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DESIGN GUIDE ON PRECAST CONCRETE FRAME WITH MOMENT CONNECTION

Descriptors: precast concrete beams, precast concrete columns, moment connections, ties, diaphragm action, hollow core slabs, precast planks, precast frames

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DESIGN GUIDE ON PRECAST CONCRETE FRAME WITH MOMENT CONNECTION
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Committee representation

The Industrialised Building System (IBS) Technical Development Committee under whose authority this Construction Industry Standard (CIS) was developed, comprises representatives from the following organisations:

Association of Consulting Engineers Malaysia
Construction Industry Development Board Malaysia
Hume Industries (M) Berhad
The Institution of Engineers, Malaysia
Jabatan Kerja Raya Malaysia
Master Builders Association of Malaysia
MTD ACPI Engineering Berhad
Pertubuhan Akitek Malaysia
Setia Precast Sdn Bhd
Star Shine Marketing Sdn Bhd
Sunway Precast Industry Sdn Bhd
UAC Steel System Sdn Bhd
Universiti Teknologi Malaysia
FOREWORD

This design guide was developed by the IBS (Industrialised Building System) Standard Committee with the assistance of the Construction Industry Development Board Malaysia (CIDB) and Universiti Teknologi Malaysia.

The objective of this design guide is to disseminate knowledge and subsequently, to promote the use of precast concrete frames through education. Hopefully, by having such design information will encourage more professionals to participate in this prefabricated component based construction.

The design guide covers:
• the overview and design principles of precast concrete frames
• analysis of precast concrete frames with moment connections
• design examples of precast columns, beams, connections and ties.

The design information, even though brief, will be useful to university students, young engineers and also professionals to understand the basic concept of designing precast concrete framed structures with moment connections.

Finally, the use of this design guide is voluntary and compliance with this design guide does not confer immunity from legal obligations.
PRECAST CONCRETE FRAME WITH MOMENT CONNECTION
DESIGN GUIDE

1. Design Principles

1.1 Introduction

This publication provides basic information on the design of multi-storey storey precast concrete framed buildings with moment or rigid connections.

This precast concrete framed building system is classified by the Construction Industry Development Board (CIDB) as one of the industrialised building systems (IBS) available in Malaysia. The precast components such as beams, columns and slabs are fabricated or "manufactured" off-site using machines and formworks and delivered to site for assemblage and erection to build a complete structure.

1.2 Frame System

This framed system is suitable for buildings that need a high degree of flexibility in terms of larger clear distances between column-to-column. The skeletal framed system gives more open space and greater freedom for designing the floor areas. The system can be used for constructing buildings that offer a certain luxury such as office buildings, school buildings, hospitals, university buildings, commercial buildings and car parks. Currently in Malaysia, the skeletal framed system becomes the dominant structural form of building construction.

A precast concrete skeletal framed building consists of main structural components of beams, slabs, and columns. In order to join the loose precast beam, column and slab components to become a framed building, structural connections are required.

1.3 Connection System

Connections form the important part of precast concrete construction. The success of precast concrete construction not only depends on the structural performance of precast beams and columns but also depends on the simplicity during erection and effectiveness of the connections. The connection types employed will not only affect the construction cost and speed but also affect the redistribution of forces from beams to columns and subsequently will affect the types of structural system.

From the structural point of view, precast concrete connections can be classified into three categories namely pinned, semi-rigid and rigid connections. However, for simplicity in analysis and design of frames, the connections can be classified as either pinned or rigid.

Rigid connections, also known as moment connections, are usually preferred due to their ability to transmit beam end moments continuously to columns.

In the United States for example, the use of precast concrete frames with rigid connections in seismic regions has gained acceptance from industries due to the ability of the overall frame systems to resist lateral loads effectively [1].
1.3.1 Beam-to-Column Connections

In this publication, only precast concrete frames with rigid connections are discussed. Typical examples of rigid beam-to-column connections used in precast concrete frames are shown in Figure 1. To determine whether a connection is pinned or rigid, an experimental testing has to be conducted to obtain the relationship between moment and rotation of the connection.

![Typical Connection Detail]

**Figure 1. Rigid beam-to-column connections.**

Numerous full-scale tests of precast concrete beam-to-column connections were conducted at the Faculty of Civil Engineering, Universiti Teknologi Malaysia, Skudai, [3]. This research, funded by the Construction Research Institute of Malaysia (CREAM-OIDB), was conducted to investigate the performance of existing and new precast concrete beam-to-column connections.

The research has contributed a significant understanding to the behaviour and performance of precast concrete beam-to-column connections. Figure 2 shows a precast concrete T-subframe for the testing. From this study, precast concrete beam-to-column connections, with reinforcement bars detail similar to monolithic construction, such that with adequate anchorage and then cast with in-situ concrete joint such as shown in Figure 2, are likely to behave close to the monolithic conventional reinforced concrete connection. Figures 3 and 4 show the testing of precast connections as part of T and H subframes respectively, conducted in the structure laboratory to determine the characteristics and performance of the precast connections.

In practice, precast concrete connections with wet joints are widely used in Malaysia. As for example, Figure 5 shows actual precast concrete beam-to-column connections using cast in-situ concrete joint with appropriate reinforcement bars to achieve monolithic connection.
Figure 2. Precast concrete beam-to-column connection using cast-in place concrete [3].

Figure 3. Testing of precast beam-to-column moment connection [3].
Figure 4. Failure mode of H-subframe with precast concrete connection [3].

Figure 5. Construction of precast concrete building with cast-in place connections.
1.3.2 Column-to-Base Connections

Connections between column-to-base can be performed in three different methods such as follows:
1. Grouted pocket
2. Steel base plate
3. Grouted sleeve (see Figure 6)

From the three types of connection above, the most popular connection used in Malaysia is the grouted sleeve shown in Figure 6, as it is the most economical. For this connection type, high tensile steel starter bars are left protruding from the foundation. After that, a precast column with vertical sleeves at the end is inserted into the starter bars. The sleeves are then filled in with grout to provide a monolithic connection. This connection type is considered rigid and has the ability to provide moment resistance at the connection between column end to footing.

![Figure 6. Grouted sleeve column-to-base connection.](image)

1.4 Floor System

The floor system in a precast framed structure consists of horizontal components of beams, slabs and connections. The success of precast concrete construction, with regards to speed and economical aspects, depends on the practical arrangement of beams and slabs in the floor system. As for example, Figure 7 shows one of the practical floor layouts that is widely used in the precast construction. In this layout, precast concrete components such as precast beams are spanning in the longitudinal direction of the building and supported by precast columns; whereas the precast slabs are spanning in the transverse direction and supported by the precast beams.

