Precast concrete floors in steel framed buildings
The contents of this technical guidance document formed the basis of three technical articles which were published in the April, May and June editions of *The Structural Engineer* in 2018.
1 INTRODUCTION

In the UK steel-framed buildings with precast concrete floors are a common form of multi-storey building. Such structures may be used for car parking, for commercial, retail or residential property developments and for public buildings, such as schools and hospitals. This hybrid form of construction has many benefits to it, including the provision of an early, secure and broad platform from which subsequent site activities can be undertaken. This technical guidance document addresses aspects of designing steel framed buildings with precast concrete floors.

2 SLAB DESIGN

The design of precast prestressed concrete planks (see Figure 2.1) will usually be undertaken by the manufacturer in consultation with the frame designer, who will need to specify the design floor loading. The major design issues with respect to load capacity are briefly discussed below:

(i) Unlike a conventionally reinforced precast concrete element, the only reinforcement in a prestressed plank comprises longitudinal prestressing tendons, which are anchored by bonding with the concrete as it cures. Since the majority (often all) of the tendons are concentrated in the lower half of a plank, floors will usually be designed to be simply supported in order to avoid tensile stresses in the top of the plank.

(ii) The bending resistance of a precast prestressed plank is determined in a similar manner to that of any prestressed concrete element. The British Precast document CCIP-034 Precast Eurocode 2: Worked Examples [1] contains a detailed example of the design of a hollowcore plank.

(iii) The shear resistance of a precast prestressed plank is determined in a similar manner to that of a conventionally reinforced concrete one-way spanning slab. The reader is again referred to the British Precast document CCIP-034 Precast Eurocode 2: Worked Examples for information about designing hollowcore planks for shear. Additionally, the manufacturer will perform design checks in the vicinity of the supports, such as:

(a) shear tension failure, which occurs when the principal tensile stress in the web of a hollowcore plank reaches the tensile strength of the concrete;
(b) sufficient anchorage of the prestressing tendon, and
(c) the effects of non-rigid supports.

The checks for (a) and (b) are strongly affected by the length from the support over which the full prestressing force is developed (known as the ‘transmission length’).

(iv) The use of a structural topping on precast prestressed planks enhances the structural capacity of the floor due to composite action between the topping and the planks. Typically, a structural topping comprises a 75 mm thick layer of C25/30 normal weight concrete reinforced with fabric reinforcement. This structural topping should not be confused with some types of screed that do not contribute any structural enhancement to the precast concrete planks.

The design of the topping must take into account:

(a) compressive stress in the topping;
(b) horizontal (complementary) shear at the interface between the topping and the floor planks;
(c) durability requirements of the concrete, and
(d) cracking of the topping due to shrinkage and thermal movement during and after construction.
Steel Construction Institute (SCI) Report P401 [2] indicates that if a structural topping is used, then typically it is possible to increase the resistance of hollowcore planks by between 20% and 60%.

**Figure 2.1 - Examples of precast hollowcore flooring**

![Diagram of precast hollowcore flooring](image)

The requirement for floor diaphragm action also needs to be considered when designing precast prestressed concrete floors. Although there is usually no mechanical fixing between structural topping and precast planks, the two parts can be designed to act compositely provided that the shear stress at the interface meets the requirements of clause 6.2.5 of BS EN 1992-1-1 [3]. Consequently, the reinforcement in the structural topping must be designed for both bending and diaphragm actions. Clause 10.9.3 of BS EN 1992-1-1 explains the principles of how a precast floor can be designed to act as a diaphragm.

**2.1 End Preparation of Hollowcore Planks**

The ends of what would otherwise be standard square-ended planks can be modified during the manufacturing process to facilitate the installation of the planks and the provision of ties.

For example, it is sometimes difficult to install planks onto their supports and so to counter this the ends of the planks can be chamfered or notched (see Figure 2.2). For precast concrete planks less than 300 mm deep typical dimensions for a chamfer comprise a horizontal length of 235 mm and a maximum reduction in unit depth of 85 mm. For deeper precast concrete planks square notches are often used. When designing the notch or chamfer the manufacturer will also check that, in the vicinity of a support, there is sufficient slab remaining to resist any vertical shear forces that may be applied during construction (including those due to the weight of any topping).

**Figure 2.2 - Chamfered and notched ends**

![Chamfered and notched ends diagram](image)

Another example is where the planks need to be tied to the support beams to satisfy disproportionate collapse requirements. The tops of a number of cores (between two and four)
Precast concrete floors in steel framed buildings

can be opened up so that reinforcement may be placed within them (see Figure 2.3). Generally, two adjacent cores should not be opened up, as it will be difficult to preserve the integrity of the chamfered rib between them. Similarly, an outer core should not be opened up since the outer rib would become vulnerable to damage during handling and erection.

