HOLLOWCORE FLOORING

TECHNICAL MANUAL

NPCAA

National Precast Concrete Association Australia

Hollowcore units are machine-made in a factory. They are nominally 1200 mm wide, range in thickness from 150 mm to 400 mm and come with a variety of core configurations depending on the type of production equipment

Hollowcore Floor Planks feature:

- *Speed of construction*
- *Immediate work platform*
- *Elimination of formwork or propping*
- *Reduction of on-site labour*

Hollowcore Floor Planks, as a structural unit:

- *Are lightweight and efficient*
- *Allow design flexibility*
- *Have high load capacity*
- *Are durable*
- *Allow long spans*

Hollowcore Floor Planks, as a floor system, provide:

- *Required fire ratings*
- *Sound insulation*
- *Complete package (manufacture, delivery and installation)*
- *Accommodation of services*
- *Flexible ceiling options*

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CONTENTS

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For manufacturers and other information on hollowcore, see the NPCAA web site: **www.npcaa.com.au**.

1 INTRODUCTION

Hollowcore floor planks are precast, prestressed elements produced on a long-line bed using an extrusion or slide-forming machine. Planks are normally 1200 mm wide, thicknesses range from 150 mm to 400 mm and cores vary in shape, size and number depending on the equipment used for manufacture. Hollowcore floor units are referred to as *planks*, *panels* or *slabs* interchangeably, the first term has been adopted in this Manual to distinguish them from wall panels in a companion manual.

The casting bed lengths are generally between 120 m to 170 m and made of steel. The beds are usually heated from underneath to shorten the curing cycle and ensure daily production. The planks are saw-cut on the bed to the required lengths.

A thin topping of concrete is often placed on-site to produce composite units and a level floor surface, but the planks can just as readily be used with grouted joints only.

Hollowcore planks, as a flooring system, have many of the advantages of precast products in general as well as some unique to themselves.

- **Machine-Made in Factory** Hollowcore floor planks are manufactured by machine in a factory, cured, cut to length and stored ahead of the construction schedule, ready for immediate delivery to the project.
- **Speed of Construction** They are rapidly installed with minimal equipment and labour, thus reducing construction time. The resulting time saving reduces risk, site costs and financial charges.
- **Immediate Work Platform** They provide an immediate and safe working platform for following trades. They can be designed to accommodate high construction loads without propping
- **Eliminates Formwork or Propping** Hollowcore planks do not require propping during construction. Clear and unrestricted access is immediately available to following trades.
- **Reduced On-Site Labour** A small erection crew can install up to 1,000 m^2 of hollowcore floor planks per day.
- **Efficient, Lightweight Section** The hollow voids and prestressing result in reduced dead load for a given strength. The depth of plank and the strand pattern can be varied at minimum cost to suit the span and load requirements.
- **Design Flexibility** Hollowcore floor planks can be used in combination with most building materials including masonry walls, precast or insitu concrete walls and beams, prestressed concrete or steel beams. Hollowcore planks can accommodate most building requirements including openings, angles and cantilevers.
- **Durability** Concrete used for the production of hollowcore meets the durability requirements of the Australian Standard AS 3600. Strand cover may be varied to suit particular exposure conditions.
- **Long Span** Hollowcore planks can accommodate long spans, resulting in column-free open space. Clear spans of up to 20 metres can be economically achieved.
- **High Load Capacity** Hollowcore planks are capable of handling the heavy loads required in shopping centres, car parks, offices, apartments, housing and warehouse structures at minimal floor depths. Some sections are able to be used as bridge members.
- **Fire Resistance** Fire resistance levels up to the regulatory maximum of 240/240/240 can be provided.
- **Sound Insulation** Hollowcore floors reduce the amount of transmitted noise. $R_w + C_{tr}$ and $L_{n,w} + C_1$ ratings, as specified in building codes, can be met for a variety of occupancy requirements.
- **Pre-Finished Ceilings** Hollowcore planks provide flat soffits, which will readily accept painted or applied textured finishes. Alternatively, the soffits can be directly lined in plasterboard or have a suspended ceiling.
- **Services** The cores in the planks can be used as service ducts to conceal plumbing, electrical and telephone cables. Large openings are pre-cut by the manufacturer during production whilst small penetrations are cut or cored on site.
- **Complete Package** Hollowcore floor planks can be manufactured, delivered and installed on site by the manufacturer as a complete package.

2 DESIGN APPROACH

The design of a hollowcore floor is usually undertaken in two stages:

- **Preliminary Design** The general layout, the overall dimensions of the planks and the typical details are selected to suit the building requirements, and
- **Final Design** The details of the planks such as strand patterns, connections, embedded items are decided and the shop drawings produced.

It is normal for the manufacturer to participate in the design process with the project management team as well as providing advice on costing. The capacity to participate varies with each manufacturer and from job to job. The responsibilities are generally divided as follows.

The project management team, which includes the Architect and Structural Engineer, usually provides the following:

- The general arrangement drawings showing the floor plan layout, building dimensions and general structure and support methods.
- The project specification.
- The codes and particular building regulations governing the project.
- The vertical and horizontal loadings on the floor.
- The required fire resistance levels.
- The noise insulation requirements.
- The vibration characteristics if required.
- Any deflection restrictions.
- Forces arising from structural frame actions.
- Checking and acceptance of the design calculations when the manufacturer carries these out.
- Overall responsibility for the structural design of all elements in the project including the integration of the hollowcore planks into the structure.

The manufacturer usually provides the following:

- Detailed specification for the manufacture of the planks using proprietary equipment.
- The detailed design of the hollowcore planks as agreed.
- Detailed layout drawings locating each plank type in the structure.
- Support and joint details.
- Product drawings showing plank details including dimensions and the location of lifting and fixing inserts.
- The erection procedure.

The structural design of the planks may be provided by either party, depending on the contractual arrangements:

- If carried out by the Structural Engineer, the manufacturer supplies design properties unique to his product, such as section properties, normal concrete strengths, strand patterns.
- If carried out by the manufacturer all design documentation is provided to the Structural Engineer who checks that the design meets the project requirements in all respects.

3 PRE-PLANNING

In planning a structure considerable benefits are achieved by establishing a framing layout to suit the hollowcore module and span capability, rather than substituting hollowcore for a traditional insitu concrete layout. The most economical grid in a framed structure maximises the span of the hollowcore and minimises the span of the support beams. Most buildings have at least one short grid (usually 7.2 to 8.4 m) which can be used as the beam span and then the span of the planks follow.

The primary consideration in developing a framing plan is the span length. For a given loading, fire rating and exposure classification, the span length and the plank thickness can be optimised using a manufacturer's published load tables as a guide.

It is strongly recommended that a manufacturer be consulted during the preliminary stages of a project so that advice can be provided on the most cost-effective and practical design.

Floor Depth

For preliminary design the general load-span graphs in **Figures 2** to **7** can be used to determine a suitable plank depth. To limit the deflection of a floor slab and its sensitivity to vibration, the ratio of span to overall depth is usually kept in the range of 30–35 but can be up to 40. Roof slabs which are not subject to foot traffic can range from 35–45. For handling, the ratio should not exceed 45.

The slab thickness may need to be increased if deflection and vibration are critical such as when there are sustained live loads, rhythmic action, masonry partitions or a large number of openings. Slender planks subject to light repetitive loading such as foot traffic in quiet environments should be checked for perceptible vibration. High fire ratings increase cover and may require an increase in overall depth. It is not practical to use continuity in the structure to minimise plank depth.

Plank Width

It is preferable that the floor plan dimensions suit a 1200-mm module width. Never the less, non-modular dimensions can be accommodated with longitudinally-cut planks or wet-cast panels. Alternatively an insitu makeup strip can be used.

Plank Length

The planks are cut to the length required for their location in the floor-plan. The ends can be cut to an angle to suit skew layouts.

Connections

The type and details of the connections between hollowcore floor planks and supporting beams or walls should be chosen in consultation with a manufacturer. Refer to **Figures 16**, **17**, and **18** for examples of connections that have been developed for typical situations.

Tolerances

Construction tolerances must be allowed for in developing a plan layout. Tolerance in plank length is absorbed by allowing a gap at the end of the plank and specifying the minimum length of bearing. A clearance gap is also required along the side of a plank that abuts a wall or beam to allow for an accumulation of width tolerances. Small increases in floor width can be accommodated by increasing joint width with the use of a strip of former to bridge the base of the joint during grouting, **Figure 15**.

Camber and Topping

Hollowcore floor planks are cambered because of the upward bending induced by the prestressing, **Figure 1**. This camber should be allowed for in detailing the planks and the joints at abutting walls, door openings and the like. A site-cast topping unifies the planks into a monolithic floor, takes out differential levels between units and provides a level working surface. The minimum thickness of the topping occurs at the highest point on the plank, usually at the centre. The minimum average thickness of a structural topping is 50 mm (AS 3600, Cl 8.4.6). For practical purposes, 60 mm topping is used for up to 200 mm deep planks and 80 mm for planks 300-mm and above. Typical reinforcement is SL62 mesh. Non-structural toppings are usually self-levelling mortars averaging 10–15 mm in thickness.

It should be clear on the structural drawings where the topping thickness is to be measured and whether it should follow the camber of the plank. The drawings should show the location of construction joints. Where joints are saw-cut the cut should be made within 18 hours of casting the topping.

4 PRELIMINARY DESIGN

1 Determine a **feasible arrangemen**t for columns, walls and beams with a minimum number of dependant load paths. Note the preferred options for structural efficiency in **Chapter 3**.

2 Establish the **basic design data**:
- Occupancy of the structure
- Fire rating (building regulations)
- Sound transmission class (building regulations)
- Exposure classifications and durability requirements (AS 3600).

3 Determine the **minimum slab thickness** from fire rating and sound transmission class. Determine the **minimum concrete strength and cover** from fire rating, exposure classification and durability requirements.

4 Select a suitable **overall depth of plank and topping** to satisfy deflection control from the guideline values. Typical span to depth ratio for floors is 30–35, 35–40 for roofs. Floor span to depth ratios in excess of 30 should be checked for adequate vibration stiffness. The thickness of an insitu topping may be determined by the cover to reinforcement at laps.

5 Determine the **dead and live loads** (AS 1l70). Local loads such as masonry partitions and wheel loads distribute themselves across a number of planks where possible (**Figures 8** to **11**). Machinery vibrations and such like, may need to be considered. Construction loads must also be taken into account.

6 Check the **strength capacity** of the plank from the preliminary selection load-span graphs in **Figures 2** to **7**. The graphs are based on typical minimum and maximum strand quantities. For economy, a particular plank depth would be used between these lines. A span/load combination cannot be to the right of the rightmost (maximum) line. Core configuration usually does not affect flexural capacity significantly but does affect shear capacity. Always confirm the shear capacity for a particular configuration.