The precast slabs normally used for the floor system are either precast hollow core slabs or precast planks.
1.4.1 Precast Hollow Core Slab

Precast concrete hollow core slabs have many excellent characteristics, such as light in weight due to the hollows, less cost of steel as only tendons are used, span longer due to the prestressed and also more importantly possess monolithic characteristic as good as conventional slab.

Precast concrete hollow core slabs are suitable for long spanning structures, in the range of 4m to 12m. With the long spanning capabilities, precast hollow core slabs are suitable for the construction of office buildings that require spacious areas for flexibility interior design.

In order to obtain a monolithic floor slab, the loose precast hollow core slabs must be connected together effectively. The monolithic aspect can be achieved at a relatively cheap cost using shear key along the edges of the slab. The shear key can be easily obtained if the slab edge is designed with appropriate profile, see Figure 8. The profiled edges, when connected together with grout, ensure adequate locking of the loose components and hence adequate transfer of horizontal and vertical shear stresses between adjacent slabs. Eventually, the loose precast slabs when connected properly will act as one unit of monolithic slab.

![Resistance to horizontal shear](Figure 8. Edge profile of precast hollow core slab)
Precast concrete hollow core slabs can be constructed as either topped or untopped. In untopped precast slab system, the loose precast slab components are tied together using steel ties in order to form a monolithic slab (see Figure 9).

In the case of topped precast slab, cast in-situ concrete topping of about 50mm-75mm is added on top of the precast hollow core slabs together with steel mesh fabric to form a monolithic structural topping (see Figure 10).

The use of topped precast slab system enables the floor system to
1. provide resistance to water leakage, in particular, in between the hollow core slab joints
2. provide good level of floor surface
3. carry moving loads and heavy concentrated loads.

Figure 9. Precast hollow core slabs without concrete topping.

Figure 10. Precast hollow core slabs with cast in-situ concrete topping and steel mesh.
1.4.2 Precast Plank

The floor system can also be constructed using precast planks, also known as half slabs or semi-precast slabs.

One unit of precast plank normally consists of steel lattice trusses and bottom steel mesh cast in the factory (see Figure 11(a)) and subsequently the top steel mesh is cast with in-situ concrete topping on site. During construction, the planks act as a formwork for concreting the topping (see Figure 11(b)).

The complete floor system uses a combination of precast concrete planks, steel mesh and cast in-situ concrete topping. The use of cast in-situ concrete topping with top steel mesh, effectively ties all the loose precast planks together provides a monolithic slab with enhance stiffness and rigidity.

In terms of spanning performance, precast planks are suitable for short spanning structures in the range of 2m to 5m. Hence, this floor system is suitably employed in residential dwellings where planks with spans of less than 5m are sufficient.

Figure 11. Precast concrete plank.
1.5 Structural Integrity

A precast concrete structure is said to have the structural integrity if it is able to withstand localised damage or failure of one component without causing progressive collapse to the neighbouring components, see Figure 12. This may be achieved by tying the loose precast components together using steel ties. So that in the event of an accident, the ties in between the components have the ability to transfer loads to un-failed members through alternate load paths.

Figure 12. Floor ties transfer loads to un-failed members through alternate load paths.

BS 8110 clause 3.12.3 specifies the guidelines for designing ties to ensure that the overall structure is tied together horizontally and vertically. The various types of ties in untopped and topped precast framed system are shown in Figure 13 and Figure 14 respectively, in which Ties Nos. 1, 2, 3, 4, 5 and 6 represent the horizontal ties, whereas Tie No.7 represents the vertical tie. In construction, these ties should be robust and durable, therefore the connection of ties should be filled with good quality grout or well compacted cast in-situ concrete, and normally the ties use high strength steel reinforcement bars.
(a) Different types of tie in a precast concrete frame

(b) Ties in chipped-out hollow core slabs

Figure 13. Method 1: Horizontal and vertical ties in untopped precast concrete framed building.
(a). Different types of tie in a precast concrete frame

(b). Ties in grouted keys of hollow core slabs

Figure 14. Method 2: Horizontal and vertical ties in topped precast concrete framed building.
Tie No.1 - Internal floor ties
The function of these ties is to provide continuity of tension between ends of precast slabs. To tie the precast hollow core slabs, the ends of the hollow core at the ends is cut, then a high strength steel tie is incorporated into the precast cores and finally filled with cast in-situ concrete, see Figure 15.

Figure 15. Ends of precast slabs are tied to provide continuity.

Tie No.2 - Internal floor ties.
These ties are used to joint longitudinal side of slabs to provide connecting link between edges of precast units so that the slab can act as a rigid floor diaphragm. Joints between loose panels are tied together using shear keys and insitu-fill to create a robust joint with minimal finishing required, see Figure 16.

Figure 16. Placement of internal floor ties in topped precast framed system.

Tie No.3 – Perimeter floor ties
These ties enable the slab ends to be tied together to edge beams, see Figures 17(a), (b) and (c).
Ties Nos.4 & 5 - Peripheral ties
These peripheral ties are provided around the perimeter of the building to create a continuous tie arrangement. Possible details of peripheral beam ties if located along edge beam are shown in Figures 17(d) and (e). In addition to ties, concrete infill will be cast in between the edge beam and slab.
(a). Cross section for topped precast concrete system

(b). End of precast slab is tied to edge-beam

(c). Ties for slabs running parallel to the edge beam
Figure 17. Floor and peripheral ties.

Ties Nos. 6 & 7
Ties Nos. 6 and 7 are actually connections between beam-to-column and connections between column-to-column respectively.
1.6 Diaphragm Action

As the precast slabs are manufactured individually, the loose components of precast slabs should be connected together to enable the floor slabs to act as a rigid horizontal diaphragm. With the availability of slab diaphragm, lateral loads acting on a building can be transferred safely to the lateral stability system through the bending action of the slab diaphragm (Figure 18).

![Diagram of diaphragm action](image)

Figure 18. Transfer of lateral loads to lateral stability system through floor diaphragm action.

For untopped precast slabs, the rigid floor diaphragm of the precast floors can be achieved by connecting the precast slabs by means of welding, or tying. Figure 19(a) shows the tying method, using steel bars, to tie ends of untopped precast slabs. In addition, the monolithic aspect of untopped precast slabs, can be enhanced further by enclosing the untopped precast slabs with a boundary (peripheral) tension tie (see Section 1.5, with reference to Ties No.4 and No.5). For topped precast slabs, the rigid diaphragm can be achieved by having reinforced structural topping consisting of cast in situ concrete with steel mesh, see Figure 19(b). The mechanism of shear transfer and diaphragm action is as shown in Figure 20.

![Diagram of topping](image)

(a). Ties for untopped precast slabs
Cast in-situ concrete topping

(b). Cast in-situ concrete topping with steel mesh

Figure 19. Precast hollow core slabs and ties.

