longitudinal prestressing tendons located in the lower half. The tendons are anchored by their bond with the concrete. Consequently, whenever possible, tensile stresses in unreinforced zones (i.e. the top half of the unit) are normally avoided by designing the floors to be simply supported.

The bending resistance of hollow core units is provided in the same way as for any prestressed member. The prestressing force induced by the longitudinal wires precompresses the concrete in the regions where tensile stresses would develop. Therefore, when the precast unit is loaded the bending stresses reduce the built-in compression in those regions (see Figure 2.10). When the load is removed the unit will return to its original state of stress.

a) Stress due to gravity loads   b) Stress due to prestress   c) Final stress condition
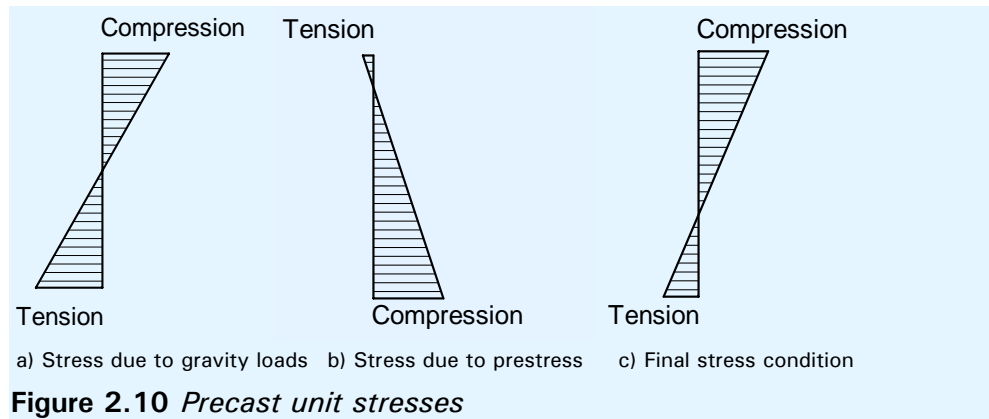
Figure 2.10 *Precast unit stresses*

In addition to the bending resistance of the precast unit, there are important design checks that the precast manufacturer will perform regarding the unit in the vicinity of the supports. These are:

- Shear resistance check (as for conventional reinforced concrete)

- Shear tension failure (which occurs when the principal tensile stress in the web reaches the tensile strength of the concrete)

- Sufficient anchorage of the prestressing steel.

The length from the support over which the full prestressing force is developed, known as the 'transmission length', is particularly important for the shear checks.

The effects of non-rigid supports and the requirement for floor diaphragm action need to be considered in the design of precast concrete floors. These design issues are covered in Sections 2.5 and 2.6 respectively.

Structural topping

The use of a structural topping on hollow core and solid prestressed precast units enhances the structural capacity of the floor by composite action between the units and the topping. The design of the topping must be adequate for:

- Durability of the concrete

- Compressive stress in the topping

- Horizontal shear at the interface between the floor units and the topping

- Cracking of the topping due to shrinkage and thermal movement during and after construction.

The minimum thickness of a structural topping should be 50 mm at the mid-span crown, increasing towards the supports due to the prestressing precamber.

Guidance regarding the durability of concrete for use as structural toppings is provided in BS 8500[16]. Depending on the application, the concrete mix required may vary significantly. Further information on durability is provided in Section 2.10.

The compression in the structural topping is limited by the strength of the topping and is accounted for by the precast manufacturer in the design of the floor.

The shear interface design is a part of the structural design of the floor carried out by the precast manufacturer. The allowable horizontal shear stress is obtained from Table 5.5 of BS 8110-1. From tests[5] is can be seen that precast floors with a structural topping will generally fail in bending before there is any bond failure at the shear interface between the unit and the structural topping. However, the surface finish to the precast units should be such that the concrete topping will adhere to the precast surface. The procedures for placing a structural topping are:

(i)   Grout all joints between precast units (The joints should have been grouted for a minimum one day before the topping is placed)

(ii) Remove of all debris, oil, mortar and any contaminants

(iii) Thoroughly wet the surface of the units (3 litres per metre square)

(iv) Place reinforcement onto the precast surface using the appropriate spacers and a layout which should avoid excessive lapping

(v)   Place the structural concrete topping in an even manner on the surface of the units and compact thoroughly using vibrating pokers.

Cracking of the structural topping due to temperature effects is controlled by providing movement joints in the floor. It is normal to provide movement joints at a maximum spacing of 64 m for frames subject to direct thermal effects (e.g. car parks) and a maximum of 120 m for frames protected from direct thermal effects (e.g. schools and offices). Direct thermal affects are considered in Appendix A of Reference 6. Normal average temperature ranges for exposed car parks are 45°C.

Shrinkage of the topping due to drying requires that the topping is restrained by reinforcement and that shrinkage (crack control) joints are provided at appropriate centres and places. Appropriate centres for crack control joints are a function of the reinforcement, the depth of the topping and the curing conditions. Typical layouts are:

- 12 m in each direction for toppings with A142 mesh reinforcement

- 18 m in each direction for toppings with A193 mesh reinforcement.

Crack control joints are generally 6 mm wide by 25 mm deep and can be formed with a disc cutter. After the joints are formed they should be filled with flexible sealant. Crack control joints should be positioned over the supports but off-set from the line of any shear studs or along the line of the joint between units in the orthogonal direction.

2.5   Non-rigid supports

Simply supported hollow core units are generally analysed based on a two dimensional stress distribution. This is theoretically valid if the unit is supported on rigid supports that are perpendicular to direction of the span.

However, when units are supported on beams that are not rigid and which deflect under imposed load, the units become supported at their corners and not across the whole width (as shown in Figure 2.11b). In units supported at their corners, additional shear stresses parallel to the longitudinal axis of the supporting beam are applied across the ends of the hollow core units. These

additional stresses are directly related to the vertical shear force in the unit due to the imposed load. 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. The precast manufacturer should make allowance for the effects of non-rigid supports in the design of the precast units.
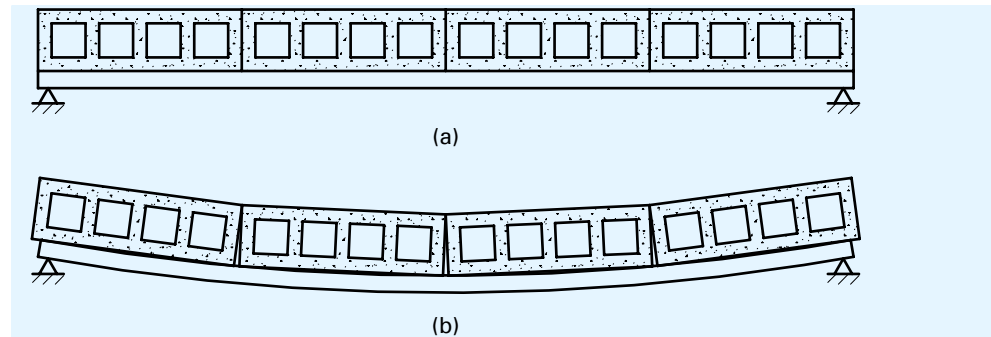
Figure 2.11 *Hollow core units on a) rigid support and b) non-rigid support*

When a structural topping is cast after the longitudinal joints between the units have been grouted and the grout has hardened (as is recommended), the topping self weight is included as part of the imposed load for the purposes of allowing for non-rigid supports. The manufacturer will check the design of the units and advise if the shear resistance of the unit should be increased.

In most practical applications, where the 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 non-rigid supports. However, when propped construction is used, the removal of the props can significantly increase the applied shear stresses within the hollow core units.

If the shear resistance of the precast unit is insufficient when the effects of non-rigid supports are included, the shear resistance of the unit can be increased or the stiffness of the support can be increased. The shear resistance of hollow core units 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. 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 the objective of increasing the shear resistance.

For unpropped non-composite beams, the influence of support stiffness need not be considered[7] 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 (non-composite or composite) 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.

It has been proposed in some guidance, from other sources, that the span to depth ratio of the units should be limited to 35. Most precast manufacturers can provide detailed analysis of the effects of non-rigid supports which means that this span to depth limitation is not relevant.

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.

In some cases, the effect of fire on the rigidity of supports and shear capacity of units needs to be considered. Section 2.8 provides guidance relating to fire resistance.

2.6   Floor diaphragm action

The floor is often required to provide diaphragm action in order to transfer wind loads to braced walls or concrete core walls (Figure 2.12). In a steel frame building with precast unit floors, the diaphragm action can be achieved through a combination of the following measures:

- Utilisation of the shear resistance of the grouted joints between the precast units.

- Provision of a continuous in-situ reinforced topping to enhance the diaphragm action provided by the grouted joints (a topping is recommended for larger floors or taller buildings).

- Ties between the perimeter members and the floor units (see Section 4.2.3 for connection details).

- Ties between the floor units and the shear walls or reinforced cores.

- Encasement of columns into the floor.

Note: * Link bars between the ends of the units may be required to transmit tensile diaphragm forces resulting from negative wind pressure on the front of the building or wind on the sides of the building.
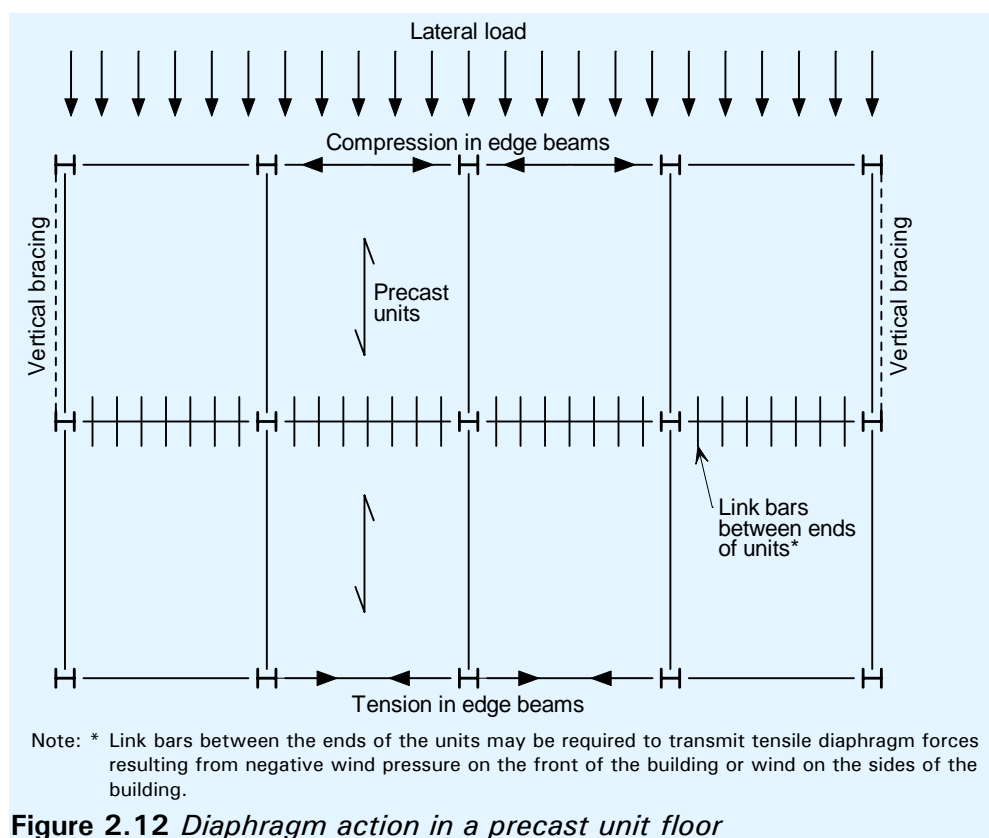
Figure 2.12 *Diaphragm action in a precast unit floor*

The measures required to achieve structural robustness for avoidance of disproportionate collapse (see Section 4.2) may also serve to ensure diaphragm action of the floor.

To ensure that the whole floor acts together, the longitudinal joints between the units must be grouted and allowed to cure before an in-situ concrete topping is poured. When a structural topping is provided with precast floors acting compositely with steel beams using shear studs, floor diaphragm action is generally adequate for buildings with regular rectangular floors of normal proportions without exceptionally large openings.

2.6.1   Shear resistance of grouted joints

BS 8110-1 gives a shear strength for joints of 0.23 N/mm^2 but this is not applicable to the smooth finish that is provided on precast units. EN 1992-1-1:2004[8] states that the shear strength of the grouted joint between two adjacent precast units may be taken as 0.10 N/mm^2 at ultimate limit state. The effective area of the joint may be taken as the length of joint multiplied by the depth of unit minus 35 mm (as shown in Figure 2.13). The strength of the precast unit is usually at least twice the strength of the grout in the joints between the units. Therefore, it is customary to calculate the resistance (shear and bearing) of the floor diaphragm using the grout strength and the effective depth of the joint.

If the shear resistance of the joints between adjacent units is not sufficient to resist the forces applied to the floor diaphragm, a more rigorous approach can be applied. Additional shear transfer mechanisms that may be utilised include: aggregate interlock, dowel action provided by reinforcement (see Figure 2.14) and ties between the floor and the supporting steel structure (see Section 4.2.3). For cases when the more rigorous approach is required to achieve the necessary diaphragm action, advice from the manufacturer of the precast units should be sought.
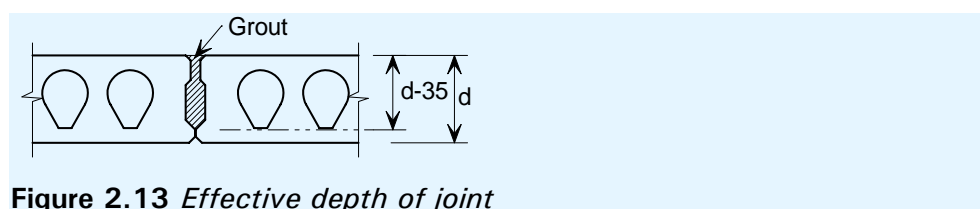
Figure 2.13 *Effective depth of joint*

For the value of 0.10 N/mm^2 for the shear strength of the grouted joint to be valid, the crack width (see Figure 2.14) must be limited; this can be achieved by restraining the perimeter of the floor. In steel framed buildings the floor is restrained by the perimeter beams and ties ensuring the floor is adequately connected to the steel frame. For Class 2A buildings, sufficient restraint to the floor is usually provided by ensuring that all joints between the precast units are grouted and the gaps between the columns and the precast units are grouted. For Class 2B buildings, restraint to the floor can be provided, in addition to the grouting as for Class 2A buildings by tying the precast units to the perimeter supporting beams and also to the perimeter tie beams. Appropriate details for tying floor units to perimeter members are shown in Section 4.2.3. Definitions of Class 2A and Class 2B buildings are provided in Table 4.1.
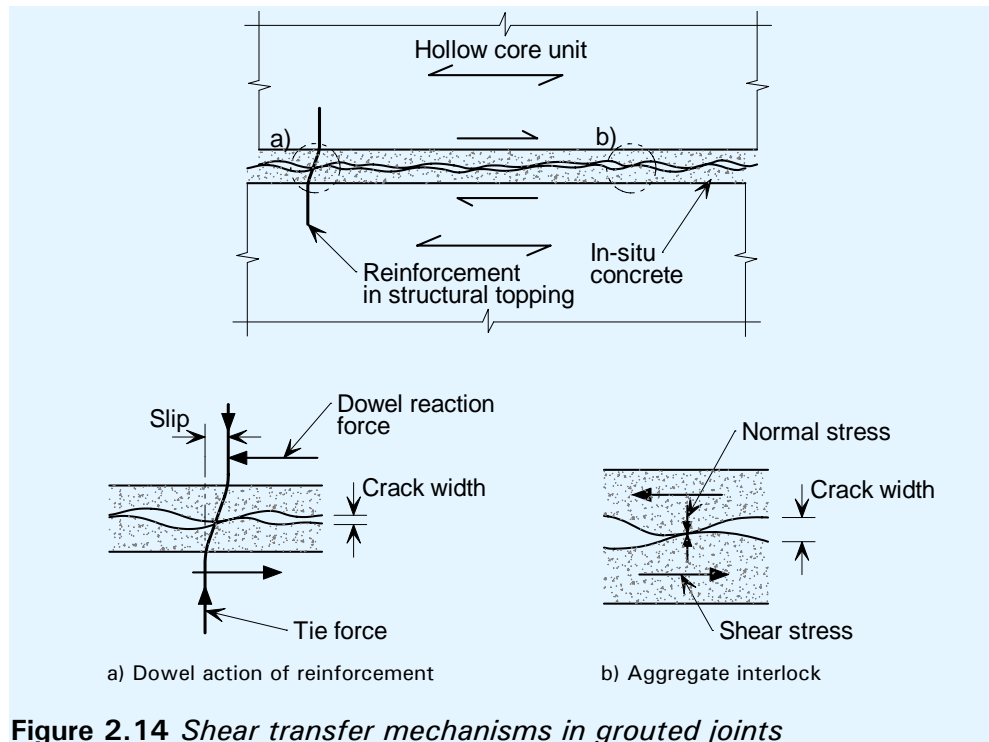
Figure 2.14 *Shear transfer mechanisms in grouted joints*

Precast floor units are selected based on the vertical loading they are required to support. The floor is also checked against the diaphragm effects of shear and bearing due to horizontal loads (as described above). It is customary not to check the precast units for the interaction of the vertical and horizontal modes in normal buildings. However, the designer should satisfy themselves that this is appropriate in all cases.

2.6.2 Perimeter ties

Lateral loads acting on the building induce axial loads into the edge beams (as shown in Figure 2.14). Therefore, steel beams around the perimeter of the building should be tied into the floor plate for diaphragm action. I-beams may be considered to act as peripheral ties, provided that they are connected mechanically to the slab through shear connectors. Beams within the depth of the floor slab (e.g. *Slimflor* beams or RHS edge beams) may also be considered to act as peripheral ties.

For situations where the floor is not tied to the supporting structure with welded shear studs or the steel beams are not within the depth of the floor then there needs to be a mechanism of transmitting the loads between the floor and the steel frame. One potential method for transmitting nominal forces in Class 2A buildings (see Table 4.1) could be by bearing of the floor diaphragm against the steel columns. This mechanism requires all the joints between the units to be properly grouted and good quality in-situ concrete to be placed between the precast units and the columns.

2.7   Steelwork connection design

The guidance in this publication is limited to design of the floor beams as simply supported beams. It would be possible to utilise moment resisting connections to enhance beam and frame stiffness, decrease sagging moments in the beams and reduce or eliminate the need for bracing. However, no test

information or design guidance presently exists on the performance of the shear connection of composite beams with hollow core units in negative moment regions. Such connection and frame design is therefore excluded.

Detailed guidance on connection design and standardised connection detailing is provided in publications P212 *Joints in Steel Construction: Simple Connections*[9], P207 *Joints in Steel Construction: Moment Connections*[10] and *Corus Slimdek Manual*[11] (for ASB connections).

The guidance in this Section relates to the particular considerations for connection design when using precast floors.

The steelwork connections in a steel frame building with precast floor units will generally be similar to those used in steel frame with a conventional metal deck and in-situ concrete floor. The main difference is that the connections will often be subjected to considerable torsional forces due to the unbalanced loading of the beams caused by the construction sequence of the precast floor units. In the majority of cases the frames will be braced and the beams will be designed as simply supported. Hence, the connections will be designed for shear (and torsion) only.

The connection design should consider the following issues:

- The width of beam and column flanges, especially where beams connect to the column on adjacent faces.

- Combined effects of vertical shear and applied torsion (particularly for edge beams).

Beam-to-column connections should be designed as non composite connections using full depth end plate connections. Full depth end plates are used because they can be designed to resist torsion (due to the out-of-balance forces on the beam) and they reduce deflections at the construction stage, as a result of the effective stiffness of the joint. Fin plate connections are not appropriate where there is significant applied torsion. Where end plates protrude above the top flange of the beam, the precast units should be detailed to ensure that there is sufficient clearance to avoid the end plate and weld.

The details of beam-to-column connections for downstand beams, asymmetric slim floor beams and shelf angle beams are shown Figure 2.15, Figure 2.16 and Figure 2.17 respectively.
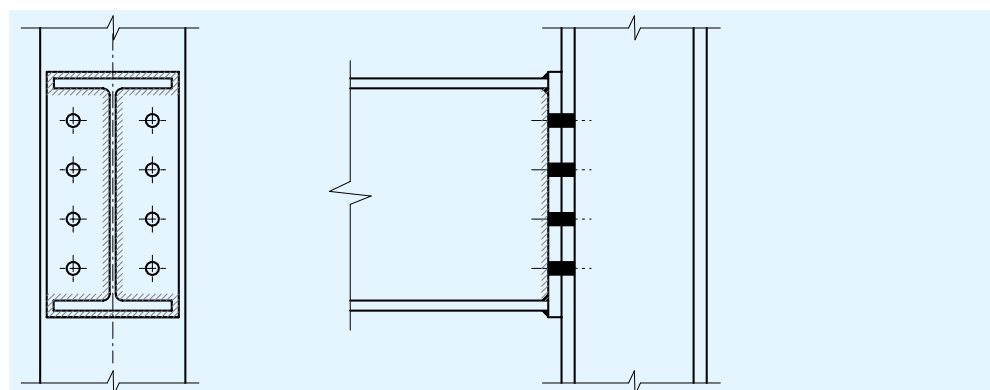
Figure 2.15 *Connection detail for downstand beam*
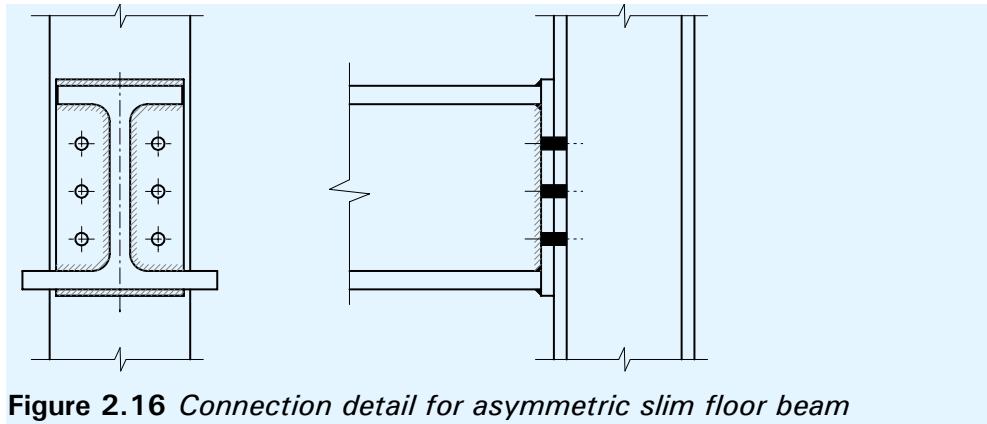
Figure 2.16 *Connection detail for asymmetric slim floor beam*
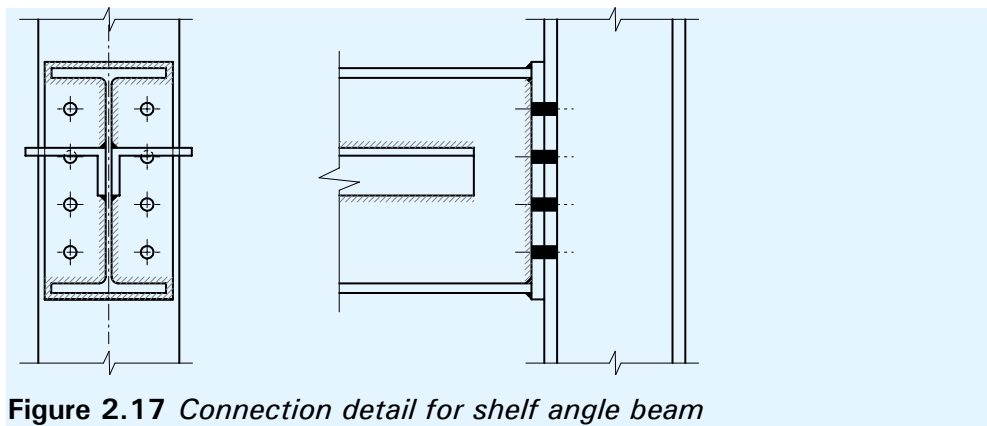
Figure 2.17 *Connection detail for shelf angle beam*

When 'wide' members, such as ASBs, are connected to the minor axis of a column, a common detail is to weld a plate across the toes of the column flanges, as shown in Figure 2.18. In this situation the welds between column flanges and the plate need to be designed for the combined effects of vertical shear and any torsion. Similar connection details are often used for RHS *Slimflor* edge beams, where the RHS is often offset from the centreline of the column to suit the detailing at the perimeter.

Plate welded
across column toes
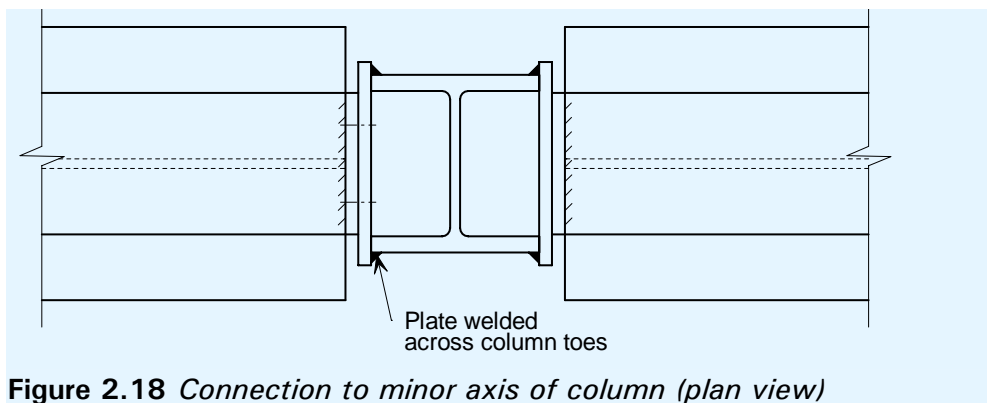
Figure 2.18 *Connection to minor axis of column (plan view)*

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2.8　Fire safety

Guidance regarding fire resistance of precast concrete floors and the supporting steel beams is provided in SCI publication P287[3] for floors with downstand beams and in SCI publication P342[4] for floors with precast units supported on the bottom flange of Asymmetric *Slimflor* Beams.　The following information presented in this section is a summary of the guidance from P287 and P342.

2.8.1　Fire resistance requirements

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:

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

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

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

All three requirements apply to separating elements (i.e. floor slab), whereas, only requirement (c) applies to supporting beams.　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.

2.8.2　Fire resistance of supporting 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'.　Members with low section factors will heat up more slowly than members with high section factors.　Different types of beams will have different section factors because they have different amounts of exposed surface.

Downstand beams and shelf angle beams

There is a choice of fire protection materials that can be applied to the steel section.

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.

The thickness of fire protection required depends on the section factor and period of fire resistance required.

SCI publication P126[12] provides design guidance and detailing rules for shelf angle beams supporting precast units that can provide 30 and 60 minutes fire resistance without applied fire protection.

ASB sections

All ASB sections used with precast floor units can achieve at least 30 minutes fire resistance. The sections designated ASB(FE) can be designed to achieve 60 minutes fire resistance without additional fire protection. For higher periods of fire resistance, applied fire protection is required. Guidance on this is given in SCI P342.

2.8.3   Fire resistance of hollow core units

Hollow core floor units generally have fire resistances of either 1 hour or 2 hours depending on the depth and mass of the unit. These fire resistances can be increased by applying additional finishes. Hollow core floors have been subject to various tests[5,13] which demonstrate the integrity of hollow core floors during fire conditions. Further guidance regarding fire design of precast units is available from manufacturers.

Hollow core units supported on fire protected downstand beams and shelf angle beams

For protected sections with a limiting temperature of $\leq 620^{\circ}$C, a summary of the fire protection requirements for different periods of fire resistance are given below. The requirements apply to both composite and non-composite beams.

For 30 minutes, there are no special requirements. However, effective tying should be provided by a three dimensional tied steel structure.

For 60 minutes, there are three options:

(a) Limit the slab thickness to no more than 265 mm and provide effective tying by a three dimensional tied steel structure.

(b) Provide longitudinal tie reinforcement embedded in joints between precast units.

(c) Provide a structural topping with mesh reinforcement.

For 90 minutes, provide longitudinal tie reinforcement embedded in joints between precast units. Note: The provision of a structural topping with mesh reinforcement is optional.

For 120 minutes, provide longitudinal tie reinforcement (suspension reinforcement) embedded in filled cores of the precast units and provide a structural topping with mesh reinforcement.

For downstand composite beams, the transverse reinforcement used to develop composite action is normally sufficient to provide satisfactory performance for 60 minutes fire resistance. The 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[13]. The effect of non-rigid supports need not be considered provided beams are protected. Typical details for periods of fire resistance are shown in Figure 2.19.