**Figure 2.3 - Section through precast concrete planks showing tie reinforcement in the cores**

During end preparation the void at the back of each opened core is blocked with concrete and a weep hole formed to allow any water that enters the unit to drain away (see Figure 2.4).

**Figure 2.4 - Longitudinal section through an opened core**

2.2 Minimum Bearing
SCI Report P401 identifies four factors affecting the actual size of the end bearing for precast planks sitting on a steel beam, namely:

- the nominal bearing length;
- variations in the size and position of the steelwork;
- length variations in the manufacture of the planks, and
- the accuracy with which the planks can be positioned on site.

In its ‘Code of Practice for the Safe Installation of Precast Concrete Flooring and Associated Components’ [4] the Precast Flooring Federation adopted a pragmatic approach to these four factors and stated that, “To allow for manufacturing and construction tolerances it is recommended that the minimum bearing length provided is 75 mm.”

The Code of Practice goes on to recommend:

- Where flooring units are to be installed on shelf angles within beams, the ends should be suitably treated to facilitate the safe installation. This can be achieved via notched ends or chamfered ends.
- The length of the units should be designed to allow a 25 mm clearance gap between the end of the unit and the supporting steelwork.
- Where shear studs for composite action or progressive collapse requirements are provided then the flange width of the supporting beam shall be reviewed and adjusted if necessary.

In some cases it may be possible, by specifying tighter tolerances on both the manufacturing and construction elements, to reduce the nominal bearing length to 50 mm. However, this should only be done in consultation (and agreement) with the precast manufacturer, the flooring installer, the steelwork supplier and the steel erector.
2.3 Allowance for Non-Rigid Supports

As discussed above, precast prestressed planks are usually designed as simply supported elements on rigid supports. However, if the planks are supported on a beam which deflects under applied loading, then the planks will not bear evenly onto the beam (see Figure 2.5) and shear stresses will be generated across the ends of the planks. These stresses will be proportional to the vertical shear forces in the planks due to the applied loading and will be in addition to the stresses within the planks, had they been rigidly supported. The manufacturer should make allowance for the effects of non-rigid supports when checking the shear resistance of the precast prestressed planks.

In most cases precast prestressed planks have sufficient shear capacity to withstand the additional stresses arising from the effect of non-rigid supports. However, if the calculated shear resistance of the precast planks is insufficient, then the following could be considered:

(i) increasing the shear capacity of the planks, or
(ii) increasing the stiffness of the beam.

Option (i) could be achieved by infilling the ends of the planks to a distance equal to their depth, or by providing a structural concrete topping over the planks. Option (ii) could be achieved by providing a heavier or deeper steel section size, or by making the steel and concrete act compositely together by infilling at least half the cores.

The reader should note that if propped construction is used in conjunction with non-rigid supports, then particular care should be exercised since the removal of the props could lead to significant increases in shear stress within the precast prestressed planks.

SCI Reports P351 [5] and P401 both state that, “For unpropped non-composite beams, the influence of support stiffness need not be considered 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 hollowcore planks provided by the manufacturer). For cases when propped construction is used, or when the factored shear force applied to the slab is greater than 0.35V_{Rd}, advice should be sought from the manufacturer of the precast planks.”

In some cases, the effect of fire on the rigidity of beams that are not fire-protected should be considered. This is because the additional stresses in the precast prestressed planks can be magnified under fire conditions due to the higher deformations and curvature of unprotected steel sections that can occur. Premature failure resulting from fire can be prevented by limiting the end shear on the precast planks, as in the normal condition.
2.4 Diaphragm Action
The floor plate is often required to act as a diaphragm transferring lateral forces to shear walls, reinforced concrete cores or the steel frame (see Figure 2.6). This action can be achieved through the following measures:

- utilisation of the shear resistance of the grouted joints between the precast planks;
- provision of a continuous in-situ reinforced structural topping to transfer the in-plane forces (recommended for larger floors or taller buildings);
- provision of ties between the edge beams and the floor planks;
- provision of ties between the floor planks and the shear walls or reinforced concrete cores, and
- provision of transverse reinforcement in composite beams (internal ties) where an in-situ topping is not used.

Figure 2.6 - Diaphragm action in a precast unit floor

The same measures are also appropriate to achieve robustness for avoidance of disproportionate collapse (see Chapter 4).