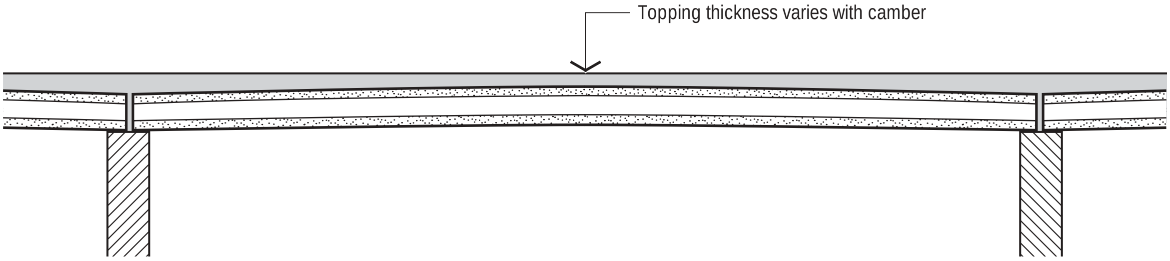


Figure 1 *Allowance for Camber*

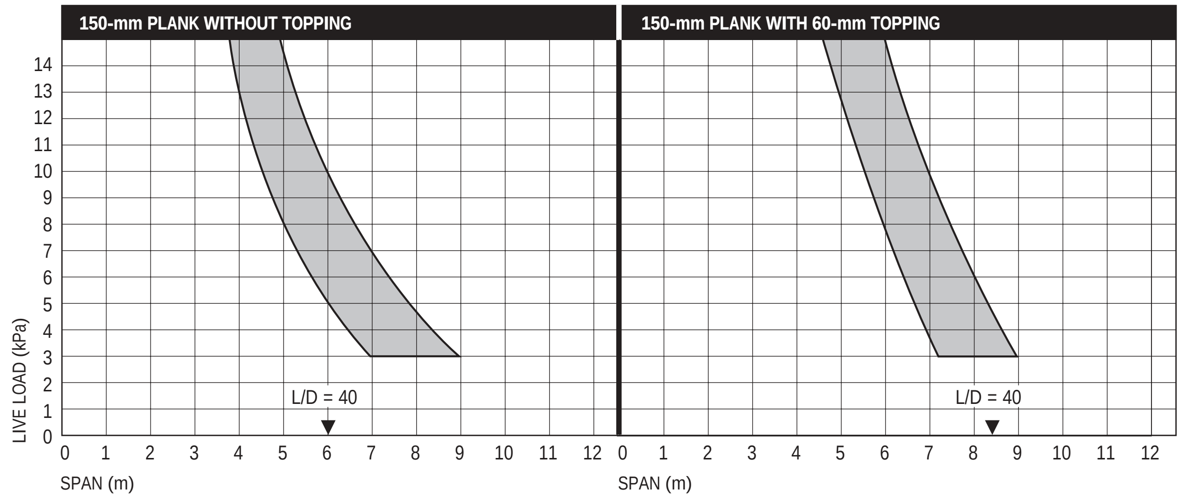


Figure 2 *150-mm Hollowcore Planks – Live Load Capacity*

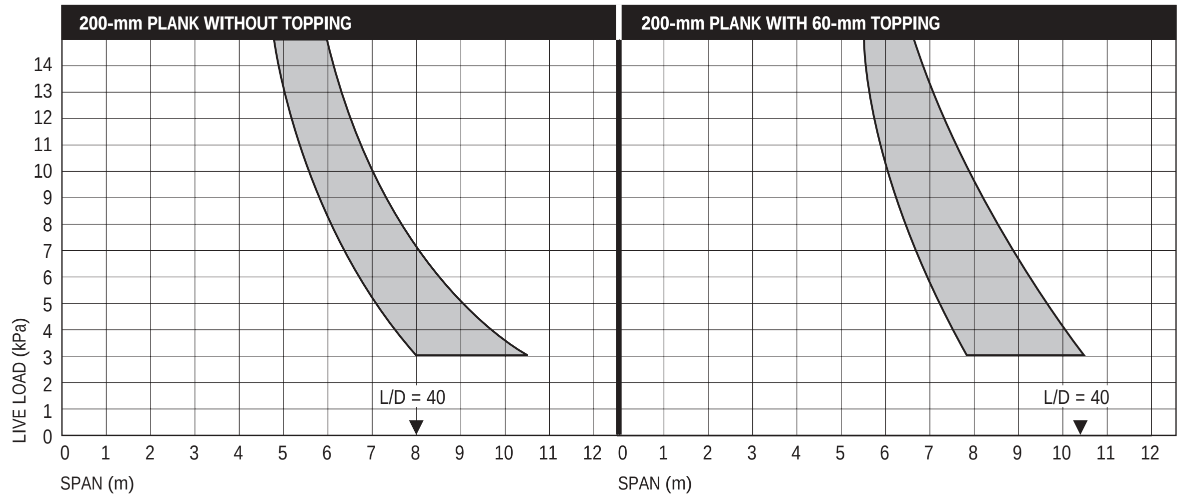


Figure 3 *200-mm Hollowcore Planks – Live Load Capacity*

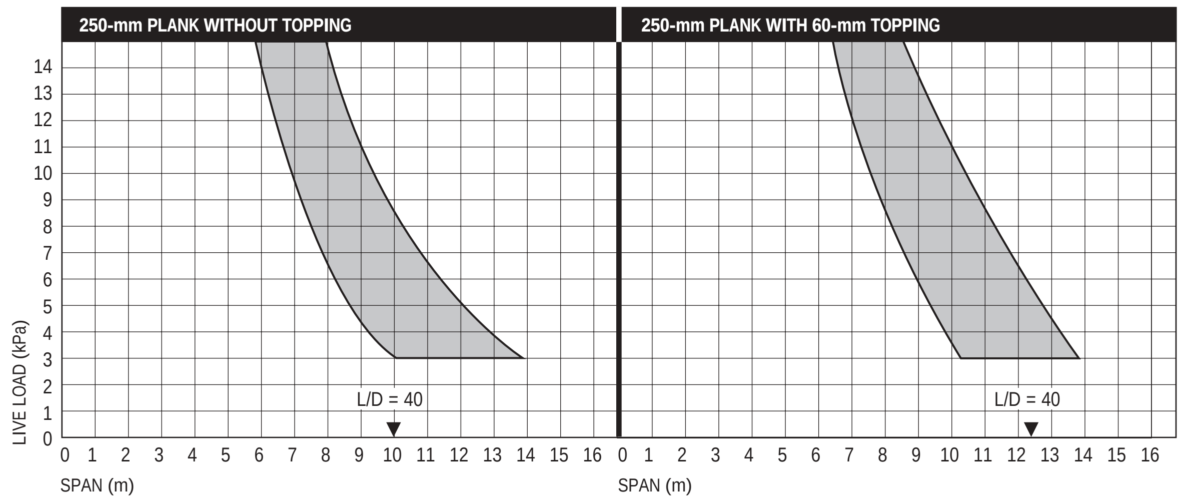


Figure 4 *250-mm Hollowcore Planks – Live Load Capacity*

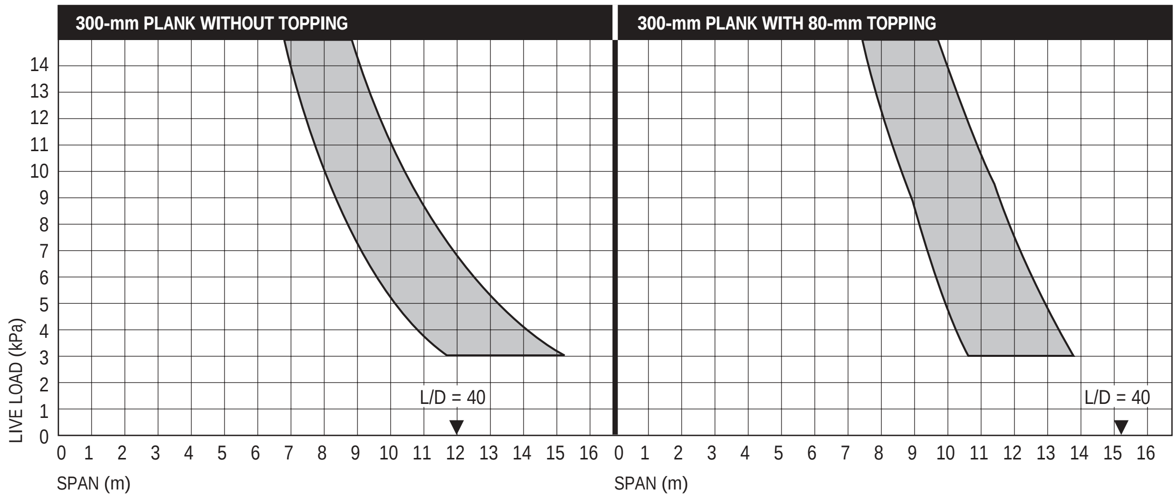


Figure 5 *300-mm Hollowcore Planks – Live Load Capacity*

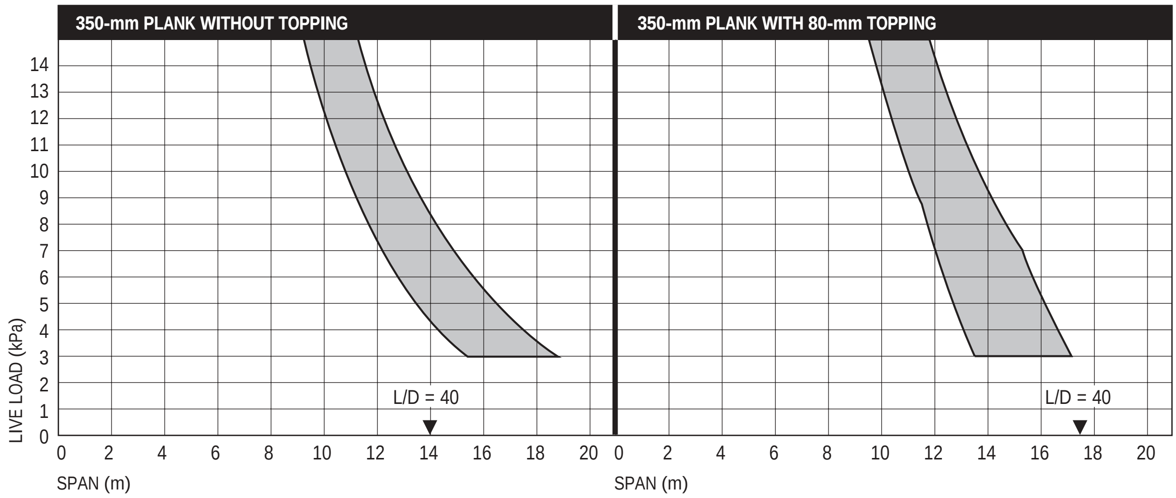


Figure 6 *350-mm Hollowcore Planks – Live Load Capacity*

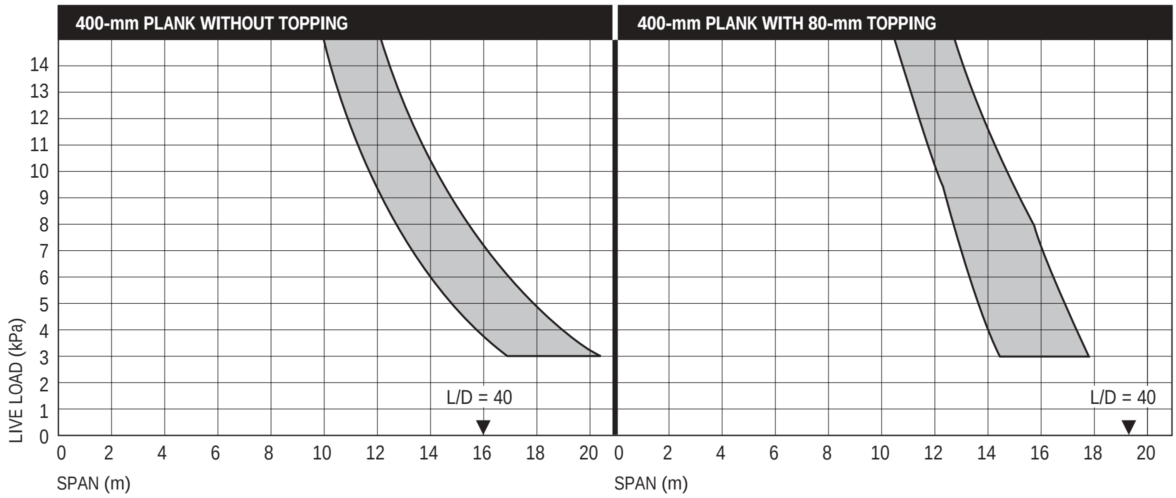


Figure 7 *400-mm Hollowcore Planks – Live Load Capacity*

5 FINAL DESIGN

5.1 General

The detailed design of a hollowcore plank follows the normal procedure for a prestressed member (see *Sample Calculation* **Chapter 9**). The plank is checked for design ultimate strength in bending, shear and at transfer, and at service load for crack control, camber at time of erection and deflection. For the standard conditions of uniformly distributed live load, the manufacturer's load tables take into account these design criteria and show the load capacity based on the governing criteria. For non-uniform load cases or non-standard conditions separate calculations are required. The design steps are outlined in the following sections.

5.2 Section Availability

Each hollowcore manufacturer has a standard set of cross-sections that can be produced by their machine. Depths vary from 150 to 400 mm. The core configuration makes each system unique, actual depths vary between manufacturers and not all are available in each system.

Hollowcore manufacturers prepare and distribute product literature that includes section properties and capacities and load span tables for their product range. These are calculated for the particular planks, strand configuration and effective depth of the section with concrete cover to meet designated fire rating and exposure classifications.

For the purpose of this Manual, typical section plank capacity charts are provided as an aid for preliminary sizing and to illustrate the procedures for a detailed final design. In practice, the amount of prestress is varied to suit the load within each plank size. Top strands are required in 350-mm and 400-mm deep sections (3 x 12.7-mm dia.) to control the stress distribution. Concrete strengths range from 40 to 65 MPa and topping strength is usually 32 or 40 MPa.

For convenience, the load-span graphs (**Figures 2** to **7)** are presented in terms of the allowable superimposed distributed live load applied to a simply-supported span. Where added dead loads or concentrated loads are applied to the floor, these can be converted to equivalent uniform live loads for a preliminary design. Refer **Figures 8** to **11**.

5.3 Flexural Strength

The design ultimate bending strength of the section is calculated in accordance with Clause 8.1 of AS 3600. The design strength ϕM_u may be calculated using the rectangular stress block for compression taking into account any area reduction due to the presence of cores. The maximum stress in the strands at ultimate may be determined by either strain compatibility or by the approximate formula in Clause 8.1.5. of AS 3600. The capacity reduction factor, ϕ, is 0.8. The strand length to develop the prestress by bond is 60 strand diameters. The bond length to develop the stress at ultimate flexure is approximately 150 diameters.

To ensure a ductile failure, upper and lower limits are placed on the area of prestressing steel. A minimum area of steel has to be provided so that the ultimate capacity of the section is not less than 1.2 times the cracking moment. This ensures that when the section cracks the strand will not simultaneously fail. The maximum quantity of prestress steel is controlled by the requirement that the neutral axis depth does not exceed 0.4 times the effective depth of the section. In practice over-reinforced hollowcore sections are precluded by the number of strands that can be bonded between the cores. The compression zone of composite sections is usually above the core zone of the plank.

