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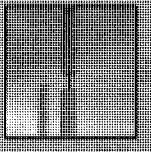
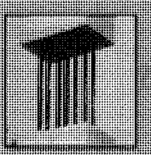
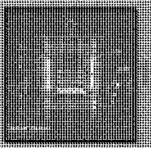
Structural Precast Concrete Handbook



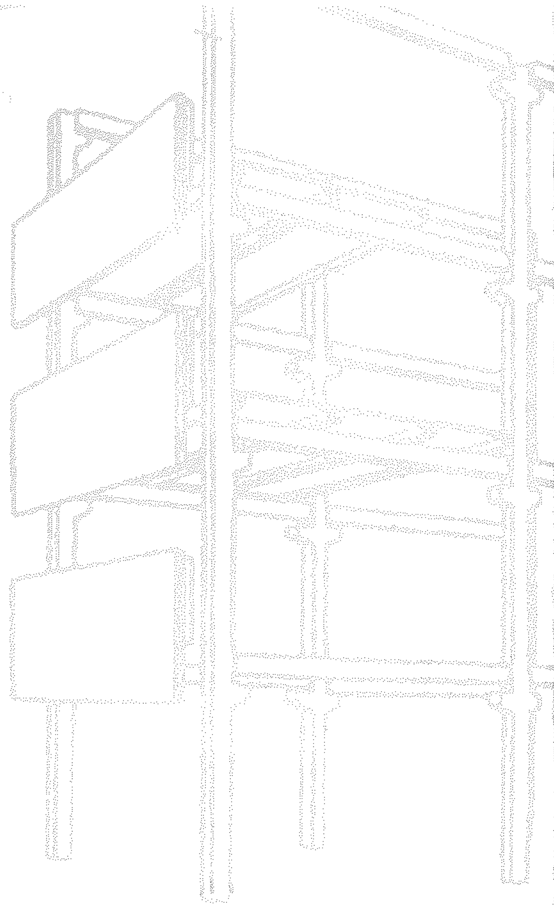




Structural Precast Concrete Handbook

2nd Edition

Building and Construction Authority



The **Structural Precast Concrete Handbook** 2nd Edition is published by the Technology Development Division of the Building and Construction Authority. The 1st edition was published in March 1999.

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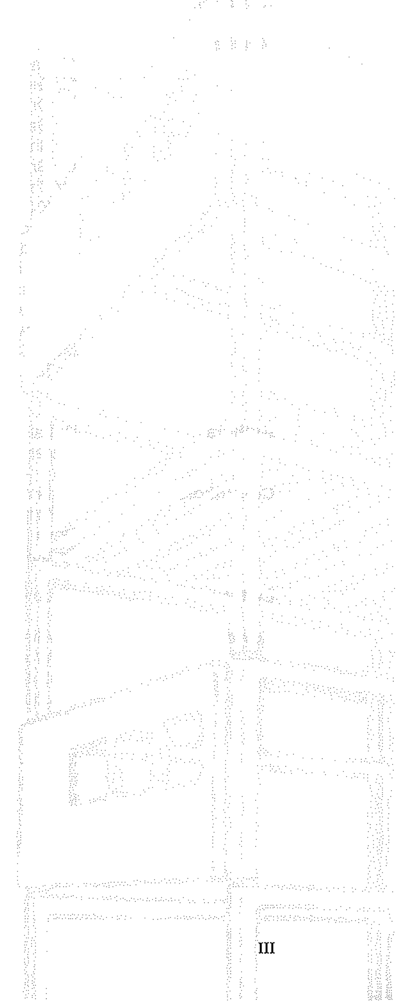
FOREWORD

The Structural Precast Concrete Handbook was first published in March 1999 and was warmly received by the industry. This Handbook is a valuable tool in assisting the industry to be more proficient in precast design. The buildable design regulations introduced in January 2001 has spurred many professionals to gear up by undergoing training in precast design. More buildings are now constructed using precast components with improved construction efficiency.

This Handbook was updated from the 1999 version to include the latest revisions from the Singapore Standard on Code of Practice For Structural Use of Concrete - CP65 : 1999.

I would especially like to thank Dr Lai Hoke Sai for his assistance in this revision. I am confident that this Handbook will continue to serve the industry well.

Lam Siew Wah
Deputy Chief Executive Officer
Industry Development
Building and Construction Authority



ACKNOWLEDGEMENT

The Handbook is the result of two years of many conscientious people within and outside BCA working towards a common objective. Sharing knowledge and experience about the many facets of structural precast concrete design has motivated the Precast Steering Committee, authors, editor and review group. The Precast Steering Committee, formed in 1997, was tasked to oversee the drafting and production of this Handbook from 1997-1999.

This Handbook was initiated by BCA in collaboration with Dr Lai Hoke Sai (PWD Consultants Pte Ltd) and Professor Per Kjærbye (Technical University of Denmark) for their work in the Handbook. Together, they had spent many hours working with all contributors.

To all contributors, we want to express our appreciation for a job well done.

Precast Steering Committee Members (1997 – 1999)

Chairman	Mr Tan Ee Ping
Members	Dr Kog Yue Choong Mr Lam Siew Wah Mr Tan Tian Chong Mr Ong Chan Leng Mr Gan Eng Oon Mr Shum Chee Hoong Mr Graeme Forrest-Brown Mr Shahzad Nasim Mr K Srivelan Mr Poon Hin Kong Mr Eddie Wong Mr Lai Huen Poh Mr Lee Kut Cheung Mr Chuang Shaw Peng Mr Edward D'Silva Mdm Chia Oi Leng Dr Tan Guan
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BCA would like to thank DSTA and PWD Consultants Pte Ltd for consent to use photographs taken from their projects.

In addition, BCA would like to acknowledge the following organisations/publishers for consent to use materials taken from their publications:

FIP – Planning and Design Handbook on Precast Building Structures
Fédération Internationale du Béton
Case Postale 88
CH-1015 Lausanne
Switzerland

Manual of Precast Concrete Construction – System Building with Large Panels (Volume 3)
Bauverlag GmbH
D-65173 Wiesbaden
Germany

PCI Design Handbook – Precast and Prestressed Concrete
Precast/Prestressed Concrete Institute
175 W. Jackson Blvd., Suite 1859
Chicago, IL 60604
USA

Multi-Storey Precast Concrete Framed Structures
Blackwell Science Ltd
Osney Mead, Oxford
OX2 OEL
UK

NOTATIONS

a	lever arm distance; column dimension
a_b	clear distance between bars
a_v	lever arm distance to shear force
a_w	weldment throat thickness
b	breadth of section; column dimension; base plate dimension
b_e	effective breadth; contact breadth in composite section
c	cover distance; distance to centre of bar
d	effective depth of section to tension steel; depth of web in steel sections; depth of shear key; base plate dimension
d'	effective depth to the compression reinforcement; distance from bolt centroid to edge of steel plate
e	eccentricity
e_a	deflection due to slenderness effect
e_x	transverse load eccentricity
f_b	perimeter bond stress; compressive strength of bearing materials
f_{bc}, f_{tc}	bottom and top fibre stress due to prestress after losses
f_{bu}	ultimate anchorage bond stress
f_c	flexural compressive stress in concrete
f_{ci}	characteristic compressive cube strength of concrete at transfer
f_{cp}	prestress at centroidal axis
f_{cu}	characteristic compressive cube strength of concrete
f_{cw}	compressive strength of mortar
f_{cyd}	cylinder compressive strength of concrete
f_{pe}	final effective prestress in tendons/wires after losses
f_{ps}	design tensile stress in tendons/wires
f_{pi}	initial prestress in tendons/wire
f_{pu}	characteristic strength of prestressing tendons/wires
f_s	steel tensile stress
f_t	limiting direct (splitting) tensile stress in concrete; transverse tensile stress in joint concrete
f_y	characteristic strength of reinforcing steel bars
f_{yb}	characteristic strength of bolts
f_{yn}	characteristic strength of horizontal steel reinforcement
f_{yv}	characteristic strength of reinforcing steel links/stirrups
g_k	characteristic uniformly distributed dead load
h	depth of section; height of shear key
h'	net height of infill wall
l	span distance between column-to-column centres; span
l_b	bearing length; overhang of base plate; compressive anchorage length of steel bars
l_e	effective length; effective contact length in composite section
l_o	clear height of column/wall between end restraints
l_p	prestress development strength; embedment length of bars/wires; tension anchorage of steel bars
l_r	distance between columns or walls (stability ties)
l_s	floor to ceiling height (structural tie design)
l_w	length of weldment
n_w	vertical load capacity per unit length in wall or in horizontal wall joint
p	perimeter of steel section
p_w	strength of weld material
p_y	yield strength of steel plate
q	distributed line load
q_k	characteristic uniformly distributed live load
r	bend radius of reinforcing bar
r_s	percentage of tension reinforcement ($100A_s/bd$)
$1/r_b$	curvature at mid-span or, for cantilevers, at the support section
s	leg length of weld; first moment of area of section
s_v	spacing of shear links
t	thickness of section; torsional strength of wall system
t_w	width of joint concrete in precast wall panel; thickness of steel web
v	ultimate shear stress
v_{ave}	average interface shear stress
v_c	design concrete shear stress
v_h	design interface shear stress
w	uniformly distributed load; breadth of compressive strut; width of bearing
w'	diagonal length of infill shear wall
x	distance to centroid of stabilising system; co-ordinate in Cartesian system
y	co-ordinate in Cartesian system
z	lever arm

NOTATIONS (CONT'D)

A	area; cross-section area
A_b	area of bolts
A_{bst}	area of bursting reinforcement
A_c	cross-section area of concrete
A_{ps}	area of prestressing reinforcement
A_s	area of tension reinforcement
A_s'	area of compression reinforcement
A_{sc}	area of vertical reinforcement in column/wall
A_{sn}	area of horizontal reinforcement
A_{SA}, A_{SB}	ring reinforcement in column socket wall design
A_{sh}	shear friction reinforcement in corbel within 2/3 x effective depth of a section
A_{sv}	area of shear reinforcement
B	breadth of void in slab; breadth of building
C	compressive force in steel section inserts design
D	depth of hollow core unit; beam depth
E	Young's modulus of elasticity
E_c	Young's modulus of concrete
E_{ce}	effective Young's modulus of concrete
E_s	Young's modulus of steel
F	force
F_c	compressive force in concrete
F_R	sliding force parallel to the slope of shear key
F_s	tensile force in reinforcing bars
F_t	notional tensile force in stability ties
G_k	characteristic dead load
H	horizontal force; beam depth; overall column dimension
H_A, H_B, H_D	horizontal force in column socket design
H_{bst}	bursting force
H_v	horizontal component of diagonal resistance of infill wall
I	second moment of inertia
I_e	effective moment of inertia
I_{em}	effective moment of inertia at mid-span of beam
I_g	gross uncracked moment of inertia
I_{cr}	cracked moment of inertia
K_t	bond length parameter
L	span; length of building; longitudinal force
L'	net length of infill wall
L_b	bond length
L_1, L_2, L_3, L_4	length of stress block (insert design)
M	bending moment
M_{cr}	cracking moment
M_o	decompression bending moment
M_p	plastic moment of resistance of steel section/plate
M_s	serviceability moment of resistance; service load moment
M_u	ultimate moment of resistance
N	ultimate axial force; ultimate column load; horizontal force at bearing
P	prestressing force; propping force
P_e	effective prestressing force after all losses
P_i	initial prestressing force
P_w	strength of fillet weld
Q	first moment of area in floor diaphragm action design
Q_k	characteristic live load
R	reaction force
R_v	diagonal resistance of infill wall
T	tension force; torque
V	shear force; vertical force
V_{co}	shear resistance in flexurally uncracked prestressed section
V_{cr}	shear resistance in flexurally cracked prestressed section
V_h	horizontal shear force
V_{uh}	ultimate horizontal shear force
Z	elastic section modulus
Z_b, Z_t	elastic section modulus at extreme bottom and top fibres
Z_p	plastic section modulus of steel section

NOTATIONS (CONT'D)

α	angle; characteristic contact length in infill wall; load distribution factor for hollow core slabs
β	column effective length factor; angle; anchorage bond stress coefficient; load distribution factor for hollow core slabs
ε	strain
ε_c	concrete strain
ε_s	steel strain
$\varnothing$, ϕ	rotation; bar diameter; creep coefficient
η	total losses in prestressing force; force reduction factor
λ	joint deformability
μ	coefficient of friction
μ_s	static coefficient of friction
θ	angle
ρ_s	reinforcement ratio (A_s/bd)
ζ	bursting coefficient
δ	deflection; ratio of joint width to joint thickness
χ	depth of neutral axis
σ	normal stress
τ	shear stress

PREFACE

To the uninitiated engineer, prefabrication of concrete structure is often considered as a variation of the cast in-situ technique. The approach has been to assemble the precast components in parts of the structure at the site in a manner that the initial cast in-situ concept structure is obtained as close as possible. This misconception is due to a lack of understanding in the design philosophy and the special characteristics and rules associated with precast concrete design and construction. Wider appreciation of the technique also suffers from a lack of design instruction at the undergraduate level and a limited exposure of the engineer to the design concepts, manufacturing and erection stages as most, if not all, precast concrete design is carried out by in-house engineers employed by precast manufacturing companies.

The publication of this Handbook aims to provide the engineering profession with a wider understanding of the procedures of precast concrete structural design. Besides serving as a useful guide and source of information, the Handbook will be particularly valuable to engineers who are less familiar with this type of method of design and construction.

Chapter 1 of the Handbook provides an overview on the general design principles of some of the most basic structural systems in precast buildings. The chapter emphasises on the design philosophy to achieve overall coherence in terms of stabilising system and structural integrity which are important and critical to precast construction.

In Chapter 2, detail information is given on the design of some of the more common type of precast components such as prestressed hollow core slabs, planks, reinforced concrete beams, columns and load bearing walls. Prestressed precast concrete beams and single or double tee slabs are intentionally excluded as they are not commonly used locally at this stage of time. Where appropriate, examples are used to illustrate the design approaches. Further design aids are also provided in the form of a series of graphs and charts.

Chapter 3 attempts to catalogue and present the principles, design criteria, design formulae or equations to some of the common types of connections used in different types of precast structures.

The design concepts and procedures outlined in Chapters 1 to 3 are applied in Chapter 4 in the design of two precast concrete buildings. The need to consider specific construction requirement in precast concrete design is demonstrated in one of the building design examples.

Chapter 5 aims to give the reader a general idea of the various possibilities of jointing and connection between different types of precast concrete structure. Apart from showing some connections commonly used locally, the readers are also exposed to overseas practices, particularly European practices, where considerable advancement in precast construction has been made.

The design methods and approaches in the Handbook are based on established design rules and revised in accordance with the Singapore Standard on Code of Practice For Structural Use of Concrete - CP65:1999 in this edition. Wherever applicable, the relevant clauses from the Code are given and when parts which are not covered by the Code, other sources or literature are quoted. A list of references is also given at the end of the Handbook for readers who are interested in furthering their understanding of precast concrete design and construction.

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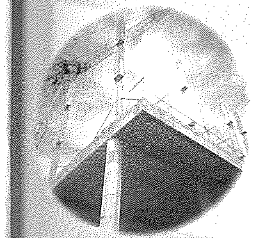


CHAPTER 1

STRUCTURAL CONCEPT FOR PRECAST CONCRETE SYSTEMS

CHAPTER 1 STRUCTURAL CONCEPT FOR PRECAST CONCRETE SYSTEMS

1.1 General

The main structural difference between cast-in-situ buildings and precast buildings lies in their structural continuity. The structural continuity of conventional cast-in-situ buildings is inherent and will automatically follow as the construction proceeds. For precast structures, there must be a conscious effort to ensure that structural continuity is created when precast components, such as slabs and walls, are connected. The connections act as bridging links between the components.

As the structural elements in precast building will only form a stable structural system after the joints are connected, structural considerations for stability and safety are necessary at all stages.

A load-bearing structure with stabilising elements which can sustain both vertical and horizontal loads and transmit these to the foundation and the soil is required. The structure must be robust and adequately designed against structural failure, cracking and deleterious deformations.

The overall behaviour of a precast structure is dependent on the behaviour of the connections which must respond to:

- resistance to all design forces
- ductility to deformations
- volume changes
- durability
- fire resistance
- production considerations
- construction considerations

1.2 Loadings And Load Tables

1.2.1 Design procedures in general

A logical design procedure for the structural engineer must include these four phases (Figure 1.1):

1. Load Assessment: Setting up load estimates and load tables.
2. Calculation Model: Defining the structural system, describing a possible load path, evaluation of stiffness of components and joints, the execution methods and load combinations.
3. Structural Analysis: Starting with the determination of loads on components and joints, calculation of the strength or carrying capacity of materials, cross-sections and joints; thereafter, the loads and the resistance are to be compared.
4. Documentation: Preparation of specifications, sketches, shop drawings and assembly drawings.

PROCEDURES	ACTIONS
1. LOAD ASSESSMENTS	Load Tables Load Estimates
2. CALCULATION MODEL	Structural System Load Path Stiffness of Components and Joints Execution Methods Load Combinations **Calculations:** Internal Forces Reactions
3. STRUCTURAL ANALYSIS (CODES OF PRACTICE)	**Design:** Stresses Deformations Deflections
4. DOCUMENTATION	Specifications Calculations Sketches Drawings

Figure 1.1 Procedure For Structural Design

1.2.2 Vertical and horizontal loads

The building design loads can usually be extracted from the load specifications in codes of practice or building bye-laws. If a load type, such as the wind load, cannot be established because it is a specially shaped building, tests must be conducted.

The vertical loads comprise the dead weight of the structure, superimposed dead loads and live loads. Load from lightweight partition walls is normally treated as a line load. It can be equated as a uniform load if the floor slabs distribute the load evenly.

For precast buildings with simply supported main structural members, it is easy to organise and accumulate the loads acting from top to bottom of the building in load tables. An example of how it deals with accumulating vertical loads in general is shown in Figure 1.2.

The horizontal loads are derived from either the wind forces or from the so-called notional force which is determined as a certain percentage of the total vertical load on the building. In CP65, the notional horizontal load is taken to be 1.5% of the characteristic dead weight of the structure and is treated as an ultimate load for which the stability of the building has to be checked against. It occurs due to eccentricities of the structure, tremors or subsoil settlements. In countries where earthquakes are frequent, a considerably higher value is used, or dynamic calculations have to be made.

The wind forces act on the facades and gables of the building. The notional force, on the other hand, is located at the points of application of the vertical forces, normally accumulated at the centres of gravity of the structural walls and of the floor slabs.

The wind forces and the notional force are assumed to act in an arbitrary direction. The designer only needs to consider the greater of the two forces.

Accidental action is action applied to the structure as the result of accidents and not due to specified imposed loads. Accidental action could occur from collisions, explosions or from vertical loads on air raid shelters.

For structures which must be assumed to be exposed to the risk of accidental action, specifications in the Code* normally give the designer a choice between:

- designing the structure in such a way that the parts of the structure subjected to the accidental action can withstand this, or
- designing the structure in such a way that failure of a given magnitude will not result in the progressive collapse or toppling of the entire structure, as detailed in Figure 1.3

Precautions Against Structural Failure
The figure demonstrates one of the principles of structural integrity. Special joint reinforcement bars are placed to avoid the progressive collapse of the building.

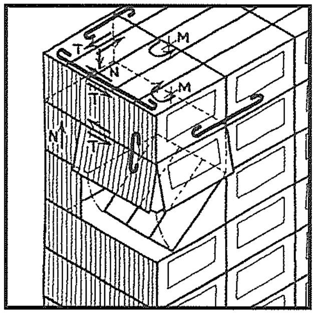
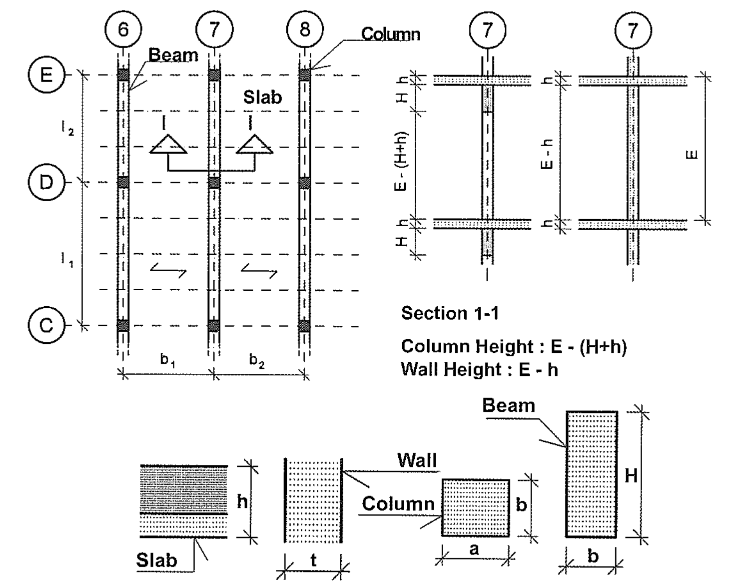


Figure 1.3 Structural Integrity Ties Against Failure

* : CP65 (1999) shall hereafter be referred to as the "Code".