(a) 60 minutes fire resistance

(b) 90 and 120 minutes fire resistance

Figure 2.19 *Detailing measures for fire resistance of hollow core units with downstand steel or composite beams*
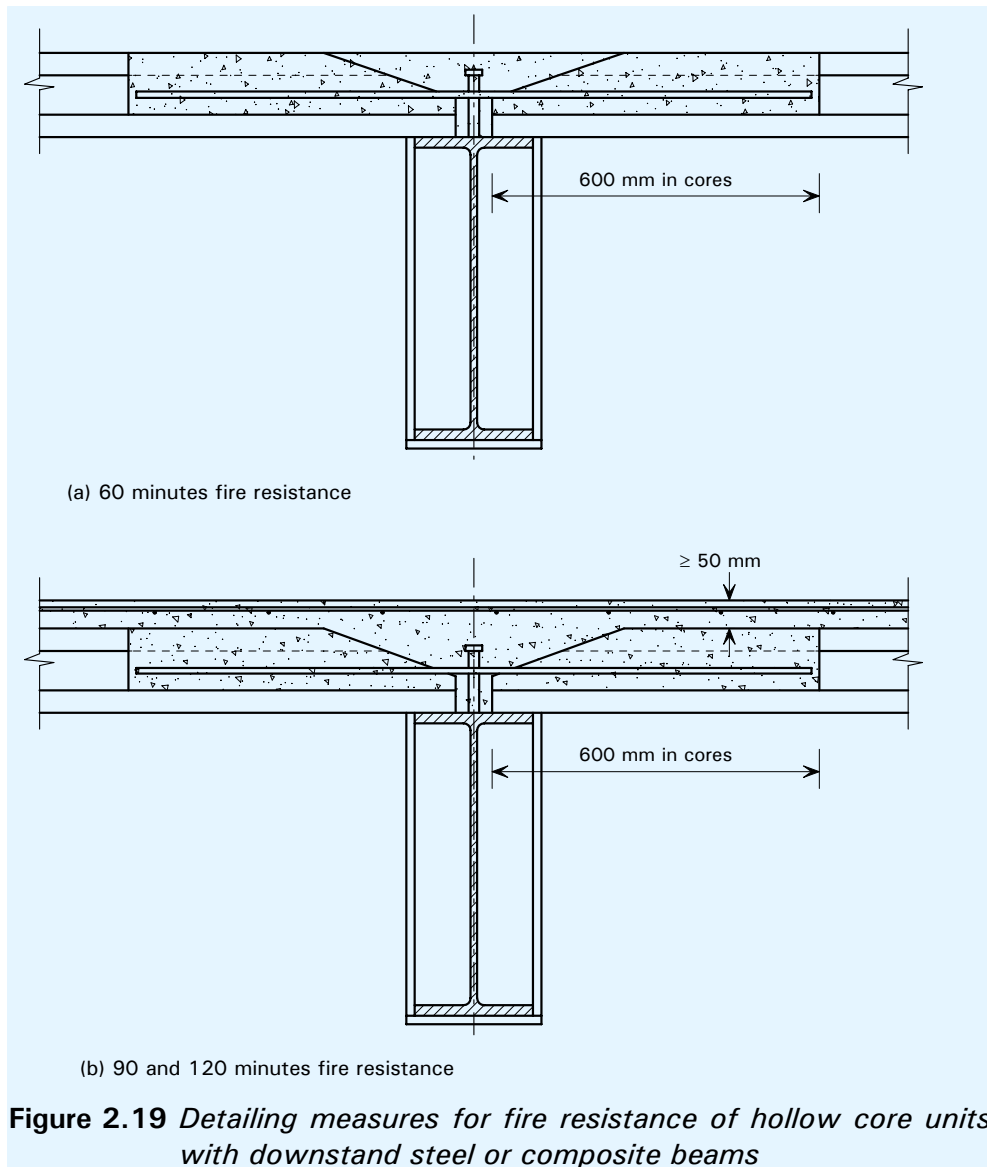
Hollow core units supported on unprotected ASB, Slimflor and shelf angle beams

The effect of non rigid supports, as discussed in Section 2.5, should be considered in the fire condition. The additional stresses in the hollow core units due to non-rigid supports can be much greater in the fire condition than in the normal condition. This is because of the high deformations and curvature of unprotected sections that can occur in a fire. Premature failure from this can be prevented by limiting the end shear on the PC Units, as in the normal condition.

For unprotected sections, a summary of the fire protection requirements for different periods of fire resistance are given below.

For 30 minutes, there are no special requirements. However, effective tying should be provided by a three dimensional tied steel structure.

For 60 minutes, there are three options:

(a) Provide longitudinal tie reinforcement passing over or through the beams and embedded in joints between precast units.

(b) Provide a structural topping with mesh reinforcement.

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(c) Design for a reduced shear resistance of 0.2 V_{Rd} and provide effective tying by a three dimensional tied steel structure.

For 90 minutes, provide longitudinal tie reinforcement (suspension reinforcement) over or through the beams and embedded in filled cores of the precast units and design for a reduced shear resistance of 0.2 V_{Rd}. Note: The provision of a structural topping with mesh reinforcement is optional.

For 120 minutes, the requirements are as for 90 minutes with the addition of a structural topping with mesh reinforcement.

Typical details for different periods of fire resistance are shown in Figure 2.20.
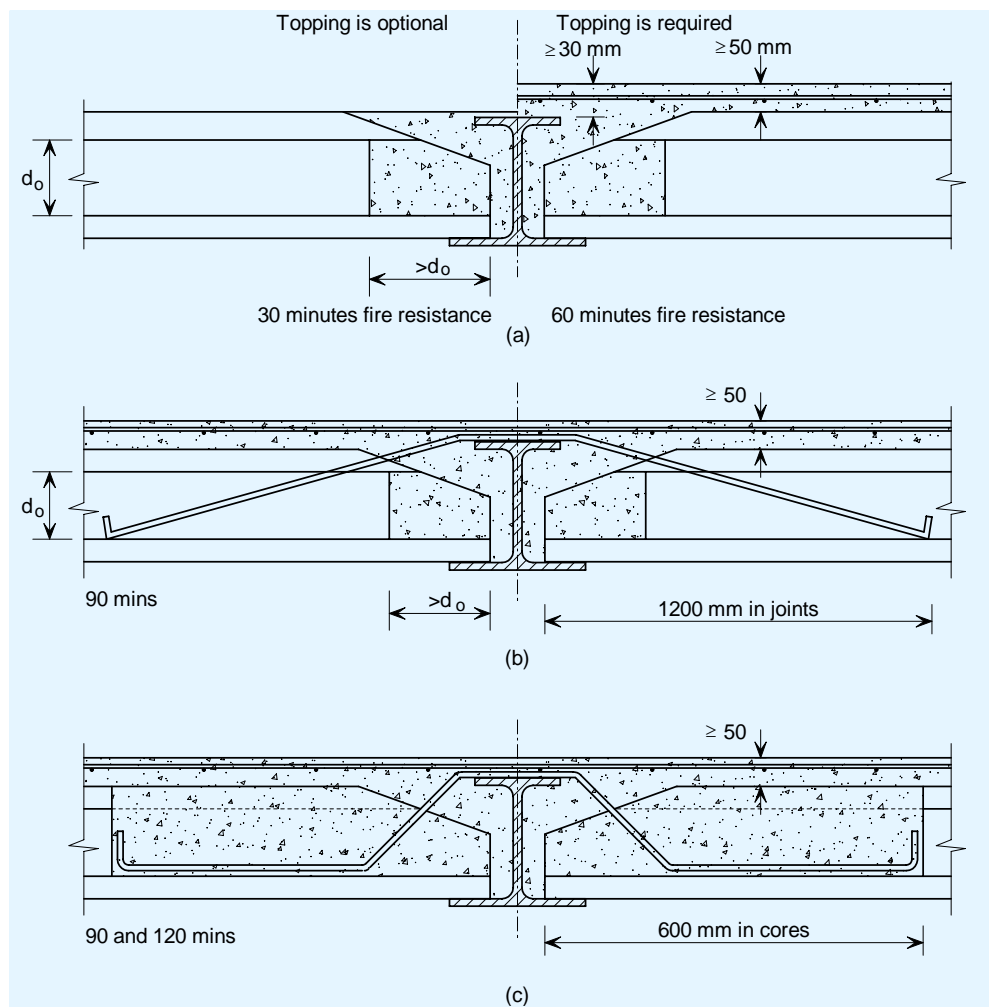
Figure 2.20 *Detailing measures for fire resistance of hollow core units with ASB*

2.9   Dynamics

In recent years, there has been an increase in demand for buildings that have large uninterrupted floor areas and are flexible in their intended final use. Steel framed buildings with precast unit floors can satisfy such demands and produce structures which are competitive in terms of overall cost. However, the trend towards longer span lightweight floor systems, which generally have lower natural frequencies and reduced natural damping, has created a greater

awareness of the dynamic performance of these when subjected to human activities.

Generic, design guidance on the vibration of floors in steel framed buildings is provided in SCI publication P354[14]. Specific guidance regarding dynamic considerations for precast concrete floors supported on steel frames is provided in SCI publication P287[3] for floor with downstand beams and in SCI publication P342[4] for floors with precast units supported on the bottom flange of Asymmetric *Slimflor* Beams.

Precast units with an insitu concrete topping and supplementary continuity reinforcement will behave in a similar manner to a metal decking composite floor systems.

Precast floor units without a structural topping supported on steel beams contribute only by virtue of their mass to the vibration characteristics of the floor as a whole. Therefore, they may not be suitable for dynamically sensitive floors.

2.10 Durability of car park structures

The durability of car park structures needs careful consideration because the environment can be particularly corrosive. The air is usually contaminated with exhaust fumes and the surfaces are sprayed with de-icing salts and water. The effects of these corrosive elements can be minimised by good design, in particular proper drainage, crack control and protection of the top surface of the floor slab. Guidance on crack control is provided in Section 2.4.

There is a range of corrosion protection systems that can be used to protect the steel frame structure. These protection systems have design lives (duration to first major maintenance) of up to 30 years. The actual design life will depend on the protection system selected and the environmental corrosion category. Details of the available protection systems are provided in the Corus publication *Steel-framed car parks*[15].

BS 8500[16] gives guidance on specifying concrete for durability including the assessment of concrete cover, strength, water/cement ratio and minimum cement content. Guidance on the use of BS 8500 is provided in Reference 17, an extract of which is provided in Table 2.1.

Table 2.1 *Concrete specification for durability*

Conditions	Minimum concrete mix requirements	Minimum cover
Car park decks, ramps and external areas subject to freezing and de-icing salts	Strength: C40/50 Water / cement ratio: 0.40 Cement content : 380(20) / 380(15) / 380(10) kg/m^3 CEM1	45 mm + Δc

Notes: Δc is an allowance for deviations.

See Reference 16 or 17 for full details of requirements for cement and aggregate specification.

Car park decks, particularly the top deck, should be waterproofed. It is important that the waterproofing system is appropriate for its intended use and that the installation and maintenance follow the manufacturer's instructions. Advice on waterproofing should be obtained from specialist manufacturers.

3 INITIAL SIZING

3.1 Typical floor loadings

Recommended imposed floor loadings are given in BS 6399-1[18].

For commercial office applications, typical design floor loading is in the range of 2.5 to 4.0 kN/m^2 for the imposed load, plus 1 kN/m^2 for partitions and 0.85 kN/m^2 for raised floors, ceiling and services.

For residential buildings a typical loading is 1.5 kN/m^2 for the imposed floor load and 1.0 kN/m^2 for partitions. Corridors will be designed for an imposed floor load of 3.0 kN/m^2.

A typical imposed floor loading for car parking areas and ramps is 2.5 kN/m^2.

3.2 Initial sizing tables for precast units

This Section provides load span tables for initial sizing of precast floor units. Three tables are presented, one for each of the following floor types:

- Solid precast units with a structural topping (Table 3.1)

- Hollow core units with a structural topping (Table 3.2)

- Hollow core units without a structural topping (Table 3.3).

Most precast floor unit manufacturers produce load span tables for their own products which will be applicable to the units manufactured by that particular producer only. The initial sizing tables presented below are generic tables and will therefore differ from manufacturers tables. The manufacturer should always be consulted prior to final design of the floor.

Table 3.1   *Initial sizing data for solid units with topping*

Imposed Load (kN/m^2)	Maximum span* (m)			
	(propped)	(unpropped)	(propped)	(unpropped)
0.75	7.29	3.75	8.88	5.00
1.5	6.86	3.75	8.44	5.00
2.0	6.61	3.75	8.18	5.00
2.5	6.38	3.75	7.94	5.00
3.0	6.18	3.75	7.72	5.00
4.0	5.83	3.75	7.33	5.00
5.0	5.53	3.75	7.00	5.00
10.0	4.52	3.63	5.82	4.65
Self weight kN/m^2	3.54	3.54	4.71	4.71

Notes:

*   Spans are for the stated imposed load plus an additional 1.5 kN/m^2 for finishes.
     Source: Bison Concrete Products Limited[19]

†   Total depth = Unit depth + Structural topping thickness (mm)

Table 3.2 *Initial sizing data for hollow core units with topping*

Imposed Load (kN/m²)	Maximum span* (m) for total depth (mm) [†]						
	200 (150+50)	250[†] (200+50)	300 (250+50)	375 (300+75)	425 (350+75)	475 (400+75)	525 (450+75)
0.75	8.2	10.7	12.1	14.3	15.7	16.8	18.0
1.5	8.2	10.3	11.6	13.8	15.2	16.2	17.5
2.0	8.2	10.0	11.2	13.5	14.9	15.9	17.1
2.5	8.2	9.7	11.0	13.2	14.6	15.6	16.8
3.0	8.0	9.5	10.7	13.0	14.3	15.3	16.5
4.0	7.6	9.0	10.2	12.5	13.8	14.7	15.9
5.0	7.3	8.7	9.8	12.1	13.3	14.2	15.4
10.0	6.0	7.2	8.1	10.4	11.6	12.4	13.4
Self weight kN/m²	3.60	4.20	4.45	5.75	6.20	6.60	7.10

Notes:

* Spans are for the stated imposed load plus an additional 1.5 kN/m² for finishes.
Source: Bison Concrete Products Limited[19]

† Total depth = Unit depth + Structural topping thickness (mm)

Table 3.3 *Initial sizing data for hollow core units without topping*

Load (kN/m²)	Maximum span* (m) for hollow core unit depth (mm)							
	150	150[§]	200	250	300	350	400	450
0.75	7.5	7.5	10.0	12.4	15.0	16.9	18.0	19.4
1.5	7.5	7.5	10.0	11.7	14.7	16.2	17.3	18.6
2.0	7.5	7.5	10.0	11.3	14.2	15.7	16.8	18.1
2.5	7.5	7.5	9.4	10.8	13.8	15.2	16.3	17.5
3.0	7.2	7.3	9.3	10.5	13.3	14.8	15.8	17.1
4.0	6.7	na	8.8	10.1	12.6	14.0	15.1	16.3
5.0	6.3	na	8.3	9.6	12.0	13.4	14.4	15.6
10.0	5.0	na	6.6	7.7	9.9	11.0	12.0	13.1
Self weight kN/m²	2.40	3.03	2.98	3.30	3.95	4.45	4.85	5.35

Notes:

* Spans are for the stated imposed load plus an additional 1.5 kN/m2 for finishes.

§ 150 mm 'sound slab' for use in residential buildings.
Source: Bison Concrete Products Limited[19, 20]

The initial sizing tables presented above include an additional allowance of 1.5 kN/m² for finishes. Traditionally this represented a 50 mm non-structural sand cement screed and a ceiling. The spans given in the tables will be conservative if this level of additional load is not present.

Design software for initial sizing of precast units can be downloaded from manufacturers' websites.

3.3   Beam sizes

SCI publication P287[3] presents initial sizing tables for the selection of Universal Beam sizes to support hollow core floors.   Additionally, beam selection should consider the minimum required beam flange width.   The minimum beam flange width which can be used to support precast units is dependant on the following factors:

- Required bearing width

- Allowance for tolerances in manufacture of precast units and erection of steelwork

- Required gap between ends of units will depend on whether shear studs are present or not and, if they are welded on site or during the fabrication

- Whether the support is an edge or internal beam

- The end condition of the precast unit

- Whether the beam design is composite or non-composite

- The type of precast unit i.e. solid or hollow core.

BS 8110-1[1] recommends that the actual bearing width of non-isolated precast elements should be at least 40 mm, increased if necessary to reduce local bearing stresses.   This increase is not normally necessary for precast units supported on steelwork.   For isolated members, BS 8110-1 recommends that the actual bearing width is 20 mm larger than for non-isolated members.

BS 8110-1 recommends that the nominal bearing width selected and used for detailing should make allowance for spalling and construction tolerances (as shown in Figure 3.1).   However, there is no need to make any allowance for spalling for steel supports (Table 5.1 of BS 8110-1) and also for prestressed precast units (Table 5.2 of BS 8110-1).   Construction tolerances should include the manufacturing tolerance on the precast units (see Section 1.4.2) and the steelwork erection tolerance (±10 mm on beam spacing is reasonable). BS 8110-1 suggests a variation allowance for each precast unit of 15 mm or 3 mm per metre span, whichever is greater.   This limit is intended to include all tolerances of manufacture and erection.
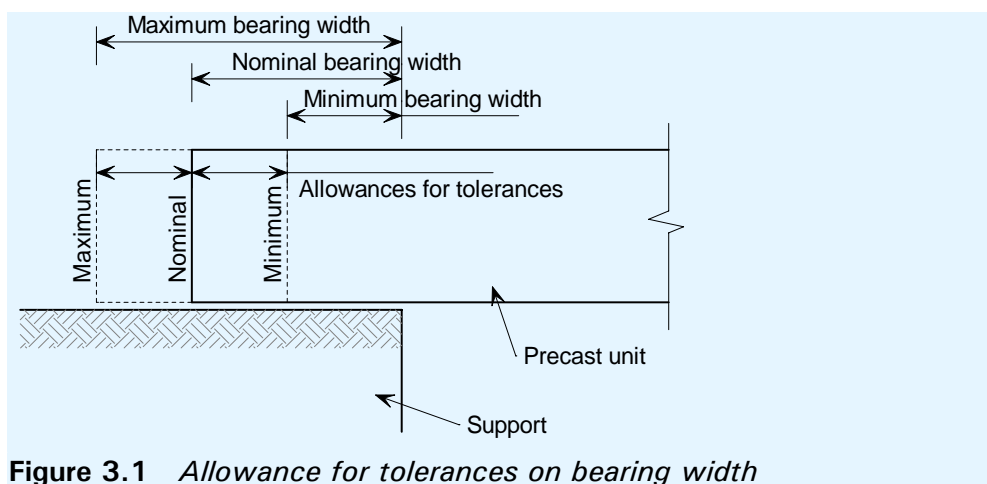
Figure 3.1   *Allowance for tolerances on bearing width*

Table 3.4 gives recommended nominal bearing widths which include allowances for construction variations to provide minimum bearing widths not less than 40 mm. Maximum bearing widths equal to the nominal width plus 10 mm are also presented.

Table 3.5 presents minimum beam flange widths for two precast unit span ranges, various combinations of stud type and different precast unit end type. The minimum beam flange widths are based on the maximum bearing widths given in Table 3.4 plus the required gap between the ends of units.

Table 3.4 *Bearing widths*

Span (m)	Nominal bearing width (mm)	Maximum bearing width (Nominal + Tolerance) (mm)
3.75	50	60
5.0	55	65
6.0	55	65
7.5	55	65
10.0	60	70

Note: Nominal bearing width taking into account all negative tolerances will provide a bearing width of 40 mm.

Table 3.5 *Minimum beam widths*

Precast unit length (m)	Maximum bearing width (mm)	Stud type	Access for stud or gap (mm)	Precast unit end type	Minimum beam width (mm)
< 7.5	65	None	25	Square or Chamfered	155
< 7.5	65	Shop welded	60	Square	190
< 7.5	65	Site welded	65	Square	195
< 7.5	65	Shop welded	50	Chamfered	180
< 7.5	65	Site welded	65	Chamfered	195
≥ 7.5	70	None	25	Square or Chamfered	165
≥ 7.5	70	Shop welded	60	Square	200
≥ 7.5	70	Site welded	65	Square	205
≥ 7.5	70	Shop welded	50	Chamfered	190
≥ 7.5	70	Site welded	65	Chamfered	205

4 DETAILING

4.1 Introduction

For buildings of all types of construction, appropriate detailing of connections and the interfaces between elements can be vital for structural adequacy, serviceability criteria, ease of construction and for compliance with building regulations. For precast floor units, these aspects are covered in this Section of the publication.

4.2 Structural Integrity

4.2.1 The Building Regulations

Regulatory requirements to avoid disproportionate collapse were first introduced in the UK following the progressive partial collapse of a block of flats at Ronan Point, London, in 1968 (Figure 4.1). A gas explosion in a flat on the 18th floor knocked out load-bearing precast concrete wall panels, the floors above collapsed due to the lack of support from below, and the impact of the collapsing floors caused further collapses all the way to the ground.

Figure 4.1   *Partial collapse of Ronan Point (1968)*

The Building Regulations of England and Wales[21] require that all buildings are designed to avoid disproportionate collapse. Requirement A3 of the Regulations states:

> *Disproportionate Collapse*
> *A3. The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause*

The primary and most common approach to avoid disproportionate collapse of steel framed buildings is to provide structural integrity by having both horizontal and vertical tying of the frame elements to ensure whole frame action. BS 5950-1[22] includes two other methods for providing structural integrity to avoid disproportionate collapse; design for notional removal of supports and key element design. The guidance given in this publication focuses on the provision of tying resistance in buildings with precast floors. Detailed explanation of the requirements of the Building Regulations of England and Wales regarding

disproportionate collapse and how to comply with them are provided in SCI publication P341[23].

The disproportionate collapse regulations in Scotland and Northern Ireland are slightly different from those in England and Wales. The regulations have slightly different limits defining when disproportionate collapse should be considered. Guidance concerning these differences is provided in Advisory Desk Note AD297[24] and AD303[25].

Approved Document A (2004 Edition)[26], which provides guidance on satisfying the Building Regulations of England & Wales, sets out required levels of robustness for different types and sizes of buildings. There are four classes of building: Class 1, Class 2A, Class 2B and Class 3. The robustness requirements are progressively more stringent from Class 1 to Class 3.

Table 11 in Approved Document A (reproduced here as Table 4.1) is used to define the class of the building.

Table 4.1   *Building Classification*

Class	Building type and occupancy
1	Houses not exceeding 4 storeys.
	Agricultural buildings
	Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height
2A	5 storey single occupancy houses
	Hotels not exceeding 4 storeys
	Flats, apartments and other residential buildings not exceeding 4 storeys
	Offices not exceeding 4 storeys
	Industrial buildings not exceeding 3 storeys
	Retailing premises not exceeding 3 storeys of less than 2000 m^2 floor area in each storey
	Single storey educational buildings
	All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey
2B	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys
	Educational buildings greater than 1 storey but not exceeding 15 storeys
	Retailing premises greater than 3 storeys but not exceeding 15 storeys
	Hospitals not exceeding 3 storeys
	Offices greater than 4 storeys but not exceeding 15 storeys
	All buildings to which members of the public are admitted which contain floor areas exceeding 2000 m^2 but less than 5000 m^2 at each storey
	Car parking not exceeding 6 storeys
3	All buildings defined above as Class 2A and 2B that exceed the limits on area and/or number of storeys
	Grandstands accommodating more than 5000 spectators
	Buildings containing hazardous substances and/or processes

Note 1:   For buildings intended for more than one type of use the Class should be that pertaining to the most onerous type.

Note 2:   In determining the number of storeys in a building, basement storeys may be excluded provided such basement storeys fulfil the robustness requirements of Class 2B buildings.

4.2.2 Class 1 and 2A Buildings

For steel framed buildings designed to BS 5950-1, the structural provisions necessary to meet Regulation A3 are the same for Class 1 and Class 2A buildings. This is because the minimum amounts of tying recommended in BS 5950-1 are sufficient to meet regulatory requirements for Class 1 and Class 2A buildings.

The robustness requirements for Class 2A buildings given in Approved Document A are:

For Class 2A buildings - Provide effective horizontal ties, or effective anchorage of suspended floors to walls, as described in the Codes and Standards listed under paragraph 5.2 for framed and load-bearing wall construction.

Hence, for framed construction there is no requirement to tie the floors to the frame. However, the framed elements must be tied together to meet the minimum recommendations of BS 5950-1 which are:

- Columns should be tied in two directions, approximately at right angles, at each principal floor level.

- All ties (which should be located along the edges of the building and along each column line) and their end connections should be capable of resisting a factored tensile load of at least 75 kN.

- Horizontal ties should also be provided at roof level, except where steelwork only supports cladding that weighs not more than 0.7 kN/m^2 and that carries only imposed roof loads and wind loads.

