Humes

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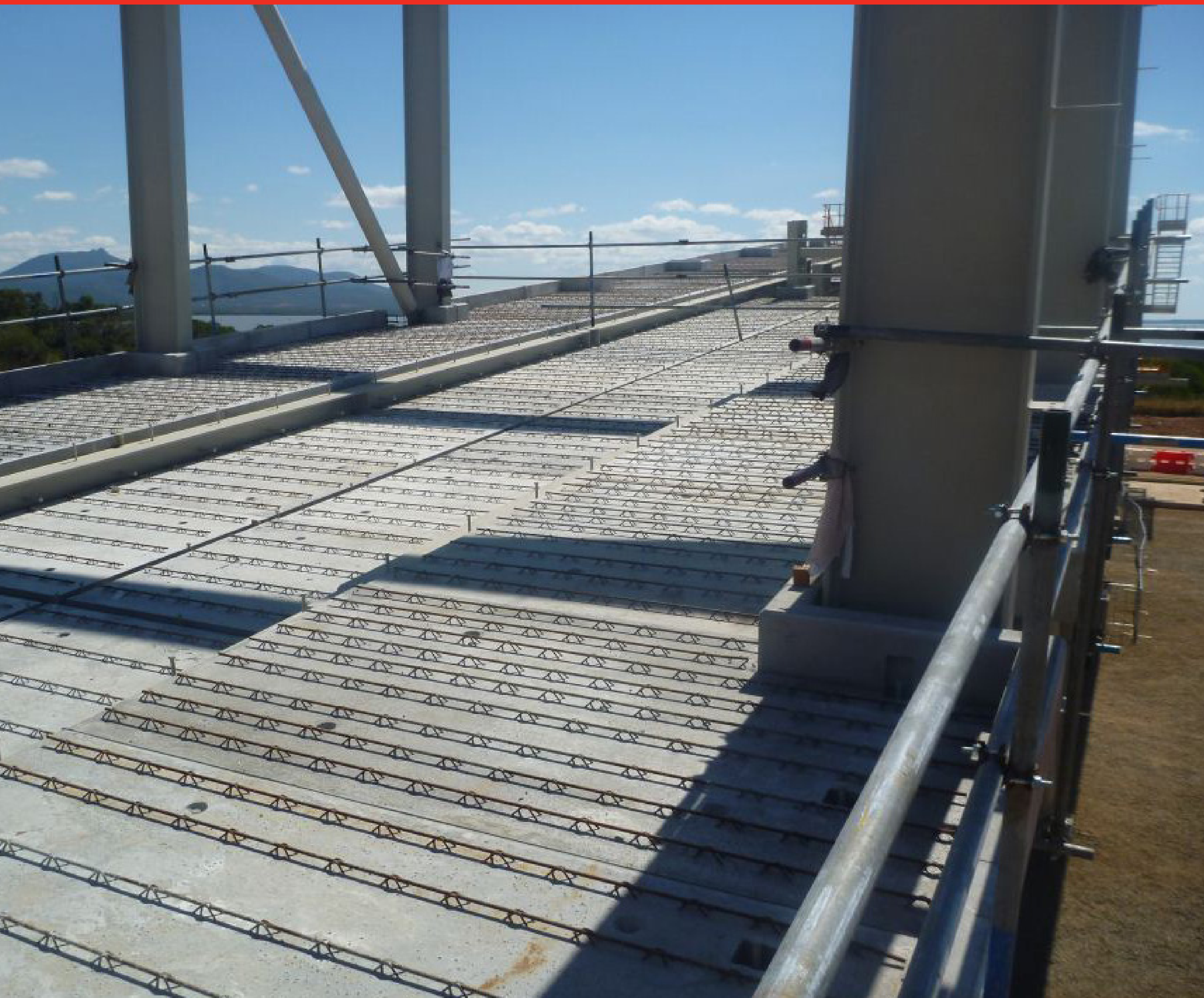
# HumeSlab® system Technical manual



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# Preamble

This Technical Manual has been prepared by Smorgon Steel Group on behalf of Humes to facilitate the design of suspended concrete slabs covering a wide range of applications using the HumeSlab™ flooring system. It is intended to be used as a technical guide for construction loading and it is a requirement of use that any designs prepared using this Technical Manual be examined and verified by a competent and qualified structural engineer.

The manual contains comprehensive data on the properties of HumeSlab™ trusses and describes some of the typical details required to achieve an equivalent monolithic slab. The procedures are based on established design methods and material properties for conventional steel reinforced concrete structures. Design criteria relating to bending, shear capacity, anchoring of reinforcement, transverse reinforcement, support conditions and any general design and construction procedures shall be referred to and approved by an industry registered structural design engineer.

# Contents



**Further Information**

For further technical information regarding HumeSlab™, contact our sales Engineers or technical representatives at Humes. For contact details, refer to back cover.

# 1.0 Introduction

The **HumeSlab™ System** (also known as Transfloor™ and by the name of the original licensors - ABE, Filigran, Kaiser-Omnia floor) has been widely used in Europe and elsewhere for over 40 years. Overseas trends indicate that this precast flooring system is a favoured method of construction for suspended concrete slabs and in some parts of Europe it accounts for 60% of all suspended work reaching production rates of 80 million square metres per year. As a precast flooring system it offers many advantages over cast in-situ floors while maintaining the full structural integrity and monolithic requirements of the slab.

In Australia this type of flooring has been in use since 1982 and in February 1988 Transfloor™ was purchased by Smorgon Steel Group and traded as Transfloor™ Australia Pty Ltd until 1991. Since 1992 the manufacture of Transfloor™ has been licensed to a number of independent precast companies. Humes, as a licensee, markets Transfloor™ as HumeSlab™, using the same expertise and technical know how developed by Transfloor™.

Figure 1: Placing a HumeSlab™ panel

Humes and Smorgon Steel Group are committed to technical support and product development of HumeSlab™.

## 2.0 The HumeSlab™ System

The HumeSlab™ system uses a combination of precast conventionally reinforced concrete panels and a poured in-situ topping as a means of constructing a typical suspended concrete slab. The use of site placed steel reinforced concrete effectively ties all the precast elements together providing safety, rigidity and structural redundancy.

### HumeSlab™ Features

**Size** - A HumeSlab™ panel is a factory made precast concrete slab of variable width up to a maximum of 2.5 metres and variable length, usually limited to about 12 metres for transport and handling purposes.

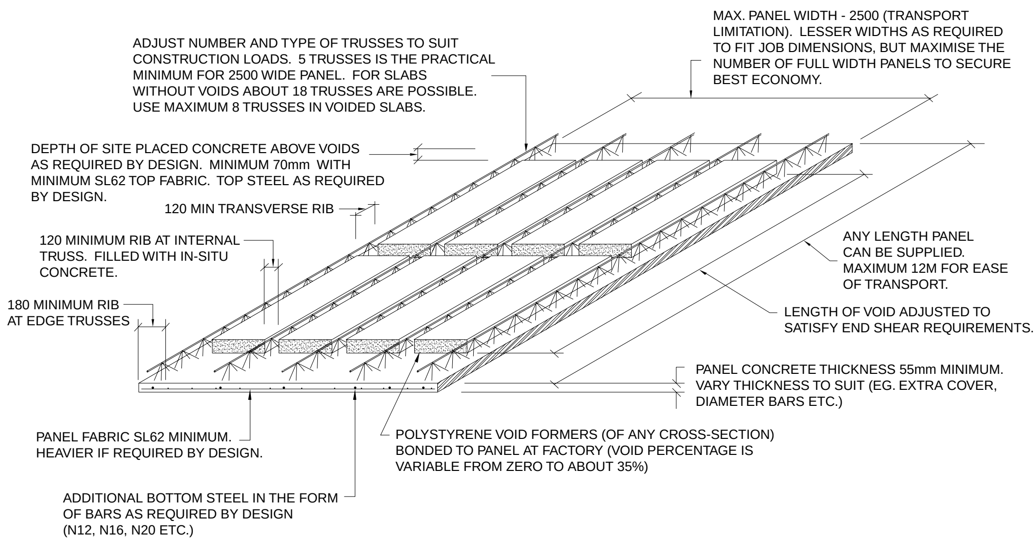
**Thickness** - The panel thickness can be varied and will depend on reinforcement size and concrete cover. For many applications a nominal thickness of 55 mm is satisfactory.

**Reinforcement** - The bottom reinforcement embedded in the panel can consist of a layer of fabric, the bottom chords of the trusses and additional reinforcing bars as required by the designer.

**Handling** - The HumeSlab™ trusses provide strength and stiffness for handling and transport, allow panels to support construction loads with a minimum of temporary propping, contribute to the bottom steel and to the top steel and can also serve as continuous bar chairs to support the top reinforcement.

**Weight Saving** - Polystyrene void formers, added at the precast factory, allow for construction of voided slabs with a significant reduction in self weight (typically 30%).

**Flexibility** - In contrast with most other prefabricated systems, HumeSlab™ imposes few restrictions on designers because there are no standard panel sizes. The length, width, thickness, plan geometry and reinforcement steel can be varied to suit design requirements and allow considerable flexibility for both the Architect and the Engineer.




NOTE
HUMESLAB UNITS CAN BE MADE TO ANY SIZE AND ANY SHAPE WITHIN THE LIMITS SHOWN ABOVE. SEMICIRCULAR OR RECTANGULAR CUT OUTS, SKEWED ENDS AND IRREGULAR SHAPES CAN BE MANUFACTURED TO SUIT PARTICULAR JOB REQUIREMENTS.

CHARACTERISTICS OF HUMESLAB™ PANELS

Figure 2: Typical characteristics of a HumeSlab™ panel

# 3.0 Advantages and Applications

The HumeSlab™ system is versatile and adaptable for use in a wide variety of structures including low-rise residential and commercial developments, high-rise steel and concrete framed structures, bridge decks, culverts and other civil applications. Generally suited to most suspended reninforced slabs.

## Cost Effective Features

**Faster construction** - Up to 150 $m^2$ per hour can be placed by crane. Total building time can be reduced significantly (refer Table 1).

**Eliminates formwork** - Most of the traditional formwork can be eliminated. HumeSlab™ panels provide both the working platform and part of the completed slab.

**Reduced propping** - Propping requirements are reduced when compared with traditional formwork which means less cluttering of the floor below and earlier access by following trades.

**Clean and safe** - Fewer trades are required resulting in a less cluttered, cleaner and safer building site. An immediate work platform is provided.

**Lighter structure** - Use of polystyrene void formers reduces the self weight of the slab and provides cost savings in foundations, columns and beams. The void formers also reduce the volume of in-situ concrete.

**Soffit finish** - A class 2 off-form grey finish is easily achieved, suitable for painting with minimum preparation (refer Figure 3). Panel joints do require filling if a flat soffit is desired (refer Figure 30).

**Balcony Upstands** - Can be provided as an integral part of the HumeSlab™ panel. Eliminates costly edge formwork and scaffolds. Allows early installation of temporary or permanent balustrades. (Generally the standard size is 300mm high x 150mm wide. For other sizes please consult your Humes representative.)

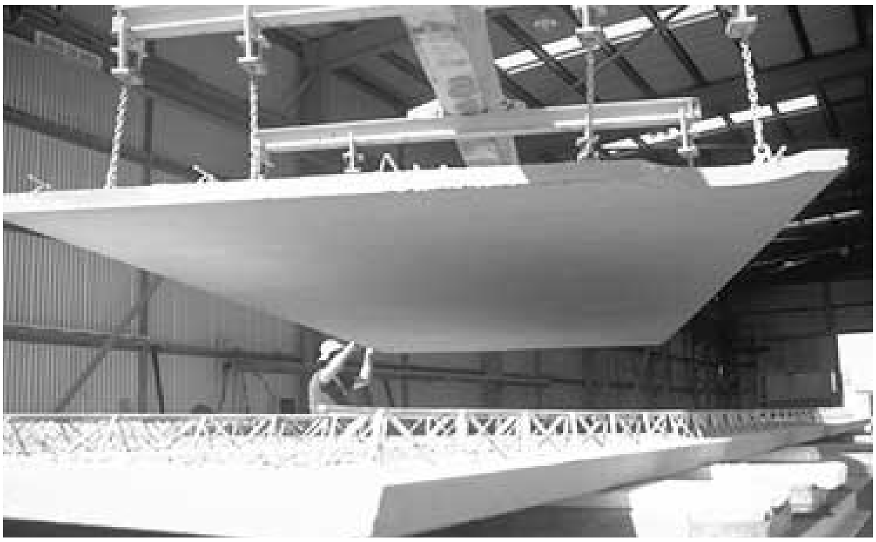


Figure 3: High quality off form soffit finish

# Advantages and Applications

| Traditional Formwork | | | | HumeSlab™ System | | | |
|---|---|---|---|---|---|---|---|
| **Activity** | **Labour** | **Day/s** | **Hours** | **Activity** | **Labour** | **Day/s** | **Hours** |
| Erect & prop wall panels | 2 Dogman<br>2 Labourers | 1<br>1 | 16<br>16 | Erect and prop wall panels | 2 Dogman<br>2 Labourers | 1<br>1 | 16<br>16 |
| Grout wall panels | 2 Labourers | 1 | 16 | Grout wall panels | 2 Labourers | 1 | 16 |
| Support frames | 3 Scaffolders | 2 | 48 | **Support frames** | **3 Scaffolders** | **1** | **24** |
| Place ply formwork | 4 Carpenters<br>2 Labourers | 3<br>3 | 96<br>48 | **Place HumeSlab™ panels** | **2 Dogman**<br>**2 Carpenters** | **1**<br>**1** | **16**<br>**16** |
| Place reinforcement | 4 Steel Fixers | 2 | 64 | **Place top reinforcement** | **4 Steel Fixers** | **1** | **32** |
| Pour concrete | 8 Labourers | 1 | 64 | Pour concrete | 8 Labourers | 1 | 64 |
| Strip formwork and clean up | 4 Carpenters<br>2 Scaffolders | 2<br>2 | 64<br>32 | **Remove propping frames** | **2 Scaffolders** | **1** | **16** |
| **Total Cycle (approximate hours)** | | | **464** | **Total Cycle (approximate hours)** | | | **216** |
| **Typical Cycle** | | | **0.62 hrs/m²** | **Typical Cycle** | | | **0.29 hrs/m²** |

Table 1: Comparison of cycle times and labour requirements for slab over precast walls-Brookland Apartments.

Figure 4: Column penetration in HumeSlab™ panel

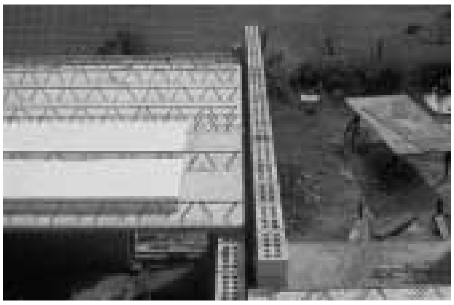


Figure 5: HumeSlab™ on load bearing block walls

**Flexibility in design** - HumeSlab™ is an engineered product made to suit individual project requirements. Penetrations, cantilevers and unusual panel shapes can be easily accommodated (refer Figure 4).

**Eliminates bar chairs** - If concrete cover and overall slab thickness are suited to the truss type, top reinforcement can be supported directly on the HumeSlab™ trusses.

**Four easy steps to build with HumeSlab™**

1. At the time of planning, contact Humes to discuss the use of HumeSlab™ for your application.

2. Supplier personnel will then assess and arrange for a preliminary design and prepare concept layout plans and a quotation.

3. Upon placement of the order a detailed layout plan is prepared based on the documentation provided. This information is returned to the builder and engineering consultant for checking and approval.

4. After approval has been obtained for dimensional accuracy and engineering integrity, the panels are produced and delivered to site at a time specified by the builder.

Figure 6: HumeSlab™ panels placed on steel frame structures

# 4.0 Material & Product Specification

### 4.1 Reinforcement

HumeSlab™ trusses are fabricated from plain round hard drawn 500L grade bar conforming to AS4671. The diagonal bars of the truss are electronically welded to both the top and bottom chords. Weld tests are carried out at regular intervals as part of the Smorgon Steel Group Quality Assurance programme.

All fabric used in the panels is welded wire fabric, grade 500L conforming to AS4671 and all bar reinforcement is grade 500N conforming to AS4671.

### 4.2 Panel concrete

The panel concrete is Normal Class Concrete as defined in AS3600. A typical concrete specification is given below but the Engineer should also nominate special class concrete if used.

Minimum strength grade . . . . . . . . . . . . . . . . . . N40
Slump . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 80 mm
Maximum size of aggregate . . . . . . . . . . . . . . . 14 mm (nominal)
Cement . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . General purpose

Figure 7: Reinforcement in casting bed ready for concrete pour

### 4.3 Polystyrene void formers

The expanded polystyrene (EPS) void formers are made of a light weight cellular plastic material comprising 98% air. A class SL material is used having a density of 13.5 kg/m3. All other physical properties of the EPS are in accordance with AS1366, Part 3-1992. Designers should note that the EPS is produced with a fire retardant additive that allows it to self extinguish almost immediately after the fire source is removed. The level of toxicity of EPS in a fire situation is not greater than that of timber or other commonly used building materials.

### 4.4 Topping concrete

It is essential that the site concrete, whether placed over panels or over void formers, is of a high quality, and that placement and curing is of a satisfactory standard to minimise surface cracking due to plastic shrinkage or other causes.

In-situ concrete thickness over void formers will be governed by cover, quantity, size and laps of top reinforcement. A minimum of 70 mm should be used.

### 4.5 Truss specifications

| Generic Truss Reference | CSR Humes Product Code | Top Chord Diameter | Height (H) (mm) | Mass (kg/m) |
|---|---|---|---|---|
| T80/10 | TRUS8010C | 9.5 | 82 | 1.77 |
| T110/10 | TRUS11010C | 9.5 | 111 | 1.86 |
| T150/10 | TRUS15010 | 9.5 | 154 | 2.06 |
| T190/10 | TRUS19010C | 9.5 | 191 | 2.21 |
| T110/12 | TRUS11012C | 11.9 | 112 | 2.21 |
| T150/12 | TRUS15012C | 11.9 | 155 | 2.41 |
| T190/12 | TRUS19012C | 11.9 | 192 | 2.56 |

**Typical Sections:**

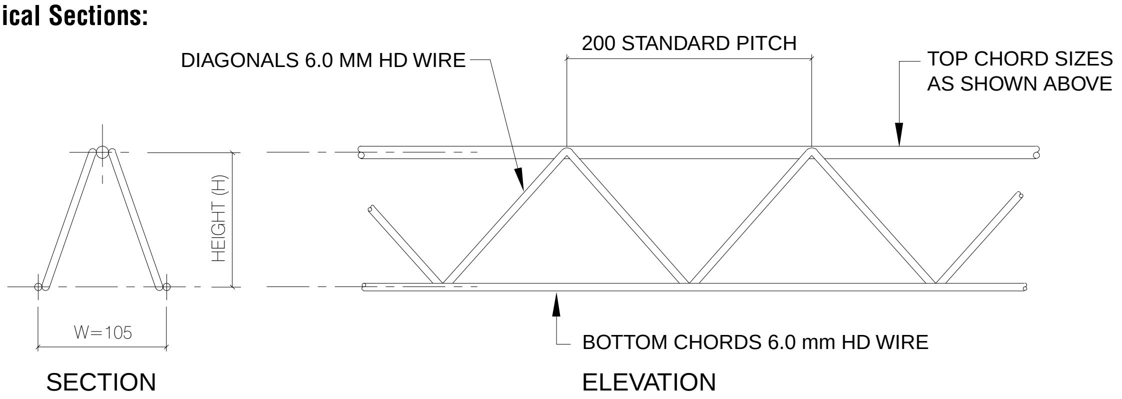



Figure 8: Truss properties and section details

# 5.0 Design Principles

The structural design of HumeSlab™, or any precast concrete floor system, should not only deal with the calculation of bending moment and shear force capacity of the separate units, but also with the total coherence of the floor. In the final stage, the individual components should be connected in a manner that ensures adequate overall capacity with interaction between the units and the supporting structure. Two distinct stages must be checked when designing with HumeSlab™.

1. The non-composite panel during construction - stresses occurring only in the precast units resulting from lifting, transportation  and the weight of the wet concrete.
2. The composite floor slab after hardening of the in-situ concrete.

## 5.1 Design for bending

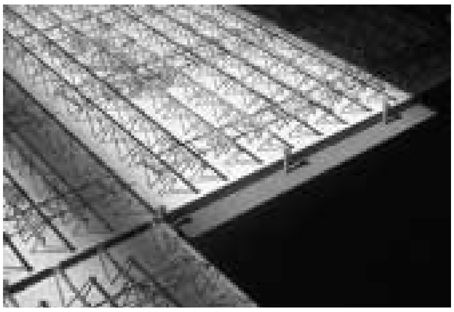


Figure 9: Biaxial trusses - panels engineered to suit project requirements

Accepted principles of Ultimate Strength Theory applies to the design of HumeSlab™ since the finished slab can be considered as monolithic. A prerequisite for this is that the uptake of shear forces at the interface between precast and in-situ concrete is proven. The shear capacity, at this interface, has been shown to be adequate by overseas research (Reference 1) and some early testing done at the University of Queensland (Reference 2).

The system is best suited to one way action, however, two way action can be achieved by eliminating void formers to allow placement of transverse bars. The transverse bars should be placed near the upper surface of the panel ensuring that in-situ concrete flows under the bars and anchorage is achieved. Note that a reduced effective depth for the transverse reinforcement will have to be used.