2.4.1 Shear resistance of grouted joints
Due to the smooth finish that is provided on precast planks, clause 10.9.3(12) of BS EN 1992-1-1 states that the shear strength of the grouted joint between two adjacent precast planks should be taken as 0.10 MPa. For the value of 0.10 MPa to be applicable, crack widths in the grouted joints must be limited and this can be achieved by restraining the perimeter of the floor, i.e., using perimeter ties to ensure that the floor is adequately connected to the steel frame.

As the strength of the concrete in the precast planks will exceed that of the grout in the joints, the grout strength is used to calculate the shear and bearing resistances of the floor plate. SCI Report P351 indicates that the effective depth of a joint may be taken as the depth of the precast planks less 35 mm and so the effective area of a joint may be taken as the length of that joint multiplied by its effective depth. If calculations show that the shear resistance of the joints between adjacent planks is insufficient to resist the forces applied to the floor plate, then a more rigorous approach can be adopted, taking into account aggregate interlock and dowel action provided by reinforcement and ties between the floor elements and the supporting steel structure. In those instances when a more rigorous design approach is required advice should be sought from the manufacturer of the precast planks.
2.4.2 Perimeter reinforcement at floor edges
Steel edge beams are usually designed as being non-composite, with the result that they have a similar section size to those used for internal beams. Lateral loads acting on a building will generate axial loads in these edge beams (see Figure 2.6). Consequently, steel edge beams should be tied into the floor plate for diaphragm action. Provided that they are connected mechanically to the slab using shear studs then the I-beams may be considered to act as peripheral ties and to transfer in-plane forces. Clause 6.6.5.3 of BS EN 1994-1-1 [6] states that U-bars must be placed around the studs to provide effective transverse reinforcement and tying action. These U-bars should be of minimum diameter equal to half the diameter of the shear studs and should be anchored in each filled core (see Figure 2.7).

Figure 2.7 - Edge beam arrangement

Beams within the depth of the floor slab (for example where flooring units are installed on shelf angles within beams) may also be considered to act as peripheral ties.

2.4.3 Beam transverse reinforcement
Reinforcing bars are placed in the opened cores, transverse to the longitudinal axis of the steel beam. For good composite action, the bars must be located a sufficient distance (15 mm) below the heads of the shear studs. Clause 6.6.5.1 of BS EN 1994-1-1 states that bottom reinforcement should be at least 30 mm clear below the heads of the studs. However, this clause is not directly applicable to applications using precast planks as it relates to solid slabs with two layers of fabric. That said, in hollowcore planks the base of a core is usually between 30 mm and 40 mm above the soffit and, therefore, the 30 mm clear distance should be readily attainable (see Figures 2.1 and 2.3).

Table 2.1 summarises the recommendations contained in SCI Report P401 concerning minimum bar sizes for transverse reinforcement:

Table 2.1 - Recommendations for transverse reinforcement
<table>
<thead>
<tr>
<th>Slab Depth</th>
<th>Bar Sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid planks</td>
<td>H10 @ 300 mm centres + A412 fabric reinforcement</td>
</tr>
<tr>
<td>Hollowcore planks (up to 200mm deep)</td>
<td>H12 @ 200 mm to 350 mm centres*</td>
</tr>
<tr>
<td>Hollowcore planks (up to 260 mm deep)</td>
<td>H16 @ 200 mm to 350 mm centres</td>
</tr>
<tr>
<td>*16 mm diameter bars should be provided if partial shear connection is used</td>
<td></td>
</tr>
</tbody>
</table>

When hollowcore planks are used the spacing of the bars should be such that bars are placed in alternate cores. (The opening of two adjacent cores and outer cores should be avoided.) As shear studs are usually fixed at 120 mm to 225 mm centres along the beam, they should not clash with the transverse reinforcement, which should extend at least 500 mm beyond the ends of the hollowcore planks to provide sufficient anchorage in the filled cores. The reader should note that longer straight bars, or even L-bars, may be necessary to satisfy fire conditions.
2.5 Fire Resistance
For downstand composite beams which are fire protected, the transverse reinforcement used to develop composite action should usually be sufficient to provide 60 minutes fire resistance. The bars should be embedded to a minimum distance of 600 mm from the ends of the planks. For periods of fire resistance of 90 and 120 minutes, a concrete topping will usually be required.

A summary of the fire protection requirements for different periods of fire resistance (based on SCI Report P351) is provided in Table 2.2.