5.4 Shear Strength

Hollowcore planks are designed for shear in accordance with Clause 8.2 of AS 3600. Stirrups cannot be provided in hollowcore planks and the capacity of the section is controlled by concrete strength. The capacity in both flexure-shear cracking and web cracking is calculated and the lesser value taken as ϕV_{uc} where $\phi = 0.7$. Negative moments can reduce the shear capacity by causing premature cracking in the web. Precautions need to be taken in the design to ensure that this does not happen, see **Clause 5.9** *Continuity*.

The shear capacity of given plank sizes varies between manufacturers due to different void configurations. These affect the web width at the neutral axis where shear stress is a maximum. The load capacity charts in this Manual assume that shear is not critical for the load ranges shown, this may not be the case for all types of cross-section, particularly those with large circular voids and needs to be checked with the individual manufacturer.

In addition to the check for vertical shear, the longitudinal shear strength of a composite section must be checked at the interface between the plank and the insitu topping in accordance with Clause 8.4, AS 3600. When the surface texture at the top of the plank is as-screeded by the machine, a shear plane coefficient of 0.2 may be used in the calculation and 0.4 when the surface is intentionally roughened or wire brushed.

5.5 Transfer Stress

When the strands are cut and apply the prestress force to the concrete, only the self-weight of the plank acts to counter the upward bending moment due to eccentricity. The strength of the section should be checked for crushing or cracking of the concrete. Although Clause 8.1.4 of AS 3600 specifies a maximum compressive stress in the concrete at release of 0.5 times the cylinder strength, a factor of 0.6 has been found to be satisfactory. In practice this limiting stress is met by choosing the release strength of the concrete. The tensile stress in the section at release is not critical for the strand patterns normally used.

5.6 Crack Control

Flexural cracking in a prestressed slab may be controlled by limiting the tensile stress in accordance with the provisions of Clause 9.4.2. of AS 3600. As the strands in hollowcore planks are fully bonded, a calculated flexural tensile stress under short-term service loads up to $0.5\sqrt{f'_c}$ may be used in the design. This means that the section is effectively uncracked at normal service loads.

Alternatively, the more general provision for partial prestressing may be adopted whereby the increment in strand stress is limited to 150 MPa as the load increases from the decompression value to the short-term service load.

5.7 Camber and Deflection

Camber is the upward deflection of a prestressed member and results from the prestressing force being eccentric to the centre-of-gravity of the section. It is therefore determined by the design loading and span for a given depth of plank and should not be specified as a limiting value. Unloaded planks tend to creep upwards when the self-weight of the plank is insufficient to balance the prestress. To avoid unnecessarily-high cambers, it is important not to over-specify the design loading or arbitrarily increase the prestress.

When the design live load is high, the resulting camber can be reduced by using additional top strand to give a less triangular stress distribution over the cross-section.

The initial camber is calculated from the formulae for elastic deflection due to the prestressing moment and the self-weight using appropriate values for the prestressing force and the modulus of elasticity at release. As these are known with reasonable accuracy the elastic deflection can be predicted fairly well. However, time-dependent camber and deflection calculations are only estimates.

The camber at erection due to prestress can be estimated using a creep factor of 1.7 to 2.0 times the initial camber.

Deflection results from the applied loads. If the section is uncracked the deflection depends on the stiffness of the gross concrete section and is independent of the amount of prestressing. Usually hollowcore planks are designed to be uncracked under service loads so the gross moment of inertia may be used, otherwise the effective moment of inertia of the cracked section must be taken into account.

Camber and deflection change with time due to concrete creep, loss of prestress and other long-term factors such as climate. A long-term load in excess of the load balanced by the prestress causes the plank to creep downwards.

Composite action with a topping results in a stiffer section. The deflection of interest is the movement from the as-cast level of the topping resulting from the long-term effect of the sustained load and the instantaneous effect of the short-term load. Limiting values for these deflections are specified in AS 3600, Table 2.4.2. As an aid to estimating deflection, long-term multiplication factors are given in reference 4, **Chapter 8** and are used in the Sample Calculation, **Chapter 9**.

5.8 Load Distribution

Hollowcore floor planks are usually designed as simple, one-way-spanning slabs. Floors are often subject to non-uniform loads such as line loads, concentrated loads or loads at openings. The shear/torsion interaction between planks with properly-filled side joints and lateral restraint against spreading, allows these concentrations to be shared by several planks.

This ability to distribute loads has been demonstrated by full scale testing and explained by a combination of shear stresses induced in the grouted keyways and by transverse bending in the planks. For design purposes, an effective load-resisting width can be used to calculate design bending moments and shears. This effective width depends upon the span.

A simple linear distribution is shown in **Figure 8**. The FIP method (see Item 3, **Chapter 8**) is a more detailed approach and is shown in **Figures 9, 10** and **11** for both point and line loads. Load sharing decreases near an edge or large opening and a conservative approach must be taken to re-distribution in these regions, particularly for large edge loads on long spans. When in doubt a finite element analysis can be carried out for more insight into the distribution between the units.

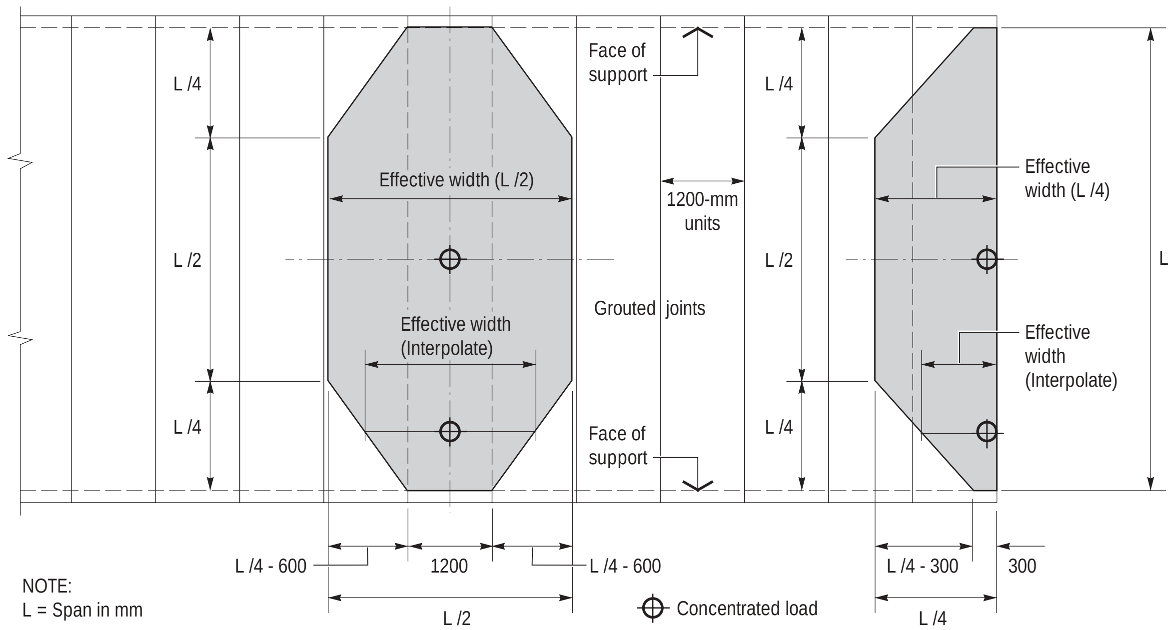


Figure 8 *Linear Load Distribution of Concentrated Loads (PCI)*

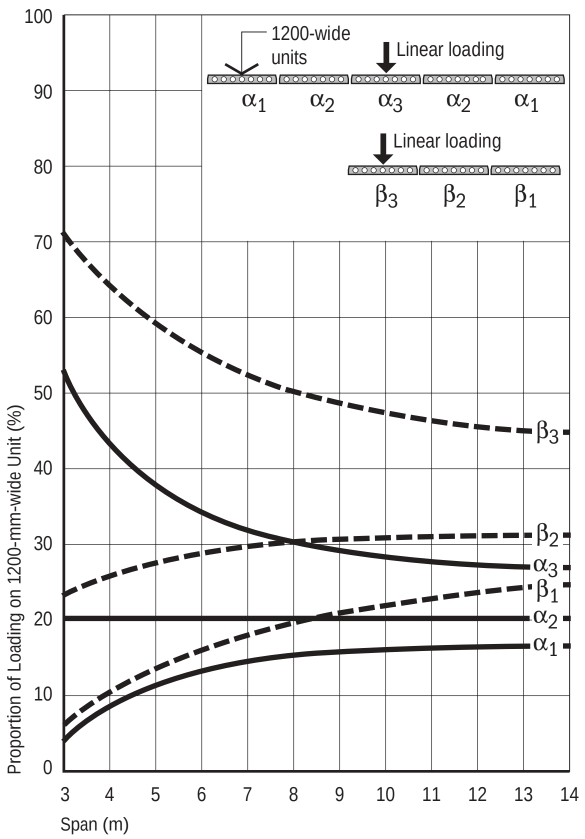


Figure 9 *Load Distribution Coefficients – Centrally-Placed Line Load (FIP)*

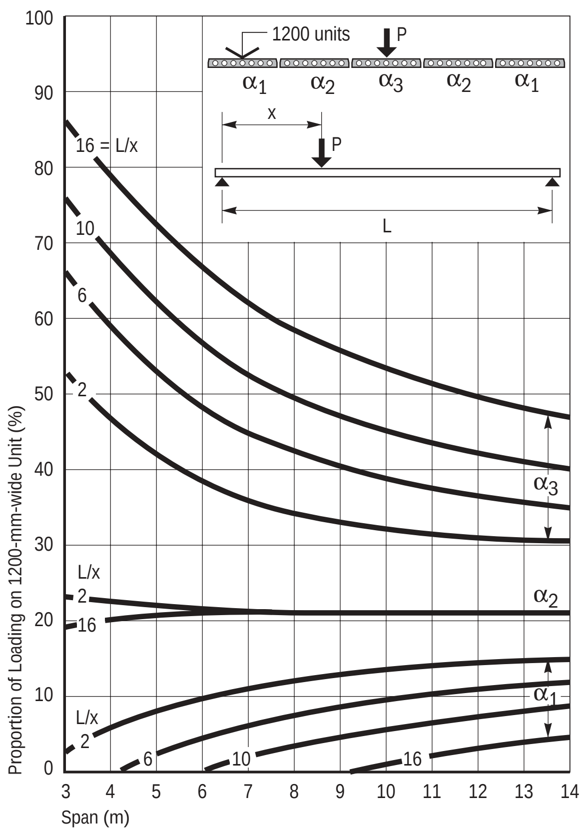


Figure 10 *Load Distribution Coefficients – Point Load (FIP)*

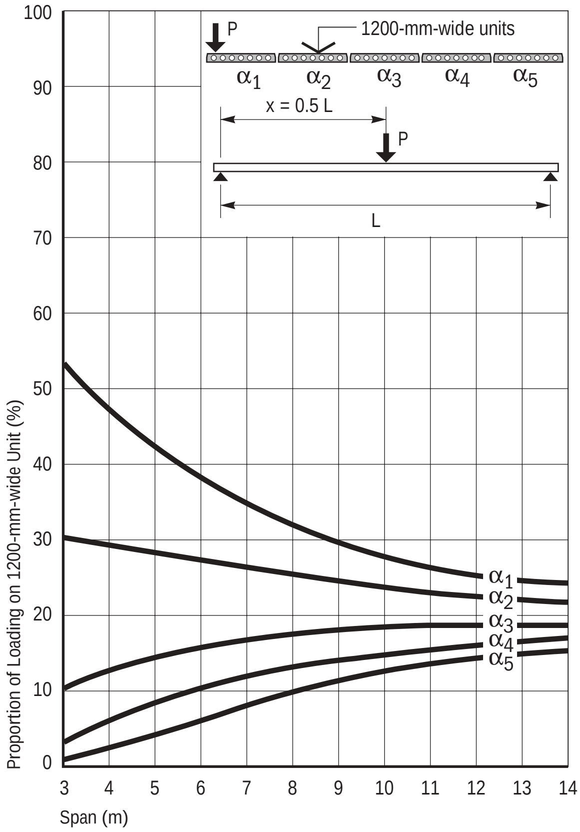


Figure 11 *Load Distribution Coefficients – Point Load at Edge (FIP)*

5.9 Continuity

The most economical use of planks in a framed structure is as simply-supported members in a layout which has a larger plank span than support beam span. Sufficient tie steel (which will give a degree of continuity) as described below must be supplied whether the planks are topped or not.

Care must be taken not to develop unintentional negative moments at the support of untopped planks. There may already be tension here due to prestress eccentricity and additional tension could initiate web cracking. Neglect of correct detailing could lead to shear failure at unexpectedly-low loading. Two typical causes of negative moments are:

- Grout from the butt-end joint penetrating into the cores and creating a stiff end-length. Core dams should always be located within 50 mm of the end of the plank.
- The ends of the planks clamped between bearing walls, restraining rotation. A clearance or compressible material can be used to prevent the upper wall bearing on the plank.

