

# STANDARD INDUSTRI PEMBINAAN

(CONSTRUCTION INDUSTRY STANDARD)

## CIS 8:2007

DESIGN GUIDE ON  
PRECAST CONCRETE HOLLOW CORE SLAB

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LEMBAGA PEMBANGUNAN INDUSTRI PEMBINAAN MALAYSIA



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**DESIGN GUIDE ON  
PRECAST CONCRETE HOLLOW  
CORE SLAB**

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## **Committee representation**

The Technical Committee on IBS which supervised the development of this Construction Industry Standard (CIS) was managed by the Construction Industry Development Board Malaysia and comprises representatives from the following organisations:-

ACP Industries Bhd  
Association of Consulting Engineers Malaysia  
Hume Industries Malaysia Bhd  
Institution of Engineers, Malaysia  
Jabatan Kerja Raya Malaysia  
Malaysia University of Science and Technology  
Malaysian Institute of Architects  
Master Builders Association Malaysia  
Setia Precast Sdn Bhd  
Sunway Precast Industries Sdn Bhd  
UAC Steel Systems Sdn Bhd  
Universiti Teknologi Malaysia

## **FOREWORD**

This Design Guide was developed by the IBS Standard Committee under the direction of the Construction Industry Development Board of Malaysia and compiled with the assistance of the Universiti Teknologi Malaysia in the efforts of upgrading the construction industries. This Design Guideline's intended to assist the construction industries in the use of Hollow Core Slab as one of IBS Components by providing guidelines for professionals and practitioners to refer to. It covers the followings:

- Design and durability of Precast Hollow Core Slabs
- Example of Design Calculations

The use of this Guideline is voluntary and compliance with this does not of itself confer immunity from legal obligations.

## DESIGN GUIDE ON PRECAST CONCRETE HOLLOW CORE SLAB

### 1. Introduction

Precast concrete refers to concrete components not cast in place, but rather cast off-site or in location different from their final location. Precast components in building construction are typically beams, column, floor, roof, or wall units. The use of precast concrete components in building construction is regarded as an economic, structurally sound and architecturally versatile form of construction. Precasting offers economies based on speed of construction and high quality materials.

In multi-storey buildings the floor units account for 50% to 60% of the material requirement. For this reason and because of large numbers of unit involved, they can be regarded as the most important structural components of all. Precast concrete floors have been widely used in various types of construction ranging from commercial to domestic building. This is due to the advantages of this type of floor over conventional flooring such as in-situ concrete or timber floor. Precast floors give maximum structural performance with minimum weight, rapid assembly on site and like many other precast components it can be produced in a highly efficient automated production method.

#### 1.1 Types of Precast Floor

The wide range of precast concrete floors can be grouped into five main types.

- Prestressed hollow core slab
- Reinforced and prestressed double-tee floor
- Composite prestressed plank-floor
- Composite beam and plank
- Beam and block floors

Table 1 gives a performance guide to the weights and maximum spans for some of the precast floor units.

**Table 1. Basic properties and performance characteristics of precast flooring [1]**

| Type                     | Width (mm)              | Typical Depth (mm) | Approximate Self weight (kN/m <sup>2</sup> ) | Approximate Max Span <sup>#</sup> (m) |
|--------------------------|-------------------------|--------------------|--|---------------------------------------|
| Hollow Core              | 1200, 600, 400, 333     | 110                | 1.90   | 7.0                                   |
|                          |                         | 150                | 2.25   | 8.5                                   |
|                          |                         | 200                | 3.00   | 10.5                                  |
|                          |                         | 250                | 3.35   | 11.0                                  |
|                          |                         | 300                | 3.60   | 14.5                                  |
|                          |                         | 400                | 4.80   | 18.5                                  |
| Double-tee               | 2400, 3000              | 500                | 5.50   | 22.5                                  |
|                          |                         | 400                | 3.60   | 14.5                                  |
|                          |                         | 500                | 4.10   | 17.0                                  |
|                          |                         | 600                | 4.50   | 19.5                                  |
|                          |                         | 700                | 4.95   | 21.0                                  |
| Composite Plank          | 2400 max                | 840                | 5.50   | 24.0                                  |
|                          |                         | 65/75*             | 3.36   | 4.50                                  |
|                          |                         | 75/75*             | 3.60   | 5.35                                  |
| Composite Beam and Plank | 900 – 2400 beam centres | 100/100*           | 4.80   | 7.35                                  |
|                          |                         | 455/115**          | 4.2-6.6                                      | 17.8                                  |
|                          |                         | 550/115**          | 4.5-7.4                                      | 20.9                                  |

#

Span for super imposed loading of 1.5 kN/mm<sup>2</sup> plus finishes 1.5 kN/mm<sup>2</sup>

\*\*

Depth of precast plank/ depth of insitu topping

\*\*

Depth of precast beam/depth of plank and topping

The choice and design of precast floor units are affected by the following factors:

- The spacing of the beams that support them;
- The superimposed load;
- The structural requirements (cavities, flat soffit, construction depth);
- The possibilities of manufacture and transport.

## 1.2 Precast Prestressed Hollow Core Slabs

Hollow core slabs are the most widely used precast floor component [3]. Floor units consisting of hollow slab type member is preferable in cases where a flat soffit is required. The cross-sections and versatility of these units varies depending on the manufacturing process. There are two common processes of producing hollow core slabs, which are slabs with oval or non-circular voids (Figure 1) and those with circular voids (Figure 2). Slabs with oval shaped voids are manufactured using the slip forming technique while those with circular voids are manufactured using the extrusion technique. The hollow core slabs are manufactured using long line extrusion or slip-forming processes. The long line extrusion is more popular than the slip-forming manufacturing method.

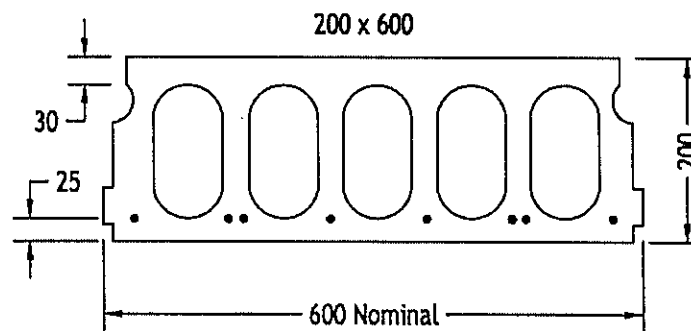


Figure 1. Slabs with Non-circular Voids



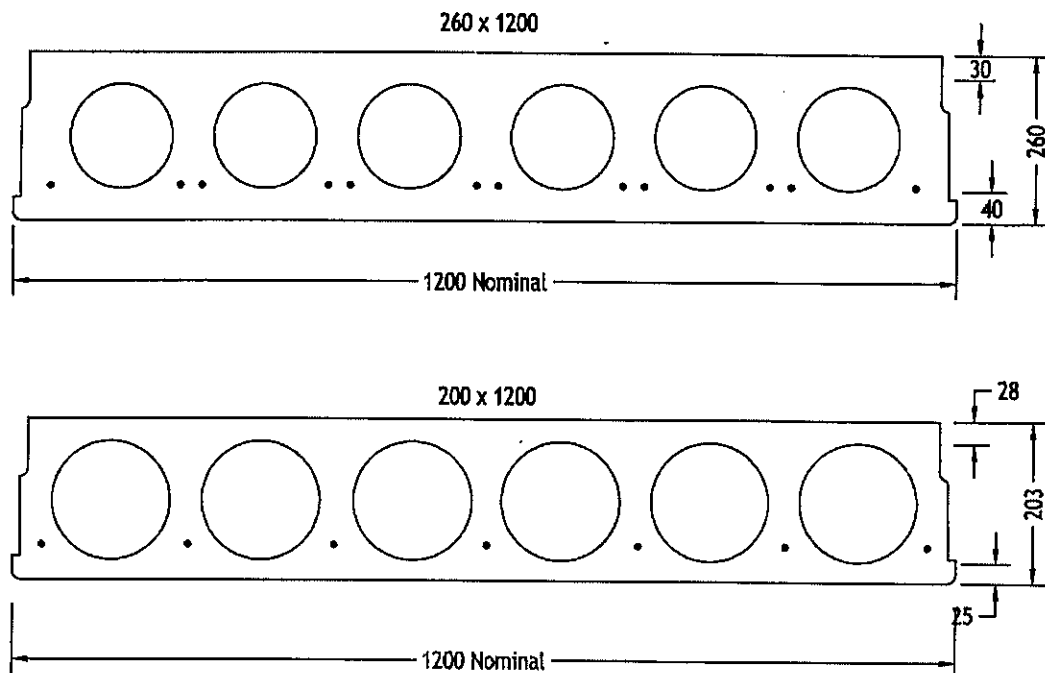


Figure 2. Slabs with Circular Voids

Hollow core slabs are generally designed to achieve two-hour fire resistance. Fire resistance of four hours can be designed and produced by either raising the level of the centroid of the tendons or by increasing the concrete covers.