ACCUMULATION OF VERTICAL LOADS	1		2	3	4 (1x2)+3		5	6 (1x2)+5		7		8	9	10 (4x8)+9		11	
CHARACTERISTIC VALUES	SLAB LOADS		SLAB SPAN	BEAM SW	BEAM LOADINGS		WALL SW	WALL LOADINGS		$\Sigma 6$		BEAM SPAN	COLUMN SW	COLUMN LOADINGS		$\Sigma 10$	
	kN/m^2		m	kN/m	kN/m		kN/m	kN/m		kN/m		m	kN	kN		kN	
	DL	IL	$\frac{1}{2}(b_1+b_2)$	DL	DL	IL	DL	DL	IL	DL	IL	$\frac{1}{2}(l_1+l_2)$	DL	DL	IL	DL	IL
TOP FLOOR (ROOF)																	
NORMAL FLOOR																	
x N																	
GROUND FLOOR																	
BASEMENT																	
FOUNDATION																	

DL is for superimposed dead load, IL for live load, and SW for self weight of structures.

The 11 columns on the load table are:

1. Load assessment based on a preliminary design
2. Geometrical determination of slab spans
3. Based on a preliminary design
4. Column (1x2)+3
5. Similar to 1 and 3, based on a preliminary design
6. Column (1x2)+5
7. Sum of 6, $\Sigma 6$
8. Geometrical determination of beam spans
9. Based on a preliminary design
10. Column (4x8)+9
11. Sum of 10, $\Sigma 10$

Figure 1.2 Accumulation Of Vertical Loads

1.2.3 Load path description

A building constructed from precast components becomes a so-called "house of cards". It requires rather simple structural calculations as most of the load-bearing structural members are considered simply supported.

Using precast floor slabs, walls, beams and columns, it is seldom possible to achieve restraint in the joints, mainly due to small dimensions of the components. This calls for special attention when evaluating the stability of the entire structure.

After having made the load assessment and the choice of calculation model, the next and very important step is the load path description. It should explain in detail a possible load path for a specific load from the point of application to where the load is transmitted to the foundation and the soil.

By using a detailed load path description for vertical as well as horizontal loads, the designer is able to calculate all internal forces acting on components and joints necessary for a proper design.

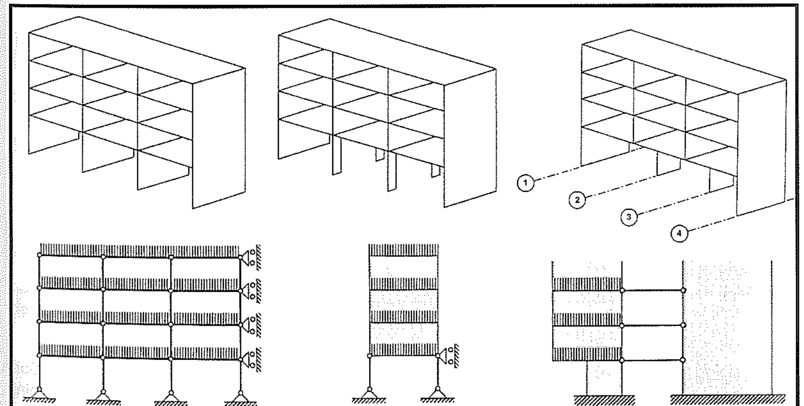
The description could also act very conveniently as a guide for the accumulation of loads and as a list of contents for the structural calculations.

1.2.4 Load distribution

The last preparatory step before the structural analyses can begin is the distribution of all loadings from their application points to the load-bearing and bracing systems. This process is very much linked to the load path description. Based on the determination of structural models and the evaluation of structural stiffness or rigidity, the distribution of loads can be easily accomplished.

With simply supported precast components, it is easy to allocate the vertical load to the load bearing elements, normally directly proportional to the span. Horizontal loads, on the other hand, are more difficult to handle. It is essential that precise and detailed structural description and evaluation are made.

The illustrations in Figure 1.4 give examples of statical models for vertical as well as horizontal loads for a panel system building with different supporting structures on the first floor.



(a) Proportionate Shear Wall (b) Wall Frame (c) Non-Proportionate Shear Walls

Figure 1.4 Statical Models For Vertical And Horizontal Load Distribution Of Large Panel Structures

1.3 Precast Concrete Systems

1.3.1 Building systems

Some examples of the more commonly used precast concrete building systems are illustrated in Figure 1.5. Skeletal frame systems are suitable for commercial buildings, schools, hospitals, parking structures and sporting facilities, where a high degree of flexibility in planning and disposition of floor areas can be achieved by using large spanning column-beam layout. Such buildings are easily adaptable to changes in use, thus giving architects a broader choice in facade claddings.

Systems with load-bearing cross walls, spine walls or facades are more suitable for domestic housing, apartments and hotels. Advantages are high construction speed, ready-to-paint surfaces, facades as architectural precast, good acoustic insulation and fire resistance.

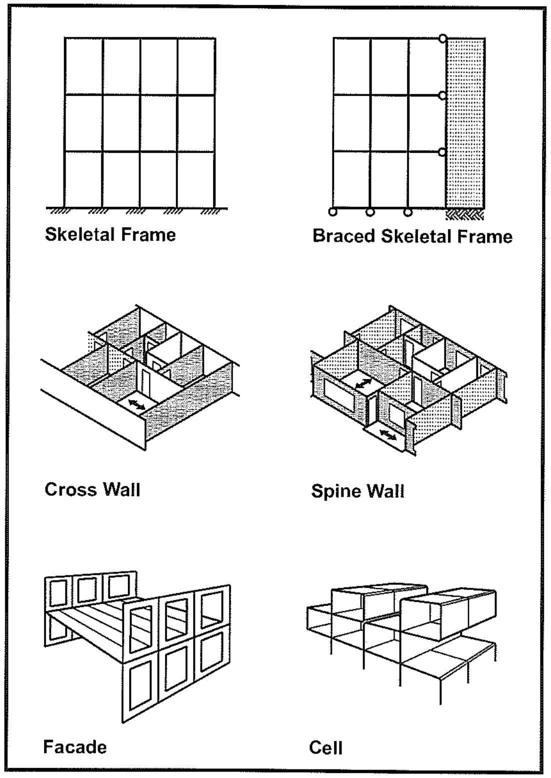


Figure 1.5 Precast Concrete Building Systems

Load-bearing facades are often used in combination with skeletal frames as internal structure. Such systems are economical, have a high construction speed and can incorporate architectural finishing.

Cell systems are mainly used for parts of a building, for instance, bathrooms or kitchens. Important elements in the design are manufacturing, assembly, transportation and erection considerations. Heavy cranage with special lifting devices may be needed.

Some assembly examples of frame and cell systems are shown in Figure 1.6.

The illustration (d) with the transverse crane shows a slab-column system in which stability is achieved by means of rigid cast-in-situ staircase towers.

Sketch (e) shows a cell system in which the individual rooms are prefabricated boxes assembled in a skeleton of precast frames.

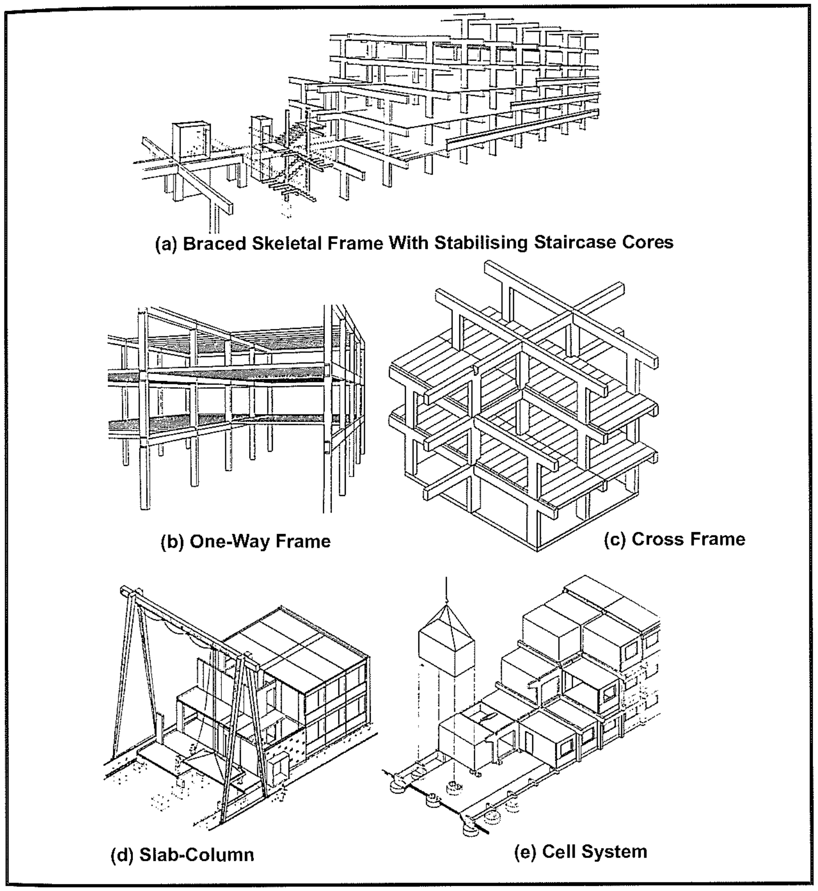


Figure 1.6 Assembly Of Precast Building System

Precast buildings with skeletal frames system may come in different forms and some examples are shown in Figure 1.7.

One- and two-storey column-beam systems together with four-storey high columns are shown. All of these are normally divided into components with joints at crossing points between the structural members.

Structures with other component divisions are also shown, for instance, with structural joints placed at the beam-span near to the zero bending moment point as in structural systems 5 and 9.

The structural joints can be broadly grouped into either hinged or pinned and rigid or fixed joints and they can either be prefabricated or formed at site. By manipulating the joints and their positions, various structural frames can be achieved from the assembly of the precast components as illustrated in Figure 1.8.

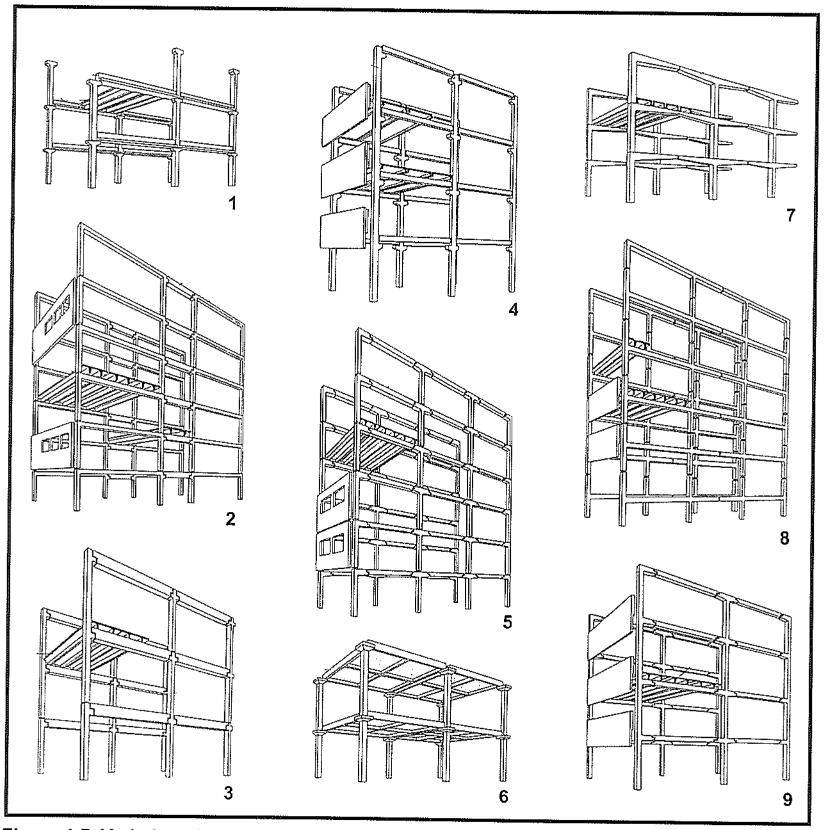


Figure 1.7 Variations In Skeletal Frame Systems (reference 2)

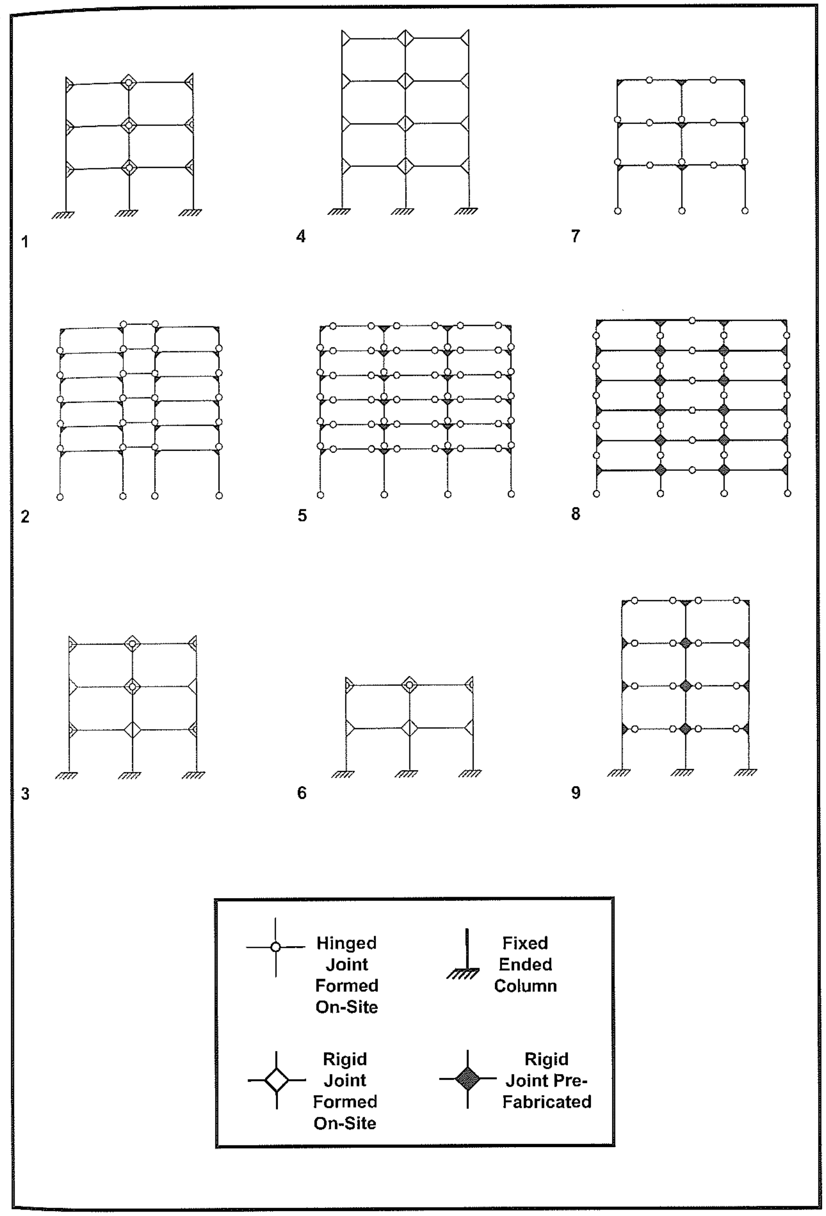


Figure 1.8 Skeletal Frames And Structural Idealisation

Structural System 1:
The system consists of columns with corbels, inverted T-beams and slab components. Columns are restrained at their bases and are spliced at each floor.

Beams are temporarily simply supported on the corbels, but rigid joints are formed afterwards. Floor slabs are simply supported.

Structural System 2:
The system consists of frames, beams and slabs. Frames are one-storey high, simply supported on top of each other, and are connected by a simply supported beam on each floor.

Floor slabs are also simply supported. (Structural System 2 is explained in detail on the following Figure 1.12.)

Structural System 3:
The system consists of one- and two-storey high columns, beams and floor slabs. Columns are restrained at their bases and erected with staggered joints.

Beams are temporarily in simply supported state. They are designed to have permanent rigid joints after construction. Floor slabs are simply supported from beam to beam.

Structural System 4:
Structural System 4 comprises unspliced four- storey high continuous columns with corbels, plus beams and floor slabs. The columns are restrained at their bases.

Beams behave as simply supported at temporary stage, but are designed to have rigid joints after construction. Floor slabs span from beam to beam, and are simply supported.

Structural System 5:
The system consists of T-shaped and L-shaped columns with beams suspended at the point of zero bending moment. Floor slabs span from beam to beam and are simply supported.

Frames of L- or T-shaped columns are placed simply supported on top of each other. At each storey, these frames are connected by beams spanning from frame to frame.

All rigid joints are prefabricated as an integrated part of the columns.

Structural System 6:
The system consists of unspliced two-storey high continuous columns and freely supported large floor units. Columns are restrained at their bases.

Floor units span in two directions and are supported only at the columns. Hidden beams are incorporated. Rigid joints between slabs and column drops are formed after erection.

Structural System 7:
The system consists of L-shaped and T-shaped frame units placed on top of each other. Simply supported floor slabs span between the frames.

The frame units are placed simply supported on the base and on top of each other. Hinged connections are made between the frame units as pin-joints at the mid-span of the beams.

Structural System 8:
The system shown is formed by H-shaped frames with a cantilever beam. Slabs are simply supported by spanning them from beam to beam.

The connections between the H-shaped frames are formed as pin-joints at the mid-height of the columns and between the cantilever beams.

All rigid joints are made as an integrated part of the frame units. Slabs span from frame to frame.

Structural System 9:
The system consists of unspliced four-storey high continuous columns with cantilever attachments for supporting the beams. Floor slabs are simply supported on the beams.

The columns are restrained at their bases. Beams are simply supported on column-attachments. All rigid joints are made as integrated parts of the columns.

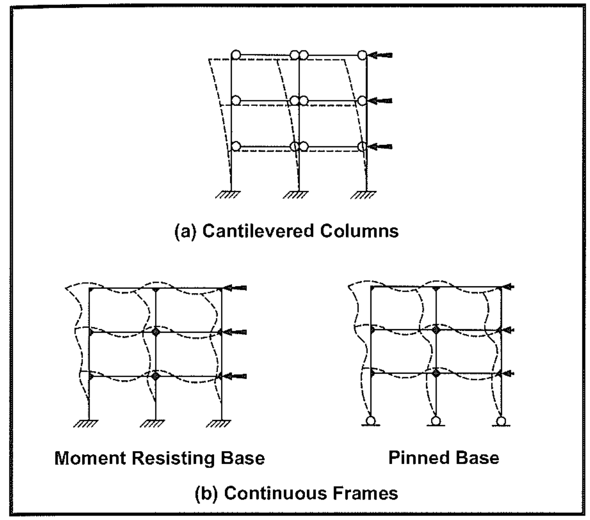


Figure 1.9 Unbraced Frame

1.3.2 Structural systems for horizontal loads

1. Unbraced frame system
- **a. Cantilever columns (Figure 1.9a)**
  - Beam-column connections are pinned. Moment resisting connections can be found at column-foundation intersection.
- **b. Continuous frame system (Figure 1.9b)**
  - Beam-column connections are rigid with flexural and shear continuity. The system allows either moment resisting connections or pinned connections at column-foundation intersection.

2. Braced skeletal system (Figure 1.10)
- Stability is provided by shear walls, shear cores or other bracing systems.
- The base may be pinned or moment resisting connections.
- Beam-column connections may be rigid or pinned.

3. Bearing wall and facade (Figure 1.11)
- Bearing walls which include core walls, spine walls, cavity walls, shaft walls and load-bearing facades can be designed to transfer vertical and horizontal forces to the foundations.
- Non-load bearing panels are designed to withstand the appropriate stresses, but are not intended to carry any load in the building.

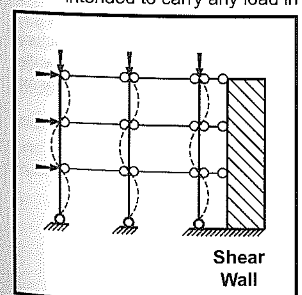


Figure 1.10 Braced Frame

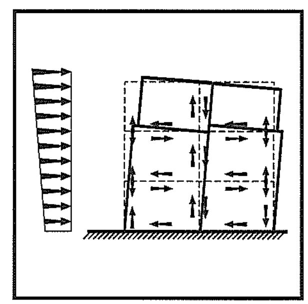


Figure 1.11 In-Plane Action Of Precast Concrete Walls

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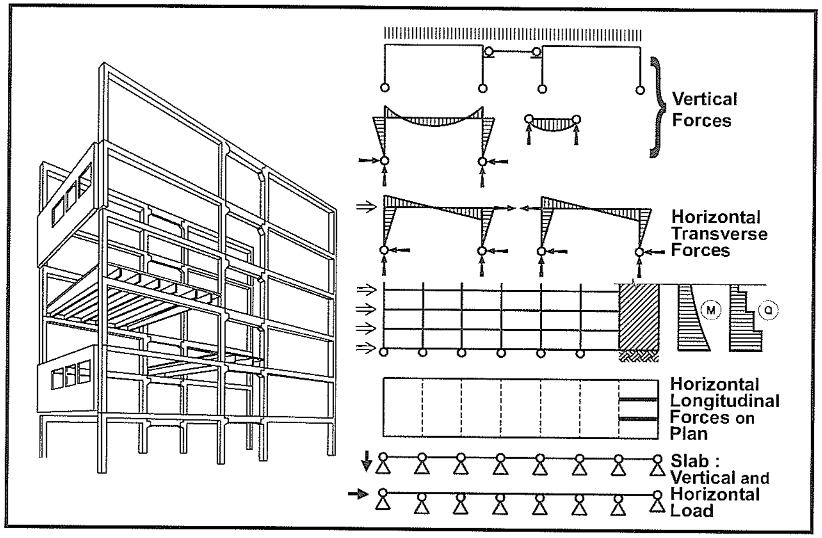

Figure 1.12 Statical Model Of Skeletal Frame For Horizontal And Vertical Forces

Figure 1.12 describes the structural model and behaviour of a skeletal frame system subjected to horizontal and vertical forces. It also covers a load path description. The vertical forces are transmitted from the double-T floor slabs to the beam and to the frames. The double-Ts are acting as simply supported slabs.