Table 2.2 - Fire protection

<table>
<thead>
<tr>
<th>Period of fire resistance (mins)</th>
<th>Protected (with a limiting temperature of ≤ 620°C)</th>
<th>Unprotected</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>No special requirements - effective tying should be provided by a three dimensional tied steel structure.</td>
<td>Provide longitudinal tie reinforcement passing over or through the beams and embedded in joints between precast planks, or Provide a structural topping with fabric reinforcement, or Design for a reduced shear resistance of 0.2 ( V_{Rd} ) and provide effective tying by a three dimensional tied steel structure.</td>
</tr>
<tr>
<td>60</td>
<td>Limit the slab thickness to no more than 265 mm and provide effective tying by a three dimensional tied steel structure, or Provide longitudinal tie reinforcement embedded in joints between precast planks, or Provide a structural topping with fabric reinforcement.</td>
<td>Provide longitudinal tie reinforcement (suspension reinforcement) over or through the beams and embedded in filled cores of the precast planks and design for a reduced shear resistance of 0.2 ( V_{Rd} ). Note: The provision of a structural topping with fabric reinforcement is optional.</td>
</tr>
<tr>
<td>90</td>
<td>Provide longitudinal tie reinforcement embedded in joints between precast planks. Note: The provision of a structural topping with fabric reinforcement is optional.</td>
<td>Provide longitudinal tie reinforcement (suspension reinforcement) embedded in filled cores of the precast planks plus Structural topping with fabric reinforcement.</td>
</tr>
<tr>
<td>120</td>
<td>Provide longitudinal tie reinforcement (suspension reinforcement) embedded in filled cores of the precast planks plus Structural topping with fabric reinforcement.</td>
<td>Provide longitudinal tie reinforcement (suspension reinforcement) over or through the beams and embedded in filled cores of the precast planks and design for a reduced shear resistance of 0.2 ( V_{Rd} ). plus Structural topping with fabric reinforcement.</td>
</tr>
</tbody>
</table>
3 BEAM DESIGN

3.1 Practical Considerations
The design of steel beams that will support precast prestressed concrete planks will be undertaken by the frame designer, possibly in consultation with the precast manufacturer. Before detailed design of the steel beams commences the frame designer will need to address various practical considerations, including:

(i) minimum beam width;
(ii) edge beam details, and
(iii) robustness for avoidance of disproportionate collapse.

Items (i) and (ii) are discussed below, whilst the third consideration is addressed in Chapter 4.

3.1.1 Minimum beam width
The minimum flange width (see Figure 3.1) is determined by:

- the minimum end bearing for precast planks sitting on the beam;
- whether the steel beam is an internal or an edge beam, and
- whether shear studs are present or not, and whether they are shop or site-welded.

Figure 3.1 - Dimensional factors affecting flange width

The Precast Flooring Federation (PFF) states in its ‘Code of Practice for the Safe Installation of Precast Concrete Flooring and Associated Components’ that “to allow for manufacturing and construction tolerances it is recommended that the minimum bearing length provided is 75 mm.”

Steel Construction Institute (SCI) Report P401 indicates that the minimum gap between the ends of abutting precast planks should not usually be less than:

- 70 mm for shop-welded shear studs (allowing 50 mm for placement and compaction of concrete around the studs and up to 10 mm for excess bearing of the precast planks), and
- 85 mm for site-welded shear studs (allowing 65 mm for positioning the welding tool and up to 10 mm for excess bearing).

Taking the minimum bearing length of 75 mm recommended by the PFF, the minimum flange width of a supporting steel beam designed to act compositely is therefore 220 mm for shop-welded shear studs and 235 mm for site-welded shear studs. However, if tighter tolerances on both the manufacturing and construction elements are specified (and agreed by all parties), then the minimum bearing length can be reduced to 50 mm, leading to a minimum flange width of 170 mm.
3.1.2 Edge beams
The detailing of edge beams requires special consideration for a variety of reasons:

- edge beams are usually needed to act as peripheral ties for the purposes of structural robustness (see Chapter 4);
- edge beams are usually connected mechanically into the floor plate so that lateral loads can be transferred to the steel frame (diaphragm action - Chapter 2);
- cladding loads will often act eccentrically on edge beams (although this action should be beneficial in reducing the torsion generated in edge beams by floor loads), and
- deflection limits may be more onerous for edge beams than for internal beams so as to ensure that the cladding system and any finishes perform adequately.

Edge beams are usually designed as being non-composite, with the result that they have a similar section size to those used for internal beams. However, if a composite design for an edge beam is necessary, then SCI Report P401 suggests that the following are required:

- a minimum edge distance of the shear studs, and
- sufficient transverse reinforcement.

Although the design of composite structures using steel and precast concrete falls outside the explicit provisions of BS EN 1994-1-1 some of the provisions in the code would seem applicable to the design of such structures. For a composite edge beam clause 6.6.5.3 of BS EN 1994-1-1 states that if the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300 mm, then the distance from the edge of the concrete flange to the centre of the nearest stud should not be less than six times the diameter of the studs (see Figure 3.2). Using 19 mm diameter studs this corresponds to a distance of 115 mm. Additionally, clause 6.6.5.3 of BS EN 1994-1-1 states that U-bars, with a diameter not less than half the diameter of the shear studs, must be placed around the studs to provide effective transverse reinforcement and tying action.