Holes in the floor unit may be created in the precast units during the manufacturing stage before the concrete is hardened. The maximum size of the opening, which may be produced in the unit depends on the size of the voids and the amount of reinforcement that can be removed without jeopardising the strength of the precast element. Figure 3 (a) and (b) shows the permitted sizes and locations of openings and cutouts in hollow core slabs where the limits of cutout shown in Figure 3 (b) is given in Table 2. Small holes up to 150 mm in diameter can be core drilled on site. The holes should pass through the hollow core, and designers must be consulted if any of the reinforcing bars removed.

Sockets and other fixing must be formed on site using appropriate anchors. Fixing by shot fired method is not recommended. Maximum fixing depth should be less than 25 mm. Manufacturer's recommendation should be referred when selecting the suitable hanger system for ceilings. Drainage holes (10mm) in every void should be provided at both ends of slab to ensure water in the voids is drained-off. In local practice, a layer of topping is usually included in the construction of hollow core structural floors. The thickness of the topping ranging from 60 to 75mm at the support and minimum thickness of 45 to 50mm is maintained at mid-span. The lower thickness of topping at mid-span is due to prestressing camber.

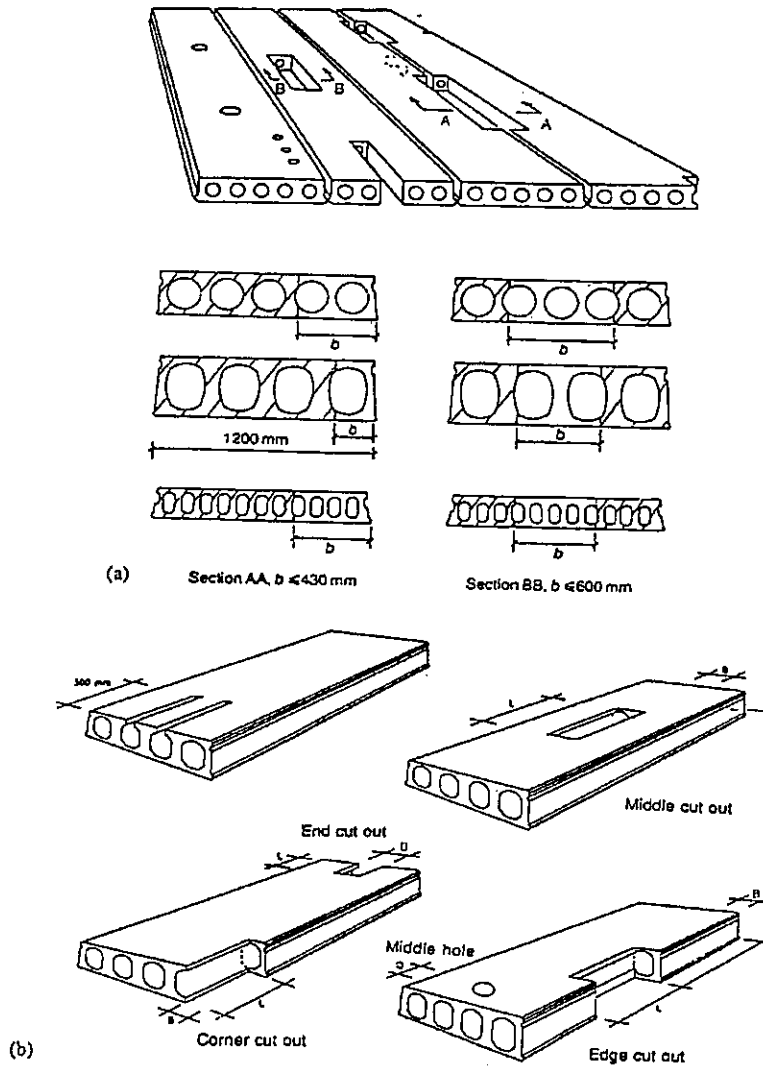


Figure 3. Permitted Sizes and Locations of Openings and Cutouts in Hollow Core Slabs [2]

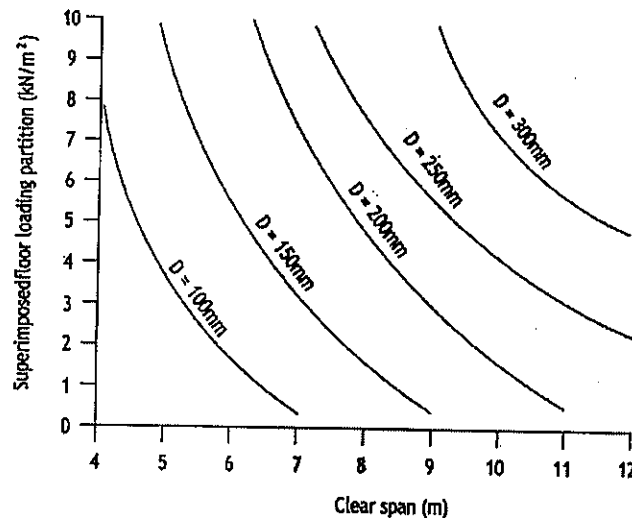
Table 2. Limits of Cut-out [2]

| Depth of Slab (mm) | Corner cut-out L x B (mm) | Edge cut-out L x B (mm) | End cut-out L x B (mm) | Middle cut-out L x B (mm) | Middle hole diameter $\phi$ (mm) |
|--------------------|---------------------------|-------------------------|------------------------|---------------------------|----------------------------------|
| 150                | 600 x 400                 | 600 x 400               | 1000 x 400             | 600 x 400                 | 80                               |
| 200                | 600 x 380                 | 600 x 400               | 1000 x 380             | 600 x 400                 | 130                              |
| 265                | 600 x 260                 | 600 x 400               | 1000 x 260             | 600 x 400                 | 130                              |
| 320                | 600 x 260                 | 600 x 400               | 1000 x 260             | 600 x 400                 | 170                              |
| 400                | 600 x 260                 | 600 x 400               | 1000 x 260             | 600 x 400                 | 170                              |

## 2. Design Considerations for Precast Prestressed Hollow Core Slabs

### 2.1 Concrete

All slabs are designed as single spanning. Hollow core slabs are manufactured using dry mix concrete with zero slump. The water/cement ratio is about 0.36. 10mm coarse aggregate is used. Hollow core slabs are designed as a series of I-section with semi circular or parallel side-slabs. The minimum concrete grade is C50 or C60. The maximum span/depth ratio is 55. The shape of the voids is usually circular, oval, or parallel sided with triangular and/or semi-circular ends. Design is carried out to the normal prestressed concrete clauses in BS 8110, however the consideration of the level of prestress in the transmission zone is based on recommendations given by FIP[5]. Typical load versus span data for these units are given in Figure 4.



**Figure 4. Applied Load vs Clear Span for Prestressed Hollow Core Slab [1]**  
 Note: details are intended for general information only. Precaster should be consulted for design of the slabs

### 2.2 Prestressing Tendons

The prestressing tendons commonly used are 270k, seven wire low relaxation (Class 2) plain or indented helical strands conforming to either the BS 5896-1980 or to ASTM A416-1980 Supplement. In local practice the tendons are prestressed to between 60-65% of the characteristic strength. The in-service effective prestressing force after losses from steel relaxation, creep, shrinkage and deformation is typically at 45-50% of the characteristic strength. The steel area is relatively low with  $\rho_{ps} = (A_{ps}/bd)$  between 0.1 to 0.25%.

### 2.3 Design of Cross-section

Some general rules for determining the size and shape of the voids are as follows:

- The height of the void should not exceed  $h-50$  mm, where  $h$  is the overall depth of the unit
- The diameter of the circular voids is usually  $h-60$  to  $75$ mm
- The minimum flange thickness is  $1.6\sqrt{h}$
- The bottom flange should be 30 mm thick to satisfy minimum cover requirement

## 2.4 Web Thickness

The critical region is near the ends of the units. At this region failure due to spalling, splitting and bursting of the concrete is quite common. The suggested minimum breadth of the web for fully stressed grade C60 concrete with depth of up to 300mm is 30mm. In general horizontal cracking due to excessive prestressing can be prevented by specifying web thickness of 30mm or greater.

## 2.5 Edge Profiles and Joint Strength

The sides of hollow core units have natural random roughness with indentation up to 2mm in depth. This is to ensure that an adequate transfer of horizontal and vertical shear between adjacent units i.e. at the joints. Typical profile geometries are shown in Figure 5. BS 8110 has indicated that for type of joint shown in this diagram the ultimate horizontal shear stress is  $0.23 \text{ N/mm}^2$ . The vertical shear resistance is provided by having indentation of 40mm x 10mm deep on the sides of hollow core slabs.