The details shown in Figure 4.2 to Figure 4.5 are examples of how precast floor units on supporting steel beams may be constructed in Class1 and Class 2A buildings. The minimum bearing recommendations for precast units given in BS 8110-1 must be satisfied.

a) Floor unit on top flange of beam   b) Floor unit on shelf angle beam
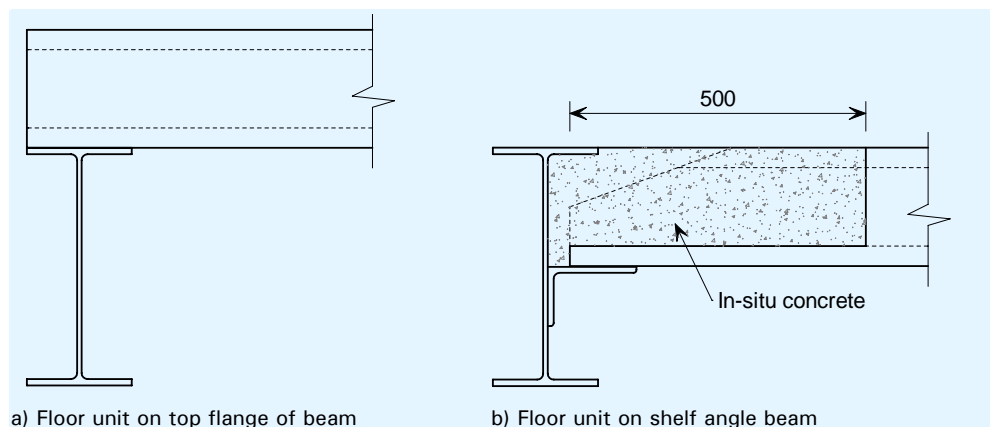
Figure 4.2 *Junction between supporting edge beam and floor unit for Class 2A*

a) Floor unit on top flange of beam

b) Floor unit on shelf angle beam
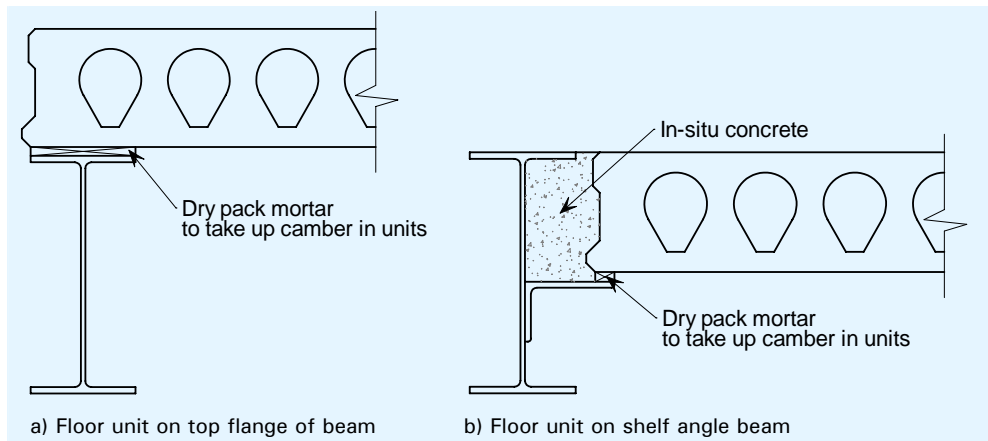
Figure 4.3 *Junction between edge tie beam and floor unit for Class 2A*

a) Floor unit on top flange of beam

b) Floor unit on shelf angle beam
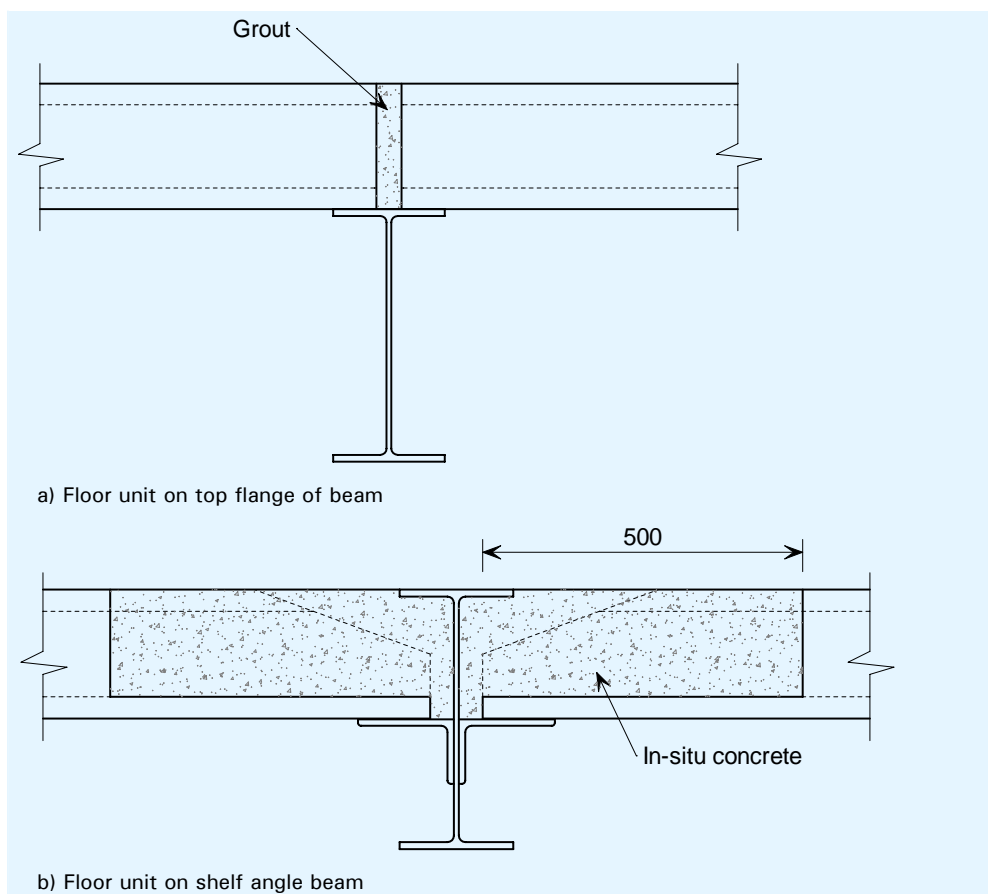
Figure 4.4 *Junction between internal supporting beam and floor units for Class 2A*

Figure showing floor unit connections with labels:
- Concrete grout
- Dry pack mortar to take up camber in units
- a) Floor unit on top flange of beam
- In-situ concrete grout
- Dry pack mortar to take up camber in units
- b) Floor unit on shelf angle beam
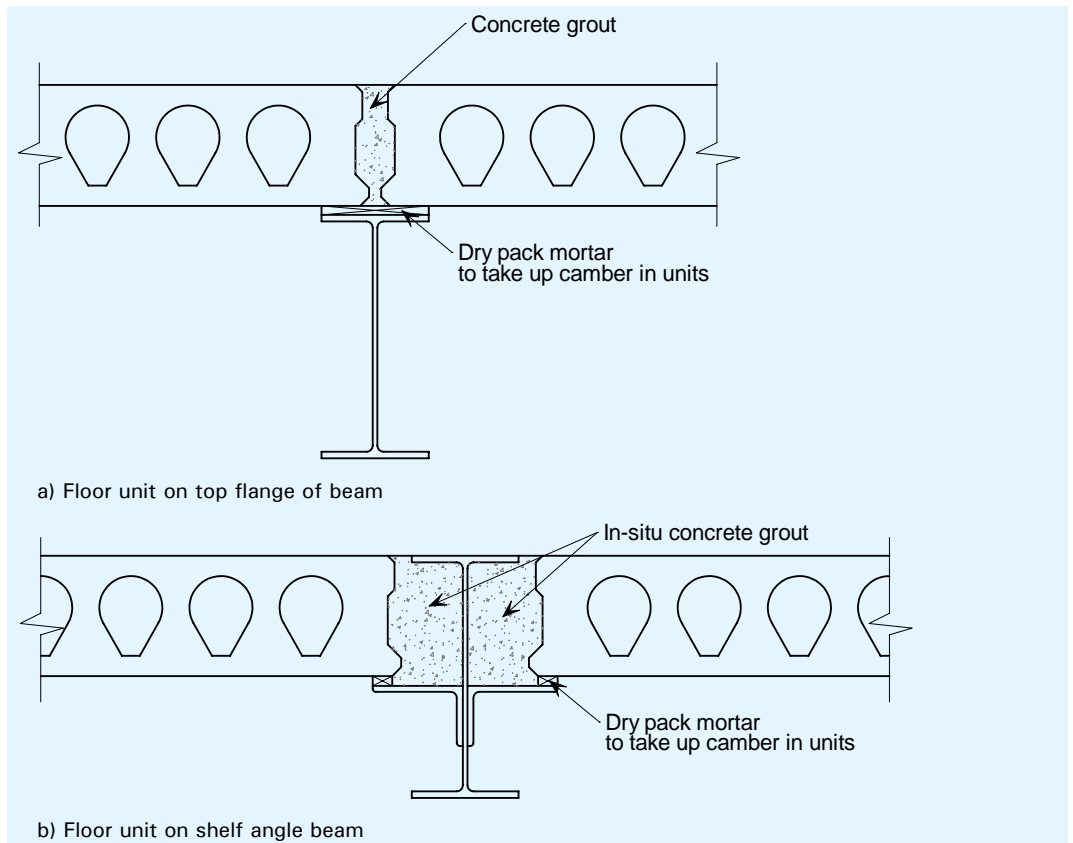
Figure 4.5 *Junction between internal tie beam and floor units for Class 2A*

4.2.3 Class 2B buildings

The robustness provisions required for Class 2B buildings are more stringent than those for Class 1 or 2A buildings. The robustness requirements for Class 2B buildings, given in Approved Document A, are:

For Class 2B buildings:

a) Provide effective horizontal ties, as described in the Codes and Standards listed under paragraph 5.2 for framed and load-bearing wall construction,… together with effective vertical ties, as defined in the Codes and Standards listed under paragraph 5.2, in all supporting columns and walls,

or alternatively

b) Check that upon the notional removal of each supporting column and each beam supporting one or more columns, or any nominal length of load-bearing wall (one at a time in each storey of the building) that the building remains stable and that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or 70 m², whichever is smaller, and does not extend further than the immediate adjacent storeys (see Diagram 25).

Where the notional removal of such columns (or beams supporting one or more columns) and lengths of walls would result in an extent of damage in excess of the above limit, then such elements should be designed as a "key element" as defined in paragraph 5.3 below.

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For hot-rolled steel-framed buildings with precast floor units, the requirements concerning load-bearing walls are not relevant.

To satisfy the robustness requirements, the main difference for steel framed buildings with precast floor units and steel framed buildings with other floor systems is the manner in which the floor units are tied to the structural frame.

For Class 2B buildings, the recommendations of Clause 2.4.5.3 of BS 5950-1 should be followed. Clause 2.4.5.3 has five sub-clauses a) to e) which should be followed. Clause 2.4.5.3 e) provides recommendations for tying heavy floor units such as precast hollow core units. All the requirements of Clause 2.4.5.3 are discussed below.

Horizontal Tying

Clause 2.4.5.3 a) *General tying*, describes which horizontal members should be designed as ties (Figure 4.6) and presents equations (reproduced below) for calculating the tensile loads that the ties and their end connections should be capable of resisting.

Note: All beams shown are designed as ties
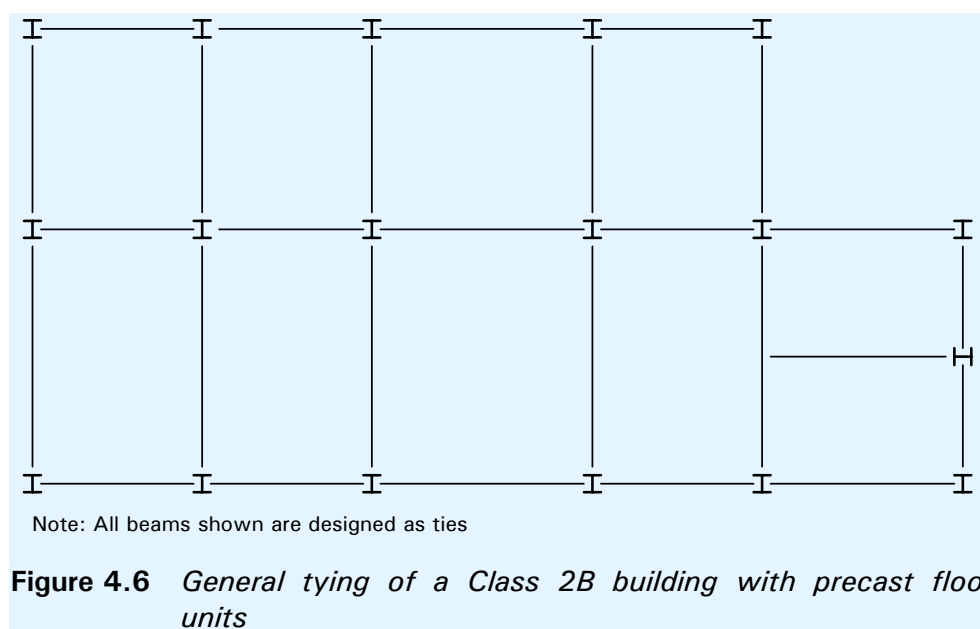
Figure 4.6 *General tying of a Class 2B building with precast floor units*

For internal ties: $0.5(1.4\ g_k + 1.6\ q_k)\ s_t\ L\ n$ but not less than 75 kN

For edge ties: $0.25(1.4\ g_k + 1.6\ q_k)\ s_t\ L\ n$ but not less than 75 kN

Where:

g_k is the specified dead load per unit area of the floor or roof

L is the span

q_k is the specified imposed floor or roof load per unit area

s_t is the mean transverse spacing of the ties adjacent to that being checked

n is a factor related to the number of storeys in the structure see Table 4.2.

Table 4.2 *Reduction factor for required tie capacities*

Number of storeys in building	Reduction factor, *n*
5 or more	1.0
4	0.75
3	0.50
2	0.25
1	0

Note: In determining the number of storeys in a building, basement storeys may be excluded provided such basement storeys fulfil the robustness requirements of Class 2B buildings.

Tying of edge columns

Clause 2.4.5.3 b) *Tying of edge columns*, states that ties connected to edge columns should be capable of resisting the larger of the following forces:

- The design loads for general tying specified in Clause 2.4.5.3 (a)

- 1% of the factored vertical dead and imposed load in the column at that level.

Tabulated capacities and calculation procedures for tying the capacities of typical beam-to-column and beam-to-beam connections are given in SCI publication P212[9].

Vertical tying

Clause 2.4.5.3 c) *Continuity of columns*, requires that all column splices should be capable of resisting an axial tension equal to the largest total factored vertical dead and imposed load reaction applied to the column at a single floor level located between that column splice and the next column splice down (or to the base).

As for beam connections, guidance on designing column splices for tying forces is given in SCI publication P212.

Bracing systems

Clause 2.4.5.3 (d) *Resistance to horizontal forces* requires at least two sets of bracing (or other system for resisting horizontal force) in each orthogonal direction; so that no substantial part of the structure is braced by only one set of bracing in any single direction.

Guidance on the design of bracing systems in steel-framed buildings is provided in SCI publication P334[27].

Heavy floor units

Clause 2.4.5.3 (e) *Heavy floor units* requires that precast concrete or other heavy floor or roof units are effectively anchored in the direction of their span, either to each other over a support, or directly to their supports as, recommended in BS 8110[1]. Clause 5.1.8.3 of BS 8110-1 recommends that the anchorage of the floor units should be capable of carrying the dead weight of the member. However, the accidental load condition from Clause 2.4.5.3 of BS 5950-1 is dead load plus a third of the live load with an overall partial load factor of 1.05.

The required anchorage force for a precast unit is calculated below.

$$\text{Anchorage force} = 0.5\,(1.05\,(1.0g_k + 0.33q_k) \times L \times w\,)$$

Where typical values may be:

g_k (Dead load) $= 3.5$ kN/m^2

q_k (Live load) $= 5$ kN/m^2

L (Span) $= 7$ m

w (Unit width) $= 1.2$ m

Which gives a typical requirement of:

$$\text{Anchorage force} = 0.5\,(1.05\,(3.5 + 0.33 \times 5) \times 7 \times 1.2) = 22.7 \text{ kN}$$

A T12 high yield reinforcement bar (with a yield strength of 500 N/mm^2) that is properly anchored will provide a tie capacity of approximately 49 kN and a T16 bar will provide a tie capacity of approximately 87 kN.

The details shown in Figure 4.7 to Figure 4.12 are examples of how precast floor units on supporting steel beams may be tied in Class 2B buildings.

Tying floor units to supporting edge beams

Figure 4.7 shows tying details for the junction between a supporting edge beam and precast floor units. For this situation BS 5950-1 recommends that "*units are effectively anchored in the direction of their span... directly to their supports*" this is achieved by tying the floor units to the supporting steel beam.

The detail in Figure 4.7a) ties the floor unit to the steel beam by placing steel reinforcement U bars around shear studs welded to the steel beam. The U bars are located in opened cores of the floor unit. The welded studs and U bars are encased in in-situ concrete.

The option for shelf angle beams, shown in Figure 4.7b), is to weld reinforcing bar to the top flange of the supporting steel beam. The other part of the reinforcing bar is located in opened cores and anchored in position with in-situ concrete. The spacing of bars and their diameter can be selected to suit the tying capacity required.

Note: Adjacent cores should not be opened – see Section 1.5.5.

Details appropriate for tying precast floor units to asymmetric *Slimflor* edge beams and RHS with welded plate edge beams are shown in Figure 4.8 and Figure 4.9.

a) Floor unit on top flange of beam

b) Floor unit on shelf angle beam
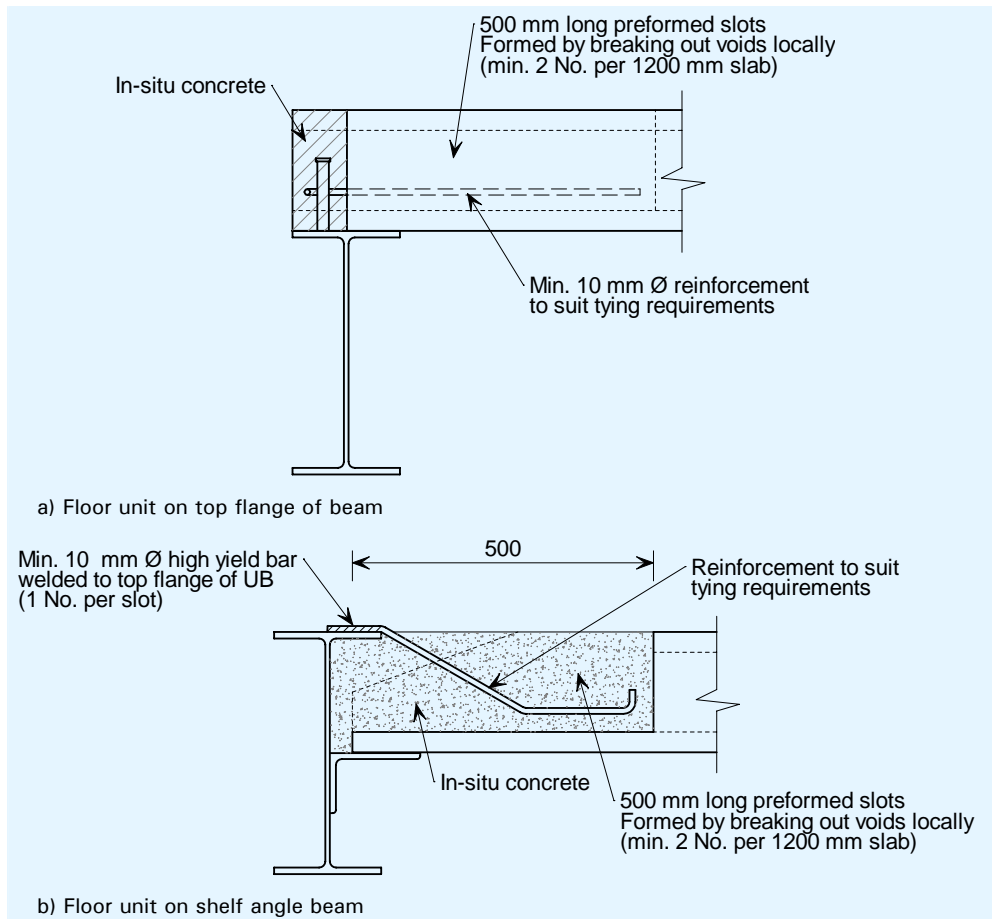
Figure 4.7 *Junction between supporting edge beam and floor units for Class 2B*

Figure 4.8 *Junction between supporting RHS edge beam and floor units for Class 2B*

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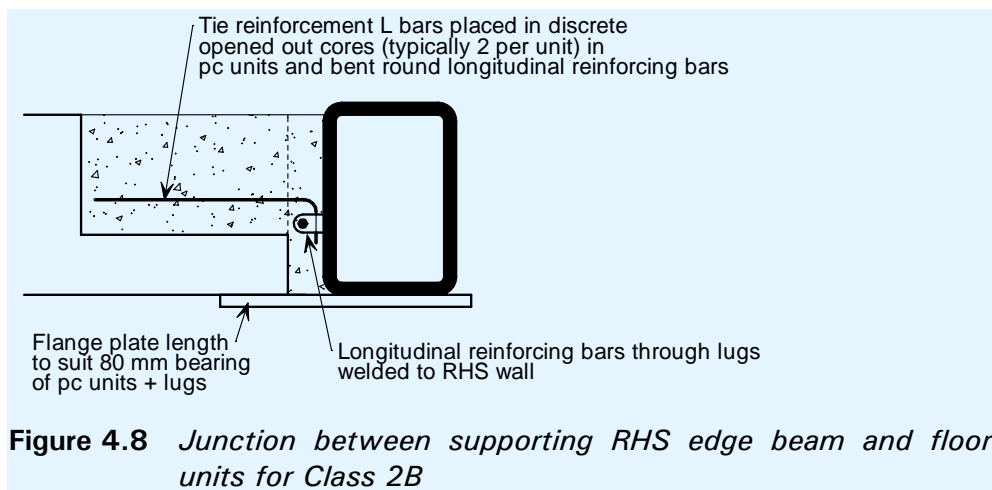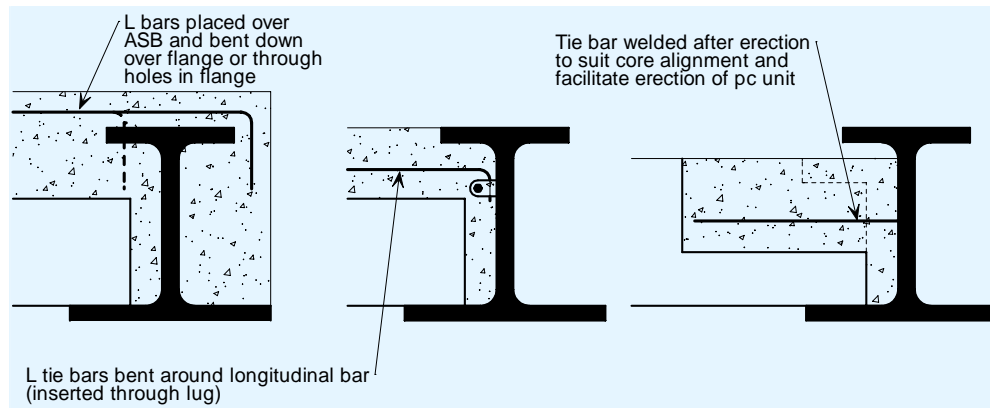
Figure 4.9 *Junction between supporting ASB edge beam and floor units for Class 2B*

Tying floor units to supporting internal beams

Figure 4.10 shows details for the floor units spanning onto internal supporting beams. For this situation BS 5950-1 recommends that "*units are effectively anchored in the direction of their span, either to each other over a support or directly to their supports*".

If the floor units are designed to act compositely with the supporting steel beams, transverse reinforcement and shear studs are required (see Section 2). In many cases these provisions will provide sufficient tying capacity between the precast units. However, the tying capacity of the details will need to be checked and the details modified if an increase in tying capacity is required.

The floor units shown in Figure 4.10(a) are tied to each other over the steel beam support by placing steel reinforcement in opened cores. The reinforcement is encased in in-situ concrete placed in the opened cores.

The option for shelf angle beams, shown in Figure 4.10(b), is to place reinforcing bar over the top flange of the supporting steel beam and into opened cores in the precast units on either side. The reinforcing bar is welded to the top flange of the beam and anchored in the opened cores with in-situ concrete. The weld helps to ensure that the reinforcing bar remains in the correct position as the in-situ concrete is placed. The spacing of bars and their diameter can be selected to suit the tying capacity required. An alternative to site welding where this facility is not available is to place bars through holes pre-drilled in the web of the beam.

a) Floor unit on top flange of beam

b) Floor unit on shelf angle beam

Figure 4.10 *Junction between internal supporting beam and floor units for Class 2B*
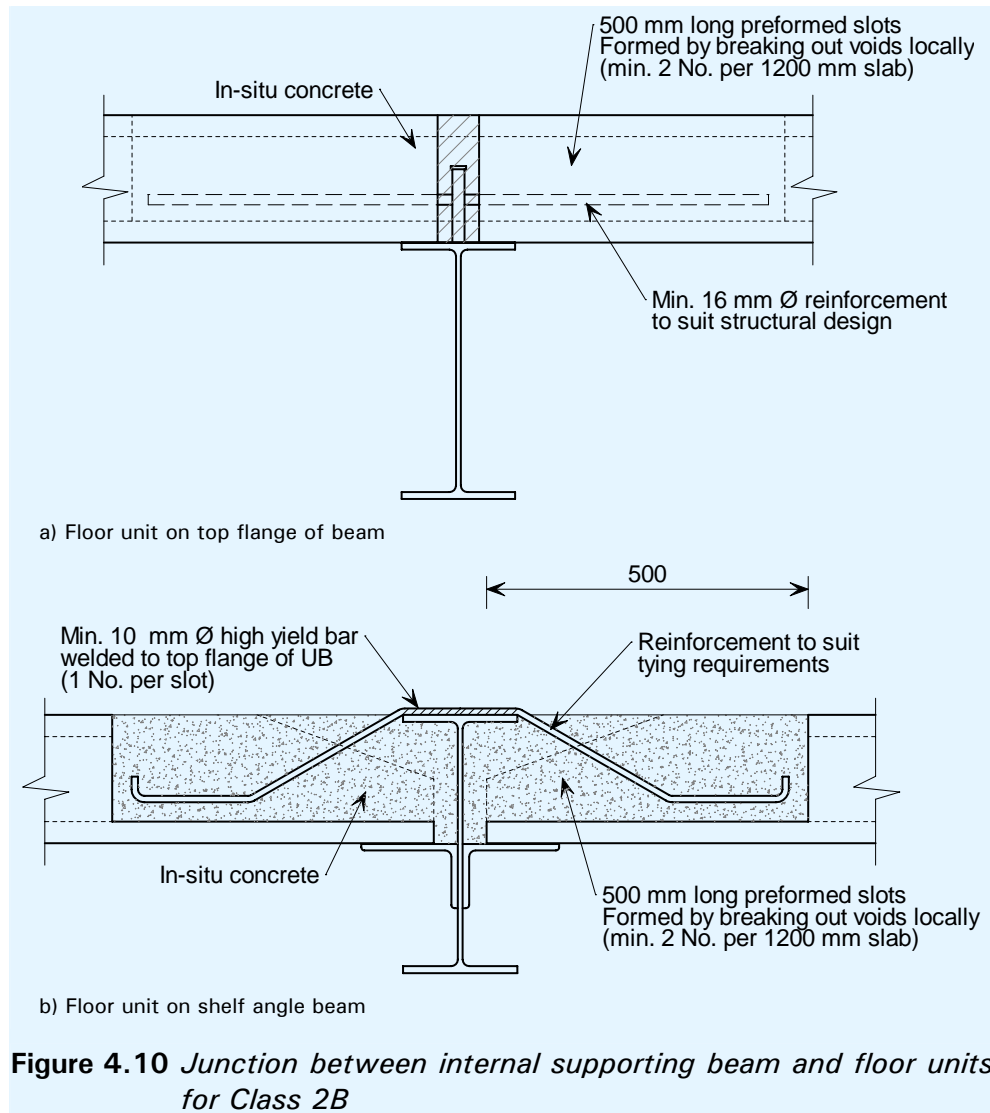
Tying precast floor units supported on the bottom flange of asymmetric *Slimflor* beams (ASB) can be achieved by placing rebar in opened cores and passing it through holes drilled in the web of the ASB.

If the precast units have a structural topping, it may be possible to use the reinforcement in the topping to provide the required tying capacity, as shown in Figure 4.11 for downstand beams.

Figure 4.11 *Tying precast units with structural topping*
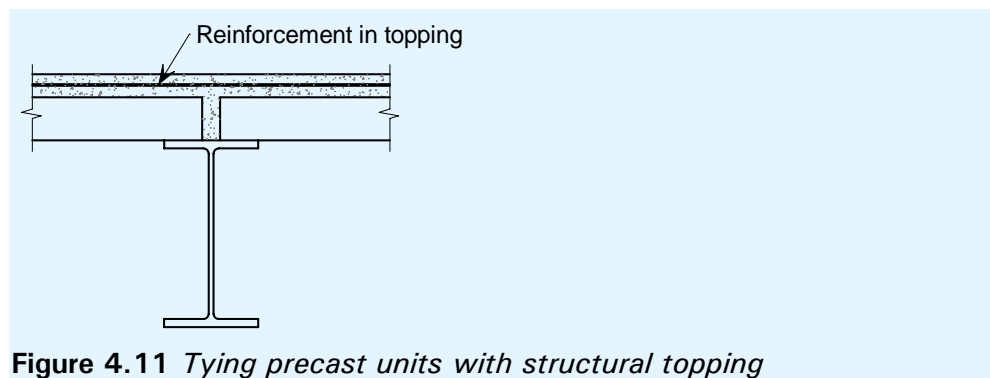
Tying floor units to edge tie beams

Figure 4.12 shows suitable details for tying floor units to edge beams which are parallel to direction in which the units span (i.e. tie beams). For this situation, BS 5950-1 does not recommend any specific tying of the floor units to the edge beams, provided that the structural steel frame is tied together such that Clauses 2.4.5.3 a) and 2.4.5.2 are satisfied. Tying the floor units to the edge beams provides additional structural integrity and may also be necessary for other structural reasons such as transferring horizontal loads, lateral restraint and stability of the beams.

The precast unit is tied to the edge beam using reinforcing bars that link the steel beam to the precast unit. The reinforcing bars are cast into side pockets (Figure 4.13) in the precast units and linked to the beam by being cast around shear studs welded to the steel beam (see Figure 4.12a)). If there are no shear studs, the reinforcing bar can be welded to the top flange of the steel beam as shown in Figure 4.12b) for the case with shelf angle beams.