In a uniaxial design the precast panel will normally contain all of the bottom reinforcement required in the final design which can consist of a light fabric, truss bottom chords and additional bar reinforcement. It should be noted that the presence of voids will not usually result in design of the section as a tee beam since large amounts of steel are required to shift the neutral axis below the top of the void. Refer to Figure 10 for a general cross section of a finished slab.

Note: Minimum slab thickness of 160mm.

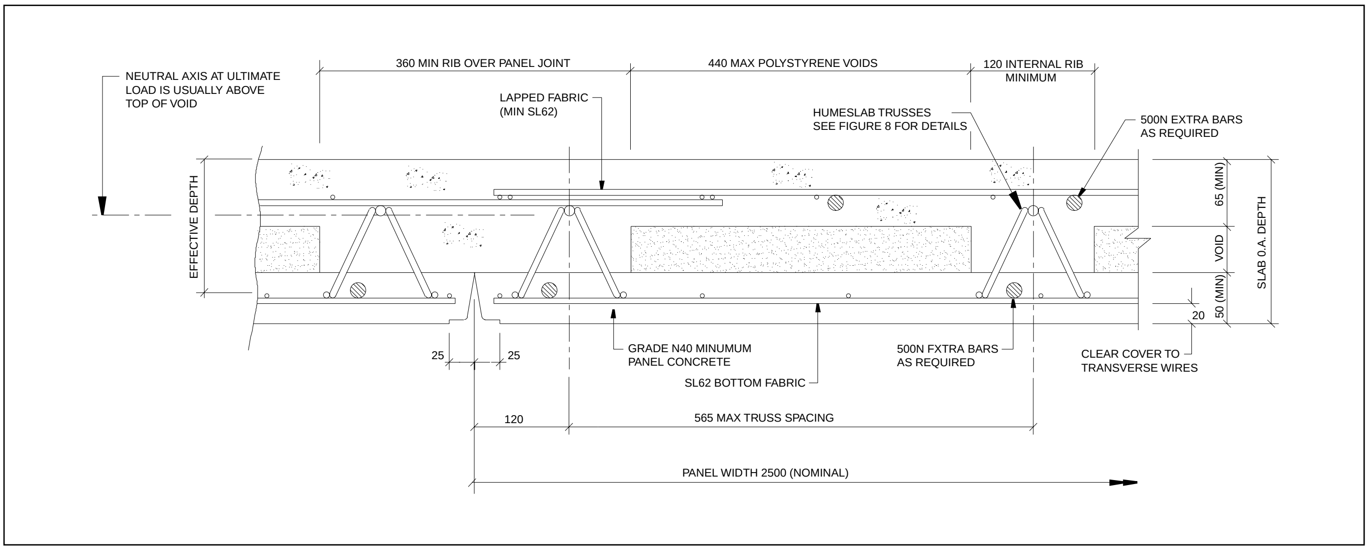



Figure 10: General cross section of finished slab

Figure 11: Penetrations for services can be cast into panel

## 5.2 Precast in-situ interface

The required capacity at the interface can be calculated in accordance with AS3600 Clause 8.4. The level of surface roughness is somewhat open to interpretation but can be considered as rough with small ridges and undulations. The surface roughness achieved during the casting process is satisfactory when, at the same time, truss web members are used as shear plane reinforcement.

If an intentionally roughened surface is specified, care should be taken not to disturb the grain structure of the concrete or dislodge aggregates near the surface. A vibrated level or light broom finish is all that is required.

## 5.3 Vertical shear

If a voided slab is used the shear forces can only be carried by the concrete in the rib sections. Voids must be terminated in regions of high shear (at supports and point loads) and will generally not be included within one slab depth from the section at which the ribs are just sufficient to resist the applied shear.

The overall slab thickness is not normally controlled by shear strength requirements but, when required, the diagonal wires of the trusses may be treated as inclined stirrups (Reference 1) provided the pitch of the wires does not exceed the depth of the slab, trusses extend through the full slab depth and truss spacing does not exceed the recommended stirrup spacing given in AS3600.

When the precast element is used to form a wide shallow beam (band beam/slab system on columns) and shear reinforcement is required, the ligatures should extend over the entire section depth and tie into the precast element. However, if the actual shear is less than the shear capacity and the beam depth is less than half the beam width, nominal shear ligatures can be incorporated as shown in Figure A8 in Appendix A.

## 5.4 Load distribution

When a slab is subjected to concentrated loads, the distribution of the load across longitudinal joints should be considered. The transverse load distribution in composite precast element floors is similar to cast in situ floors. Load distribution between precast elements is provided by the shear resistance of the in-situ concrete section at the joint (Figure 10). Where trusses are not located adjacent to the joint, additional transverse bars may be placed in the site concrete over the panel joints. The inclusion of transverse ribs (Figure 2) would also contribute to the load distribution capabilities.

Figure 12: Slab and band beam system

| Panel Thickness | Panel Width |
|---|---|
| 55mm<br>60mm<br>65mm<br>70mm | 2502mm<br>2504mm<br>2506mm<br>2508mm |

Table 2 Actual panel thickness and width

Table 3: Standard truss thickness and typical fire rating for voided slabs

| Truss type | Slab thickness | Fire rating |
|---|---|---|
| T80 | 160mm | 2 Hours |
| T110 | 190mm | 3 Hours |
| T150 | 230mm | 3 Hours |
| T190 | 270mm | 3 Hours |

Note:

1. Table 3 is based on 20 mm cover to top and bottom reinforcement and minimum 65 mm topping concrete over polystyrene void formers.
2. The overall slab thickness is the minimum that can be used with the nominated truss type.
3. The actual panel width will depend on the panel thickness used due to the tapered edge forms.
4. Top reinforcement can be supported directly on trusses when the above slab/truss combinations are used and reinforcement is arranged as shown in Figure 10.

### 5.5 Durability requirements and fire rating

Since HumeSlab™ panels are cast on rigid steel forms and are subjected to intense compaction the reinforcement cover requirements at the bottom of the slab can be reduced compared to in-situ slabs (AS3600 Table 4.10.3.4). If severe exposure conditions are specified the panel thickness is increased to allow for the increased cover requirements.

Fire resistance requirements for slabs constructed with HumeSlab™ panels can be determined by referring to clause 5.5.1 (b) and 5.5.3 (a) of AS3600. If a voided slab is used the effective thickness of the slab is calculated as the net cross sectional area divided by the width of the cross section. Typical fire resistance periods are shown in Table 3. Higher fire ratings can be achieved with increased cover to reinforcement and decreased thickness of polystyrene voids.

## 5.6 Support conditions

Figure 13: panels connecting to precast walls

The correct detailing of precast concrete involves the consideration of the design, manufacture and construction requirements at the start of the project. It is important to consider detailing during the early design stages so as to obtain the full benefits of any precast system.

As with in-situ floors, when designing with HumeSlab™, attention must be given to anchoring of steel reinforcement at the supports. Steel reinforcement end details are specified in AS3600 clause 9.1.3 and the amount of steel reinforcement to be carried into the support will depend on the end restraint condition.

Connections between HumeSlab™ panels and supporting members present few problems since continuity can be provided by lapping the panel steel reinforcement with steel bars projecting from the supporting beams or walls. In general, it is sufficient to anchor 50% of the total positive moment steel reinforcement required at mid span. The bottom chords of the HumeSlab™ trusses which end at the front edge of the support do not constitute part of this requirement. Therefore the details suggested in Figure 14 can be safely used provided the field steel reinforcement is satisfactorily anchored above the support.

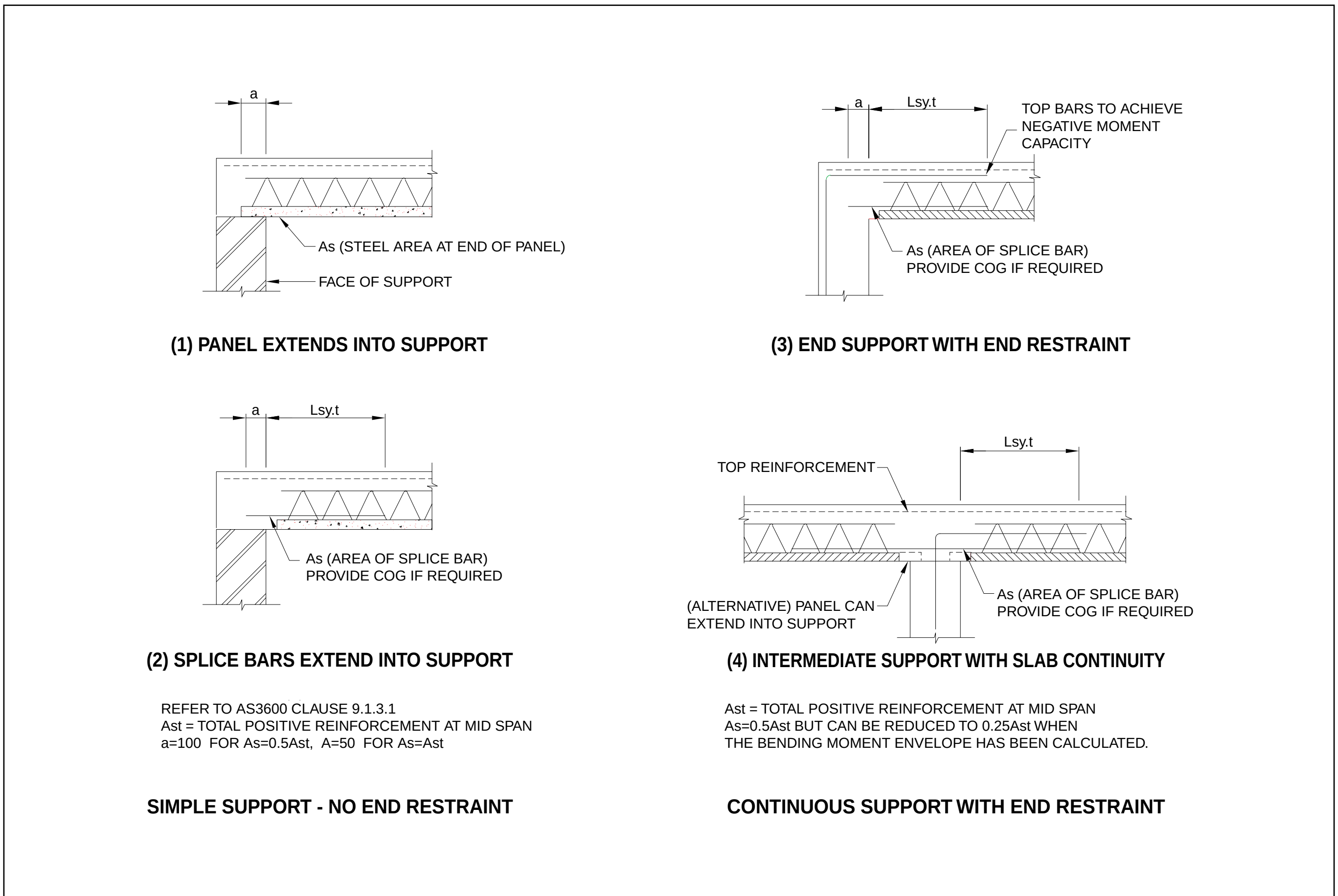



Figure 14: Reinforcement end details conforming to AS3600

## 5.7 Design for construction loads

Figure 15: Unpropped HumeSlab™ panels on precast beams

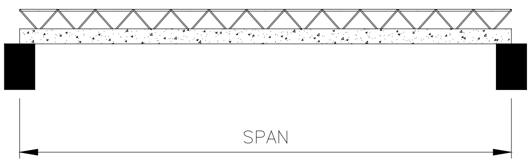
Selecting a panel specification to support construction loads should provide a panel with sufficient strength and stiffness to carry the mass of wet concrete and construction live loads without exceeding safe limits for stress and/or deflection. The loading to be considered at this stage of the design is based on the Formwork Code AS3610 and will include:

- Precast panel self weight
- Dead load of wet in situ concrete
- Live loads due to stacked materials
- Live load due to workmen and equipment
- Localised mounding of in-situ concrete during placing.

Prop spacing during construction will be controlled by one of the following criteria.

- Bending moment capacity determined by limiting the tensile stress in the panel concrete to less than the characteristic tensile strength.

| Truss Type | Overall Slab Thk | Truss spacing (mm) | | | | | | | Maximum span between temporary supports (m) |
|---|---|---|---|---|---|---|---|---|---|
| | | 565 | 450 | 320 | 200 | 565 | 450 | 320 | |
| T80/10 | 160 | 2.1 | 2.2 | 2.3 | 2.5 | 2.3 | 2.3 | 2.3 | |
| | 180 | 2.0 | 2.1 | 2.2 | 2.4 | 2.2 | 2.3 | 2.4 | |
| T110/10 | 190 | 2.2 | 2.3 | 2.5 | 2.7 | 2.5 | 2.5 | 2.7 | |
| | 200 | 2.2 | 2.2 | 2.4 | 2.7 | 2.4 | 2.5 | 2.6 | |
| | 220 | 2.1 | 2.2 | 2.3 | 2.6 | 2.4 | 2.5 | 2.6 | |
| T150/10 | 230 | 2.4 | 2.6 | 2.8 | 3.1 | 2.8 | 3.0 | 3.1 | |
| | 250 | 2.3 | 2.5 | 2.7 | 3.0 | 2.8 | 2.9 | 3.1 | |
| T190/10 | 270 | 2.6 | 2.8 | 3.0 | 3.4 | 3.2 | 3.3 | 3.5 | |
| | 300 | 2.5 | 2.6 | 2.9 | 3.3 | 3.1 | 3.2 | 3.4 | |
| | 320 | 2.4 | 2.6 | 2.8 | 3.2 | 3.1 | 3.2 | 3.4 | |
| | 350 | 2.3 | 2.5 | 2.7 | 3.1 | 3.0 | 3.1 | 3.3 | |
| | 400 | 2.2 | 2.3 | 2.5 | 2.9 | 2.9 | 3.0 | 3.1 | |
| T110/12 | 190 | 2.4 | 2.5 | 2.7 | 3.1 | 2.7 | 2.8 | 3.0 | |
| | 200 | 2.4 | 2.5 | 2.7 | 3.0 | 2.7 | 2.8 | 3.0 | |
| | 220 | 2.3 | 2.4 | 2.6 | 2.9 | 2.7 | 2.8 | 2.9 | |
| T150/12 | 230 | 2.8 | 2.9 | 3.2 | 3.6 | 3.2 | 3.4 | 3.6 | |
| | 250 | 2.7 | 2.8 | 3.1 | 3.4 | 3.2 | 3.3 | 3.5 | |
| T190/12 | 270 | 3.0 | 3.2 | 3.5 | 3.9 | 3.7 | 3.8 | 4.0 | |
| | 300 | 2.9 | 3.0 | 3.3 | 3.7 | 3.6 | 3.7 | 3.9 | |
| | 320 | 2.8 | 2.9 | 3.2 | 3.6 | 3.5 | 3.7 | 3.8 | |
| | 350 | 2.7 | 2.8 | 3.1 | 3.5 | 3.5 | 3.6 | 3.7 | |
| | 400 | 2.4 | 2.7 | 2.9 | 3.3 | 3.4 | 3.5 | 3.6 | |
| | | Solid Slab | | | | Voided Slab | | | |




For a more detailed design, go to the VIP section at www.smorgonarc.com.au

Table 4: Propping requirements - single span during construction

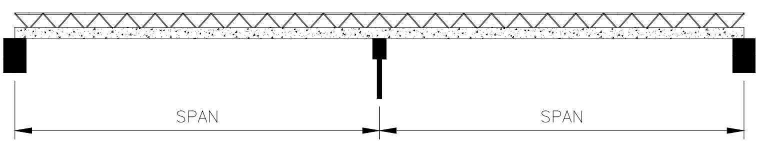
- Bending moment capacity may also be governed by the compressive stress in the top chord of the truss. This should be limited so that buckling of the top chord does not occur.
- Shear capacity will be determined by the buckling strength of the truss diagonal wires.

Load capacities and thus distance between temporary supports will depend on panel thickness, truss spacing and whether the slab is voided or solid. The unpropped spans given in Tables 4 and 5 have been calculated by analysing the HumeSlab™ panel as an uncracked section using a transformed area method to determine stresses in concrete and steel during construction.

Since this is a serviceability limit state design, unfactored loads have been used. The tensile stress in the panel concrete is limited to $0.6\sqrt{f'c}$ (AS3600, Clause. 6.1.1.2) and the compressive force in the truss wires is limited to AS4100 Clause. 6.1

Maximum span between temporary supports (m)

| Truss Type | Overall Slab Thk | Truss spacing (mm) Solid Slab 565 | Solid Slab 450 | Solid Slab 320 | Solid Slab 200 | Voided Slab 565 | Voided Slab 450 | Voided Slab 320 |
|---|---|---|---|---|---|---|---|---|
| T80/10 | 160 | 2.4 | 2.5 | 2.6 | 2.8 | 2.6 | 2.7 | 2.8 |
| | 180 | 2.3 | 2.4 | 2.5 | 2.7 | 2.5 | 2.6 | 2.7 |
| T110/10 | 190 | 2.5 | 2.6 | 2.8 | 3.1 | 2.8 | 2.9 | 3.1 |
| | 200 | 2.5 | 2.6 | 2.7 | 3.0 | 2.8 | 2.9 | 3.0 |
| | 220 | 2.4 | 2.5 | 2.6 | 2.9 | 2.8 | 2.8 | 3.0 |
| T150/10 | 230 | 2.8 | 2.9 | 3.2 | 3.6 | 3.3 | 3.4 | 3.6 |
| | 250 | 2.7 | 2.8 | 3.0 | 3.4 | 3.2 | 3.3 | 3.5 |
| T190/10 | 270 | 2.7 | 3.1 | 3.4 | 3.9 | 3.6 | 3.8 | 4.0 |
| | 300 | 2.5 | 3.0 | 3.3 | 3.7 | 3.6 | 3.7 | 3.9 |
| | 320 | 2.4 | 2.8 | 3.2 | 3.6 | 3.5 | 3.6 | 3.8 |
| | 350 | 2.6 | 2.8 | 3.0 | 3.4 | 3.4 | 3.6 | 3.8 |
| | 400 | 1.9 | 2.3 | 2.9 | 3.2 | 3.3 | 3.4 | 3.6 |
| T110/12 | 190 | 2.8 | 2.9 | 3.1 | 3.5 | 3.1 | 3.2 | 3.4 |
| | 200 | 2.7 | 2.8 | 3.1 | 3.4 | 3.1 | 3.2 | 3.4 |
| | 220 | 2.6 | 2.7 | 2.9 | 3.3 | 3.1 | 3.1 | 3.3 |
| T150/12 | 230 | 3.1 | 3.3 | 3.6 | 4.0 | 3.7 | 3.9 | 4.1 |
| | 250 | 3.0 | 3.2 | 3.5 | 3.9 | 3.6 | 3.8 | 4.0 |
| T190/12 | 270 | 2.7 | 3.3 | 3.9 | 4.4 | 4.1 | 4.4 | 4.6 |
| | 300 | 2.5 | 3.0 | 3.8 | 4.2 | 3.9 | 4.2 | 4.5 |
| | 320 | 2.4 | 2.8 | 3.6 | 4.1 | 3.8 | 4.2 | 4.4 |
| | 350 | 2.2 | 2.6 | 3.5 | 3.9 | 3.7 | 4.1 | 4.3 |
| | 400 | 1.9 | 2.3 | 3.1 | 3.7 | 3.4 | 3.9 | 4.1 |




For a more detailed design, go to the VIP section at www.smorgonarc.com.au

Table 5: Propping requirements - two or more spans during construction

Tables 4 and 5 can be used to determine propping requirements, provided construction loads are specified as in AS3600, the HumeSlab™ panel has a minimum thickness of 55 mm and is reinforced with at least SL62 fabric, and the load from stacked materials does not exceed 4 KPa prior to placement of top concrete.

Where special construction loads are specified and the above conditions do not apply, the determination of prop spacing is possible using the TranSpan™ software. This software is available from Humes or can be accessed by visiting Smorgon Steel Group's web site on http://www.smorgonsteel.com.au/reinforcing

### 5.8 Deflection during construction

At typical propping spans of up to 2.7 m tests have shown that deflections under construction loads should not exceed 2 mm.

In cases where unpropped spans exceeding 3.0 m are proposed the deflection should be checked to ensure it does not exceed the limits set in AS3600. Conventional transformed section methods can be used to predict the elastic behaviour of a HumeSlab™ panel but note that the load used to calculate deflections during construction should be the dead load only (wet concrete and panel).