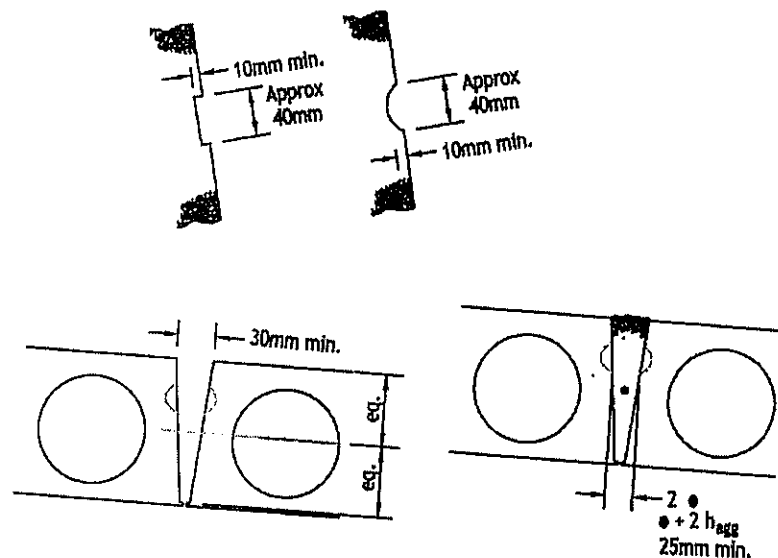


Figure 5. Typical Profile Geometry [1]

## 2.6 Flexural Capacity

The flexural strength of the precast unit at service and ultimate limit states are checked. The ratio of the ultimate moment to the serviceability moment of resistance is normally in the order of 1.7 to 1.8 indicating the serviceability condition will always be critical.

The serviceability flexural resistance  $M_{sr}$  is limited by the flexural tensile and compressive stresses in the concrete, precamber and deflections. Precamber and deflections are more critical for highly stressed units exceeding 7m in length.

Based on the allowable flexural tensile stress  $f_{ct}$  the precast units can be classified as follows:

- Class I No flexural tensile stress.
- Class II Flexural tensile stress not exceeding  $0.45\sqrt{f_{cu}}$  or  $3.5 \text{ N/mm}^2$  whichever is the smaller, but no visible cracking.
- Class III Flexural tensile stresses allowed but for crack widths less than  $0.1 \text{ mm}$  in units in severe environments where the stress is  $(1.34\sqrt{f_{cu}} - 2.2 \text{ N/mm}^2)$  or  $6.3 \text{ N/mm}^2$  whichever is smaller. In all other conditions the flexural stress allowed is  $(1.34\sqrt{f_{cu}} - 3.2 \text{ N/mm}^2)$  or  $7.3 \text{ N/mm}^2$ .

Precast units that meet Class II requirements are quite common. The flexural stress in the top surface of simply supported units is limited to  $0.33f_{cu}$ . This is not a major problem in most precast slab design.

The stresses due to the following loading arrangements should be checked in design:

- Immediately after the transfer of prestress
- After prestress losses have occurred
- After prestress losses have occurred but in service using  $\gamma_f = 1.0$

The transfer stress  $f_{ct}$  is a function of the final concrete strength  $f_{cu}$ . For  $f_{cu} = 60 \text{ N/mm}^2$ ,  $f_{ct}$  should be  $38$  to  $40 \text{ N/mm}^2$ . The design compressive and tensile strengths for class II members are  $0.5f_{ci}$  and  $0.45\sqrt{f_{ci}}$  respectively. Since in local practice the tendons are prestressed up to 65% of the characteristic strength, therefore the magnitude of the prestressing force per wire of tendon is taken as  $0.65(1-\eta)A_{ps}f_{pu}\gamma_m$  where  $\gamma_m = 1.0$  and  $\eta$  is the losses in the prestressing tendons. If low relaxation wires are used  $\eta = 0.04$  at transfer which is half the final value of 8 percent relaxation loss given in the specification. In the majority of cases  $f_{pu} = 1750 \text{ N/mm}^2$ .

Similarly the calculation of stresses after prestress losses has taken place will be the same except that the value of  $\eta$  refers to total losses. The losses of prestress can be attributed to the followings:

- i. Prestressing steel relaxation loss  
The prestressing steel relaxation loss can be calculated by multiplying a factor of 1.2 to the 1000 hours relaxation test value for cases involve class II design.
- ii. Concrete shrinkage losses  
The concrete shrinkage losses can be calculated by multiplying the shrinkage per unit length (taken as  $300 \times 10^{-6}$  for indoor manufacture) and modulus of elasticity of the tendons (taken as  $200 \text{ kN/mm}^2$ ) which results in  $60 \text{ N/mm}^2$  of shrinkage loss. If the tendon is prestressed at 65% and  $f_{cu} = 1750 \text{ N/mm}^2$  the concrete shrinkage loss will be 5.3 % of the total stress. i.e.  $100 \times 60/0.65 \times 1750$ .
- iii. Elastic shortening losses  
The elastic shortening losses can be calculated based on the initial short-term modulus of elasticity for the concrete. An average value of  $28 \text{ kN/mm}^2$  can be taken. Elastic shortening is calculated using the prestress in concrete  $f_{cc}$  at the level of the tendon. Therefore the elastic loss in steel is  $f_{cc}E_s/E_{ci}$
- iv. Creep of concrete losses  
The creep of concrete losses is proportional to the stress in the concrete at the level of the tendon. A specific creep strain value found in BS 8110 (Part 2, Figure 7.1) can be used to calculate prestress losses due to creep. Therefore the creep loss is creep strain value times  $f_{cc}E_s/E_{ci}$

Total prestress losses is normally in the range of 20 to 30 percent. The design effective prestress in the tendons after all losses is given by  $f_{pe}$

**2.7 Serviceability Limit State Design**

The design method is standard prestressed concrete procedures. The service moment of resistance  $M_{sr}$  is given by the following equations:

$$M_{sr} = (f_{bc} + 0.45\sqrt{f_{cu}})Z_b$$

Or

$$M_{sr} = (f_{tc} + 0.33f_{cu})Z_t$$

Where,

$$f_{bc} = P_f(1/A + e/Z_b) \text{ and } f_{tc} = P_f(1/A - e/Z_t)$$

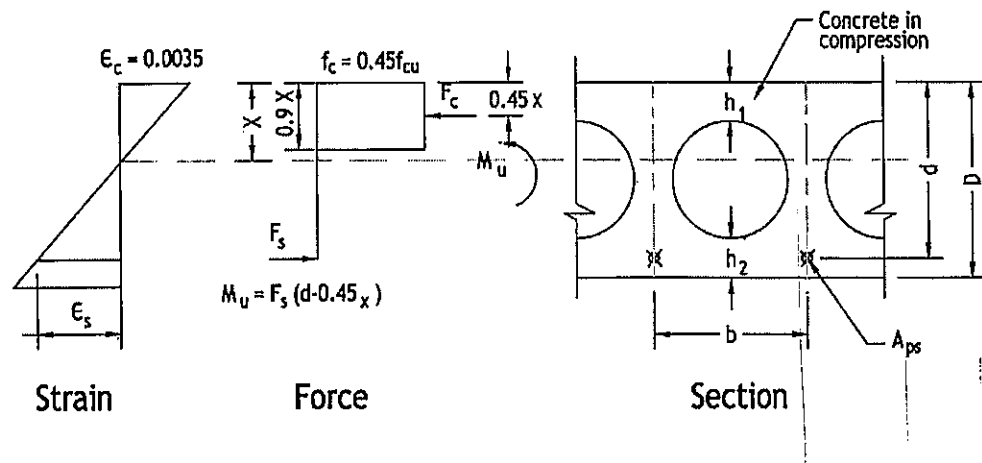
$f_{tc}, f_{bc}$  are the top and bottom fibre stresses

$Z_t, Z_b$  are the top and bottom section moduli

$e$  is the eccentricity of prestressing force from the geometrical neutral axis

**2.8 Ultimate Limit State Design**

The ultimate flexural resistance  $M_{ul}$  is limited by the ultimate compressive strength of concrete and design tensile stress in the tendons  $f_{pb}$ , this is when bonded tendon is being used. Calculation of the ultimate flexural resistance of a cross section can be based on the stress distribution diagram shown in Figure 6. Table 4.4 of BS 8110, Part 1 can be used to calculate the flexural capacity of a section as long as  $X$  is within the top flange thickness. However when  $X$  falls within the void are, the value of  $X$  can only be obtained by geometrical or arithmetic means.



**Figure 6. Strain and Force Distribution in Hollow Core Slab at Ultimate Limit State**

$$M_{ul} = f_{pb} A_p s (d - 0.45 X)$$

The values of  $X/d$  and  $f_{pb}$  can be found from BS 8110, Part 1, Table 4.4. Since the compressive stress block is not rectangular,  $X >$  covers to cores, the depth to the neutral axis must be found by geometrical or equilibrium of the section. An allowance is made for the existence of non-prestressed bars in the tension zone by replacing the area of reinforcement  $A_s$  by an equivalent area  $A_s f_y / f_{pu}$ .

## 2.9 Precamber and Deflections

Camber and deflection are calculated based on the flexural stiffness of the section, loading arrangement and support conditions. The calculations of short and long-term camber should be treated as approximation due to fact that the stiffness is affected by many variables.