Limited continuity can be achieved by placing reinforcement in the topping concrete at supports. Continuity should only used where there is a clear advantage to the structure to do so, and then, only in composite construction. The cost per kN of force supplied by prestressing strand is about a third that of reinforcement. Thus, it is uneconomical to use ordinary reinforcement as negative-moment steel to replace positive-moment strand steel.

Continuity can be used to achieve a higher fire-rating for a given cover to the strand; this is useful when it is not possible, or inadvisable, to raise the strand any higher in the cross-section. Continuity is also useful for increasing the robustness of a structure to catastrophic overload. The following points should be observed:

- Do not interchange the sections of different manufacturers without re-calculating the shear capacity. Web widths between different core profiles can vary by up to 60% with a similar effect on the shear capacity.
- Core filling may be required to make up shear capacity. Check flexural tension.
- Check that the strand eccentricity does not exceed the cracking moment in the top surface at the end of the plank. If necessary, provide top strand in the plank (two or three is usually sufficient).
- Extend negative reinforcement in the topping well past the expected point of contraflexure to ensure that uncontrolled cracking cannot penetrate into the precast unit.

5.10 Cantilevers

Cantilevers are a special case of continuity over a support. Frequently untopped planks are used in this way, **Figure 12**. For the cantilever portion of the plank, the bottom strands add to the bending moment at the support. Depending on the system of manufacture, it may be possible to de-bond some of these strands over the length of the cantilever. To control deflection, the length of a cantilever is limited to 8 to 10 times the plank thickness.

Where a composite topping is used, normal reinforcing bar or fabric is provided in the topping layer for the negative moment.

If the floor is untopped then sufficient top strands are required in the plank to meet ultimate shear and flexure requirements and ensure that the top surface is in residual compression under service loads. However, for short cantilevers (say up to 750-mm) bars may be grouted in the cores or joints between planks. Both the construction loads and final service loads must be considered. Some temporary propping may be required during construction.

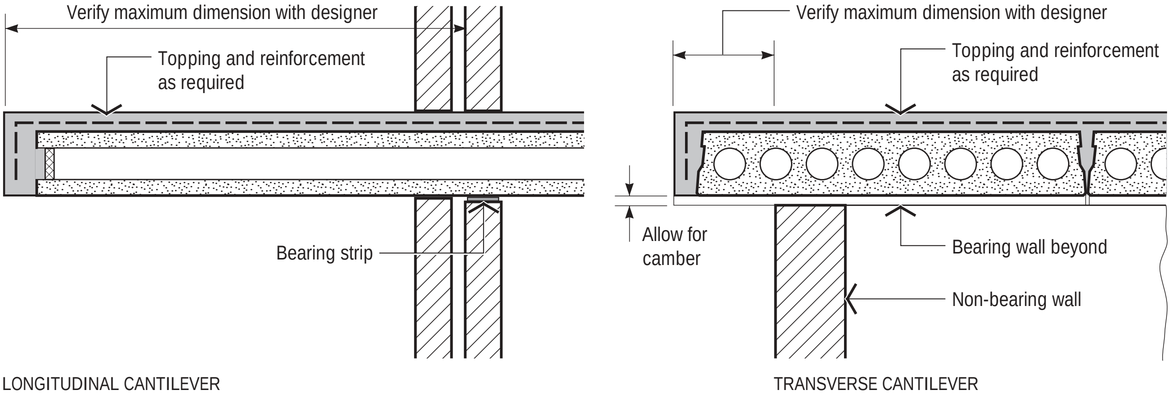


Figure 12 *Longitudinal and Transverse Cantilever Arrangements*

5.11 Structural Integrity

Structural integrity is too broad a subject to cover in this Manual and items 2, 3, 4 and 13 in the Bibliography (**Chapter 8**) should be consulted for further information and design methods.

Where hollowcore planks have a precast supporting structure, the floor and frame members should be tied together to form a structural entity. In addition to normal loadings, the hollowcore floor should be detailed to act with the building structure to resist unusual and dynamic load conditions. These may arise from earthquake, blast or the accidental removal of a support. The starting point in securing integrity is to provide a structural layout of planks, beams, walls and columns in which the fewest components rely on others for support.

Hollowcore planks should be tied to the support structure directly or indirectly through insitu concrete or grout. Reinforcement may be provided as continuous ties in insitu strips parallel or transverse to the planks, in the topping concrete over the supports or ties concreted into the cores of the hollowcore planks. For perimeter ties enclosing a group of floor units, unstressed strand laid in joint spaces in long lengths may be used. A bond length of at least 1.2 m should be provided for strand.

Suggested minimum tie capacities and typical details are illustrated in **Figure 13**. These types of detail allow the structure to act in a ductile manner during overload and are a cost-effective means of providing structural integrity in the event of catastrophic loss of support. Refer to Item 3, **Chapter 8** for further information.

Shear forces can be transferred by shear-friction effect at transverse or longitudinal joint faces by ties that act perpendicularly to the interface. The shear-friction capacity is in addition to any other design force requirement.

5.12 Diaphragm Action

Hollowcore floor planks can be used as a horizontal diaphragm to resist lateral loads. The easiest way to achieve this is with a reinforced insitu topping to the planks. In resisting lateral loads, the requirement is to transfer the load from the point of application to the foundations. Several elements are in this load path and each must be capable of accepting the load and transferring it to the next element.

If hollowcore planks are used without a composite structural topping, connections must be provided to transfer the forces from the floor to the vertical bracing system. The floor diaphragm must have adequate shear and bending strength to span as a horizontal beam between the bracing elements. The basic concept is illustrated in **Figure 14** and refer to Items 2, 3 and 4 in **Chapter 8**. Note that the full width of the floor may not participate in the beam action.

For untopped planks, longitudinal shear along the planks is transmitted through the grouted keyways. Where ductile behaviour is required, the shear capacity of this grout can be augmented by the shear-friction effect of transverse ties placed in the butt joints at the ends of the planks. When shear forces are transferred by shear-friction effect at transverse (or longitudinal) joint faces, the necessary tensile-force capacity should be added to the other design forces of the ties. The possible combination of horizontal and vertical shear in longitudinal joints should also be considered.

The chord forces induced by the bending of the diaphragm can be resisted by reinforcement in the supporting beam or wall. Generally, topped planks are simpler to design and detail as the reinforced insitu topping provides an efficient diaphragm restrained in the vertical plane by bond to the plank.

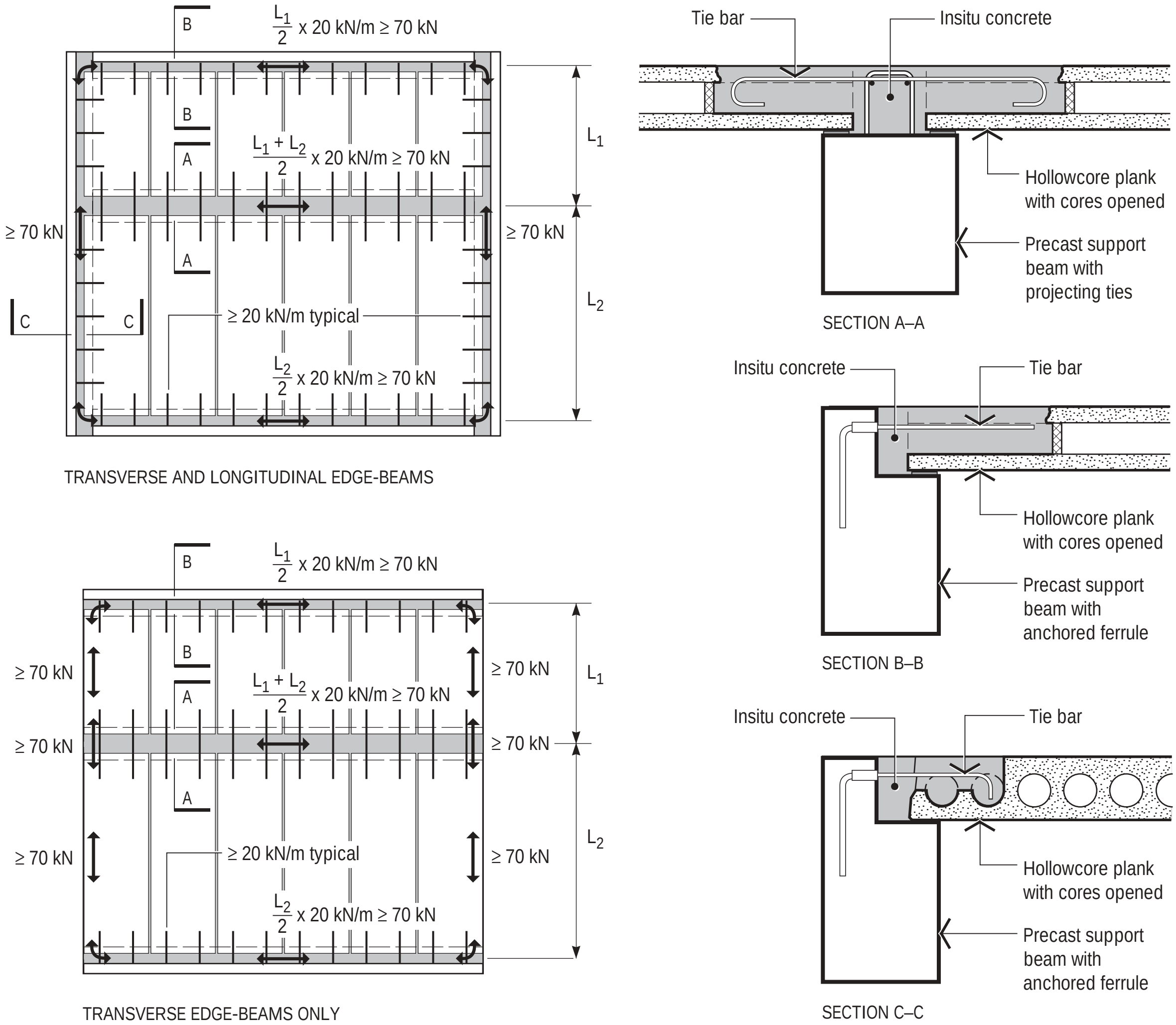


Figure 13 *Minimum tensile Capacities of Integrity Ties* [After Item 3, Chapter 8]

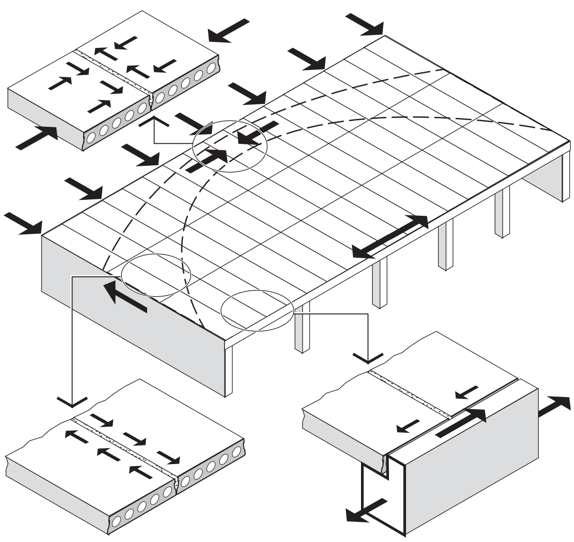


Figure 14 *Horizontal Load Transfer by Diaphragm Action*

5.13 Fire Rating

The fire rating or fire resistance level (FRL) of a floor is specified in the building codes as the period in minutes during which the floor must retain its structural adequacy, integrity and insulation when subjected to the standard fire test and is expressed as *180/90/90* for example (ie 180 minutes structural adequacy/90 minutes integrity/90 minutes insulation).

The FRL is normally varied by changing the cover to the strands and perhaps adding topping to the floor. This is effective up to a fire resistance period of 180 minutes. Beyond that it is usually necessary to vary the number of cores or increase the soffit thickness or insulate the soffit. Fire protection must be maintained at penetrations through cores. This can be achieved by back-filling them with concrete for the requisite length.

The Concrete Structures Code, AS 3600, specifies that fire resistance be met either by testing or calculation or by proportioning members to comply with certain rules. In practice the deemed-to-comply rules are adopted as a convenient method of compliance. Two criteria must be satisfied.

- *Insulation* requires a minimum effective thickness of concrete and a minimum thickness of concrete between adjacent cores and between a core and the exposed surface.
- *Structural adequacy* requires a minimum concrete cover to the strand.

The deemed-to-comply requirements are summarised in **Table 1**.

Proximity to cores and difficulty in ensuring full bond make strand covers in excess of 55 mm undesirable. It is good practice to group the strands in the flange section, and not locate them high in the web.

The effective thickness of a hollowcore plank is taken as the nett cross-sectional area divided by its width. If the effective thickness is not sufficient to achieve the required fire rating, this can be increased by providing fewer cores, a concrete topping or an insulating layer to the soffit.

As the size and spacing of the cores vary with the particular machine there are some small differences in the deemed-to-comply fire resistance levels of the planks provided by different manufacturers. In practice these differences tend to be significant only for the higher fire ratings and for planks that are not topped.