Figure 20. Shear Transfer and Diaphragm Action.
1.7 Lateral Stability System

The unbraced frame is designed to carry both vertical and lateral loads.

Under the action of vertical loads, the frame response is shown in Figure 21(a) in which the degree of bending in the beams and columns depends on the stiffness and strength of beam-to-column connections.

Under the action of lateral loads (see Figure 21(b)), the frame sways and the lateral stability of the overall unbraced frame relies on the degree of bending in the beams and columns that eventually relies on the strength and stiffness of beam-to-column connections.

Under the action of both vertical and gravity loads, the bending action of beams and columns that provides resistance to both the vertical and lateral loads is shown in Figure 21(c) and Figure 22.

In continuous frame, moment connections between beam-to-column must have the ability to develop sufficient strength to resist the applied loads and must have the ability to provide sufficient stiffness to limit the lateral deflection of the global frame. Clause 3.2.1.3.1 BS8110 specifies that where the global frame provides the lateral stability, sway should be considered in the analysis.

![Figure 21. Response of moment frame due to: (a) gravity load only (b) lateral load only (c) gravity and lateral loads](image)

![Figure 22. Deformed shape of rigid frame](image)
1.8 Scope of Publication

This publication gives introduction aspects on the analysis and design of precast concrete frames with moment connections. The examples given in the following chapters consider the aspects of analysis and design at the ultimate limit state only, while the aspects of handling and erection are not included in designing the components.

In this design example, a three-storey and one-bay unbraced frame shown in Figure 23 is considered for the analysis and design calculations. The frame is unbraced, as no independent lateral stability systems such as shear wall, core or bracing is included in both directions of the system (see clause 3.8.1.5 BS 8110).

The frame has the following features:
- An orthogonal beam or cross beam layout. The layout of floor system consists of precast beams in both transverse and longitudinal directions of the building
- Rigid beam-to-column connections
- Rigid column-to-base connections
- Structural precast concrete slabs may be used with topped or untopped concrete
- Lateral stability system is provided by the frame bending action

![Diagram of unbraced precast concrete frame]

Figure 23. Unbraced precast concrete frame.
2. Analysis of frame with moment connections

2.1 Introduction

This section discusses the analysis of the three-storey and one-bay unbraced frame with rigid beam-to-column connections (see Figure 24).

![Rigid beam-to-column diagram](image)

**Figure 24. Transverse section of unbraced frame with rigid beam-to-column connections.**

2.2 Analysis of Frame

2.2.1 Load Cases

- The analysis of the overall frame needs to consider the following load cases:
  - Load case 1: 1.4Gk + 1.6Qk
  - Load case 2: 1.2Gk + 1.2Qk + 1.2Wk
  - Load case 3: 1.0Gk + 1.4Wk

2.2.2 Design Data

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Roof Level</th>
<th>1st and 2nd floor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load, G_k</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st and 2nd floor</td>
<td>= 5.0 kN/m²</td>
<td></td>
</tr>
<tr>
<td></td>
<td>= 6.0 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Imposed load, Q_k</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof level</td>
<td>= 1.5 kN/m²</td>
<td></td>
</tr>
<tr>
<td>1st and 2nd floor</td>
<td>= 3.0 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Wind load, W_k</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof level</td>
<td>= 5.4 kN</td>
<td></td>
</tr>
<tr>
<td>2nd floor</td>
<td>= 10.1 kN</td>
<td></td>
</tr>
<tr>
<td>1st floor</td>
<td>= 9.0 kN</td>
<td></td>
</tr>
</tbody>
</table>
Materials
Concrete \( f_{cm} = 40 \text{ N/mm}^2 \)
Reinforcement \( f_y = 460 \text{ N/mm}^2 \)

Member Properties
Columns 400mm x 400mm (square column)
Beams 400mm x 600mm (B x D)

Connection Types
Beam-to-column connection: Rigid
Column-to-base: Rigid

Floor System
The floor system consists of one way spanning precast floor slabs seated on transverse beams.

Lateral Stability System
Lateral stability of the building in transverse direction is provided by the bending action of the frame with the availability of rigid beam-to-column connections.

Plane Frame Model
The building can be modelled as a two-dimensional plane frame shown in Figure 25.

![Diagram of a plane frame model with dimensions and connections.]

Figure 25. Plane frame model

2.2.3 Analysis
Figures 26 to 28 show the three different load cases considered in the analysis.
Load Case 1: Gravity load

1.4G_k + 1.6Q_k

**Roof:**
Design load per unit area, q

\[ = 1.4G_k + 1.6Q_k \]
\[ = 1.4(5kN/m^2) + 1.6(1.5kN/m^2) \]
\[ = 9.4 \text{ kN/m}^2 \]

Design load per unit length

\[ = q \times \text{width} \]
\[ = 9.4 \text{kN/m}^2 \times 5 \text{m} \]
\[ = 47 \text{ kN/m} \]

1st floor and 2nd floor:
Design load per unit area, q

\[ = 1.4G_k + 1.6Q_k \]
\[ = 1.4(6kN/m^2) + 1.6(3.0kN/m^2) \]
\[ = 13.2 \text{kN/m}^2 \]

Design load per unit length

\[ = 13.2 \text{kN/m}^2 \times 5 \text{m} \]
\[ = 66 \text{ kN/m} \]

---

*Figure 26. Load case 1.*

---

Load Case 2: Gravity load + Wind load

1.2G_k + 1.2Q_k + 1.2W_k

**Roof:**
Design load per unit area, q

\[ = 1.2G_k + 1.2Q_k \]
\[ = 1.2(5kN/m^2) + 1.2(1.5kN/m^2) \]
\[ = 7.8 \text{kN/m}^2 \]

Design load per unit length

\[ = q \times \text{width} \]
\[ = 7.8 \text{kN/m}^2 \times 5 \text{m} \]
\[ = 39 \text{ kN/m} \]
1st floor and 2nd floor:
Design load, q
\[ = 1.2G_k + 1.2Q_k \]
\[ = 1.2(6kN/m^2) + 1.2(3.0kN/m^2) \]
\[ = 10.8 \text{kN/m}^2 \]

Design load per unit length
\[ = 10.8 \text{kN/m}^2 \times 5m \]
\[ = 54 \text{kN/m} \]

Design wind load
\[
\begin{align*}
\text{Roof,} & \quad 1.2W_k = 1.2(5.4kN) = 6.5 \text{kN} \\
\text{2nd floor,} & \quad 1.2W_k = 1.2(10.1kN) = 12.1 \text{kN} \\
\text{1st floor,} & \quad 1.2W_k = 1.2(9kN) = 10.8 \text{kN}
\end{align*}
\]

![Diagram](image)

Figure 27. Load case 2.