An efficient approach to estimating camber and deflection is by determining the curvatures and then applying the curvature-area theorem [2]. For simply supported members with straight tendons the curvature due to prestress consists of three parts:

- i. Instantaneous curvature at transfer

$$1/r_b = P_i e / (E_c I) \text{ (upwards)}$$

- ii. Due to prestress losses

$$1/r_b = \delta P e / (E_c I) \text{ (downwards)}$$

- iii. Due to long term creep effect

$$\frac{1}{r_b} \text{ (longterm)} = \phi \frac{(P_i + (P_i - \delta P))}{2 E_c I} \times e \text{ (upwards)}$$

The value of  $(1/2)(P_i + (P_i - \delta P))$  is the average value of prestressing force and  $\phi$  is the creep coefficient which can be figured out from Figure 7.1 BS8110, Part 2.

Therefore, the total long term curvature due to prestress is given by i + ii + iii

$$\frac{1}{r_b} \text{ (longterm)} = \frac{P_i e}{E_c I} \left[ \eta + \frac{1 + \eta}{2} \right]$$

where,  $\eta$  is the prestress loss ratio given by  $(P_i - \delta P) / P_i$

The calculations of curvatures due to applied load are given by the following equations:

$$1/r_b \text{ (short-term)} = M_s / (E_c I) \text{ (downwards)}$$

$$1/r_b \text{ (long-term)} = \phi M_s / (E_c I) \text{ (downwards)}$$

Therefore, the total long-term curvature due to applied loads summation of the short-term and long-term curvatures.

$$\frac{1}{r_b} (\text{longterm}) = \frac{1+\phi}{E_c} \frac{M_s}{I} = \frac{M_s}{E_{ce}I}$$

where,  $E_{ce}$  is the effective modulus given by  $E_c/(1+\phi)$

#### iv. Total Deflection

For straight tendons in hollow core units, the total deflection,  $\delta$  due to self weight and applied loadings can be estimated using the following equations [1,2]:

$$\delta = \frac{5wL^2}{384 E_c I} \quad (\text{downwards})$$

$$\delta = \frac{\eta p_e L^2}{8 E_c I} \quad (\text{upwards})$$

As a general guide the camber for hollow core slabs is about  $L/300$ . Deflections are limited by span/500 or 20mm where brittle finishes are to be applied or span/350 or 20mm for non-brittle finishes. The net deflection should be less than span/1500.

## 2.10 Shear Capacity

For precast units of span more than 3m, shear is not a problem. As in any prestressed concrete elements the design shear capacity is calculated for two conditions i.e. the cracked and uncracked section in flexure.

It is not possible to specify shear reinforcement in hollow core slab, which warrant the prestressed concrete to do the work. Two modes of shear failures may occurs flexural shear and shear tension. Flexural shear occurs when flexural crack develops into a shear crack. Shear tension failures are diagonal crack found close to the units propagating through the unprestressed and unreinforced regions of the slab.

### 2.10.1 Shear Capacity in the Uncracked Region, $V_{co}$

The ultimate shear capacity is given by:

$$V_{co} = 0.67 b_v h \sqrt{f_i^2 + 0.8 f_{cp} f_t}$$

where,

$$f_i = 0.24 \sqrt{f_{cu}}$$

$f_{cp}$  is the concrete compressive stress at the centroidal axis due to effective prestress at the end of prestress transmission zone.

Although the above expression is derived based on rectangular section and not the actual flanged section appropriate to hollow core slab, the difference is accepted as being small. The Code of Practice [4] also take account that critical shear plane may occur in the prestressed development zone where  $f_{cp}$  is not fully developed. At this region prestressing force develop parabolically therefore a reduced  $f_{cp}$  is used to calculate the uncracked ultimate shear capacity.



$$f_{cpx} = \left(\frac{x}{l_p}\right) \left(2 - \frac{x}{l_p}\right) f_{cp}$$

where,

- $x$  is measured from the ends of the unit, measured at  $45^\circ$  to the inner bearing edge
- $l_p$  is taken as the greater of transmission length  $K_t \phi \sqrt{f_{ci}}$  or the overall depth  $D$  of the member

### 2.10.2 Shear Capacity in the Region Cracked in Flexure, $V_{cr}$

The value of  $V_{cr}$  can be calculated using the following semi-empirical equation:

$$V_{cr} = \left(1 - 0.55 \frac{f_{pe}}{f_{pu}}\right) v_c b_v d + M_o \frac{V}{M}$$

where,

$f_{pe}/f_{pu}$  is the ratio of the effective prestress after losses to the characteristic strength of the prestressing tendons.

$v_c$  is the permissible shear stress in Part I, Table 3.9, BS 8110

$M_o$  is the moment necessary to produce zero stress in the concrete at the effective depth level ( $d$ ). The following equation may be used to calculate  $M_o$ :

$$M_o = 0.8 f_{pt} I / y$$

where,

$f_{pt}$  is the concrete compressive stress due to effective prestressing force at depth  $d$  and distance  $y$  from the centroid axis of section which has a second moment of inertia  $I$ .

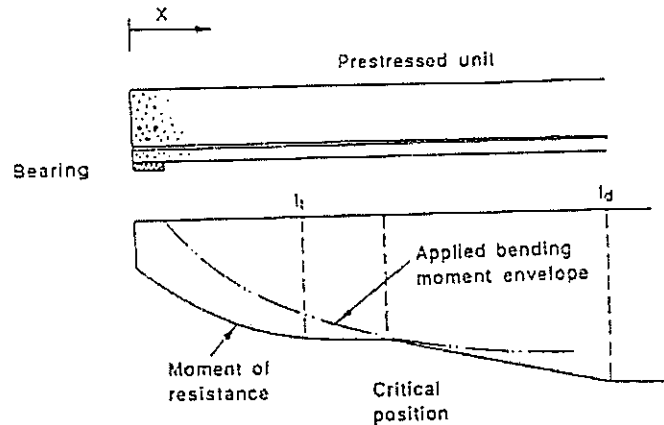
$V$  and  $M$  are the ultimate shear force and bending moment at the section considered

### 2.11 Anchorage and Bond Development Lengths

Transfer length  $l_t$  is the length that is needed to reach the state of equilibrium where the bond force equals the prestressing force, and the tendon stop slipping. Typical values for  $l_t$  are 50 to 80 diameters. The value of 70 diameters is normally used in design. If a flexural crack occurs within the transfer length, the state of equilibrium will not be achieved. An additional length is necessary to ensure that ultimate stress in the tendon is developed. This additional distance is known as flexural-bond length.

The total of transfer length and flexural-bond length is the development length  $l_d$ . According to BS 8110, Part 1, clauses 4.3.8.4 and 4.10.1 gives  $l_d =$  greater of  $D$  (depth) or  $K_t \phi \sqrt{f_{ci}}$  where  $\phi$  is the bar diameter  $K_t = 240$  for strand, 400 for crimped wire and 600 for plain wire. By knowing the development length, designer may be able to assess the implication of flexural cracking in this region. If a flexural crack occurs at a distance of less than the development length measured from the end of the slab, then full flexural capacity has been achieved. If the crack occurs in between the transfer and development lengths, then the capacity is larger than the flexural cracking moment but smaller than the ultimate moment. If the crack is located before the transfer length, then the anchorage capacity is smaller than the cracking moment, thus

failure occurs upon flexural cracking. Figure 7 shows the development and anchorage lengths in hollow core units.



**Figure 7. Development Length and Anchorage Length in Hollow Core Slab [1]**

The stress at the end of the transfer length is equal to the prestress after losses. The stress at the end of the development length is equal to the rupture stress in the tendon.

The ACI Code [1] formulae for the transfer and development lengths are as follows:

$$l_t = f_{pe} \frac{\phi}{2I}$$

$$l_d = l_t + \left[ (f_{pu} - f_{pe}) \frac{\phi}{7} \right]$$

where  $\phi$  is the bar diameter.

## 2.12 Transverse Load Distribution

The load distribution is determined based on tests or theoretical analyses. The assumptions made for determination the load distribution are as follows:

- Floor units are simply supported and 1.2m wide
- Concrete loading is linear and acting parallel to the span of the slab, and
- The units are provided with transverse ties to prevent them separating from each other

Reference should be made to the load distribution curves that are available in handbook such as [2] in order to determine the lateral load distribution coefficients.

## 2.13 Cantilever Design Using Hollow Core Slabs

In order to design cantilever spans of up to 1.5m long for units 200 to 250mm deep, two methods can be followed.

- If no structural topping is used, top steel should be provided in the cantilevered part and in the main span for a considerable distance to provide a tie-back with an adequate factor of safety of 2.

- Use of debonded strands so that no upward prestressing force is applied in the cantilevered portion. In this arrangement the transmission lengths for top steel should be increased by 50% to cater for the varying nature of concrete close to the top units.