The horizontal transverse force is transmitted from the facade components to the floor slab structure, and by diaphragm action to the frame structure. The floor slab structure can act as a continuous beam spanning from frame to frame.

The horizontal longitudinal force from wind or notional force is transmitted from the gable or from gravity centres to the floor slab structure and further on to the bracing walls. These walls are considered as vertical beams restrained at the foundation or at the basement structure. They are able to sustain the longitudinal force as bending moment and shear at the wall-foundation intersection.

Figure 1.13 describes a panel system, possible load paths, and a structural model for vertical and horizontal forces. The system is a cross-wall structure with longitudinal bracing by three spine walls, two near the gables and one in connection with the stairwell structure. On the left, the isometric drawing shows how the floor slabs and the walls respond to the loadings.

To the right, alternative floor spanning directions are shown. The isometric drawing shows a structural system as a combination of load-bearing cross walls and of load-bearing facades and spine walls. As both cross walls and longitudinal walls are load bearing, this system is more suitable to absorb horizontal forces than a system with parallel load-bearing walls only.

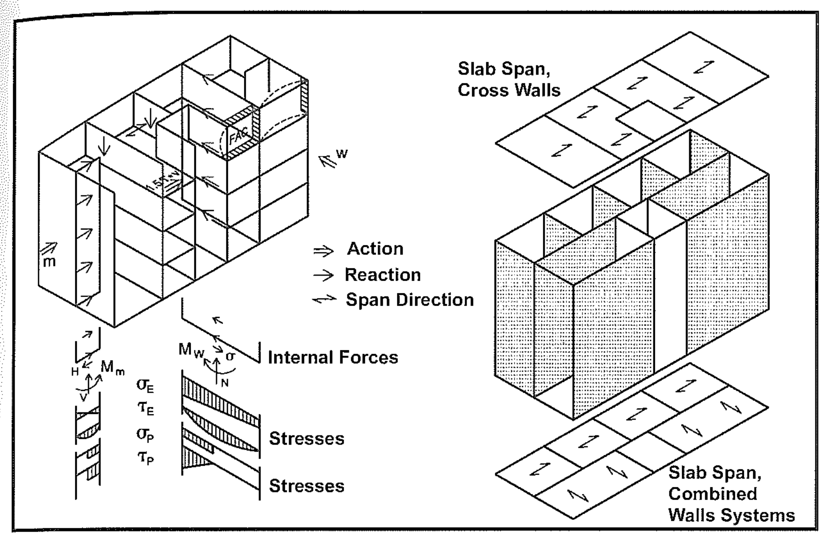

Figure 1.13 Statical Model Of Panel System For Vertical And Horizontal Forces

The bracing walls are all restrained at the foundations, so that internal forces as well as normal, bending and shear stresses can be calculated using either the theory of elasticity or the theory of plasticity.

1.4 Slab Wall Structures

1.4.1 Statically determinate slab-wall systems

Sketches 1, 2, 3, 4 and Sketches 1a, 2a, 3a, 4a in Figure 1.14 show respectively a floor slab supported by eight columns, one bracing wall and columns, two bracing walls and columns, and finally three bracing walls and columns. All columns are considered as pinned, and all walls are only able to sustain and transmit in-plane forces. A building is said to be stable when its individual structural members are in stable equilibrium and can resist the acting forces.

The structure in Sketch 1 (1a) is therefore unstable and unable to sustain any horizontal loads.

The structure in Sketch 2 (2a) is able to sustain only the horizontal force acting in-plane to wall 0. All other horizontal forces will cause movement in the structure.

The structure shown in Sketch 3 (3a) has two walls to carry horizontal loads. This system is sufficient if the horizontal loads can be resolved into components which are in-plane with walls 0 and 1. In other words, the result of the acting forces must go through the crossing between walls 0 and 1. All forces in other directions will cause movement in the structure.

The structure shown in Sketch 4 (4a) comprises the floor slab plus three bracing walls in lines 0, 1 and 2. The walls are placed in such a way that they are not crossing each other at the same point. Such a system is able to sustain any horizontal force. This structure is therefore in stable equilibrium.

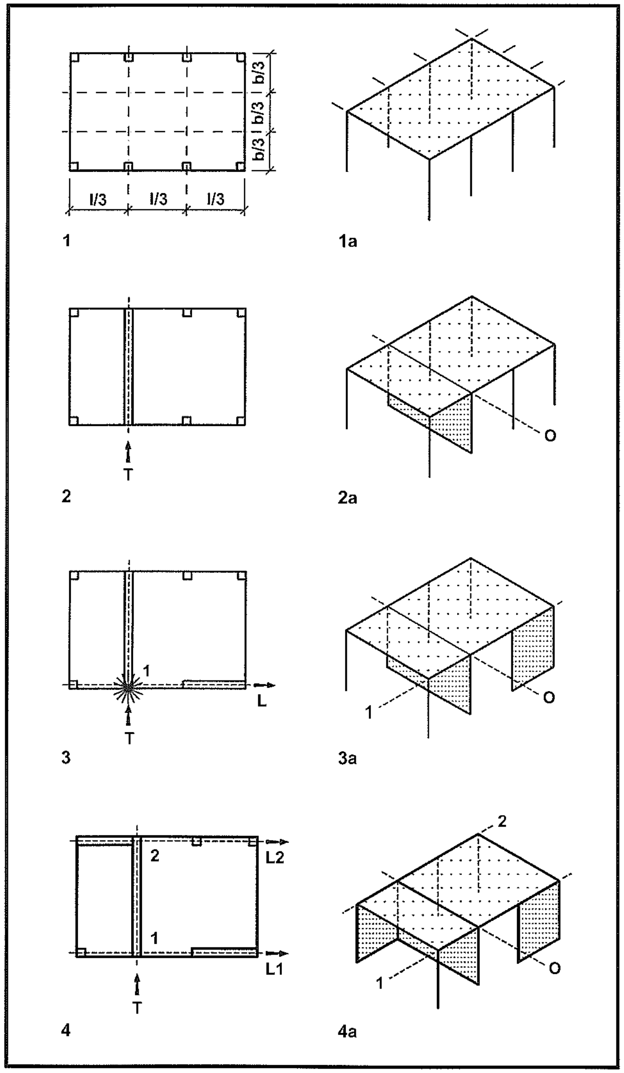

Figure 1.14 Stability Of Statically Determinate Slab-Wall System

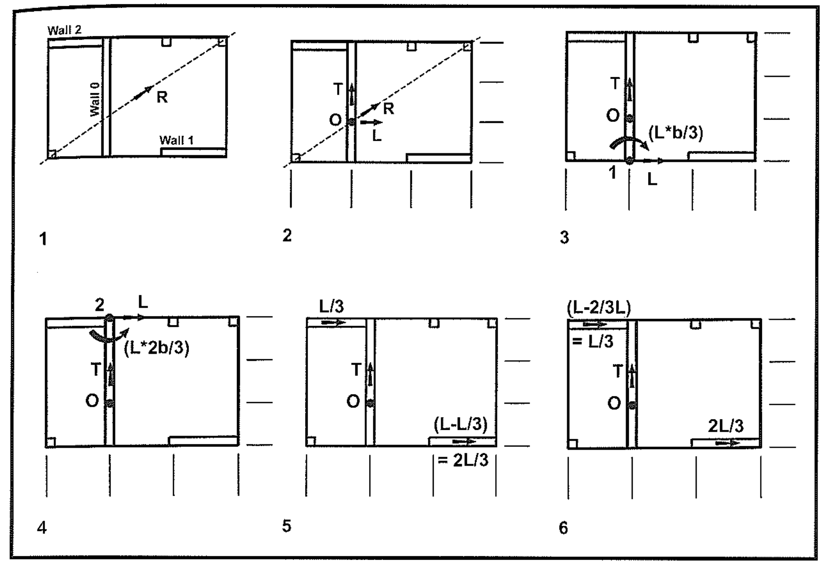

Figure 1.15 Resolution Of Horizontal Forces In Walls

1.4.2 Resolution of horizontal forces

Sketch 1 in Figure 1.15 shows a horizontal force R acting in-plane to the floor slab in the diagonal direction, from one building corner to the opposite corner.

The force R can be moved to point 0 and resolves into a transverse force T and a longitudinal force L as shown in Sketch 2.

Force T can now be sustained by wall 0. Force L has to be transferred to either the crossing point between walls 0 and 1, or to the crossing point between walls 0 and 2.

Sketches 3 and 5 show one of the possibilities where force L is displaced from 0 to the crossing point between walls 0 and 1. A moment L x b/3 has to be added.

Force L will go directly to wall 1, and the moment will be resolved into a pair of forces in walls 1 and 2 with a value of L/3. The final wall actions will be at:

wall 0 : T
wall 1 : L - L/3
wall 2 : L/3

Sketches 4 and 6 show the situation where force L is displaced to the crossing point between walls 0 and wall 2.

The moment to be added is now L x 2b/3 which has to be taken as a force pair still in walls 1 and 2. The value of these forces will now be : 2L/3, which again lead to the final actions at the three-wall system:

wall 0 : T
wall 1 : 2L/3
wall 2 : L - 2L/3 = L/3

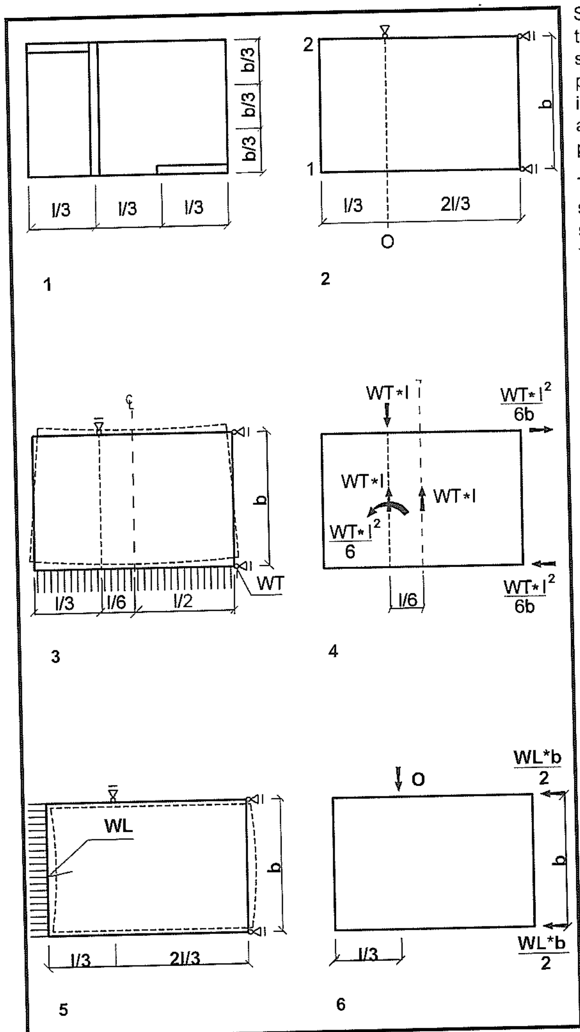

Figure 1.16 **Statical Model Of Wall System For The Transfer Of Wind Load**

Sketch 1 of Figure 1.16 shows the same very simple panel system as described in the previous pages. The system is statically determinate for any horizontal load acting in-plane in the floor slab.

The statical model of the floor slab for horizontal loadings is shown in Sketch 2 in which the supporting points for horizontal loads are depicted.

Transverse wind load WT is acting horizontally on the floor edge as shown in Sketch 3.

The wind WT x l, displaced from wall 0 by l/6, results in a moment WT x l x l/6 which is resolved into walls 1 and 2 with a value of WT x l^2/6b.

The final wall reactions from the horizontal line load WT are shown in Sketch 4 at the supporting points.

Longitudinal wind load WL is depicted in Sketch 5. This wind load acts along walls 1 and 2 with a value at WL x b/2 to each wall.

The final wall reactions from the horizontal line load WL are shown in Sketch 6 at the supports.

Design Example 1

Floor slab A in Figure 1.17 is supported by three walls which are subjected to vertical as well as horizontal forces.

The floor span is 6 m, walls 1 and 2 are 4x3 m, and wall 3 is 6x3 m. The panel system is loaded only with a longitudinal force L, acting in the gravity centre of the floor slab.

It is assumed that all walls act as shear walls and actions are displaced to supporting lines and moments are resolved into a pair of forces. The resultant forces as determined are shown in the figure.

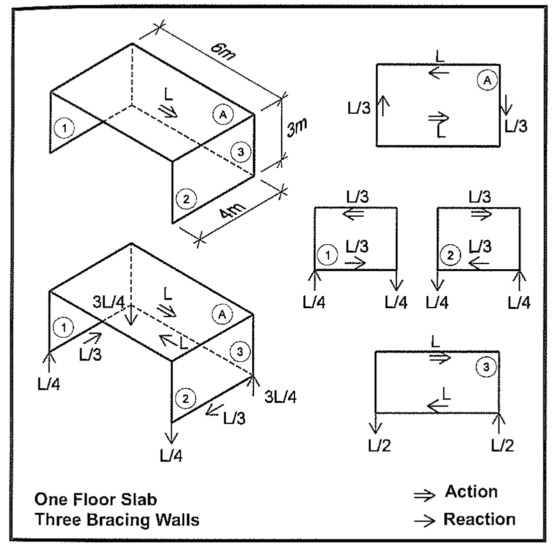

Figure 1.17 Example Of Action And Reaction Forces Due To Horizontal Load In A Determinate Slab-Wall System

Resolution Of Longitudinal Force

One floor slab interacts with three bracing shear walls which transmit the force L to the foundation by pure shear.

Design Example 2

Figure 1.18 shows one part of a building which is made up of a panel structure with load-bearing cross walls. The building is unusual, as the floor below is opened in the sense that cross walls corresponding to walls 6 and 7 are missing.

The normal floor spans at 4 m, but because of the missing walls 6 and 7, is now increased to 12 m. The structure could be considered as a three-dimensional panel structure. Floor slabs A and B together with the walls 1 to 7 act as diaphragm members.

The vertical actions on walls 6 and 7 are resolved into shear forces acting on floor slabs A and B and on the end-wall 5.

Floor slabs A and B as well as wall 5 are then considered as supported by the cross walls 3 and 4, which are, in turn, supported by cross walls 1 and 2.

One Open Floor
The floor slab A or B is supported by walls 6 and 7 which transmit the load to wall 5 via slabs A and B to walls 3 and 4.

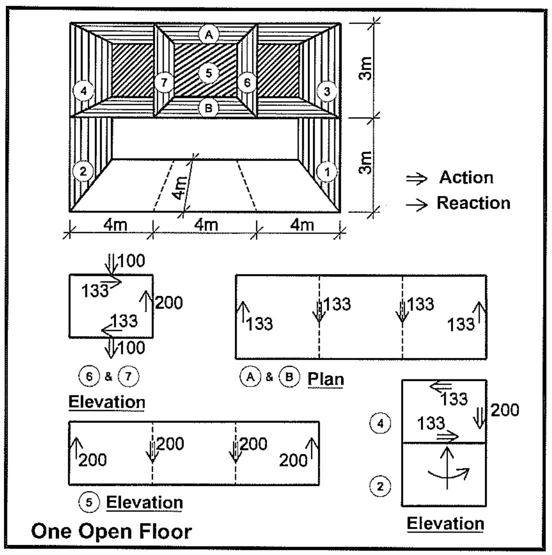

Figure 1.18 Horizontal And Vertical Load Transfer Of Determinate Discontinuous Slab-Wall System

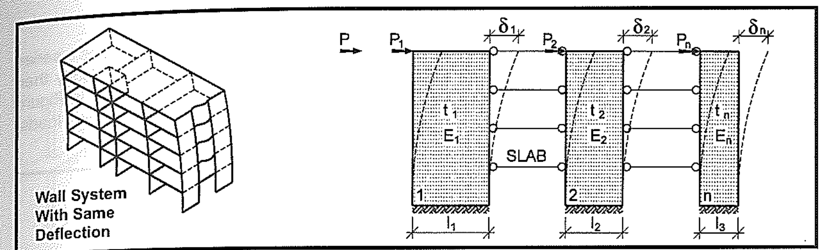

Figure 1.19 Statically Indeterminate Slab-Wall System

1.4.3 Statically indeterminate slab-wall systems

Panel systems in normal cases consist of more than three bracing walls. This means that such systems are statically indeterminate to horizontal loadings.

The wall systems may be regarded as a number of vertical beams restrained at the bottom and coupled together by the floor slabs or beams. They are assumed to be completely stiff due to horizontal deflection.

The illustrations in Figure 1.19 show symmetrical panel systems where all the bracing walls are having the same deflection at a given level, example $\delta_1 = \delta_2 = \delta_n$.

As a rough calculation, it is normally assumed that the deflection of the walls is due only to moment contributions. Contributions from shear can be neglected.

These assumptions lead to a distribution of the total horizontal load onto the individual wall profiles in proportion to their lateral stiffness.

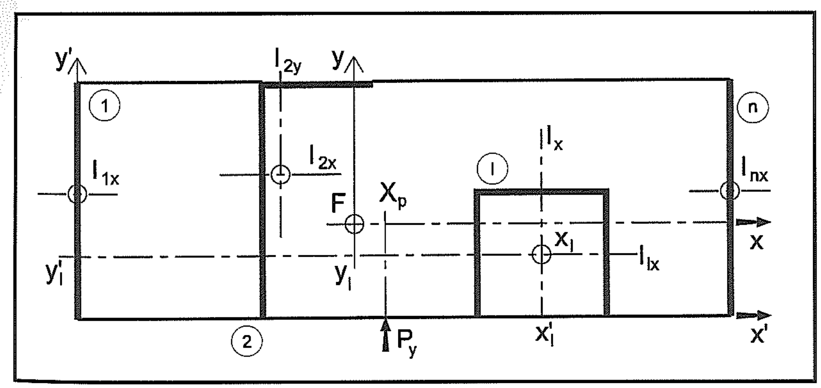

Figure 1.20 Wall System Subjected To Torsion

If a wall system is subjected to torsion as in the case of asymmetrical buildings, or if the acting horizontal loading is asymmetrical, the effect of torsion has to be taken into account. Figure 1.20 shows such a system. The distribution of horizontal load in this instance is proportional to the moments of inertia of the wall sections.

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1.5 Shear Wall Behaviour

If the stiffness of wall joints or slab joints is nominal, the wall or slab structure will behave as a series of beams. Together, they will not be as stiff and will have a smaller load carrying capacity than homogeneous structures. However, if stiffness of such joints is significant as illustrated in Figure 1.21, the wall and slab structure will behave as homogeneous plates in respect of horizontal loads.

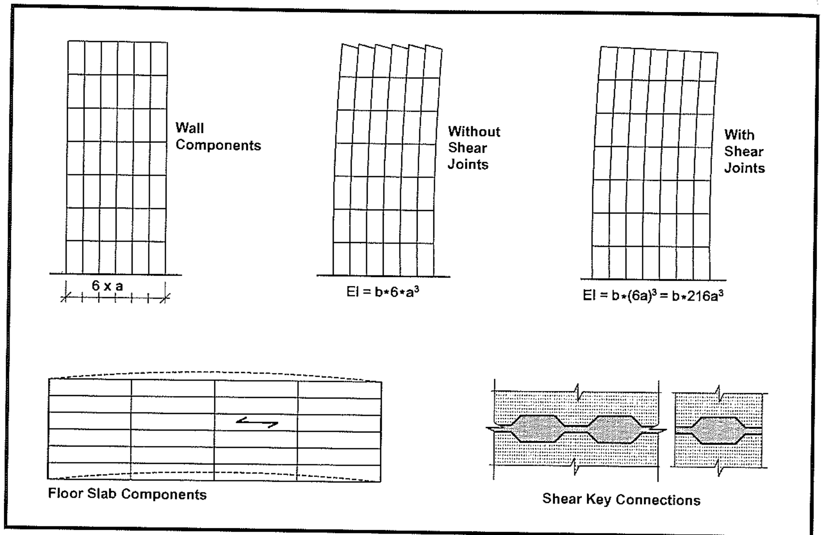
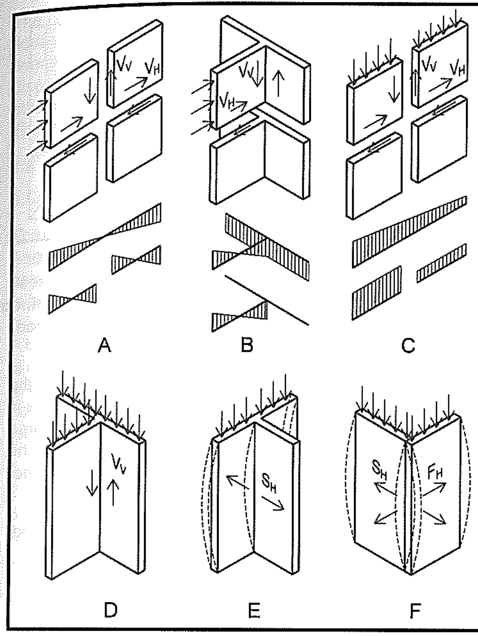


Figure 1.21 Connections And Shear Wall Behaviour

A horizontal load results in internal moment and shear force in the walls, which in turn, gives rise to normal and shear stresses in the vertical and horizontal sections as depicted in Figure 1.22. These stresses can be calculated using Navier and Grashof formulae.