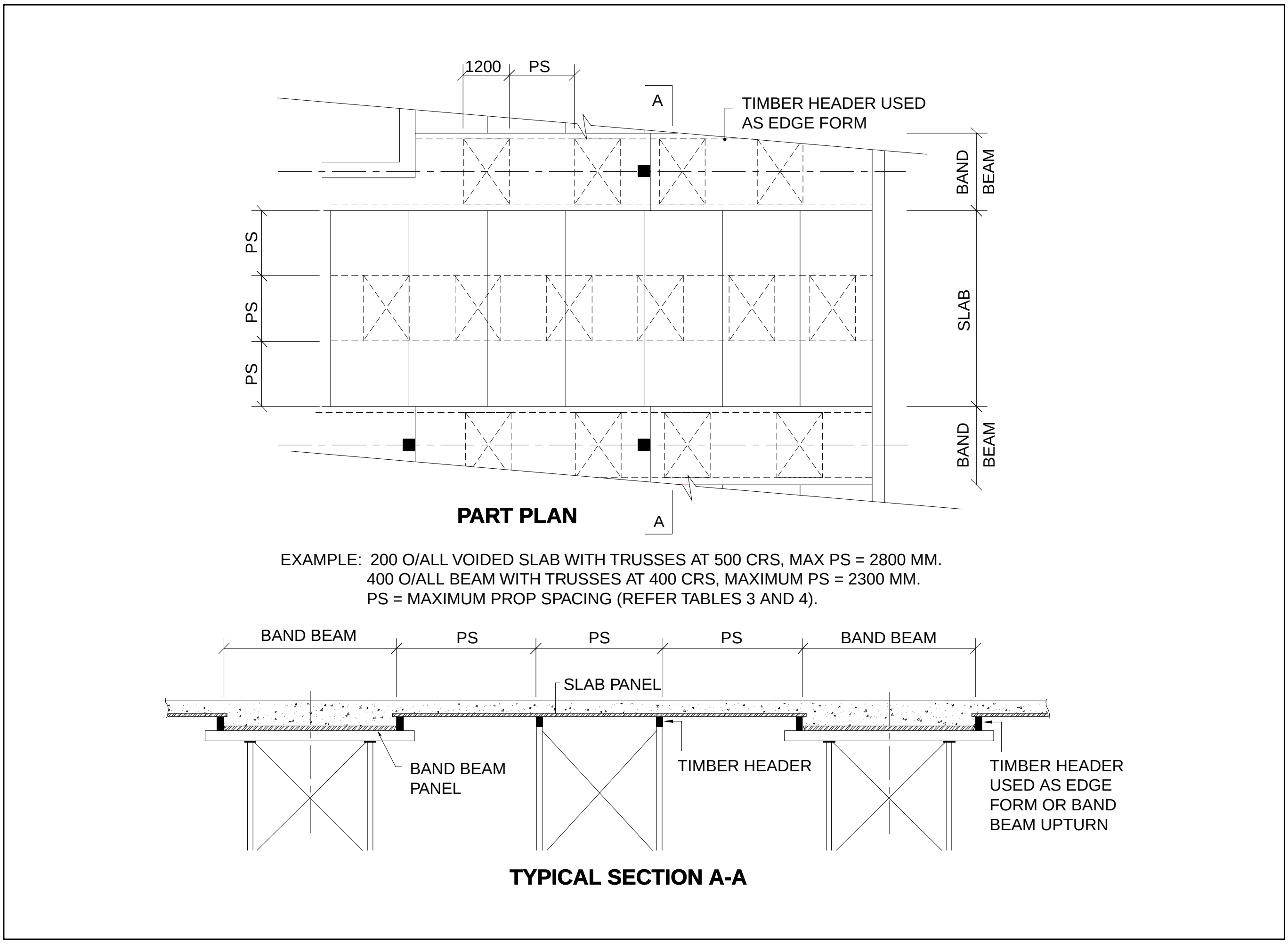



Figure 16: Typical propping layout

# 6.0 Final Slab Design

Tables 6 and 7 can be used to to estimate values for the final slab design. However, it should be noted that this information is indicative and should only be used for estimating purposes and does not replace the need for a qualified design Engineer. The calculations are based on the following criteria.

1. Design is to AS3600 Clause 7.2 and Section 9.
2. Cover to reinforcement = 20 mm (exposure classification B1).
3. Concrete class: 32 MPa for in situ topping and 50 MPa for precast.
4. Bottom steel reinforcement content to include SL62 fabric (min).
5. Superimposed loads include a dead load of 0.5 KPa, the remainder is live load.
6. In-Situ concrete allows for polystyrene void formers and is given in $m^3/m^2$

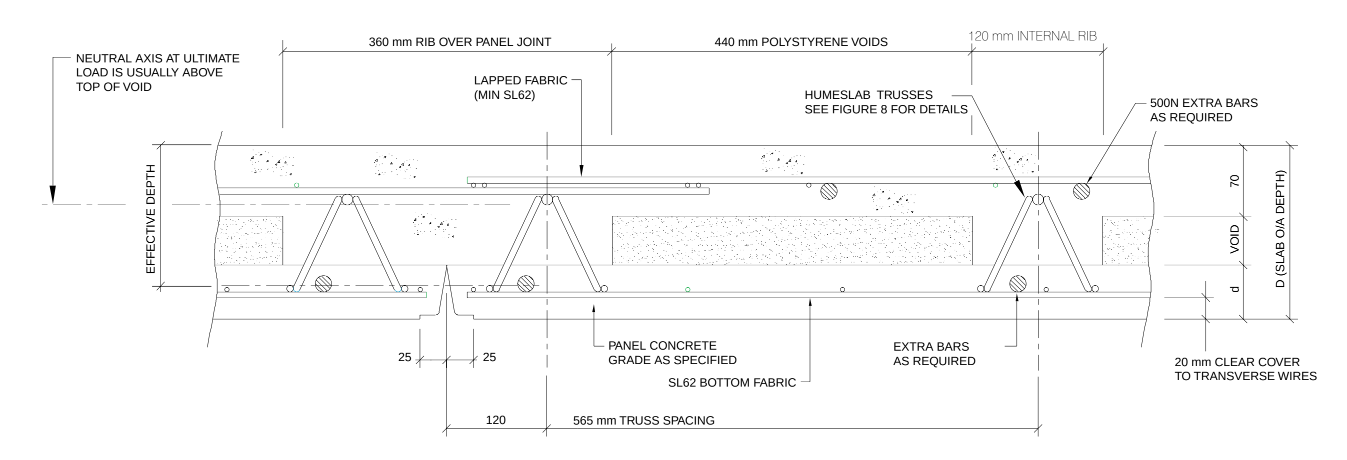



Figure 17: Slab section relating to Tables 6 and 7

| Slab Depth | | SW | Reinf. (Kg/m²) | | In situ Conc | Superimposed load (KPa) for span (m) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| D | d | KPa | Top | Bot. | | 8.5 | 8.0 | 7.5 | 7.0 | 6.5 | 6.0 | 5.5 | 5.0 | 4.5 | 4.0 | 3.5 |
| **160** | 55 | 3.3 | 3.1 | 4.3 | 0.080 | | | | | | | | | 2.2 | 4.1 | 5.9 |
| **190** | 55 | 3.6 | 3.1 | 5.4 | 0.090 | | | | | | | 2.0 | 3.5 | 5.2 | 7.3 | 10.5 |
| **230** | 55 | 3.9 | 3.1 | 6.5 | 0.104 | | | | | 2.5 | 3.7 | 5.0 | 6.7 | 9.1 | 12.3 | |
| **270** | 55 | 4.3 | 3.1 | 7.5 | 0.120 | | | 2.5 | 3.7 | 4.8 | 6.2 | 8.1 | 10.5 | 13.8 | | |
| **300** | 55 | 4.6 | 3.1 | 7.9 | 0.130 | 2.0 | 2.8 | 3.7 | 4.7 | 6.1 | 7.8 | 10.0 | 12.8 | | | |

Table 6: Single simply supported span (voided slab)

| Slab Depth | | SW | Reinf. (Kg/m²) | | In situ Conc | Superimposed load (KPa) for span (m) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| D | d | KPa | Top | Bot. | | 9.5 | 9.0 | 8.5 | 8.0 | 7.5 | 7.0 | 6.5 | 6.0 | 5.5 | 5.0 | 4.5 |
| **160** | 55 | 3.3 | 4.4 | 4.3 | 0.080 | | | | | | | | | 0.5 | 2.5 | 4.5 |
| **190** | 55 | 3.6 | 4.9 | 5.0 | 0.090 | | | | | | | 1.5 | 2.5 | 4.1 | 5.5 | 7.5 |
| **230** | 55 | 3.9 | 5.6 | 6.1 | 0.104 | | | | | 2.0 | 3.4 | 4.5 | 5.8 | 7.5 | 9.7 | |
| **270** | 55 | 4.3 | 6.1 | 6.8 | 0.120 | | | 2.5 | 3.3 | 4.3 | 5.4 | 6.8 | 8.6 | 10.9 | | |
| **300** | 55 | 4.6 | 6.7 | 7.6 | 0.130 | 2.5 | 3.2 | 4.0 | 5.0 | 6.2 | 7.6 | 9.4 | 11.7 | | | |

Table 7: Multiple continuous span (voided slab)

# 7.0 Seismic Conditions

Although Australia is classified as a “low risk” area, in terms of earthquake damage, the need for seismic design in building structure was highlighted by the Newcastle earthquake of 1989. Building structures are to be designed for earthquake loading depending on the “earthquake design category” as specified in AS1170.4.

Seismic considerations for HumeSlab™ will follow the same design rules as for in-situ floors but will require adequate detailing to achieve seismic integrity at the connections. The main criteria to consider is:

- maintain structural integrity without collapse of all or a significant part of the structure;
- achieve ductility of both precast elements and their connections;
- provide structural continuity;
- design and detail structural elements such that they may be produced economically and erected easily.

## 7.1 Structural integrity

Figure 18: HumeSlab connecting to precast walls

It has generally been found that in-situ floor slabs, acting monolithically with supporting beams, are very capable of transmitting lateral forces unless the number of large openings is excessive.

HumeSlab™, acting monolithically, will adequately transmit lateral loads through diaphragm action. The strength and ductility of the overall structural system will depend on the integrity of the joint detailing and in particular, the connections between the floor (horizontal diaphragm) and the supporting structure.

The majority of reported damage (Reference 3.0) caused to precast construction during earthquakes is confined to the joints and connections and can be summarised as follows:

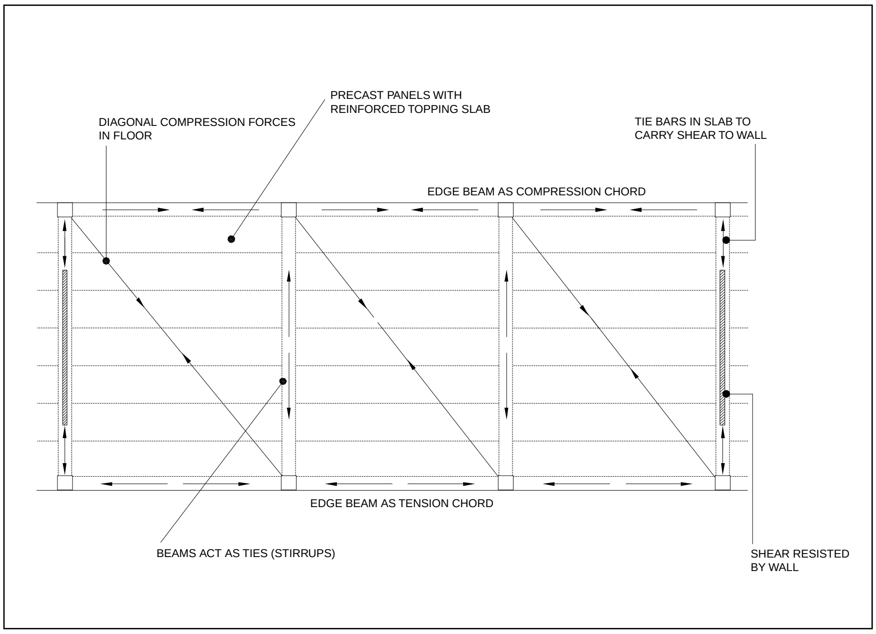



Figure 19: Actions in a typical diaphragm

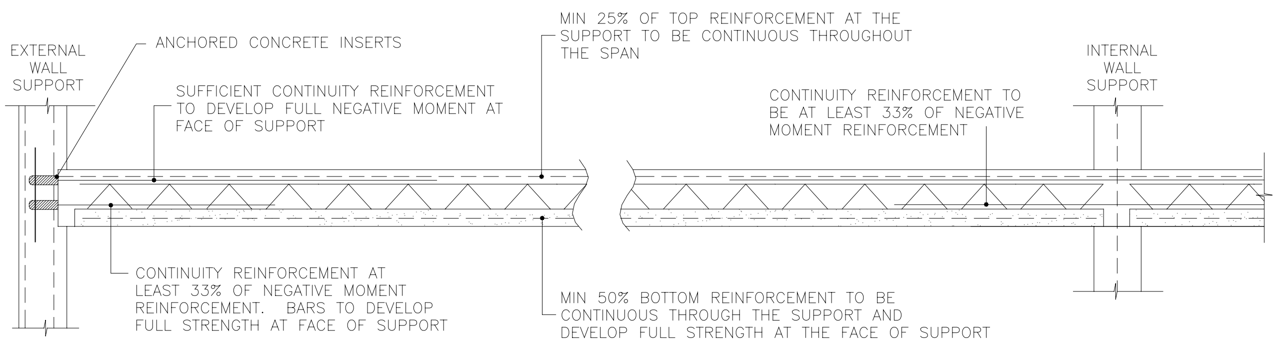
- Failure of connection between wall panel and roof system resulting in roof failure, tilting of wall panels and increased stresses in the lower level floor connections.
- Failure of connection between wall panel and floor system.
- Flexibility of thin cast in-situ topping slab that forms the horizontal diaphragm causing overstressing and cracking resulting in separation from the precast elements.

The 1988 earthquake in Armenia highlighted some of the problems caused by inadequately detailed precast construction (Reference 3). A common form of construction for medium rise residential buildings was to use precast concrete panels or frames for the vertical elements and precast concrete floor planks without the addition of a topping slab. These precast systems performed poorly due mainly to the inadequate provision of viable load paths through inadequate tying of the horizontal floor planks to the vertical elements and to each other for effective diaphragm action.

### 7.2 Diaphragm action

Horizontal loads from earthquakes are usually transmitted to the vertical cores or shear walls by the roof and floor acting as horizontal diaphragms. The floor can be analysed by the ‘strut and tie’ method or by considering the floor to act as a deep horizontal beam. The central core, shear walls or other stabilising components act as supports with the lateral loads being transmitted to them as shown in Figure 19.

As stated by Clough (Reference 4), “In zones of high seismic intensity, or with configurations which impose large in-plane compatibility forces under lateral load, diaphragms joined by cast in place reinforced concrete usually are satisfactory”. It is essential to ensure that the topping is adequately bonded to the precast elements such as in precast element floors where the topping is bonded by mechanical connectors (wire truss as in-plane reinforcement). Without this, separation can occur and the topping may buckle when subject to diagonal compression from diaphragm action.




**TYPICAL SLAB REINFORCEMENT DETAILS FOR EARTHQUAKE LOADING (Refer AS3600, Appendix A)**

Figure 20: Detailing requirements for earthquake loading

## 7.3 Detailing requirements for seismic loads

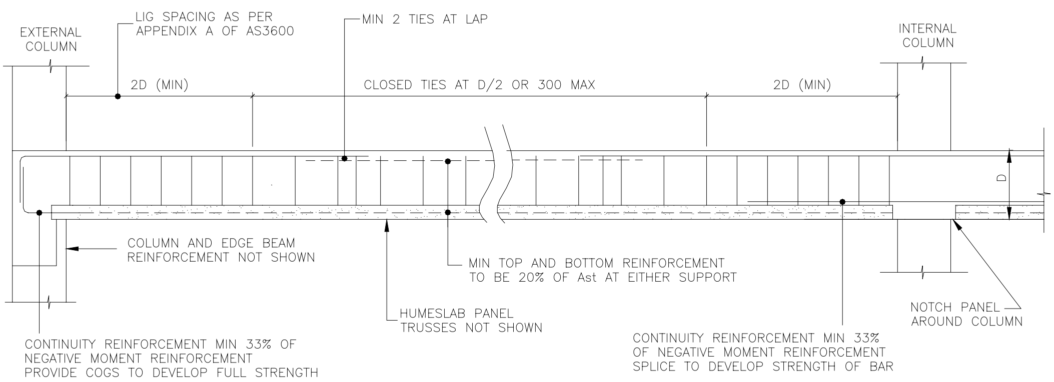
Designers should ensure that not only is there an adequate load path for forces that need to be transferred between the diaphragm and any lateral force resisting elements, such as walls or frames, but that connections are detailed such that they adequately transfer the anticipated loads.

The comments in this section relate to ‘Intermediate Moment Resisting Frames’, defined in AS3600 (Reference 8) as ‘moment resisting frames of ductile construction’, complying with the additional requirements of ‘Appendix A’ in AS3600. The intent of these special detailing requirements is to improve the ductility and reduce the vulnerability of concrete structures in a manner consistent with the relatively low seismic hazard in Australia. The detailing requirements shown in Figure 20 are therefore not onerous and relate to steel reinforcement continuity, anchorage and lapping.

## 7.4 Slab and band beam systems

In high seismic regions building codes (ACI and New Zealand Standard) tend to discourage wide shallow beams by imposing limitations on the maximum beam width. Also, 75% of the longitudinal beam bars are required to be within the column width. Since the main difficulties with wide beams is placing all the required joint ties, Irvine and Hutchinson (Reference 5) recommend that the steel reinforcement ratio ($A^{st}$/bd) be restricted to 0.02 or less, so as to reduce this problem. The designer should ensure that the column has sufficient ductility to prevent a column side sway failure (soft storey collapse).

The above requirements apply to high seismic regions. The University of Melbourne has conducted research to investigate the behaviour of wide band beams in low seismic regions. At this stage the current requirements of AS3600 (Reference 8) can be used, see Figure 21.




**TYPICAL BEAM REINFORCEMENT DETAILS FOR EARTHQUAKE LOADING (Refer AS3600, Appendix A)**

Figure 21: Detailing requirements for earthquake loading

# 8.0 Manufacture and Installation

## 8.1 Manufacture

Figure 22: Concrete is discharged at a controlled rate by an electrically operated concrete spreader

The manufacture of HumeSlab™ panels takes place in a factory environment where a system of controls and checks ensures optimum product quality.

Panels are cast on steel forms using high strength concrete, externally vibrated, to ensure thorough compaction and uniform density.

After an initial curing period of approximately 12 hours the panels are stripped, stacked and stored ready for delivery.

## 8.2 Delivery

Figure 23: Lifting of HumeSlab panels from casting table

Panels are stacked and transported by semi-trailers in approximately 150 $m^2$ loads. Stacking bearers should be provided at approximately 1.5 m centres to minimise stresses during transport.

The laying sequence should be pre-determined and communicated to the HumeSlab™ supplier prior to manufacture. This will enable stacks to be stored and then loaded in reverse order of placement so that the top panel on the stack is the first to be placed on site. The only exception being in the case of a load of mixed panel sizes when small panels are loaded on the top of the stack irrespective of the placing sequence. The erector should be prepared to site stack units delivered out of sequence due to loading requirements. However, such panels may be placed directly in position if their locations can be accurately fixed prior to commencement of panel placing.

## 8.3 Installation

Figure 24: Panels in storage ready for delivery to site

Where the HumeSlab™ panels are not designed to sustain construction loads over the clear span without intermediate supports, a simple system of frames and props with timber headers is normally erected prior to arrival of panels on site (see Figure 16).

Prop spacing should be specified by the design engineer and will vary according to the type and number of trusses in the HumeSlab™ panels and the construction loads to be supported. Tables 4 and 5 can be used to determine the required prop spacing or alternatively contact Humes for more information.

## 8.4 Lifting and placing

Figure 25: Site lifting - typical 4-point lift

Panels 55 mm thick have a typical weight of 145 to 160 kg/m $^2$. In cases where the spreader is used for lifting, the weight of the spreader (about 500 kg) must be added to the panel weight to determine the maximum load for lifting.
It is important to ensure that the crane selected has adequate capacity at the reach required to place all panels. If crane capacity is limited, it may be necessary to limit the size of panels to ensure that the load/reach capacity of the crane is not exceeded.

During production each panel is marked with an identification number corresponding to the panel numbers on the layout drawing. This ensures that panels are placed in the correct position in the structure.

Panels up to 8.5 metres in length can be lifted by crane using four chains*. The chain hooks must be attached to the top chord of the trusses as shown in Figure 26. The lifting capacity has been verified by testing for this method. Panels between 8.5 and 10 m long may require a lifting frame. Lifting point locations should be marked on shop drawings.

Placing rates of up to 10 panels per hour can be achieved with a crew of two men on the deck, crane driver and dogman. Where panels of 6 m length or greater are supplied, the placing rate can be approximately 150 m $^2$ per hour.

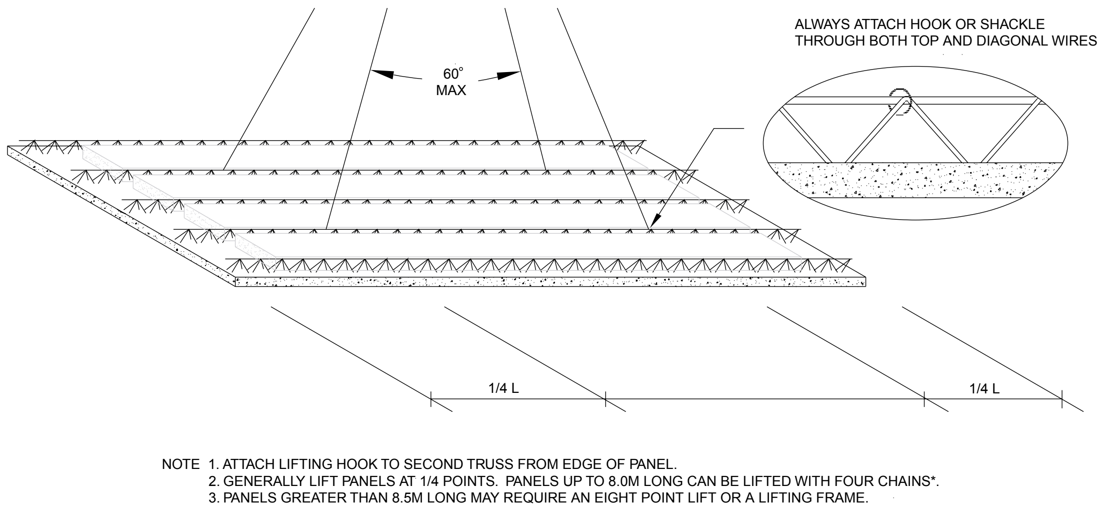



Figure 26: Lifting of HumeSlab™ Panels

All bearing surfaces for HumeSlab™ panels should be level to ensure alignment between units and to minimise twisting of panels. Where panels are to sit on block work or precast walls, the bearing surfaces may require levelling with concrete mortar. An alternative is to provide temporary, carefully aligned props immediately adjacent to the walls.