Load Case 3: Gravity load + Wind load
\[ 1.0G_k + 1.4W_k \]

Roof:
Design load per unit area, q
\[ = 1.0G_k \]
\[ = 1.0(5.0kN/m^2) \]
\[ = 5.0 \text{kN/m}^2 \]

Design load per unit length
\[ = 5.0kN/m^2 \times 5m \]
\[ = 25 \text{kN/m} \]

1st floor and 2nd floor:
Design load, q
\[ = 1.0G_k \]
\[ = 1.0(6kN/m^2) \]
\[ = 6.0 \text{kN/m}^2 \]

Design load per unit length
\[ = 6kN/m^2 \times 5m \]
\[ = 30 \text{kN/m} \]
Design wind load
Roof level, \[1.4W_k = 1.4(5.4\text{kN}) = 7.6\text{ kN}\]
2\textsuperscript{nd} floor, \[1.4W_k = 1.4(10.1\text{kN}) = 14.1\text{ kN}\]
1\textsuperscript{st} floor, \[1.4W_k = 1.4(9\text{kN}) = 12.6\text{ kN}\]

Figure 28. Load case 3.

Figures 29 to 40 show the analysis results as obtained from the first order elastic computer program. From these results, precast concrete components such as beams, columns and connections are designed to carry the respective loads.

Figure 29. Bending moment diagram due to load case 1.
Figure 30. Bending moment diagram due to load case 2.

Figure 31. Bending moment diagram due to load case 3.
Figure 32. Shear force diagram due to load case 1.

Figure 33. Shear force diagram due to load case 2.
Figure 34. Shear force diagram due to load case 3.

Figure 35. Axial column loads due to Load case 1.
Figure 36. Axial column loads due to Load case 2.

Figure 37. Axial column loads due to load case 3.
Figure 38. Frame deflection due to load case 1.

Figure 39. Frame deflection due to load case 2.
<table>
<thead>
<tr>
<th>Node</th>
<th>$U_x$ (mm)</th>
<th>$U_y$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7,8</td>
<td>9.0</td>
<td>-0.9</td>
</tr>
<tr>
<td>5,6</td>
<td>8.0</td>
<td>-0.8</td>
</tr>
<tr>
<td>3,4</td>
<td>5.8</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

**DEFLECTED SHAPE**
Load case 3
1.0$G_x + 1.4W_k$

**Figure 40.** Frame deflection due to load case 3.
3. Design of Column

3.1 Design Procedures

(a). Moment due to ultimate lateral and vertical loads
- No eccentric moments because of rigid beam-to-column connections.
- The analyses from Chapter 2 give the results of axial loads and maximum end moments in the columns.

(b). Moment due to second order deflection
- Use Clause 3.8.3 BS 8110.
- In this clause, the additional moment due to the axial load and additional deflection (i.e. second order P-Δ effect) is calculated.
- All vertical loads from the beams are transferred to the columns in terms of axial loads. The total axial load in the column is denoted as \( N \).
- Additional column moment due to additional deflection is, \( M_{add} = N \cdot a_u \)

(c). Moment combination
- The overturning moment due to lateral and vertical loads, and additional moment due to second order effect are combined to obtain the total design moment.
- The column section is then designed to withstand the design ultimate axial load, plus the total design moment.
- Design of unbraced columns is specified in section 3.8.3.7 BS 8110.
- Effective length of unbraced column is in accordance with section 3.8.1.6 BS 8110.
3.2 Column Design Example

Consider column 2, see Figure 41
Clause 3.8.1.3 BS 8110

For unbraced column:

if both \( \frac{I_x}{h} < 10 \) and \( \frac{I_y}{b} < 10 \) \} short column.

\[ b = 400\text{mm}, \quad h = 400\text{mm}, \]

Clear height between end restraints, \( l_o = 5250\text{mm} - 600\text{mm} = 4650\text{mm} \)

About x-x axis:
Table 3.20: Condition of end restraint:
End condition at top: \( \text{Condition 1- } h_{\text{beam}}=600\text{m} > h_{\text{column}}=400\text{mm} \} \}
End condition at bottom: \( \text{Condition 1-Connected to foundation} \} \beta = 1.2 \}
Therefore \( l_x = \beta l_o = 1.2(4650) = 5580\text{mm} \).

About y-y axis:
Table 3.20: Condition of end restraint:
End condition at top: \( \text{Condition 1-edge beam connected to main beam } \} \}
End condition at bottom: \( \text{Condition 1-connected to foundation} \} \beta = 1.2 \}
Therefore \( l_y = \beta l_o = 1.2(4650) = 5580\text{mm} \).

Figure 41. Column end conditions.
\[ \frac{l_{ex}}{h} = \frac{5580}{400} = 13.95 > 10 \quad \text{and} \quad \frac{l_{ev}}{b} = \frac{5580}{400} = 13.95 > 10 \] \) Slender column.

Design column 2 as slender and unbraced.

**Load case 1**
Clause 3.8.3
Additional moment due to second order effect
\[ M_{add} = N. \ a_u \]
\[ a_u = \beta_a K h \]
\[ \beta_a = \frac{1}{2000} \left( \frac{l_e}{b'} \right)^2 = \frac{1}{2000} (13.95)^2 = 0.097 \]
\[ a_u = 0.097 \times 1 \times 400 = 38.8 \text{mm} \]

\[ P = 716 \text{ kN} \]
\[ M_{add} = 716 \text{ kN} \times 38.8 \text{mm} \times 10^{-3} = 27.8 \text{ kNm} \]
\[ M = 106 + 27.8 = 129.6 \text{ kNm} \]

**Load case 2**
Clause 3.8.3
Additional moment due to second order effect
\[ M_{add} = N. \ a_u \]
\[ a_u = \beta_a K h \]
\[ \beta_a = \frac{1}{2000} \left( \frac{l_e}{b'} \right)^2 = \frac{1}{2000} (13.95)^2 = 0.097 \]
\[ a_u = 0.097 \times 1 \times 400 = 38.8 \text{mm} \]

\[ P = 608 \text{ kN} \]
\[ M_{add} = 608 \text{ kN} \times 38.8 \text{mm} \times 10^{-3} = 23.6 \text{ kNm} \]
\[ M = 106 + 23.6 = 129.6 \text{ kNm} \]
Design for load case 1

\[
\frac{M}{bh^2} = \frac{114.8 \times 10^6 \text{Nmm}}{400 \text{mm} \times (400)^2 \text{mm}^2} = 1.79 \text{N/mm}^2
\]

\[
\frac{N}{bh} = \frac{716 \times 10^3 \text{N}}{400 \text{mm} \times 400 \text{mm}} = 4.48 \text{N/mm}^2
\]

\[
d = h - \text{cover} - \phi_{bar}/2 = 400 - 30 - 20/2 = 360 \text{mm},
\]

\[
\frac{d}{h} = \frac{360}{400} = 0.9, f_{cu} = 40 \text{N/mm}^2, f_y = 460 \text{N/mm}^2 \} \text{ Chart No. 39}
\]

\[
\frac{100 A_{sc}}{bh} = 0.4
\]

\[
A_{sc} = \frac{0.4bh}{100} = \frac{0.4 \times 400 \text{mm} \times 400 \text{mm}}{100} = 640 \text{mm}^2
\]