### 3. Design Examples

#### Example 1

Design a 6.0 m span 200mm deep non-composite dry-cast hollow core slab with the following design data:

- Superimposed loading 5.0 kN/m<sup>2</sup>
- Finishes 1.5 kN/m<sup>2</sup>
- Partition 1.0 kN/m<sup>2</sup>
- Services and Suspended Ceiling 0.5 kN/m<sup>2</sup>
- Fire rating 2 Hrs
- End bearing length (nominal) 75 mm

#### Solution:

Effective span = 6000 + 75/2 + 75/2 = 6075 mm

#### Loading

| Loading     |   | Service | Ultimate |
|-------------|---|---------|----------|
| Dead loads: | Self weight of slab*                      | 2.60    | 3.64     |
|             | Partition, finishes, services and ceiling | 3.00    | 4.20     |
|             |   | 5.60    | 7.84     |
| Live loads: | Superimposed                              | 5.00    | 8.00     |
|             | Total                                     | 10.60   | 15.84    |

\* ignore the insitu concrete in the longitudinal gaps between units

#### Design in Flexure

$$M_s = 10.60 \times 6.075^2/8 = 48.9 \text{ kNm} \times 1.2 \text{ (for unit width)}$$

$$= 58.7 \text{ kNm per unit}$$

$$M_u = 15.84 \times 6.075^2/8 = 73.1 \text{ kNm} \times 1.2 \text{ (for unit width)}$$

$$= 87.9 \text{ kNm per unit}$$

#### Design in Shear

$$V_u = 15.84 \times 6.075/2 = 48.1 \text{ kN/m} \times 1.2 \text{ (for unit width)}$$

$$= 57.7 \text{ kN per unit}$$

Assuming design data for a series of hollow core slab is as shown in table below. Select 20SS77 unit, requiring 7 nos. 10.9 mm diameter strands of total  $A_{ps} = 497 \text{ mm}^2$  ( $0.204\%A_c$ ).


## Design in Bearing

Net bearing length (BS8110, clause 5.2.3.3) = 600 mm

Net bearing width (BS8110, clause 5.2.3.7) = 75 - 15 = 60 mm

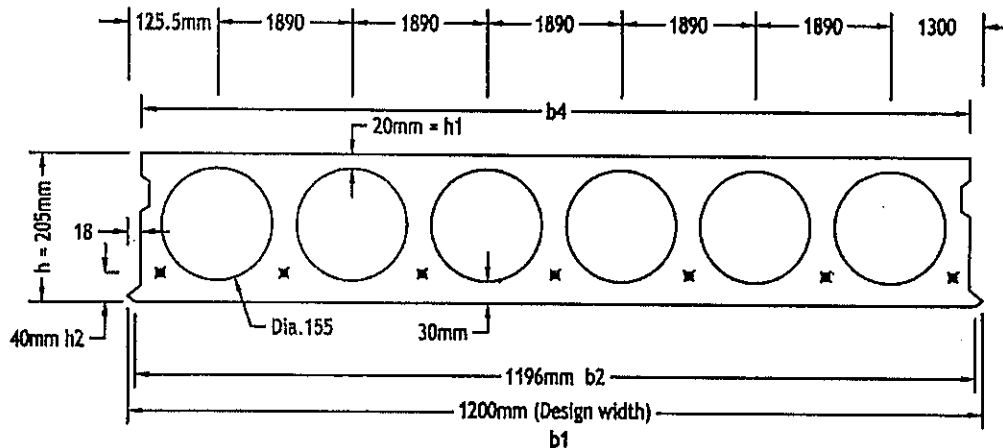
Bearing stress =  $57.7 \times 1000 / 600 \times 60 = 1.60 \text{ N/mm}^2 < 0.4f_{cu} = 16 \text{ N/mm}^2$

Example data for 200 mm deep prestressed concrete hollow core slabs, non-composite section properties and capacities [1]

| <b>Basic section properties</b>  |            |             |             |               |                   |
|--|------------|-------------|-------------|---------------|-------------------|
| $b = 1200 \text{ mm}$  |            |             |             |               |                   |
| $h = 200 \text{ mm}$   |            |             |             |               |                   |
| Number of circular voids = 6   |            |             |             |               |                   |
| $A_c \text{ (net)} = 135\,030 \text{ mm}^2$  |            |             |             |               |                   |
| $y_t = 97.96 \text{ mm}$   |            |             |             |               |                   |
| $y_b = 102.04 \text{ mm}$  |            |             |             |               |                   |
| $I_{xx} = 678.45 \times 10^6 \text{ mm}^4$   |            |             |             |               |                   |
| $Z_t = 6.72 \times 10^6 \text{ mm}^3$  |            |             |             |               |                   |
| $Z_b = 6.65 \times 10^6 \text{ mm}^3$  |            |             |             |               |                   |
| $f_{cu} = 60 \text{ N/mm}^2$   |            |             |             |               |                   |
| Unit Type  | Width (mm) | $M_s$ (kNm) | $M_u$ (kNm) | $V_{co}$ (Kn) | $100 A_{ps}/A_c'$ |
| 20SS76   | 1200       | 64.65       | 83.74       | 105.6         | 0.164             |
| 20SS77   | 1200       | 75.78       | 110.71      | 109.4         | 0.204             |
| 20SS78   | 1200       | 88.29       | 140.29      | 113.0         | 0.270             |
| <p><math>A_c'</math> = gross cross-sectional area, e.g. <math>1200 \times 200 \text{ mm}^2</math><br/>           Serviceability moment, <math>M_s</math> are for Class 2 BS 8110, Clause 4.3.4.3(b) using <math>f_{ct} = 3.50 \text{ N/mm}^2</math><br/>           Ultimate moments, <math>M_u</math> are calculated based on Clause 4.3.7.3 BS 8110, using <math>f_{pb} = 0.87f_{pu}</math><br/>           Shear forces are calculated based on Clause 4.3.8.4 using <math>f_{cpX}</math> measured at approximately 175mm from end of unit and using transmission length of 515 mm.</p> |            |             |             |               |                   |
| Code: e.g. 20SS77  |            |             |             |               |                   |
|  <p>77 – 7 wire 10.9mm dia<br/>           SS – 1200mm wide<br/>           20 – depth in cm</p>  |            |             |             |               |                   |

**Example 2**

The following design calculations appear courtesy of Eastern Pretech (M) Sdn. Bhd.

**i) Material Properties****Concrete**

$f_{ci} = 35$  MPa, HCS cube strength at transfer

$f_{cu} = 50$  MPa, HCS cube strength at 28 days

$f_{ct} = 35$  MPa, HCS cube strength at 28 days

$E_{ci} = 27000$  MPa, HCS modulus of elasticity at transfer

$E_{cs} = 30000$  MPa, HCS modulus of elasticity at 28 days

$E_{ct} = 26000$  MPa, Topping modulus of elasticity at 28 days

**Strand**

$f_{pu} = 1860$  MPa, Characteristic strength

$E_s = 195000$  MPa, Modulus of elasticity

Span = 7.405 m

$$\begin{aligned}
 \text{Loading} &= \text{HCS s/w} && = 2.6 \text{ k/m}^2 \\
 &\text{Concrete topping} && = 1.68 \text{ k/m}^2 \\
 &\text{SDL} && = 0.5 \text{ k/m}^2 \\
 &\text{Live load} && = 2.5 \text{ k/m}^2
 \end{aligned}$$

$$\begin{aligned}
 \therefore \text{ Try using EV6 - 2.05 - 2M - 3S, } A_s &= 2 \times 100 + 3 \times 55 \\
 &= 365 \text{ mm}^2
 \end{aligned}$$

***Stress Limit at Transfer***

$$\text{Tensile stress} = 0.45 \sqrt{f_{cu}} = -2.66 \text{ N/mm}^2$$

$$\text{Compressive stress} = 0.5f_{cu} = 17.5 \text{ N/mm}^2$$

***Stress Limit at Service***

$$\text{Tensile stress} = 0.45 \sqrt{f_{cu}} = -3.18 \text{ N/mm}^2$$

$$\text{Compressive stress} = 0.33f_{cu} = 16.5 \text{ N/mm}^2$$

***ii) Section Properties***

$$a = 18 \text{ mm}$$

$$b_1 = 1200 \text{ mm}$$

$$b_2 = 1196 \text{ mm}$$

$$b_3 = (b_2 + b_4 - 2a) / 2 = 1157 \text{ mm}$$

$$b_4 = 1154 \text{ mm}$$

$$\begin{aligned}
 A_c &= b_3 \times h - n_v \times A_v \\
 &= 1157 \times 205 - 6\pi(155)^2 / 4 \\
 &= 123970 \text{ mm}^2
 \end{aligned}$$

$$A_s = 365 \text{ mm}^2$$

Area with strand area,

$$\begin{aligned}
 A_i &= A_c + \left( \frac{E_s}{E_{50}} - 1 \right) A_s \\
 &= 123970 + (195000 / 30000 - 1) \times 365 = 125978 \text{ mm}^2
 \end{aligned}$$