* This applies to standard panels only. For non-symmetrical panels, panels with block-outs, voids or other non-standard inclusions, contact Humes.

**8.5 Services and edge forms**

Figure 27: Temporary props are positioned prior to placing panels

Electrical junction boxes, fire collars for plumbing, ferrules etc, can be cast into the panels as detailed on architectural drawings  These items need to be supplied by the contractor.  A hot wire cutter is used to quickly cut polystyrene void formers to accommodate conduits. Generally penetrations are formed by using polystyrene edge forms in the factory. Smaller penetrations can be accommodated by casting in polystyrene blockouts or alternatively they can be core drilled onsite.

Fixing of edge forms can usually proceed while services are being installed. A turnbuckle engaging truss wires can be used as a connection device for edge forms. Appendix A, Figure A6, includes edge form details.

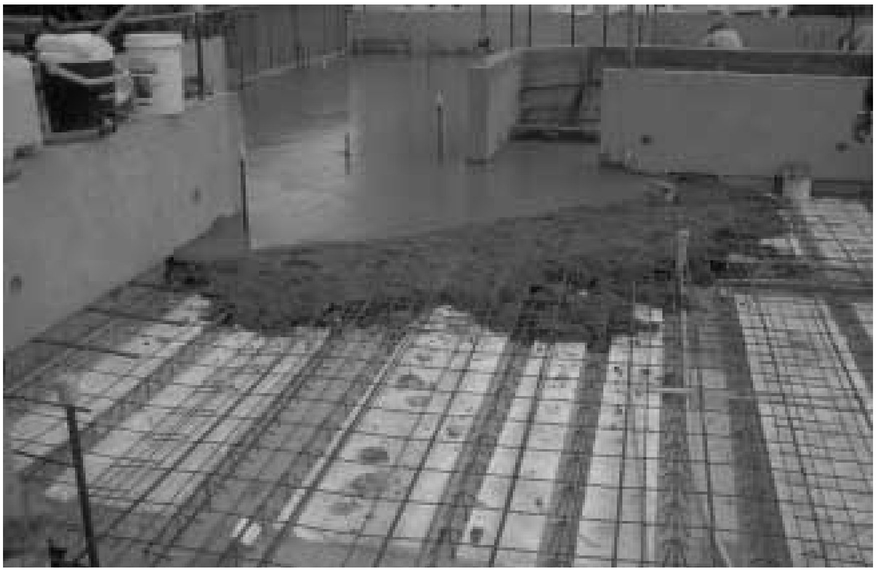


Figure 28: In-situ concrete is reduced by use of polystyrene void formers

**8.6 Top reinforcement and in-situ concrete**

Immediately following installation of services and edge forms, fixing of top reinforcement steel is carried out and the slab is then ready for pouring the top layer of concrete.

The thickness of topping concrete above polystyrene generally should not be less that 65 mm. Additional top steel reinforcement, fabric (mesh) laps, fabric (mesh) wire diameter and other factors may require this topping thickness to be increased to ensure that steel reinforcement is fully embedded and adequate cover is provided. This aspect should be considered at the design stage (refer to section 5.0, ‘design principles’).

**8.7 Ceiling finish**

HumeSlab™ panels are manufactured in rigid steel beds and the soffit finish achieved is ‘Class 2’ as described in AS3600. The joint between panels, if left unfilled, is referred to as a shadow joint, in that a light and shade effect is created between the two prefabricated units. This type of ceiling finish requires no treatment and is quite acceptable as an off-form grey concrete finish. In fact, the surface finish achieved is quite superior to that achieved with conventional forming products.

Figure 29: Unpainted soffit finish showing shadow joint

In situations where the slab soffit is to be used as an exposed ceiling and a painted surface is required, then the joint can be filled and a textured paint finish applied directly to the panel. If the joints are subject to differential movement then the use of a cement based repair mortar with high bond strength should be used to fill the joint. If no differential movement were expected then the use of a plaster-based material would be acceptable. A flat paint finish is possible after a skim coat of plaster.

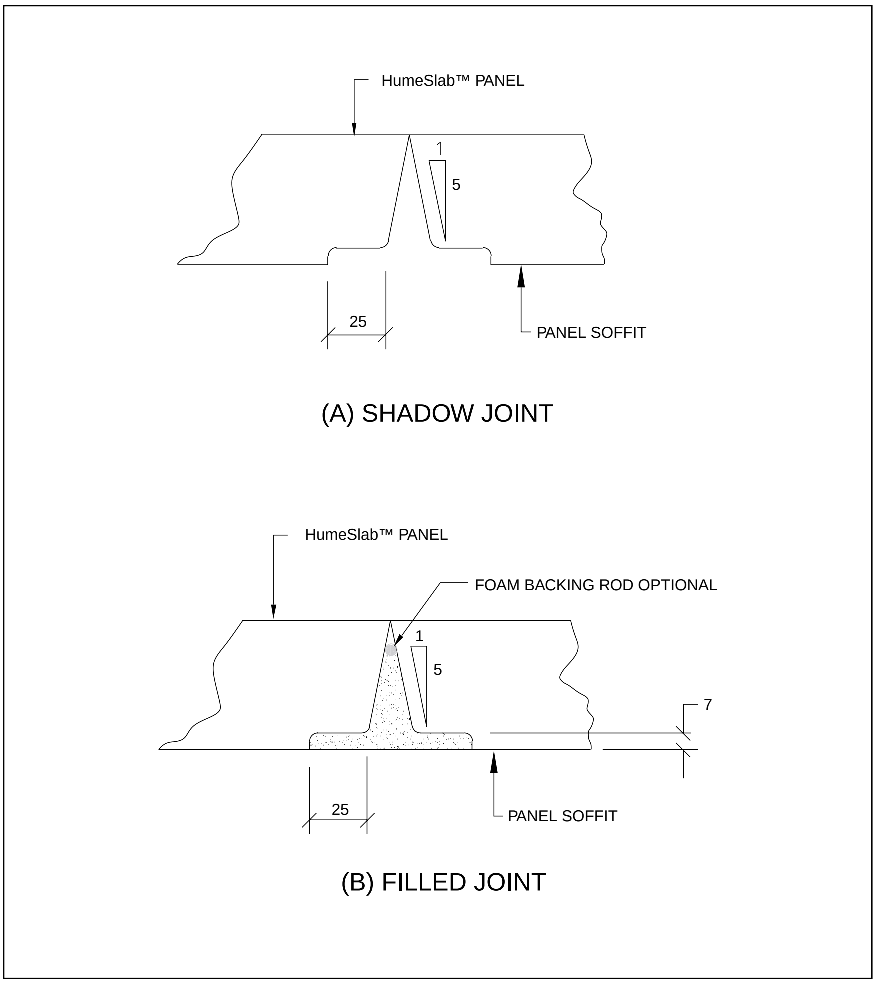



Figure 30: HumeSlab™ panel soffit joint

## 8.8 Construction Pratice

### Delivery

Panels are delivered in stacks on semi-trailers, approximately 150m2 per load. Stacks are normally loaded onto the truck in reverse order of placement so that the top panel on the stack is the first to be placed on site. The only exception being in the case of a load of mixed panel sizes when small panels are loaded on the top of the stack irrespective of placing sequence. This should be the only circumstance which requires a panel to be grounded on site before placing. However, such panels may be placed directly in position if their location can be accurately fixed prior to commencement of panel placement.

### Installation

Except in cases where the HumeSlab™ panels are designed to sustain construction load over the clear span without propping, a simple system of frames and props with 150 x 100 timber headers is normally erected prior to arrival of panels on site. Prop spacing should be specified or shown on the engineer's drawing and will vary according to the type and number of trusses in the HumeSlab™ panels and the construction loads to be supported. Prop spacing generally varying from 1.8 to 2.4 metres is typical for slabs.

### Crane Capacity

HumeSlab™ 55mm thick has an average weight of 145kg/m2. In cases where the spreader is used for lifting, the weight of the spreader (500kg) must be added to the panel weight to determine the maximum load for lifting. It is important to ensure that the crane selected has adequate capacity at the reach required to place all panels.

Alternatively, where crane capacity is limited it may be necessary to limit the size and weight of panels to ensure that the load/reach capacity of the crane is not exceeded.

### Lifting and Placing

During production, each panel is marked with an Identification number corresponding to the panel layout drawing so that the placement of each panel in its correct position in the structure is simplified.

Most panels up to about 8 metres in length containing truss types T110 or T150 can be lifted by crane using four chains, the hooks being attached to the top bars of the HumeSlab™ trusses. See Fig 26, page 19 for correct hook placement.

In windy conditions it may be preferable to lift long panels using a 16 hook spreader.

For lifting and placing panels a crew of two men on the deck should achieve a placing rate of approximately 10 panels per hour.

### Services

After a reasonable area of floor has been covered with panels a stable deck is available for following trades to commence work. Conduits for electrical and communications services and water reticulation pipes are installed as for in-situ concrete slabs. A hot wire cutter is used to quickly chase into polystyrene void formers to accommodate conduits.

Most penetrations can be accommodated during the panel design. However, small penetrations such as those required for waste pipes and electrical outlets can be made by core drilling through the 55mm HumeSlab™.

### Cracking of Panel

The HumeSlab™ panel may exhibit cracking for a number of reasons, eg.

- Incorrect loading of stacking on site.
- Poor handling techniques.
- Inadequate propping.

Minor cracking will not affect the structural Integrity of the final slab, however, if more severe cracking (I.e. crack widths greater than 0.2mm) has occurred it should be inspected by a suitably qualified engineer.

### Top Steel and In-Situ Concrete

Immediately following Installation of services, fixing of top steel is carried out and the slab is then ready for pouring of site placed concrete.

The thickness of topping concrete above polystyrene will be shown on drawings, but generally should not be less than 70mm, additional top reinforcement fabric wire diameter and other factors may require this topping thickness to be increased in some cases, to ensure that reinforcement is fully embedded and adequate cover provided.

### Edge Forms

Fixing of edge forms can usually proceed while services are being installed. A turn buckle engaging truss wires may be used as a connection device for edge forms.

# 9.0 HumeSlab™ Bridge Decking

HumeSlab™ is used in composite bridge construction and has been approved by most state road authorities, providing safer and more efficient construction of bridge superstructures. Figure 31 shows a typical bridge deck section constructed with HumeSlab™ panels (70 to 90 mm thick) which, when topped with in-situ concrete, become an integral part of the deck slab. Panels are made with trusses and steel reinforcement uninterrupted but with full-length gaps or continuous concrete block-outs, which coincide with beam locations to accommodate the shear connectors. This allows placement of panels directly over precast concrete or steel beams.

The HumeSlab™ panel can cover the entire width of a bridge, including the cantilever beyond the external beams, thus eliminating the need for formwork and additional scaffolding. This application for HumeSlab™ has been widely accepted and shown to be very cost effective in terms of speed of erection, safety in construction (instant safe working platform), efficient use of materials (no lost formwork) and significantly reduced traffic interference.

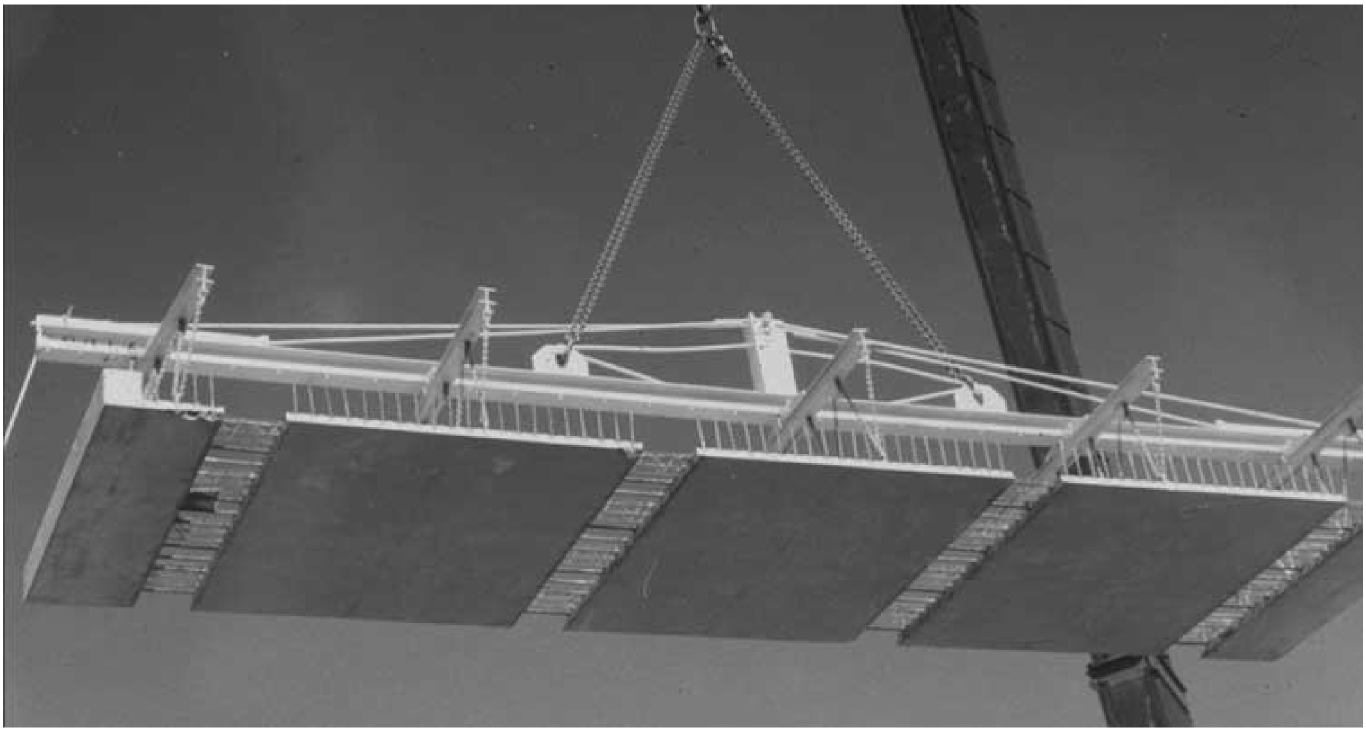


Figure 31: Ready made bridges - HumeSlab™ bridge deck panel being lifted into position

## 9.1 Design details

The design can be carried out assuming full composite action between the precast panel, in situ topping and the supporting beams. The in-situ concrete topping fills the gaps over the beams and ensures an effective connection with ligatures on precast beams or shear studs on steel beams. During construction HumeSlab™ trusses provide the cantilever strength and negative moment strength over the beams.

The slab reinforcing steel can be designed in accordance with the Austroads Bridge Design Code and, in view of the discontinuity at panel joints, the slab could be considered as spanning one way transversely over the beams. However, research carried out by Buth et al (Reference 6), for similar precast systems, has demonstrated that using this approach is conservative and that the joints can be disregarded.

## 9.2 Load distribution - panel to panel connection

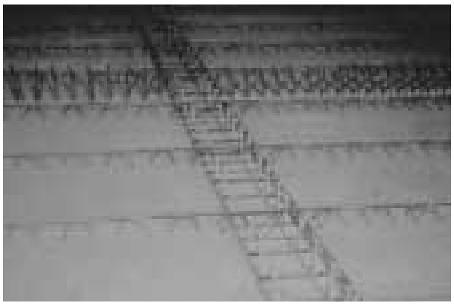


Figure 27: Temporary props are positioned prior to placing panels

A commonly debated topic of past and current research has been the ability of similar deck systems to distribute wheel loads in the longitudinal direction and the corresponding effect of the joints between adjacent panels.

Continuity at the joint is provided by the in-situ portion of the deck and research results indicate that the presence of the joint is not detrimental to the load distribution performance of the bridge deck system (Reference 6,9 and 10).

Test results on two systems of longitudinal reinforcement:

1. longitudinal reinforcement placed directly on top of panels, and
2. splice bars, on top of panels and across joints,

in addition to the normal longitudinal steel reinforcement indicated the in-situ concrete topping successfully transferred wheel loads across joints. The supplementary joint reinforcing steel did not improve the performance and in all tests with wheel loads near the panel joint, the mode of failure was punching shear (Reference 6). Even at failure loads there were no tensile cracking observed at the bottom of the in-situ topping directly over the panel joints.

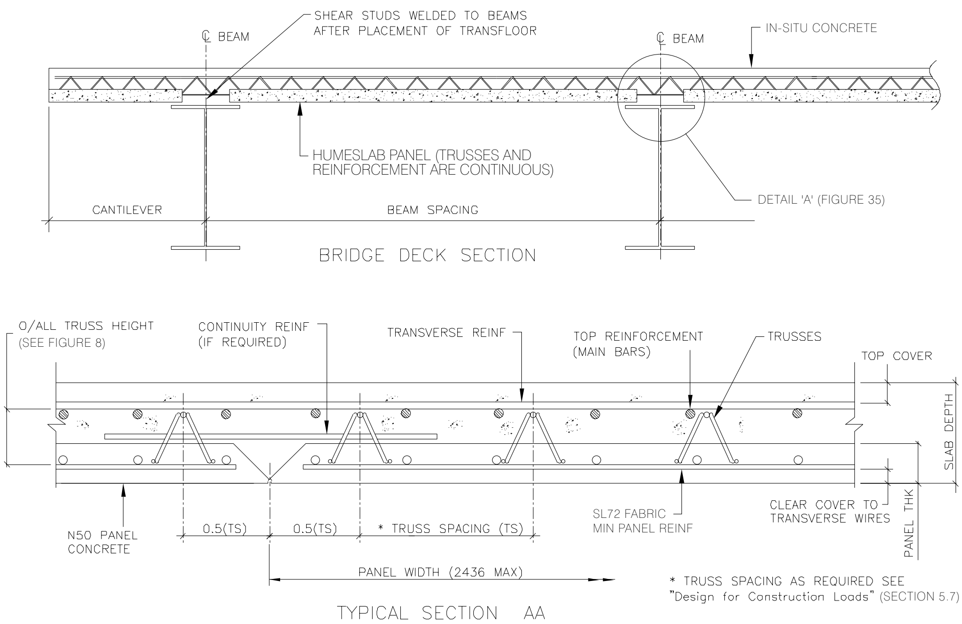



Figure 33: Typical bridge deck details

Figure 34: Thomson River Bridge - Victoria

Further research and testing (Reference 10) has indicated a tendency for shrinkage and thermal cracks to form directly over the panel joints but these cracks do not adversely affect the ability of the deck slab to transfer wheel loads across joints. Since these cracks extend down approximately half way through the topping slab, it was concluded that the distribution reinforcement performs better when placed toward the top to control shrinkage and thermal cracking than when placed at the bottom of the topping slab in an attempt to control flexural cracking.

American studies on in-service bridges (Reference 10) have indicated that a level of transverse reinforcement (reinforcement in the same direction as beams) equivalent to 230 $mm^2/m$ is satisfactory. AASHTO has adopted 230 $mm^2/m$ as the minimum transverse steel reinforcement in deck panels of similar decking systems. The level of this steel reinforcing content should be left to the discretion of the design Engineer. However, it should be noted that projects in Australia have been completed with the steel reinforcement content between 230 and 985 $mm^2/m$.

### 9.3 Bearing of bridge deck panels

Composite bridge deck panels must be supported on the bridge girders by a permanent bearing material providing continuous and solid support. The permanent bearing material should consist of mortar, grout, concrete or steel. Use of soft fibrous material may lead to the bridge deck acting as simple spans over girders rather than continuous spans and delamination at the ends of the precast panels may occur.