Details appropriate for *Slimflor* edge beams (parallel to the span of the precast unit) are shown in Figure 4.14.

a) Floor unit on top flange of beam

b) Floor unit on shelf angle beam
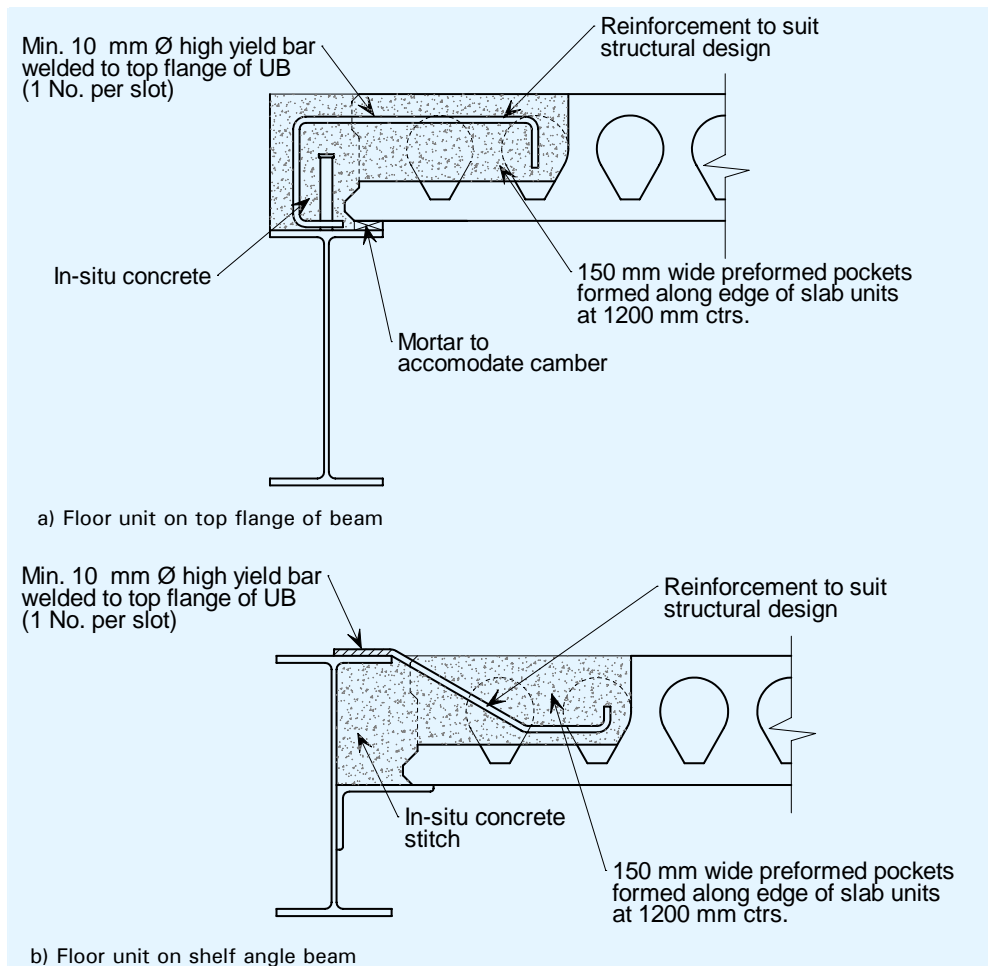
Figure 4.12 *Junction between edge tie beams and floor units for Class 2B*

Figure 4.13 *Side pocket in precast unit*

Reinforcing bar should generally be at least 10 mm diameter and side pockets are normally spaced at 1200 mm centres along the precast unit. The spacing of the pockets and the diameter of the reinforcement can be changed to suit the structural requirements.
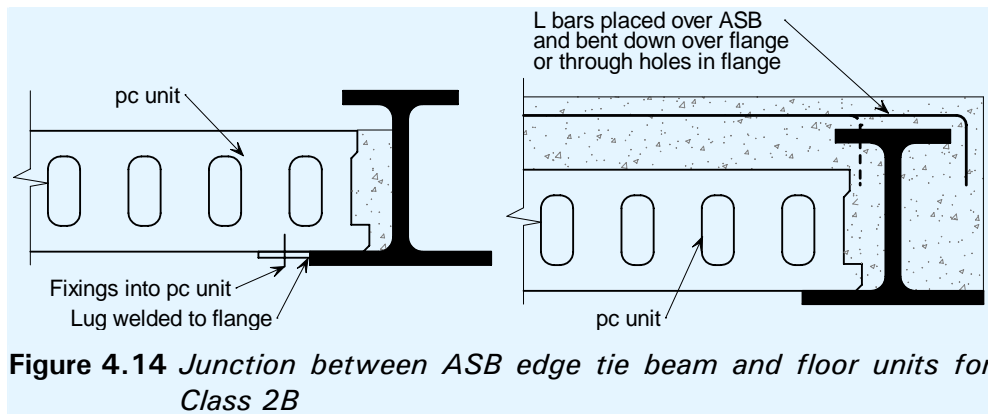
Figure 4.14 *Junction between ASB edge tie beam and floor units for Class 2B*

4.2.4   Stair units

In an accidental loading situation, the stair units are likely to be the main form of escape for occupants and the access route for emergency service personnel. Therefore, it is important that the staircase units are tied into the structure to reduce the likelihood of their collapse in an accidental loading situation.

For robustness purposes, precast stair units should be treated in a similar manner to precast floor units. The stair units should be tied to the structural steel frame. Figure 4.15 shows an angle section that is cast into the staircase and used as the bearing point. The tying capacity is usually provided by reinforcing bars cast into the stair unit, which are then cast into the floor.
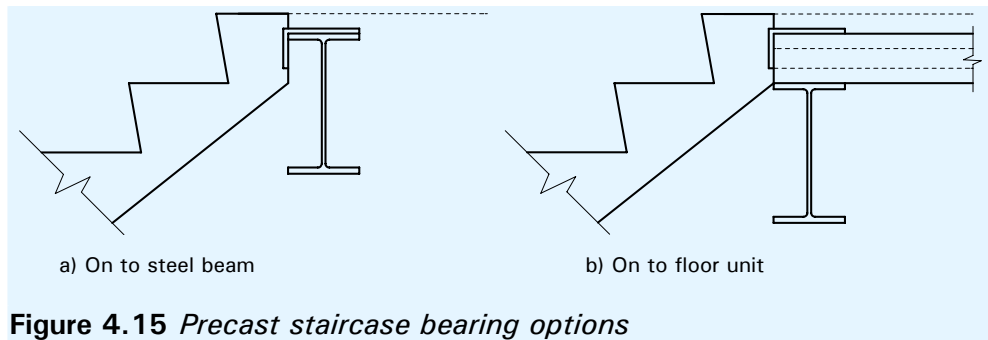
Figure 4.15 *Precast staircase bearing options*

Figure 4.16 shows two connection details between a precast stair units and a supporting steel beam where shear studs are present on the supporting beam. The detail shown in Figure 4.16b) is more appropriate where the beam flange width is narrow (e.g. <180 mm).
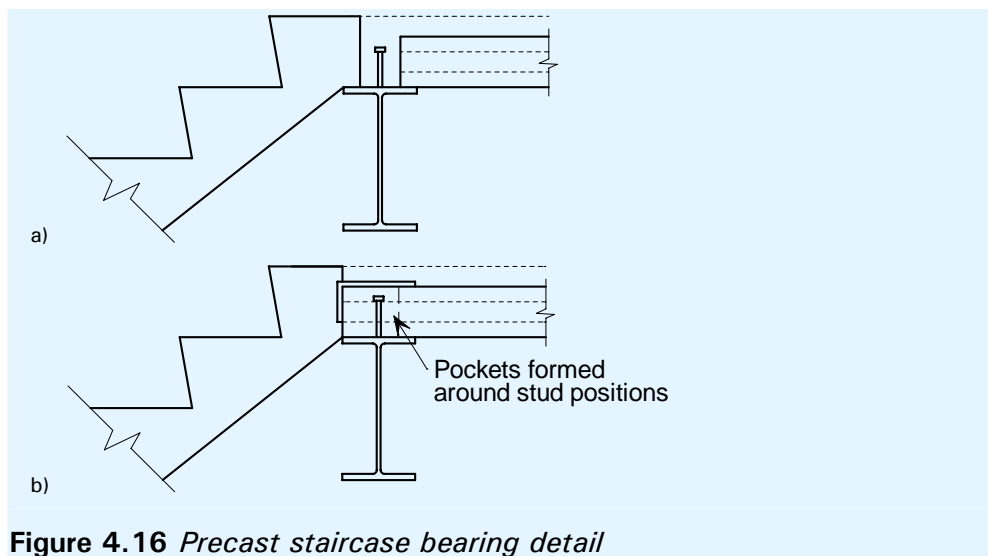
Figure 4.16 *Precast staircase bearing detail*

It is often advantageous to specify stair units with integral landings as shown in Figure 4.17 because the number of supporting steel beams is reduced and the landings are automatically tied to the stairs by design.
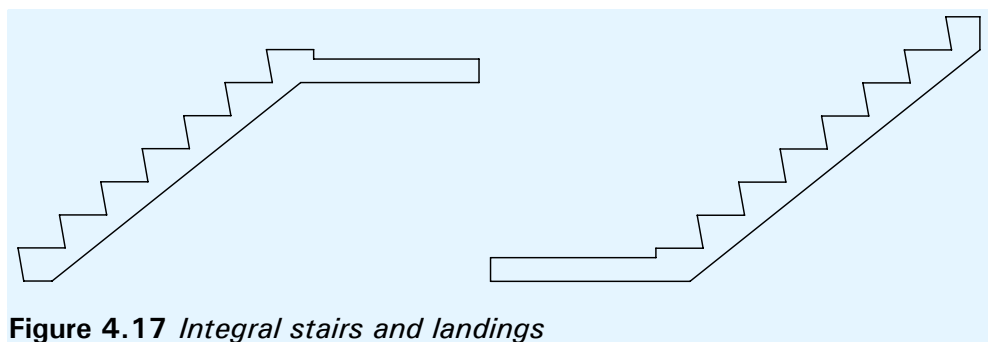
Figure 4.17 *Integral stairs and landings*

4.3   Acoustic performance

High standards of sound insulation and acoustic performance of buildings contributes significantly to the quality of life for those people who live and work in them. This is particularly true for residential, educational and health care buildings.

The acoustic requirements of residential buildings are normally given in national building regulations and associated guidance documents. For England and Wales this is Part E of the Building Regulations 2000 and Approved Document E[26]. Similar equivalent documents exist for use in Scotland and Northern Ireland.

For school buildings the acoustic requirements are given in Building Bulletin 93 *The Acoustic design of schools*[28]; for Hospitals the requirements are given in *Health Technical Memorandum 2045: Acoustics - Design considerations*[29] and *Health Technical Memorandum 56: Partitions*[30].

BS 8233:1999 *Sound insulation and noise reduction for buildings* [31] gives recommendations for acoustic performance in a range of building types.

4.3.1   Residential Buildings

Part E of the Building Regulations

Only Requirement E1 (see below) of Part E of the Building Regulations has an influence on the construction and detailing of the structural frame and floor. Requirement E1 relates to separating walls and floors, and their junction details. The other regulations in Part E refer to surface finishes, partition walls and other building types. Requirement E1 states:

> *E1: Protection against sound from other parts of the building and adjoining buildings*
>
> *Dwelling-houses, flats and rooms for residential purposes shall be designed and constructed in such a way that they provide reasonable resistance to sound from other parts of the same building and from adjoining buildings.*

Rooms for residential purposes, as referred to in Requirement E1, includes rooms in hotels, hostels, boarding houses, halls of residence and residential homes etc. but does not include rooms in hospitals, or other similar establishments, used for patient accommodation.

Approved Document E provides guidance on how these requirements may be satisfied and sets acoustic performance standards. The required levels of insulation to airborne and impact sound are summarised in Table 4.3 and Table 4.4 respectively.

Table 4.3   *Airborne sound insulation requirements*

Building type	Element	Airborne sound insulation performance $D_{nT,w} + C_{tr}$ (dB)
Purpose built: Dwelling houses and flats	Separating walls	≥ 45
	Separating floors and stairs	≥ 45
Purpose built: Rooms for residential purposes	Separating walls	≥ 43
	Separating floors and stairs	≥ 45

Table 4.4   *Impact sound insulation requirements*

Building type	Element	Impact sound insulation performance $L'_{nT,w}$ (dB)
Purpose built: Dwelling houses and flats	Separating floors and stairs	< 62
Purpose built: Rooms for residential purposes	Separating floors and stairs	< 62

The measurement term $D_{nT,w} + C_{tr}$ is a site measured property that represents the airborne sound level difference between a pair of adjacent rooms over a range of frequencies. The C_{tr} term is used to specifically take account of low frequency sounds such as that which can be emitted by home audio equipment.

The performance measurement term $L'_{nT,w}$ is a site measured property that represents the impact sound pressure level in the receiving room of the test.

Demonstrating compliance with Part E

There are two methods of demonstrating compliance with Part E of the Building Regulations; pre-completion testing (PCT) and by use of Robust Details.

PCT is carried out on-site and the onus is on the builder to demonstrate compliance. It is recommended that 1 in 10 of each type of construction detail is tested. PCT only applies to separating walls and floors and is not necessary for internal elements. PCT should be carried out when the rooms either side of the separating element are essentially complete, except for decoration. Tests should be carried out without non-permanent decorative floor coverings (e.g. carpet, laminate flooring, vinyl). The revisions to Part E and therefore the need for PCT took effect from 1 July 2004.

Robust Details (RDs) were developed as an alternative to PCT. A range of details have been developed which consistently satisfy (and exceed) the acoustic performance requirements specified in Approved Document E. The available RDs are published in a handbook by Robust Details Limited[32]. To use a Part E Robust Detail in the construction process, builders must first obtain permission from Robust Details Limited and pay the requisite fee for each dwelling. Provided that the Robust Details are built correctly, this will be accepted by building control bodies in England and Wales as evidence that the homes are exempt from PCT.

For steel frames with precast floor units, PCT is currently the most appropriate route for demonstrating compliance with Part E. This is because there are as yet no Robust Details for precast floor units supported on steel frames. The range of possible details combined with the strict testing and approval process for Robust Details means that acquiring approval for new steel construction RDs is laborious. However, significant progress has already been made for a range of details of precast units supported on steel frames towards obtaining Robust Details Limited approval.

Guidance constructions are provided in Approved Document E that should provide the required acoustic performances, provided they are built correctly. However, these are often conservative solutions and still require PCT to be carried out.

Floor unit density

To satisfy the sound insulation requirements of residential buildings, a precast unit with a minimum density of 300 kg/m^2 is generally required. For standard hollow core units at least a 200 mm deep unit is normally required to provide a unit floor density 300 kg/m^2. Specially designed 'sound slab' precast units are available which are only 150 mm deep but have a density of at least 300 kg/m^2. However, the 200 mm deep unit has advantages over the 150 mm 'sound slab' in terms of detailing both for acoustics and for providing tying for disproportionate collapse. For acoustics the 200 mm unit only requires a 100 mm gap from the underside of the unit to the ceiling whereas a 150 mm unit requires a 150 mm gap. Providing tie bars between 'sound slab' units is more difficult than the 200 mm deep unit because there are fewer cores and they are smaller.

Recommended details

The level of sound insulation between two adjacent spaces depends on direct sound transmission and also flanking sound transmission. Direct sound passes through the separating element and is relatively straightforward to control. Flanking sound bypasses the separating element by travelling through gaps and other elements of the structure. It is important that the junctions between separating elements are detailed correctly to minimise flanking sound transmission. The junction details shown in Figure 4.18 to Figure 4.21 are recommended details for use in steel framed buildings with precast floor units. Propriety versions of these details are being developed by Bison Concrete Products Ltd and their industry partners. The details have been tested and, if built correctly, will satisfy the acoustic performance requirements of Approved Document E.

The acoustic performance of a building is sensitive to the quality of the on-site workmanship, as gaps or absent insulation can seriously impair the sound insulation performance by increasing the amount of flanking sound transmission. Isolation of different elements and changes in material density help to reduce sound transmission. For this reason, the screed is separated from the precast floor units with a 10 mm proprietary isolating foam layer. The room linings (plasterboard on the walls and ceiling) are not fixed directly to the structural frame or precast floor units.

The details in Figure 4.18 to Figure 4.21 show the precast units without a structural topping. If a structural topping is provided for structural reasons, the isolating foam layer and screed will be required over the structural topping.

The recommended acoustic details presented in this Section do not include junction details for floors supported on ASB sections because this arrangement has not been acoustically tested by Bison Concrete Products Ltd. Other acoustic detailing guidance e.g. Robust Details Handbook and SCI publication P336[33] do not include details for precast floors supported on ASB sections. However, the principles used in these documents can be used to develop appropriate acoustic details. In practice, an acoustic consultant should be employed to advise on the suitability of selected details.

Figure 4.18 *Acoustic detailing of a separating wall junction with a separating floor (precast unit spanning perpendicular to separating wall)*

Light steel stud wall with two layers of gypsum based board 22 kg.m² (total)

50 mm glass mineral wool density 19.5 kg/m³

min 40 mm thk gyvlon screed or 65 mm sand/cement screed

Isolating foam layer

Timber sole plate

Acoustic edge strip

Jointing tape

10 mm isolation layer

Grouted void between p.c. units

Acoustic quilt

150 or 200 mm hollow core floor units (min. 300 kg/m²)

Gypsum based ceiling board (min. 8 kg/m²) suspended from a proprietary support system

Mineral wool packing

Two layers of gypsum based board 22 kg/m² (total) encasement to steelwork. Restrained via proprietary framing system

Deflection head detail

150 pc units / 200 pc units / 150 min. / 100 min.

Figure 4.19 *Acoustic detailing of an external wall junction with a separating floor (precast unit spanning perpendicular to separating wall)*

Light steel insulation wall with two layers of gypsum based board 22 kg.m² (total)

Insulation

Jointing tape

min 40 mm thk gyvlon screed or 65 mm sand/cement screed

Isolating foam layer

Breather membrane and rigid insulation

Acoustic edge strip

Timber sole plate

10 mm isolation layer

Flexible cavity closer with cavity tray over

External cladding

150 or 200 mm hollow core floor units (min. 300 kg/m²)

Gypsum based ceiling board (min. 8 kg/m²) suspended from a proprietary support system

Mineral wool packer

Two layers of gypsum based board 22 kg/m² (total) encasement to steelwork. Restrained via proprietary framing system

Deflection head detail

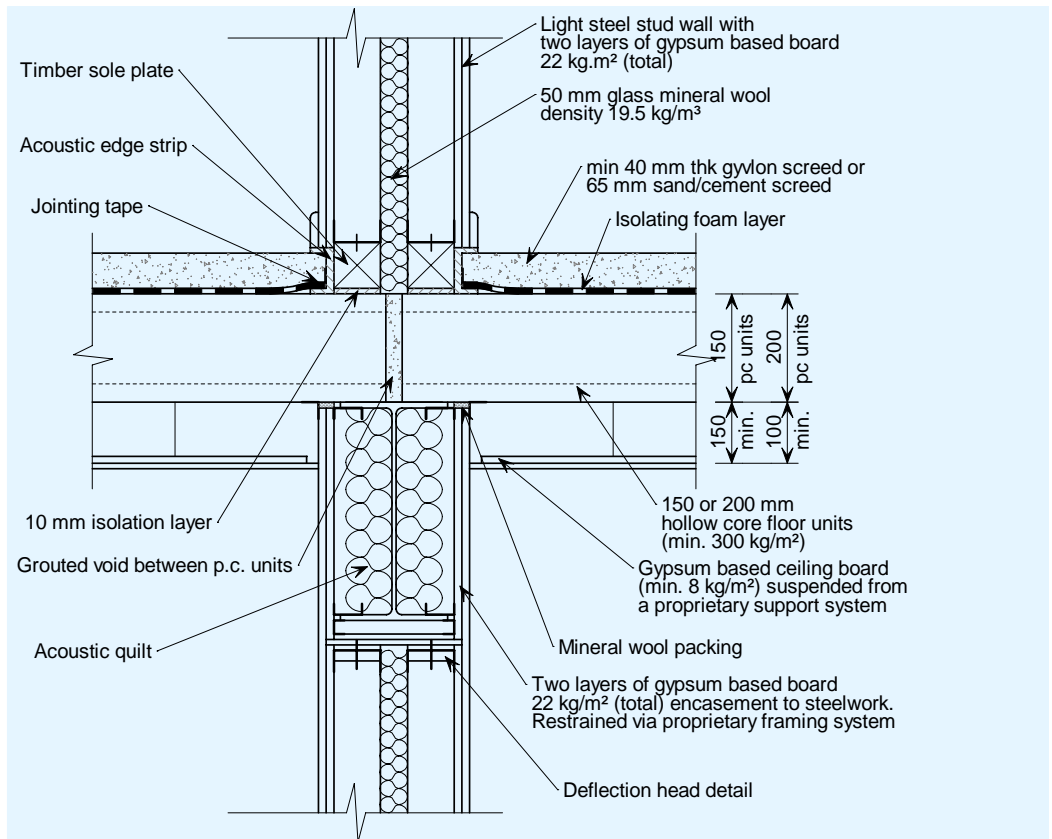150 pc units / 200 pc units / 150 min. / 100 min.

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Figure 4.20 *Acoustic detailing of a separating wall junction with a separating floor (precast unit spanning parallel to separating wall)*
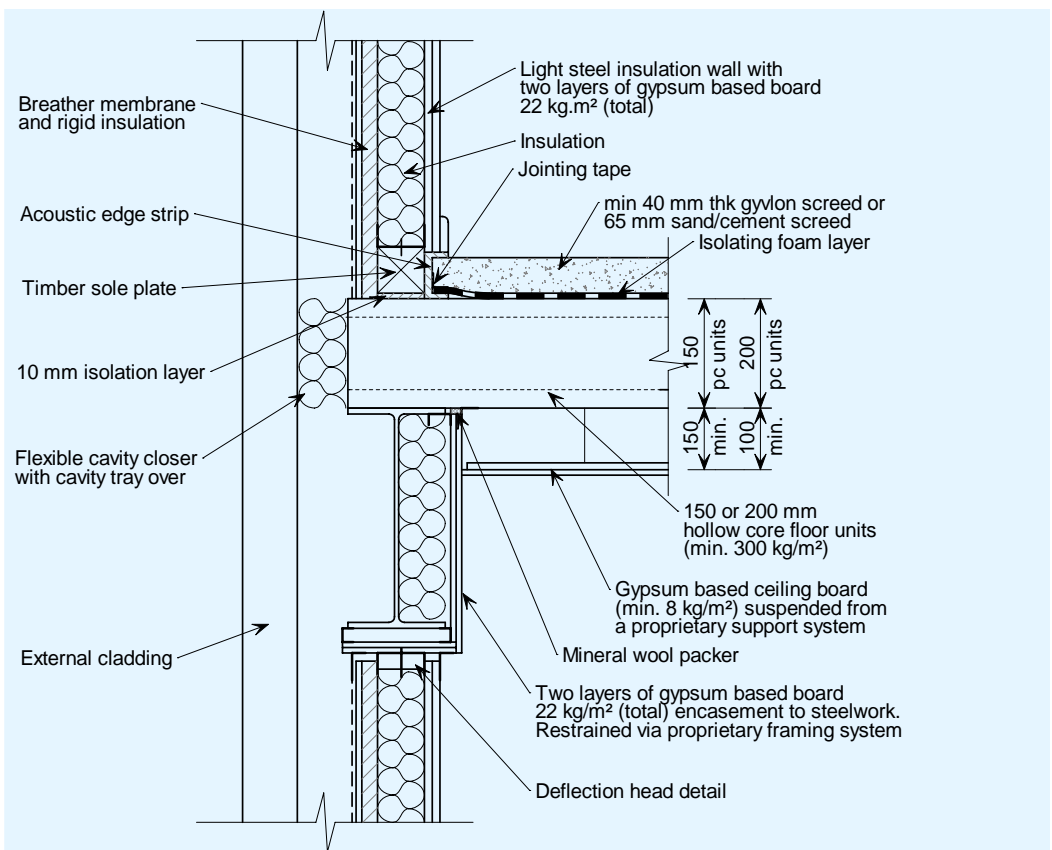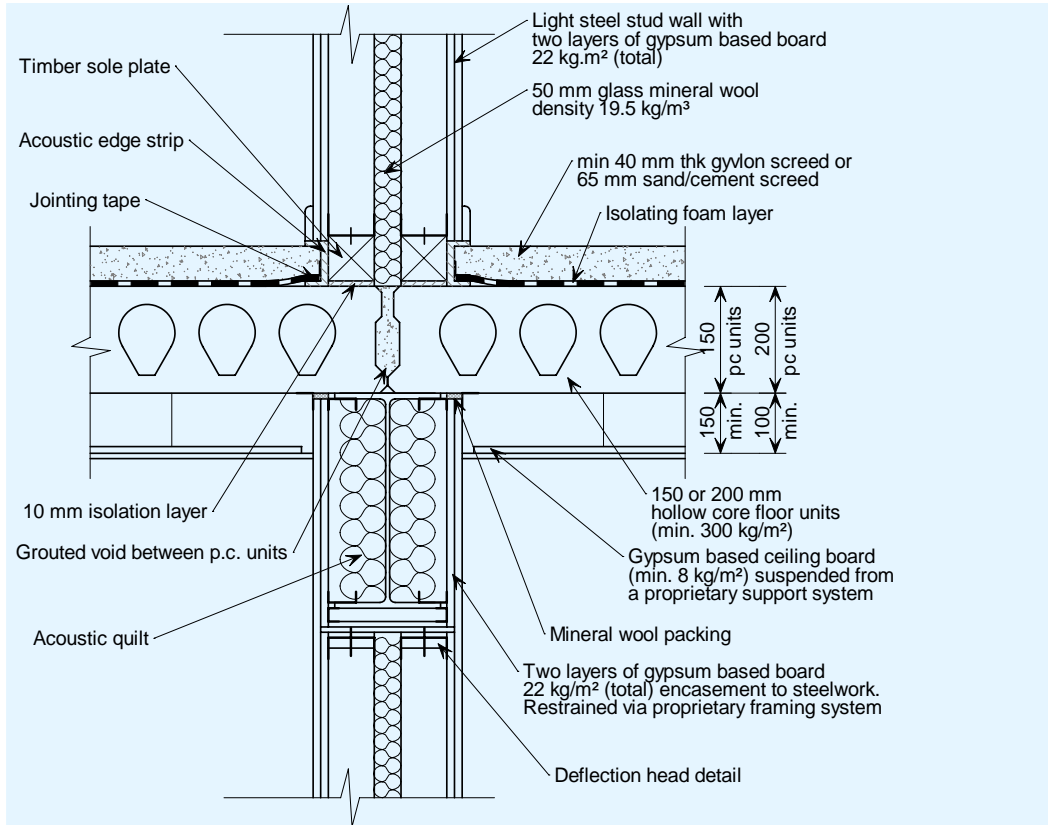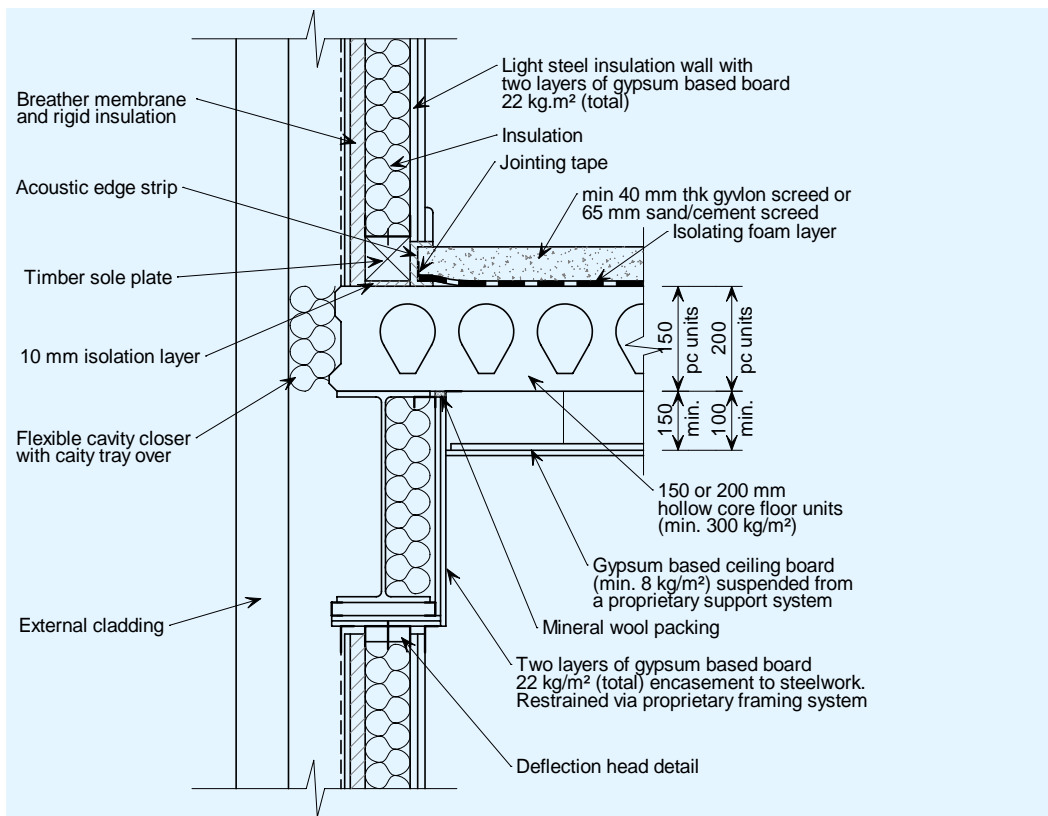

Figure 4.21 *Acoustic detailing of an external wall junction with a separating floor (precast unit spanning parallel to separating wall)*

4.3.2   Health care buildings

The acoustic requirements for health care buildings are given in HTM 2045 *Acoustics - Design considerations*[29] and HTM 56 *Partitions*[30].

The level of airborne sound insulation provided by walls and floors in health care buildings depends on the relative uses of the adjacent spaces.   In most situations a weighted sound reduction index (R_w) of at least 52 dB will be acceptable. R_w is a laboratory measured property; it does not include any allowance for flanking transmission.