A: A horizontal force calls for a horizontal and vertical shear joint if the components are assumed to interact.

B: If the two walls are not locked together with, for instance, a castellated joint, the profile will not act as a T-section.

C: The vertical load can differ from one wall to another due to differences in slab spans. This will result in shear forces at adjoining connections.

D: The vertical joint between the two walls has to resist shear forces resulting from differences in the acting vertical loads.

E: Stirrups and locking bars have to be installed to avoid possible horizontal wall deflections.

D: At wall corners, the vertical joint must be locked to withstand the risk of horizontal deflections.

Figure 1.22 Normal And Shear Stresses In Walls

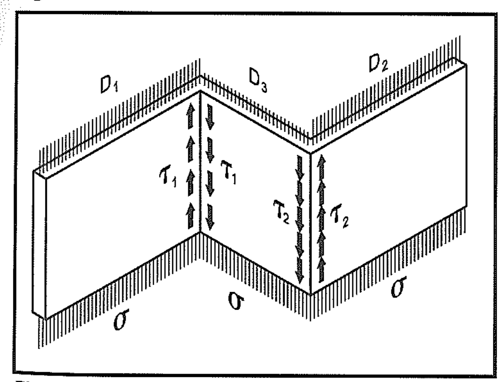


Figure 1.23 Normal And Shear Stresses In Wall Subjected To Non-uniform Vertical Load

Due to high in-plane stiffness of the wall, a non-uniformly distributed vertical load will normally result in uniformly distributed vertical stresses in the next horizontal joint. This will induce shear force in the wall to wall connections as shown in Figure 1.23. Simple equilibrium considerations can be used to determine these forces.

There are numerous connection techniques in the wall to wall joints. A popular method is the castellated joint with or without interlacing steel.

In the evaluation of castellated shear joints in bracing wall structures, the normal practice is to assume that the horizontal load on the structure can be increased until the shear stress at the most heavily loaded point of the joint reaches a permissible value. Up to this load, the wall can be assumed to be homogeneous. The horizontal bearing capacity of the structure at this loading point is also assumed to be fully utilised based on the theory of plasticity.

The ultimate load carrying capacity of castellated shear joints has to be determined by tests. To evaluate and design the joint, it is especially important to study the crack pattern and the distribution of cracks.

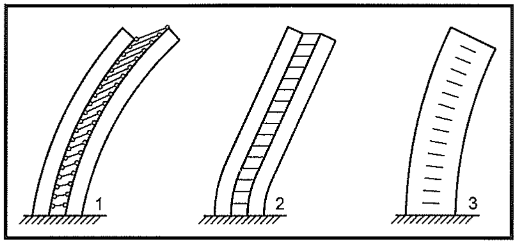


Figure 1.24 Shear Wall With Lintels

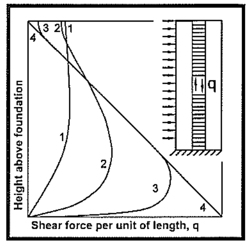


Figure 1.26 Variations Of Shear Forces Using Continuous Layer Method

1.5.1 Continuous layer method

There are various theoretical analyses of shear walls subjected to horizontal forces and one of the more common techniques is the continuous layer method. As the name suggests, the structure is simplified by making the assumption that all horizontal connecting elements are effectively smeared over the height of the building to produce an equivalent, continuous connecting layer between the vertical elements. The two dimensional planar structure is transformed into an essentially one-dimensional one.

The illustrations in Figure 1.24 show three different models of a shear wall subjected to pure bending. The two wall elements are coupled by door lintels.

1. System with completely flexible door lintels.
2. System with an elastic continuous layer in accordance with the continuous layer method.
3. System with completely stiff door lintels.

In Figure 1.25, the continuous layer method is applied to a wall with one row of doors.

1. The structural model.
2. The geometrical behaviour of the chosen structural model.
3. The main system with redundant shear forces.
4. The geometrical behaviour of the continuous layer model.
5. The main system using the continuous layer model with redundant shear.

Curves 1, 2 and 3 in Figure 1.26 show the results of calculations derived from the continuous layer method for increasing stiffness of the layer.

Curve 4 shows the result of a calculation based on the beam theory with a completely stiff layer.

For a stiff layer, there seems to be good agreement between the two theories, except at the boundary area near to the wall-foundation intersection.

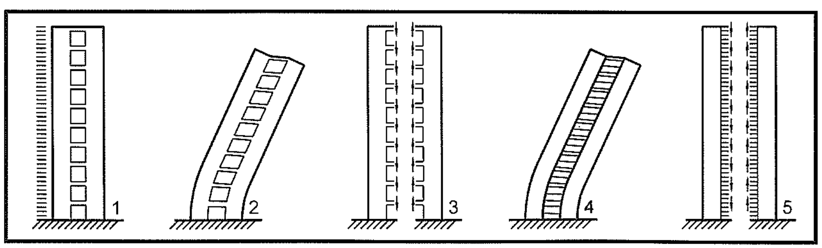


Figure 1.25 The Continuous Layer Method

The continuous layer method is reasonably accurate for uniform system of connecting beams or floor slabs. In many practical situations, the building layout will involve walls that are not uniform over their height but have changes in height, width or thickness, or in the location of openings. Such discontinuity does not lend itself to the uniform smearing of continuous layer representation and other analytical techniques such as the finite element, analogous frame etc, will need to be employed. The designer should refer to relevant literature in this matter.

1.6 Structural Integrity And Design For Progressive Collapse

1.6.1 General

The overall behaviour of a precast structure depends to a large extent on the behaviour of connections. Apart from the design of force transfer between individuals units, there should also be continuity across and ductility within the connections, structural members and of the structure as a whole. This is to ensure structural integrity which is the ability of the structure to bridge local failure.

A badly designed and/or badly detailed precast building is susceptible to progressive collapse which is a chain-reaction failure causing extensive damage or total collapse as a result of localised failure to a small portion of a structure. The failure is initiated by the so-called accidental loads which are not generally considered in the design. The accidental loadings which can be structurally significant include:

1. errors in design or construction
2. local overloading
3. service system (gas) explosion
4. bomb explosion
5. vehicular and falling material impacts
6. intense localised fire
7. foundation settlement
8. seismic effects

1.6.2 Design for progressive collapse

The most direct way to prevent progressive collapse is to reduce or eliminate the risk of accidental loadings by measures such as prohibiting gas installations or erecting barriers to prevent vehicular impact. This, however, may not be practical because every conceivable hazards must be eliminated and all accidental loading conditions fully dealt with.

CP65 adopts three alternative methods in the design for accident damages:

1. Design and protection of structural members

The design, construction and protection of structural members are covered in the clauses 2.2 and 2.6 in Parts 1 and 2 of the Code respectively. The protected members or in the Code's terminology, "key elements", are elements which include connections to adjacent members on which the stability of the structure is to depend. All other structural components that are vital to the stability of the key elements should also be considered as key elements. To prevent accidental removal, the key elements and their connections are designed to withstand an ultimate pressure of 34 kN/m^2, to which no partial safety factors should be applied. The Code recommends that key elements should be avoided as much as possible by revising the building layout within the architectural constraints.

2. Alternative load paths

In this method, which is covered in Part 2, clause 2.6.3, the beams, walls, columns or parts thereof, are considered to have failed and the loads supported by the failed members are transferred by bridging elements to other load bearing members. In designing the bridging elements, the following materials and design load safety factors may be used:

a. Materials (Part 1, clause. 2.4.4.2) concrete = 1.3
steel = 1.0

b. Design loads (Part 1, clause. 2.4.3.2)

dead load	=	1.05
live load	=	1.0 for warehouses and industrial buildings and 0.33 for others.
wind load	=	0.33

3. Provision of structural ties

The design method is aimed at providing minimum levels of strength, continuity and ductility. It is the most commonly adopted solution to prevent progressive collapse in precast structures.

The structural ties are continuous and fully anchored tensile elements consisting of reinforcing bars or prestressing tendons (stressed or unstressed). They are placed in in-situ toppings, infill strips, pipe sleeves or joints between precast components and form a three dimensional network in the longitudinal, transverse and vertical directions as illustrated in Figure 1.27. The design of structural ties is covered in Part 1, clause 3.12.3 of the Code.

The above three methods may be employed separately or use in combination in different part of the structure. It is not permitted, however, to superimpose the effect of the three methods ie, a member must be either fully protected or fully tied and not partially protected and partially tied.

Apart from designing for progressive collapse, the Code also requires that all buildings must be robust and are capable to resist the greater of:

1. an ultimate notional horizontal load of not less than 1.5% of the total charactertisic dead weight of the structure acting at each floor or roof level simultaneously, or
2. the wind load.

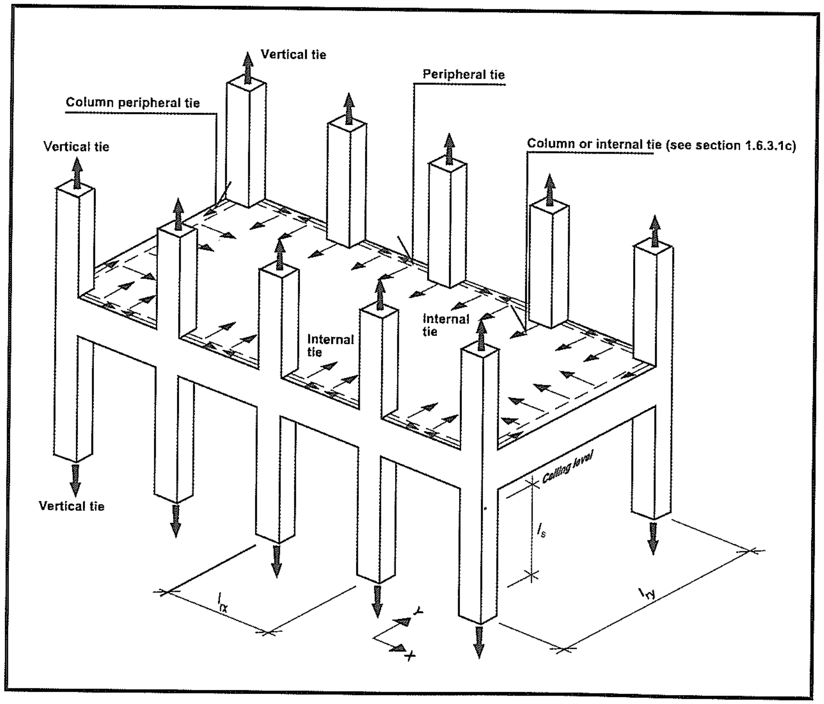


Figure 1.27 Structural Integrity Ties

1.6.3 Design of structural ties

The following ties should be provided and detailed in precast structures:

1. Horizontal floor ties

The basic tie force on each floor or roof should be the lesser of

F_t = 60 kN or
F_t = 20 + 4 x number of storey (in kN)

Horizontal floor ties are further divided into peripheral, internal and column/wall ties.

a. Peripheral Ties

Design tie force = 1.0F_t . Peripheral ties should be located within 1.2m from the edge of a building or within the perimeter walls or beams. Perimeter reinforcement for floor diaphragm action may be considered as peripheral ties. From the maximum basic tie force of 60kN above, the steel area for peripheral ties is 130mm^2 (= 60 x 10^3 /460) which is equivalent to 1 number of T13 bar.

Structures with internal edges eg. atrium, courtyard, L- or U-shaped floor layout etc., should have peripheral ties detailed in Figure 1.28. At re-entrant corner of the perimeter, the tie reinforcement should be anchored straight inwards on both sides.

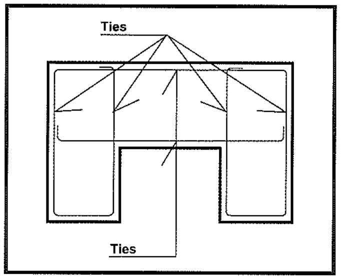


Figure 1.28 Peripheral Ties In Floor Layout With Internal Edges

b. Internal Ties

The ties are in two orthogonal directions and anchored to peripheral ties or to columns and walls. The spacing of these ties must not be greater than 1.5l_r where l_r is the greater distance between centres of vertical load bearing elements in the direction of the tie being considered.

The tie should be capable of resisting a tensile force equal to the greater of (in kN/m)

i. $$\frac{g_k + q_k}{7.5} \times \frac{l_r}{5} F_t$$

$$= 0.0267(g_k + q_k)\, l_r \times F_t \qquad \text{or}$$

ii. $1.0 \times F_t$

where $(g_k + q_k)$ is the sum average of the characteristic dead and imposed floor loads (kN/m^2).

The reinforcement acting as internal ties may be spaced evenly across the floor or grouped within the beams or walls as convenient.

c. **Column and wall horizontal ties**

Design tie force will be the greater of (in kN)

i. $2 \times F_t$ or $(l_s/2.5)F_t$ or

ii. 3% of the total vertical ultimate load carried by the column or wall at the floor or roof level being considered.

where l_s is the floor to ceiling height (m)

At corner columns, the ties are to be in each of the two directions. If peripheral ties are located within the columns and walls and the internal ties anchored to the peripheral ties, no other horizontal ties to columns and walls need be provided. Otherwise, the columns and every metre length of the walls should be tied back to the floor or roof.

2. Vertical ties

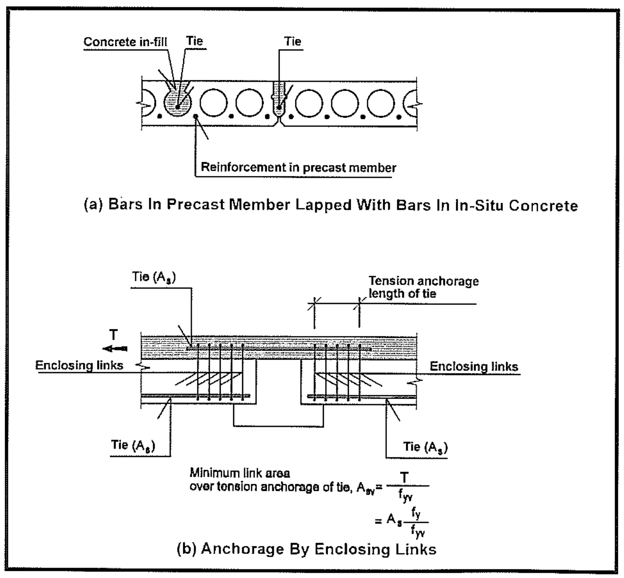
Each load bearing column and wall should be tied continuously from foundation to roof. The tie force in tension will be the maximum design ultimate dead and live load imposed on the column or wall from any one storey or the roof.

3. Proportioning of ties

Reinforcement bars acting as ties are designed to its characteristics strength and the bars provided for other purposes may be used as part or whole of the tie requirement. Ties may be located partly or wholly within precast member as long as continuity of the tie is assured.

4. Continuity of ties

Continuity of tie reinforcement can be achieved by lapping or welding of reinforcement, or by using threaded couplers, cast-in sockets or anchors. Tie continuity created by lapping with precast member reinforcement or using enclosing links is permitted by the Code as illustrated in Figure.1.29.



(a) Bars In Precast Member Lapped With Bars In In-Situ Concrete

(b) Anchorage By Enclosing Links

Figure 1.29 Tie Continuity By Lapping And Enclosing Links

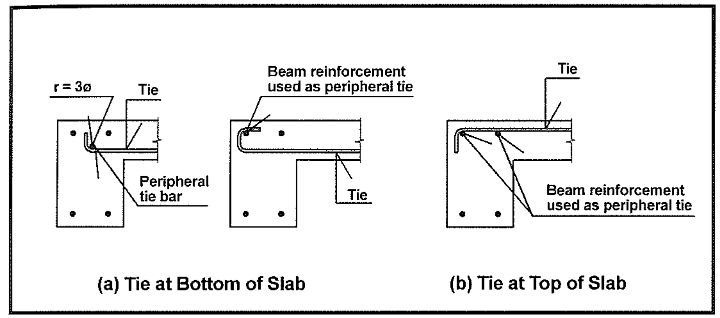
5. Anchorage of ties

The Code requires that the internal tie reinforcement is to be effectively anchored to that in the peripheral ties. Tie bars are considered fully anchored to the peripheral tie if they extend:

a. 12ϕ or equivalent anchorage length beyond all the bars forming the peripheral tie

b. an effective anchorage length (based on the actual force in the bar) beyond the centre-line of the bars forming the peripheral ties.

Figure 1.30 illustrates the above anchorage requirements of internal ties to peripheral ties.

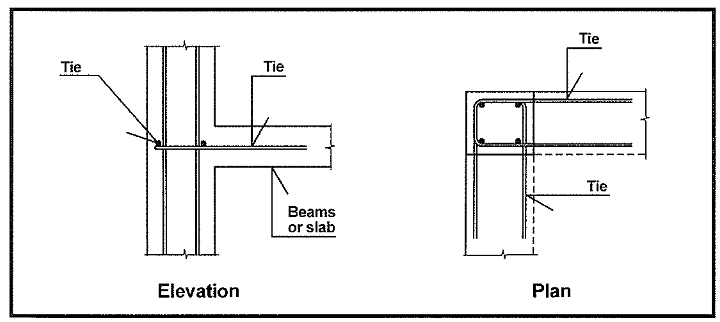


(a) Tie at Bottom of Slab

(b) Tie at Top of Slab

Figure 1.30 Anchorage Of Ties To Peripheral Ties

Figure 1.31 illustrates the tie backs from edge column in two orthogonal directions. It should be noted that the tie backs may also be part of the main reinforcement from the perimeter beams framing into the column.

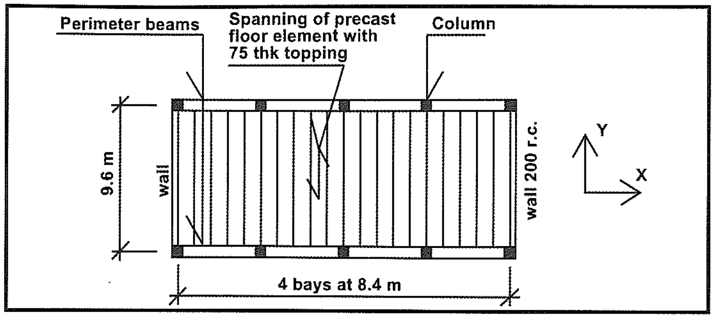


Elevation

Plan

Figure 1.31 Tie Backs For Edge Columns

Design Example 3 : Structural Integrity Ties



Design Data

Total no. of storey	=	8 (including roof)
Floor to floor height	=	3.5m
Characteristics dead load, q_k	=	10 kN/m²
Characteristics live load, g_k	=	3.5 kN/m²
Characteristics steel strength, f_y	=	460 N/mm² for T-Bars
	=	485 N/mm² for steel mesh
Column size	=	400 x 600

1. Basic Tie Force

F_t = 60 kN or

F_t = 20 + 4 x 8

= 52 kN < 60kN

Use F_t = 52 kN

2. Horizontal Ties

a Peripheral tie

Design tie force = $1.0F_t$

Steel area, A_s = F_t/f_y

= $(1.0 \times 52 \times 10^3) / 460$

= 113 mm² (1T13, A_s = 132 mm²)

b Internal Tie

Tie force the greater of

i. $0.0267(q_k + g_k)\, l_r\, F_t$

ii. $1.0F_t$

i. Ties in the X-direction :

l_r = 8.4m

Tie force = $0.0267 \times (10 + 3.5) \times 8.4 \times 52$

= 157.4 kN/m

or = $1.0 \times F_t$

= 1.0×52

= 52 kN/m

Use design tie force = 157.4 kN/m. Tie reinforcement will be provided by the steel mesh embedded within the 75mm thick concrete topping. The total steel area required:

A_s = $157.4 \times 10^3 / 485$

= 325 mm²/m

ii. Ties in the Y-direction

l_r = 9.6 m

Tie force = $0.0267(10 + 3.5) \times 9.6 \times 52$

= 180 kN/m > 52 kN/m

Use mesh within 75 mm thickness topping, steel area required:

A_s = $180 \times 10^3 / 485$

= 371 mm²/m

Use D7 steel mesh (ϕ7 @ 100 c/c both ways) within topping (A_s = 385 mm²/m)

c Column Horizontal Ties

Total ultimate load carried by the column at 1st floor

N = $(1.05 \times 10 + 0.33 \times 3.5) \times 8.4 \times 9.6 \times 7/2$

= 3289.5 kN

Design tie forces :

i. $2 \times F_t$ = 2×52

= 104 kN or

$l_s \times 52/2.5$ = $3.5 \times 52/2.5$ (assume l_s = 3.5m)

= 72.8 kN

ii. 3% of ultimate vertical load = 0.03×3289.5

= 98.7 kN

From above, the design tie force for column F_t =98.7 kN (the greater of (i) & (ii))

A_s = $98.7 \times 10^3/485$

= 203 mm²

According to the Code, if the internal ties are properly anchored into the peripheral ties, no additional ties for column is needed. Hence D7 mesh is adequate as column tie.