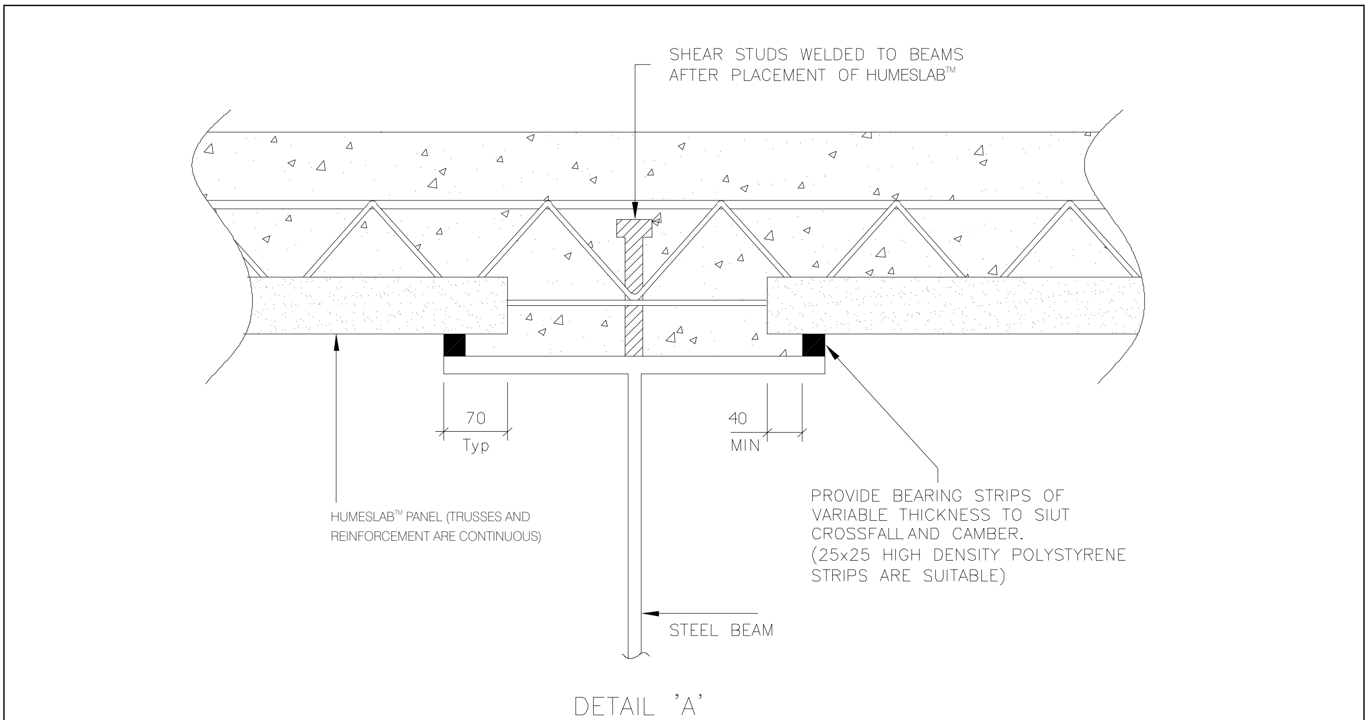



Figure 35: Temporary bearing detail for bridge deck panels

Figure 36: Cantilever portion of bridge deck is unpropped during construction

If grout or concrete is used as the permanent bearing, a temporary bearing system must be used to support the panels during construction. Temporary bearing systems designed to remain in place include continuous strips of compressible material such as high density polystyrene and bituminous fibreboard. Rigid material such as hard plastic shims, which are left in place, will continue to provide the primary support for deck panels should the permanent grout or concrete shrink. This may result in undesirable cracking over these rigid bearing points.

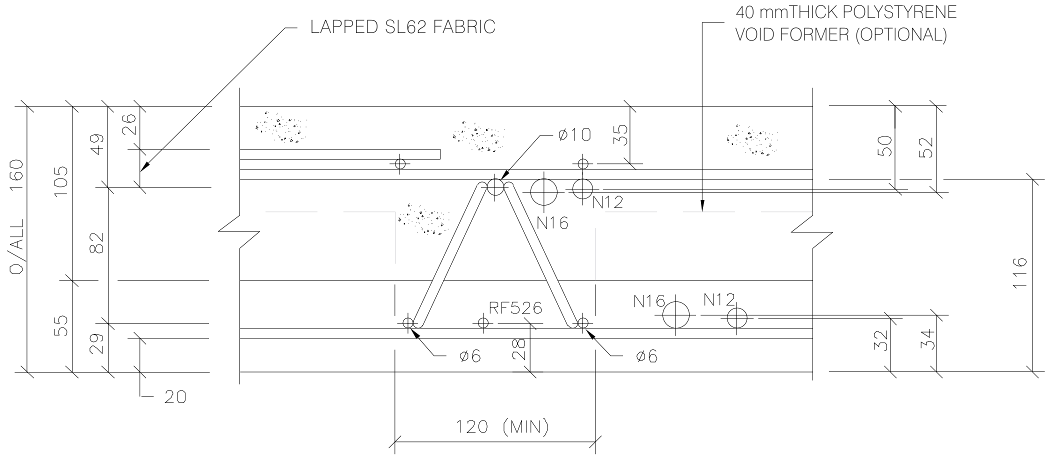
### 9.4 Construction practice for bridge decks

1. Temporary bearing materials, which are designed to remain in place, must be compressible.
2. The height of the temporary bearing strip must be adequate to allow grout or concrete to flow under the panel.
3. Deck panels should extend a minimum of 40 mm beyond the temporary bearing material.
4. Venting is required when grout or concrete is used. This can be accomplished by leaving small gaps in the bearing strips at approximately 1200 mm intervals.
5. The top concrete should first be placed in continuous strips over girders and allowed to flow under panels before being placed on the remaining deck. This procedure improves the flow of concrete under panel ends, helps eliminate air pockets and places concrete under panel ends before the temporary bearing strips are compressed due to the weight of wet concrete.

Figure 37: Placing bridge deck panels

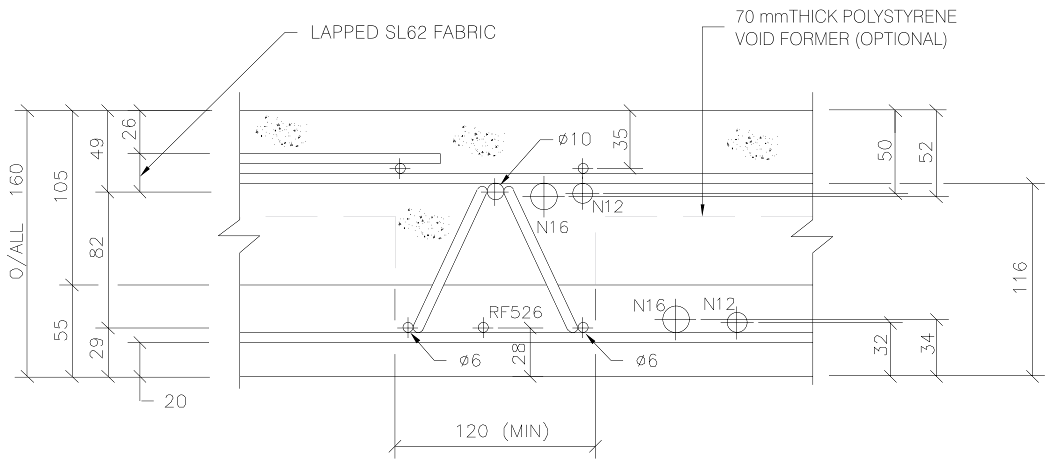
# 10.0 References


1. Hartmut Koblenz, “Precast Concrete Floors, Part 2” Betonwerk Fertigteil - Tecknik, (BFT), Bauverlag GmbH, Concrete Precasting Plant and Technology, issue 6/1994.
2. J. Glynn, “Test of HumeSlab™ Precast Floor Units”, Glynn Tucker and Associates, University of Queensland, Report No. 7650, 1981.
3. Sanders P.T. (et al), “Seismic Behaviour of load Bearing Precast Construction in Australia”, Steel Reinforcement Institute of Australia, 1995.
4. Clough D.P., “Considerations in the Design of Precast Concrete for Earthquake loads”, Journal of Prestressed Concrete Institute, Vol. 27, No. 2. pp 78-107.
5. Irvine H.M. and Hutchinson G.L., “Australian Earthquake Engineering Manual” 3rd Edition, Techbooks, 1993.
6. Buth, Eugene, Furr H.L., and Jones H.L., “Evaluation of a Prestressed Panel, Cast in Place Concrete Bridge”, Research Report 145-3, Texas Transportation Institute.
7. Furr H.L. and Ingram L.L., “Cyclic Load Tests of Composite Prestressed-Reinforced Concrete Panels”, Research Report 145-4F, Texas Transportation Institute.
8. Standards Australia, “AS3600 - 2001 Concrete Structures”.
9. Kluge, Ralph W. and Sawyer H.A., “Interacting Pretensioned Concrete Form Panels for Bridge Decks”, PCI Journal, Vol. 20, No. 3.
10. Jones H.L. and Furr H.L., “Study of In Service Bridges Constructed with Prestressed Panel Sub-decks”, Research Report 145-1, Texas Transportation Institute.



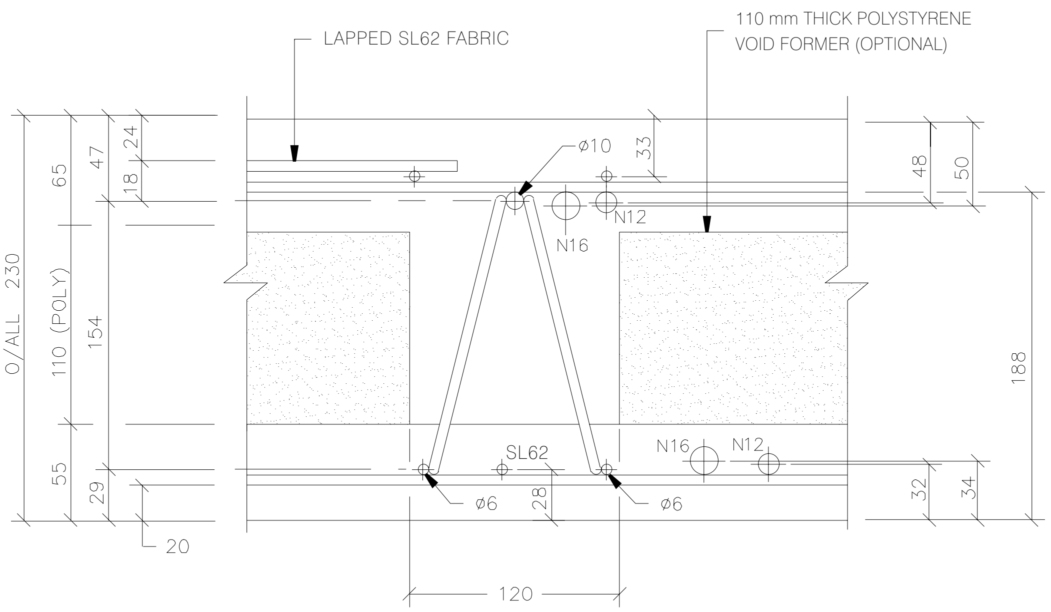

TYPICAL SLAB SECTION T80/10 HUMESLAB™ TRUSS
(All dimensions in millimetres (mm))

Figure A1: Typical, but not restricted to, reinforcement arrangements in slabs



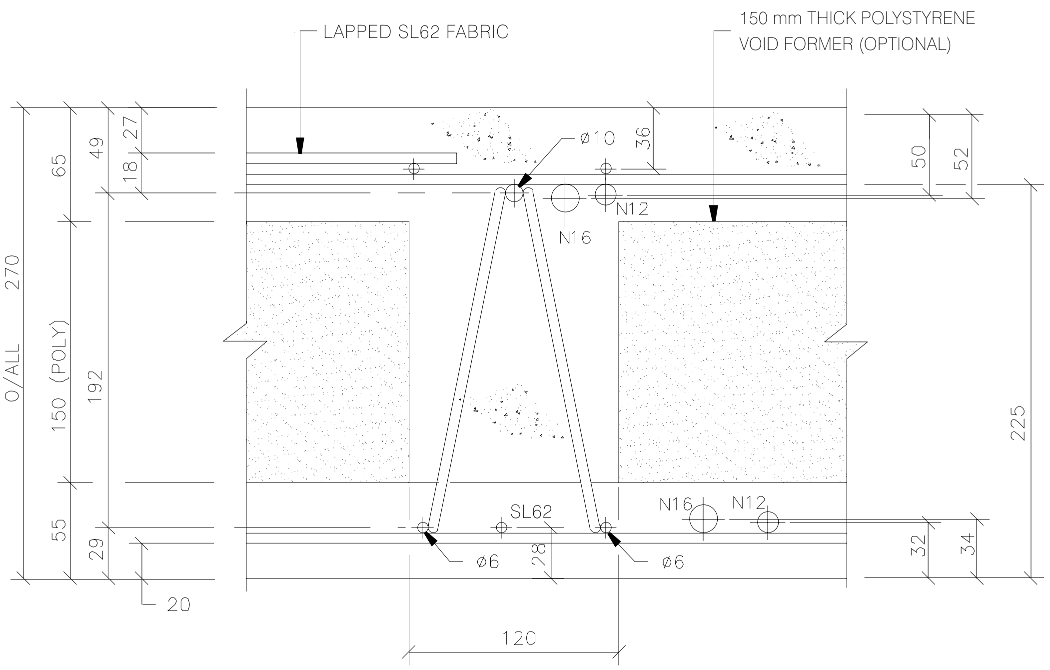

TYPICAL SLAB SECTION T110/10 HUMESLAB™ TRUSS
(All dimensions in millimetres (mm))

Figure A2: Typical, but not restricted to, reinforcement arrangements in slabs




TYPICAL SLAB SECTION T150/10 HUMESLAB™ TRUSS
(All dimensions in millimetres (mm))

Figure A3: Typical, but not restricted to, reinforcement arrangements in slabs




TYPICAL SLAB SECTION T190/10 HUMESLAB™ TRUSS
(All dimensions in millimetres (mm))