Require 4 No. T16 (A_{sc} provided) = 805 \text{mm}^2

Design for load case 2

\[
\frac{M}{bh^2} = \frac{129.6 \times 10^6 \text{Nmm}}{400 \text{mm} \times (400)^2 \text{mm}^2} = 2.03 \text{N/mm}^2
\]

\[
\frac{N}{bh} = \frac{608 \times 10^3 \text{N}}{400 \text{mm} \times 400 \text{mm}} = 3.8 \text{N/mm}^2
\]

\[
d = h - \text{cover} - \phi_{bar}/2 = 400 - 30 - 20/2 = 360 \text{mm},
\]

\[
\frac{d}{h} = \frac{360}{400} = 0.9, f_{cu} = 40 \text{N/mm}^2, f_y = 460 \text{N/mm}^2 \} \text{ Chart No. 39}
\]

\[
\frac{100 A_{sc}}{bh} = 0.4
\]

\[
A_{sc} = \frac{0.4bh}{100} = \frac{0.4 \times 400 \text{mm} \times 400 \text{mm}}{100} = 640 \text{mm}^2
\]

Provide 4 No. T20 (A_{sc} provided) = 1257 \text{mm}^2
4. Design of Connection

4.1 Design of Column-to-Base Connection

Grouted Sleeve Connection

Design of starter bars
Design the connection based on the most severe load case, i.e. largest moment from load case 2 and largest axial load from load case 1 (see Figure 42).

\[ M = 77.7 \text{ kNm} \]
\[ N = 718 \text{ kN} \]

Use \( f_{cu} = 40 \text{ N/mm}^2 \)
\( F_y = 460 \text{ N/mm}^2 \)

Cover to link = 30mm

\[ \frac{N}{bh} = \frac{716 \times 10^3 N}{400 \text{mm} \times 400 \text{mm}} = 4.48 \text{ N/mm}^2 \]

\[ \frac{M}{bh^2} = \frac{77.7 \times 10^6 N \text{mm}}{400 \text{mm} \times (400 \text{mm})^2} = 1.21 \text{ N/mm}^2 \]

Effective depth, \( d = h - \text{cover} - \Phi_{\text{bar}/2} = 400 - 30 - (20/2) = 360 \text{mm} \)

\[ \frac{d}{h} = \frac{360 \text{mm}}{400 \text{mm}} = 0.9 \]

Use chart No. 39

\[ \frac{100A_s}{bh} = 0.4 \]

\[ A_s = \frac{0.4(400 \text{mm})(400 \text{mm})}{100} = 640 \text{mm}^2 \]

Provide 4T16 (As provided = 805 mm\(^2\))

Figure 42. Bending moment and axial load are transferred from column to base.
Check the anchorage length of the starter bar.

Compressive stress at column bottom end (section 1-1)

\[ \sigma = \frac{N}{A} = \frac{716 \times 10^3 N}{400 \text{mm} \times 400 \text{mm}} = 4.48 \text{N/mm}^2 \]

Bending stress at section 1-1, see Figure 43

\[ \sigma = \frac{My}{I} = \frac{6M}{BD^2} = \frac{6(77.7 \times 10^6) N\text{mm}}{400 \text{mm} \times (400 \text{mm})^2} = 7.28 \text{N/mm}^2 \]

Bending stress is greater than compressive stress, hence starter bars are in tension. From Table 3.27 BS 8110,

Tension bond length = 35\#bar = 35 (16\text{mm}) = 560\text{mm}

Provide anchorage length, L = 600\text{mm} for all 4T16 starter bars.

Provide holes for starter bars of 50\text{mm} diameter, see Figure 43.

Figure 43. Column-to-base connection using grouted sleeve method.

4.2 Design of Beam-to-Column Connection

The connection between beam-to-column must be capable to provide continuity for shear and moment transfer. In this design example, the mechanism of force transfer is as follows:

(i). the beam end reaction will be transferred to the column through the corbel
(ii). the beam end moment will be transferred to the column by the top reinforcement bars that are anchored to the column using cast in-situ concrete.

Design of Corbel

\[ M = 288 \text{kNm} \]

Reaction, \( R = 264 \text{ kN} \)
Shear check

\[ V < \frac{V}{0.8\sqrt{f_{cu}} \times bd} \]

\[ d > \frac{V}{0.8\sqrt{f_{cu}} \times b} = \frac{264 \times 10^3 N}{0.8\sqrt{40 \text{ N/mm}^2} \times (400 \text{mm})} = 130.4 \text{mm} \]

Provide, \( d = 200 \text{mm} \)

\( h = d + \text{cover} + \frac{d}{2} = 200 + 25 + 10/2 = 230 \text{mm} \)

\( a_v = 80 \text{mm} < 0.6d = 200 \text{mm} \) \} Shallow Corbel

where \( a_v \) is the lever arm distance to shear force (see Figure 44)

\[ a_v = 80 \text{mm} \]

\[ R = 297 \text{kN} \]

\[ d = 200 \text{mm} \]

Figure 44. Corbel definitions.

\[ X = 0.5d = 0.5(200 \text{mm}) = 100 \text{mm} \]

\[ \beta = \tan^{-1} \left( \frac{d - 0.5X}{a_v} \right) = \tan^{-1} \left( \frac{200 \text{mm} - 0.5(100 \text{mm})}{80 \text{mm}} \right) = 61.9^\circ \]

Tension force in reinforcement bar,

\[ F_t = \frac{R}{\tan \beta} = \frac{264 \text{kN}}{\tan 61.9^\circ} = 141 \text{kN} \]

Tension reinforcement

\[ A_{sh} = \frac{F_t}{0.95 f_y} = \frac{141 \times 10^3 \text{N}}{0.95(460 \text{ N/mm}^2)} = 322.7 \text{mm}^2 \]
Horizontal friction force, \( F_f = \mu R = 0.4(264kN) = 105.6 \text{kN} \)

\[
A_s = \frac{F_f}{0.95f_y'} = \frac{105.6 \times 10^3 N}{0.95(460N/mm^2)} = 241.6 mm^2
\]