The corner columns at both ends of the floor require ties in two orthogonal directions with A_s = 203 mm² in each direction. This will be provided by the main steel in perimeter beams framing into the columns.

3. Vertical Column Ties

Design tie force = ultimate load on column at the floor being considered

= 3289.7/7

= 470.7 kN

A_s = $470.7 \times 10^3 / 460$

= 1022mm² (=0.43% of A_c)

This area of tie will not be in addition to the column reinforcement if it is greater than 0.43%. If the columns are precast with floor to floor joint, the connections must be designed for the tension force of 470.7 kN and tie continuity from foundation to roof properly detailed.

1.7 Floor Diaphragm Actions

Horizontal loads on the structure are transmitted to the vertical stabilising cores, shear walls, structural frames or bracings, etc, by the floors and roofs which act as rigid horizontal diaphragms as shown in Figure 1.32.

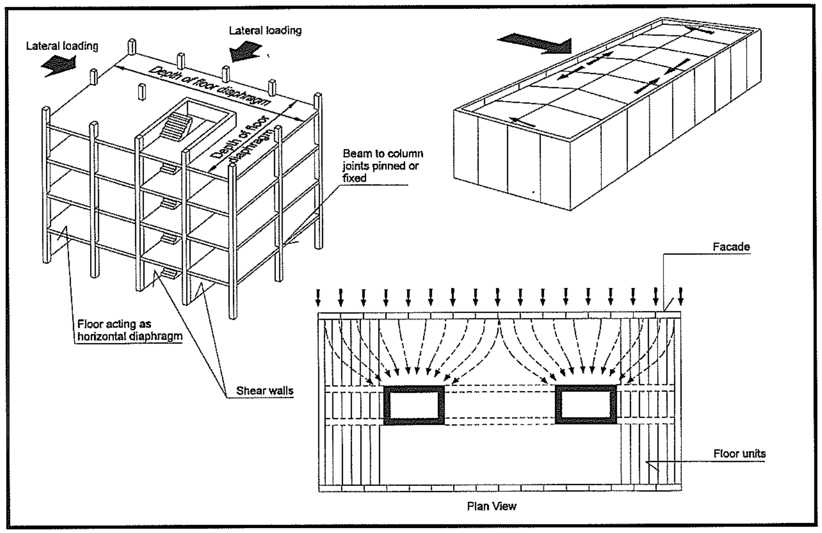


Figure 1.32 Diaphragm Action In Precast Floors And Roofs (reference 3)

1.7.1 Method of analysis

The precast concrete floor or roof is analysed by considering the slab as a deep horizontal beam, analogous to a plate girder or beam containing chord elements as shown in Figure 1.33.

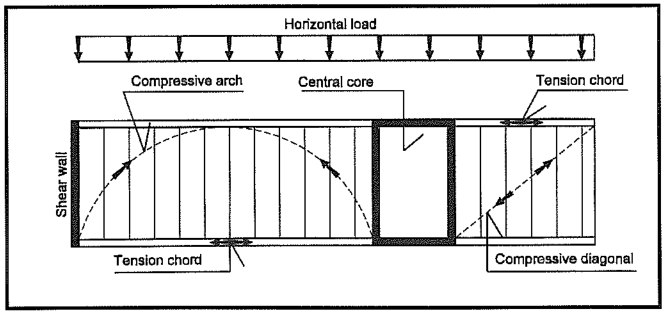


Figure 1.33 Analogous Deep Beam (reference 3)

The stabilising cores, shear walls, frames or other bracing components act as supports for this analogous deep beam and the lateral loads are transmitted as reactions.

The model for a deep beam is usually an arch and tie structure. The tensile, compressive and shear forces in the diaphragm can be calculated by normal statical method as shown in Figure 1.34.

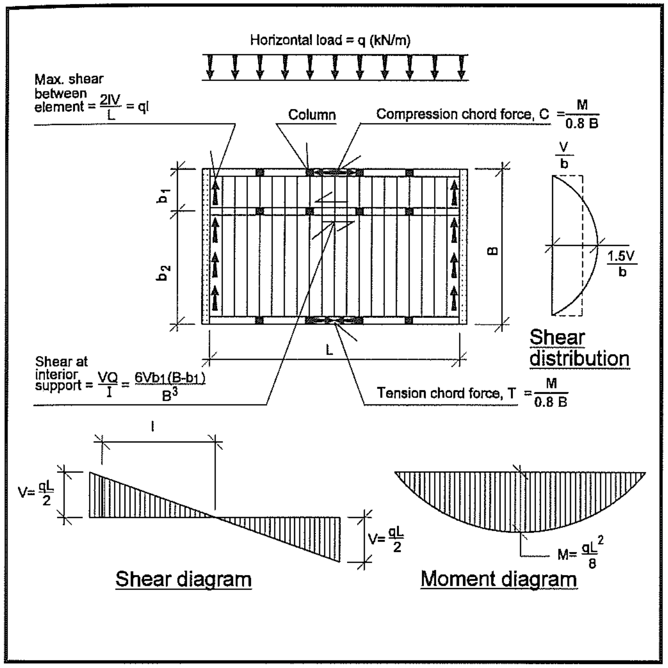


Figure 1.34 Analogous Beam Design Of A Floor Diaphragm

1.7.2 Transfer of horizontal forces

In general, the horizontal forces are transferred between precast units (Figure 1.35) by a combination of shear friction, aggregate interlocking, dowel action and mechanical welding. To resist these forces, it is necessary that the units are tied together so that shear forces can be transferred across the joints even when the units are cracked.

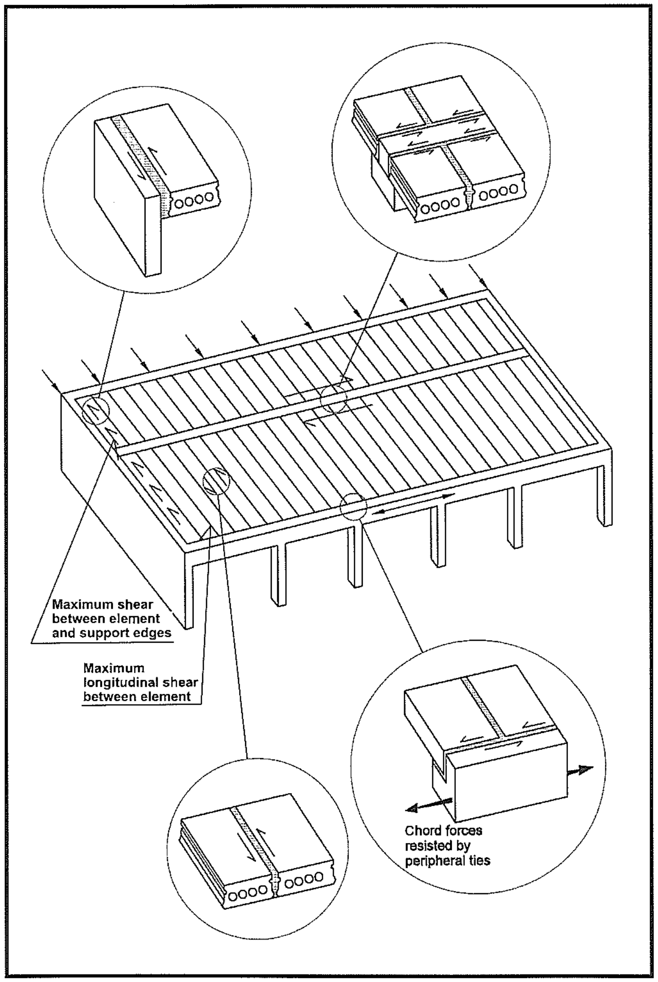


Figure 1.35 Horizontal Force Transfer

1. **Chord forces**

The chord forces are calculated as shown in Figure 1.34. At the floor perimeter, the chord tension is usually resisted by the peripheral ties or by the reinforcement in the perimeter beams.

2. **Shear transfer between elements**

The most critical sections are at the joints between the floor and the stabilising elements where the shear forces are at their maximum. Examples of joint details are shown in Figure 1.36.

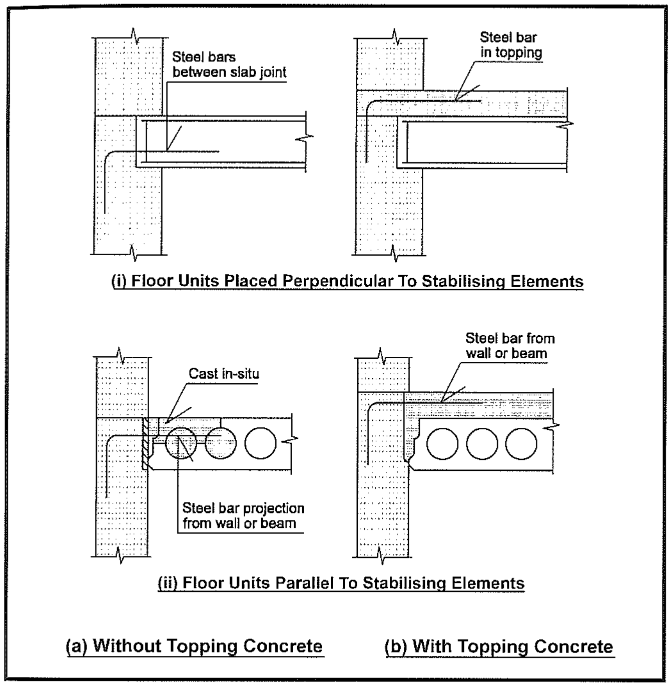


Figure 1.36 Shear Transfer Across Edge Supports

At intermediate supports, the shear force is carried across by reinforcing bars as shown in Figure 1.37. The reinforcing bars for shear transfer is usually determined by the shear friction design method. In general, the forces are quite low and only as many bars as required should be used.

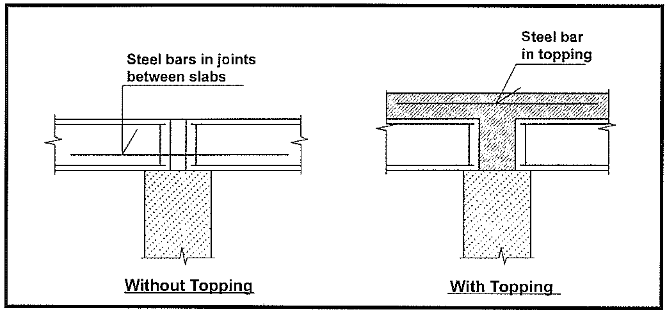


Figure 1.37 Shear Transfer Across Intermediate Support

In floors without composite topping, the longitudinal shear transfer between units is usually accomplished by welded plates or bars in flanged deck elements or grouted keys in hollow core slabs. The welded plates or bar connection may be analysed as shown in Figure 1.38. Variations of the connection are possible from different precast manufacturers.

For elements with infill concrete or grout along the joints, the design of average ultimate shear stress between units over the effective depth of the joints should not exceed 0.10 N/mm² (reference 4). In general, the shear stress calculated at the joint is seldom critical.

For floors with composite topping, the topping enhances the diaphragm action of the floor. The topping is usually reinforced by welded steel mesh which serves both as structural floor ties as well as shear friction reinforcement between units.

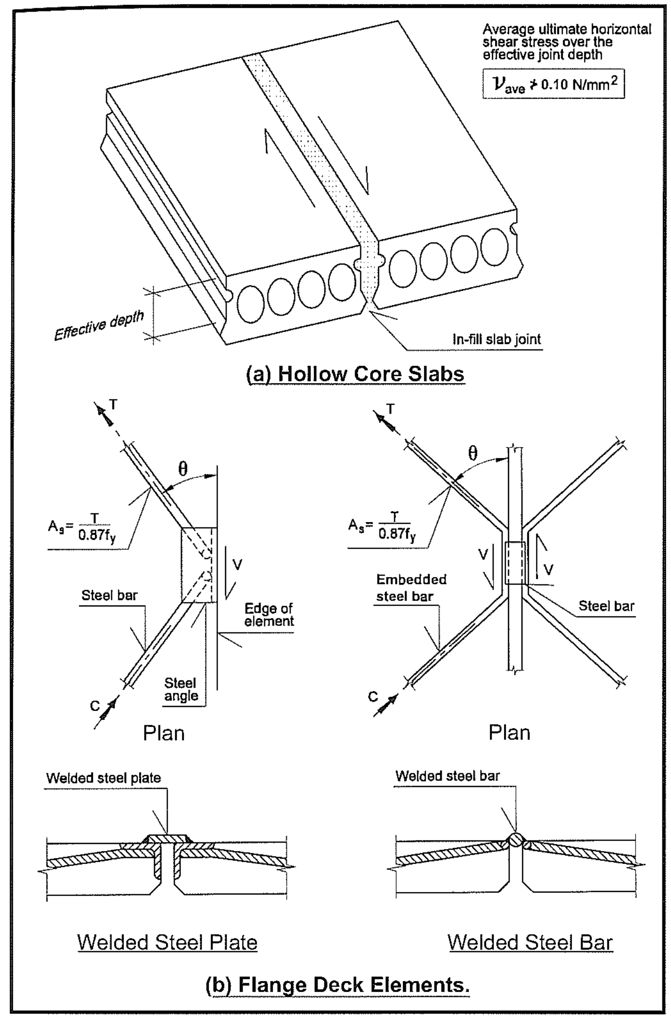
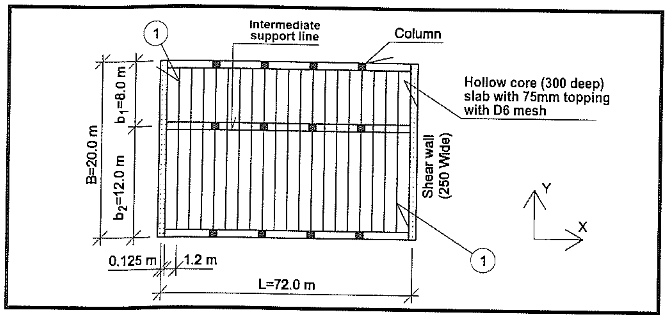


Figure 1.38 Examples Of Shear Transfer Along Longitudinal Joint Between Elements In Floor Diaphragm Design

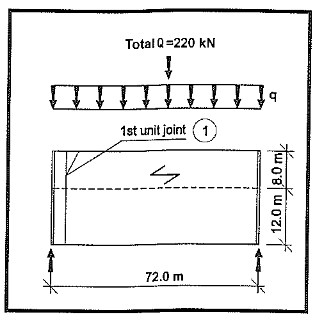
Design Example 4 : Floor Diaphragm Action

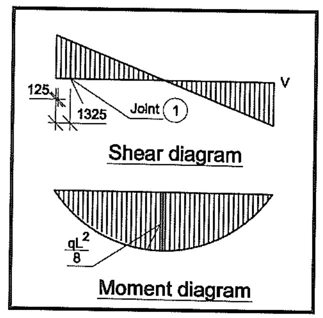
A precast floor is subjected to a total notional horizontal load of 220 kN in each orthogonal direction. The floor elements consist of 300 mm deep hollow core slabs with 75 mm thick concrete topping reinforced with D6 mesh (ϕ6 @ 100 both ways). Determine the critical forces and design the reinforcement for effective floor diaphragm action.



Typical Floor Plan

A. Notional load acting in Y-direction





1. **Shear in floor diaphragm**

a. Reaction at wall as supports :

$$V = 220/2 = 110 \text{ kN}$$

b. Shear at joint between hollow core slabs and wall :

$$V = 110 \times (36.0 - 0.125)/36.0 = 109.6 \text{ kN}$$

c. At first joint between hollow core slabs (Joint ①) :

$$V = 110 \times (36 - 1.325)/36 = 105.9 \text{ kN}$$

d. Shear at intermediate support

$$V_i = 6Vb_1(B - b_1)/B^3 = 6 \times 110 \times 8(20 - 8)/20^3 = 7.9 \text{ kN}$$

2. **Bending moment in floor diaphragm**

Maximum mid-span moment :

$$M = qL^2/8 = 220 \times 72/8 = 1980 \text{ kNm}$$

Chord forces at floor perimeter :

$$T = C = M/0.8B = 1980/(0.8 \times 20) = 123.8 \text{ kN}$$

3. **Design of reinforcement for shear transfer**

a. Across the joint between hollow core slab and wall:

$$V = 109.6 \text{ kN}$$

$$\text{Average shear} = 109.6/20 = 5.5 \text{ kN/m}$$

$$A_s = V/(0.87f_y) = 5.5 \times 10^3/(0.87 \times 460) = 14 \text{ mm}^2\text{/m}$$

Note : No additional reinforcement needed if D6 mesh in topping concrete continues and anchored into the walls.

b. Joint location 1

$V = 105.9$ kN

Average shear $= 105.9/20$

$= 5.3$ kN/m

Check horizontal shear stress at hollow core joint :

Assume effective depth $\approx 0.8h$

$= 0.8 \times 300$

$= 240$ mm

Average shear stress $= (5.3 \times 10^3/240) \times 10^{-3}$

$= 0.02$ N/mm^2 < 0.10 N/mm^2 O K

No additional steel needed.

c. At intermediate support

$V_i = 7.9$ kN/m

$A_s = 7.9 \times 10^3/(0.87 \times 460)$

$= 20$ mm^2/m

No additional steel needed as D6 mesh ($A_s = 283$ mm^2/m) is placed continuous throughout the floor.

d. Peripheral chord ties

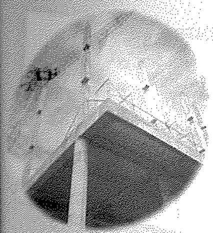
Tension chord forces $= 123.8$ kN

$A_s = 123.8 \times 10^3/(0.87 \times 460)$

$= 309$ mm^2

Note : The required A_s should be compared with the peripheral tie reinforcement as calculated from Part 1 clause.3.12.3.5, of the Code. In most instances, no additional steel is specifically needed for diaphragm action at the perimeters.

Separate calculations may need to be carried out for notional load acting in the X-direction. The computation is similar as above. It can easily be shown that the floor diaghragm action is adequately ensured by the D6 steel mesh placed within the concrete topping.



CHAPTER 2

DESIGN OF PRECAST REINFORCED CONCRETE COMPONENTS

CHAPTER 2 DESIGN OF PRECAST REINFORCED CONCRETE COMPONENTS

2.1 Precast Prestressed Hollow Core Slabs

Hollow core slabs are the most widely used precast floor component in prefabricated buildings. The success is largely due to the highly efficient automated production method, good quality surface finish, saving of concrete, wide choice of structural depths, high strength capacity and rapid assembly on site.

The hollow core slabs are manufactured using long line extrusion or slip-forming processes; the former process being the most popularly used. Cross section, concrete strength, and surface finish are standard to each system of manufacture. Other variations include increased fire resistance, provision of penetrations, opening of cores for on-site fixings, cut-outs for columns/walls, etc.

The hollow core slabs are based on 1.2 m nominal width. Most manufacturers produce the units at 1196 mm width to allow for construction tolerance. The units are available from standard depth of 150 mm to 500 mm.

The slabs are sawn after detensioning which normally takes place six to eight hours after casting and typically when the concrete strength reaches 35 N/mm². Non-standard units are produced by splitting the units longitudinally. Although the cut gives the rectangular plane ends as the standard, skewed or cranked ends in non-rectangular floor layout may also be specified.

Hollow core slabs are generally designed to achieve two-hour fire resistance. Fire resistance up to a maximum of four hours can be designed and produced by either raising the level of the centroid of the tendons or by increasing the concrete cover. To prevent spalling of concrete for covers exceeding 50 mm at elevated temperature, a light transverse steel mesh below the prestressing tendons is often cast at the bottom of the slabs in four-hour fire resistance units.

Holes in the floor may be created in the precast units during the manufacturing stage before the concrete has hardened. The maximum size of the opening which may be produced in the units depends on the size of the voids and the amount of reinforcement that can be removed without jeopardising the strength of the unit. Holes should preferably be located within the void size which may vary in different sections. The designer must be consulted for larger openings which involve the removal of prestressing tendons. General information on the restrictions of opening size and location is given in reference 4 and guides from the manufacturer are shown in Table 2.1 and Figure 2.1.

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 ø (mm)
165	600 x 400	600 x 400	1000 x 400	1000 x 400	80
215	600 x 380	600 x 400	1000 x 380	1000 x 400	130
265	600 x 260	600 x 400	1000 x 260	1000 x 400	130
300	600 x 260	600 x 400	1000 x 260	1000 x 400	170
400	600 x 260	600 x 400	1000 x 260	1000 x 400	170

Table 2.1 Opening In Hollow Core Slabs

It is not practical to cast sockets or surface fixtures into the soffits of the precast units. These must be formed on-site. Fixing by shot fired methods is not recommended. There are limits to the maximum fixing depths at soffits in the webs of the units to prevent accidental severing of the tendons. The list of acceptable proprietary fixing anchors should be obtained from the manufacturer when planning services routing or selecting the suitable hanger system for ceilings.