For impact sound, the arithmetic mean of $L'_{nT,w}$ from all tests should not be greater than 61 dB and no individual value of $L'_{nT,w}$ greater than 65 dB. $L'_{nT,w}$ is the same performance measurement term that is used in Approved Document E for residential buildings.

For health care buildings, screeded floors with bonded vinyl or rubber coverings are popular.   These provide durability, hygiene, ease of maintenance and slip resistance combined with sound absorption.

HTM 2045 includes recommendations for the maximum amount of noise from mechanical services and intrusive external noise that is permitted in certain areas depending on the use.   These requirements need to be considered in the specification of the external walls and their junction details.

The maximum reverberation time of sound in certain rooms is subject to a requirement in HTM 2045.   The maximum allowable reverberation time for a specific room depends on its volume and function.   This requirement does not significantly affect the structural aspects of the building, as internal surface finishes can be selected to achieve the required reverberation time.

4.3.3   Educational buildings

The acoustic requirements for education buildings are given in Building Bulletin 93 (BB93)[28], which is referred to directly in Part E of the Building Regulations for England and Wales.

As for health care buildings, the level of airborne sound insulation required between adjacent spaces depends on the relative uses of the adjacent spaces. The requirement can vary between 30 and 60 dB for a minimum $D_{nT(Tmf,max),w}$. For walls between classrooms $D_{nT(Tmf,max),w}$ should be at least 45 dB.   The term $D_{nT(Tmf,max),w}$ is similar to $D_{nT,w}$ used for residential buildings except that the reference reverberation time is different.

The impact sound requirement for floors depends on the use of the space.   The maximum allowable value of $L'_{nT(Tmf,max),w}$ varies between 55 and 65 dB depending on the use of the space.

BB93 also has requirements for speech transmission and limits for sound reverberation in school buildings.   However, these aspects are largely unaffected by the structural aspects of the building. The, room shape, layout of the rooms, open plan areas and surface finishes are critical factors for speech transmission and sound reverberation.

4.4   Thermal insulation and energy efficiency

Thermal insulation and energy efficiency are covered by Part L of the Building Regulations for England & Wales *Conservation of Fuel and Power*[26].   Part L1A is for new dwellings and Part L2A is for new buildings other than dwellings. The 2006 version of Part L has five conditions for compliance:

1.   Target CO_2 Emission Rate (TER)

2.   Minimum standards for building fabric and services

3.   Limiting summer solar gain

4.   Demonstrating actual performance (testing)

5.   Providing information to users.

The first condition requires energy calculations to be performed; this allows considerable design flexibility. To prove compliance in design a calculation of the Building CO_2 Emission Rate (BER) for a proposed building is compared with a calculation of Target CO_2 Emissions Rate (TER).

The second condition limits the amount of design flexibility by imposing minimum performance standards.   Designers must consider U-values, air permeability and building services efficiency. For ground floors, the maximum U-value is 0.25 W/m^2K.   The maximum permitted air leakage of buildings is generally 10 $m^3/hour/m^2$ (at a pressure differential of 50 Pa).

The third condition requires that buildings should be constructed so that naturally ventilated spaces do not overheat when subject to a moderate level of heat gain and mechanically ventilated or cooled spaces do not require excessive cooling plant.

For the fourth condition, the quality of construction must be demonstrated by a final calculation that the BER still meets the TER.   The calculations must include field measured figures from air leakage testing of the building and commissioning of building services, together with any changes in the final construction.

For the fifth and final condition at the start and end of the design-build-test sequence, the Building Control Body will require reports that the BER is no higher than the TER calculated. The final report should include site check lists, air tightness test certificates and building services test certificates.

4.4.1   Ground floor slabs

Insulated hollow core units can be used for ground floor units and provide U-values of 0.22 W/m^2K or better for floors.   The insulated units usually consist of expanded polystyrene (EPS) bonded to the soffit of hollow core units and specially formed end and side bearing points to provide support where required.
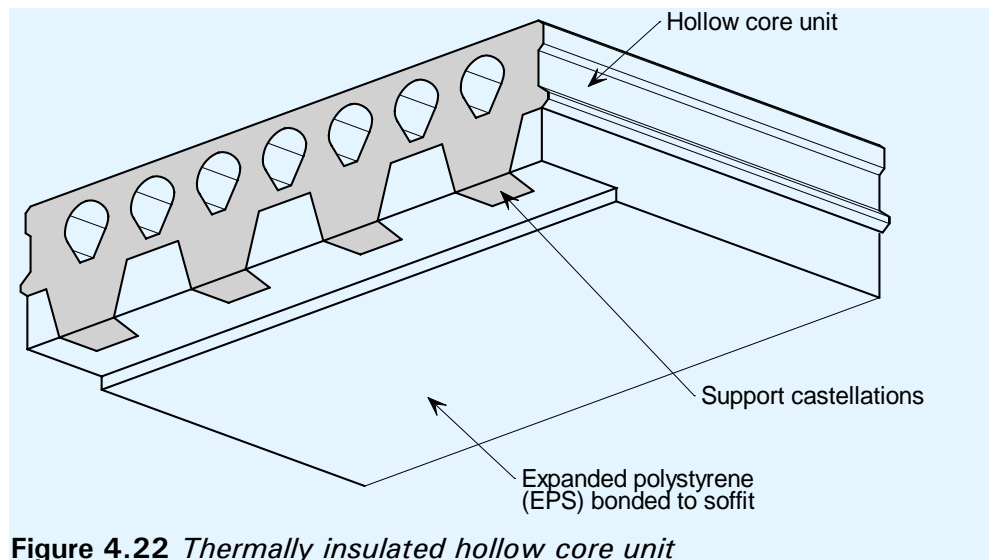
Figure 4.22 *Thermally insulated hollow core unit*

Ground floor junction details

Accredited Construction Details (ACD's) have been developed for a range of construction materials, including precast floor units. The purpose of the details is to assist the construction industry achieve the performance standards required to demonstrate compliance with the energy efficiency requirements (Part L) of the Building Regulations. The details focus on the issues of insulation continuity (minimising cold bridging) and airtightness. The details contain checklists which should be used to demonstrate compliance. The details are available from the Department for Communities and Local Government[34].

4.5 Attachments and fixings

There are a number of options available for attaching ceiling battens, service hangers and other elements to the underside of precast floors in steel frame buildings. It is important that prestressing tendons in the units are not damaged during the installation of the fixings. Information on the location of the prestressing tendons and suitable fixing positions should be obtained from the precast manufacturer. Generally, suitable fixing positions are along the centreline of the hollow cores.

Various proprietary products are available for making fixings to precast floor units and, used in accordance with the manufacturers' instructions, are simple and economical. The fixing methods include self tapping screws, toggle bolts, expanding anchor systems and chemical mortar anchors all of which are shown in Figure 4.23. The fixing method selected must be appropriate for the load that it is required to carry. Manufacturers of the fixings generally have carried out testing of their products and publish load tables. Capacities typically vary from 0.7 kN to 1.6 kN in 150 to 260 mm deep units and from 2.9 kN to 4.9 kN in 300 to 450 mm deep units.

Shot fired connectors should not be used.

A self tapping screw fastening is made straight into the concrete without using a separate plug. A hole is drilled into the concrete unit and the screw is driven straight into the hole. The self tapping screws are available with a range of head types for different applications. Flat head screws are generally used for fixing timber battens and other soft materials, hexagon head screws are used for

fixing light metal elements and screws with heads with threaded connection can be used for fastening light duty pipe rings. Self tapping screws come in a range of lengths to allow for elements of various thicknesses to be fastened and to ensure that the correct embedment (generally 30 – 40 mm) is achieved.

Toggle bolts (Figure 4.23b) are installed through a hole drilled in the precast unit directly below a core. It is important that the hole is drilled centrally to the core. After the toggle bolt has been installed, it should be secured into position and the hole closed with a nut and plate washer.

Expanding anchor systems usually have an internal thread suitable for M6, M8 or M10 fixings. The anchor is placed in a hole drilled in the precast unit. The method by which the fixing is secured in position depends on the product; either a central expansion cone is driven into the anchor with a purpose designed setting punch or a central cone expands the fixing as it is tightened (as shown in Figure 4.23c).

Chemical anchoring systems usually consist of a specially formulated chemical or resin mortar, a gauze sleeve and a fastening insert, which can be either an anchor rod or a threaded socket. A hole is drilled into the unit penetrating the hollow core. The gauze sleeve is placed in the drilled hole and filled with the mortar. The fastening element is then pushed into the mortar, which displaces some mortar through the gauze to form a mechanical key into the hollow section (see Figure 4.23d). The resin mortar must be allowed to cure before the fixing is loaded.

Guidance regarding the integration of services with structures is provided in SCI/BSRIA publications IEP2 *Services co-ordination with structural beams* [35] and IEP4 *Supporting services from structure* [36].
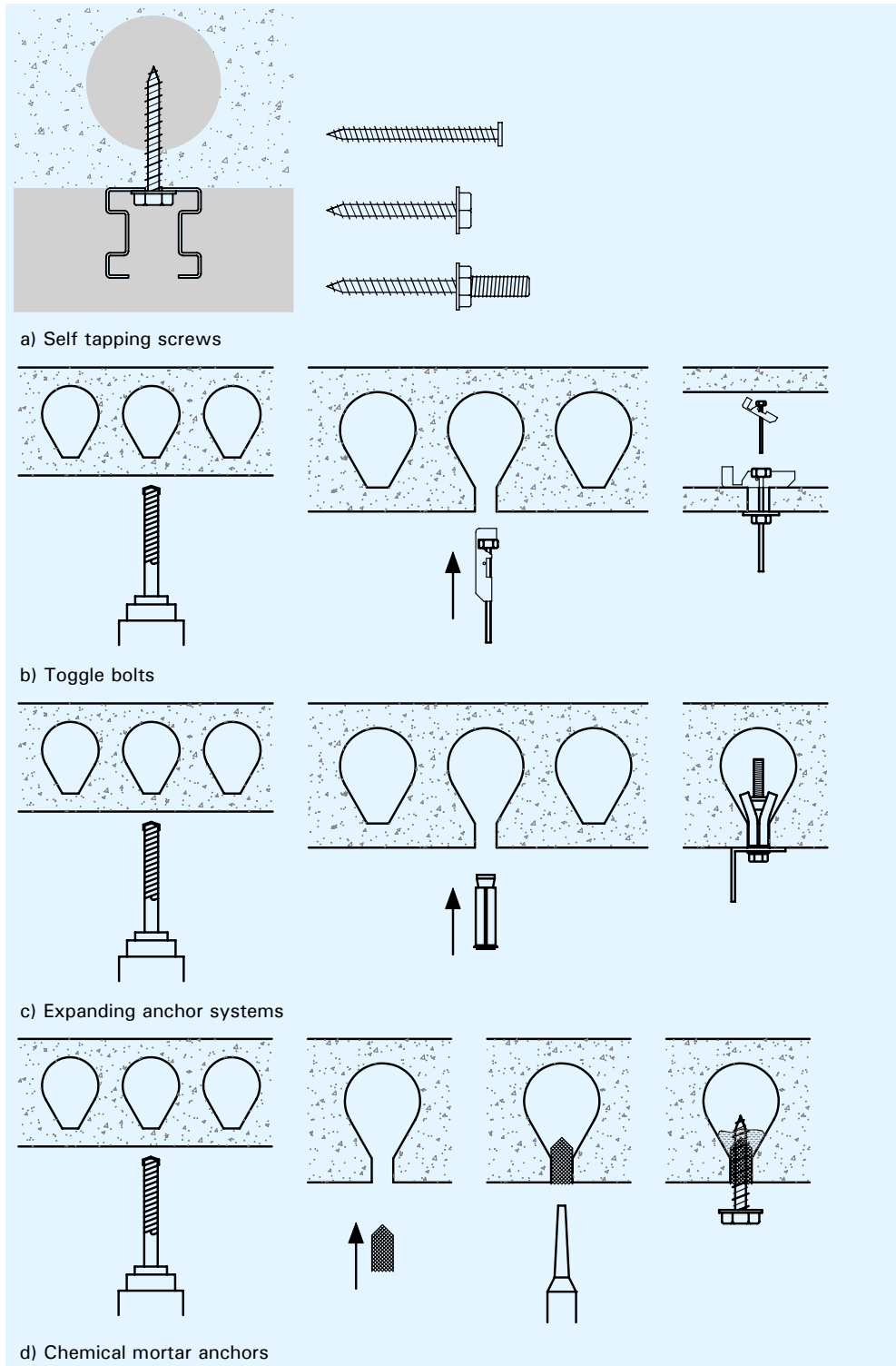
a) Self tapping screws

b) Toggle bolts

c) Expanding anchor systems

d) Chemical mortar anchors

Figure 4.23 *Fixings to precast units*

5 INSTALLATION AND ERECTION

5.1 Erection sequence

Erection of steel frames with precast floor units is a fast and efficient method of construction. Careful sequencing of activities is required to make the most of the potential benefits.

The ideal erection sequence for steel frame buildings with precast hollow core floor units is described below. However, factors such as site constraints may make other sequences more appropriate.

(a) Erection of steelwork to first splice level. This is usually just above the second or third floor level. The braced bay should preferably be constructed first to provide stability to the remainder of the structure. Generally, shear studs will be welded to the beams off-site during fabrication.

(b) Establish edge protection around the perimeter of the structure and a fall arrest system for the internal area. Possible fall arrest systems include nets and inflatable air mats. Figure 5.1 shows building edge protection and a net fall arrest system installed.

Figure 5.1 *Edge protection and net fall arrest system*

(c) Crane precast floor units into position using integrated lifting hooks as shown in Figure 5.2. The units should be positioned on to the lowest floor first and then progress upwards, to avoid creating obstructions to lowering units into position by crane. The sequence in which units are positioned onto supporting steelwork should be commensurate with unbalanced loading conditions assumed at the design stage. Section 5.3 discusses the issues associated with the temporary condition and unbalanced loading.

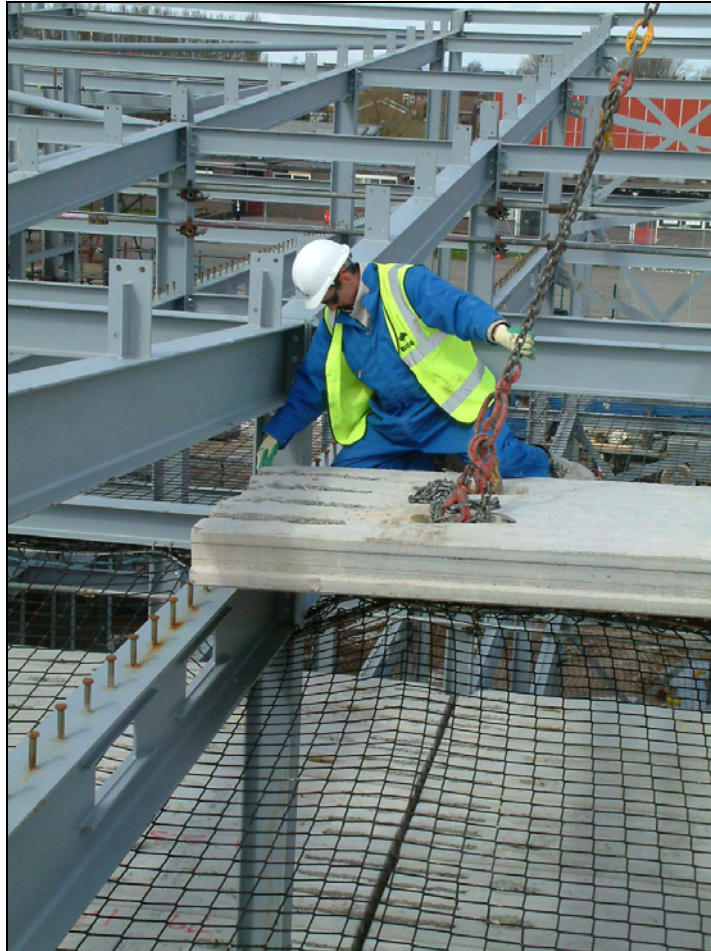(d) Any precast infill pieces that are required should be positioned in gaps between units.

Figure 5.2 *Placing of precast units*

(e) Eliminate differential cambers if the soffit is to be exposed in the final condition (e.g. car parks). Adjacent units may have differential cambers due to natural variations. Differential cambers can be eliminated by applying a temporary load to the unit prior to the joint being grouted.

(f) Place grout in joints between adjacent floor units and around columns. The joints between the units should first be 'wetted' and then in-filled with a low-shrinkage workable concrete (75-100 mm slump) as soon as is possible after erection of units to avoid accumulation of rubbish/debris in the joints. The concrete should have a minimum cement content of 290 kg/m³ and should be well compacted into the joints.

Note: Generally, the floor should not be used for heavy construction plant e.g. motorised mobile access platforms and concrete skips until the joints have been grouted and the grout has cured. Where it is unavoidable to place heavy construction loads on the units before the grout has cured, the units should be propped locally to prevent differential movement. All heavy loading should be avoided until the joint concrete has cured for a minimum of three days. In any case the designed loading of the floor must not be exceeded.

(g) Where required, position transverse reinforcing bar in open cores across internal supports and U-shaped reinforcing bar at edge beams to tie the slab to the frame. Loose debris must be removed from opened cores. If pre-welded shear studs have not been used, shear studs should be welded

on site or bars must be site welded or passed through pre-drilled holes in the beam webs (see Section 4.2.3).

(h) Fix edge trims to perimeter beams to form a permanent shutter for the in-situ concrete.

(i) If specified, place reinforcement, pour in-situ structural topping and level.

(j) Start sequence again with erection of steelwork up to the next splice level.

Further guidance and advice concerning the erection of precast flooring is provided in the Precast Flooring Federation *Code of Practice for the Safe Erection of Precast Concrete Flooring*[37].

5.2 Treatment of precast units on site

5.2.1 Storage and handling

Handling

Careful handling and close supervision are required during offloading, hoisting and fixing of units to prevent damage. Units must never be rolled over or placed upside down because they will crack and break. A fork-lift truck or any other method that supports the units near the centre should never be used to lift prestressed units because this will cause them to crack and break.

Stacking

Stacking of units on site will not be necessary in all cases as the units can often be unloaded and erected into their final position straight from the delivery vehicle. However, where stacking on site is necessary, the following guidance should be followed.

Where possible, the units should be stacked flat in the same arrangement that they are transported to site. The stacking area should be firm, level ground and a bottom row of sleepers should be carefully levelled in. Where battens are used they should be placed vertically one above the other and as near to the ends as possible, see Figure 5.3. Separate stacks should be built for units of different lengths. The ground conditions will influence how many units can be stacked vertically. However, stacks higher than approximately 1.6 m are not recommended[37].

The stacking of stair units is generally more complex than flat units. Where possible, the units should be stacked flat in the same arrangement that they are transported to site. Battens and spacers should be used to ensure units are level and vertically above each other. Separate stacks should be formed for units of different length or shape, see Figure 5.4 (a) and (b). Battens should be as near to the ends of the units as possible and, where stair units incorporate bottom landings of 1200 mm or longer, additional support should be provided, as shown in Figure 5.4 (c).

5.2.2 Lifting and positioning

Lifting

The units should normally be hoisted flat by crane using all the lifting points in the top surface provided for that purpose. Precast units should generally be lifted using a four leg chain with a yoke or boom to ensure that there is equal distribution of load. Prior to lifting, safety chains should be attached to the

lifting chains and secured close to the underside of the unit as shown in Figure 5.5. The safety chains should only be released immediately before the unit is placed in its final position. The lifting chains must be of sufficient length that the angle between the pairs of chain legs is not greater than 90° to avoid over-stressing the lifting points. If using slings, they must not be more than four times the unit depth from the ends of the unit. Part width units may only have two lifting points cast in but these can be offset such that the unit can be lifted level (Figure 5.6).

When using lifting scissors to place units, care should be taken to ensure that these are suspended from a boom and that they are placed close to the end of the pre-cast unit (i.e. within four times the unit depth from the ends of the unit). Generally, it is not recommend that lifting scissors are used for units greater than 250 mm deep.

As for all crane operations, precast units should not be lifted during periods of high wind speed.

Precast stair units should only be lifted from the built-in lifting points.

Figure 5.3 *Stacking and storage of units on site*

4 No. levelling blocks
per layer, cut from 100 x 500 mm

a) Units with no landing

Battens and spacers as near
to ends of units as possible

b) Units with top landing

Support for long landings
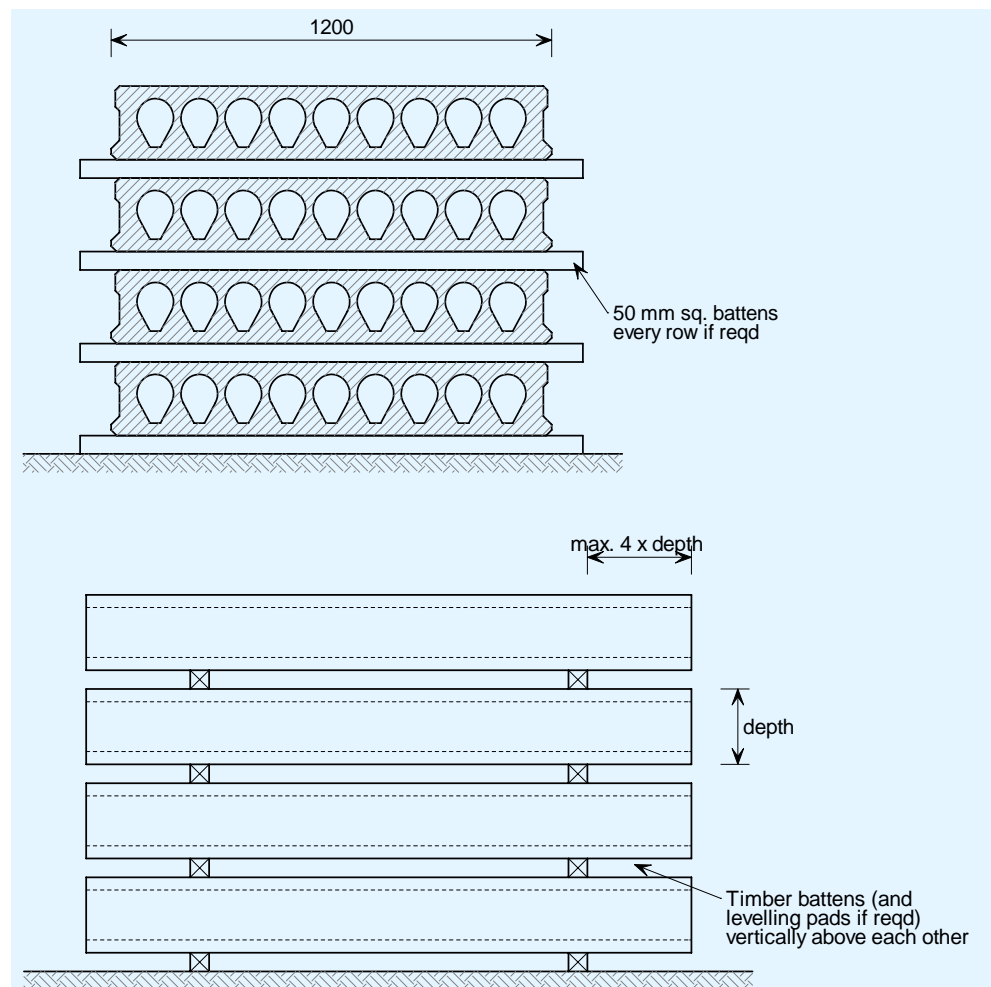
c) Units with bottom landing

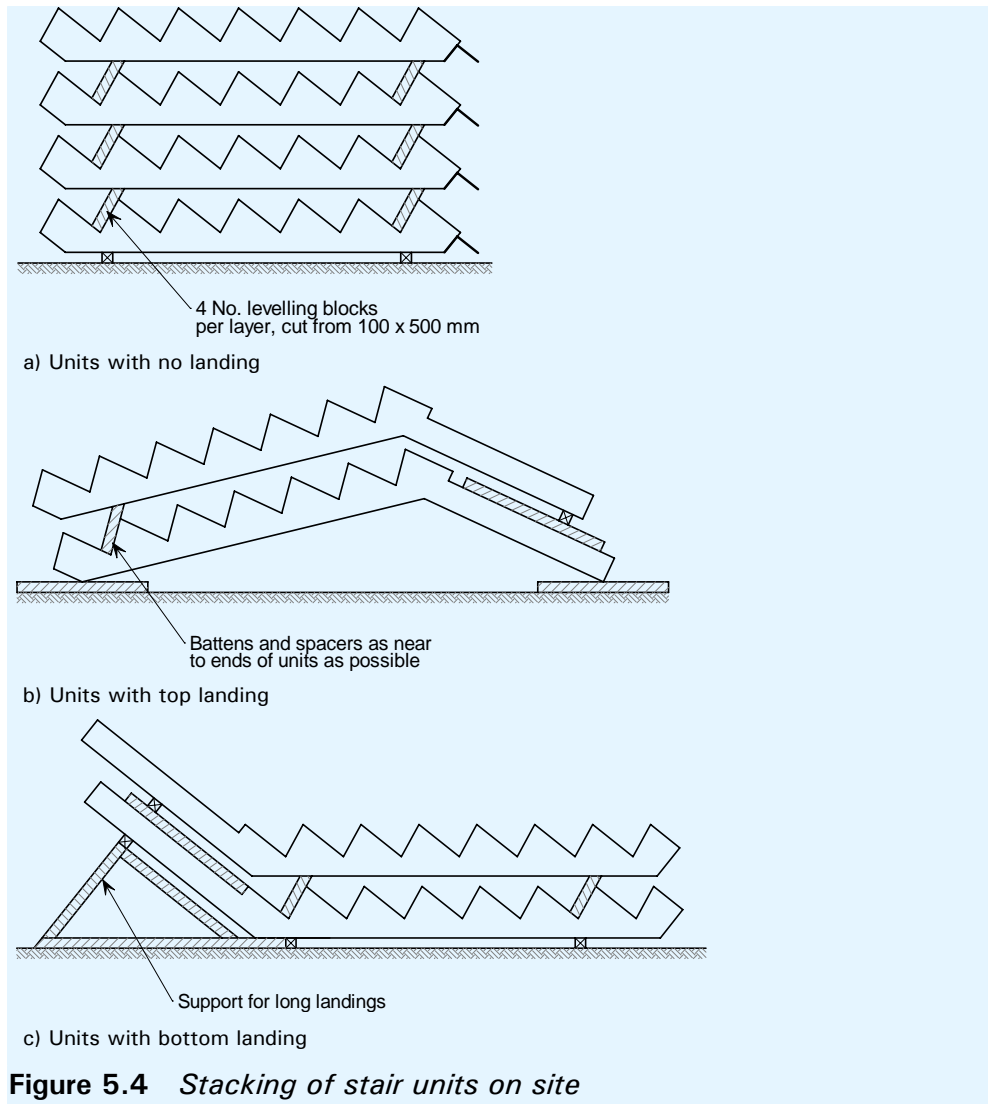**Figure 5.4** *Stacking of stair units on site*
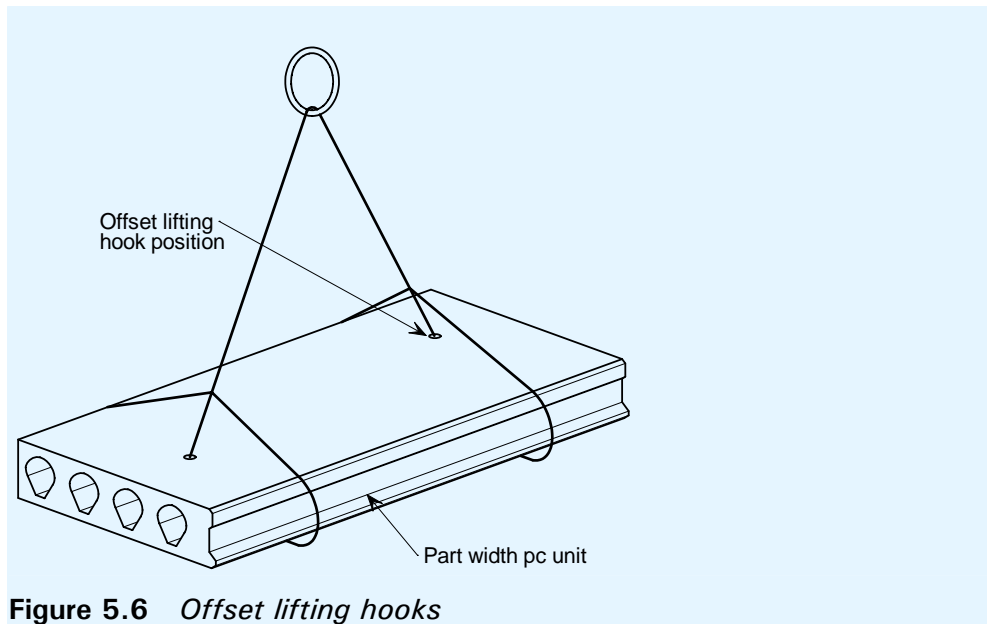

Figure 5.5 *Lifting of units*

Figure 5.6 *Offset lifting hooks*

Positioning

The units should be placed side by side with the edges of the units touching. The bearings (i.e. the steel beam flanges or shelf angles) should be clean, level and even to prevent damage to the units. The units should be centralised on their supports to ensure there is an equal bearing width at each end.