$$\begin{aligned}
 Y_b &= \left[ (b_3 \times h^2)/2 - n_v \times (h - h_1 - D_v/2) + (E_s/E_{s0} - 1) \times A_v \times h_2 \right] / A_i \\
 &= \left[ (1157 \times 205^2)/2 - 6 \times 18869.2 \times (205 - 20 - 155/2) + \left( \frac{195000}{30000} - 1 \right) \times 365 \times 40 \right] / 125978 \\
 &= 97 \text{ mm}
 \end{aligned}$$

$$Y_t = h - Y_b = 205 - 97 = 108 \text{ mm}$$

$$\begin{aligned}
 I_i &= b_3 \times h^3/12 + b_3 \times h \times (h/2 - Y_b)^2 - n_v \times \pi \times D_v^4/64 - n_v \times D_v^2/4 \times \pi \\
 &\quad \times (h_1 + D_v/2 - Y_t)^2 + (E_s/E_c - 1) \times A_s \times (Y_b - h_2)^2 \\
 &= 1157 \times 205^3/12 + 1157 \times 205 \times (205/2 - 97)^2 - 6 \times \pi \times 155^4/64 - 6 \times 155^2/4 \times \pi \\
 &\quad \times (30 + 155/2 - 108)^2 + (195000/30000 - 1) \times 365 \times (97 - 40)^2 \\
 &= 6.743 \times 10^8 \text{ mm}^4
 \end{aligned}$$

$$\begin{aligned}
 Z_t &= I_i/Y_t = \frac{6.743 \times 10^8}{108} \\
 &= 6.2435 \times 10^6 \text{ mm}^3
 \end{aligned}$$

$$\begin{aligned}
 Z_b &= I_i/Y_b = \frac{6.743 \times 10^8}{97} \\
 &= 6.952 \times 10^6 \text{ mm}^3
 \end{aligned}$$

### iii) Tension Losses

#### Prestressing Losses

$$P_t = 1000 \text{ MPa}$$

$$\begin{aligned}
 F_{ct} &= P_t \times A_s/A_i + (P_t \times A_s \times e_o^2)/I_i \\
 &= 1000 \times 365/125978 + (1000 \times 365 \times 57^2)/6.743 \times 10^8 \\
 &= 4.656 \text{ MPa (average concrete stress at strands at transfer)}
 \end{aligned}$$

**Bending Moment due to self weight**

$$\begin{aligned}
 M_{sw} &= s/w \times L^2 \times 1.2 \times 10^6 / 8 \\
 &= 2.6 \times 7.405^2 \times 1.2 \times 10^6 / 8 \\
 &= 21.385 \times 10^6 \text{ Nmm}
 \end{aligned}$$

$$\begin{aligned}
 F_{mf} &= F_{ef} - M_{sw} \times e_o / I_t \\
 &= 4.656 - 21.385 \times 10^6 \times 57 / 6.743 \times 10^8 \\
 &= 2.848 \text{ MPa (tension at lifting)}
 \end{aligned}$$

$$\begin{aligned}
 F_{av} &= (F_{mf} + F_{ef}) / 2 \\
 &= (4.656 + 2.848) / 2 \\
 &= 3.752 \text{ MPa (average compression at strands)}
 \end{aligned}$$

$$\begin{aligned}
 L_1 &= \left[ F_{av} \times \frac{E_s}{E_{35}} \times P_i \right] \times 100 \\
 &= \left[ \frac{3.752 \times 195000}{27000 \times 1000} \right] \times 100 \\
 &= 2.7\%
 \end{aligned}$$

**Relaxation Loss**

$$L_2 = 2.5\% \text{ (from manufacturer)}$$

**Shrinkage loss**

BS 8110.2 Fig.7.2

$$\begin{aligned}
 L_3 &= 100 \varepsilon_{sc} \times \frac{E_s}{P_i} \quad \varepsilon_{sc} = 2.25 \times 10^{-4}, RH_{av} = 75\% \\
 &= 100 \times 2.25 \times 10^{-4} \times 195000 / 1000 \\
 &= 4.39\%
 \end{aligned}$$

**Creep Loss**

BS8110.2 Fig.7.1

 $RH_{av} = 70\%$ ,  $\varepsilon_{sc} = 2.16$  Loading at 14 days

$$\begin{aligned}
 L_4 &= 100 \varepsilon_{sc} \times F_{av} \times E_s / (E_{50} \times P_i) \\
 &= 100 \times 2.16 \times 3.752 \times 195000 / (30\,000 \times 1000) = 5.268\% \quad \text{Total losses} = 14.858\%
 \end{aligned}$$



**iv) Serviceability Check**

**Stresses at Support at Transfer**

$$T_f = \left(1 - \frac{L_1}{100}\right) \times P_i$$

$$= \left(1 - \frac{2.7}{100}\right) \times 1000$$

$$= 973 \text{ MPa}$$

$$T_f \times \frac{A_s}{A_i} = 973 \times \frac{365}{125978}$$

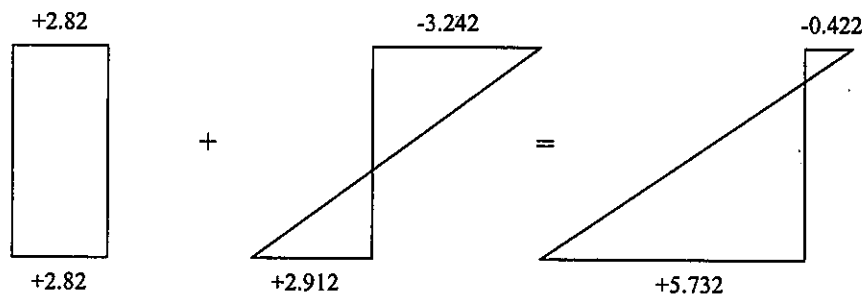
$$= +2.82 \text{ MPa}$$

$$T_f \times A_s \times e_o / Z_i = \frac{973 \times 365 \times 57}{6.2435 \times 10^6}$$

$$= 3.242 \text{ MPa}$$

$$T_f \times A_s \times e_o / Z_b = \frac{973 \times 365 \times 57}{6.952 \times 10^6}$$

$$= 2.192 \text{ MPa}$$



Tensile stress =  $0.422 \text{ N/mm}^2 > -2.66 \text{ N/mm}^2$

O.K.

Compressive stress =  $5.732 \text{ N/mm}^2 < 17.5 \text{ N/mm}^2$

O.K.

**Stresses at Transfer Length at Transfer**

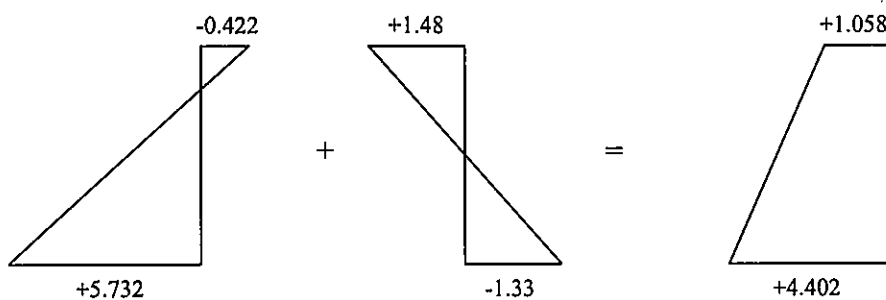
$$\text{Transfer length} = \frac{240 \times 12.9}{\sqrt{35}}$$

$$= 523 \text{ mm}$$

$$\begin{aligned}
 M_s &= \frac{WL_x}{2} - \frac{Wy^2}{2} \\
 &= \frac{1.2 \times (2.6 + 1.68) \times 7.405 \times 0.523}{2} - \frac{1.2 \times (2.6 + 1.68) \times 0.523^2}{2} \\
 &= 9.243 \text{ kNm}
 \end{aligned}$$

$$\begin{aligned}
 M_s/Z_t &= \frac{9.243 \times 10^6}{6.2435 \times 10^6} \\
 &= 1.48 \text{ MPa}
 \end{aligned}$$

$$\begin{aligned}
 M_s/Z_b &= \frac{9.243 \times 10^6}{6.952 \times 10^6} \\
 &= 1.33 \text{ MPa}
 \end{aligned}$$



$$\text{Tensile stress} = 1.058 \text{ N/mm}^2 > -2.66 \text{ N/mm}^2$$

O.K.

$$\text{Compressive stress} = 4.402 \text{ N/mm}^2 < 17.5 \text{ N/mm}^2$$

O.K.