Grouted joints of untopped planks have been shown by fire tests to provide a fire resistance level at least equal to that of the plank section. However, untopped planks should generally be restricted to lower fire resistance levels unless adequate restraint or tie reinforcement at the ends of the planks can be provided to stop spreading.

If the required cover to the strand results in an inefficient design for the specified load capacity, the cover can be reduced by applying an appropriate thickness of insulating material to the soffit. Continuity can also be used to increase the structural adequacy for a given cover. The requirement (AS 3600, Clause 5.5.3) is that the floor is structurally-continuous at one end under the superimposed loads. Note that it is more economical to provide the required cover and use simply-supported members.

Table 1 *Section Dimensions for Deemed-to-Comply Fire Resistance Levels*

	Fire Resistance Period (minutes)					
Section	30	60	90	120	180	240
Effective thickness *(mm)*	60	80	100	120	150	170
Concrete thickness between cores and between cores and exposed surface *(mm)*	25	25	25	25	30	34
Required cover to strand *(mm)*						
Simply-supported span	20	25	35	40	55	65
Continuous span	15	20	25	25	35	45

5.14 Sound Insulation

Hollowcore floor planks provide a high level of acoustical insulation to airborne sound transmission and are similar to solid concrete slabs in the transmission of impact noise. The parameter which is used to measure the resistance of a wall or floor to airborne sound transmission is the R_w rating. R_w stands for *Sound Reduction Index* and is a single-number measure of the sound transmission loss of air-borne sound through the component. The larger the value of R_w the greater the sound insulation. A negative adjustment factor C_{tr} is added to the R_w value to compensate for low frequency sounds in the environment, such as traffic noise.

Building codes specify minimum values of sound insulation for floors and walls separating different occupancies in residential buildings. A floor separating dwellings must have an $R_w + C_{tr}$ not less than 50 dB. A 150-mm deep plank with 60 mm of topping meets this requirement.

Impact noise is conducted easily through building materials and can travel great distances. The *Normalised Impact Sound Pressure Level* ($L_{n,w}+C_1$) is a single-figure rating of the overall impact sound insulation performance of a floor-ceiling assembly. It is to be 62 dB or greater for floors between dwellings.

Impact noises are very dependent on the floor surface and it is best to arrest the sound at the impact point. Hollowcore floors, in combination with resilient materials, are effective at this. Carpet on a resilient underlay provides the best insulation. Overlays such as rubber and vinyl are less effective. Hard surface areas, such as tiling, require the working surface to be resiliently-supported on the hollowcore plank to achieve the required impact rating. All these are normal construction techniques.

5.15 Durability

Hollowcore planks are cast using concrete with a low water-cement ratio and high strength, typically in excess of 40 MPa. This concrete is inherently durable and protects the strands provided that the thickness of cover is appropriate to the degree of exposure.

The Concrete Structures Code, AS 3600, provides a classification of typical exposure conditions to cover the range from an internal enclosed environment to exterior environment with exposure conditions of different degrees of severity. Typical values are shown in **Table 2**.

For very severe conditions of exposure protective coatings may be used to reduce the ingress of corrosive agents. Further guidance on design for durability may be obtained from the Commentary on AS 3600.

Table 2 *Required Concrete Cover for Various Exposure Classifications*

Class-ification	Typical exposure conditions	Concrete cover (mm) for 40 MPa concrete
AI	Enclosed within building or exterior inland non-industrial, arid zone	20
A2	Exterior inland non-industrial, temperate zone	20
B1	Exterior 1–50 km from coast and inland industrial, any zone	30
B2	Exterior 0–1 km from coast, any zone	45

6 CONSTRUCTION DETAILS

6.1 Connections

The connection details at a support must not only transfer load but also contribute to the monolithic behaviour of the entire structure. Tie arrangements in the longitudinal and transverse directions should provide the required diaphragm action, transverse distribution of vertical loads, differential settlements and restrained deformations. The detailing of the connection is usually at the discretion of the Manufacturer as he is familiar with the economical arrangements to suit his product. Any special forces to be taken into account, such as those due to frame action, should be specified by the Structural Engineer for the project. The Manufacturer details the planks to include all the items that are cast into or supplied with the planks. Any other items necessary to complete the connection are supplied by the Building Contractor.

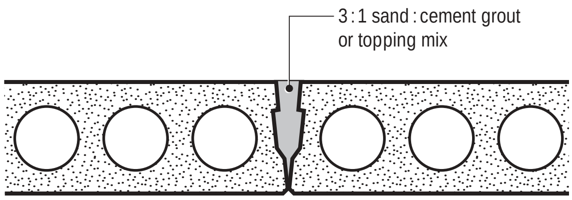
6.2 Bearing Support

Typical connection and support details are shown in **Figures 15** to **17**. Hollowcore units are normally designed to be simply-supported and the connections at the supports should be detailed accordingly, taking into account the requirement for overall structural integrity. Unintended restraining effects at the supports by stiff structural elements such as cross-walls may need to be considered. Continuity reinforcement in topping over the supports must have sufficient cover, allowing for transverse steel and laps.

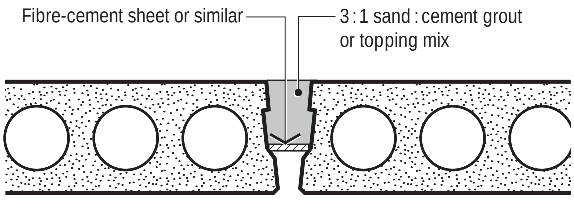
A connection should not unnecessarily restrain volume change movements that occur in precast concrete. Movement joints should be provided at regular intervals in the structure, usually at every third span longitudinally. If these movements are restrained, significant axial forces can develop in the planks, reducing shear capacity in particular.

The length of compression bearing required to adequately support a plank is a function of the reinforcing detail across a joint and the construction tolerances of the members coming together at the support. Detailing of the bearing length should allow for realistic tolerances and provide adequate clearances for erection and concreting of reinforcement details. The bearing lengths set out in **Table 3** are for planks in simple bearing. A generally-accepted absolute minimum to support any size plank in bearing is 50 mm. Tolerances for the dimensions of planks, as supplied, are set out in **Chapter 7** *Guide Specification*.

The bearing strip shown in **Figures 16** and **17**, must be specified by the designer if the plank is not to sit directly on its support. A concrete bearing surface is often provided with an elastomer strip set back at the edge. These strips are usually 2 or 3 mm thick. Planks supported on masonry walls require a bearing strip or slip joints to separate the different materials and to prevent cracking or spalling of the brickwork. Rendered walls and cornices should be detailed to permit some movement in the floor planks.



GROUTED KEY DETAIL

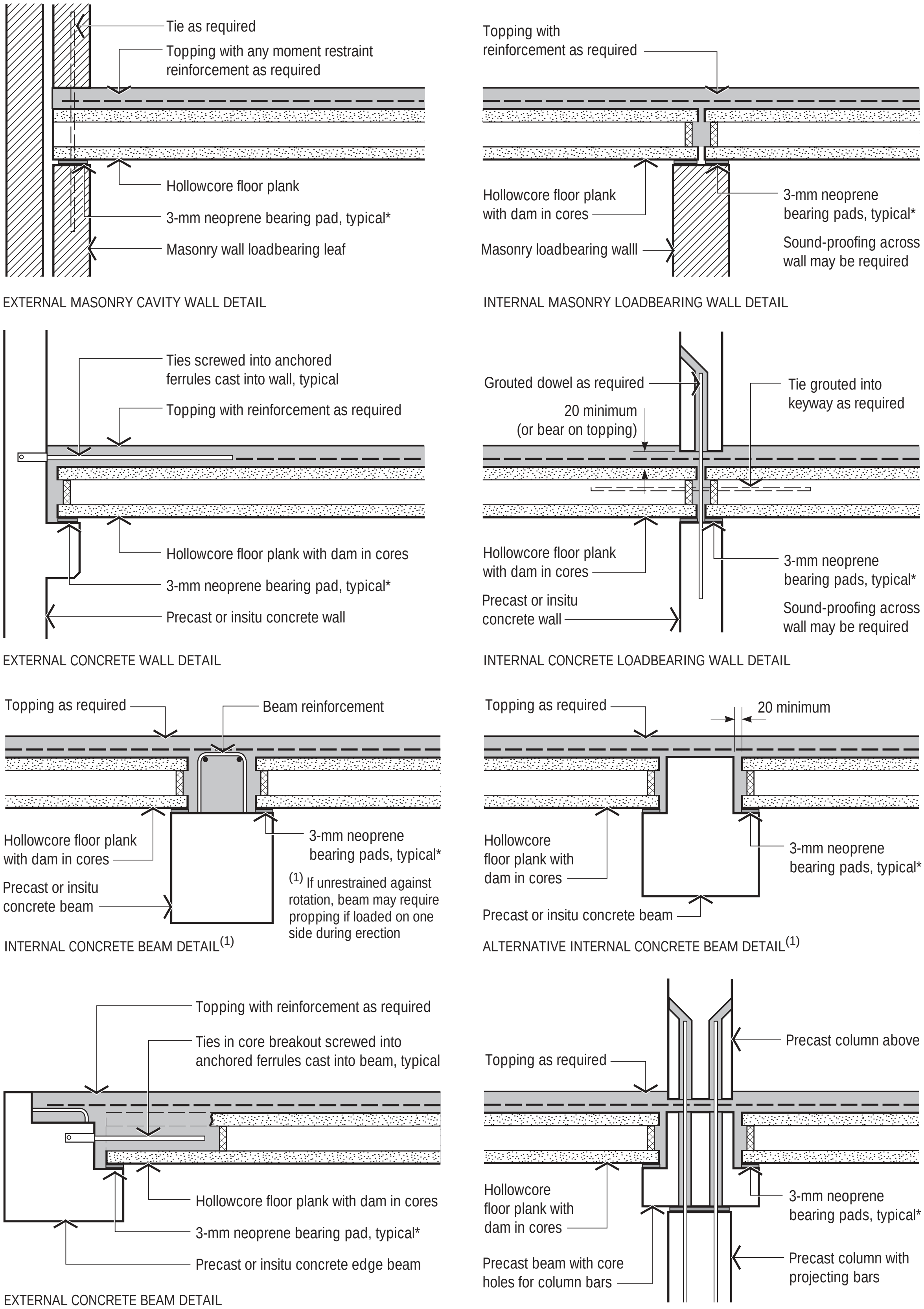


TOLERANCE MAKE-UP STRIP DETAIL

Figure 15 *Plank-to-Plank Connections*

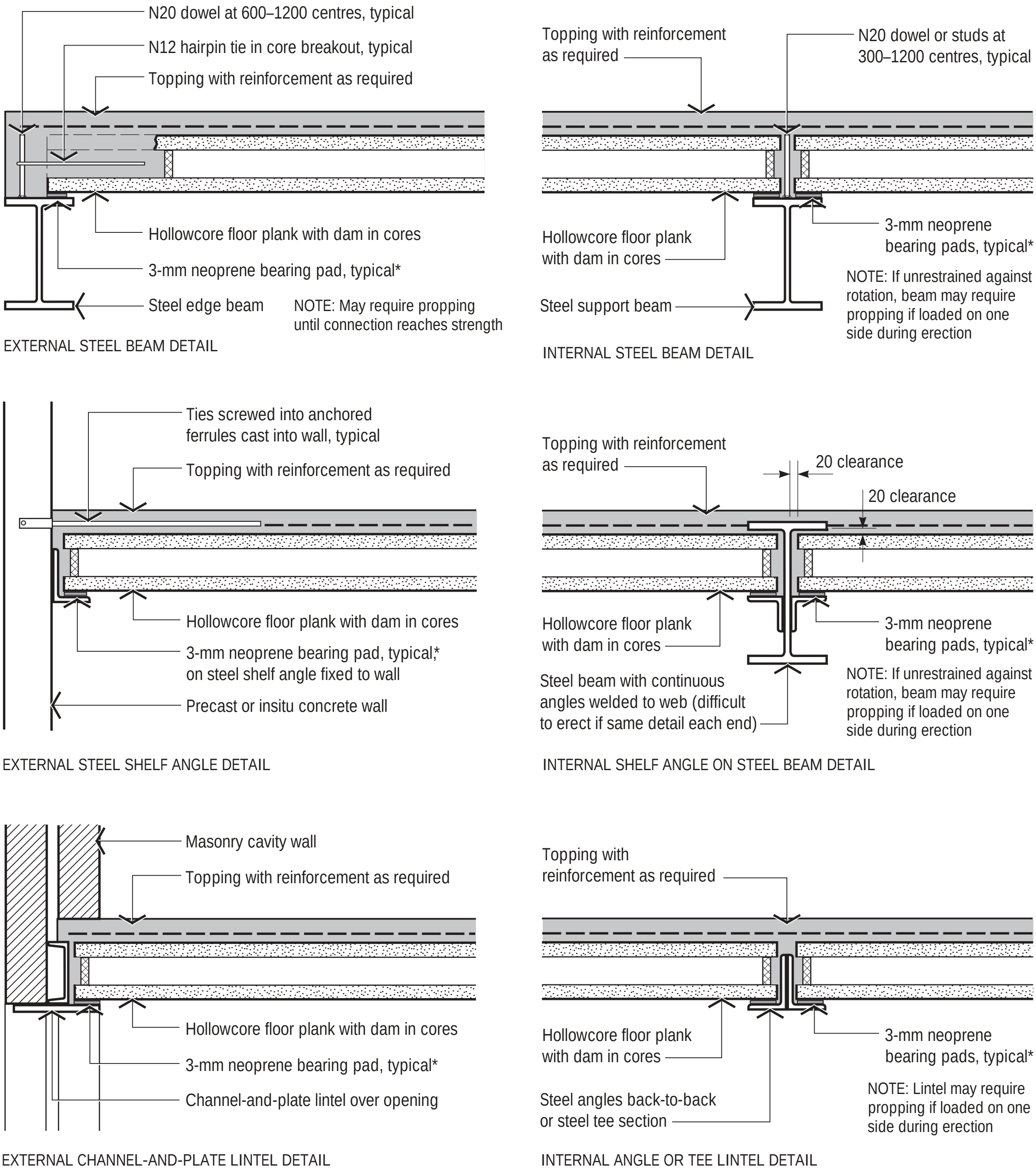
Table 3 *Recommended Bearing Lengths on Simple Supports*