Figure A4: Typical, but not restricted to, reinforcement arrangements in slabs

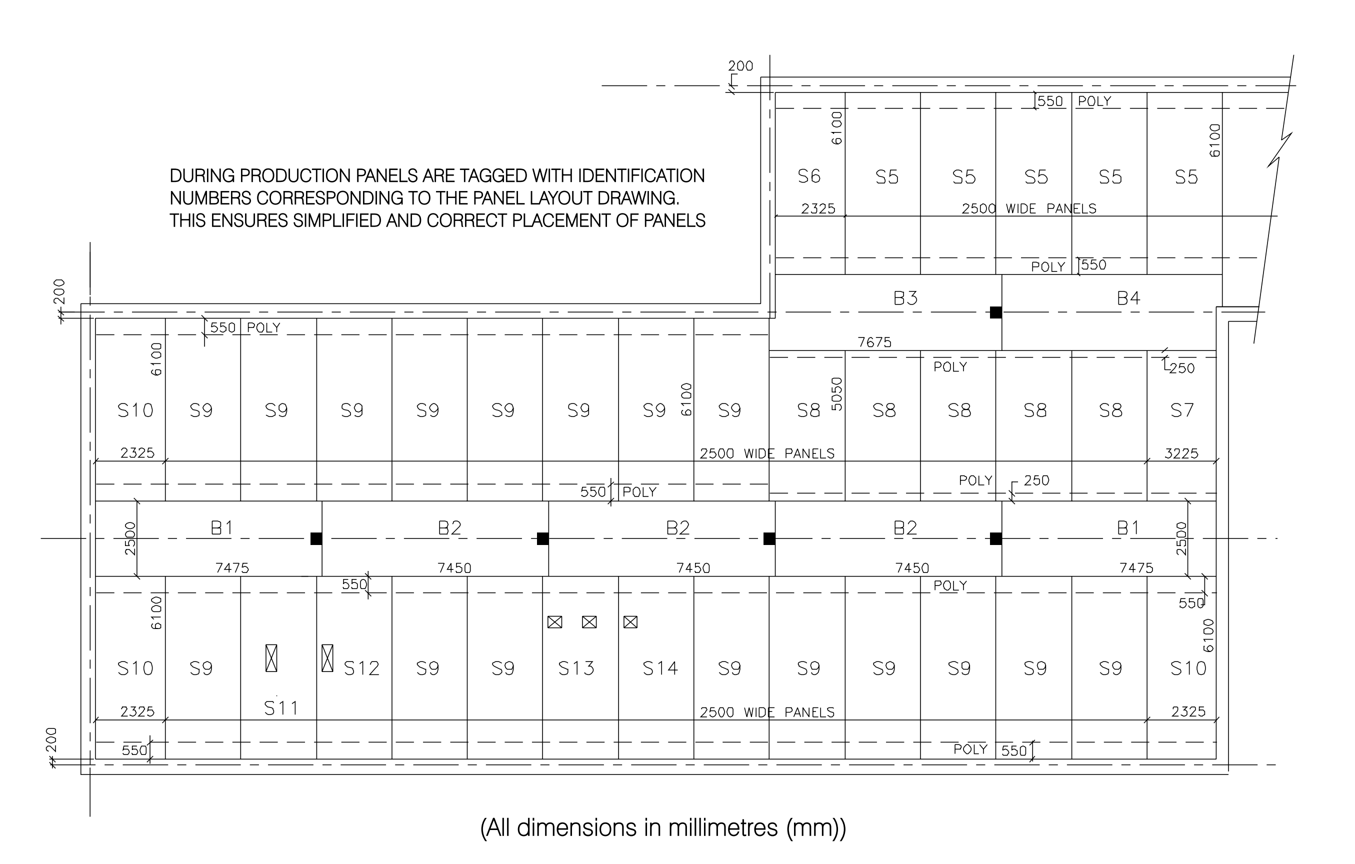



Figure A5: Typical panel layout diagram

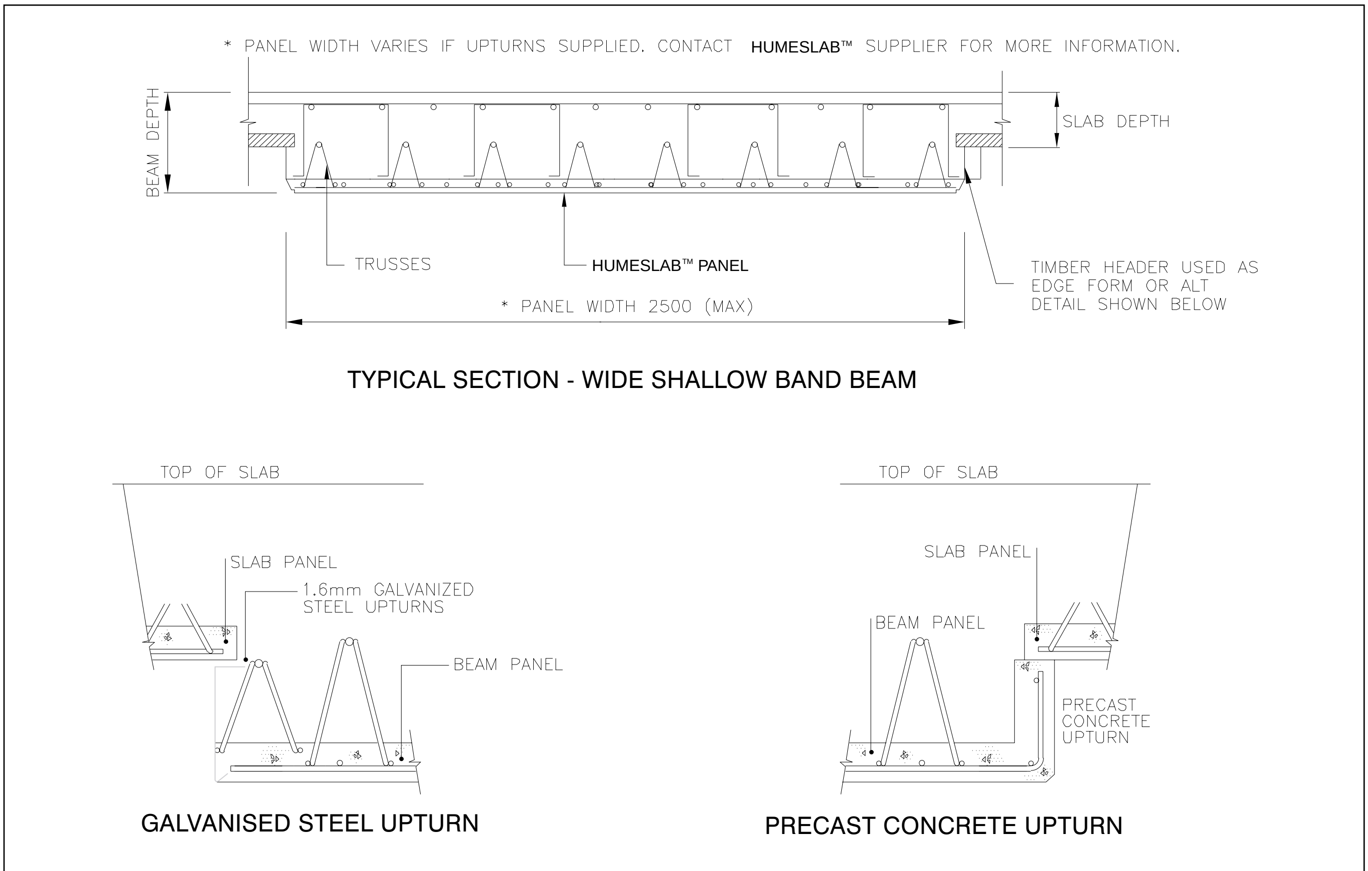



Figure A6: Band beam edge form options

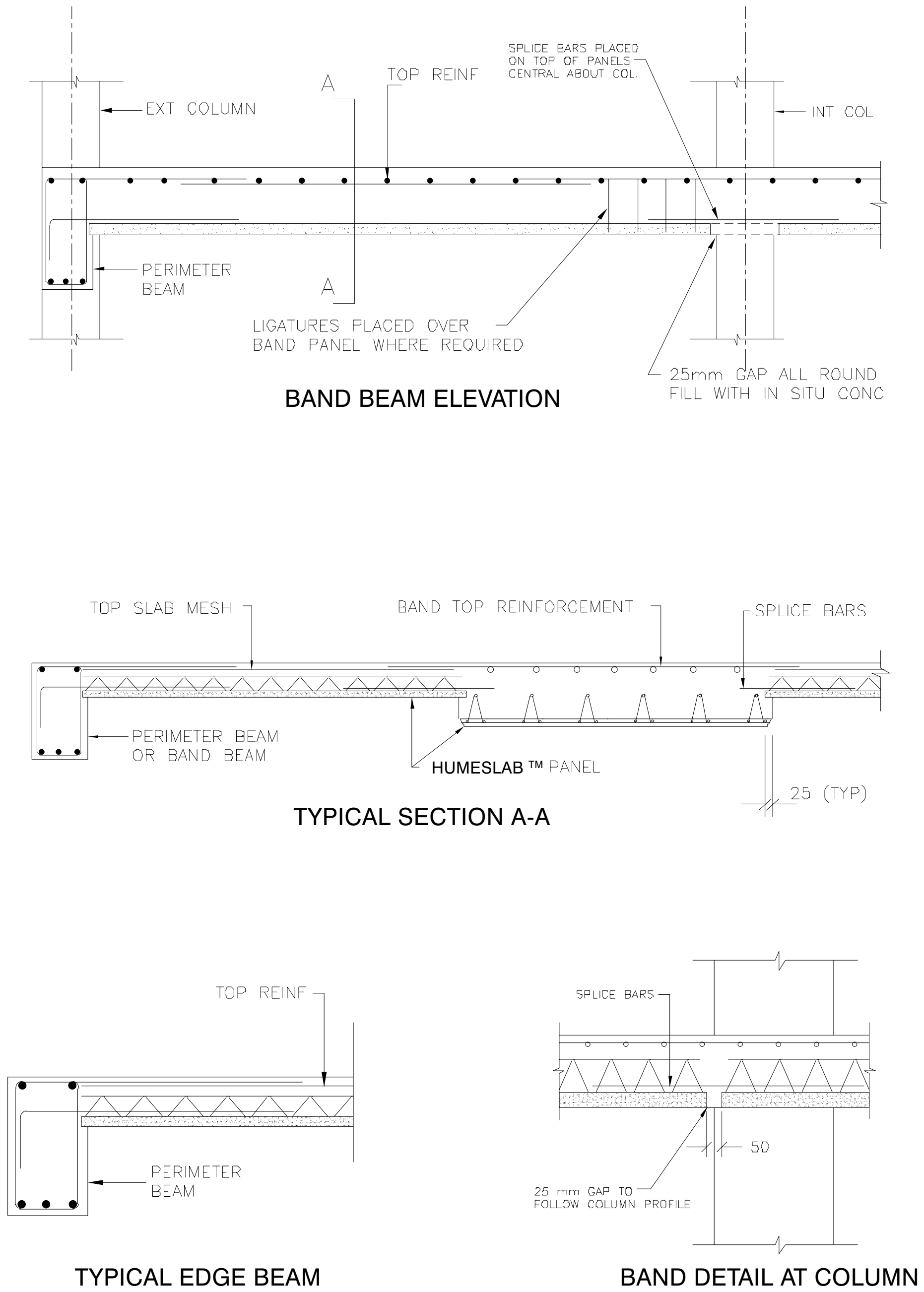
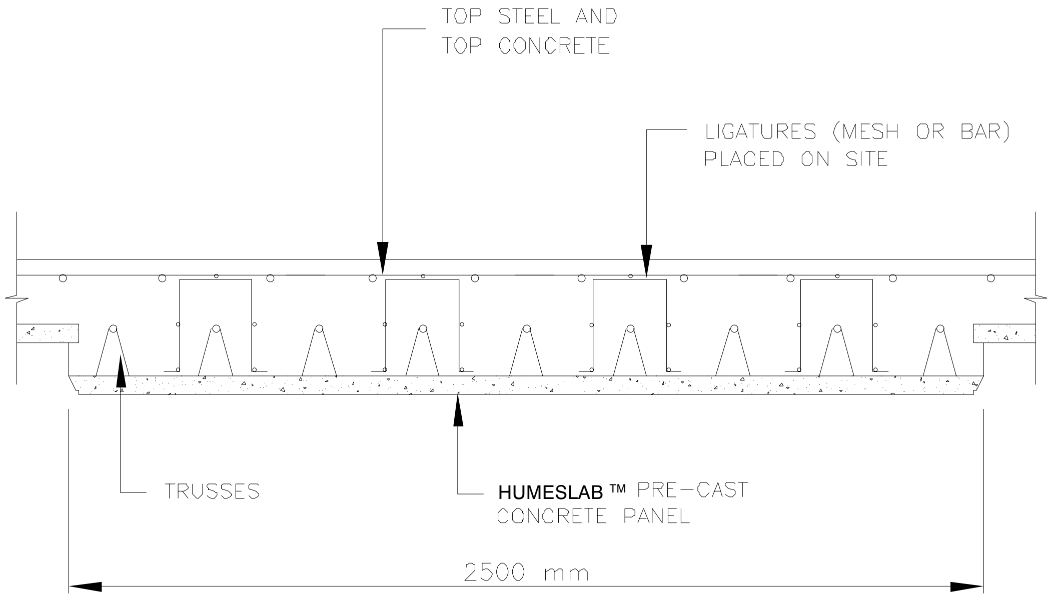
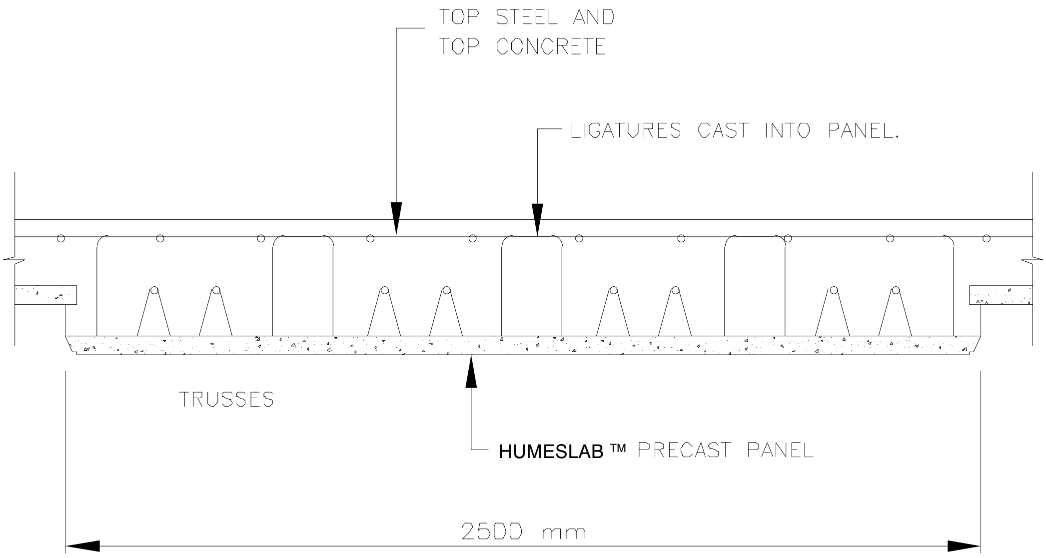



Figure A7: Typical slab and band beam sections

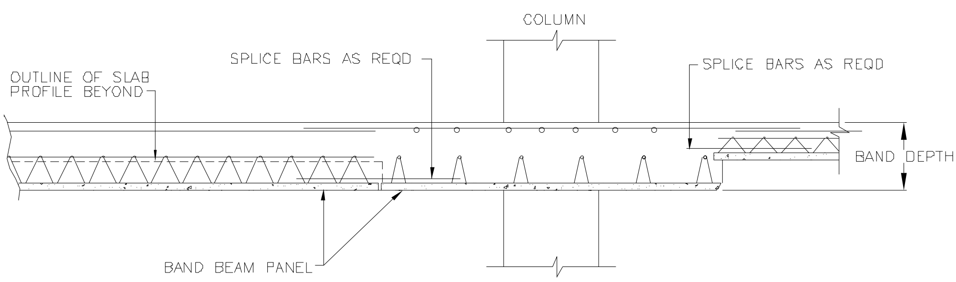



(a) USE ONLY FOR NOMINAL SHEAR REINFORCEMENT
(NO EXCESS SHEAR)

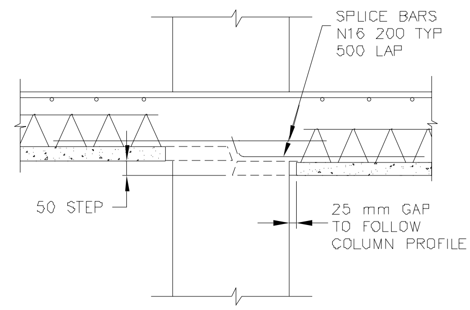



(b) USE WHEN SHEAR REINFORCEMENT IS REQUIRED
TO RESIST EXCESS SHEAR

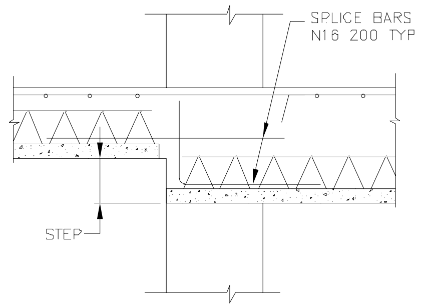
Figure A8: Band beam with ligatures



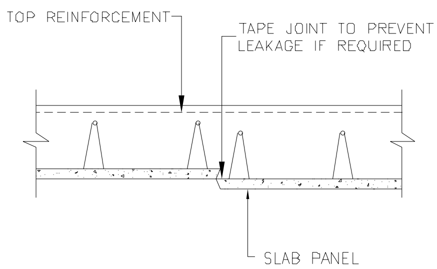

SECTION ALONG SLAB BAND WHERE BANDS INTERSECT



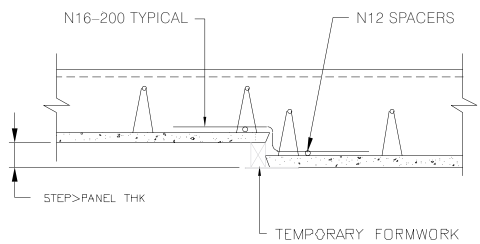

BAND BEAM TO COLUMN DETAIL
50 mm STEP IN SOFFIT




BAND BEAM TO COLUMN DETAIL
LARGE STEP IN SOFFIT

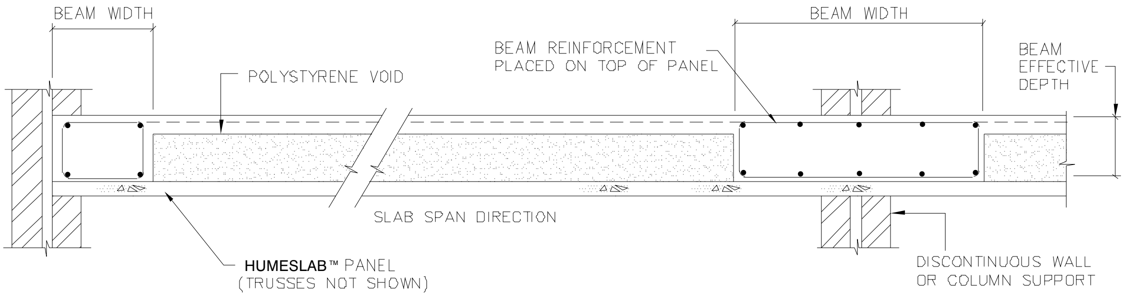



SLAB CONNECTION (SIDE TO SIDE)
WITH 50 mm STEP IN SOFFIT

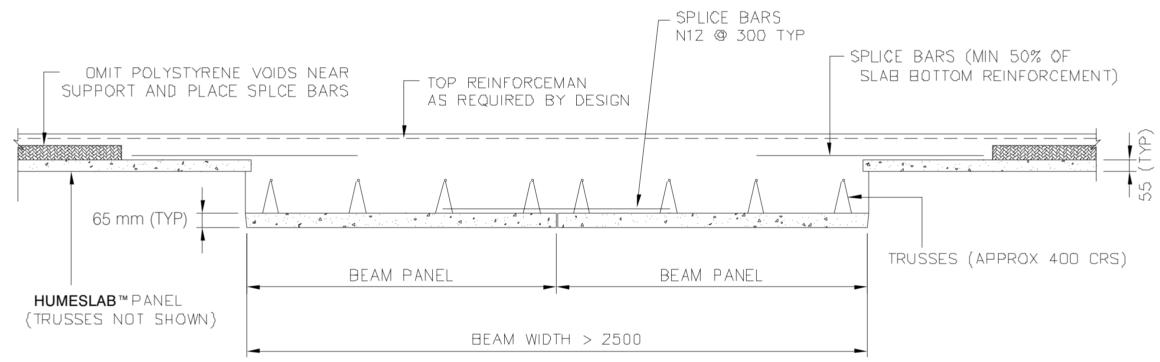



SLAB CONNECTION (SIDE TO SIDE)
WITH 100 mm STEP IN SOFFIT

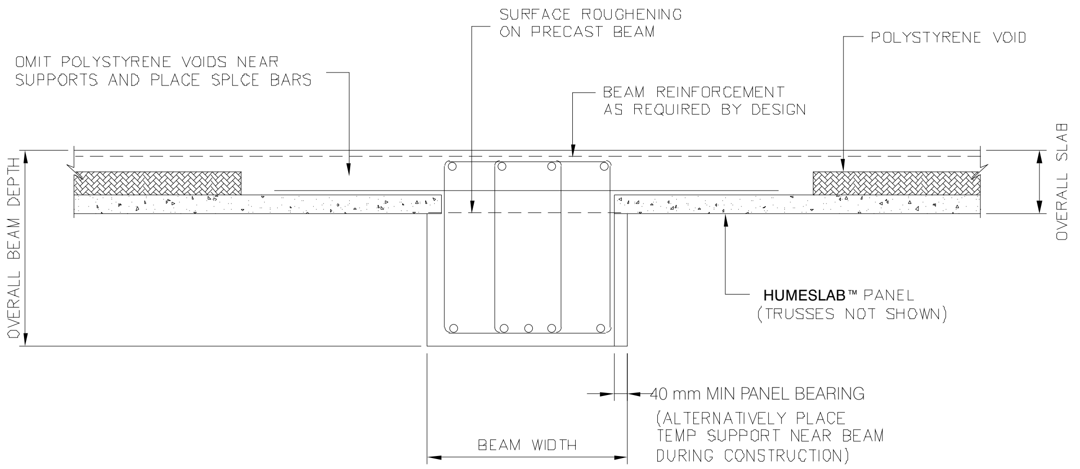
Figure A9: Steps in band beam soffit




BEAMS WITHIN SLAB THICKNESS

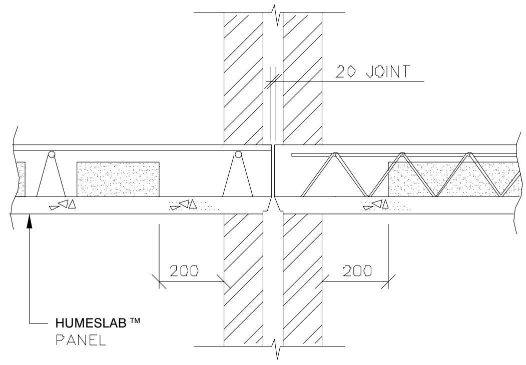



BAND BEAM SECTION WHERE BEAM WIDTH EXCEEDS 2500 mm

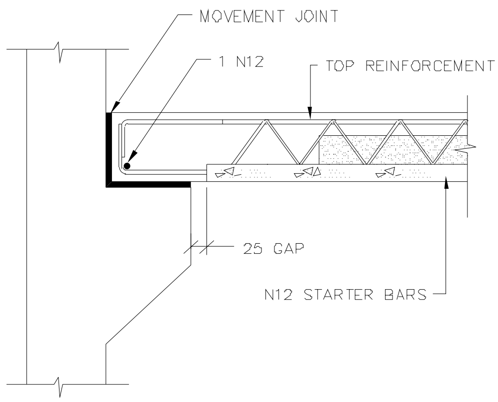



SLAB TO PRECAST BEAM

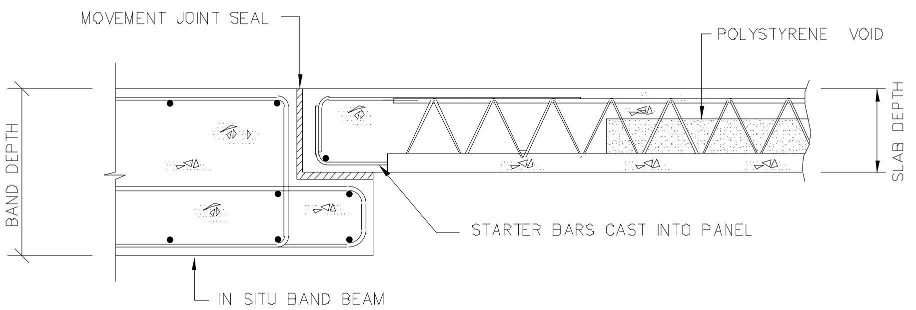
Figure A10: Typical beam details



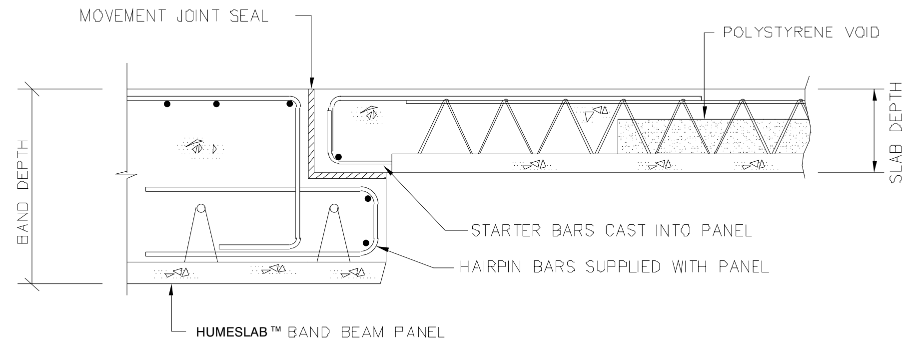

MOVEMENT JOINT AT BLOCK WALL




MOVEMENT JOINT AT CORBEL

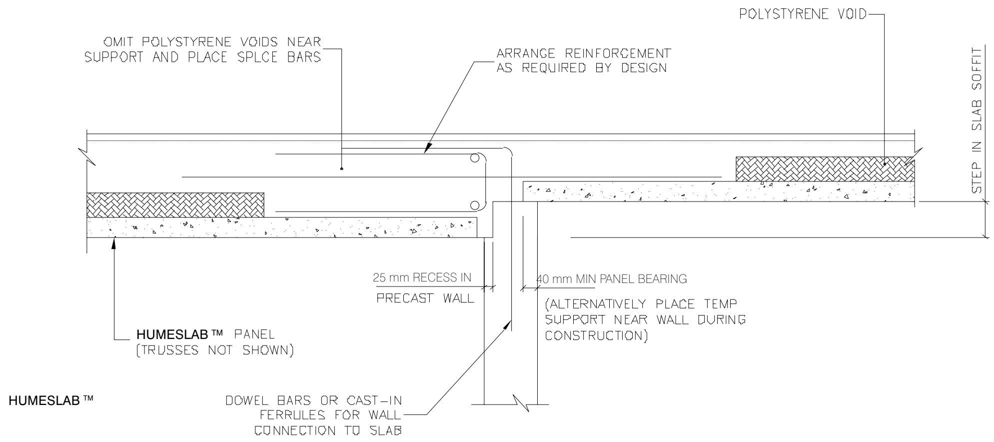



SLAB/BAND EXPANSION JOINT (IN-SITU BEAM)

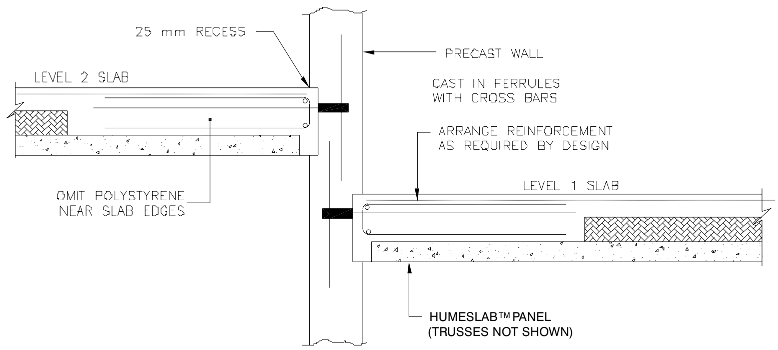



SLAB/BAND EXPANSION JOINT (HUMESLAB™ BEAM)