Lifting of precast units with integrated lifting hooks rather than slings means that a unit can be landed in its final position adjacent to the previous unit. Then, there is no need for barring to manually shift the units in to their final position.

The construction details should include allowances for the positioning of the precast units, taking due account of construction and manufacturing tolerances (see Section 1.4.2).

5.2.3   Alterations to precast units

Precast units should not be altered on site without permission from the unit manufacturer. Altering the product may compromise its structural integrity and lead to failure.

Where permission for alterations has been acquired, diamond core drills should be used to form holes. Alternatively, a light percussive rotary drill is suitable provided that cutting commences from the underside of the units (to avoid excessive spalling of the soffit). In any case, pneumatic and other heavy percussion drills must not be used. Permitted alterations should be carried out on a flat stable platform, not *in situ* supported on the steel frame. Small holes, usually up to 65 mm diameter, can usually be cut through on site at core positions or joints. Typically, a minimum of 300 mm of unit should remain between adjacent holes. However, coring holes on site is approximately 10 times more costly than factory formed holes.

5.2.4   Weep holes

Water can potentially accumulate within the cores of precast units due to exposure during the construction programme. Weep holes drilled in the underside of the unit are used to prevent water accumulation. Weep holes can

be formed during manufacture (if specified) or drilled on site by the general contractor with the permission of the manufacturer, taking care to ensure the holes coincide with the hollow cores. The storage and treatment of units on site should ensure that the weep holes are kept clear.

Wherever there is a construction detail that effectively produces a solid end to hollow core units, weep holes should be specified. Weep holes can be formed by special machines in the factory using drills which are set up on a jig to suit the core centres and drill from the underside of the unit.

Weep holes are usually 10 mm diameter approximately 750 mm back from the end of the unit.

5.3   Stability during construction

5.3.1   Frame stability

The stability of the frame during construction can be ensured by constructing the system that is used for providing lateral and longitudinal stability to the building in the permanent condition and then erecting the frame from this stable part. The system for providing lateral and longitudinal stability will commonly be triangulated bracing or a concrete core. If it is not possible to construct the braced bay or concrete core first, temporary vertical bracing will need to be provided.

5.3.2   Temporary condition for beams

The stability of the steel beams during the erection of the floor units, and the placement of the structural topping, must be considered by the designer who should take due account of the floor erection process. The construction will usually require erection in bays, to avoid excessive re-location of the crane. Hence, most beams, at some stage during construction, will be subject to unbalanced loading and will not be laterally restrained. Beams will be subject to combined bending and torsion and will be susceptible to lateral torsional buckling. If a particular sequence of erection or temporary support is 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. Edge beams and beams around openings 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 of SCI publication P287[3] and is included in Example 2 (Appendix B.2) of this publication. When the units are supported above the shear centre of the beam (e.g. on the top flange), the buckling resistance moment should be checked for a destabilising load condition. Therefore, the effective length for lateral torsional buckling should be taken as the length between lateral restraints × 1.2.

Temporary stability to resist unbalanced loading can be achieved by placing ties between the compression flanges of the beams, as shown in Figure 5.7 (a), at a maximum spacing of 40 times the beam flange width for UB sections. Ties between the tension flanges are insufficient to prevent torsion unless combined with a U-frame or other measures, see Figure 5.7 (b). Structural calculations should be performed to ensure the adequacy of the temporary bracing.
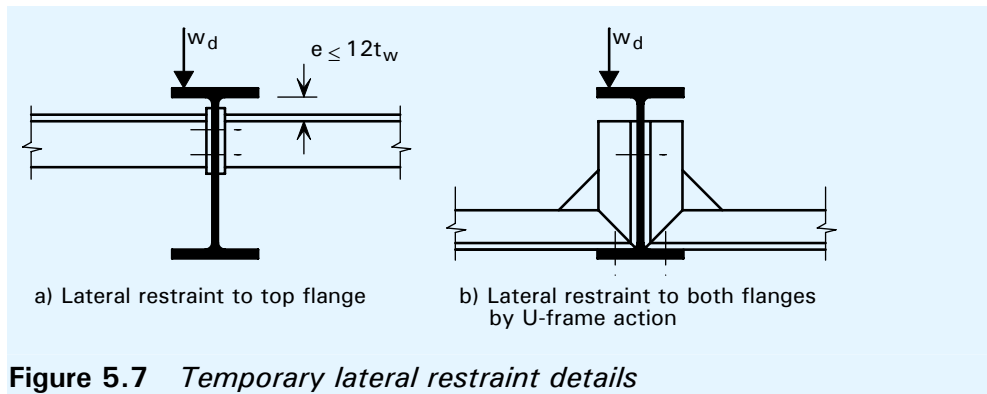
Figure 5.7 *Temporary lateral restraint details*

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 the precast unit bearing width[38]. The restraint is provided by the restoring moment of the unit and the friction between the unit and beam (as illustrated in Figure 5.8). Based on a typical bearing width of say 50 mm, beams with spans up to 8 m may be assumed to be fully laterally restrained by the precast units. However, restraint to beams with a span over 8 m should not be relied upon and they should be designed as unrestrained beams.

For cases when there is a large gap between the ends of the hollow core units (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 minimise the possibility of accidental damage arising from the installation of the adjacent unit.
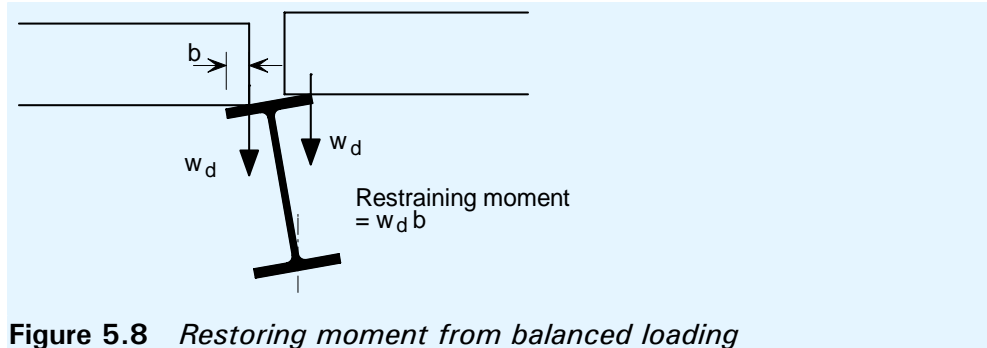

Figure 5.8 *Restoring moment from balanced loading*

For edge beams or beams with non-symmetric loading, the restoring moment may be absent. Therefore, restraint should not be relied upon and they should be designed as unrestrained beams. Also, it may be necessary to consider the effects of torsion on the design of edge beams (irrespective of the requirements for lateral stability). For guidance on the design of members subject to combined bending and torsion refer to the SCI publication P057[39]. A special case is where the units are supported by the entire width of the flange. In this case lateral restraint is provided, and torsional effects from the floor loading may be ignored. This is a practical rule based on development of restoring moments if torsional movements occur.

5.3.3 Long span beams

As discussed above, beam spans greater than 8 m cannot be considered as being fully laterally restrained by the precast units during the construction stage.

To give a structure with unimpeded vehicle movement, beam spans in car parks are commonly about 16 m. Figure 5.9 show a typical framing arrangement for a steel framed car park with precast units. The temporary bracing required for the construction stage is shown. Lateral restraints are provided at the centre of the long span beams which are then braced back to the columns in braced bay of the frame. Figure 5.10 shows the temporary bracing (coloured yellow) used in the construction of a car park.
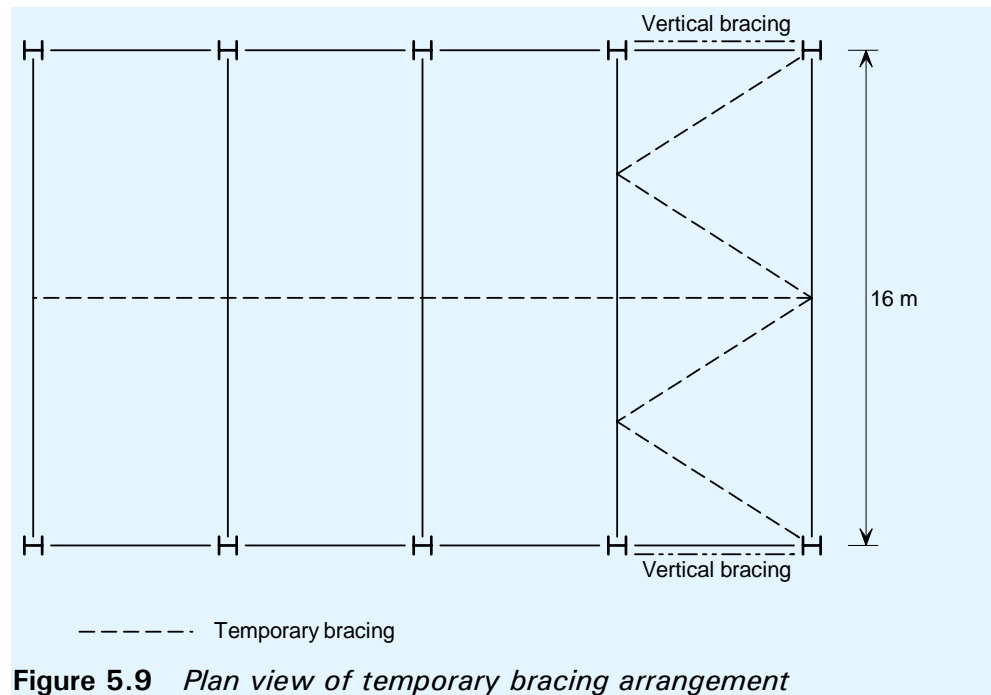

Figure 5.9 *Plan view of temporary bracing arrangement*

Figure 5.10 *Temporary bracing for long span beams*

To be effective the lateral restraint needs to be located close (i.e. within a distance of 12 times the web thickness) to the compression flange of the beam.

Figure 5.11 *Temporary lateral restraint detail to compression flange*

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APPENDIX A Case Studies

The combination of steel frames and precast concrete floors produce an effective and economic solution for a wide range of building types. The versatility of this structural combination is demonstrated in the case studies in Sections A.1 to A.6.

A.1 Multi-storey residential

Figure A.1 shows a multi-storey residential development using precast floors in a steel frame. The development used 1500 m^2 of 200 mm deep hollow core floor units and 29 m^3 of precast stair units as shown in Figure A.2. The development is located in Walsall, Birmingham.

The frame requirements were to provide a flat soffit between downstand beams which were at separating wall positions. Hollow core floor units spanning 6.1 m provided the uninterrupted flat soffit required. Utilising composite action between the hollow core slabs and the steel beams enabled significant savings on the steel tonnage. A tested sound insulation solution was provided by Bison Concrete.

Figure A.1 *Multi-storey residential development, Walsall, Birmingham*

Figure A.2   *Precast stair units in multi-storey steel frame*

A.2   Commercial buildings

Figure A.3 shows the British Gas offices at Solihull, West Midlands.   The construction of the building uses a steel frame with precast hollow core floor units.   The structural form consisted of 7.5×8.7 m bays with 200 mm precast hollow core units spanning 7.5 m.   A total of 9,100 m^2 of precast units were used and 40 m^3 of precast stair units.

No structural topping was used on the units although the steel beams were designed as composite with the precast floor to reduce the required depth of beam.   A raised access floor and suspended ceiling were installed.   The total office width is 17.4 m with a central concrete core.

Figure A.3   *British Gas office at Solihull, West Midlands*

A.3   Retail premises

Ikea, Manchester

Figure A.4 shows the Ikea development in Manchester. This retail outlet is constructed using a steel frame with precast unit floors. Over 40,000 m² of 200 mm deep hollow core precast floor units were used in the construction (Figure A.5).

The column grid is 8 x 16 m with the precast units spanning 8 m. The grid reflected the mixed use of car park, retail and offices on different floors. The regular grid allowed the use of repeatability in the type of floor units that were used. The steel beams are composite with the floor units which have 75 mm structural topping applied.

Figure A.4   Ikea, Manchester

Figure A.5   Construction of the floor at Ikea, Manchester

Drake Circus

The Drake Circus shopping centre, Plymouth is shown in Figure A.6. The steel frames support almost 19,000 m^2 of precast floor units. A combination of 200, 260, 350, 400 and 450 mm deep precast hollow core floor units were used. The placement of which is shown in Figure A.7.

The structural form required precast unit spans varying from 12.0 m in the retail areas to 15.6 m in the car park. In some areas the column centres dictated smaller spans which only required shallow floor units. The long spans and durability meant that hollow core units were the obvious choice. In the retail areas the hollow core units were left without a topping to allow the retailers the choice during fit out. This also reduced the wet trades required. The car park specification required flat unobstructed soffits and 400 mm deep hollow core units with 75 mm structural topping that provided a simple and elegant solution.

Figure A.6   *Drake Circus shopping centre, Plymouth*

Figure A.7   *Precast floor units at Drake Circus shopping centre, Plymouth*

A.4 Hospitals

In Exeter, a new hospital was built using a steel frame with precast floors (Figure A.8). The construction used 4,250 m^2 of 200 and 300 mm deep hollow core floor units (Figure A.9).

Figure A.8 *Hospital, Exeter*

The use of the floor plate dictated a variety of spans for the precast units and the supporting steel frame. Hollow core units provided the flexibility for the variety of span lengths and an optimized design for the applied loads. Precast units were used to span up to 9 m. A mixture of support types were used in the construction including shelf angle beams and downstand beams. The floor units were tied into a concrete core using embedded reinforcing bars. A 75 mm structural topping was applied which helped to ensure the dynamic behaviour of the floor was within acceptable limits.

Figure A.9 *Placing units during construction*

A.5 Car Parks

Figure A.10 shows a multi-storey steel framed car park with precast concrete floors. The car park is situated in Nuneaton, West Midlands and has 10,000 m^2 of 150 mm deep floor units and 67 m^3 of precast stair units. The top deck of the car park during construction is shown in Figure A.11.

Precast hollow core floors where chosen as the most economic and practical flooring solution to comply with the client's requirements. The bay centres are 15.6 m $\times$ 7.2 m. The floor system chosen was a composite floor consisting of 150 mm prestressed hollow core units and 75 mm structural topping. The use of precast concrete with a supporting steel frame provides excellent durability performance in the car park environment.

Figure A.10 *Multi-storey car park, Nuneaton*

Figure A.11 *Car park deck during construction*

A.6   Stadia

Millennium Stadium, Cardiff

The Millennium Stadium in Cardiff (shown in Figure A.12) used over 48,000 m^2 of precast hollow core floor units and over 380 precast stair units.

The structural form of this development required that the structural grid of the concourse areas followed the structural grid of the main seating areas.   The structure of the Millennium Stadium is a true bowl with the main structural members splayed.   Therefore, each individual hollow core unit was required to have splayed ends and the spans of the units increased towards the outside of the building.   This true bowl arrangement was chosen to reduce the number of steel members required and to optimize the composite interaction between the hollow core units and the main steel grid.

Figure A.12 Millennium stadium, Cardiff

Burton Albion Stadium

Figure A.13 shows the Burton Albion football stadium, Burton on Trent. The construction used 1,900 m^2 of 200 mm deep hollow core floor units and 71 m^3 precast stairs.

The stadium utilised 200 mm deep hollow core units spanning 7.60 m in the concourse and hospitality areas (Figure A.14). The structural grid was fixed by the span of the terrace units and to allow for the maximum allowable distance to the access and egress position. The regular grid and the integration of the concourse and terrace areas dictated the use of hollow core to avoid excessive steel members.

Figure A.13 *Burton Albion stadium*

Figure A.14 *Burton Albion stadium during construction*

APPENDIX B   Design Examples

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![SCI logo]	Job No.	CDA 155
	Job Title	Example no. 1
	Subject	Two-storey industrial office building
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570 **CALCULATION SHEET**	Client SCI / Bison	Made by **MEB** Date **Nov 2006**
		Checked by TCC Date **Feb 2007**

B.1 Two-storey industrial office building

Introduction

The purpose of this example is to demonstrate:

  a) the design considerations for temporary stability of beams

  b) the application of the truss model for diaphragm floor action using precast floor units

  c) the structural integrity requirements for a Class 2A building.

Other aspects of the design process that would need to be considered in practice are not included in this example.

The structure is a simple construction braced frame with precast units supported on the top flange of non-composite downstand beams. There are no expansion joints within the structure. The joints between the precast units and around the columns are grouted with low shrinkage C25 concrete with 10 mm aggregate. There is no topping applied to the units. A raised floor will be installed. The dimensions of the building and column grid are shown in Figure B.1.1.
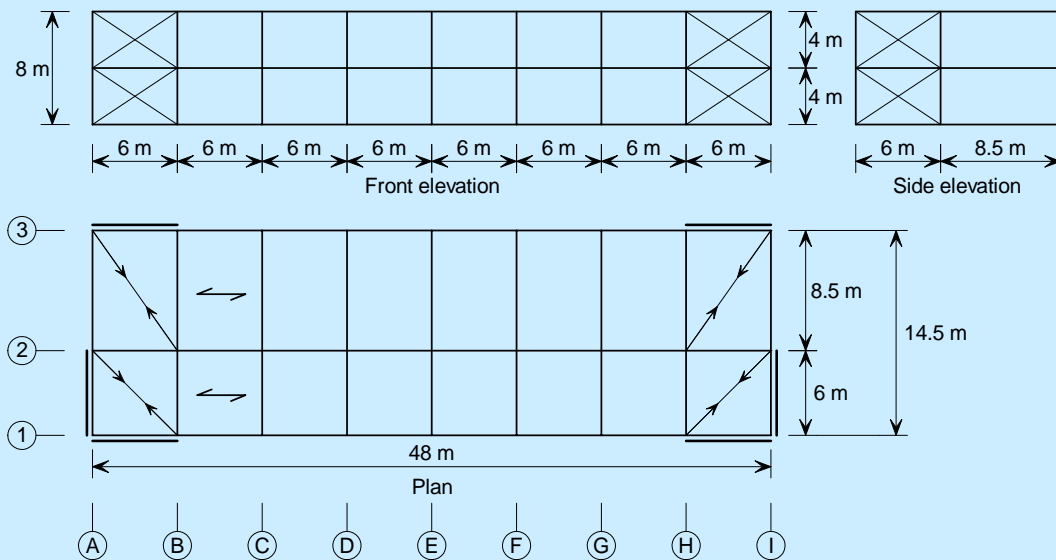

Figure B.1.1 *Building dimensions and column grid*

B.1.2 Loading

Floor loading

Dead load $\quad = 3.5$ kN/m^2 $\quad$ (this includes self weight of precast units, raised floor

and ceiling)

Imposed load $= 5.0$ kN/m^2

Roof loading

Dead load $\quad = 3.0$ kN/m^2 $\quad$ (this includes self weight of precast units)

Imposed load $= 0.6$ kN/m^2

Wind Load

Wind load $\quad = 1.1$ kN/m^2

Note: For this example, the wind load is greater than 1% of the factored dead load for all load cases. For the purposes of this example, the wind load is taken as a consistent value from all directions.

B.1.3 Precast unit selection

The maximum span of the precast units in this building is 6 m. From manufacturers load span tables for an imposed load of 5 kN/m^2, a 200 mm deep hollow core unit can span over 6 m without a topping. Therefore, select a 200 mm deep hollow core unit with a nominal width of 1200 mm and a self weight of 2.98 kN/m^2. This is a conservative precast unit selection because Table 3.3 includes an allowance of 1.5 kN/m^2 for finishes.

Table 3.3

Note: Manufacturers load span tables are only given for guidance. Final precast unit selection should be carried out in consultation with the manufacturer.

B.1.4 Steel frame members

The beams and columns for the steel frame have been selected based on the dead and imposed loads given above. The tie beams are assumed to carry a 1 m strip of floor load (and cladding load for edge ties) but are assumed to be unrestrained. The beams supporting the precast units are restrained by the precast units in the normal condition.

Columns:

$\quad$ 254 × 254 UC S275 $\quad$ for all columns

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Beams:

$\quad$ 406 × 178 × 74 UB S275 $\quad$ for 6.0 m supporting beams (e.g. Grid B1 to B2)

$\quad$ 533 × 210 × 122 UB S275 $\quad$ for 8.5 m supporting beams (e.g. Grid B2 to B3)

$\quad$ 406 × 178 × 74 UB S275 $\quad$ for 6 m tie beams (e.g. Grid A2 to B2)

Note: For this example, it is assumed that the section sizes used for edge beams will be the same sections as for the corresponding internal beams.

Bearing length

The width of the beams supporting the precast units must be wide enough to provide the minimum bearing length for the precast units. For the construction used in this example, the minimum required flange width is 155 mm. The supporting beams selected above are adequate.

Table 3.5

B.1.5 Temporary stability of beams

Internal beams

During the construction, it is generally impractical for the precast units to be placed alternately either side of the beam. Therefore, the beams will be subject to unbalanced loading and will not be laterally restrained by the precast units. The beams need to be checked from the case when precast units are in place on one side and also for the case when the precast units have been installed on both sides of the beam.

Internal beams – Loaded one side only

In this case the beams are subject to combined bending and torsion and are not laterally restrained by the precast units.

An example of how this check should be carried out is included in the Example 2.

Section B.2.6

Internal beams – Loaded both sides

Beams with spans greater than 8m should be checked for lateral stability in the temporary condition before the joints are grouted.

Section 5.3.2

The requirement that needs to be satisfied to have sufficient restraint is:

$$L \le 160f$$

P093[38]
Case 4a

where:
   L is the span of the beam
   f is the bearing length.

For the 533 × 210 × 122 UB
   L = 8.5 m

   f = (Beam width – Gap between units)/2 – allowance for tolerances

   f = (211.9 – 25)/2 – 10 = 83.5 mm,

   $160f$ = 160 × 0.0835 = 13.4 m > 8.5 m   OK

Therefore, sufficient restraint is provided to the beam in the temporary condition.

For the 406 × 178 × 74 UB
   L = 6 m

   f = (Beam width – Gap between units)/2 – allowance for tolerances

   f = (179.5 – 25)/2 – 10 = 67.3 mm, $\Rightarrow$ $160f$ = 160 × 0.067   = 10.7 m

Therefore, sufficient restraint is provided to the beam in the temporary condition.

Edge beams

This example assumes that the entire width of the edge beam flange is used to support the precast units (as shown in Figure B.1.2). In this case lateral restraint can be assumed and torsion effects from the floor loading may be ignored.
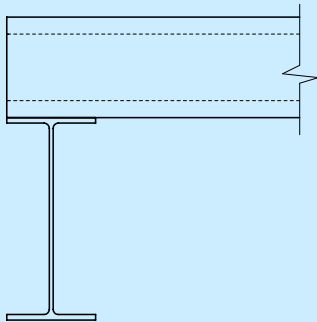
P093
Case 4b

Figure B.1.2 *Edge beam supporting precast unit on entire width*

B.1.6 Floor diaphragm

The wind loads are transferred to the vertical bracing by diaphragm action of the floor. The model used in this example is the truss model, which requires the precast units and the steel beams act together as a deep beam. For this model to be realistic, it is important that the joints between the precast units and around the columns are grouted. This prevents the diagonal length of each bay from changing significantly and thus racking of the precast units will not occur.

B.1.6.1 Edge beams

In the truss model, the edge beams which span perpendicular to the direction of the wind will carry axial forces due to the wind loads (see Figure B.1.3). The selected edge beam (406 × 178 × 74 UB S275) is checked for the axial load and bending.

Wind load on floor diaphragm = storey height × wind load = 4 × 1.1 = 4.4 kN/m
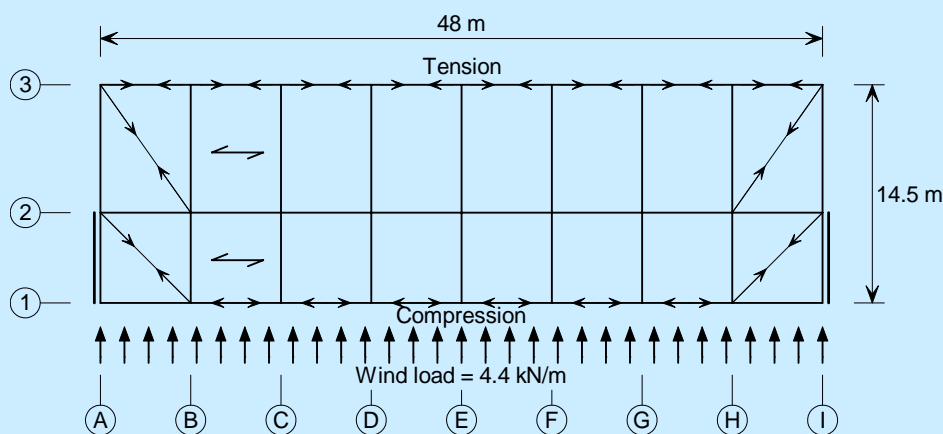

Figure B.1.3 *Truss model diaphragm action*

Load case 1: 1.4 Dead + 1.6 Imposed + Notional horizontal forces

Notional horizontal forces $= 0.005 \times$ factored vertical loads

$$= 0.005 (1.4 \times 3.5 + 1.6 \times 5) \times 14.5 = 0.94 \text{ kN/m}$$

Max moment due to NHF $= \dfrac{wL^2}{8} = \dfrac{0.94 \times 48^2}{8} = 271 \text{ kNm}$

Axial load in edge beams $= 271 / 14.5 = 18.7 \text{ kN}$

Factored gravity load on edge beam $= 1.4 \times 3.5 + 1.6 \times 5 = 12.9 \text{ kN/m}$

Max moment due to vertical load $= \dfrac{wL^2}{8} = \dfrac{12.9 \times 6^2}{8} = 58.1 \text{ kNm}$

The $406 \times 178 \times 74$ UB S275 is Class 2 compact when subject to axial load and bending.

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Combined axial and bending check

Cross-section capacity for Class 2

BS 5950-1
Cl 4.8.2.3

$M_x < M_{rx}$

$F/P_z = 18.7 / 2600 = 0.01$, $M_{rx} = 406 \text{ kNm} > M_x$ therefore section is adequate

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Page C-112

Member buckling resistance, More exact method

The effective length of the section is taken as 6 m about both the major and minor axes.

BS 5950-1
Cl 4.8.3.3.2a

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}} \left(1 + 0.5 \frac{F_c}{P_{cx}} \right) \leq 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \leq 1$$

$P_{cx} = 2490 \text{ kN}$, $P_{cy} = 715 \text{ kN}$

P202
Page C-113

$M_{cx} = 413 \text{ kNm}$, $M_b = 176 \text{ kNm}$

$m_x = 0.95$, $m_{LT} = 0.925$

BS 5950-1
Table 26
Table 18

$$\frac{18.7}{2490} + \frac{0.95 \times 58.1}{413} \left(1 + 0.5 \frac{18.7}{2490} \right) = 0.01 + 0.13(1.004) = 0.14 < 1.0$$

$$\frac{18.7}{715} + \frac{0.925 \times 58.1}{176} = 0.03 + 0.31 = 0.34 < 1.0$$

Therefore, the $406 \times 178 \times 74$ UB S275 is adequate for load case 1.

Load case 2: 1.4 Dead + 1.4 Wind

Factored wind load = 1.4 × 4.4 = 6.16 kN/m

Max moment due to wind load $= \dfrac{wL^2}{8} = \dfrac{6.16 \times 48^2}{8} = 1774$ kNm

Axial load in edge beams due to wind = 1774 / 14.5 = 122.3 kN

Factored dead load on edge beam = 1.4 × 3.5 = 4.9 kN/m

Max moment due to dead load $= \dfrac{wL^2}{8} = \dfrac{4.9 \times 6^2}{8} = 22$ kNm

Section is Class 2 compact when subject to axial load and bending

Combined axial and bending check

Cross-section capacity for Class 2

$M_x < M_{rx}$

$F/P_z = 122.3 / 2600 = 0.05$, $M_{rx} = 406$ kNm $> M_x$ therefore section is adequate

Member buckling resistance, More exact method

The effective length of the section is taken as 6 m about both the major and minor axes.