Plank depth (mm)	Bearing length (mm) on concrete	Bearing length (mm) on steel
150, 200	60	70
250, 300	100	100
350, 400	150	125



* For recommended bearing lengths, see Table 3 (50 mm absolute minimum)

Figure 16 *Concrete and Masonry Bearing Support Details*

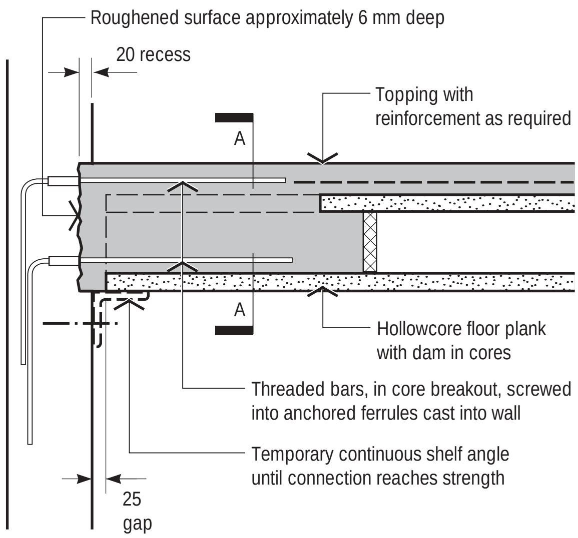


* For recommended bearing lengths, see Table 3 (50 mm absolute minimum)

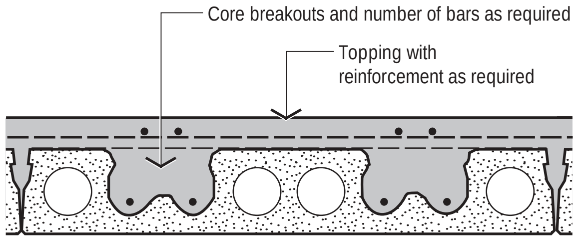
Figure 17 *Steel Bearing Support Details*

6.3 Insitu Connection

A connection type, which does not require a direct bearing or corbel support for the plank, is shown in **Figure 18**. An insitu reinforced concrete interface transfers moment and shear between the topped plank and the supporting wall panel. During construction a temporary angle is bolted to the wall to support the plank and topping until the connection gains strength. The projects in which this type of support has been used, along with the results of test programmes and advice on design, are available from manufacturers.



TYPICAL CONNECTION DETAIL



SECTION A–A

Figure 18 *Insitu Connection Detail*

6.4 Steel Support Beams

Steel support beams may be designed for composite action with the plank and topping (**Figure 17**) by providing for shear transfer at the interface. If the overall depth of construction is critical, support angles may be welded to the web, thus reducing the floor height. Note that clearances are needed between the planks and beam for satisfactory erection and this arrangement should not be used at both ends without careful consideration of the installation sequence. Bolted splices in steel beams should be detailed so that they do not interfere with the bearing of the planks.

6.5 Torsion in Support Beams

A common oversight during erection is the eccentric loading of planks onto one side only of a support beam leading to dangerous twisting of the beam. Both precast and steel beams are susceptible but the latter are particularly at risk. Precast beams can be detailed with a double-bolted moment connection to the column to resist the torsion. The column may need propping to resist the horizontal component of the torsion as well as other erection loads. For a steel beam, it, and possibly the planks will need to be propped.

The situation can also arise at an end-bay where the floor and beam system may require propping until the designed torsion-resisting connections are active.

6.6 Topping Layer

When concrete or masonry walls continue above topped floor planks, and some degree of continuity is intended, the topping will have to be placed before the wall above is erected.

6.7 Ties

Structural integrity ties may be required in both longitudinal and transverse directions. These may be reinforcement or strand grouted into keyways or joints at the ends of planks. Connection to the structure is usually by cast-in inserts. Care is needed in setting out the planks to ensure that the keyways line up with adjacent spans, projecting bars or ferrule locations.

6.8 Penetrations

Penetrations and block-outs in planks should not cut through the strand unless this has been allowed for in the design, **Figure 19**. Any coring on site should be restricted to defined parts of members. Where large penetrations are required full-width headers (**Figure 20**) may be used to support the end of a plank and transfer the load to adjacent planks. These planks must be designed for the additional load.

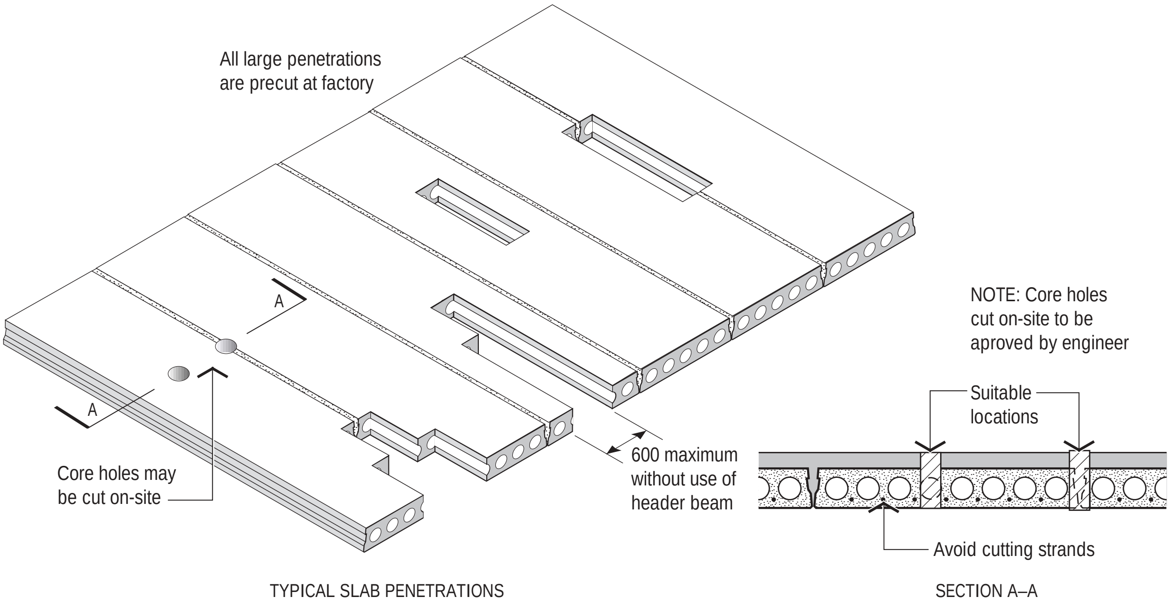


Figure 19 *Penetration Positions*

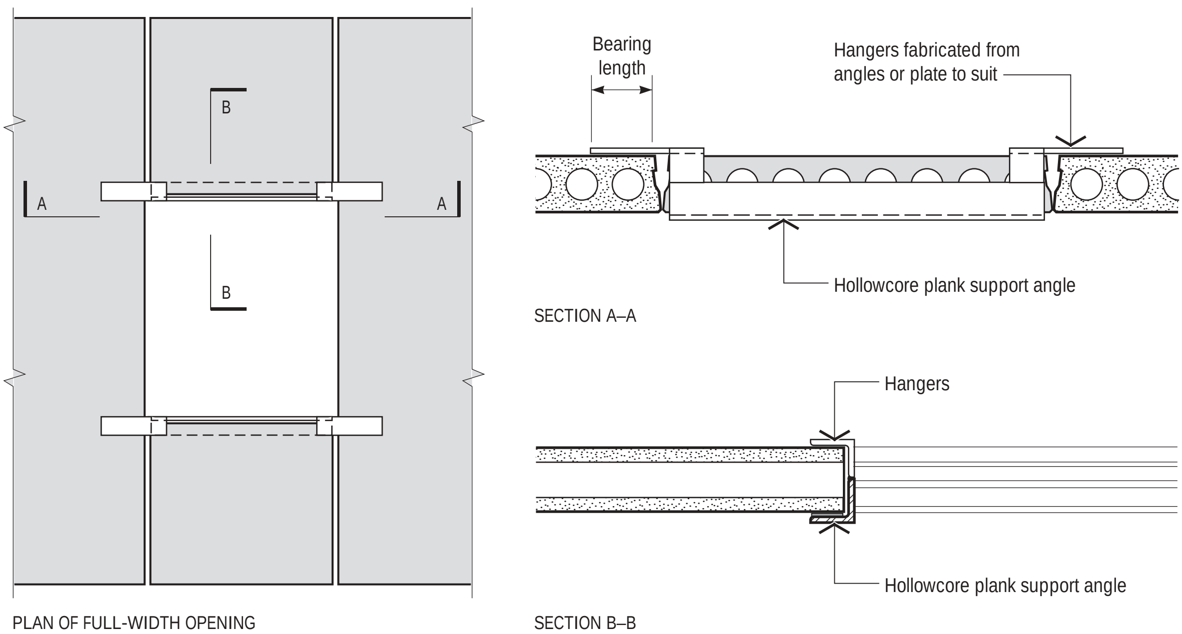


Figure 20 *Typical Header Arrangement*

6.9 Hangers

Fixings and support hangers (**Figure 21**) should be installed strictly in accordance with the supplier's directions. Where heavy loads are to be supported, through-bolts to a distribution plate or beam may be required. These are required to be in accordance with the design engineer's specification.

6.10 Waterproofing

Hollowcore decks which are open to the weather can be made waterproof in the usual ways. These include:

- Post-tensioning a topping of minimum 100-mm thickness
- Using a proprietary waterproofing admixture which is capable of sealing cracks to 0.4-mm width. The Supplier's recommendation must be strictly followed
- Membranes.

In each case, only careful construction practice will ensure a satisfactory result.

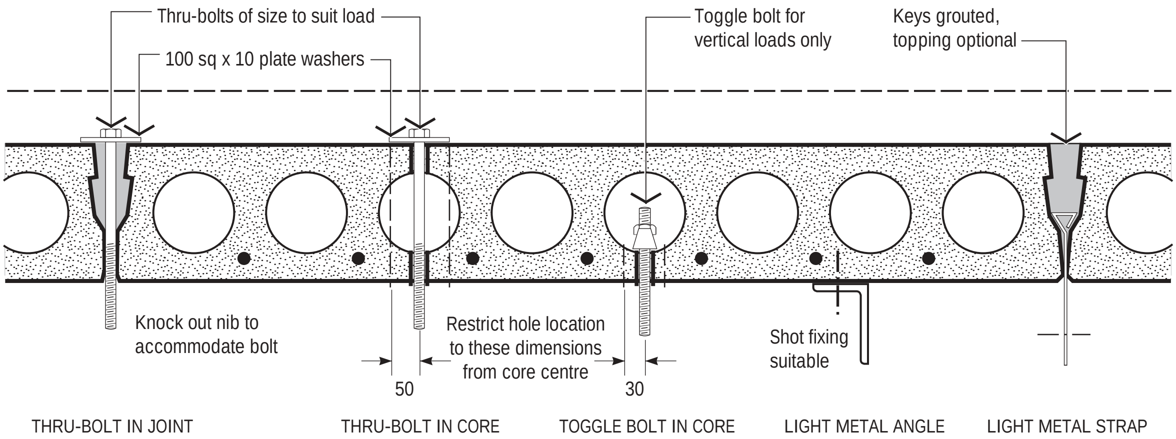


Figure 21 *Typical load hangers*

7 GUIDE SPECIFICATION

This guide specification is intended to be used in the preparation of the specification for a particular project. It should be checked for compatibility with the particular job requirements by deleting any provisions that do not apply and adding special provisions needed.

Scope

This guide specification covers the manufacture and erection of hollowcore floor planks produced by an approved manufacturer.

Design

Planks shall be designed in accordance with AS 3600, except where industry practice provides a proven alternative method.

The manufacturer shall prepare and submit shop drawings for approval of the general arrangement of the planks, adequacy and dimensions prior to manufacture. Shop drawings shall show the location of all planks with all major openings detailed. Shop drawings detailing each unit, cast-in inserts and its strand configuration shall be submitted to the building contractor for approval.