In local practice, a layer of topping concrete varying from 60 to 75mm is usually included in the construction of hollow core units as structural floors. The topping thickness is generally specified at the support of the units with minimum thickness of 40mm to 55mm maintained at mid-span. The reduction of topping thickness at mid-span is due to prestressing cambers.

In wet weather, water may penetrate into the voids of hollow core slab through the open ends or surface cracks. This should be drained off before permanent floor connections are made. A simple method is to drill weep holes in the slabs at each void location, usually during the production stage.

It is common to find cracks on the surface of the precast units. These cracks may be inherent during the production stage or as a result of the handling and delivery. The types of cracks and their effects on the structural behaviour of the precast units are published by FIP and are shown in Figure 2.2. In doubtful cases, testing of the units should be carried out to verify their structural performance.

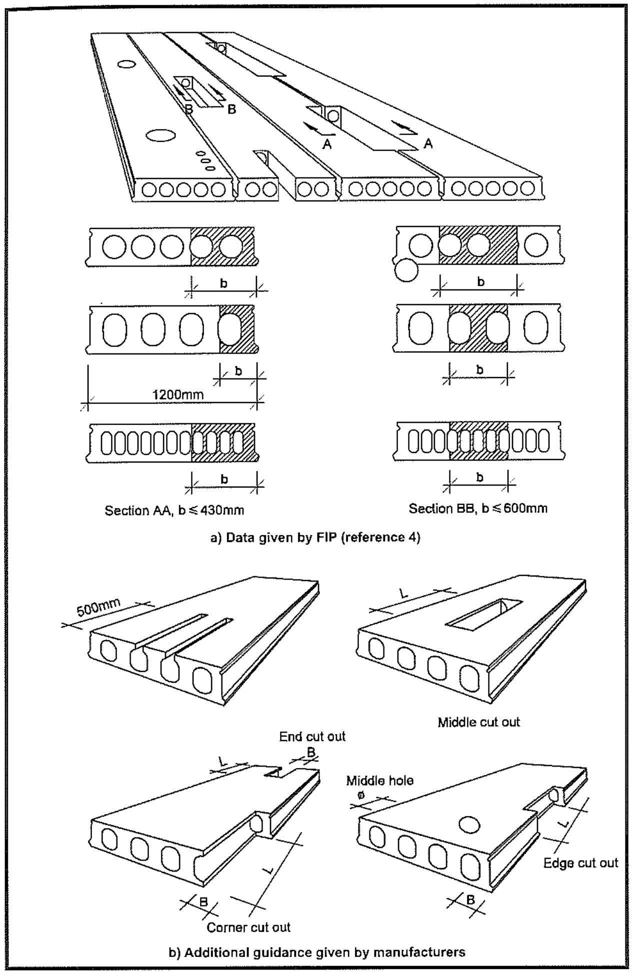


Figure 2.1 Rules For The Permitted Sizes And Locations Of Openings And Cut-Outs In Hollow Core Slabs (Also see Table 2.1)

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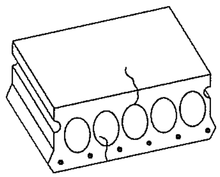
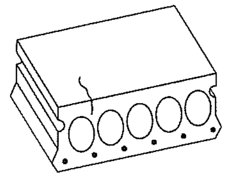
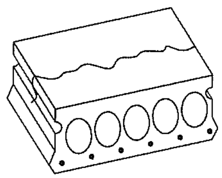
Possible types of cracks on hollow core slabs can be categorised according to FIP report as follows:



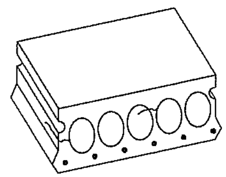
1. Longitudinal crack at void



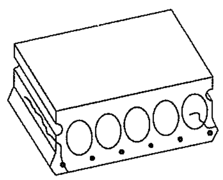
2. Longitudinal crack at web



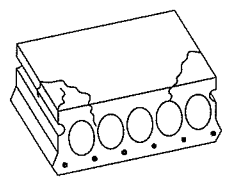
3. Transverse crack



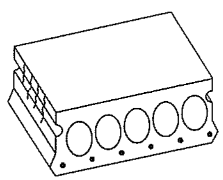
4. Web crack



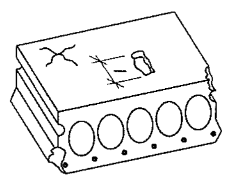
5. Cracks penetrating to strands



6. Corner cracks



7. Tearing cracks



8. Miscellaneous cracks

Figure 2.2 Types Of Cracks

Type	Effect on serviceability	Repair
1.	Severe or full-depth cracks in an untopped system can affect load distribution when there are concentrated loads, or transverse cantilevers. A whole-length crack at the bottom flange is dangerous at the lifting stage.	Voids can be grouted solid or the crack may be epoxied. Repair is not required if cracks are localised, not full-depth or full-length.
2.	Only little effect.	If the crack is deep, possibly penetrating up to the top strand and long, then the slabs should be used only with topping. With minor cracks, repair is not required.
3.	Potential shear capacity reduces if crack occurs at the end of the slab. It can have a significant effect on shear and moment capacities of cantilevers.	Slabs can be repaired if the crack is further away from the end than the anchorage length and is at a maximum 1/3 x depth. For minimal cracks, epoxy or solid grouted voids can be used. Minor cracks (≤ 0.2 mm) in top flange at areas of positive moment or, in bottom flange at areas of negative moment, may not require any repair.
4.	It can be dangerous at the lifting stage and can reduce shear capacity.	If many of the webs are cracked, the slab should be rejected. Small single crack can be accepted.
5.	It can reduce shear capacity. Evaluation must include the effects of the associated strand slippage.	Slipped strands cause load reduction, and load bearing capacity has to be checked.
6.	Usually, a minimal effect.	Smaller cracks need not be repaired. If the corner is cracked, it should be calculated similar to opening and voids and should be grouted solid.
7.	Usually no effect.	Joint grout will automatically fill the cracks.
8.	Usually no effect.	Dropped flanges can be repaired by grouting if the damaged area is limited (l ≤ 0.5 ~ 1.0 m)

Figure 2.2 (cont'd)

2.1.1 Design considerations for hollow core slabs

Hollow core slabs are normally produced by high strength concrete of not less than 50 N/mm^2. This is made possible by using zero slump concrete (w/c = 0.36) together with placing machineries which produce high internal pressure, shear movement compaction and high vibration energy during the extrusion process. The extrusion process makes it difficult to incorporate any other reinforcement than longitudinal prestressing tendons into the precast units. Therefore, unlike conventional reinforced or prestressed concrete structures, the strength of hollow core slabs depends on the stresses induced by prestressing force and the tension and compression capacity of the concrete.

The prestressing tendons or strands 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. The tendons are pretensioned to between 60 - 65% of the characteristics strength in local practice. The in-service effective prestressing force after losses from steel relaxation, creep, shrinkage and deformation is typically at 45 - 50% of the characteristics strength. The steel area is relatively low with ρ_{ps} (= A_{ps}/bd) between 0.1 to 0.25%.

The tendons are anchored by bond and are exposed at the open ends of the units. The effective pull-in of the tendons, determined from depth gauges on the centre wire of the helical strand, is typically less than 3 mm.

The design of hollow core slabs may be based either on the guidance in reference 4 or on the stipulations for general prestressed concrete design in CP65. Some important aspects specifically related to hollow core slabs design will be discussed in the following sections.

1. Design for serviceability limit state

The serviceability limit state design is based on satisfying the limits on :

- flexural tensile and compressive stresses in the concrete, and
- camber and deflection

a. Flexural tensile and compressive stresses in the concrete

The stress limitations apply to hollow core slab section at all ages and under all possible loading conditions. Most designers specify class 2 prestressed concrete structure which permits flexural tensile stresses not exceeding $0.45\sqrt{f_{cu}}$ or a maximum 3.5 N/mm^2 (C60 concrete) but without any visible crack. The flexural concrete compression is limited to $0.33f_{cu}$ at service and $0.5f_{ci}$ at prestress transfer.

Under class 2 stress limitations, the section is considered uncracked and the net cross-sectional properties are used to compute the maximum fibre stress at the top and bottom of the section.

The service moment of resistance is being the lesser of

$$M_s = (f_{bc} + 0.45\sqrt{f_{cu}}) \times Z_b$$

or

$$M_s = (f_{tc} + 0.33f_{cu}) \times Z_t$$

where $f_{bc} = P_e (1/A_c + e/Z_b)$ and
$f_{tc} = P_e (1/A_c - e/Z_t)$
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

b. Camber and deflection

The calculated values of camber and deflection are based on the flexural stiffness E_cI of the section, the support condition and the loading arrangement. Many variables affect the stiffness such as concrete mix especially the water/cement ratio, curing method, strength of concrete at the time of transfer and at the time of erection, relative humidity, etc. Because of these factors, the calculation of short-term and long-term camber and deflection should be treated only as estimations.

An efficient and general procedure in the calculation of camber and deflection is to determine the curvatures and then apply the curvature-area theorem. For straight tendons and simply supported members, 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

$$\frac{1}{r_b} \text{ (long-term)} = \phi \frac{[P_i + (P_i - \delta P)] \times e}{2E_cI} \text{ (upwards)}$$

$(1/2)[P_i + (P_i - \delta P)]$ represents the average value of prestressing force and ϕ is the creep coeffcient which can be determined from Part 2, Figure 7.1, CP65.

The total long-term curvature due to prestress is given by (i) + (ii) + (iii) as

$$\frac{1}{r_b} \text{ (long-term)} = \frac{P_i\, e}{E_cI} [\eta + \frac{1+\eta}{2} \phi]$$

where $\eta = (P_i - \delta P)/P_i$ is the prestress loss ratio.

The curvatures due to applied load are simply :

$$\frac{1}{r_b} \text{ (short-term)} = \frac{M_s}{E_cI} \text{ (downwards) and,}$$

$$\frac{1}{r_b} \text{ (long-term)} = \phi \frac{M_s}{E_cI} \text{ (downwards)}$$

Added together, the total long-term curvatures due to applied load is

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

where $E_{ce} = E_c/(1 + \phi)$ is known as the effective modulus.

iv. Total deflection

For straight tendons in hollow core units, the total deflections can be estimated simply:
Deflection due to self weight and applied loadings:

$$\delta = \frac{5wl^4}{384E_cI} \text{ (downwards)}$$

Deflection due to self weight and applied loadings:

$$\delta = \frac{\eta P_i e l^2}{8E_cI} \text{ (upwards)}$$

An alternative approach to the estimation of long-term camber and deflection is to use the creep multipliers recommended in the PCI Design Handbook (reference 5) as shown in Table 2.2 below:

	Without composite topping	With composite topping
At Erection		
1. Deflection (downward) component – apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
2. Camber (upward) component -apply to the elastic camber due to prestress at the time of release of prestress	1.80	1.80
Final :		
3. Deflection (downward) component - apply to the elastic deflection due to the element mass weight at release of prestress	2.70	2.40
4. Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.20
5. Deflection (downward) – apply to elastic deflection due to super-imposed dead load only	3.00	3.00
6. Deflection (downward) - apply to elastic deflection caused by composite topping	-	2.30

Table 2.2 Suggested Multipliers To Be Used As A Guide In Estimating Long-term Cambers And Deflections For Typical Members

When designing camber and deflection, the following considerations need to be taken into account :

i. Aesthetic deflection limits of 1/250 is applied to units not supporting nonstructural elements which might be damaged by large deflection.

ii. When the units carry non-structural elements sensitive to large deflection, a more conservative approach is needed and guidance is given in Part 2, clause 3.2.1.2, of the Code.

iii. Transverse load distribution due to concentrated or line loads should be considered.

iv. When estimating long-term deflections, suitable levels of design load should be considered as outlined in Part 2 clause 3.3, of the Code.

2. Modes of failure and ultimate limit state design

Under increasing loads, four modes of failure of prestressed hollow core slabs may be distinguished.

a. flexural failure,
b. shear tension failure,
c. shear compression failure, and,
d. bond and anchorage failure.

a. Flexural failure

Flexural failure, shown in Figure 2.3, may occur in critical sections of maximum bending. Due to the relatively small steel area, the failure mode is characterised by flexural cracking at the slab soffits, excessive deflection and, finally, rupture in the prestressing tendons.

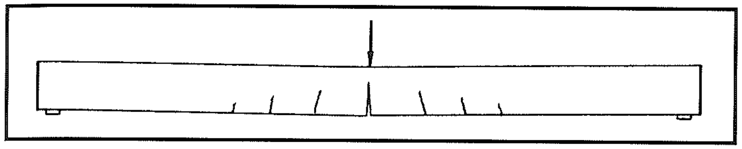


Figure 2.3 Flexural Failure

Calculations of the flexural bending capacity of a cross section can be based on the stress distribution diagram shown in Figure 2.4. In so far as χ is within the top flange thickness, the flexural capacity of the section can be calculated from Part 1, Table 4.4, BS 8110. When χ is within the void area, the value of χ can only be obtained by geometrical or arithmetic means.

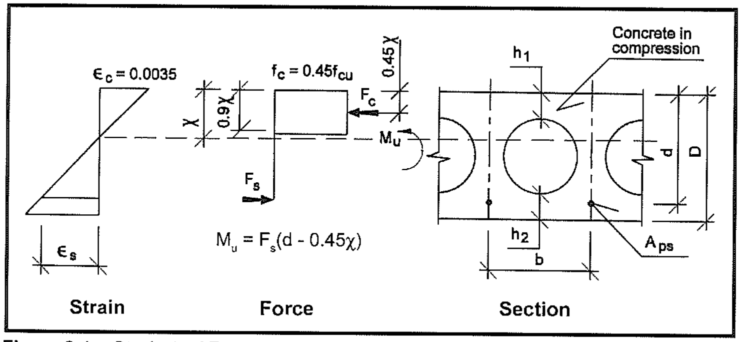


Figure 2.4 Strain And Force Distribution In Hollow Core Slab At Ultimate Limit State

b. **Shear tension failure**

If the principal tensile stress in the web reaches the tensile strength of concrete in an area containing no flexural cracks, an inclined crack may appear and failure may occur suddenly. The crack usually appears at the critical section where the favourable influence of support reaction is no more significant and where the prestressing force is not yet fully developed.

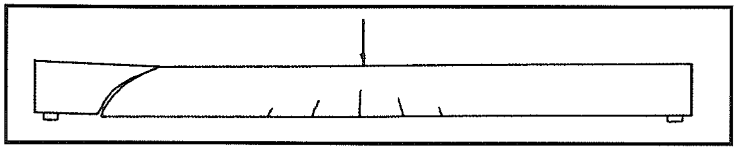


Figure 2.5 Shear Tension Failure

The existence of void in hollow core slabs complicates the theoretical calculation of stresses in the web area and it is necessary to introduce simplifications in the design method. Part 1, clause 4.3.8.4, assumes that the extremity of the support reaction spreads at an angle of 45° from the inner bearing edge. The critical section is taken as the distance from the bearing edge which is equal to the height of the centroid of the section above the soffit. The ultimate shear capacity is then calculated as:

$$V_{co} = 0.67bh\sqrt{(f_t^2 + 0.8f_{cp}f_t)}$$

where $f_t = 0.24\sqrt{f_{cu}}$

f_{cp} = concrete compressive stress at the centroidal axis due to effective prestress at the end of prestress transmission length

The expression 0.67bh is based on rectangular section and for hollow core slabs, it may be replaced Ib/S; where I and b are the respective second moment of area and breadth of the hollow core section and S the first moment of area about the centroidal axis. Ib/S usually works out to be about 0.7 to 0.8bh.

The Code recognises the fact that critical shear may occur in the prestress development length where f_{cp} is not fully developed. The prestressing force is assumed to develop parabolically according to the expression :

$$f_{cp\chi} = (\chi/l_p)(2 - \chi/l_p)\, f_{cp}$$

where χ is measured from the ends of the unit.
l_p is taken as the greater of the transmission length $K_t \o/\sqrt{f_{ci}}$ or the overall depth, D, of the member.

When the critical section is within the transmission length, the uncracked ultimate shear capacity V_{co} will need to be assessed with reduced $f_{cp\chi}$.

c. **Shear compression failure**

Shear compression failure occurs if a flexural crack develops into a shear crack which propagates through the member into the compression zone, leading to an eventual crushing of the concrete. The failure will occur most likely in the vicinity of a concentrated load near to the supports. If the load is uniformly distributed, there is a high probability that shear compression failure will not occur as the shear tension capacity near the support will normally be exhausted before shear compression becomes too excessive.

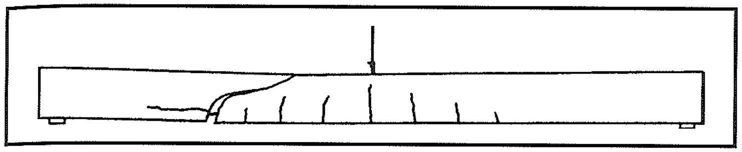


Figure 2.6 Shear Compression Failure

The design ultimate shear resistance of a flexurally cracked section is calculated using the semi-empirical equation in the CP65:

$$V_{cr} = (1 - 0.55\ f_{pe} / f_{pu})v_c\, bd + M_o V/M$$

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

v_c is the permissible shear stress in Part 1, Table 3.9, of the Code.

M_o is the moment necessary to produce zero stress in the concrete at the effective depth, d, level. It may be calculated as $0.8f_{pt}I/y$ where f_{pt} is the concrete compressive stress due to effective prestressing force at depth d and distance y from the centroid axis of the section which has a second moment of inertia I.

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

In design, V_{co} and V_{cr} must always be determined and the lesser of the values governs the shear capacity of the precast unit.

d. Bond and anchorage failure

This mode of failure usually occurs when the slab is subjected to heavy concentrated loads near the support or when heavy loads are applied over a rather short span. The failure is initiated by a flexural crack resulting in a loss of bond around the prestressing tendons due to insufficient anchorage beyond the crack in the uncracked region. Figure 2.7 shows tensile stress, f_{ps}, in the prestressing tendons at point A, where the resulting flexural stress from prestressing and bending moment reaches the flexural tensile capacity of the concrete and :

$$f_{ps} = (\frac{1}{A_{ps}})(\frac{M}{Z} + V)$$

where the additional V/A_{ps} is due to the development of direct tensile stress in the tendons resulting from shear displacement of the cracked section and Z the section modulus at the level of prestressing tendons.

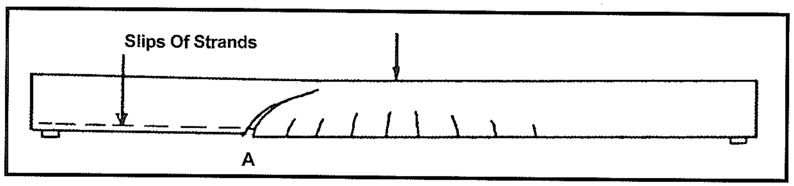


Figure 2.7 Bond And Anchorage Failure

The limiting values of f_{ps} can be determined from the anchorage failure envelope shown in Figure 2.8.

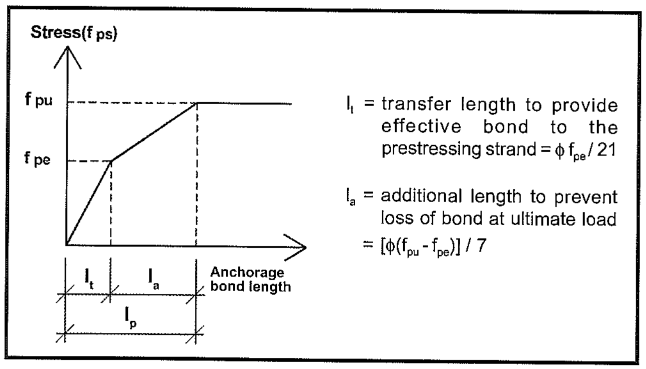


Figure 2.8 Anchorage Failure Envelope

3. Transverse load distribution and joint strength

The transverse distribution of line and point loads on precast hollow core slabs gives rise to :

a. bending moments in the transverse direction of the slab units, and
b. vertical shear forces in the longitudinal joints

Generally, hollow core slabs are manufactured without any transverse bottom reinforcement except light wiremesh for fire resistance purposes. The flexural resistance to transverse bending in the slab is hence solely dependant on the tensile strength of the concrete. From extensive field experience gathered in Europe and America, such omission of transverse reinforcement is justified in hollow core slabs.

The determination of the load distribution is generally based on tests or from theoretical analyses using the following assumptions :

a. floor units are simply supported and 1.2 m wide,
b. concentrated loading is linear and acting parallel to the span of the slab, and
c. the units are provided with transverse ties to prevent them separating from each other.

Four sets of load distribution curves are shown in the Handbook and they cover the following cases :

a. linear loading in hollow core slab floor with and without concrete topping, and
b. point loads in the span and edge of the floor without concrete topping

Alternatively, the designers may adopt the more conservative approach provided by the Code in Part 1, clause 5.2.2. It allows load distribution over an effective width which is equivalent to that of the total width of three precast units or 1/4 of the span on either side of the loaded area. For floors with reinforced concrete topping, load distribution over the total width of four precast units is permitted.