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) \leq 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \leq 1$$

$P_{cx} = 2490$ kN, $P_{cy} = 715$ kN

$M_{cx} = 413$ kNm, $M_b = 176$ kNm

$m_x = 0.95$, $m_{LT} = 0.925$

$$\frac{122.3}{2490} + \frac{0.95 \times 22}{413}\left(1 + 0.5\frac{122.3}{2490}\right) = 0.05 + 0.05(1.025) = 0.10 < 1.0$$

$$\frac{122.3}{715} + \frac{0.925 \times 22}{176} = 0.17 + 0.12 = 0.29 < 1.0$$

Therefore, the 406 × 178 × 74 UB S275 is adequate for load case 2.

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BS 5950-1
Cl 4.8.2.3

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BS 5950-1
Cl 4.8.3.3.2a

P202
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BS 5950-1
Table 26
Table 18

Load case 3: 1.2 Dead + 1.2 Imposed + 1.2 Wind

Factored wind load $\qquad = 1.2 \times 4.4 \quad = 5.28$ kN/m

Max moment due to wind load $\quad = \dfrac{wL^2}{8} \quad = \dfrac{5.28 \times 48^2}{8} \qquad = 1521$ kNm

Axial load in edge beams due to wind $\quad = 1521 / 14.5 \qquad = 104.9$ kN

Factored gravity load on edge beam $= 1.2 \times 3.5 + 1.2 \times 5 \qquad = 10.2$ kN/m

Max moment due to dead load $\quad = \dfrac{wL^2}{8} \qquad = \dfrac{10.2 \times 6^2}{8} \quad = 45.9$ kNm

Section is Class 2 compact when subject to axial load and bending

P202
Page C-112

Combined axial and bending check
Cross-section capacity for Class 2

BS 5950-1
Cl 4.8.2.3

$M_x < M_{rx}$

$F/P_z = 104.9 / 2600 = 0.04$, $M_{rx} = 406$ kNm $> M_x$ therefore section is adequate

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Page C-112

Member buckling resistance, More exact method

The effective length of the section is taken as 6 m about both the major and minor axes.

BS 5950-1
Cl 4.8.3.3.2a

$$\frac{F_c}{P_{cx}} + \frac{m_x M_x}{M_{cx}}\left(1 + 0.5\frac{F_c}{P_{cx}}\right) \leq 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \leq 1$$

$P_{cx} = 2490$ kN, $P_{cy} = 715$ kN

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Page C-113

$M_{cx} = 413$ kNm, $M_b = 176$ kNm

$m_x = 0.95$, $m_{LT} = 0.925$

BS 5950-1
Table 26
Table 18

$$\frac{104.9}{2490} + \frac{0.95 \times 45.9}{413}\left(1 + 0.5\frac{104.9}{2490}\right) = 0.04 + 0.11(1.021) = 0.15 < 1.0$$

$$\frac{104.9}{715} + \frac{0.925 \times 45.9}{176} = 0.15 + 0.24 = 0.39 < 1.0$$

Therefore, the 406 × 178 × 74 UB S275 is adequate for load case 3.

B.1.6.2 Precast units

Shear between units

Maximum shear due to wind load $= 1.4 \times 4.4 \times 48 \,/\, 2 \qquad = 148$ kN

$$\text{Shear resistance} = \text{Joint shear strength} \times \text{depth} \times \text{length}$$

$$= 0.10 \text{ x}10^{-3} \times (200\text{-}35) \times 14.5 \times 10^3 = 239 \text{ kN} < 148 \text{ kN}$$

Therefore, the shear resistance of the joints between the precast units is adequate.

Concrete strut action

In the truss model for floor diaphragm action, the precast units are required to resist compressive force due to the horizontal wind load.

Total factored wind load $= 1.4 \times 4.4 \times 48 \qquad = 296$ kN

Force resisted by each set of bracing $= 296 \,/\, 2 \qquad = 148$ kN

The following calculation conservatively assumes that the force is resisted by the smaller (6 × 6 m) end bay. In reality the force will be resisted by both the 6 × 6 m and the 6 × 8.5 m end bays.

Angle of compressive force in slab $= \text{Arctan} (6 \,/\, 6) \qquad = 45°$

Compressive force in slab $= 148 \,/\, \text{Sin } 45° \qquad = 209$ kN
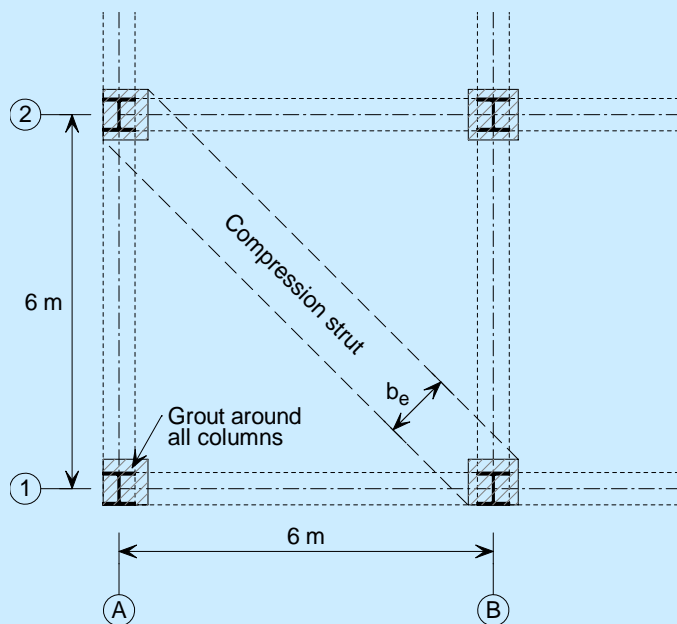

Figure B.1.4 *Compression strut in end bay*

Cross-sectional area of concrete resisting force, $A_e = t_e \times b_e$

where:

   t_e is the effective depth of the units $= 200 - 35 = 165$ mm

   b_e is the effective width of the compression strut $= (B_{col}^2 + D_{col}^2)^{0.5}$

$$= (254^2 + 254^2)^{0.5} \quad = 359 \text{ mm}$$

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Section 2.6.1

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Cross-sectional area of concrete resisting force, A_e = 165 × 359 = 59,235 mm^2

Compressive resistance of grout around the column, F_{cu} = 0.6 f_{cu} A_e

$$= 0.6 \times 25 \times 59,235 \times 10^{-3} \quad = 711 \text{ kN} > 209 \text{ kN}$$

Therefore, compressive resistance of the slab and grout around column is adequate.

The above calculation checks the compression resistance of the grout rather than the hollow core unit. This is because although the precast unit is not solid, its strength is considerably greater than that of the grout.

It is considered that the slab will not buckle under this compressive load as the compressive resistance is much greater than the applied force. The cross-sectional area of concrete resisting the compressive force is conservative as dispersion of load within the concrete has been ignored.

For completeness, the compression resistance of the grout is also checked on the orthogonal faces of the columns.

Cross-sectional area of concrete resisting force, A_e = 165 × 254 = 41,910 mm^2

Compressive resistance of grout on column face , F_{cu} = 0.6 f_{cu} A_e

$$= 0.6 \times 25 \times 41,910 \times 10^{-3} \quad = 629 \text{ kN} > 148 \text{ kN}$$

Therefore, compressive resistance of the slab and grout around column is adequate.

Bracing connection arrangement

Figure B.1.5 shows a possible beam to column connection arrangement including the diagonal bracing. The design of the bracing arrangement is outside the scope of this guide. However, the following should be considered:

- The edge beams shown in Figure B.1.5 should be checked for combined axial load and bending. The moments and forces in the beams will be from the combined effects of bracing system loads, diaphragm action and floor loading.

- The column in the bracing system may need to be checked for high shear depending on the load path of the connections.
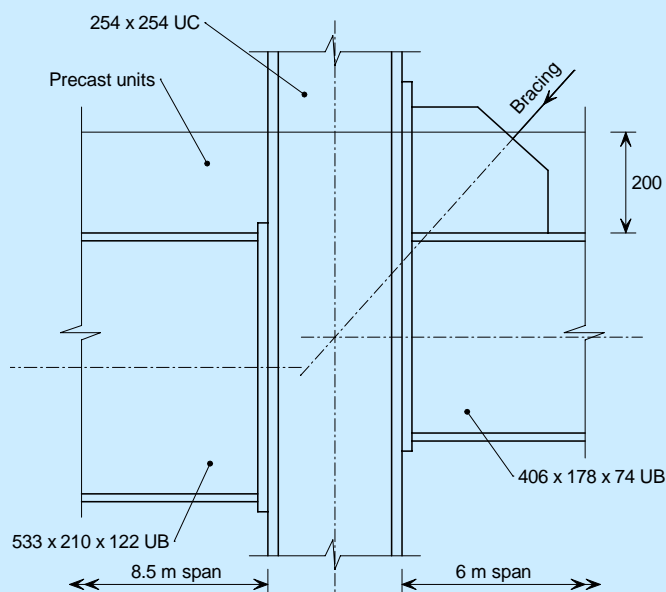

Figure B.1.5 *Connection arrangement*

B.1.7 Structural integrity

Industrial buildings up to three storeys and office buildings up to four storeys are Class 2A. Therefore, this building is Class 2A.

Approved Document A[26]

The requirement for Class 2A buildings is that effective horizontal ties should be provided. BS 5950-1 recommends that all horizontal frame elements in a Class 2A building are capable of resisting a tensile load of at least 75 kN.

Section 4.2.2

The beam-to-column connections used in this building are full depth end plate connections (as shown in Figure B.1.6). Full depth end plates are used because they can resist torsion. The end plates will be at least 10 mm thick and will have at least 8 no. M20 bolts. By comparison with the tying capacity of partial depth flexible end plates, the beam-to-column connections will easily satisfy the 75 kN minimum tying capacity.
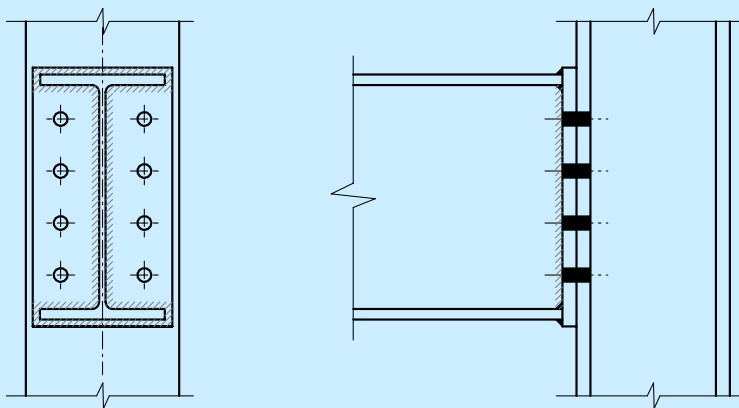
SCI-P212 page 390

Figure B.1.6 *Full depth end plate beam-to-column connection*

For Class 2A buildings, precast floor units are not required to be tied to the structural frame or to each other. However, the minimum bearing requirements of BS 8110 must be complied with. The bearing details used in this example are adequate.

	Job No.	CDA 155		Sheet	1	of	10	Rev	B
SCI	Job Title	Example no. 2							
	Subject	Four-storey car park							
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Client	SCI / Bison	Made by	AW		Date	Jan 2007		
CALCULATION SHEET			Checked by	TCC		Date	Feb 2007		

B.2 Four-storey car park

B.2.1 Introduction

The purpose of this example is to demonstrate:

    a)   design of temporary lateral restraint for long span beams

    b)   design of beam subject to unbalanced construction load

    c)   the application of the tying details required for Class 2B buildings .

Other aspects of the design process that would need to be considered in practice are not included in this example. An example of the design process for a composite beam with precast units in a car park application is provided in SCI publication P287.

P287[3]

The car park structure is a braced steel frame with precast units supported on the top flange of downstand beams. The beams supporting the precast units are designed as composite beams with a structural topping. The dimensions of the building are shown in Figure B.2.1 and the column grid is shown in Figure B.2.2. The cross-section of the floor is shown in Figure B.2.3.
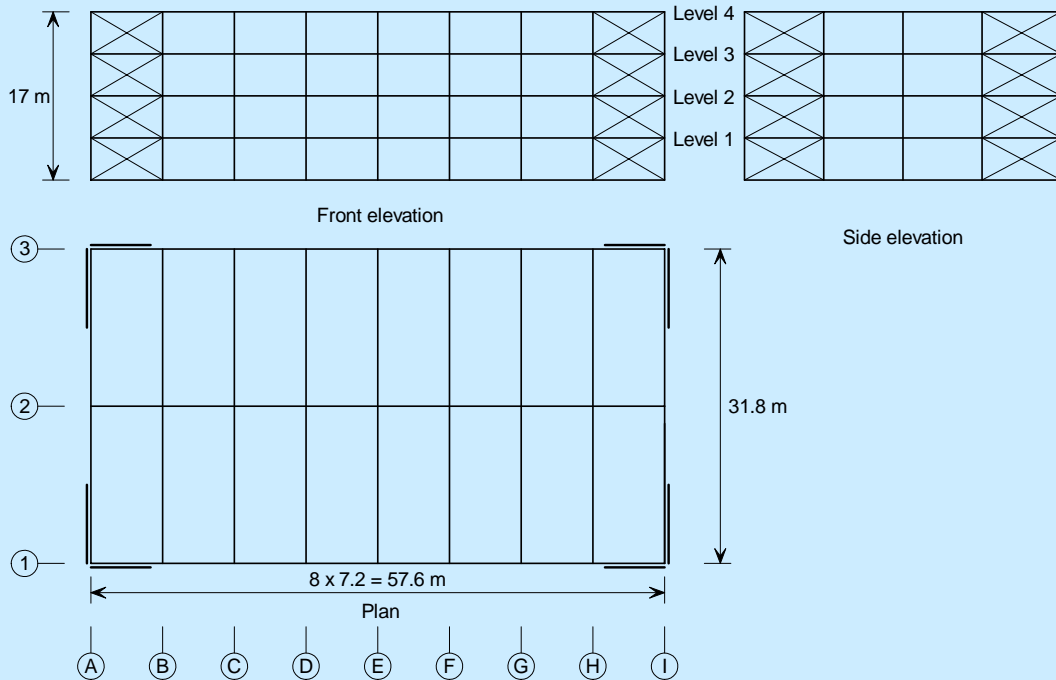

Figure B.2.1 *Building dimensions*

B.2.2 Loading

Floor loading

Dead load = 3.9 kN/m² (assuming a 150 mm precast unit, 50 mm topping and including the beam weight)

Imposed load = 2.5 kN/m² (normal condition)

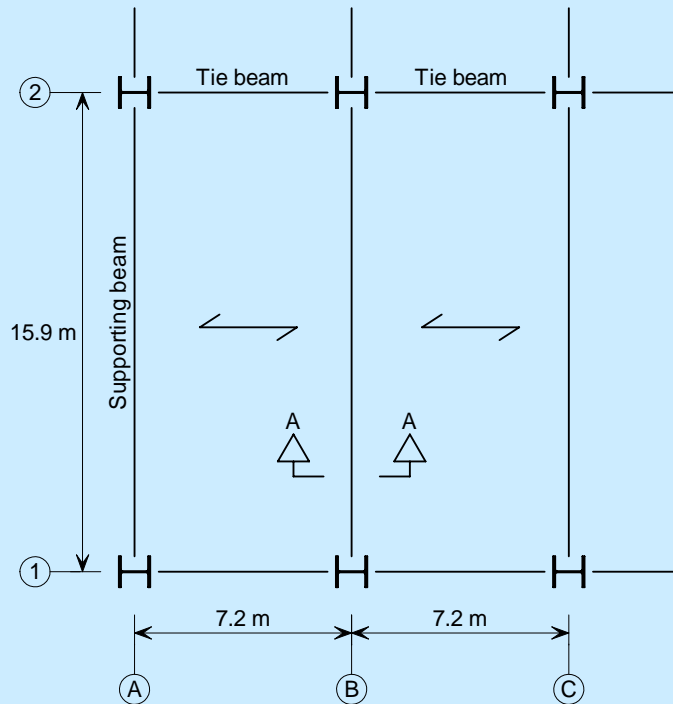Imposed load = 0.5 kN/m² (construction condition)

Figure B.2.2 *Column grid and beam arrangement*
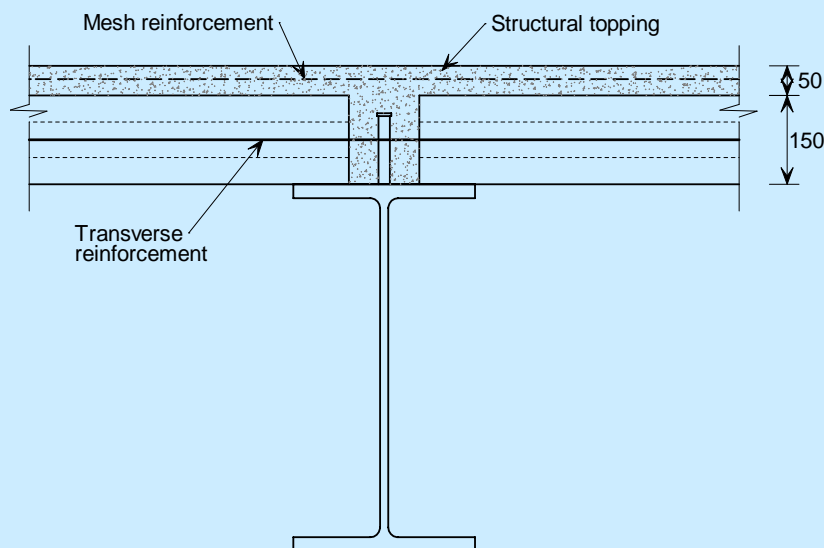

Figure B.2.3 *Composite beam (cross-section A-A)*

This example uses a 50 mm structural topping. However, for durability in a car park application, the topping may need to be 75 or 85 mm

B.2.3 Precast unit selection

The maximum span of the precast units in this building is 7.2 m. From manufacturers' load span tables for an imposed load of 2.5 kN/m² a 150 mm deep hollow core unit with a topping can span 8.2 m. Therefore, select a 150 mm deep hollow core unit with a structural topping.

Table 3.2

This example uses 150 mm hollow core units, however, 200 mm units can be more practical in some situations because the cores are larger; this enables easier placement of the reinforcing bar.

Note: Manufacturers load span tables are only given for guidance. Final precast unit selection should be carried out in consultation with the manufacturer.

B.2.4 Steel frame members

The beams and columns for the steel frame have been selected based on dead and imposed loads given above. The tie beams are assumed to carry a 1 m strip of floor load but are assumed to be unrestrained. The beams supporting the precast units are composite in the normal condition.

Columns:

305 × 305 × 137 UC S355   for columns up to Level 2

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305 × 305 × 97 UC S355   for columns from Level 2 to Level 4

Beams:

610 × 305 × 238 UB S275   for 15.9 m supporting beams (e.g. Grid C1 to C2)

305 × 127 × 48 UB S275   for 7.2 m tie beams (e.g. Grid C2 to D2)

Note: For this example, it is assumed that the section sizes used for edge beams will be the same sections as for the corresponding internal beams.

Bearing length

The width of the beams supporting the precast units must be wide enough to provide the minimum bearing length for the precast units. For the construction used in this example the minimum required flange width is 190 mm (it is assumed that the shear studs will be shop welded). The supporting beam selected above is adequate.

Table 3.5

B.2.5 Temporary lateral restraint for long span beams

As discussed in Section 5.3.2, the temporary condition for beams must be considered in design. This is particularly true for long span beams (> 8 m) because even when the beam is subjected to balanced loading, lateral restraint provided by the precast units can not be relied upon in the construction condition. In this example, the supporting beams span 15.9 m and therefore temporary restrain will be required. The arrangement adopted for temporary restraint is shown in Figure B.2.4.
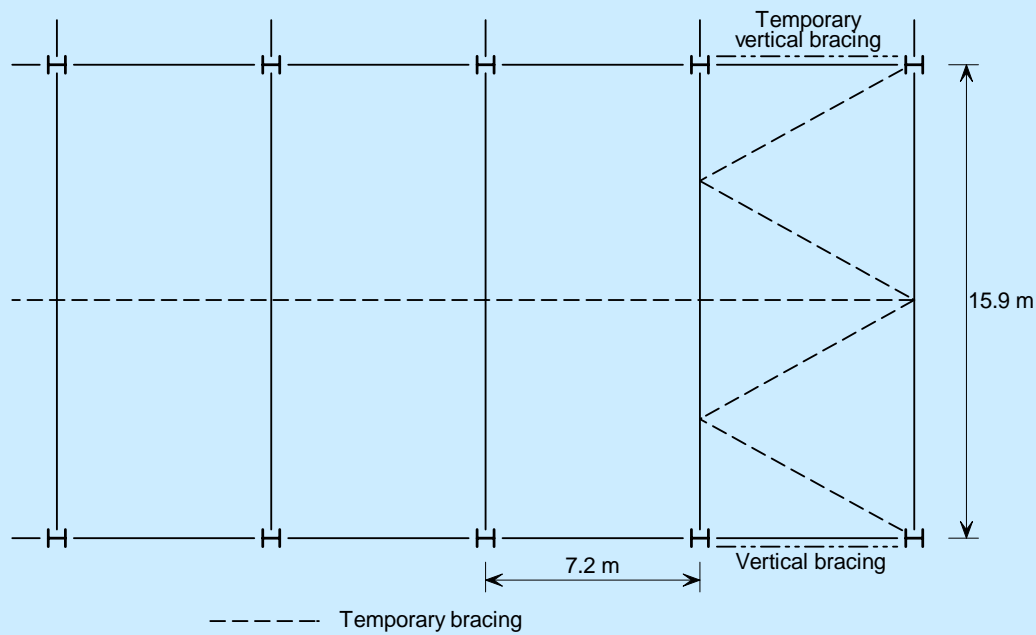
Figure B.2.4 *Temporary bracing arrangement*

The intermediate lateral restraint should be capable of resisting 2.5% of the force in the compression flange of the beam.

BS 5950-1
Cl 4.3.2.2.1

Floor load during the construction stage:

$$= 1.4 \times \text{dead load} + 1.6 \times \text{construction imposed load}$$

$$= 1.4 \times 3.9 + 1.6 \times 0.5 \quad = 6.26 \text{ kN/m}^2$$

Maximum moment in supporting beam during the construction stage:

$$= \frac{wL^2}{8} = \frac{6.26 \times 7.2 \times 15.9^2}{8} \quad = 1424 \text{ kNm}$$

Maximum force in flange $= 1424 / (635.8 - 31.4) \times 10^{-3} = 2356$ kN

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2.5% of flange force $= 2.5\% \times 2356 = 59$ kN

The temporary bracing system will be used to restrain the supporting beams for the whole length of the building. Therefore, the restraint forces from the restrained beams need to be summed together. There are nine parallel beams requiring restraint.

Total restraint force $= 9 \times 59 = 531$ kN

Note: This is a conservative value because the flange force in the edge beams is half the calculated value and the temporary horizontal truss can restrain the right hand edge beam directly.

The reduction factor for multiple restraints, $k_r = (0.2 + 1/N_r)^{0.5}$

BS 5950-1
Cl 4.3.2.2.3

$k_r = (0.2 + 1/9)^{0.5} = 0.558$

Required restraint force $= 0.558 \times 531 = 296$ kN

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The effective length of the members providing temporary restraint is 7.2 m.

Select 193.7×6.3 CHS S275 (P_c for 8 m = 436 kN)

To be effective the restrain must be located within a distance of 12 times the beam web thickness of the compression flange of the beam.

Maximum distance from centre of flange to centre of CHS = $12 \times 18.4 = 221$ mm

There is a second condition that the restraint should also be positioned closer to the level of the shear centre of the compression flange than to the shear centre of the section. Therefore, the maximum distance from the centre of the restraint to the top of the section is given by:

$$\frac{D-T}{4} + \frac{T}{2} = \frac{635.8 - 31.4}{4} + \frac{31.4}{2} = 167 \text{ mm}$$

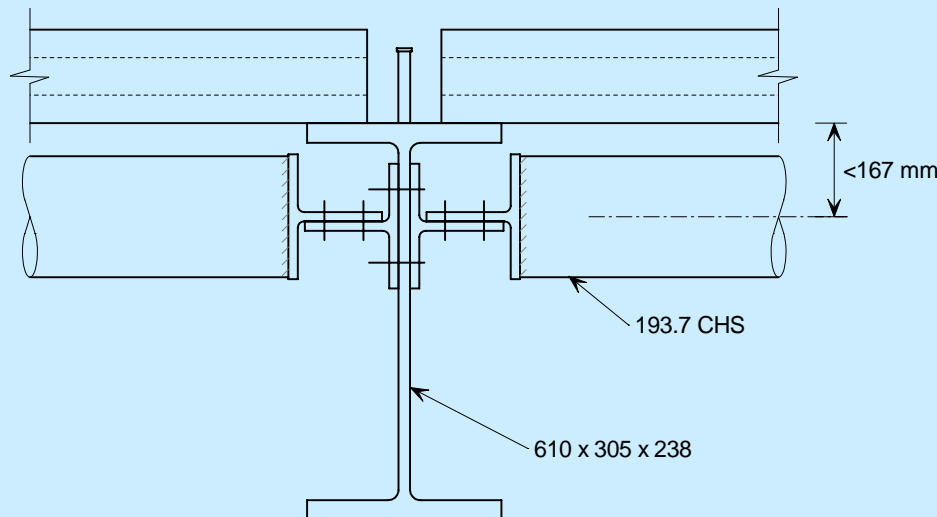A possible detailing arrangement for the temporary restraint is shown in Figure B.2.5.

Figure B.2.5 *Connection of temporary restraint to supporting beam*

B.2.6 Design of beam subject to torsion

During the construction, it is generally impractical for the precast units to be placed alternately either side of the beam. Therefore, the beams will be subject to unbalanced loading and will not be laterally restrained by the precast units. The supporting beams need to be checked for the construction condition of unbalanced loading due to precast units being placed on only one side.

The construction loads for the unbalanced condition are:

Hollow core units = 2.40 kN/m²

Steel beam   = 2.4 kN/m

Construction load = 0.5 kN/m²

Design load   = $1.4 \times (2.4 \times 7.2 / 2 + 2.4) + 1.6 \times 0.5 \times 7.2 / 2$ = 18.34 kN/m

Side notes:
P202 page C-9

Figure 5.7

BS 5950-1 Cl 4.3.2.1

Table 3.3

Design shear F_v $= \dfrac{wL}{2} = \dfrac{18.34 \times 15.9}{2}$ $= 146$ kN

Design moment M_x $= \dfrac{wL^2}{8} = \dfrac{18.34 \times 15.9^2}{8}$ $= 580$ kNm

The shear capacity (1860 kN) and moment capacity (1980 kN) of the $610 \times 305 \times 238$ UB S275 beam are adequate.

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The buckling resistance moment is checked for a destabilising load condition because the units are supported on the top flange of the beam which is above the shear centre of the beam. The effective length for lateral torsional buckling, taking account of the mid-span restraint is given by:

$L_E = L \times 0.5 \times 1.2$

BS 5950-1
Cl 4.3.5

$L_E = 15.9 \times 0.5 \times 1.2$ $= 9.54$ m

The buckling resistance moment (1020 kNm for $L_E = 10$ m) is adequate.

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page C-59

Combined bending and torsion

The design process for designing supporting beams subject to combined bending and torsion is described in Section 4.1 of SCI publication P287. For simplicity the mid-span restraint is neglected for the torsional bending check. This is a conservative approach.

P287[3]

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

Section 3.3

$e = B/2 - 20$ $= \dfrac{311.4}{2} - 20$ $= 135.7$ mm

The torsional moment is:

$T_q = F_v \times e$ $= 146 \times 135.7 \times 10^{-3}$ $= 19.8$ kNm

Buckling check

The criterion in P287 is $\dfrac{\overline{M}_x}{M_b} + \dfrac{(\sigma_{byt} + \sigma_w)}{p_y} \left[1 + 0.5 \dfrac{\overline{M}_x}{M_b} \right] \leq 1.0$

SCI-P287
Equ. 1

Where:

$\overline{M}_x$ is the equivalent uniform moment given by: $\overline{M}_x = m_{LT} M_x$, in which m_{LT} is 1.0 for a destabilizing load condition

BS 5950-1
Cl 4.3.6.6

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: $M_{yt} = \phi \times M_x$

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: $\sigma_w = -EW_{n0}\phi''$

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.

To calculate σ_w, it is assumed that the connections provided (end plates) are simple connections, i.e. ends torsionally fixed, warping free.