Figure 3.2 - Composite edge beam

3.2 Beam Design - Construction Stage
For the construction stage, the steel beams that are supporting the precast prestressed concrete planks will need to be checked using BS EN 1993-1-1 [7] for both out-of-balance and balanced loading conditions. These loading conditions are briefly described in Table 3.1.
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Table 3.1 - Construction stage loading conditions

<table>
<thead>
<tr>
<th>Out-of-balance</th>
<th>Balanced</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Unbalanced loads may exist due to the sequence of installation of the precast planks.</td>
<td>• Balanced loads will exist when all the precast planks are installed (provided that they are of equal span on either side of the beam).</td>
</tr>
<tr>
<td>• Steel beams will be laterally unrestrained.</td>
<td>• Steel beams may be assumed to be fully restrained for beam spans less than, or equal to, 500/3 times the precast plank's bearing width, i.e., the width of flange covered by each plank (see Figure 3.1). Typically spans up to 8 m may be assumed to be restrained.</td>
</tr>
<tr>
<td>• Steel beams will be subject to bending and (in some cases) torsion, due to the self-weight of the precast planks.</td>
<td>• Steel beams will be subject to bending due to construction loading plus the self-weight of the precast planks and any topping.</td>
</tr>
</tbody>
</table>

Table 3.2 outlines what needs to be checked for the construction stage (refer to SCI Report P401 for detail). Since designers will normally utilise Class 1 or 2 cross-sections, for economy, Table 3.2 is based on the use of a Class 1 or 2 cross-section.

Table 3.2 - Design steps for the construction stage using a Class 1 or 2 cross-section (refer to SCI Report P401 for detail)

<table>
<thead>
<tr>
<th>Activity</th>
<th>BS EN 1993-1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Out-of-balance loading condition</strong></td>
<td></td>
</tr>
<tr>
<td>Calculate the plastic moment resistance</td>
<td>6.2.5</td>
</tr>
<tr>
<td>Calculate the plastic shear resistance.</td>
<td>6.2.6</td>
</tr>
<tr>
<td>If shear force is high (unlikely for the construction stage), then calculate the reduced moment resistance.</td>
<td>6.2.8</td>
</tr>
<tr>
<td>Calculate torsional resistance</td>
<td></td>
</tr>
<tr>
<td>6.2.7(7) states that for I-beams and H-beams the effects of St Venant torsion can be neglected and so torsional resistance can be calculated using warping alone. Such an approach is simple and if it delivers the required torsional resistance, then there is no need for a more refined approach.</td>
<td>6.2.7 Note 1 Note 2</td>
</tr>
<tr>
<td>If torsion is present, then calculate the reduced plastic shear resistance.</td>
<td>6.2.7(9)</td>
</tr>
<tr>
<td>Consider interaction of torsion with lateral torsional buckling.</td>
<td>Note 3</td>
</tr>
<tr>
<td>Calculate the vertical deflection.</td>
<td>NA.2.23</td>
</tr>
<tr>
<td>Calculate the twist.</td>
<td>Note 4</td>
</tr>
<tr>
<td><strong>Balanced loading condition</strong></td>
<td></td>
</tr>
<tr>
<td>Calculate the plastic moment resistance</td>
<td>6.2.5</td>
</tr>
<tr>
<td>Calculate the plastic shear resistance.</td>
<td>6.2.6</td>
</tr>
<tr>
<td>If shear force is high (unlikely for the construction stage), then calculate the reduced moment resistance.</td>
<td>6.2.8</td>
</tr>
<tr>
<td>Calculate the lateral torsional buckling resistance (unless the beam may be assumed to be restrained).</td>
<td>6.3.2.1 to 6.3.2.4 NA.2.16 - NA.2.20</td>
</tr>
<tr>
<td>Calculate the vertical deflection.</td>
<td>NA.2.23</td>
</tr>
</tbody>
</table>
Notes:

1) 6.2.7(1) does not provide a rule to evaluate torsional resistance in the presence of bending, whilst
   6.2.7(4) states that stresses due to torsion should be taken into account, without explaining how.
   The reader is referred to SCI Reports P385 and P401 for further information.

2) At the ends of a member subject to torsional load, torsional restraint must be provided and the connections designed to resist the forces that provide restraint, e.g., end plates.