When the precast unit is designed with the entire load acting directly on it, the vertical shear in the joints need not be considered as there will be zero shear force in the transverse joint. However, when transverse load is unevely distributed or if point or line loads are present, the vertical shear capacity in the longitudinal joint will have to be assessed.

Extruded hollow core slabs have a natural random surface roughness of up to about 2 mm deep indentation. This surface is, however, classified as smooth in the Code and a design ultimate shear stress value of $0.23N/mm^2$ is permitted in Part 1, clause 5.3.7a of the Code for the shear transfer at the joint. However, the value appears to be high when compared with $0.1N/mm^2$ in reference 4. The designer should use the lower value of $0.1N/mm^2$ when designing the average horizontal shear stress in the longitudinal joints as in reference 4 which deals with hollow core floor design specifically.

In general, the vertical and horizontal shear capacity at the transverse joints are rarely critical in design provided the joint infill is placed and well compacted. Although FIP stipulates minimum C20 concrete, local practice tends to cast the joints together with topping concrete which is normally specified with minimum C30 in design.

2.1.2 Design charts

The design charts on the selection of hollow core slabs for different loading and span are shown in Figure 2.9.

Transverse load distribution factors for line and point load on hollow core slab floor with or without topping are given in Figures 2.10 to 2.13. The actual design and disposition of prestressing tendons may be different between precast manufacturers and they should be consulted in the final design of the components.

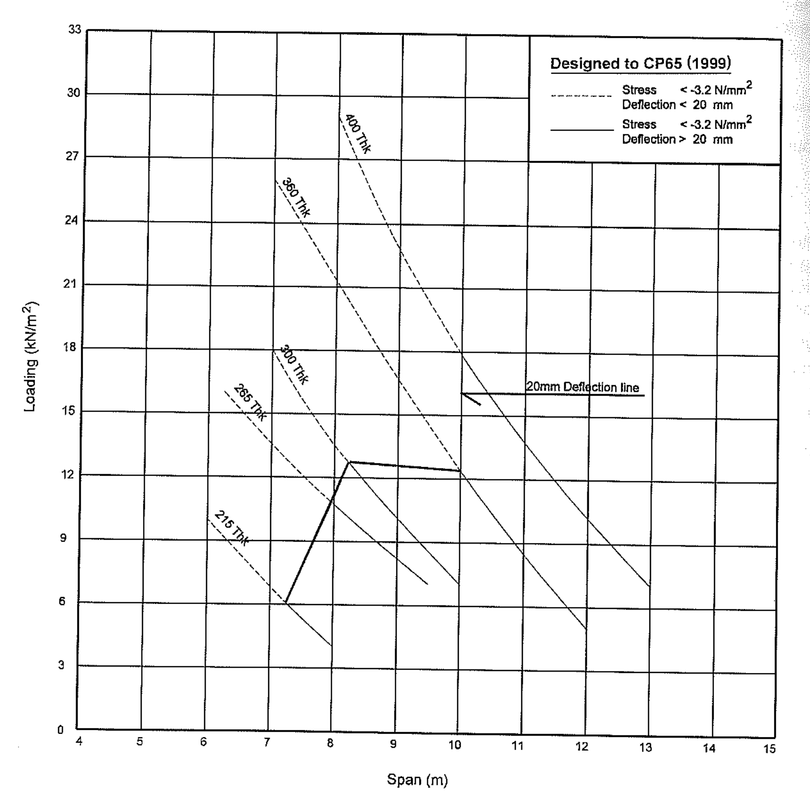


Figure 2.9 Loading Chart Based On Bending And Shear Capacity For Hollow-Core Slab

Details are intended for general information only. Precasters should be consulted for actual design of the slabs.

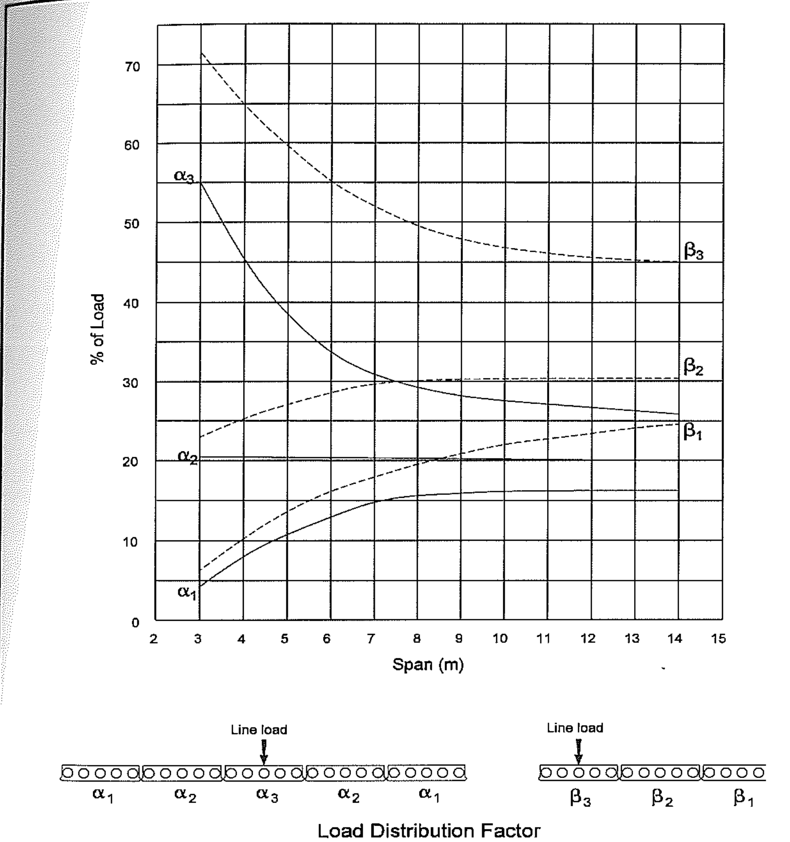


Figure 2.10 Load Distribution Factors For Linear Loadings In Hollow-Core Slab Floors Without Topping

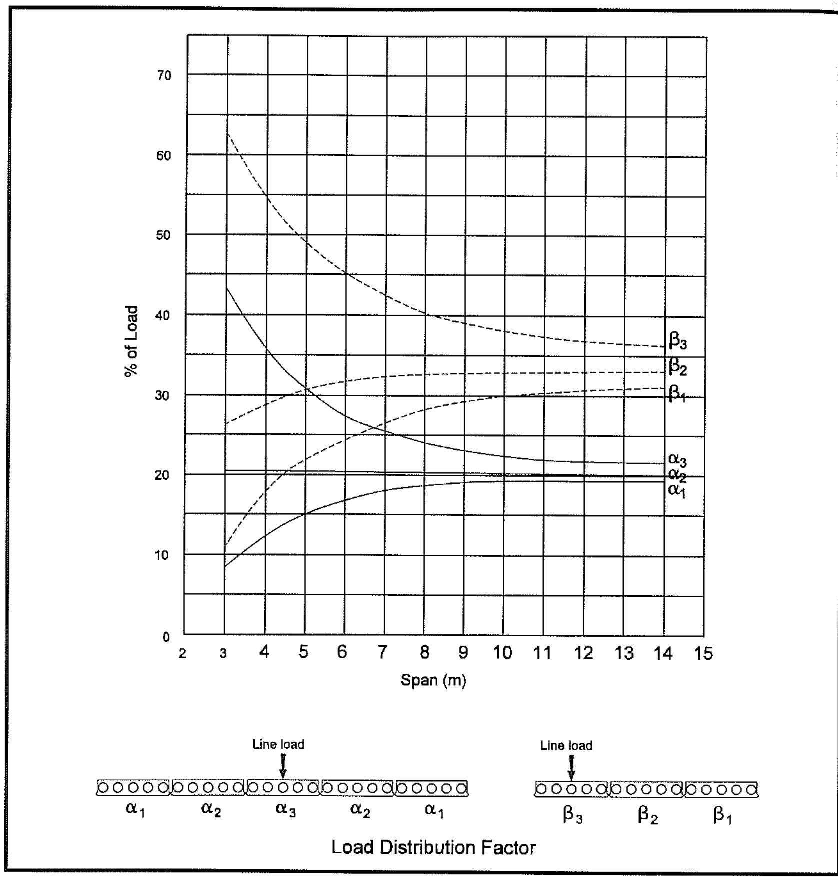


Figure 2.11 Load Distribution Factors For Linear Loadings In Hollow-Core Slab Floors With Toppings

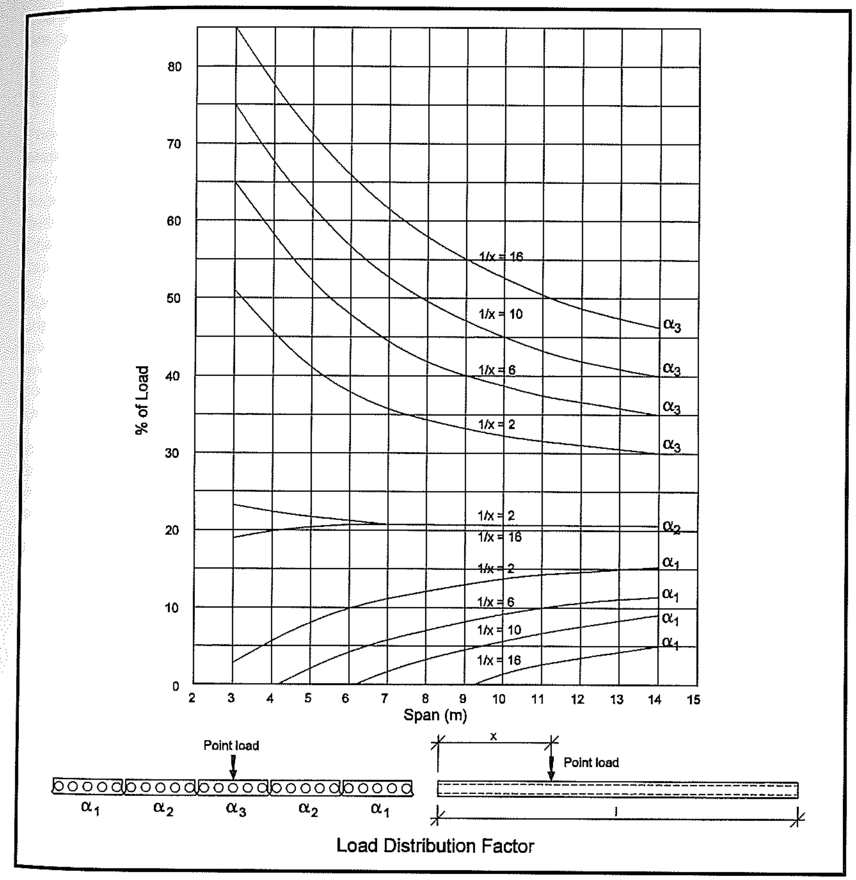
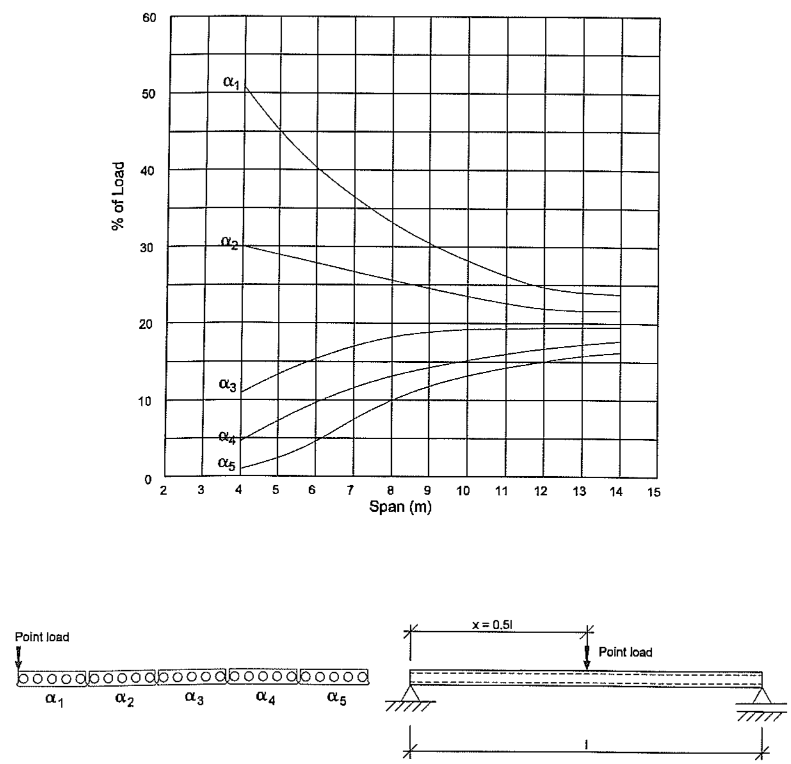


Figure 2.12 Load Distribution Factors For Point Load In The Centre Area Of Hollow-Core Slab Floors With Topping



Load Distribution Factor

Figure 2.13 Load Distribution Factors For Mid-span Point Load At The Edge Of The Slab Field Without Topping

2.2 Design Of Precast Concrete Planks

2.2.1 General

Solid precast concrete floor planks have the advantages of smooth soffit finish, no formwork and rapid assembly at site. The planks are laid between supporting members and used as permanent formworks for in-situ topping concrete which may vary between 50 mm and 200 mm thickness. In terms of performance, the precast floors are considered equal to that of cast in-situ construction.

The precast planks are usually 75 mm or 100 mm thick although planks of 80 mm and 110 mm thick are common in the market. The choice of thickness is primarily based on :

1. construction economy, and
2. preferred method of construction

The planks can be designed either ordinarily reinforced or prestressed. Steel bars, meshes or prestressing tendons are placed in the precast units as flexural bottom reinforcement. Steel meshes are generally incorporated in the topping concrete to serve both as structural floor ties as well as negative reinforcement over the support if the slabs are designed as continuous structure. Minimum design concrete grade is generally C35 for ordinary reinforced and C40 for prestressed planks. Despite the introduction of steel meshes in the topping or in the precast units, the planks are usuallly designed as one-way spanning without utilising the two-way capabilities.

During the temporary installation stage, the planks are designed as simply supported with an unpropped span of up to a maximum of about 4 m and 6 m for 75 mm and 100 mm thick prestressed units respectively. For longer spans, intermediate props are needed. The effect of props must be taken into design considerations. The plank units are most critical when the self weight of the wet concrete topping is added to the self-weight of the plank. An allowance of 1.5 kN/m^2 of construction loads should generally be considered in checks during the temporary stage.

In the permanent condition, the hardened in-situ concrete topping provides the compression resistance and the flexural resistance in the span provided by the bottom reinforcing bars or prestressing tendons. The hogging moment resistance at the supports is provided by in-situ placed meshes or steel bars.

2.2.2 Prestressed concrete planks

Precast planks prestressed with pretensioned tendons offer the advantages of longer unpropped spans and the resistance to cracking due to handling.

The planks are usually designed to class 2 stresses. Class 3 structures are appropriate for thicker units with heavy imposed loads. The use of class 3 stresses in the plank design will facilitate detensioning of thicker units as they will be subjected to unbalanced effects due to eccentric prestressing force.

The design of prestressed planks is similar to those in the design of prestressed concrete and two design examples are presented to illustrate the various design considerations.

2.2.3 Reinforced concrete planks

The design of flexural resistance and shear follows the usual methods for reinforced concrete.

If the maximum bending moment at the temporary installation stage due to self weight and topping concrete is M_1 at a given section, the area of reinforcement is calculated as:

$$A_{s1} = \frac{M_1}{z_1 \times 0.87f_y}$$

where z_1 is the lever arm in the precast unit.

If the ultimate moment due to imposed loads, including the effect of props, is M_2, the total area of steel at the section is given as:

$$A_s = A_{s1} + A_{s2}$$

$$= \frac{1}{0.87f_y}\left(\frac{M_1}{z_1} + \frac{M_2}{z_2}\right)$$

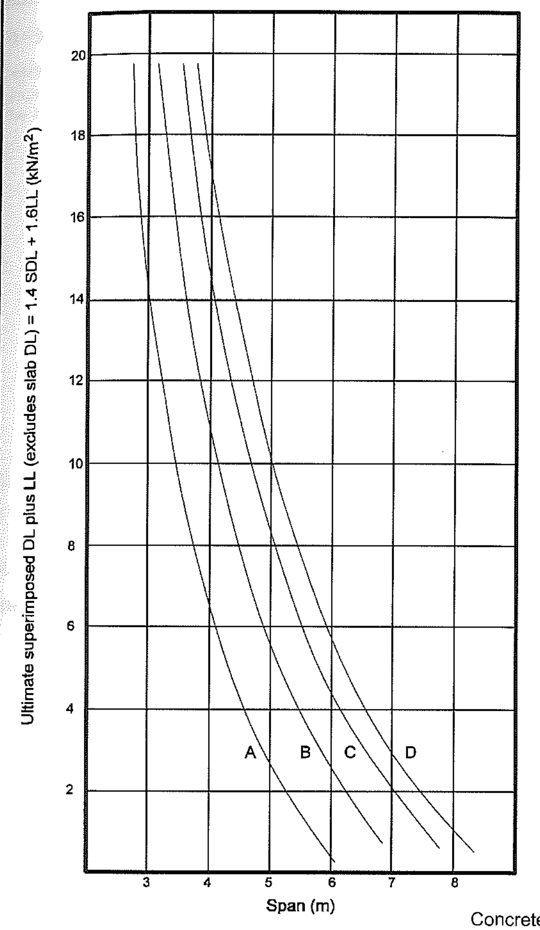
where z_2 is the lever arm in the composite section.

Vertical shear is rarely critical as there is a larger effective support width for the floor slabs. It may, however, be critical if there are openings which reduce the effective widths at the supports. Additional shear reinforcement in the form of stirrups projecting above the precast units may be provided, if necessary.

The effect of point and line loads is considered in the same manner as in cast in-situ construction. Interface shear is checked as per the code requirements in Part 1, clause 5.4.7.

2.2.4 Design charts

Two design charts, Figures 2.14 and 2.15, are presented as aids to the preliminary sizing of prestressed plank members for general design purposes.

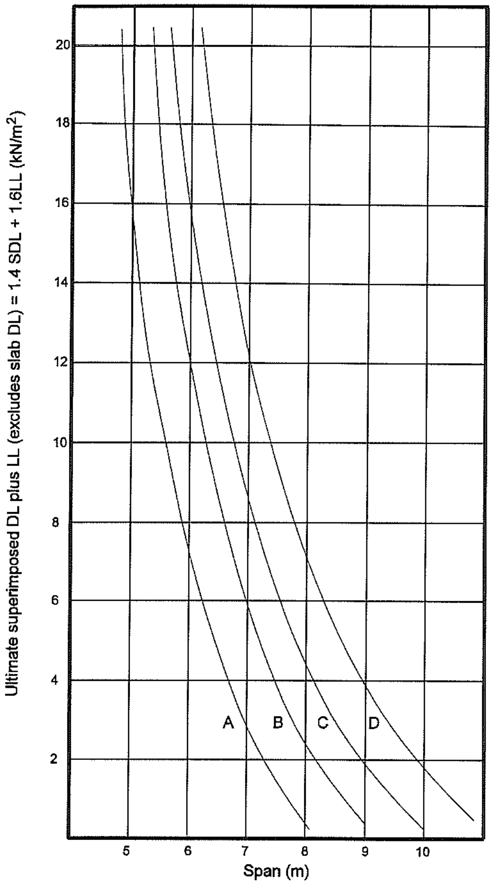


Concrete topping thickness = 55mm

Plank concrete: Grade 40
9.6mm diameter stress relieved strands conforming to BS 5896-1980, (grade 270k) at the following strand spacing:

Curve A	300mm c/c
Curve B	200mm c/c
Curve C	150mm c/c
Curve D	120mm c/c

Figure 2.14 Prestressed Solid Planks (80mm)



Concrete topping thickness = 55mm

Plank concrete: Grade 40
9.6mm diameter stress relieved strands conforming to BS 5896 - 1980, (grade 270k) at following strand spacing

Curve A	300mm c/c
Curve B	200mm c/c
Curve C	150mm c/c
Curve D	100mm c/c

Figure 2.15 Prestressed Solid Planks (110mm)

gn Example 1 : Unpropped Prestressed Precast Concrete Planks

gn the prestressing reinforcement in 80 mm thick pretensioned prestressed planks with a 3.6 m ly supported span. The planks will act compositely with 65 mm thick concrete topping eventually. design data are as follows :

Design loading :

Finishes	=	1.20 kN/m²
Services	=	0.50 kN/m²
Live load	=	2.00 kN/m²

Materials

a. Prestressing tendons :
- Ultimate tensile stress f_{pu} = 1860 N/mm²
- Initial prestress f_{pi} = 1395 N/mm² (75% of f_{pu})
- Prestress loss ratio (assumed) η = 0.75

b. Concrete :
- planks f_{cu} = 40 N/mm²
- topping f_{cu} = 35 N/mm²
- at transfer f_{ci} = 25 N/mm²
- cover to steel c = 30 mm

The plank is to be designed as class 2 structure.