Total area of tension reinforcement required, \( A_{st} = A_{s1} + A_{s2} = 322.7 + 241.6 = 564.3 \text{ mm}^2 \)

Min reinforcement, \( A_{s\text{min}} = \frac{0.4bh}{100} = \frac{0.4(400mm)(230mm)}{100} = 368 mm^2 \)

\( A_{st} > A_{s\text{min}}, \text{OK} \)

Therefore, provide 4T16 ( \( A_{s\text{prov}} = 805 \text{ mm}^2 \))

**Design of horizontal stirrup (shear reinforcement)**

Shear stress, \( \nu = \frac{V}{bd} = \frac{264 \times 10^3}{400 \times 200} = 3.3 N/mm^2 \)

\[0.8\sqrt{f_{cu}} = 0.8\sqrt{40} = 5.06 N/mm^2\]

\( \nu = 3.3 N/mm^2 \quad < \quad 0.8\sqrt{f_{cu}} = 5.06 N/mm^2, \text{OK} \)

\[
\frac{100As}{bd} = \frac{100(805mm^2)}{(400mm)(200mm)} = 1.01
\]

Table 3.8 BS 8110
\[
\frac{100As}{bd} = 1.01, \quad d = 200mm \quad \Rightarrow \quad \nu_c = 0.75
\]

Since \( f_{cu} = 40 \text{N/mm}^2 \quad > \quad 25 \text{N/mm}^2 \), \( \nu_c \) should be multiplied by \( \left( \frac{f_{cu}}{25} \right)^{1/3} \)

\[

\nu_c \left( \frac{f_{cu}}{25} \right)^{1/3} = (0.75) \left( \frac{40}{25} \right)^{1/3} = 0.88 N/mm^2
\]

Enhancement factor, \( \frac{2d}{a_s} = \frac{2(200mm)}{(80mm)} = 5 \)

\( \nu_c = 5 \times 0.88 = 4.4 N/mm^2 \quad > \quad \nu = 3.3 N/mm^2 \)

5.2.7.2.3 BS8110

Provide minimum shear reinforcement in the upper two third of the effective depth of corbel
\[ A_{sv} = \frac{0.4 \sigma_v S_v}{0.95 f_{yw}} = \frac{0.4(400\text{mm})(80\text{mm})}{0.95(250\text{N/mm}^2)} = 53.9\text{mm}^2 \]

Or \( A_{sv} = 0.5 \ A_{sl} = 161.4\text{ mm}^2 \)

Therefore minimum \( A_{sv} = 161.4\text{ mm}^2 \)

Adopt 2R8 links \( A_{sv} = 201\text{mm}^2 \) (for two legs) at 50mm spacing.

Figure 45 shows the arrangement of the reinforcement bars in corbel.

![Diagram of reinforcement bars in corbel](image)

**Figure 45.** Reinforcement bars required in the corbel.

**Design of anchorage length of beam reinforcement bars**

Maximum moment, \( M = 288 \text{ kNm} \)

\[ M = T 	imes z = 288 \text{ kNm} \]

Where \( z = \text{lever arm} = \text{distance between top and bottom reinforcement} = 600\text{mm} - 2(40\text{mm}) - 2(10\text{mm}) - 2(20\text{mm}/2) = 480\text{mm} \)

\[ T = \frac{M}{z} = \frac{288 \times 10^3 \text{kNm}}{480\text{mm}} = 600\text{kN} \]

Reinforcement bars provided: 4T25

Force for each bar, \( F_{sv} = \frac{600\text{kN}}{4} = 150\text{kN} \)

Table 3.26:

Bar type 2, in tension \( \beta = 0.50 \)

3.12.8.4 BS8110 Part 1:

Design ultimate anchorage bond stress,

\[ f_{bu} = \beta \sqrt{f_{cu}} = 0.50 \sqrt{40} = 3.16\text{N/mm}^2 \]
3.12.8.3 BS8110 Part 1:

\[ f_b = \frac{F_s}{\pi \phi l} \]

Therefore, \( I \geq \frac{F_s}{\pi \phi f_{sb}} = \frac{150 \times 10^3}{\pi \times 25 \times 3.16 \times 10^3} = 604.4 \text{mm} \)

Table 3.27 BS8110
Minimum tension anchorage and lap length = 35 \( \phi \) = 875 mm

Provide anchorage length, \( l = 900 \text{mm} \)

See Figure 46,
Due to the space constraint, hook end anchorage has to be adopted.

Try radius of bend, \( r = 4 \phi = 100 \text{mm} \)
Effective anchorage length = (200 + 157 + 100 + 157 + 100) mm = 714 mm

\[ F_{bond} = f_b \times \pi \times \phi \times l \]
\[ = \left( 3.16 \frac{N}{mm^2} \times \pi \times 25 \times 714 \right) / 1000 \]
\[ = 177.2 \text{kN} \]

\[ F_{yield} = 0.95 \times f_y \times A_z \]
\[ = \left( 0.95 \times 460 \frac{N}{mm^2} \times \left( \frac{\pi}{4} \times 25^2 \right) \right) / 1000 \]
\[ = 214.5 \text{kN} \]

\[ F_{bearing} = F_{yield} - F_{bond} \]
\[ = 214.5 \text{kN} - 177.2 \text{kN} \]
\[ = 37.3 \text{kN} \]

\[ F_{bearing} = f_{cu} \times A_{plate} \]

\[ A_{plate} = \frac{37300N}{40 \frac{N}{mm^2}} = 932.5 \text{mm}^2 \]

Total area required = 4 x 932.5 mm\(^2\) = 3730 mm\(^2\)

Provide steel plate with thickness of 12 mm, 200 mm wide and 100 mm high.
Figure 46. Anchorage length of beam reinforcement bar.

Details of the beam-to-column connection are shown in Figure 47.

Figure 47. Details of connection between beam and column.
5. Design of Beam

This section explains the design of transverse beam assuming that the beam is unpropped. In this design, beam section size of B = 400mm, and h=600mm is used.

5.1 Moment

![Beam Bending Moment Diagram](image)

**Figure 48.** Bending moment due to case 1 is adopted as it is the most critical.

Design is based on the most critical load case, i.e. load case 1 (see Figure 48).