3) BS EN 1993-1-1 and the UK NA do not address the interaction of torsion and lateral torsional bucking. However, Annex A of BS EN 1993-6 does address this issue.

4) BS EN 1993-1-1 and the UK NA do not suggest limits for twist. Historically, a 2º limit to the angle of rotation has been used. Consideration should be given as to whether the design situation is transient since some, or all, of the twist might be reversed at a later stage during construction.

3.3 Beam Design - Normal Stage
If adequate shear connection has been provided, then composite action will occur and for the normal stage the beams may be designed according to BS EN 1994-1-1 for both out-of-balance and balanced loading conditions. These loading conditions are briefly described below:

Table 3.3 - Normal stage loading conditions

<table>
<thead>
<tr>
<th></th>
<th>Out-of-balance</th>
<th>Balanced</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Unbalanced loads</td>
<td>• Balanced loads will exist when spans (and loads) on either side of the beam are equal.</td>
<td></td>
</tr>
<tr>
<td>• Beams will be</td>
<td>• Composite beams will be subject to bending due to self-weight (steel and concrete), finishes, services and variable loading.</td>
<td></td>
</tr>
<tr>
<td>subject to bending</td>
<td></td>
<td></td>
</tr>
<tr>
<td>alone, as torsion</td>
<td></td>
<td></td>
</tr>
<tr>
<td>is precluded by the</td>
<td></td>
<td></td>
</tr>
<tr>
<td>presence of the slab.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The frame designer will need to address various aspects of the shear connection between the steel and concrete elements, and the resulting resistance and stiffness of the composite beam, including:

(i) shear studs;
(ii) effective flange width;
(iii) longitudinal shear and transverse reinforcement;
(iv) bending resistance, and
(v) serviceability conditions.

These matters are discussed overleaf.
3.3.1 Shear studs
Clause 6.6.3.1 of BS EN 1994-1-1 presents two equations to determine the design shear resistance of a headed shear stud in a solid in-situ concrete slab. The design resistance is taken as the lesser of the two values determined from these expressions.

\[ P_{Rd} = \frac{0.8 f_u \pi d^2/4}{\gamma_V} \]

and

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

where,
- \( f_u \) is the ultimate tensile strength of the shear stud;
- \( d \) is the diameter of the shank of the shear stud;
- \( f_{ck} \) is the characteristic cylinder strength of the concrete;
- \( E_{cm} \) is the secant elastic modulus of concrete;
- \( \alpha = 0.2 \left( \frac{h_{sc}}{d} + 1 \right) \) for \( 3 \leq \frac{h_{sc}}{d} \leq 4 \);
- \( \alpha = 1.0 \) for \( 4 < \frac{h_{sc}}{d} \), and
- \( h_{sc} \) may be taken as the as-welded height of the shear stud.

Since hollowcore planks and solid planks may perform slightly differently to solid in-situ concrete slabs, a reduction factor, \( k \), needs to be applied to the two equations. In the case of solid planks, the performance in shear of a headed stud is only influenced by the width of the gap between abutting planks. SCI Report P401 suggests that the value of the reduction factor, \( k \), may be taken as 1.0 provided that its recommendations for minimum gap width are followed.

In the case of hollowcore planks the shear performance of a headed stud is influenced by the presence of transverse reinforcement and the geometry of the gap between the ends of adjacent hollowcore planks. SCI Report P401 suggests that the value of the reduction factor, \( k \), should be taken as 0.9 provided that its recommendations for provision of transverse reinforcement and minimum gap width are followed.

Clause 6.6.5.5(3) of BS EN 1994-1-1 states that in buildings the maximum longitudinal spacing of shear connectors should not exceed six times the total slab thickness or 800 mm, whilst clause 6.6.5.7(4) of BS EN 1994-1-1 stipulates that for headed shear studs the minimum longitudinal spacing is five times the diameter of the studs (95 mm for 19 mm studs) and the minimum transverse spacing is four times the diameter of the studs (76 mm for 19 mm studs). Whilst BS EN 1994-1-1 is not strictly applicable to the design of composite structures using steel and precast concrete it would seem reasonable to satisfy these detailing rules.

3.3.2 Effective flange width
For composite construction comprising steel beams and in-situ concrete BS EN 1994-1-1 adopts an effective flange width of \( \text{span}/4 \) (but not exceeding the spacing of the steel beams) with some reduction near to the ends of the steel beams (for ultimate limit state design). Details can be found in Figure 3.3 and in clauses 5.4.1.2 and 6.1.2 of BS EN 1994-1-1.
For composite construction utilising solid precast planks the effective width may be calculated
in a similar manner to that outlined above. However, in the case of hollowcore planks the
effective width of the slab will be affected by the strength of the in-situ concrete and the
amount of transverse reinforcement. Whilst the rules from EN 1994-1-1 may be applicable,
research findings (see Further Reading) indicate that the effective slab width should not
exceed the total width of the concrete infill plus the width of the gap between the abutting
planks.