The design of the structure, including checking the adequacy of the hollowcore planks for their intended use in the structure, shall be the responsibility of the Structural Engineer for the project.

Materials

Cement shall comply with AS 3972 and supplementary cementitious materials with AS 3582 parts 1 and 2. Aggregates shall comply with AS 2758.1. Chemical admixtures shall comply with AS 1478.1.

Prestressing steel shall be stress-relieved low-relaxation strand complying with AS 1311. Strand shall be clean and free of deleterious substances at the time of concreting.

Concrete shall have a minimum characteristic 28-day strength of 40 MPa and shall conform to the requirements of AS 3600. Concrete strength at release of prestress shall be a minimum of 25 MPa or as required by the structural drawings.

Topping concrete shall have a minimum characteristic 28-day strength of 32 MPa or as shown on the drawings. If topping concrete is used to grout the keyways, the maximum aggregate size shall be 14 mm.

Manufacture

Hollowcore floor planks shall be cast on a long-line bed by an approved machine and mechanically compacted. The top surface shall be finished as cast by the machine or intentionally-roughened to achieve the specified bond characteristics of the topping or other finish applied after erection.

The underside finish shall be as cast against the bed in accordance with good industry practice. Some surface voids and colour variations shall be acceptable in accordance with the sample agreed prior to casting.

Tolerances

Floor planks shall be supplied in accordance with the following tolerances.

Length	+ 10 mm	- 10 mm
Width	+ 3 mm	- 6 mm
Thickness	+ 3 mm	- 3 mm
Squareness of end	+ 6 mm	- 6 mm
Wind	10 mm per 3000 mm	
Location of ferrules	+ 20 mm	- 20 mm
Location of strand	+ 3 mm	- 3 mm
Differential camber adjacent units	2 mm/m span but not greater than 15 mm	

Delivery and Handling

Hollowcore floor planks shall be lifted and supported during manufacture, storage, transport and erection operations at the nominated lifting positions only.

Erection

The Building Contractor shall be responsible for providing true and level bearing surfaces for the support of the hollowcore planks. Temporary shoring and bracing shall also be provided as necessary to ensure the stability of the structure during erection. The hollowcore planks shall be installed by a competent erection contractor. Where the manufacturer also erects the planks, the Building Contractor shall be responsible for providing suitable access at the site to enable trucks and cranes to operate under their own power.

Bearing strips shall be accurately set where required. Place any reinforcement required by the drawings. Keyways shall be filled and compacted with a 3:1 sand-cement grout mix or by the topping concrete using a maximum aggregate size of 14 mm. Voids at plank ends shall be sealed to prevent penetration of topping into the cores by more than 50 mm. It should not be more than the support length without considering the implications on the shear capacity.

Attachments and Penetrations

Attachments and fixings to the hollowcore planks shall be in accordance with the approved details only and shall not impair or reduce the strength of the floor planks.

Penetrations and chases to the hollowcore planks shall be in accordance with details approved by the Structural Engineer and agreed by the Manufacturer.

Insitu Topping

The Building Contractor shall provide a well-compacted insitu structural concrete topping to the floor planks as detailed on the drawings. Reinforcement is to be placed in accordance with structural details. The plank surface shall be clean and free of loose material and surface-moist (saturated surface-dry) immediately prior to placing the topping. Finish and cure the topping so that plastic shrinkage cracks are controlled to acceptable levels. Construction joints in the topping shall be located as shown on the drawings.

Inspection and Acceptance

The manufacturer shall provide access to its production facilities for inspection of work in progress by the Structural Engineer to verify conformity of the product with the project specifications.

8 BIBLIOGRAPHY

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9 SAMPLE CALCULATION

A hollowcore plank floor is required to span 8 m between centreline of bearings and carry a live load of 3 kPa plus a superimposed dead load of 1.5 kPa. It will have an insitu screed of 60 mm thickness forming a composite section. The strands are located at 40 mm from the soffit. Bearing length is 80 mm at each end.

AS 3600–2001 *Concrete structures* and AS 1170–2002 *Structural design actions* are the relevant standards.

1 TRIAL SECTION

Choose a 200 deep hollowcore section with 60 mm of structural topping, from the load-span charts (**Figure 3**). Span/depth ratio is 31 and thus vibration response will be satisfactory.

1.1 Section Properties

These properties are illustrative for this example and refer to a section with non-circular cores. Actual properties are available from each manufacturer for their cross-sections and must be used in design calculations.

Precast section

Nominal width	B_{hc} = 1200 mm
Area	A_{hc} = 150,840 mm^2
Distance to Neutral axis	Y_{bhc} = 99.8 mm
Moment of Inertia	I_{hc} = 694.8 x 10^6 mm^4
Bottom section modulus	Z_{bhc} = 6.96 x 10^3 mm^3
Width of web	b_{whc} = 430 mm

Composite section

Distance to Neutral axis	Y_{bcomp} = 138.8 mm
Moment of Inertia	I_{comp} = 1479 x 10^6 mm^4
Bottom section modulus	Z_{bcomp} = 10.66 x 10^3 mm^3
Shear area moment for bottom flange	Q_{bfcomp} = 7.2 x 10^6 mm^3

Concrete Properties

Plank strength at release	f'_{chci} = 25 MPa
Elastic modulus at release	E_{hci} = 25,250 MPa
Plank characteristic strength	f'_{chc} = 40 MPa
Elastic modulus at 28days	E_{hc} = 31,900 MPa
In-situ screed strength	$f'_{c.screed}$ = 32 MPa
In-situ elastic modulus	E_{screed} = 28,500 MPa
Concrete density	$\gamma_{concrete}$ = 2,500 kg/m^3

Other Variables

Span	L_s = 8.0 m
Bearing length	L_{brg} = 80 mm
Overhang past bearing	$L_{o'hang}$ = 0 mm

1.2 Design Values

Load factors

Ultimate load combination	$E_{d.dst}$ = [1.2G, 1.5Q]
Moment strength reduction factor	ϕ_m = 0.8
Shear strength reduction factor	ϕ_v = 0.7
Short term combination factor	ψ_s = 0.7
Long term combination factor	ψ_l = 0.4

Working loads

Plank mass	W_{hc} = 3.8 kN/m
Topping mass	W_{screed} = 1.8 kN/m
Live load	W_{ll} = 3.6 kN/m
Superimposed dead load	W_{sdl} = 1.8 kN/m

Ultimate loads

Plank mass	W^*_{hc} = 3.8 x 1.2 = 4.56 kN/m
Topping mass	W^*_{screed} = 1.8 x 1.2 = 2.16 kN/m
Live load	W^*_{ll} = 3.6 x 1.5 = 5.4 kN/m
Superimposed dead load	W^*_{sdl} = 1.8 x 1.2 = 2.16 kN/m

Working load moments

Plank mass	M_{hc} = 30.4 kNm
Topping mass	M_{screed} = 14.4 kNm
Live load	M_{ll} = 28.8 kNm
Superimposed dead load	M_{sdl} = 14.4 kNm

Factored (ultimate) moments and shears

Total design loading	W^* = 4.56 + 2.16 + 5.4 + 2.16 = 14.28 kN/m
Bending moment at mid-span	M^* = 114 kNm
Shear at support	V^* = 57 kN

2 SECTION ANALYSIS

2.1 Number of Strands

Assuming 9.3-mm diameter strand at 35 mm cover, the effective depth (d_p) of the composite section is 260 - 40 = 220 mm.

The area of prestressing steel can be estimated as follows. The product of the ϕ-factor, lever arm factor (j') and stress in the strand at ultimate is generally about 1200 to 1250 MPa. Taking a value of 1200:

$$A_p = \frac{M^*}{1200\, d_p} = 432 \text{ mm}^2$$

Select an integer number, N_p, of strands of suitable size. Strand diameters available are 9.3, 12.7 and, less commonly, 15.2 mm. Using 9.3 mm diameter super strand:

$$N_p = \frac{A_p}{54.7 \text{ mm}^2} = 7.9$$

Allow some margin

Say, $N_p = 9$

Then, $A_p = 492.3 \text{ mm}^2$

Adopt 9 off 9.3 mm dia. strand with an ultimate stress, f_p, of 1860 MPa.

The jacking load of strand usually lies in the range 65% to 80% of ultimate stress, with 70% as a typical value. Prestress loss occurs with time and ranges from 18 to 25%, experience suggests to allow 11% at release and 22% total with this amount of prestress until checked.

For typical loss calculations, *see Chapter 6 of Bibliography Item 1*.

$$P_j = (70\%\ f_p)A_p = 641 \text{ kN}$$

$$P_i = 89\%\ P_j = 570.5 \text{ kN}$$

$$P_f = 78\%\ P_i = 445 \text{ kN}$$

2.2 Prestress on the Precast Cross-Section

In the following calculations the stress due to prestress or bending at a particular level above the bottom (soffit) of the cross-section is often required. The generic equations used are as follows.

For computing the stress due a bending moment, M:

$$\sigma = M \frac{(Y - Y_b)}{I}$$

Where Y_b is the distance from the soffit to the neutral axis of the precast or composite section and I is the corresponding moment of inertia.

For computing the stress in the precast section due to a prestress force, P, at an eccentricity, e, from the precast neutral axis:

$$\sigma = \frac{P}{A_{hc}} + \frac{P\, e(Y - Y_{bhc})}{I_{hc}}$$

Where the eccentricity, e, is the distance from strand centreline to the precast neutral axis and Y is the distance from the soffit to the level at which the stress is required The section properties are as defined in item 1.1 above. The section modulus, Z, at the top or bottom fibre is the moment of inertia, I, divided by the distance to the neutral axis. Compression stress is taken as positive.

Check that the bottom fibre stress gives a reasonable ratio to release strength (say, about 0.6) and calculate the residual prestress at the critical section for flexure for later use. The top fibre stress at release can also be checked at the prestress development length from the end of the plank. It includes some self-weight stress and may be tensile. It is usually not critical for the amount and eccentricity of prestress used in hollowcore planks.

The strand eccentricity, e, for the precast section is 99.8 - 40 = 59.8 mm.

$$\sigma_{bpi} = \frac{P_i}{A_{hc}} + \frac{P_i\, e}{Z_{bhc}} = 8.7 \text{ MPa}$$

$$\sigma_{bphc} = \frac{P_f}{A_{hc}} + \frac{P_f\, e}{Z_{bhc}} = 6.8 \text{ MPa}$$

2.3 Flexural Capacity of Composite Section

The flexural capacity can be determined by standard formulae for a rectangular section provided the neutral axis is above the zone of the cores. The rectangular stress block factor, g, and the stress in the strand at ultimate moment, σ_{st}, are calculated in accordance with AS 3600 Clause 8.1. The equations for k_u and M_u can be found in standard concrete texts, see *Chapter 7 of Bibliography Item 13*.

The concrete strength in the calculation is that of the insitu topping for a composite section. The value of f_p is the appropriate ultimate strength for the strand diameter. The effective depth of the composite section is 220 mm.

$$\gamma = 0.85 - 0.007(f'_{c.screed} - 28) = 0.822$$

$$k_1 = 0.4$$

$$k_2 = \frac{A_p\, f_p}{B_{hc}\, d_p\, f'_{c.screed}} = 0.108$$

$$\sigma_{st} = f_p \left[1 - \frac{k_1\, k_2}{\gamma}\right] = 1761.9 \text{ MPa}$$

$$k_u = \frac{A_p\, \sigma_{st}}{0.85\, f'_{c.screed}\, \gamma\, d_p\, B_{hc}} = 0.147$$

The neutral axis is at $k_u{\cdot}d_p$ = 32.3 mm and is above the void zone.

$$M_u = d_p\, A_p\, \sigma_{st} \left[1 - \frac{\gamma\, k_u}{2}\right] = 179.3 \text{ kNm}$$

$$\phi\, M_u = \phi_m\, M_u = 143.4 \text{ kNm}$$

$$> M^* \quad \text{OK}$$

The ultimate moment capacity of the section must exceed its cracking moment by a margin of 1.2 (*AS 3600, Clause 8.1*). The cracking moment of a composite section is affected by the mass of the insitu screed since it is carried on the precast section alone and reduces the bottom fibre prestress. However the composite action also increases the section properties. Thus at mid-span, where the cracking moment is a minimum:

$$M_{cr} = \left[\sigma_{bphc} + 0.6\sqrt{f'_{chc}} - \left(\frac{M_{hc} + M_{screed}}{Z_{bhc}}\right)\right] Z_{bcomp} + M_{hc} + M_{screed} = 89 \text{ kNm}$$

$$\frac{M_u}{M_{cr}} = 2.02 \qquad > 1.2 \quad \text{OK}$$

Check that the maximum flexural tensile stress under short-term service loading does not exceed $0.5\sqrt{f'_c}$. Values for the short-term factor, ψ_s, are given in AS 1170.0. For office loading ψ_s = 0.7.

$$\sigma_{ft} = 0.5\sqrt{f'_{chc}} = 3.2 \text{ MPa}$$

The net bottom fibre stress in the precast after topping is:

$$\sigma_{bhc} = \sigma_{bphc} - \frac{M_{hc}}{Z_{bhc}} - \frac{M_{screed}}{Z_{bhc}} = 0.37 \text{ MPa compression}$$

The bottom fibre stress at service load is:

$$\sigma_{bcomp} = \sigma_{bhc} - \frac{M_{sdl}}{Z_{bcomp}} - \psi_s \frac{M_{ll}}{Z_{bcomp}} = -2.9 \text{ MP tension}$$

$$< \text{allowable } \sigma_{ft} \quad \text{OK}$$

2.4 Shear

The requirements of AS 3600, Clause 8.2, for flexure-shear must be met at all sections of the plank. This capacity can be calculated directly. The provisions for web-shear (principal tension cracking) only apply to cross-sections uncracked in flexure. The calculation for web-shear capacity is more involved except at the neutral axis. Refer to a standard text for derivation of the formulae, such as *Chapter 12 of Bibliography Item 13*.