At mid-span,
Maximum moment, \( M = 273 \text{kNm} \)

Effective depth, \( d = h - \text{cover} - \text{bar diameter}/2 \)
\( d = 600\text{mm} - 40\text{mm} - 20\text{mm}/2 = 550\text{mm} \)

\[
K = \frac{M}{bd^2.f_y} = \frac{273 \times 10^6 \text{Nmm}}{(400\text{mm})(550\text{mm})^2(40\text{N/mm}^2)} = 0.0564 < K' = 0.156
\]

\[
z = d \left[ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right]
\]
\[
= d \left[ 0.5 + \sqrt{0.25 - \frac{0.0564}{0.9}} \right] \\
= 0.93d \leq 0.95d
\]

\[
A_e \text{ required} = \frac{M}{0.95 f_y \cdot Z} = \frac{273 \times 10^6 Nmm}{0.95(460N/mm^2)(0.93 \times 550mm)} = 1221.4 mm^2
\]

Provide 6T20 (\(A_e\) provided = 1,886 mm²)

**At support**

Taking maximum end moment, \(M = 288\) kNm and using similar calculations as above, the following results are obtained:

- \(K = 0.0595\)
- \(Z = 0.93d\)
- \(A_e \text{ required} = 1288.4 \ mm^2\)

Provide 4T25 (\(A_e\) provided = 1964 mm²)

**5.2 Shear**

Figure 49. Shear force diagram due to load case 1.
Design Of Shear Reinforcement

Only 50% of flexural reinforcement is needed from centre of beam to 0.08L end of the beam.
0.5A_s = 0.5 (1886)
= 943 mm^2

Only 4T20 need to continue until the end of beam
A_s = 1257 mm^2 > 943 mm^2

3.4.5.2 BS 8110 Part 1:
See Figure 49,

\[ \nu = \frac{V}{bd} \]

= 264 x 10^3 / 400 (550)
= 1.2 N/mm^2

Table 3.8 BS 8110

\[ \frac{100A_s}{bd} = \frac{100 (1257)}{400 (550)} \]

= 0.571 < 3

\[ \frac{400}{d} = \frac{400}{550} \]

= 0.73 < 1.0

Therefore, 400 / d = 1.00

\[ f_{cu} \]

25

= 40 / 25

= 1.60

\[ \nu_c = \frac{0.79 \left( \frac{100A_s}{bd} \right)^{\frac{1}{3}} \left( \frac{400}{d} \right)^{\frac{1}{4}} \left( \frac{f_{cu}}{25} \right)^{\frac{1}{3}}}{1.25} \]

= 0.79 (0.571)^{1/3} (1.00)^{1/4} (1.60)^{1/3} / 1.25

= 0.61 N/mm^2
Table 3.7 BS 8110

0.8\sqrt{f_{cu}} = 0.8 \times (40)^{0.5} = 5.08 \text{ N/mm}^2

\nu_c + 0.4 = 0.61 + 0.4 = 1.01 \text{ N/mm}^2

\nu_c + 0.4 < \nu < 0.8 \times (f_{cu})^{0.5}

1.01 \text{ N/mm}^2 < 1.2 \text{ N/mm}^2 < 5.06 \text{ N/mm}^2

Therefore, shear reinforcement is needed.

\[ A_{sv} = \frac{b(v - \nu_c)}{0.95f_{yu}} \]

\[ = 400 (1.2 - 0.61) / 0.95 (250) \]

\[ = 1.0 \]

Try 2R10 \ (A_w = 314 \text{ mm}^2)

\[ S_v = \frac{A_{sv}}{1.0} \]

\[ = 314 / 1.0 \]

\[ = 314 \text{ mm} < 0.75d (412.50 \text{ mm}) \]

Use 2R10-200

Nominal Link

Table 3.7 BS 8110:

\[ A_{sv} = \frac{0.4b}{0.95f_{yu}} \]

\[ = 0.4 \times (400) / 0.95 (250) \]

\[ = 0.67 \]

Try R10 = 157 \text{ mm}^2

\[ S_v = \frac{A_{sv}}{0.67} \]

\[ = 157 / 0.67 \]

\[ = 234.3 \text{ mm} < 0.75d (412.50 \text{ mm}) \]

Therefore, use R10-200
Strength of R10-200 Nominal Link

\[
\frac{A_{sv}}{S_v} = \frac{b(v - v_c)}{0.95 f_{yu}}
\]

\[
v = \frac{0.95 f_{yu} A_{sv}}{b S_v} + v_c
\]

\[
= \frac{0.95(250)(157)}{(400)(200)} + 0.61
\]

\[
= 1.08 \text{N/mm}^2
\]

\[
V = vb_v d
\]

\[
= 1.08 \cdot 400 \cdot (550) / 1000
\]

\[
= 237.6 \text{kN}
\]

Arrangement Of Link

![Diagram of link arrangement]

\[
X = 4.5 \cdot (264 - 237.6) / 264
\]

\[
= 0.45 \text{m}
\]

Take \(X = 1.0 \text{m}\)

Figure 50 shows the reinforcement bar details of the transverse beam.
5.3 Checking the capacity of the designed beam at construction stage.

During construction, a precast beam is installed and joined to the corbel through the starter bar only. At this stage, the connection between beam-to-column behaves as pinned. And, in practice, to increase the speed of erection, slabs are installed before the moment beam-to-column connections are completed. Therefore, during construction, the precast beam has to be designed as simply supported to carry the self-weight of the beam, self-weight of hollow core slabs and weight of machinery equipment and workers during construction, see Figure 51.

Figure 51. Simply supported beam carrying construction loads.
Design checks for the precast beam in Figure 50, considering the beam as simply supported

**Flexural Reinforcement**

Dead load (without concrete topping and finishing) = 4kN/m²
Construction live load = 0.75 kN/m²
Total factored loads = 1.4 (4kN/m²) + 1.6 (0.75kN/m²) = 6.8 kN/m²
Factored loads per unit length = 6.8 kN/m² x 5m = 34 kN/m

Maximum moment \[ M = \frac{wL^2}{8} = \frac{34 \times 8^2}{8} = 272kNm \]

\[ K = \frac{M}{ba^2f_{cu}} = \frac{272 \times 10^6 Nmm}{(400mm)(550mm)^2(40N/mm^2)} = 0.0562 \quad \text{K'} = 0.156 \]

\[ z = d \left[ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right] \]
\[ = d \left[ 0.5 + \sqrt{0.25 - \frac{0.0562}{0.9}} \right] \]
\[ = 0.933d \leq 0.95d \]

\[ A_s \text{ required} = \frac{M}{0.95f'_{y}Z} = \frac{272 \times 10^6 Nmm}{0.95(460N/mm^2)(0.933 \times 550mm)} = 1213 mm^2 \]

Reinforcement provided = 6T20 (\( A_s = 1,886 mm^2 \)) > \( A_s \) required \quad OK! Longitudinal reinforcement bars provided are adequate.

**Shear Reinforcement**

Maximum shear, \[ V = \frac{wL}{2} = \frac{34kN/m \times 8m}{2} = 136kN \quad \text{< V design = 264kN} \]
Shear reinforcement provided is adequate.