The angle of twist and its derivatives depend on the value L/a, where a is the torsional bending constant given by:

$$a = \sqrt{EH/GJ}$$

P287
Equ. 4

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

Section properties P202

$$= 2192 \text{ mm}$$

$L/a = 15900 / 2192 = 7.25$

Hence, $\dfrac{\phi GJ}{T_q a} = 0.769$ and $-\dfrac{\phi'' GJa}{T_q} = 0.132$

P287
Table 4.1

Rearranging gives

$$\phi = \frac{0.769 \times 19.8 \times 10^6 \times 2192}{78.8 \times 10^3 \times 785 \times 10^4} = 0.054 \text{ radians}$$

$M_{yt} = \phi \times M_x = 0.054 \times 580 = 31.3 \text{ kNm}$

$\sigma_{byt} = M_{yt} / Z_y = 31.3 \times 10^3 / 1020 = 30.7 \text{ N/mm}^2$

$$-\phi'' = \frac{0.132 \times 19.8 \times 10^6}{78.8 \times 10^3 \times 785 \times 10^4 \times 2192} = 1.93 \times 10^{-9}$$

For a symmetrical I-section, the normalised warping function at the flange tips is given by:

P287
page 23

$W_{n0} = h \times B / 4 = (635.8 - 31.4) \times 311.4 / 4 = 47053 \text{ mm}^2$

$\sigma_w = -EW_{n0}\phi'' = 205 \times 10^3 \times 47053 \times 1.93 \times 10^{-9} = 18.6 \text{ N/mm}^2$

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

P287
Equ. 1

$$= \frac{580}{1020} + \frac{(30.7 + 18.6)}{265}\left[1 + 0.5 \times \frac{580}{1020}\right] = 0.81 < 1.0 \quad \text{OK}$$

Local capacity check

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

| P287 Equ. 2

$$\sigma_{bx} = \frac{M_x}{Z_x} = \frac{580 \times 10^3}{6590} = 88 \text{ N/mm}^2$$

$$88 + 30.7 + 18.6 = 137 \text{ N/mm}^2 \quad < 265 \text{ N/mm}^2 \quad \text{OK}$$

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. The shear check is not included in this example. An example of such a check is included in SCI publication P287.

Combined bending and torsion summary

The section is adequate for buckling, local capacity and shear when subject to unbalanced construction load. The section is adequate for combined bending and torsion.

Other unbalanced construction load cases (e.g. precast units on both sides but wet topping and construction load on only one side) may also need to be considered depending on the construction sequence. The required temporary restraint force should be checked for the appropriate unbalanced load cases.

B.2.7 Detailing for robustness

Car parks up to six storeys are Class 2B buildings. Therefore, the recommendations of BS 5950-1, clauses 2.4.5.3 a) to e) should be followed. This includes the steel frame having horizontal and vertical ties and the precast units to be tied to each other over supports or to the support directly.

| Approved Document A [Ref 26]

Horizontal tying

For internal ties (supporting beams e.g. Beam from C1 to C2):

| BS 5950-1 Cl 2.4.5.3a

$$T = 0.5 (1.4 \, g_k + 1.6 \, q_k) \, s_t \, L \, n \quad \text{but not less than 75 kN}$$

$$T = 0.5 (1.4 \times 3.9 + 1.6 \times 2.5) \, 7.2 \times 15.9 \times 0.75 = 406 \text{ kN}$$

A standard flexible end plate connection for a $610 \times 305 \times 238$ UB with seven rows of bolts has a tying resistance of 379 kN. For this connection, the critical check is the tension capacity of the end plate.

| P212 page 388

For construction with precast units, full depth flexible end plates should be used. The tying capacity of full depth end plate connections is significantly greater than partial depth end plate connections. This is because the end plate is longer and usually thicker for the full depth end plates and the end plate is welded to the flanges as well as the web of the beam section.

Therefore, a full depth flexible end plate on a $610 \times 305 \times 238$ UB will have a tying capacity that exceeds 406 kN.

Tying of edge columns

As the horizontal tying requirement is satisfied for internal beams; by inspection it can be seen that tying of edge columns condition is satisfied because the same connection details are used.

| BS 5950-1 Cl 2.4.5.3b

Vertical tying

The column splice is required to have a tension capacity equal to the largest total factored dead and imposed floor load at any level below the splice down to the next splice.

BS 5950-1
Cl 2.4.5.3c

Adequate vertical tying can be provided by a splice with flange cover plates.

Bracing systems

The bracings systems shown in Figure B.2.1 are sufficient to satisfy this requirement.

BS 5950-1
Cl 2.4.5.3d

Heavy floor units

Section
4.2.3

The magnitude of the required anchorage force per unit is given by:

$$T = 0.5(1.05 \ (1.0g_k + 0.33q_k) \ L \ w$$

$$T = 0.5(1.05 \ (1.0 \times 3.9 + 0.33 \times 2.5) \ 7.2 \times 1.2) \quad = 21.4 \text{ kN}$$

For internal supporting beams, the composite beam design requires that there is transverse reinforcement of 16 mm diameter bars placed in at least four cores per unit.

P287
page 88

The anchorage force of four 16 mm diameter bars properly anchored is

$$= 4 \times \pi \times 8^2 \times 500 \times 10^{-3} \ / \ 1.15 = 350 \text{ kN}$$

This is more than adequate to satisfy the tying requirements.

Supporting edge beams can be designed as composite or non-composite beams. If they are composite beams, the transverse reinforcement will be sufficient to provide the required tying to the edge beam. If the supporting edge beams are non-composite, a detail similar to that shown in Figure B.2.6 should be used.
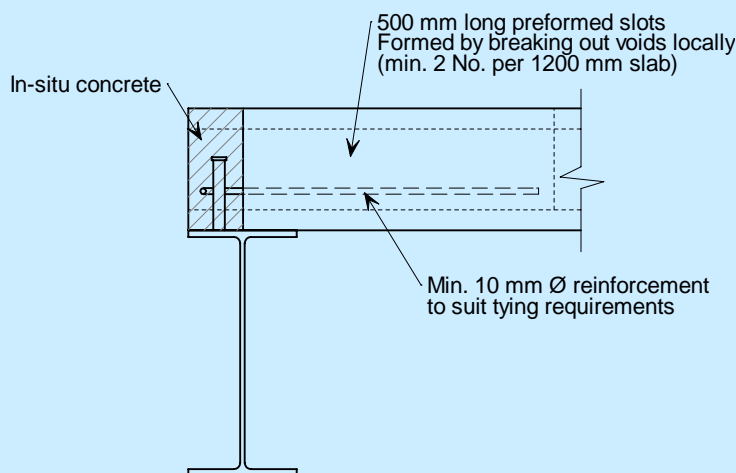

Figure B.2.6 *Detail for tying precast units to a supporting edge beams*

B.2.8 Diaphragm action

When a structural topping is provided with precast floors acting composite with supporting steel beams, floor diaphragm action is generally adequate for buildings with regular rectangular floors of normal proportions without exceptionally large openings. Therefore, no specific check is performed in this example.

	Job No.	CDA 155		Sheet	1	of	4	Rev	A
SCI	Job Title	Example no. 3							
	Subject	Multi-storey residential building							
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Client								
		SCI / Bison	Made by		AW		Date		Feb 2007
CALCULATION SHEET			Checked by		TCC		Date		Mar 2007

B.3 Multi-storey residential building

B.3.1 Introduction

The main purpose of this example is to demonstrate:

  a) Structural integrity calculations

  b) Acoustic detailing for residential buildings.

The structural integrity requirements are discussed briefly but other aspects of the design process that would need to be considered in practice are not included in this example.

The building is a multi-storey, multi-occupancy residential development that is designed to comply with Approved Document E for the resistance to the passage of sound.

The structure is a simple construction braced frame with precast units supported on the bottom flange of asymmetric beams (ASB). The joints between the precast units and around the columns are grouted with low shrinkage C20 concrete with 6 m aggregate. There is no structural topping applied to the units. An isolated screed will be installed as part of the sound insulation solution as shown in Figure B.3.4. The floor construction and column grid are shown in Figure B.3.1 and Figure B.3.2.
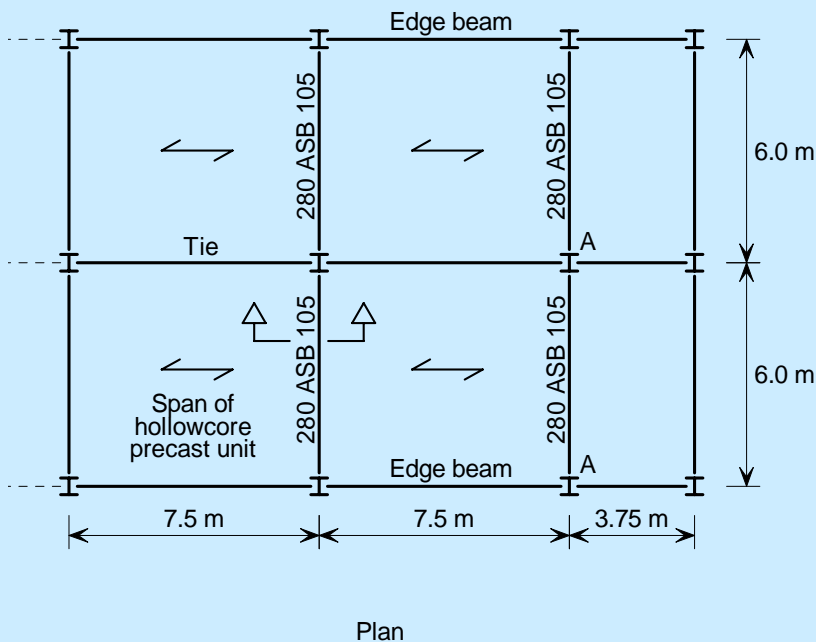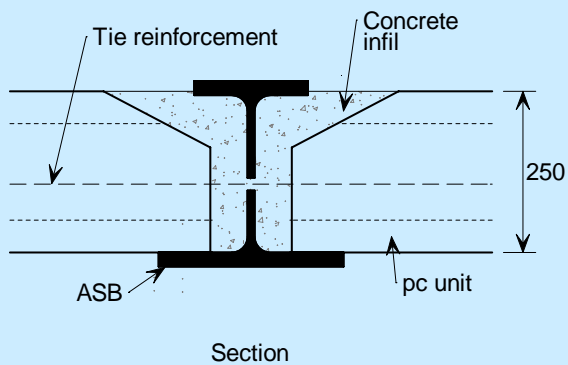

Figure B.3.1 *Column grid*

Figure B.3.2 *Floor cross-section*

The structural aspects of the floor and beam design are covered in Example 1 of SCI publication P342[4]. Only the acoustic detailing considerations are included in this example.

B.3.2 Precast unit selection

To satisfy the acoustic requirements for residential buildings it is generally necessary to use precast units which have a mass of at least 300 kg/m^2 (which is equivalent to 2.94 kN/m^2). The structural solution presented above has a weight of 3.30 kN/m^2. Therefore, the minimum density requirement is easily satisfied with a 250 mm hollow core unit.

Section 4.3.1

B.3.3 Steel frame members

The beams and columns for the steel frame have been selected based on factored gravity loads for ULS and SLS conditions. (Dead load = 3.8 kN/m^2 and Live load = 3.5 kN/m^2).

SCI-P342

Columns: 254 × 254 × 73 UC S275   for all columns

Beams: 280 ASB 105 S275   for 6.0 m supporting beams

To maintain the shallow floor construction depth, the tie beam needs to be within the depth of the precast floor. Therefore, an inverted tee section can be used. However, it should be noted that the tee section will provide minimal flexural stiffness and therefore temporary bracing may be required during construction. The floor slab is broken into sections by the inverted tee sections between the units which can adversely affect the sensitivity of the floor to vibration. Moreover, if the shear resistance of the low shrinkage concrete between the precast units can not be relied on due to the presence of the tee, the diaphragm action of the floor should be justified using a truss model.
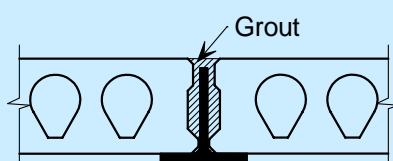

Figure B.3.3 *Joint between adjacent floor units*

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B.3.4 Structural integrity

The structural integrity requirements for the steel frame are satisfied by complying with the relevant sub clauses of Clause 2.4.5 of BS 5950-1. Which clauses are relevant will depend on the classification of the building to Approved Document A which will depend on the number of storeys.

For Class 2A buildings (not exceeding four storeys), the bearing details of precast concrete floor units should comply with the BS 8110. The 280 ASB 105, used in this example, has a bottom flange width of 286 mm which will easily enable the minimum bearing of 40 mm.

BS 5950-1
Cl 2.4.5.2

For Class 2B buildings (greater than four but not exceeding 15 storeys), the floor units should be anchored in the direction of their span. Clause 5.1.8.3 of BS 8110-1 recommends that the anchorage of the floor units should be capable of carrying the dead weight of the member. However, this example uses a more onerous anchorage based on the accidental load condition from Clause 2.4.5.3 of BS 5950-1 given by:

Anchorage force per unit $= 0.5 \, (1.05 \, (1.0 \, g_k + 0.33 \, q_k) \times L \times w \,)$

where:

g_k (Dead load) $= 3.8$ kN/m^2

q_k (Live load) $= 3.5$ kN/m^2

L (Span) $= 7.5$ m

w (Unit width) $= 1.2$ m

Anchorage force per unit $= 0.5 \, (1.05 \, (3.8 + 0.33 \times 3.5) \times 7.5 \times 1.2) = 23.5$ kN

A T10 high yield reinforcement bar (with a yield strength of 500 N/mm^2) that is properly anchored will provide a tie capacity of approximately 34 kN, a T12 bar will provide 49 kN and a T16 bar will provide approximately 87 kN. It is recommended that at least two bars are provided per unit. Therefore, provide two T10 bars per unit. The bars should be placed in opened cores and passed through holes drilled in the web of the ASB. Details for tying to edge beams are provided in Figure 4.9.

The shear resistance of the tie bars may be utilised in the diaphragm analysis of the floor.

B.3.5 Acoustic details

For compliance with Approved Document E separating floors between dwellings should have:

- An airborne sound insulation, $D_{nt,w} + C_{tr} \geq 45$ dB
- An impact sound insulation, $L'_{nT,w} \leq 62$ dB

An appropriate floor construction to satisfy the sound insulation requirements is shown in Figure B.3.4. The floor construction is based on details presented in Section 4.3.1.
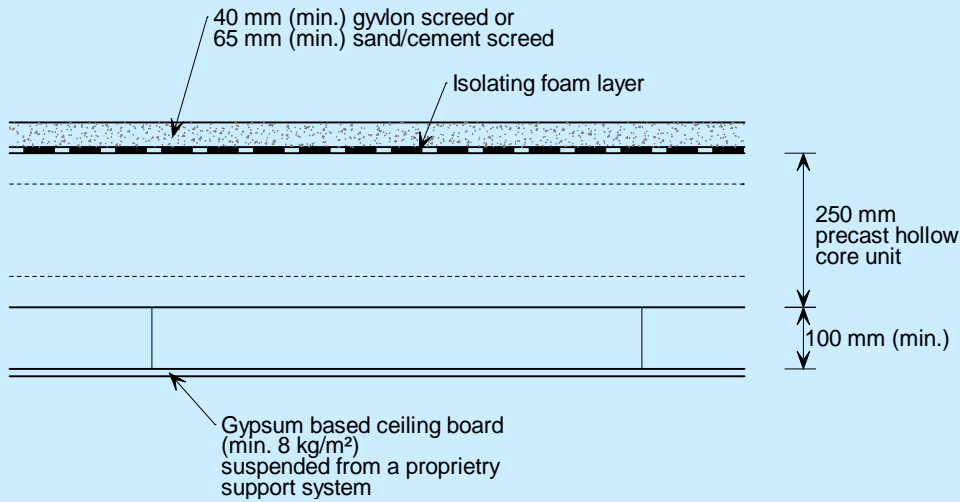
40 mm (min.) gyvlon screed or
65 mm (min.) sand/cement screed

Isolating foam layer

250 mm
precast hollow
core unit

100 mm (min.)

Gypsum based ceiling board
(min. 8 kg/m²)
suspended from a proprietry
support system

Figure B.3.4 *Separating floor detail*

An appropriate junction detail is shown in Figure B.3.5. The principles described in Section 4.3 and those used in the Robust Details Handbook and SCI publication P336[33] have been used to develop the detail.
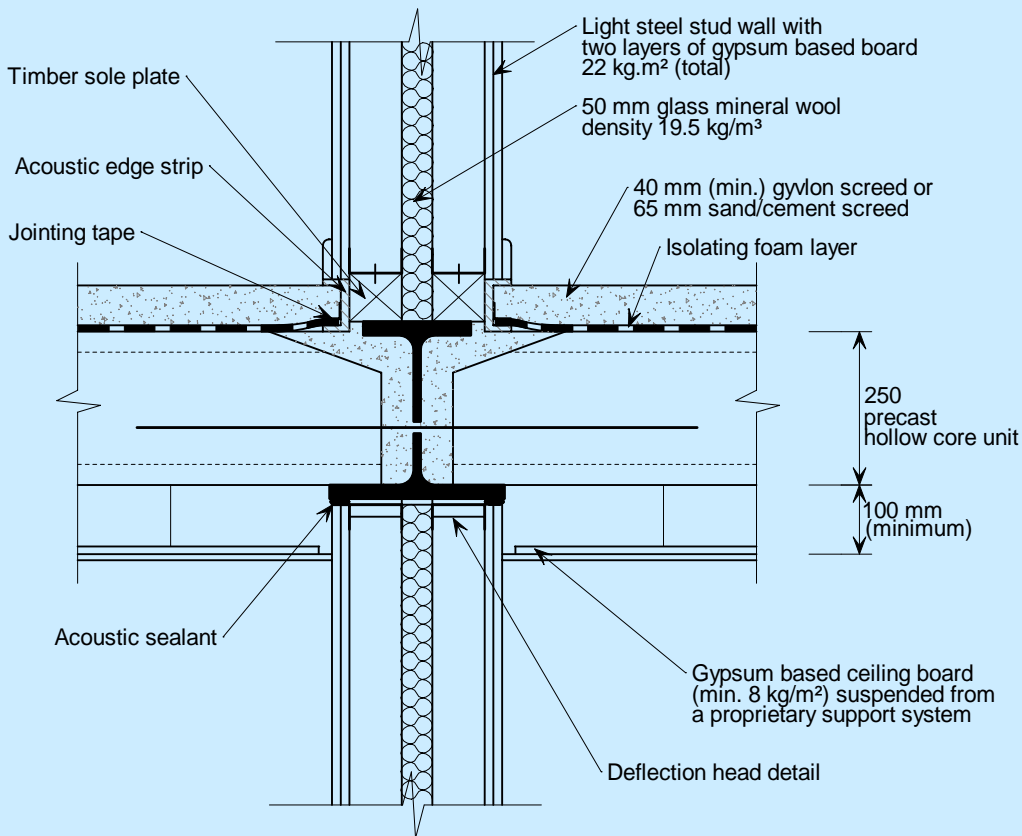
Light steel stud wall with
two layers of gypsum based board
22 kg.m² (total)

Timber sole plate

50 mm glass mineral wool
density 19.5 kg/m³

Acoustic edge strip

40 mm (min.) gyvlon screed or
65 mm sand/cement screed

Jointing tape

Isolating foam layer

250
precast
hollow core unit

100 mm
(minimum)

Acoustic sealant

Gypsum based ceiling board
(min. 8 kg/m²) suspended from
a proprietary support system

Deflection head detail

Figure B.3.5 *Junction details between separating floor and wall*

APPENDIX C Floor design using lattice slab

C.1 Introduction

The purpose of Appendix C is to highlight the design considerations for a floor using precast lattice slabs supported on steel beams. The guidance given in this Appendix is for lattice slab floors that are designed to act compositely with the supporting steel beams, welded shear studs and transverse reinforcement. A typical cross-section of a composite lattice slab floor construction is shown in Figure C.1.

Many of the calculations specific to using precast lattice slabs will be carried out by the precast manufacturer. The design of steelwork supporting lattice slabs should follow the same guidance as provided for steelwork supporting other types of precast floor units (see Section 2).
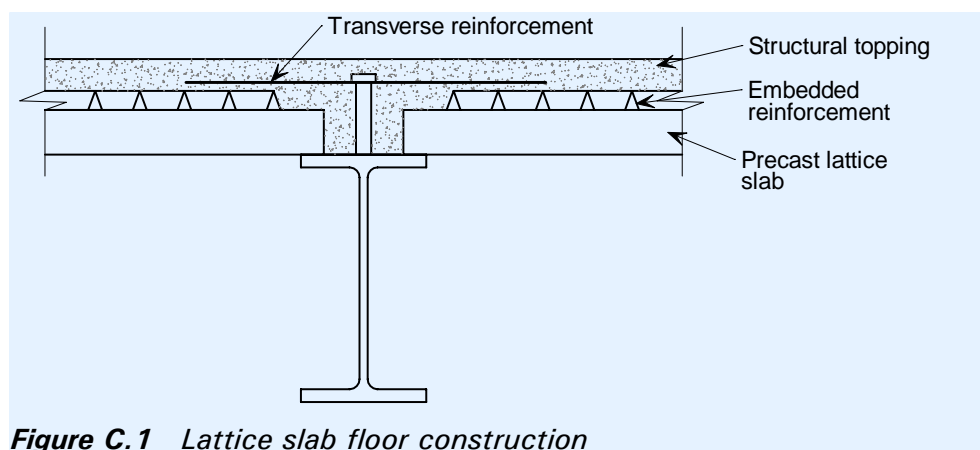

Figure C.1   *Lattice slab floor construction*

Lattice slabs for composite construction are manufactured in single span lengths, which aids erection and allows the shear studs to be welded to the steel beam in the fabrication shop.

Manufacturer's data can be used to estimate a slab thickness.

C.2 Slab design

Design of precast lattice slabs requires specialist knowledge and is therefore carried out by the precast manufacturer.

In the normal condition, the floor slab is designed as a continuous reinforced concrete beam in accordance with BS 8110-1, typically for three spans.

The slab should be designed for the shear force and bending moment envelopes for the load cases shown in Figure C.2 for a slab continuous over three spans. This is to ensure the combination of maximum shear and moment is included. The manufacturer should check that if other more critical cases arise that these are also checked e.g. only two continuous spans, unequal spans and short end spans where uplift can occur.
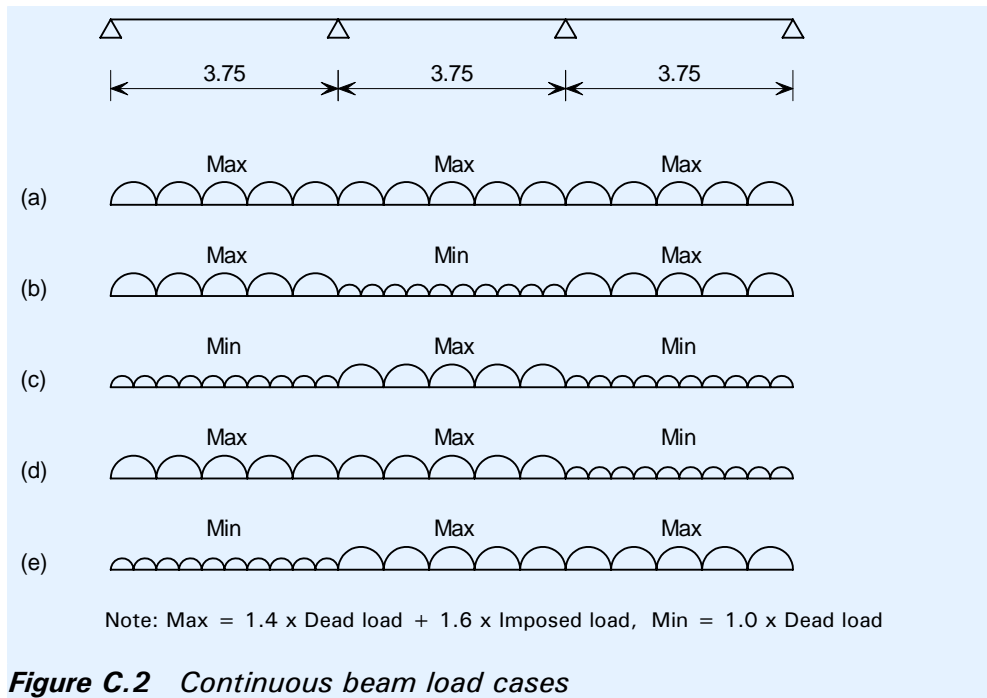
Note: Max = 1.4 x Dead load + 1.6 x Imposed load, Min = 1.0 x Dead load

Figure C.2 *Continuous beam load cases*

The manufacturer will specify tension reinforcement over the supports and the beam designer will specify transverse reinforcement for composite beam design. The manufacturer and designer must coordinate the specification of these additional reinforcements. All additional reinforcement is fixed on site.

The manufacturer will check the horizontal shear along the interface between the precast lattice slab and the structural topping to ensure the composite action of the slab in the normal condition. The shear resistance at the interface is provided by the embedded lattice reinforcement and the shear bond between the precast and the in-situ concrete. The horizontal shear force to be resisted is calculated in accordance with BS 8110-1, Clause 5.4.7. A typical bending moment diagram is shown in Figure C.3. The horizontal shear force associated with the hogging moment at the internal support 'x' must be carried by the shear resistance of the interface over the distance 'c'. The horizontal shear force associated with the sagging moment at 'y' must be carried by the shear resistance of the interface over the smaller distance of 'a' and 'b'.
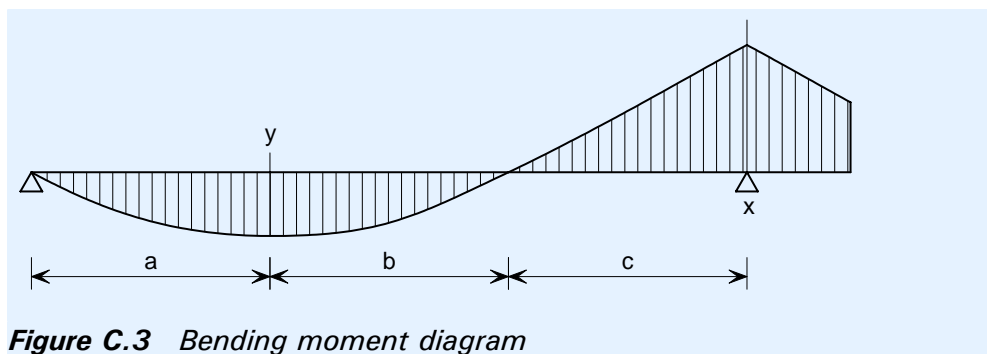

Figure C.3 *Bending moment diagram*

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C.3 Propping

Generally, depending on the slab depth, spans up to 6 m need one central row of props for the construction stage while the structural topping is poured and until it has gained adequate strength. For spans over 6 m two rows of props are likely to be required.

The precast manufacturer will calculate the propping requirements in the temporary condition. An imposed construction loading allowance of 0.5 kN/m^2 will usually be assumed.

The props should be in place, at the correct level, prior to the lattice slabs being placed.

Unlike hollow core units, the joints between the units do not need to be grouted.

The cross-section of the slab is checked using the design equations from Section 3.4 of BS 8110-1. Therefore, the propping and loading sequence of the slab does not influence its ultimate limit state resistance.

C.4 Diaphragm action

Lattice slab floors will always have a structural topping. Therefore, diaphragm action is generally adequate for buildings with rectangular floors of normal proportions without exceptionally large openings. Further guidance on floor diaphragm action is provided in Section 2.6.

C.5 Detailing for robustness

The design requirements for robustness of steelwork supporting lattice slab floors are the same as for steelwork supporting other types of precast floors (see Section 4.2).

For the lattice slabs in a Class 2A building, the minimum bearing requirements from BS 8110-1 must be provided.

For lattice slabs in Class 2B buildings, tying reinforcement may be provided in the structural topping to tie the lattice slabs together over internal supports. The tension reinforcement provided over the supports may also be adequate for tying reinforcement. At the edges of the building, reinforcement is provided to tie the lattice slab to the edge beams. Similar details to those provided in Section 4.2.3 may be used for lattice slabs.

Tying reinforcement may also be placed and lapped in the perpendicular direction to the span of the unit thereby producing an alternative load path for the tying.

APPENDIX D   Sources of further information

IPHA
International Prestressed Hollowcore Association
The Ramblers
Bank Road
Penn
High Wycombe
Buckinghamshire
HP10 8LA
Tel: 01494 812842
Fax: 01494 812842
Website: www.iphaweb.org

Bison
Bison Concrete Products Limited
Millenium Court
First Avenue Centrum 100
Burton-Upon-Trent
DE14 2WR
Tel: 01283 495000
Fax: 01283 544900
Website: www.bison.co.uk

British Precast
60 Charles Street
Leicester
LE1 1FB
Tel: 0116 253 6161
Fax: 0116 251 4568
Website: www.britishprecast.org

SCI
Silwood Park
Ascot
Berkshire
SL5 7QN
Tel: 01344 636525
Fax: 01344 636570
Website: www.steel-sci.org