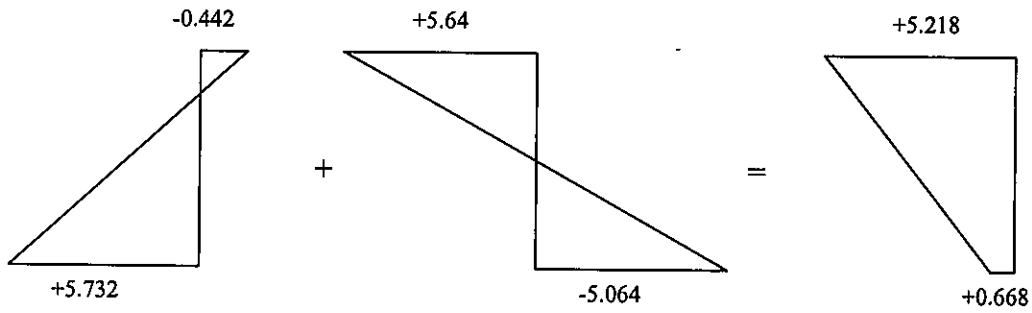
*Stresses at Midspan at Transfer*

$$\frac{M_{sw}}{Z_t} = \frac{(2.6 + 1.68) \times 1.2 \times 7.405^2 \times 10^6}{8 \times 6.2435 \times 10^6}$$

$$= 5.64 \text{ MPa}$$

$$\frac{M_{sw}}{Z_b} = \frac{(2.6 + 1.68) \times 1.2 \times 7.405^2 \times 10^6}{8 \times 6.952 \times 10^6}$$

$$= 5.064 \text{ MPa}$$



Tensile stress =  $5.218 \text{ N/mm}^2 > -2.66 \text{ N/mm}^2$

O.K.

Compressive stress =  $0.668 \text{ N/mm}^2 < 17.5 \text{ N/mm}^2$

O.K.

*Final Stresses at Support*

$$T_{pe} = (1 - \frac{L_i}{100}) \times P_i = (1 - \frac{14.858}{100}) \times 1000 = 851.42 \text{ MPa}$$

$$T_{pe} \times \frac{A_s}{A_i} = \frac{851.42 \times 365}{125978}$$

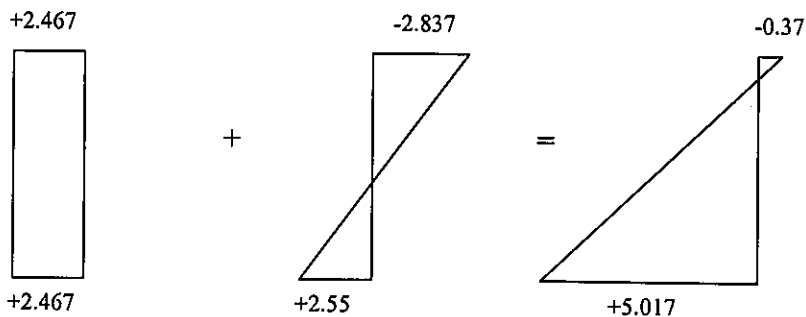
$$= +2.467 \text{ N/mm}^2$$

$$T_{pe} \times A_s \times \frac{e_o}{Z_i} = \frac{851.42 \times 365 \times 57}{6.2435 \times 10^6}$$

$$= -2.837 \text{ N/mm}^2$$

$$T_{pe} \times A_s \times \frac{e_o}{Z_b} = \frac{851.42 \times 365 \times 57}{6.952 \times 10^6}$$

$$= +2.55 \text{ N/mm}^2$$



Tensile stress =  $-0.37 \text{ N/mm}^2 > -3.18 \text{ N/mm}^2$

O.K.

Compressive stress =  $5.017 \text{ N/mm}^2 < 16.5 \text{ N/mm}^2$

O.K.

*Final Stresses in Transfer Length*

$$M_s = \frac{WL_x}{2} - \frac{Wx^2}{2}$$

$$= \frac{(0.5 + 2.5) \times 1.2 \times 7.405 \times 0.523}{2} + \frac{(0.5 + 2.5) \times 1.2 \times 0.523^2}{2}$$

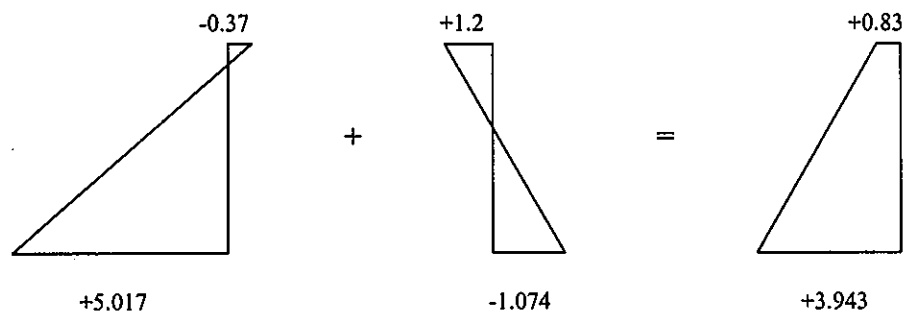
$$= 7.463 \text{ kNm}$$

$$M/Z_t = \frac{7.463 \times 10^6}{6.2435 \times 10^6}$$

$$= +1.2 \text{ N/mm}^2$$

$$M/Z_b = \frac{7.463 \times 10^6}{6.952 \times 10^6}$$

$$= -1.074 \text{ N/mm}^2$$



Tensile stress =  $0.83 \text{ N/mm}^2 > -3.18 \text{ N/mm}^2$

O.K.

Compressive stress =  $3.943 \text{ N/mm}^2 < 16.5 \text{ N/mm}^2$

O.K.

*Final Stresses in Midspan*

$$M_f = (0.5 + 2.5) \times 1.2 \times 7.405^2 / 8$$

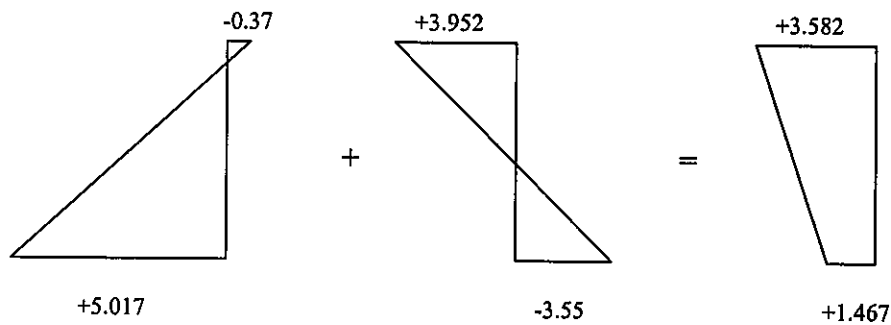
$$= 24.675 \text{ kNm}$$

$$M/Z_t = \frac{24.675 \times 10^6}{6.2435 \times 10^6}$$

$$= +3.952 \text{ N/mm}^2$$

$$M/Z_b = \frac{24.675 \times 10^6}{6.952 \times 10^6}$$

$$= -3.55 \text{ N/mm}^2$$



Tensile stress =  $3.582 \text{ N/mm}^2 > -3.18 \text{ N/mm}^2$

O.K.

Compressive stress =  $1.467 \text{ N/mm}^2 < 16.5 \text{ N/mm}^2$

O.K.

v) **Ultimate Limit Stage**

## Bending Moment Capacity

$$M_u = \frac{[(2.6+1.68) \times 1.4 + 0.5 \times 1.4 + 2.5 \times 1.6] \times 1.2 \times 7.405^2}{8}$$

$$= 87.9 \text{ kNm}$$

$$\frac{f_{pu} A_{ps}}{f_{cubd}} = \frac{1860 \times 365}{50 \times 1200 \times 218}$$

$$= 0.0519$$

$$\frac{f_{pe}}{f_{pu}} = \frac{973}{1860}$$

$$= 0.523$$

From Table 4.4, BS 8110(1997)

$$x/d = 0.14112$$

$$x = 0.14112d$$

$$\frac{f_{pb}}{0.95 f_{pu}} = 1$$

$$\therefore f_{pb} = 0.95 \times 1 \times 1860$$

$$= 1767 \text{ N/mm}^2$$

$$F_{pb} = 1767 \text{ N/mm}^2 \times 365 \text{ mm}^2 \times 10^{-3}$$

$$= 645 \text{ kN}$$

$$Z = d - 0.45x$$

$$= d - 0.45(0.14112 d)$$

$$= 204.16 \text{ mm}$$

$$M_{resistance} = 645 \text{ kN} \times 0.20416 \text{ m}$$

$$= 131.7 \text{ kNm} > 87.9 \text{ kNm} \quad \text{O.K.}$$

**vi) Shear**

$$b_w = b_3 - n_v \times D_v = 1157 - 6 \times 155 \\ = 227 \text{ mm}$$

$$V_{act} = (1.4 \times s/w + 1.4W_1 + 1.6 \times W_2) \times (L/2 - x) \times 1.2 \\ = (1.4 \times (2.6 + 1.68)) + 1.4 \times 0.5 + 2.5 \times 1.60) \times (7.405/2 - x) \times 1.2 \\ = 47.505 - 12.8304x$$