The precast beam shown in Figure 50 when simply supported is safe to carry the construction loads mentioned above.
6. Design of Ties

6.1 Introduction

BS8110 clause 3.12.3 specifies the design of steel ties to ensure that precast concrete buildings are robust against any accidental loadings and consequences of progressive collapse. By incorporating appropriate ties, a precast concrete building is said to have the structural integrity and robustness against any accidental loadings.

Ties are designed and incorporated in a building to resist calculated horizontal force, \( F_t \), based on several factors, such as:

1. Number of stories, \( n \)
2. Size of span, \( l_x \)
3. Total loads carried, i.e. total dead load and imposed load

The basic horizontal tie force, \( F_t \), is given in clause 3.12.3 BS8110 as the lesser of:

\[
F_t = (20 + 4n) \text{kN/m} \quad \text{or} \quad F_t = 60 \text{kN/m}
\]

However, if the total characteristic of (dead load, \( g_k \) + live load, \( q_k \)) is greater than 7.5kN/m² and/or the distance \( l_x \) between the columns in the direction of the tie is greater than 5m, then \( F_t \) is modified as the greater of:

\[
F'_t = F_t \left( \frac{g_k + q_k}{7.5} \right) \left( \frac{l_x}{5.0} \right) \text{kN/m} \quad \text{or} \quad F'_t = 1.0 F_t \text{kN/m}
\]

where

- \( n \) is the number of stories
- \( l_x \) is the greater of the distances (in metres) between the centres of the columns, frames or walls supporting any two adjacent floor spans in the direction of the tie under consideration.
- \( g_k \) is the dead load
- \( q_k \) is the imposed load
- \( F_t \) is defined based on the number of storeys as the consequences of collapse are generally more serious for high buildings. \( F_t \) varies from 24 kN/m for a single storey and 60 kN/m for buildings of 10 storeys and more.

6.2 Design of Floor Ties

The following calculations show the design of floor ties at the 2\textsuperscript{nd} floor of the building shown in Figure 14. In this example, precast hollow core slabs are used for the floor system, see Figure 52.
The gravity loads carried by the precast slabs are:
Dead load, Gk = 6.0kN/m², Imposed load, Qk = 3.0kN/m²

**Internal floor ties 1**
Total Gk+Qk = 6+3 = 9.0kN/m² > 7.5kN/m² and distance l_r = 5.0m
The tensile force to be resisted at the end of 5m slabs is the greater of:

\[ F' t = F_t \left( \frac{g_k + q_k}{7.5} \right) \left( \frac{l_r}{5.0} \right) \text{kN/m} \]

\[ F' t = 1.0F_t \text{kN/m} \]

where \( F_t \) is the lesser of:
\[ F_t = 20 + 4n = 20 + (4 \times 3) = 32 \text{kN/m} \]
\[ F_t = 60 \text{kN/m} \]

\[ \{ \text{choose the lesser, } F_t = 32 \text{kN/m} \} \]

Therefore,
\[ F' t = 32 \left( \frac{6 + 3}{7.5} \right) \left( \frac{5.0}{5.0} \right) = 38.4 \text{kN/m} \]

\[ F' t = 1.0(32) = 32 \text{kN/m} \]

\[ \{ \text{choose the greater, } F' t = 38.4 \text{kN/m} \} \]

Area of tensile steel bars required, \( A_s = \frac{F' t}{f_y} = \frac{38.4 \times 10^3 N}{460N/mm^2} = 83.5mm^2/m \)

Use T12@ 1000 mm centres (A_s = 113mm²/m).
These ties are placed in slots at ends of hollow core slabs.
Anchorage length = 35\( \phi \) = 35(12mm) = 420mm
Perimeter floor ties 2

Total $G_k + Q_k = 6 + 3 = 9.0kN/m^2 > 7.5kN/m^2$ and distance $l_r = 5.0m$

The tensile force to be resisted at the end of 5m slabs is the greater of:

$$ F''t = Ft \left( \frac{g_k + q_k}{7.5} \right) \left( \frac{l_r}{5.0} \right) \text{kN/m}$$

or

$$ F''t = 1.0Ft \text{kN/m}$$

where $F_t$ is the lesser of:

$$ F_t = 20 + 4n = 20 + (4 \times 3) = 32 \text{ kN/m} \quad \{ \} $$

$$ F_t = 60 \text{ kN/m} \quad \{ \text{choose the lesser}, F_t = 32 \text{ kN/m} \} $$

Therefore,

$$ F''t = 32 \left( \frac{6 + 3}{7.5} \right) \left( \frac{5.0}{5.0} \right) = 38.4kN/m \quad \{ \} $$

$$ F''t = 1.0(32) = 32kN/m \quad \{ \text{choose the greater}, F''t = 38.4 \text{ kN/m} \} $$

Area of tensile steel bars required,

$$ A_s = \frac{F''t}{f_y} = \frac{38.4 \times 10^3 N}{460 N/mm^2} = 83.5mm^2/m $$

Use T12@ 1000 mm centres ($A_s = 113mm^2/m$).

These ties are placed in slots at ends of hollow core slabs.

Anchorage length = $35 \phi = 35(12\text{mm}) = 420\text{mm}$

Perimeter floor ties 3

Floor slabs parallel with the edge beam, use nominal tie.

$$ F_t = 1.0Ft = 1 \times 32 = 32kN/m \text{ run, irrespective of span, and } A_s = 69.6mm^2/m. $$

Use T 10 ties at 1000mm centres (78.6mm²/m).

Anchorage length = $35 \phi = 35(10\text{mm}) = 350\text{mm provided by hook end}.$
7. Precast Concrete Frame Model

The following sections show two precast concrete frame models of the precast concrete designed building designed in Chapters 2 to 6, using planks and hollow core slabs respectively.

7.1 Precast Concrete Frame with Planks

Figures 53 to 58 show the structural details of the precast frame using precast planks floor system.

![Diagram of precast concrete frame]

Figure 53. Structural system of precast concrete frame.
Figure 54. Three dimensional model of precast concrete frame with precast planks.

Figure 55. Detail 'A': Precast planks supported by precast beam with one layer of steel mesh is provided on top of the planks.
Figure 56. Detail 'B': Cast in-situ concrete topping on top of the precast planks.

Figure 57. Detail 'C': Moment connection between beam-to-column.
7.2 Precast Concrete Frame with Hollow Core Slabs

Figures 59 to 63 show the structural details of the precast frame using precast hollow core slabs.

Figure 59. Three dimensional model of precast concrete frame with precast hollow core slabs.
Figure 60. Detail ‘A’: Precast hollow core slabs supported by precast beam.

Figure 61. Detail ‘B’: Precast hollow core slabs with concrete topping supported by precast beam.
Figure 62. Detail 'C': Beam-to-column connection.

Figure 63. Detail 'D': Beam-to-column connection with hollow core slab.
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