3.3.3 Longitudinal shear and transverse reinforcement
The concrete flange in the composite beam must be able to resist the longitudinal shear forces
transferred to it by the shear studs. The transfer of longitudinal shear force between the steel
beam and the concrete flange is addressed in clause 6.2.4 of BS EN 1992-1-1 and the required
area of transverse reinforcement per unit length is determined using equation 6.21. This
transverse reinforcement should extend over the effective width of the concrete flange.

It should be noted that where, due to the presence of an edge or an opening, a flange is
asymmetric then the side with the larger flange area will need to resist a greater proportion of
the shear force.

Information regarding the detailing of transverse reinforcement is contained in Chapter 2.

3.3.4 Bending resistance
For a composite beam constructed using a rolled steel section to BS EN 10365 [8] the bending
resistance is usually the plastic bending resistance, as BS EN 10365 rolled sections are normally
Class 1 or Class 2 cross-sections. As explained in SCI Report P401 the magnitude of the plastic
bending resistance is affected by:

(i) whether full or partial shear connection exists;
(ii) the size and strength of the precast concrete effective flange, and
(iii) the size and grade of the steel beam.

With respect to item (ii) if the composite construction uses precast solid planks, then a
minimum of 25 mm should be deducted from the total slab depth because of the lack of
concrete at the bottom edge of the interface between the planks. Furthermore, if there is a
dry butt joint detail between the solid planks, then the depth of the slab should be taken as
only being the thickness of the structural topping as the butt joint will not transfer
compression forces effectively. For composite construction using hollowcore planks the design
should be such that if there is full shear connection then the plastic neutral axis is not located within the hollowcore slab.

3.3.5 Serviceability conditions
Clause 7.3.1 of BS EN 1994-1-1 indicates that the deflection of composite beams should be calculated using elastic analysis. The designer only needs to allow for the extra deflection that occurs due to slip between the steel and concrete elements for very low degrees of shear connection. The standard does not state any deflection limits for composite beams and so limits would need to be selected by the frame designer based on the sensitivity of finishes, cladding, etc.. Typical limiting values for the vertical deflection of composite beams are provided in SCI Report P401.
4 DISPROPORTIONATE COLLAPSE

4.1 Introduction

Disproportionate collapse
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.

This chapter gives clear advice on the measures which may be taken to reduce the sensitivity of a steel framed building with precast concrete floors to disproportionate collapse in the event of an accident.

In Part A of the Approved Documents for England structures are divided into categories depending on the risk factor and the consequences of an accidental failure. These parameters depend on the type of building, the likelihood of accidents and the number of people that may be affected. Buildings are categorised by consequence class as shown in Table 4.1.

Table 4.1 - Building consequence classes (extract taken from Table 11 of Part A)

<table>
<thead>
<tr>
<th>Class</th>
<th>Building type and occupancy</th>
<th>Action required</th>
</tr>
</thead>
</table>
| 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 | No additional measures |
| 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 2000m² 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 2000m² at each storey | Effective anchorage of floors to supports in accordance with NHBC guidance and BS EN 1996-1-1 for horizontal ties |
| 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 and which contain floor areas exceeding 2000m² but less than 5000m² at each storey  
       | • Car parking not exceeding 6 storeys | Horizontal ties to be provided together with either vertical ties or allowance made for the notional removal of support |
Table 4.1 (continued)

<table>
<thead>
<tr>
<th>Class</th>
<th>Building type and occupancy</th>
<th>Action required</th>
</tr>
</thead>
</table>
| 3     | • All buildings defined above as consequence Class 2A and 2B that exceed the limits on area and/or number of storeys  
       • All buildings containing hazardous substances and/or processes  
       • Grandstands accommodating more than 5000 spectators | Specific consideration to take account of the likely hazards |

4.2 Design Aspects

The specific requirements relating to ties in precast concrete structures are given in Part 1-1 of Eurocode 2: Design of concrete structures. These are satisfied either by using individual continuous ties provided explicitly for this purpose in in-situ concrete strips, or using ties partly in the in-situ and partly in the precast planks. Figure 4.1 defines types of floor ties, whilst Figure 4.2 illustrates the use of ties within hollowcore floors.

From a structural viewpoint, when considering notional removal of support the objective is that in the event of the complete loss of a supporting column, beam or nominal length of load-bearing wall 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 100 m², whichever is smaller, and does not extend further than the immediate adjacent storeys.