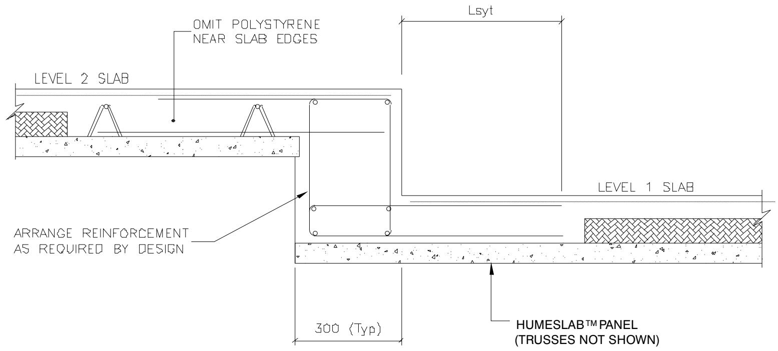
Figure A11: Typical movement joints




SOFFIT STEP AT INTERNAL WALL

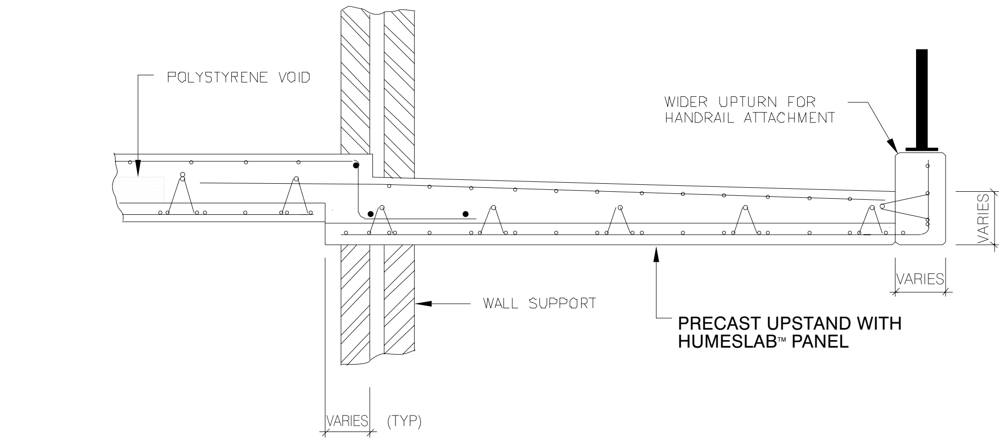



CHANGE IN SLAB LEVEL AT PRECAST WALL

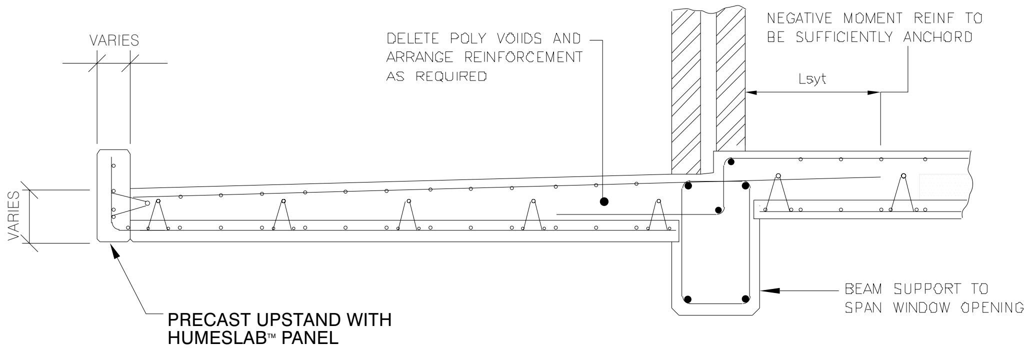



IN-SITU STEP

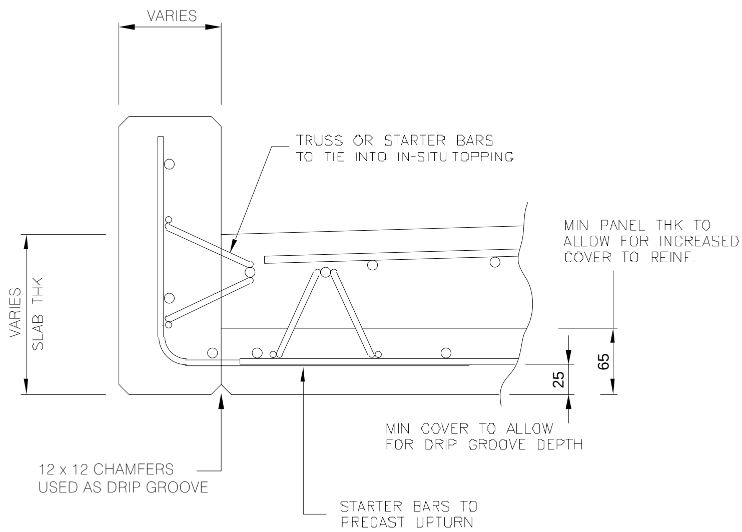
Figure A12: Change in slab soffit level




CANTILEVER SLAB ON WALL SUPPORT

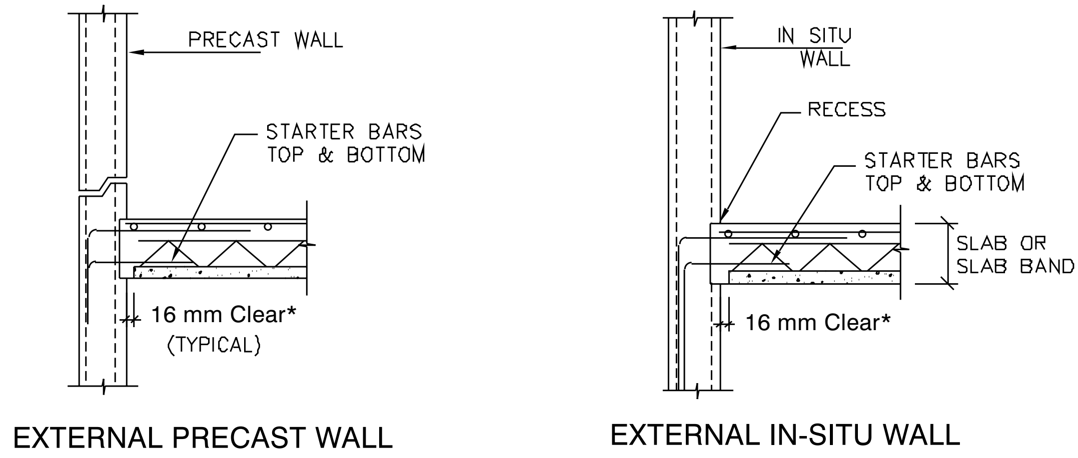



CANTILEVER SLAB ON BEAM SUPPORT

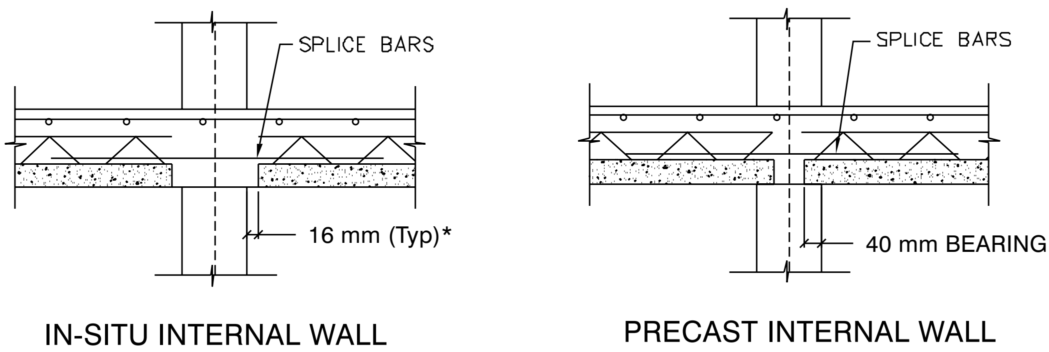


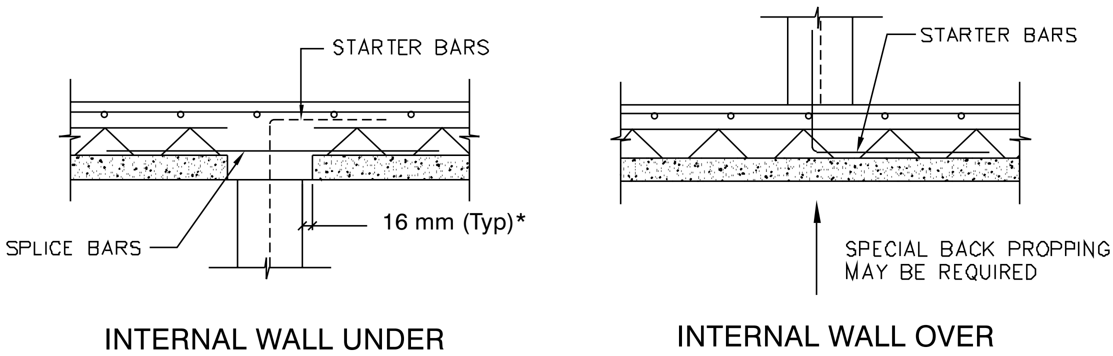

TYPICAL PRECAST UPTURN
(All dimensions in millimetres (mm))

Figure A13: Cantilever and balcony details



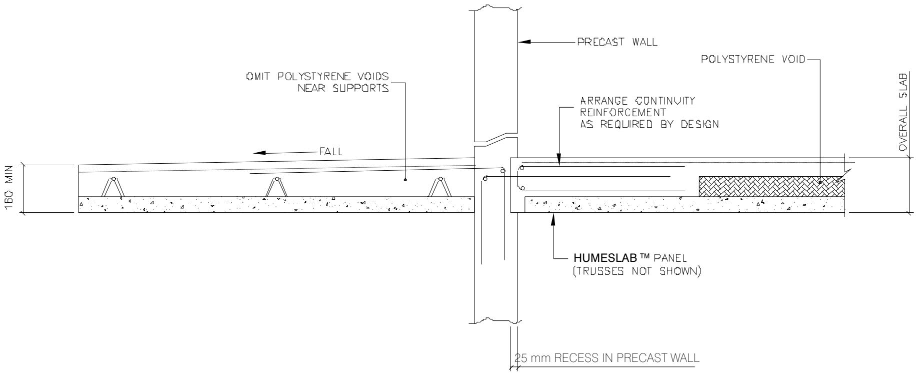

MOMENT CONTINUITY CAN BE MAINTAINED. REINFORCEMENT TO BE CARRIED INTO THE SUPPORT DEPENDS ON THE END RESTRAINT CONDITION. REFER TO AS3600 (9.1.3)



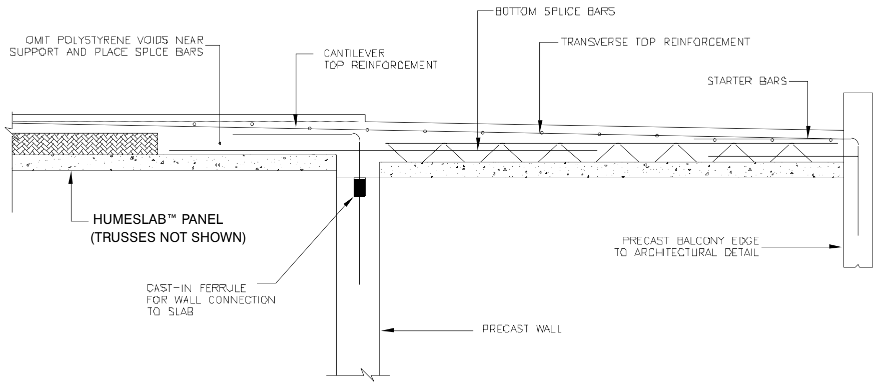




*16mm clear provides tolerance for location of the wall. An alternative detail is to place the panel on the wall with 40mm bearing as shown for “PRECAST INTERNAL WALL”.

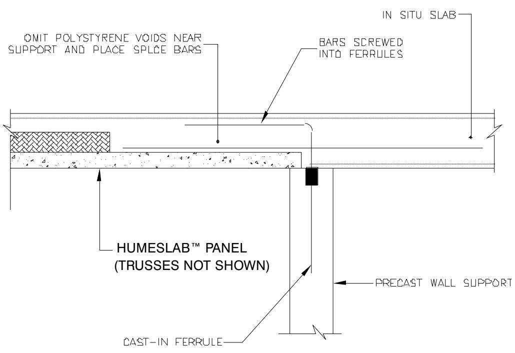
Figure A14: Typical wall connection details



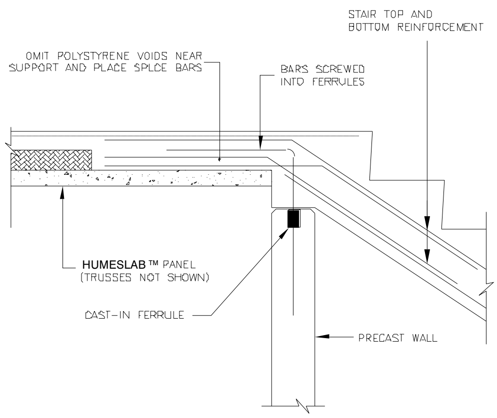

EXTERNAL WALL WITH BALCONY




PRECAST BALCONY EDGE




WALL SUPPORT AT TRANSITION
TO IN-SITU SLAB




HUMESLAB™ CONNECTION
TO IN-SITU STAIRS

# Appendix B

## Estimate Design & Detailed Design

### B1 - EXAMPLE CALCULATION - ESTIMATE DESIGN

| | | | |
|---|---|---|---|
| Effective span (continuous slab) | 6000 | mm | |
| Superimposed dead load | 0.5 | kPa | |
| Live Load | 2 | kPa | |
| Deflection limit | L/250 | | |
| Cover to reinforcement | 20 | mm | |
| | | | |
| From Table 7: | | | |
| Overall slab thickness | 190 | mm | |
| Panel thickness | 55 | mm | |
| Polystyrene void former thickness | 65 | mm | (from Appendix A) |
| Slab self weight | 3.6 | kPa | |
| Average bottom reinforcement | 5.0 | $kg/m^2$ | |
| Average top reinforcement | 4.9 | $kg/m^2$ | |
| Average in-situ concrete | 0.090 | $m3/m^2$ | |

### B2 - EXAMPLE CALCULATION - DETAILED DESIGN

| | | | |
|---|---|---|---|
| Effective span (continuous slab) | 6000 | mm | |
| Overall slab thickness | 190 | mm | |
| Cover to reinforcement | 20 | mm | |
| In-situ concrete strength | 32 | MPa | |
| | | | |
| Panel length | 6000 | mm | |
| Panel Width | 2500 | mm | |
| Panel Thickness | 55 | mm | |
| | | | |
| Polystyrene voids length | 5525 | mm | |
| Polystyrene voids width | 440 | mm | |
| Polystyrene voids thickness | 65 | mm | |
| Number of Polystyrene voids | 4 | | |
| Polystyrene volume | 0.0421 | $m^3/m^2$ | |
| | | | |
| Slab self weight | 3.62 | kPa | |
| Superimposed dead load | 0.5 | kPa | |
| Live load | 2.0 | kPa | |
| Ultimate load | 8.15 | kPa | 1.25DL + 1.5LL |

# Appendix B

| | | | | |
|---|---|---|---|---|
| **DEFLECTION CRITERIA** | | | | |
| Short term factor Xs | 0.7 | | | |
| Long term factor X1 | 0.4 | | | |
| Deflection limit | 0.004 | | span/250 | |
| Span/depth Lef/d | 39.2 | | | |
| | | | | |
| **SLAB DESIGN** | | | | |
| MINIMUM THICKNESS | | | | |
| $L_n/D_s$ | 31.58 | | | |
| $70(1/wK)^{1/3}$ | 32.06 | | $L_n/D_S < 70(1/wK)^{1/3}$ | |
| | | | | |
| BOTTOM REINFORCEMENT | | | | |
| BENDING | | | | |
| Ultimate design bending moment | 26.68 | kNm | $wl^2/11$ | |
| Effective depth d | 158 | mm | | |
| $F_{sy}$ | 500 | MPa | | |
| $f_c'$ | 32 | MPa | | AS3600-2001 Cl 8.1.2.2 (b) |
| gamma | 0.822 | | | |
| $A_{st}$ | 449 | $mm^2/m$ | | |
| b | 1000 | mm | | |
| $M_u$ | 33.37 | kNm | | |
| phi $M_u$ | 26.96 | kNm | phi $M_u \geq M^*$ | |
| $A_{st}/bd$ | 0.00284 | | | |
| $0.22(D/d)^2 f'_{cf}/f_{sy}$ | 0.00216 | | $Ast/db > 0.22(D/d)^2 f'_{cf}/f_{sy}$ | AS3600-2001 Cl 8.1.4.1 |
| $k_u$ | 0.0636 | | <0.4 | AS3600-2001 Cl 8.1.3 |
| $d_n$ | 8.25 mm | | gamma $k_u$ d | AS3600-2001 Cl 8.1.2.2 (b) |
| | | | | |
| TOP REINFORCEMENT | | | | |
| BENDING | | | | |
| Ultimate design bending moment | 29.35 | kNm | $wl^2/10$ | |
| Effective depth d | 159 | mm | | |
| $F_{sy}$ | 500 | MPa | | |
| $f_c'$ | 50 | MPa | Humeslab panel strength | |
| gamma | 0.696 | | | AS3600-2001 Cl 8.1.2.2 (b) |
| $A_{st}$ | 480 | $mm^2/m$ | | |
| b | 1000 | mm | | |
| $M_u$ | 36.72 | kNm | | |

# Appendix B

| | | | | |
|---|---|---|---|---|
| phi $M_u$ | 29.38 | kNm | phi $M_u \geq M^*$ | |
| $A_{st}/bd$ | 0.00302 | | | |
| 0.22 $(D/d)^2 f'_{cf}/f_{sy}$ | 0.00267 | | $Ast/db > 0.22(D/d)^2 f'_{cf}/f_{sy}$ | AS3600-2001 Cl 8.1.4.1 |
| $k_u$ | 0.051 | | <0.4 | AS3600-2001 Cl 8.1.3 |
| $d_n$ | 5.65 | mm | gamma $k_u$ d | AS3600-2001 Cl 8.1.2.2 (b) |
| | | | | |
| SHEAR | | | | |
| Ultimate Shear Force at support $V_s^*$ | 28.13 | kN | 1.15 $wl/^2$ | |
| $V_{uc}$ | 127.53 | kN | $B_1B_2B_3b_vd_o(A_{st}\ f'_c/b_v\ d_o)^{.333}$ | AS3600-2001 Cl 8.2.7.1 |
| phi $V_{uc}$ | 89.27 | kN | phi $V_{uc} \geq V_s^*$ | |
| Ultimate Shear Force of void $V_v^*$ | 26.19 | kN | 1.15 $wl/^2$ | |
| $b_v$ | 296 | mm | | |
| $V_{uc}$ | 56.62 | kN | $B_1B_2B_3b_vd_o(A_{st}\ f'_c/b_v\ d_o)^{.333}$ | AS3600-2001 Cl 8.2.7.1 |
| phi $V_{uc}$ | 39.63 | kN | phi $V_{uc} \geq V_v^*$ | |
| | | | | |
| DEFLECTION | | | | |
| $L_{ef}/d$ | 37.74 | | | |
| $k_3\ k_4\ ((delta/l_{ef})\ E_c/F_{d.ef})^{0.333}$ | 39.05 | | $L_{ef}/d<k_3\ k_4\ ((delta/l_{ef})\ E_c/F_{d.ef})^{0.333}$ | AS3600-2001 Cl 9.3.4.1 |
| | | | | |
| CRACK CONTROL | | | | |
| $F_{d.ef1}$ | 6.12 | kPa | G + 1xQ | AS3600-2001 Cl 9.4.1(a) |
| $F_{d.ser}$ | 5.52 | kPa | G + Xs x Q | |
| | | | | |
| $M^*_{s.1\ bottom}$ | 20.04 | kNm | $wl^2/11$ | |
| $M^*_{ser\ bottom}$ | 18.07 | kNm | $wl^2/11$ | |
| $d_{n.uncr}$ | 97.38 | | | |
| $I_g$ | 5.52E+08 | $mm^4$ | $bD^3/12$ | |
| $M_{crit}$ | 17.43 | kNm | $3\ I_g/(D/2)$ | AS3600-2001 Cl 9.4.1(a) |
| | $M^*_{s.1}$ > Mcrit so critical tensile zone | | | |
| | Wider cracks can be tolerated | | | AS3600-2001 Cl 9.4.1(b) |
| Max bar spacing | 300 | mm | | AS3600-2001 Cl 9.4.1(b) iii |
| Actual bar spacing | 200 | mm | | |
| | | | | |
| $M^*_{s.1\ top}$ | 22.04 | kNm | $wl^2/10$ | |
| $M^*_{ser\ top}$ | 19.88 | kNm | $wl^2/10$ | |
| $I_g$ | 5.72E+08 | $mm^4$ | $bD^3/12$ | |
| $M_{crit}$ | 18.05 | kNm | $3\ I_g/(D/2)$ | AS3600-2001 Cl 9.4.1(a) |
| | $M^*_{s.1}$ > Mcrit so critical tensile zone | | | |
| | Assuming wider cracks can not be tolerated, for example with tiled floor finishes. (If carpeted floor or timber floating floor, then wider cracks could be tolerated) | | | |

# Appendix B

| | | | | |
|---|---|---|---|---|
| $A_{st.min}$ | 570 | $mm^2$ | $3\ k_s A_{ct}/f_s$ | AS3600-2001 Cl 9.4.1(b) ii |
| Max bar spacing | 300 | mm | | AS3600-2001 Cl 9.4.1(b) iii |
| Max steel stress | 300 | MPa | | AS3600-2001 Cl 9.4.1(b) Note 2 |
| $E_c$ | 28600 | MPa | | |
| n | 6.99 | | $E_s/E_c$ | |
| $d_n$ | 29.49 | mm | $0.5\ b\ d_n^2 = nA_{st}\ (d-d_n)$ | |
| $I_{cr}$ | 6.48E+07 | $mm^4$ | $0.33\ b\ d_n^3 + n\ A_{st}\ (d-d_n)^2$ | |
| $I_{ef.ser}$ | 4.44E+08 | $mm^4$ | $I_{cr} + (I_g - I_{cr})(M_{cr}/M_{ser})^3$ | |
| $f_{scr}$ | 5.8 | MPa | M y/I | |
| Max steel stress | 300 | MPa | $>f_{scr}$ | AS3600-2001 Cl 9.4.4 (b) iv |
| $I_{ef.s.1}$ | 3.43E+08 | $mm^4$ | $I_{cr} + (I_g - I_{cr})(M_{cr}/M_{s.1})^3$ | |
| $f_{scr.1}$ | 8.3 | MPa | M y/I | |
| $0.8f_{sy}$ | 400 | MPa | $>f_{scr.1}$ | AS3600-2001 Cl 9.4.4 (b) v |

PROPPING DURING CONSTRUCTION

From Table 5:

| | | | |
|---|---|---|---|
| Truss Type | T110/0 | | 190 thick slab |
| Voided Slab-Truss spacings | 320 | mm | |
| Span between props | 3.1 | m | |

320 mm truss spacings selected to reduce propping of 6 m span to a single prop mid-span of the panel
reduce void width by 40 mm to 400 mm to accommodate the 8 trusses required for the 2500 wide panel

REINFORCEMENT

PANEL (BOTTOM REINFORCEMENT)

| | | | |
|---|---|---|---|
| Required from design | 449 | $mm^2/m$ | Ultimate Bending |
| Provided in Panel | | | |
| 8 T110/10 trusses over 2500 mm | 181 | $mm^2/m$ | |
| RF92 | 287 | $mm^2/m$ | |
| Total | 468 | $mm^2/m$ | |

IN-SITU SLAB (TOP REINFORCEMENT OVER SUPPORTS)

| | | | |
|---|---|---|---|
| Required from design | 570 | $mm^2/m$ | Flexural Crack Control Governs |
| Provide N12-175 | 628 | $mm^2/m$ | |

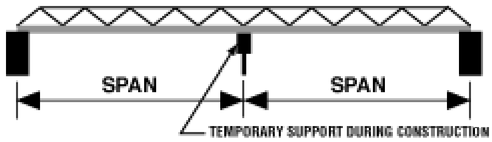
If wider cracks can be tolerated in the top of the slab, the reinforcement could be reduced to 480$mm^2$ (N12-225)

# Appendix C

## Transpan™ HumeSlab™ Design Software Output

***Introduction***

The purpose of the following calculations is to determine the maximum simply supported double span for a HumeSlab panel given structural properties and construction loads.