At sections uncracked in flexure, the criteria used below is that both the flexure-shear must be less than the calculated capacity and the maximum principal tension must be less than the allowable value in AS 3600, Clause 8.2. The section cannot be reinforced for shear so it is not necessary to know the actual web-shear capacity.

2.5 Flexure-shear

The flexure-shear capacity at a cross-section must be calculated at several locations along the span to locate the critical section. For a UDL the shear capacity equation in AS 3600, Clause 8.2, is a descending curve from the support to mid-span. The minimum difference between the capacity and the shear force at a section occurs at about quarter span where shear is significant and bending moment is reasonably high.

Check the capacity at quarter span.
The un-factored plank and topping dead loads act on the precast plank section, reducing the effective prestress in the composite section, this moment is:

$$x = \frac{L_s}{4} = 2.0 \text{ m}$$

$$M_x = \frac{(W_{hc} + W_{screed})L_s\, x}{2} - \frac{(W_{hc} + W_{screed})\, x^2}{2}$$

$$= 33.4 \text{ kNm}$$

The decompression moment, M_0, taking M_x into account, is:

$$M_0 = \left[\sigma_{bphc} - \frac{M_x}{Z_{bhc}}\right] Z_{bcomp} + M_x = 54.4 \text{ kNm}$$

Then at this location:

$$V_0 = M_0 \frac{V^*_x}{M^*_x} = 18.1 \text{ kN}$$

and the flexural-shear capacity is:

$$\beta_1 = 1.1 \left[1.6 - \frac{d_p}{1000}\right] = 1.518$$

$$\beta_2 = 1.0$$

$$\beta_3 = 1.0$$

$$\phi V_{uc} = \phi_v \left[\beta_1 \beta_2 \beta_3\, b_{whc}\, d_p \left(\frac{A_p\, f'_{chc}}{b_{whc}\, d_p}\right)^{1/3} + V_0\right]$$

$$= 72.3 \text{ kN}$$

which is compared to the factored shear at location ‘x’

If this section is uncracked then the web-shear provisions also apply.

2.6 Web-shear

The principal tension under the design loading at a cross-section is limited to $0.33\sqrt{f'_{chc}}$ by AS 3600, Clause 8.2. It is not obvious where the maximum value will occur and a number of locations in the section will require examination. These include the minimum web width, the neutral axis and the bottom web-flange junction. The latter only applies to planks with non-circular voids. At positions other than the neutral axis there will be flexural stress on the cross-section due to superimposed loads in addition to the self weight and prestress.

The critical section for web-shear is likely to be at d_p from the support. In a hollowcore plank this section will almost always be within the development length of the strand where shear is at a maximum and the prestress will be less than the full amount.

Check intersection of web and bottom flange.
As an example, check the intersection of web and bottom flange at a section located at the effective depth from the support. There are corner fillets commencing at 85 mm from the soffit.

The development length of a strand is 60 diameters. The available prestress development length, L_d, from the end of the plank to the section includes the bearing length and any overhang beyond the bearing.

$$L_d = d_p + L_{brg} + L_{o'hang} = 300 \text{ mm}$$
$$\leq 60 \text{ strand diameters}$$

The first 10% of the total development length is assumed to be unstressed (AS 3600, Clause 13.3). Thus the prestress force at a section within the development length is:

$$P_x = \frac{L_d - 0.1(60\, d_b)}{0.9(60 d_b)}\, P_f = 216.4 \text{ kN}$$

The maximum allowable principal tension is:

$$\sigma_p = 0.33\sqrt{f'_{chc}} = 2.087 \text{ MPa}$$

Location of section from centre of support is:

$$x = d_p + \frac{L_{brg}}{2} = 260 \text{ mm}$$

Factored shear and moments here are:

$$V^*_x = V^* - W^*\, x = 53.3 \text{ kN}$$

$$M^*_{xhc} = \frac{(W^*_{hc} + W^*_{sdl})\, x}{2}(L_s - x) = 6.7 \text{ kNm}$$

$$M^*_x = \frac{(W^*_{ll} + W^*_{sdl})\, x}{2}(L_s - x) = 7.6 \text{ kNm}$$

The section is uncracked, hence web-shear provisions apply.

$$Y = 85 \text{ mm}$$

Direct stress at this location is:

$$\sigma_x = \frac{P_x}{A_{hc}} - P_x\, e\left[\frac{Y - Y_{bhc}}{I_{hc}}\right] + M^*_{xhc}\left[\frac{Y - Y_{bhc}}{I_{hc}}\right] + M^*_x\left[\frac{Y - Y_{bcomp}}{I_{comp}}\right]$$
$$= 1.29 \text{ MPa}$$

compression

$$b_v = b_{whc}$$

$$\sigma_s = \frac{V^*_x\, Q_{bfcomp}}{I_{comp}\, b_v} = 0.6 \text{ MPa}$$

$$\sigma_{px} = \sqrt{\left[\frac{\sigma_x}{2}\right]^2 + \sigma_s^{\,2}} - \frac{\sigma_x}{2} = 0.24 \text{ MPa}$$

tension
$< \sigma_p$ OK

The web width, b_v, is the total width across the plank at the level being considered, in this case below the flange fillet. Q is the first moment about the neutral axis of the cross-section area below that level. Note that compressive direct stress is positive in the above equations. The flexure-shear capacity must also be checked.

2.7 Interface shear

Check the interface shear (AS 3600, Clause 8.4) between the topping and the plank, using the appropriate surface roughness, in this case assume a smooth machine-screeded surface.

$$\beta_5 = 0.2$$

$$f_{ct} = 0.4\sqrt{f'_{c.screed}} = 2.3 \text{ MPa}$$

$$\phi V_{uf} = \phi_v\, \beta_5\, B_{hc}\, d_p\, f_{ct} = 83.6 \text{ kN}$$

$> V^*$ OK

2.8 Deflection

Calculate the deflection of the plank at installation and under long-term service load using the deflection multipliers from *Table 3 in Bibliography Item 12* (reproduced below). The insitu topping provides a level surface initially and is the datum from which subsequent deflections are measured. Downward deflection is negative in these calculations.

The hog due to prestress at release is:

$$\Delta_{ps} = \frac{P_i \, e \, L_s^2}{8E_{hci} \, I_{hc}} = 15.6 \text{ mm}$$

The self-weight deflection is:

$$\Delta_{hc} = \frac{-5 \, W_{hc} \, L_s^4}{384 \, E_{hci} \, I_{hc}} = - 11.5 \text{ mm}$$

at release the plank has a net hog upwards of 4.1 mm.

The deflection components for the insitu topping, superimposed dead load and live load are:

$$\Delta_{screed} = \frac{-5 \, W_{screed} \, L_s^4}{384 \, E_{hc} \, I_{hc}} = - 4.3 \text{ mm}$$

$$\Delta_{sdl} = \frac{-5 \, W_{sdl} \, L_s^4}{384 \, E_{screed} \, I_{comp}} = - 2.3 \text{ mm}$$

$$\Delta_{ll} = \frac{-5 \, W_{ll} \, L_s^4}{384 \, E_{screed} \, I_{comp}} = - 4.5 \text{ mm}$$

At erection, the hog of the plank is:

$$\Delta_{erection} = 1.8 \, \Delta_{ps} + 1.85 \, \Delta_{hc} = 6.8 \text{ mm}$$

Immediately after pouring the screed, the soffit of the plank is at:

$$\Delta_{erection} + \Delta_{screed} = 2.5 \text{ mm}$$

The long-term service load deflection of the datum top screed surface has only the long-term components of the multipliers for the prestress, plank mass and topping mass in the deflection calculation. Therefore the elastic deflection component must be subtracted from the multipliers in the Table (below), e.g. for Δ_{ps} the long-term component of the multiplier for a composite unit is 2.2 -1 = 1.2. Values for the long-term load factor, ψ_l, are given in AS 1170.0. For office loading $\psi_l = 0.4$.

$$\Delta_{top.surface} = (1.2 \, \Delta_{ps} + 1.4 \, \Delta_{hc} + 1.3 \, \Delta_{screed}) + 3.0 \, \Delta_{sdl} + 3.0 \, \psi_l \, \Delta_{ll}$$
$$= - 15.3 \text{ mm}$$

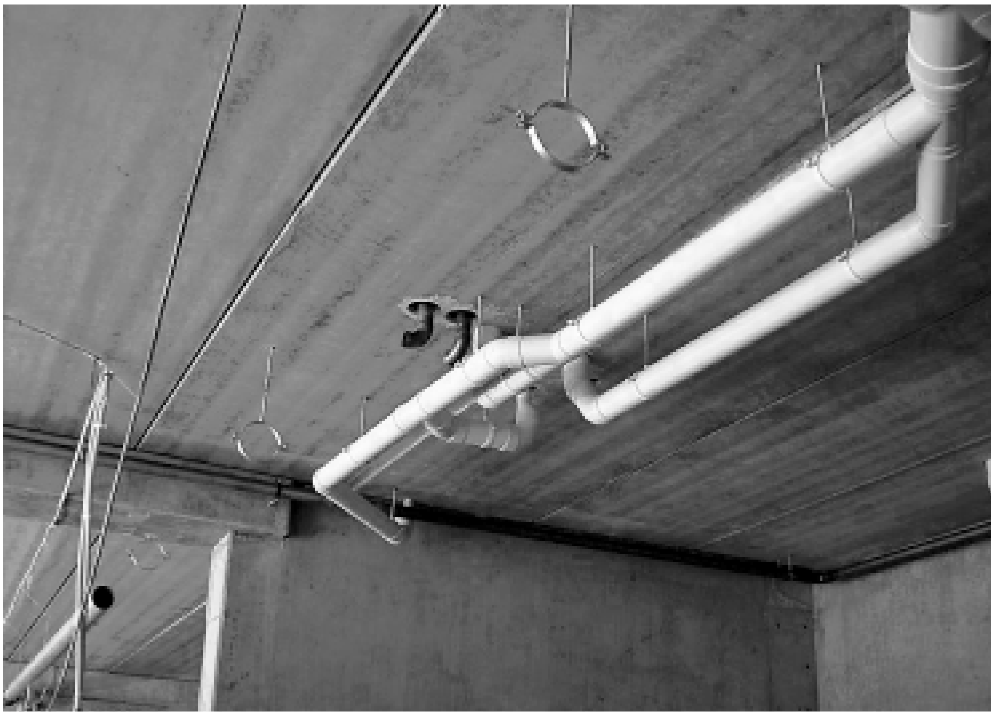
The span/deflection ratio is:

$$\frac{L_s}{- \Delta_{top.surface}} = 524$$

Suggested Multipliers for Estimating Long-term Cambers and Deflections for Typical Elements
[Based on Table 3 in Bibliography Item 12]

			Composite topping	
Stage	**Component**	**Application**	**Without**	**With**
Erection	Deflection (downward)	Elastic deflection due to element mass at release of prestress	1.85	1.85
Erection	Camber (upward)	Elastic camber due to prestress at time of release of prestress	1.80	1.80
Final	Deflection (downward)	Elastic deflection due to element mass at release of prestress	2.70	2.40
Final	Camber (upward)	Elastic camber due to prestress at time of release of prestress	2.45	2.20
Final	Deflection (downward)	Elastic deflection due only to superimposed dead load or to long-term live load	3.00	3.00
Final	Deflection (downward)	Elastic deflection caused by the composite topping	–	2.30

Core break-outs are used to grout in integrity ties, see Figure 13 (page 13)



Penetrations for services and provision for hangers are readily accommodated by hollowcore planks, see Figure 19 (page 20) and Figure 21 (page 21)

Typical detail using internal precast support beams and precast columns, see Figure 16 (page 17)

Hollowcore Floor Planks may be supported on precast beams (above) or masonry walls (below) as detailed in Figure 16 (page 17)

Hollowcore Floor Planks may be supported on precast walls (above) as detailed in Figure 16 (page 17) or steelwork (below) as detailed in Figure 17 (page 18)

Topping concrete is often used to produce composite units and a level floor surface, see Figure 1 (page 5)

Hollowcore Floor Planks are ideal for residential construction. They easily accommodate set-downs for wet areas, penetrations for services and step-downs for balconies

Hollowcore Floor Planks are becoming the preferred flooring system for many multi-storey projects because of their distinctive advantages (see Page 2)

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For other information on hollowcore, see the NPCAA web site: **www.npcaa.com.au**

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