Figure 4.1 - Definition of Floor Ties

- Peripheral tie: tie over a peripheral support.
- Internal tie: tie over an intermediate support perpendicular to the span of the floor.
- Internal floor ties: ties connecting floors over an internal support.
- Perimeter floor ties: ties connecting floors to a perimeter support.
- Vertical ties: ties connecting vertical walls or columns to provide continuity.

Figure 4.2a - Plan view of hollowcore floor

- Width at tie bar level ≥ max of (Ø + 20) or (Ø + 2 dₐ), where dₐ = max aggregate size

Figure 4.2b - Section through hollowcore
4.3 Typical Details for Hollowcore Floors
Where a structural concrete topping is used, opening of cores for tying purposes may not be required since the reinforcement in the topping is usually sufficient to comply with the tying requirements. However, the specific design must be checked by the frame designer. Some brief points regarding detailing are listed below.

- The minimum bearing of slabs should be in accordance with Part 1-1 of Eurocode 2: Design of concrete structures.
- The recommended maximum aggregate size in the in-situ concrete is 10 mm.
- The recommended minimum compressive strength class of the in-situ concrete is C25/30.
- Opening of edge cores or two adjacent cores is not recommended.
- The recommended maximum length of an open core is 600 mm.
- Notched and chamfered ends should be in accordance with the manufacturer’s details.
- Tying reinforcement may be placed in the joints provided that there is sufficient space.
- The design of all tying reinforcement is the responsibility of the frame designer.
- Where longitudinal tying reinforcement is provided, no additional reinforcement is required at the slab edges.

Figure 4.3 shows some typical details for tying. Further detailed information can be found in SCI reports P341 [10], P351 and P391 [11].

Figure 4.3a
Tying precast planks over an internal non-composite beam with structural topping.

Figure 4.3b
Tying precast planks over an internal beam with internal floor ties placed in opened cores.

Figure 4.3c
Tying precast planks to an edge beam with perimeter floor ties (U-bars) placed in opened cores.
Precast concrete floors in steel framed buildings

Figure 4.3d
Tying precast planks supported on an internal beam with shelf angles using cranked internal floor ties placed in opened cores.

Figure 4.3e
Tying precast planks supported on an edge beam with a shelf angle using cranked perimeter floor ties placed in opened cores.
GLOSSARY

**Class 1 cross-section** - Steel beams with Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance. A Class 1 cross-section is the least susceptible to local buckling.

**Class 2 cross-section** - Steel beams with Class 2 cross-sections are those which can develop their plastic moment resistance but have limited rotation capacity because of local buckling.

**Diaphragm** - a structural element (such as a floor slab) that transmits lateral load to other elements of a structure (such as shear walls or frames). Note that a shear wall is a vertical diaphragm that can transmit forces into floors and foundations.

**Disproportionate collapse** - a collapse which is judged (by some measure defined by the observer) to be disproportionate to the initial cause.

**Fabric reinforcement** - a welded prefabricated joined grid consisting of a series of parallel longitudinal wires welded to cross wires at a set spacing; also known as mesh reinforcement.

**Hollowcore plank** - a monolithic prestressed or conventionally reinforced precast concrete element with a constant overall depth and incorporating continuous longitudinal voids to reduce self-weight whilst providing an efficient structural section.

**Robustness** - the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error without being damaged to an extent disproportionate to the original cause.

**Tendon** - these are typically high strength steel wires in prestressed precast concrete planks.
REFERENCES


[8] BS EN 10365: Hot rolled steel channels, I and H sections. Dimensions and masses


FURTHER READING

BS EN 1992-1-1 UK National Annex to Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules for Buildings

BS EN 1993-1-1 UK National Annex to Eurocode 3: Design of Steel Structures - Part 1-1: General Rules for Buildings


The Institution of Structural Engineers (2010) Practical guide to structural robustness and disproportionate collapse in buildings, London, UK: The Institution of Structural Engineers

The Institution of Structural Engineers (2013) Manual for the systematic risk assessment of high-risk structures against disproportionate collapse, London, UK: The Institution of Structural Engineers


Precast Flooring Federation (2016) Building Regulations Approved Document A (Datasheet), Leicester, UK: Precast Flooring Federation

Precast Flooring Federation (2016) Hollowcore Floors (Datasheet), Leicester, UK: Precast Flooring Federation

Precast Flooring Federation (2016) Hollowcore Composite Floors (Datasheet), Leicester, UK: Precast Flooring Federation

Precast Flooring Federation (2016) Lattice Girder Composite Floors (Datasheet), Leicester, UK: Precast Flooring Federation