*Uncracked Section*

$$V_{co} = 0.67 b_w h \sqrt{f_t^2 + 0.8 f_{cp} f_t}$$

$$f_t = 0.24 \sqrt{f_{cu}} \\ = 0.24 \sqrt{50} \\ = 1.697$$

$$f_{cp} = \frac{f_{pe} \times A_s}{A_i} \\ = 937 \times 365 / 125978 \\ = 2.819$$

$$f_{cp} = \frac{x}{l_p} (2 - x/l_p) f_{cp} \\ = \frac{x}{523} (2 - \frac{x}{523}) (2.819) \\ = \frac{2.819x}{523} (2 - \frac{x}{523}) \\ = 0.01078x - 1.0306 \times 10^{-5} x^2$$

$$V_{co} = 0.67 \times 227 \times 205 \times \sqrt{1.697^2 + 0.8(0.01078x - 1.0306 \times 10^{-5} x^2)} (1.697) \\ = 31178.45 x \sqrt{2.88 + 0.014635x - 1.4 \times 10^{-5} x^2}$$

**Cracked Section**

$$V_{cr} = (1 - 0.55 f_{pe} / f_{pu}) V_c b_w d + M_o V / M \geq 0.1 b_w d \sqrt{f_{cu}}$$

$$= 35 \text{ kN}$$

$$M_o = 0.8 \frac{f_b I}{Y_b} = 0.8 \times f_b \times Z_b$$

$$= 0.8 \times 5.017 \times 6.952 \times 10^6$$

$$= 27.9 \text{ kNm}$$

$$f_{pe} / f_{cu} = 0.523 < 0.6$$

$$\frac{100 A_s}{b_w \times d} = \frac{100 \times 365}{227 \times 218}$$

$$= 0.7376$$

$$V_c = 0.6992 \times \sqrt[3]{f_{cu} / 30}$$

$$= 0.829 \text{ N/mm}^2$$

$$V_{cr} = (1 - 0.55 \times 0.523) \times 0.826 \times 227 \times 218 + 27.9 V / M$$

$$= 29.22 + 27.9 V / M$$

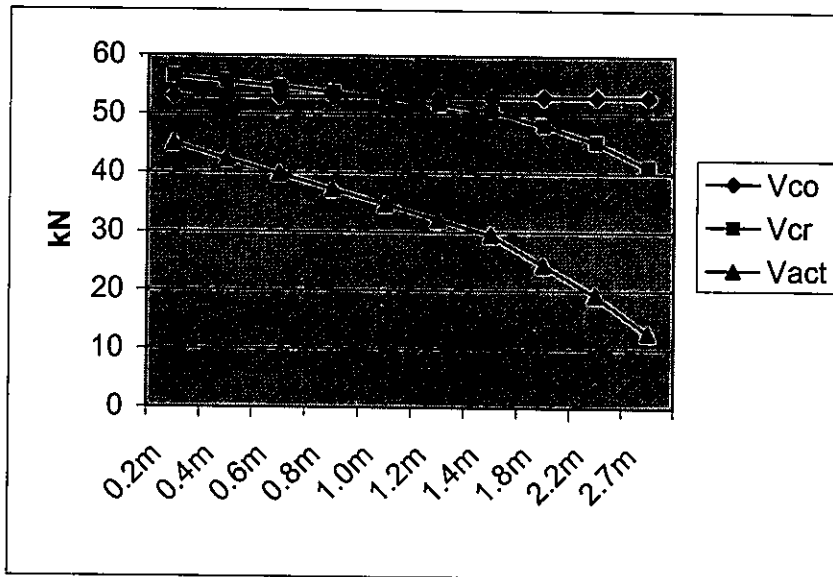
$$M = \frac{1.2 W \lambda_x}{2} \left(1 - \frac{x}{\lambda}\right)$$

$$= \frac{1.2(1.4 \times 4.78 + 2.5 \times 1.6) \times 7.405 x}{2} \left(1 - \frac{x}{7.405}\right)$$

$$= 47.5 \times \left(1 - \frac{x}{7.405}\right)$$



| $x$ | $V_{act}$ | $M$   | $V/M$ | $V_{co}$ | $V_{cr}$ | $V_u$ |
|-----|-----------|-------|-------|----------|----------|-------|
| 0.2 | 45.00     | 46.22 | 0.974 | 52.94    | 56.40    | 52.94 |
| 0.4 | 42.37     | 44.93 | 0.943 | 52.97    | 55.53    | 52.97 |
| 0.6 | 39.80     | 43.65 | 0.912 | 52.97    | 54.66    | 52.97 |
| 0.8 | 37.24     | 42.37 | 0.879 | 52.97    | 53.74    | 52.97 |
| 1.0 | 34.68     | 41.09 | 0.844 | 52.97    | 52.77    | 52.77 |
| 1.2 | 32.11     | 39.80 | 0.807 | 52.97    | 51.74    | 51.74 |
| 1.4 | 29.54     | 38.52 | 0.767 | 52.97    | 50.62    | 50.62 |
| 1.8 | 24.41     | 35.95 | 0.679 | 52.97    | 48.16    | 48.16 |
| 2.2 | 19.28     | 33.39 | 0.577 | 52.97    | 45.32    | 45.32 |
| 2.7 | 12.86     | 30.18 | 0.426 | 52.97    | 41.10    | 41.10 |



vii) SLS Deflection/Flatness of Floor at Transfer

At Transfer

From Prestressing Force

$$\begin{aligned}
 D_{17} &= \frac{P_i \times A_s \times e_o \times \lambda^2}{8 \times E_{35} \times I_i} \\
 &= \frac{1000 \times 365 \times 57 \times 7.405^2 \times 10^6}{8 \times 27000 \times 6.743 \times 10^8} \\
 &= -7.83 \text{ mm}
 \end{aligned}$$

From Self Weight

$$\begin{aligned}
 D_{27} &= \frac{5 \times s/w \times f^4 \times 0.2}{384 \times E_{35} \times I_i} \\
 &= \frac{5 \times 2.6 \times 7.405^4 \times 1.2 \times 10^{12}}{384 \times 27000 \times 6.743 \times 10^8} \\
 &= 6.7 \text{ mm}
 \end{aligned}$$

$$D_T = 6.7 - 7.83 = -1.13 \text{ mm upwards}$$

At 28 days (erection see variac manual  $A_p \beta$ )

RH 70%

$$\Rightarrow D_1 = 1.8 \times (-7.83) = -14.094 \text{ mm}$$

$$D_2 = 1.8 \times 6.7 = \frac{12.06 \text{ mm}}{}$$

$$= -2.034 \text{ mm} = D_c$$

At Final

$$D_1 = D_{17} \times 2.45 = 2.45 \times (-7.83) = -19.1835 \text{ mm}$$

$$D_2 = D_{27} \times 2.7 = 2.7 \times 6.7 = \frac{18.09 \text{ mm}}{-1.0935 \text{ mm}}$$

## Topping Concrete

$$\begin{aligned}
 D_3 &= \frac{5 \times W_t \times 1.2 \times \lambda^4}{384 \times E_{50} \times I_i} \\
 &= \frac{5 \times 1.68 \times 1.2 \times 7.405^4 \times 10^{12}}{384 \times 30000 \times 6.743 \times 10^8} \\
 &= 3.9 \text{ mm}
 \end{aligned}$$

## Dead Load (SDL)

$$\begin{aligned}
 D_4 &= \frac{5 \times (W_b + W_s) \times 1.2 \times \lambda^4 \times 3}{384 \times E_{50} \times I_i} \\
 &= \frac{5 \times 0.5 \times 1.2 \times 7.405^4 \times 10^{12} \times 3}{384 \times 30000 \times 6.743 \times 10^8} \\
 &= 3.48 \text{ mm (long term)}
 \end{aligned}$$

## Live Load

$$\begin{aligned}
 D_5 &= \frac{5 \times W_{LL} \times 1.2 \times \lambda^4 \times 0.7}{384 \times E_{50} \times I_i} \\
 &= \frac{5 \times 2.5 \times 1.2 \times 7.405^4 \times 0.7 \times 10^2}{384 \times 30000 \times 6.743 \times 10^8} \\
 &= 4.06 \text{ mm (short term)}
 \end{aligned}$$

$$\begin{aligned}
 D_6 &= \frac{5 \times W_{LL} \times 1.2 \times \lambda^4 \times 0.3 \times 3}{384 \times E_{50} \times I_i} \\
 &= \frac{5 \times 2.5 \times 1.2 \times 7.405^4 \times 0.3 \times 3}{384 \times 30000 \times 6.743 \times 10^8} \\
 &= 5.226 \text{ mm (long term)}
 \end{aligned}$$

## Total flatness of the floor

$$\begin{aligned}
 D_T &= D_2 - D_1 + D_4 - D_3 + D_5 + D_6 + D_c \\
 &= 9.8065 \text{ mm}
 \end{aligned}$$

## References

- [1] Elliot, K.S. (1996), Multi-storey Precast Concrete Framed Structures, Blackwell Science Limited
- [2] Construction Industry Development Board (1999), Structural Precast Concrete Handbook, Singapore
- [3] Design of Multi-storey Precast Concrete Structures (1983), Thomas Telford Ltd., London
- [4] British Standards Institutions (1985), The Structural Use of Concrete, BSI, BS 8110, London
- [5] Federation Internationale de la Precontrainte (1994), FIP Recommendations, Precast Prestressed Hollow Cored Floors, FIP Commissions on Prefabrication, Thomas Telford, London

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