The panel must comply with the stability, strength and service limit state criteria specified in AS3610-1995 Formwork for Concrete (Ref. [2]).

***Stability***

The panel must resist overturning, uplift and sliding under the action of all the appropriate load combintions:

a) *Overturning*: In the case of a simply supported span overturning is not applicable

b) *Uplift*: The panel must resist forces from the appropriate load combination causing uplif Uplift is commonly cause by wind loads that are beyond the scope of this analysis and th fore uplift is not considered.

c) *Sliding*: The panel and its supports must resist forces from the appropriate load combination causing sliding.
AS3610 requires that formwork resist an applied horizontal live load of 1 kN/m plus the lat pressure of concrete.

Stage II – during placement of concrete.

$$1.5Q_{uh}^{C} + 1.5P^{C} < 0.8G^{R} + (fR) \quad (1)$$

***Strength***

The Panel must resist the bending and shear action effects from all the appropriate load combinations. In the case of a simply supported panel the following load combinations ar appropriate:

Stage I – prior to placement of concrete.

$$1.25G + 1.5Q_{uv} + 1.5M_1 \quad (2)$$

Stage II – during placement of concrete.

$$1.25G + 1.25G_c + 1.5Q_{uv} + 1.5M_2 \quad (3)$$

$$1.25G + 1.25G_c + Q_c \quad (4)$$

Stage III – after placement of concrete.

$$1.25G + 1.5G_c + 1.5Q_{uv} + 1.5M_3 \quad (5)$$

***Stiffness***

The panel stiffness must be such that the deformation under the appropriate load combination does not exceed the limits specified in Ref. [2]. In the case of a simply supported panel the following load combinations are appropriate:

Stage II – during placement of concrete.

$$G + G_c \quad (6)$$

Stage III – after placement of concrete.

$$G + G_c + M_3 \quad (7)$$

***Surface Finish***

The surface finish of the panel soffit conforms with the physical quality of a "Class 2" surface finish as specified in Ref. [2].

# Appendix C

***Panel Capacity*** The strength and stiffness of the panel is dependent on the truss, panel size and geometry. During construction the applied loads are resisted by the action of the truss members and panel concrete. The resistance provided by any mesh or additional reinforcement bars is ignored.

The following structural checks are performed:

| | |
|---|---|
| Stability | a) Sliding<br>b) Overturning |
| Strength | a) Top Chord Tension<br>b) Top Chord Tension<br>c) Bottom Chord Compression<br>d) Bottom Chord Tension<br>e) Concrete Compression<br>f) Diagonal Compression |
| Service | a) Deflection<br>b) Cracking |

The limit state resistance is calculated for each case.

***Maximum Span*** The maximum span is selected on the basis that the design action, calculated from the factored load combinations, does not exceed the capacity of the panel.

A summary of the calculations showing the maximum span for each action is given in the table below:

| Design Action | Max. Span (m) |
|---|---|
| Positive Bending | 3.25 |
| Negative Bending | 3.20 |
| Shear | 14.18 |
| Cracking | 3.60 |
| Deflection | 3.54 |

The Maximum span for the given configuration is therefore:

| | | |
|---|---|---|
| Maximum Span | 3.20 | mm |

***Stability*** The formwork assembly including the HumeSlab panel, falsework and connections are required to be designed to transfer the following limit state design load to anchorage or reaction points:

**Limit State Sliding Load, *H**   1.5   kN/m**

***Stacked Materials***

**The maximum span is based on the live load for stacked materials, before and after placement of concrete, being limited to a maximum of 2.0 kPa.**
**This load must be clearly indicated in the formwork documentation and construction control put in place to ensure it is not exceeded**.

# Appendix C

***Assumptions***

***1*** Vertical and horizontal action effects from environmental loads have been ignored.

***2*** The value for stacked materials during Stage **I** ($M_1$) applies also to Stage **III** ($M_3$) and during Stage **II** the value for stacked materials ($M_2$) is 0 kPa.

***3*** The effects of form face deflection and construction tolerances can be ignored.

***4*** The deviations specified for surface undulation, in Ref[2], will be interpreted as the deflection criteria for the panel as per the following table:

| Surface Quality Class | Surface Undulation Tolerance (mm) | Span/Defelection Ratio |
|---|---|---|
| 2 | 3 | 500 |
| 3 | 5 | 300 |
| 4 | 8 | 188 |

***5*** The welds connecting the diagonal wires to the top and bottom chord of the truss are capable of transmitting the full design action effects.

***6*** Truss geometry is as per the following table:

| Truss Type | Wire Size (mm) | | | |
|---|---|---|---|---|
| | Top | Bottom | Diagonal | Height |
| T80/10 | 10 | 6.3 | 6.3 | 82 |
| T110/10 | 10 | 6.3 | 6.3 | 111 |
| T150/10 | 10 | 6.3 | 6.3 | 154 |
| T190/10 | 10 | 6.3 | 6.3 | 191 |
| T110/12 | 12.5 | 6.3 | 6.3 | 112 |
| T150/12 | 12.5 | 6.3 | 6.3 | 155 |
| T190/12 | 12.5 | 6.3 | 6.3 | 192 |

# Appendix C

***Panel Properties***

| | | |
|---|---|---|
| Overall Slab Thickness, $d$ | 190 | mm |
| Minimum Cover to Bottom Reinforcement | 20 | mm |
| Concrete Density, r | 2500 | $kg/m^3$ |
| Concrete Strength at Loading, $f_{cm}$ | 50 | mPa |
| Concrete Modulus of Elasticity, $E_{cj}$ | 38007 | mPa |
| Panel Width, $b$ | 2500 | mm |
| Panel Thickness, $t_p$ | 55 | mm |
| Number of Truss per Panel, $n_t$ | 8 | |
| Number of Voids, $n_v$ | 4 | |
| Void Width, $b_v$ | 400 | mm |
| Void Thickness, $t_v$ | 75 | mm |
| Class of Surface Finish | 2 | |

***Construction Loads***

| | | |
|---|---|---|
| Panel Dead Load, $G$ | 1.3 | kPa |
| Insitu Slab Dead Load, $G_c$ | 2.1 | kPa |
| Construction Live Load, $Q_{uv}$ | 1.0 | kPa |
| Concrete Mounding Load, $Q_c$ | 3.0 | kPa |
| Stacked Materials, $M_1$ | 2.0 | kPa |
| Stacked Materials, $M_2$ | 0.0 | kPa |
| Stacked Materials, $M_3$ | 2.0 | kPa |

***Load Combinations***

| Stage | Load Combination | Load | Unit | Equation |
|---|---|---|---|---|
| II | Stability<br>$1.5Q_{uh}^C + 1.5P^C < 0.8G^R + (\phi R)$ | | | (1) |
| | Strength | | | |
| I | $1.25G + 1.5Q_{uv} + 1.5M_1$** | 6.2 | kPa | (2) |
| II | $1.25G + 1.25G_c + 1.5Q_{uv} + 1.5M_2$** | 5.9 | kPa | (3) |
| II | $1.25G + 1.25G_c + Q_c$* | 7.4 | kPa | (4) |
| III | $1.25G + 1.25G_c + 1.5Q_{uv} + 1.5M_3$** | 8.9 | kPa | (5) |
| II | *Stifness* | | | |
| II | $G + G_c$ | 3.5 | kPa | (6) |
| | $G + G_c + M_3$** | 5.5 | kPa | (7) |

(7) * Although AS3610 specifies that $Q_c$ will apply over an area of 1.6 m x 1.6 m, it has been applied over the full area of the panel.

** - The loads from stacked materials (*M*) may apply to one span only.

***Design Load***

Therefore the design loads are as follows:

| | | |
|---|---|---|
| Strength, w* | 8.9 | kPa |
| Service, $w_s$* | 5.5 | kPa |

# Appendix C

***Truss Properties***

| | | | | |
|---|---|---|---|---|
| | HumeSlab Truss Type, $T$ | T110/10 | | |
| | Average Truss Spacing, $T_s$ | 312 | mm | Ref.[1] |
| | Truss Height, $T_h$ | 111 | mm | |
| | Truss Bar Yield Strength, $f_{syt}$ | 450 | mPa | |
| ***Top Chord*** | Bar Diameter, $d_t$ | 10 | mm | |
| | Area, $A_t$ | 628 | $mm^2$ | |
| | Strut Length, $L_t$ | 200 | mm | |
| | Effective Length, $l_t$ | 180 | mm | |
| | Radius of Gyration, $r_t$ | 3 | mm | |
| | Slenderness Ratio, $l_t / r_t$ | 72 | | **OK** |
| ***Diagonal*** | Bar Diameter, $d_w$ | 6.3 | mm | |
| | Area, $A_w$ | 499 | $mm^2$ | |
| | Angle of Web, $q$ | 48 | degrees | |
| | Strut Length, $L_w$ | 149 | mm | |
| | Effective Length, $l_w$ | 120 | mm | |
| | Radius of Gyration, $r_w$ | 1.6 | mm | |
| | Slenderness Ratio, $l_w / r_w$ | 76 | | **OK** |
| ***Bottom Chord*** | Bar Diameter, $d_b$ | 6.3 | mm | |
| | Area, $A_b$ | 499 | $mm^2$ | |
| | Strut Length, $L_b$ | 200 | mm | |
| | Effective Length, $l_b$ | 180 | mm | |
| | Radius of Gyration, $r_b$ | 1.6 | mm | |
| | Slenderness Ratio, $l_b / r_b$ | 114 | | **OK** |
| ***Mesh*** | Mesh Size | F62 | | |
| | Wire Diameter, $d_m$ | 6.3 | | |
| | Area, $A_m$ | 390 | $mm^2$ | |

***Transformed Section*** For serviceability limit state the panel is analysed as an uncracked section using the Transformed Area method to determine the stresses in the steel and concrete.

| | | |
|---|---|---|
| Steel Elastic Modulus, $E_s$ | 200000 | mPa |
| Concrete Elastic Modulus, $E_{cj}$ | 38007 | mPa |
| Modular Ratio, $n$ | 5.3 | |
| Distance from Soffit to Top Chord | 140 | mm |
| Transformed Top Chord Area | 3306 | mm2 |
| Distance from Soffit to Bottom Chord | 29 | mm |
| Transformed Bottom Chord Area | 2625 | mm |
| Panel Concrete Area | 137500 | mm |
| Distance to the Neutral Axis, $y_g$ | 30.1 | mm |
| Second Moment of Inertia, $I_g$ | 7.59E+07 | $mm^4$ |

# Appendix C

***Capacity Calculations***

***Top Chord Compression***

In accordance with AS4100 – 1998 Steel Structures (Ref.[3]), Clause 6.1

$$N^* \leq \phi \, \alpha_c \, N_s$$

| | | | |
|---|---|---|---|
| where | $\phi$ | 0.9 | |
| | Section Jcapacity, $N_s = A_t f_{syt}$ | 283 | kN |
| | $\lambda_n$ | 96.6 | |
| | $\alpha_a$ | 17.6 | |
| | $\alpha_b$ | -1.0 | |
| | $\lambda$ | 79.0 | |
| | $\eta$ | 0.2 | |
| | $\varepsilon$ | 1.3 | |
| | $\alpha_c$ | 0.7 | |
| | Limit State Capacity, $\phi\alpha_c N_s$ | 175.0 | kN |
| | Truss Height, $T_h$ | 111 | mm |
| | Limit State Moment Capacity, $M^*_{tc}$ | 19.4 | kNm |

***Top Chord Tension***

In accordance with AS4100 – 1998 Steel Structures (Ref.[3]), Clause 7.1

$$N^* \leq \phi \, A_g \, f_y$$

| | | | |
|---|---|---|---|
| where | $\phi$ | 0.9 | |
| | Limit State Capacity, $\phi A_g f_y$ | 254.5 | kN |
| | Truss Height, $T_h$ | 111 | mm |
| | Limit State Moment Capacity, $M^*_{tt}$ | 28.2 | kNm |

***Bottom Chord Compression***

In accordance with AS4100 – 1998 Steel Structures (Ref.[3]), Clause 6.1

$$N^* \leq \phi \, \alpha_c \, N_s$$

| | | | |
|---|---|---|---|
| where | $\phi$ | 0.9 | |
| | Section Capacity, $N_s = A_b f_{syt}$ | 224.4 | kN |
| | $\lambda_n$ | 153.3 | |
| | $\alpha_a$ | 12.6 | |
| | $\alpha_b$ | -1.0 | |
| | $\lambda$ | 140.7 | |
| | $\eta$ | 0.4 | |
| | $\varepsilon$ | 0.8 | |
| | $\alpha_c$ | 0.3 | |
| | Limit State Capacity, $\phi\alpha_c N_s$ | 66.0 | kN |
| | Truss Height, $T_h$ | 111 | mm |
| | Limit State Moment Capacity, $M^*_{bc}$ | 7.3 | kNm |

***Bottom Chord Tension***

In accordance with AS4100 – 1998 Steel Structures (Ref.[3]), Clause 7.1

$$N^* \leq \phi A_g f_y$$

| | | | |
|---|---|---|---|
| where | $\phi$ | 0.9 | |
| Limit State Capacity, $\phi A_g f_y$ | | 202.0 | kN |
| Truss Height, $T_h$ | | 111 | mm |
| Limit State Moment Capacity, $M^*_b$ | | 22.4 | kNm |

***Panel Concrete Compression***

The maximum concrete compressive force is given by:

$$N^* \leq 0.68 f_{cm} t_p b$$

| | | | |
|---|---|---|---|
| therefore | | | |
| Limit State Capacity, $N^*_c$ | | 4675.0 | kN |
| Truss Height, $T_h$ | | 111 | mm |
| Limit State Moment Capacity, $M^*_{pc}$ | | 518.9 | kNm |

***Diagonal Compression***

In accordance with AS4100 – 1998 Steel Structures (Ref.[3]), Clause 6.1

$$N^* \leq \phi \alpha_c N_s$$

| | | | |
|---|---|---|---|
| where | $\phi$ | 0.9 | |
| Section Capacity, $N_s = A_b f_{syt}$ | | 224.4 | kN |
| | $\lambda_n$ | 101.8 | |
| | $\alpha_a$ | 17.1 | |
| | $\alpha_b$ | -1.0 | |
| | $\lambda$ | 84.7 | |
| | $\eta$ | 0.2 | |
| | $\varepsilon$ | 1.2 | |
| | $\alpha_c$ | 0.6 | |
| Limit State Capacity, $N^*$ | | 130.8 | kNm |

***Tensile Cracking***

AS3600, Ref[4] requires the maximum flexural stress in the concrete under short term service loads to be limited to

$$0.5\sqrt{f'_c}$$

The limit state service moment can be calculated from:

$$M^* = \frac{0.5\sqrt{f_{cm}}\, I_g}{y_g}$$

therefore

| | | |
|---|---|---|
| Limit State Service Moment Capacity, $M^*_c$ | 8.9 | kNm |

***or***

AS3600 also provides an alternative of limiting the increment in steel stress to 150 mPa.

| | | |
|---|---|---|
| Steel Stress | 150 | mPa |
| Area of Bottom Chord Steel, $A_b$ | 499 | $mm^2$ |
| Area of Mesh, $A_m$ | 390 | $mm^2$ |
| Total Area of Steel | 889 | $mm^2$ |
| Equivalent Axial Force, $N^*$ | 133.3 | kN |
| Truss Height, $T_h$ | 111 | mm |

***Stability Check***

| | | |
|---|---|---|
| Limit State Service Moment Capacity, $M^*_c$ | 14.8 | kNm |

***Sliding***

The force causing sliding must be transferred to an anchorage or reaction point on the permanent structure or foundation.

| | | |
|---|---|---|
| Limit state horizontal live load, $Q_{uh}$ | 1.0 | kN/m |
| Limit state lateral concrete pressure, $P$ | 0.1 | kN/m |
| Limit State horizontal design load, $H^*$ | 1.5 | kN/m |

***Span***

***Calculations***

***Positive Bending***

The positive moment capacity of the panel is given by the following:

| | | |
|---|---|---|
| Top Chord Compression, $M^*_{tc}$ | 19.4 | kNm |
| Bottom Chord Tension, $M^*_{bt}$ | 22.4 | kNm |
| Design Moment, $M^*$ | 19.4 | kNm |

The maximum span can be calculated from the following equation:

$$S_b = \sqrt{\frac{12M^*}{wb}}$$

therefore

| | | |
|---|---|---|
| Positive Bending Maximum Span, $S_b$ | 3.25 | m |

***Negative Bending***

The negative moment capacity of the panel is given by the following:

| | | | |
|---|---|---|---|
| Top Chord Tension, $M^*_{tt}$ | 28.2 | kNm | |
| Compression of Concrete Panel, $M^*_{pc}$ | 518.9 | kNm | |
| Bottom Chord Compression, $M^*_{bc}$ | 7.3 | kNm | Ignored in this calculation |
| Negative Bending Design Moment, $M^*$ | 28.2 | kNm | |

The maximum span can be calculated from the following equation:

$$S_b = \sqrt{\frac{8M^*}{wb}}$$

therefore

| | | |
|---|---|---|
| Negative Bending Maximum Span, $S_b$ | 3.20 | m |

***Shear***

The shear capacity of the panel is governed by the compression of the truss diagonal

The maximum span can be derived from the following equation:

$$S_v = \left(\frac{2N^*\sin\theta}{w_s b}\right)$$

therefore

| | | |
|---|---|---|
| Shear Maximum Span, $S_v$ | 14.18 | m |

***Cracking*** The moment capacity of the panel is given by the following:

Flexural Cracking, $M^*_c$ 14.8 kNm

The maximum span can be calculated from the following equation:

$$S_c = \sqrt{\frac{12M^*_c}{w_s b}}$$

therefore

Cracking Maximum Span, $S_c$ 3.60 m

***Deflection*** The maximum deflection of the panel can be calculated from the following equation:

$$\Delta = \frac{0.0074 w_s b S^4}{E_{ch} I_g}$$

To maintain the specified class the following Span/Deflection ratio must be achieved:

Span/Deflection Ratio, $\beta$ 500

Therefore the maximum span can be calculated from:

$$S_d = \left(\frac{135 E_{cj} I_g}{\beta w_s b}\right)$$

therefore

Deflection Maximum Span, $S_d$ 3.54 m

***References***


**1** Smorgon Steel Group, *Transfloor™ Technical Manual,* Smorgon Steel Group, Melbourne.

**2** Standards Association of Australia, *AS3610-1995 Formwork for Concrete*, Standards Association of Australia, Sydney, 1995.

**3** Standards Association of Australia, *AS4100-1998 Steel Structures*, Standards Association of Australia, Sydney, 1998.

**4** Standards Association of Australia, *AS3600-1994 Concrete Structures*, Standards Association of Australia, Sydney, 1994.

# Appendix D

## HumeSlab™ Quotation Checklist

| | | |
|---|---|---|
| 1. | HumeSlab™...................................................................= $ As quoted per $m^2$ | |
| 2. | Crane Hire...................................................................= $____________per $m^2$ | |
| | Typically 10 panels can be placed per hour after the crane is conveniently located | |
| 3. | Propping...................................................................= $____________per $m^2$<br>See quotation for propping centres. | |
| 4. | Labour...................................................................= $____________per $m^2$<br>Typically 2 men are required to place the above 10 panels/hr. | |
| 5. | Topping concrete...................................................= $____________per $m^2$<br>*See quotation for topping thickness estimate | |
| 6. | Top steel...................................................................= $____________per $m^2$<br>See quotation for top steel mass estimate. | |
| 7. | Labour for steel and concrete placement...................= $____________per $m^2$ | |
| 8. | Edge Boards...................................................................= $____________per $m^2$ | |
| 9. | Pumps, Buckets, etc...................................................= $____________per $m^2$ | |
| 10. | Sealing the joint between the panel............................= $____________per $m^2$<br>See Figure 30 for details. | |
| | Total Cost per $m^2$...................................................= $____________per $m^2$ | |
| | Total Area...................................................................= $____________$m^2$ | |
| | Total Cost...................................................................= $____________ | |

**End of Report**

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