The method of construction is unpropped.

Service Stress Design

ep 1 : **Calculate bending stresses at installation**

Loading

Dead load :
- plank s/w = 0.080 x 24 = 1.92 kN/m²
- topping = 0.065 x 24 = <u>1.56 kN/m²</u>
- Total = 3.48 kN/m²

Live load (construction) = 1.50 kN/m²

Moment at mid-span

a. Due to self weight, M_1 = 3.48 x 3.6²/8
= 5.64 kNm/m

Top and soffit concrete stress in plank f_c = ± 6M_1 / bh²
= ± (6 x 5.64 x 10⁶) / (1000 x 80²)
= ± 5.28 N/mm²

b. Due to s/w and construction live load M_1 = (3.48 + 1.50) x 3.6²/8
= 8.07 kNm/m

Top and soffit concrete stress in plank f_c = ±(8.07 x 10⁶ x 6) / (1000 x 80²)
= ± 7.56 N/mm²

Step 2 : **Calculate bending stresses at service**

Loading

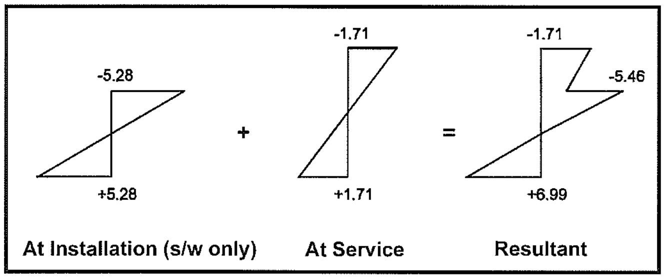
Dead load :	finishes	=	1.20 kN/m²
	services	=	0.50 kN/m²
			1.70 kN/m²
Live load		=	2.00 kN/m²
Total		=	3.70 kN/m²

At mid-span, moment M_2 = $3.70 \times 3.6^2/8$
= 6.00 kNm/m

Top and soffit concrete stress in plank f_c $= \pm (6.00 \times 10^6 \times 6) / (1000 \times 145^2)$
$= \pm 1.71$ N/mm²

Step 3 : **Resultant bending stress**

At mid span :



Note : +ve denotes concrete tensile stress
-ve denotes concrete compressive stress

Step 4 : **Calculate effective prestressing force and reinforcement**

Permissible tensile stress (class 2) at plank soffit (Part 1, clause 4.3.4.3)

$f_t = 0.45\sqrt{f_{cu}}$
= 2.8 N/mm²

Ignoring eccentricity effect of prestressing (e ≈ 5 mm), the required effective prestressing force in the plank is

$P_e = (6.99 - 2.8) \times 80 \times 1000 \times 10^{-3}$
= 335.2 kN/m

Effective prestress f_{pe} = ηf_{pi}
= $0.75 \times (0.75 \times 1860)$
= 1046 N/mm²

Hence A_{ps} = P_e / f_{pe}
= $335.2 \times 10^3/1046$
= 320 mm²/m

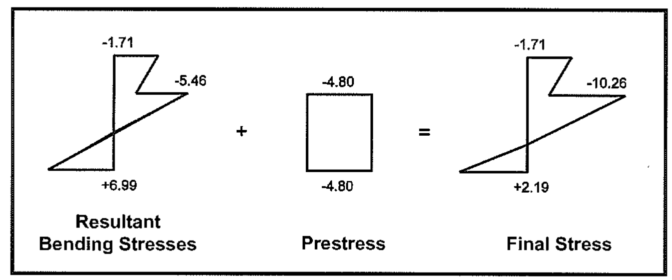
Use ϕ9.6 strands at 150 c/c (A_{ps} = 367 mm²/m)

Actual P_e = $367 \times 1046 \times 10^{-3}$
= 383.9 kN/m

Axial concrete stress in plank f_{cp} = $383.9 \times 10^3/(80 \times 1000)$
= 4.80 N/mm²

Step 5 : **Resultant concrete stresses**

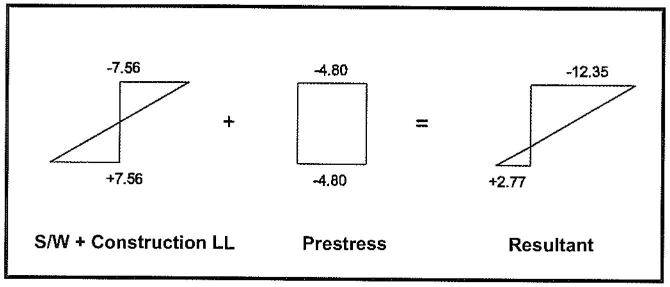
At mid span :



Maximum concrete tension at plank soffit
= 2.19 N/mm² < $0.45\sqrt{40}$
= 2.8 N/mm² OK

Maximum concrete compression at interface
= 10.26 N/mm² < $0.33 f_{cu}$
= 0.33×40
= 13.2 N/mm² OK

Step 6 : **Check mid-span concrete stresses during installation (assume taking place immediately after transfer)**



Maximum concrete tension = 2.77 < 2.8 N/mm² OK

Maximum concrete compression = 12.35 < $0.50f_{ci}$
= 0.50 x 25
= 12.5 N/mm² OK

Step 7 : **Deflection**

a. **At installation**

Total service load (including construction live load)
q = 3.48 + 1.50
= 4.98 kN/m²

Deflection $\delta = 5ql^4/(384E_cI)$

l = 3600mm
E_c = 28 kN/mm²
I = $bh^3/12$

$$\delta = \frac{5 \times 4.98 \times 3600^4 \times 12}{384 \times 28 \times 10^3 \times 1000 \times 80^3}$$

= 9.1 mm

δ/l = 1/395 < 1/250 (c1.3.2.1.1, Part 2) OK

b. **At service**

Total service load (imposed dead load + live load)
q = 1.70 + 2.0
= 3.70 kN/m²

$$\delta = \frac{5 \times 3.70 \times 3600^4 \times 12}{384 \times 28 \times 10^3 \times 1000 \times 145^3}$$

= 1.1 mm
= 1/3272 < 1/350 (c1.3.2.1.2, Part 2) OK

B. **Ultimate Limit State Design**

Step 8 : **Design for bending**

a. **At installation**

Ultimate UDL = 1.4 x 3.48 +1.6 x 1.50
= 7.30 kN/m²

Ultimate bending moment M_u = 7.30 x 3.6²/8
= 11.78 kNm/m

From Part 1, clause 3.4.4.4

$\chi/d = 1.11\,(1 - \sqrt{1 - 4.44M_u / bd^2f_{cu}})$

d = 80 - 35
= 45 mm

b = 1000 mm

$\chi/d = 1.11\,[\,1 - \sqrt{1 - (4.44 \times 11.78 \times 10^6/(1000 \times 45^2 \times 40))}\,]$
= 0.449

χ = 20.2 mm < d/2 = 22.5 mm

Concrete compression, $F_c = 0.45f_{cu}\,b\,\chi$
= 0.45 x 40 x 1000 x 20.2 x 10^{-3}
= 363.6 kN/m

b. **At final stage**

Ultimate UDL = 1.4 x 1.70 + 1.6 x 2.0
= 5.58 kN/m²

Ultimate bending moment M_u = 5.58 x 3.6²/8
= 9.04 kNm/m

d = 145 - 35
= 110 mm

$\chi/d = 1.11\,[\,1 - \sqrt{1 - (4.44 \times 9.04 \times 10^6/(1000 \times 110^2 \times 35))}\,]$
= 0.0539

χ = 5.9 mm

Concrete compression = $0.45f_{cu}\,b\,\chi$
= 0.45 x 35 x 1000 x 5.9 x 10^{-3}
= 92.9 kN/m

$f_{pu}A_{ps} / f_{cu}bd$ = 1860 x 367 / (35 x 1000 x110)
= 0.18

f_{pe} / f_{pu} = 1046 / 1860
= 0.56

Part 1,Table 4.4, $f_{pb} = 0.87f_{pu} \times 0.95$
= 0.83 x 1860
= 1537 N/mm²

Total tension to be provided by prestressing tendons,

P_s = 1537 x 367 x 10^{-3}
= 564.1 kN/m > 456.5 kN/m (=363.6 + 92.9) OK

Step 9 : Design for composite action (refer to Section 2.3.4)

Total horizontal force = 456.9 kN/m

Contact width b_e = 1000 mm
Contact length l_e = 3600/2
= 1800 mm

Average v_h = 456.9 x 10^3/(1000 x 1800)
= 0.25 N/mm² OK

For a triangular distribution of shear force, the maximum horizontal shear stress at the support is
$v_{h/max}$ = 0.25 x 2
= 0.50 N/mm²
which is less than 0.60 N/mm² in Table 5.5, Part 1 of the Code. Hence shear friction reinforcement is not necessary to ensure composite behaviour.

Step 10 : Design for vertical shear

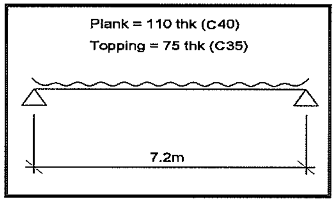
Total vertical shear at support

V = [1.4(3.48 +1.7) + 1.6 x 2.0] x 3.6/2
= 18.8 kN/m

v = 18.8 x 10^3/(1000 x 110)
= 0.17 N/mm² < 0.35 N/mm² (minimum) OK

Design Example 2 : Propped Prestressed Precast Concrete Planks

Design a 110mm thick prestressed pretensioned plank simply supported at 7.2 m and acting compositely with 75 mm thick concrete topping. All other design data are to be as per Design Example 1. Design live load is 2.5 kN/m² and the planks are to be propped at mid-span during installation.

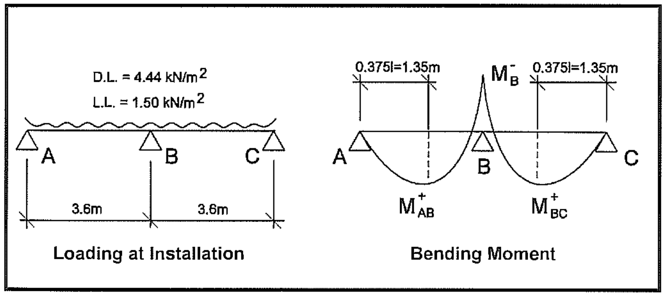


A. Service Stress Design

Step 1 : Calculate bending stress at installation stage.

Loading :

Dead load : plank self weight	= 0.110 x 24	= 2.64 kN/m²
topping	= 0.075 x 24	= 1.80 kN/m²
		4.44 kN/m²
Live load (construction)		= 1.50 kN/m²



Support :

M_B^- = 0.125ql²

a. Due to DL only : M_B^- = 0.125 x 4.44 x 3.6²
= 7.19 kNm/m

Top and soffit concrete stress in planks,
f_c = ± 7.19 x 10^6 x 6/(1000 x 110²)
= ± 3.57 N/mm²

b. Due to DL and construction live load :
M_B^- = 0.125 x (4.44 + 1.50) x 3.6²
= 9.62 kNm/m

Top and soffit concrete stress in planks,
f_c = ± 9.62 x 10^6 x 6/(1000 x 110²)
= ± 4.77 N/mm²

Span :

$M_{AB}^{+} = M_{BC}^{+} = 0.07ql^2$ at 1.35 m from end support

a. Due to DL only : $M_{AB}^{+} = 0.07 \times 4.44 \times 3.6^2$
$= 4.03$ kNm/m

Top and soffit concrete stress in planks,

$f_c = \pm 4.03 \times 10^6 \times 6/(1000 \times 110^2)$
$= \pm 2.00$ N/mm²

b. Due to DL and construction live load :

$M_{AB}^{+} = 0.07 \times (4.44 + 1.50) \times 3.6^2$
$= 5.39$ kNm/m

Top and soffit concrete stress in planks,

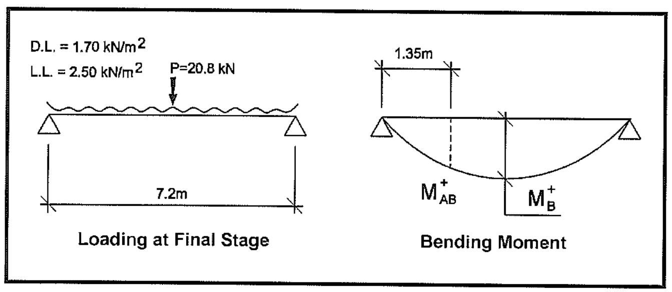
$f_c = \pm 5.39 \times 10^6 \times 6/(1000 \times 110^2)$
$= \pm 2.67$ N/mm²

Step 2 : Calculate bending stress at final stage

Loading

Dead load :	Finishes	= 1.20 kN/m²
	Services	= 0.50 kN/m²
		1.70 kN/m²
Live load		= 2.50 kN/m²
Total		= 4.20 kN/m²

Prop reaction at point B, P $= 1.30$ ql
$= 1.30 \times 4.44 \times 3.6$
$= 20.8$ kN/m



Loading at Final Stage **Bending Moment**

At mid-span :

$M_B^{+} = (4.20 \times 7.2^2/8) + (20.8 \times 7.2/4)$
$= 64.66$ kNm/m

Top and soffit concrete stress in composite section,

$f_c = \pm 64.66 \times 10^6 \times 6/(1000 \times 185^2)$
$= \pm 11.34$ N/mm²

At 1.35 m from support :

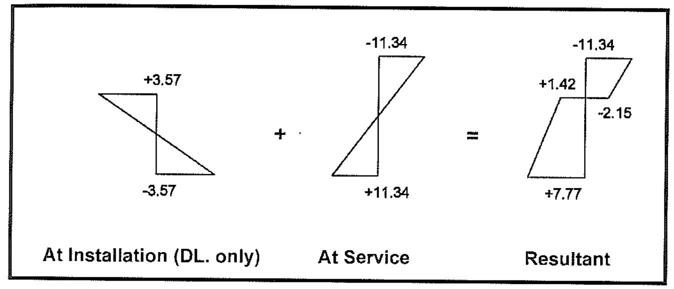
$M_{AB}^{+} = (10.4 + 15.12) \times 1.35 - 4.20 \times 1.35^2/2$
$= 30.62$ kNm/m

Top and soffit concrete stress in composite section,

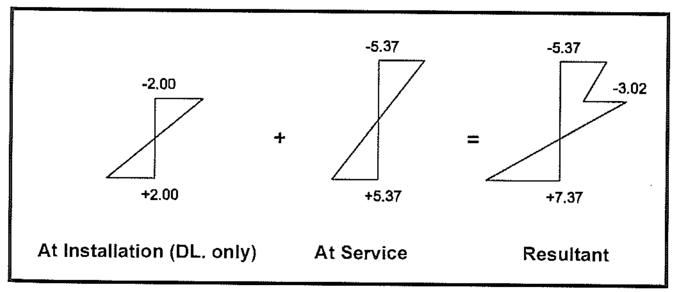
$f_c = \pm 30.62 \times 10^6 \times 6/(1000 \times 185^2)$
$= \pm 5.37$ N/mm²

Step 3 : Resultant bending stress

a. At mid span :



b. At 1.35m from support :



Maximum concrete tensile stress is at mid-span at 7.77N/mm²

Step 4 : **Calculate effective prestressing force and reinforcement**

Permissible tensile stress at plank soffit (class 2)
$f_t = 0.45\sqrt{f_{cu}}$
$= 2.8\ N/mm^2$

Eccentricity of prestressing force to plank centroid
$e = (110/2) - 35$
$= 20\ mm$

Concrete stress at the soffit of plank at mid-span,

$7.77 - P_e / A_c - P_e / Z_b = 2.8$

$7.77 - P_e / (1000 \times 110) - P_e \times 20 \times 6 / (1000 \times 110^2) = 2.8$

$P_e = 261.5\ kN/m$

Effective prestress $f_{pe} = \eta f_{pi}$
$= 0.75 \times (0.75 \times 1860)$
$= 1046\ N/mm^2$

Hence $A_{ps} = P_e/f_{pe}$
$= 261.5 \times 10^3/1046$
$= 250\ mm^2/m$

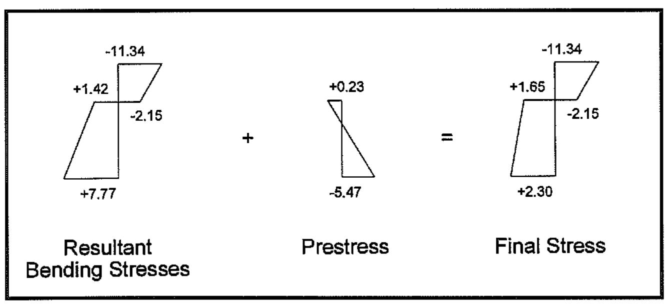
Use ϕ9.6 strands at 200 c/c ($A_{ps} = 275\ mm^2/m$)

Actual $P_e = 287.7\ kN/m$

$P_e/A_c = -2.62\ N/mm^2$

$P_e\, e / Z_b = \pm (287.7 \times 20 \times 10^3 \times 6)/(1000 \times 110^2)$
$= \pm 2.85\ N/mm^2$

Step 5 : **Resultant final concrete stresses**



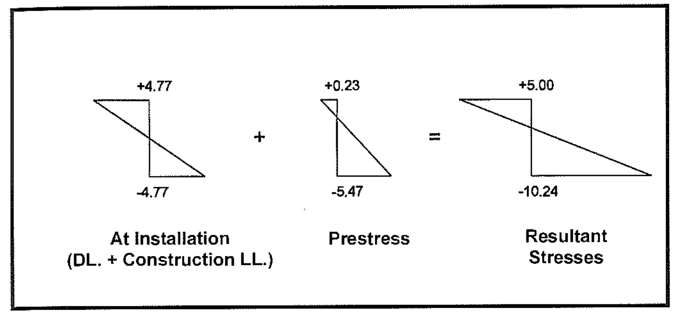
Stresses At Mid-Span

Maximum tensile stress $= 2.30 < 2.80\ N/mm^2$ OK

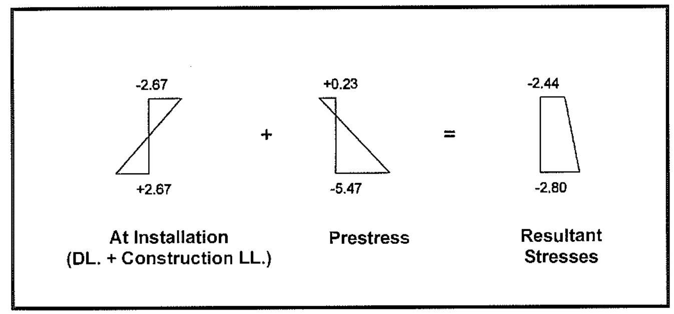
Maximum compression $= 11.34 < 0.33\ f_{cu}$ OK
$= 11.5\ N/mm^2$

Step 6 : **Check stresses at installation**

a. At Propping Support :



b. At 1.35m From End Support :



Maximum stresses are at the propping point.
Maximum compression $= 10.24\ N/mm^2 < 0.5\ f_{ci}$ $(0.5 \times 25 = 12.5 N/mm^2)$ OK

Note: Maximum tension found at the top face of the plank is $+ 5.00\ N/mm^2$. This is within the class 3 hypothetical tensile stress of $5.5 N/mm^2$ (0.2 mm crack width) for C40 concrete. The top section will eventually be under permanent compression as the interface is above the neutral axis of the composite action. Another point to note is that the construction live load is transient and when it is removed, the actual tension at the top face over the propping support is $(3.57 + 0.23) = 3.8\ N/mm^2$, which is well within class 3 stresses with 0.1 mm crack width. Cracks at the top face over the prop, if any, will not affect the structural integrity of the planks.

Step 7 : Check concrete stresses at transfer

Assume prestress loss ratio 0.9 at transfer

f_{pi} = 0.9 x 0.75 x 1860
= 1256 N/mm²

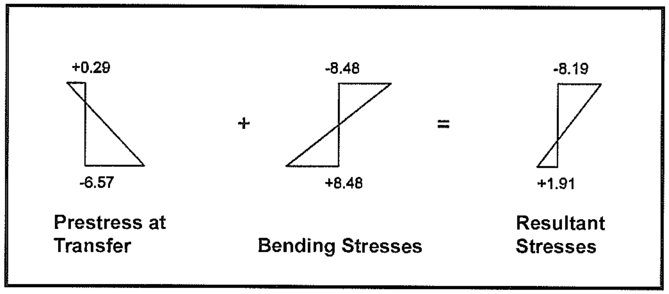
P_i = 1256 x 275 x 10^{-3}
= 345.4 kN/m

f_{ci} = -345.4 x 10^3/(1000 x 110)
= -3.14 N/mm²

$P_i\, e/Z_b$ = ± (345.4 x 20 x 10^3 x 6)/(1000 x 110^2)
= ± 3.43 N/mm²

Assuming that the plank is simply supported after transfer, the bending stresses due to a self-weight moment of 2.64 x 7.2^2/8 = 17.11 kNm/m at mid-span is

f_c = ± (17.11 x 10^6 x6)/(1000 x 110^2)
= ± 8.48 N/mm²



Maximum compression = -8.19 N/mm²
Maximum concrete compressive strength required at transfer
f_{ci} = 8.19/0.5
= 16.4 N/mm² < 25 N/mm² OK

Step 8 : Deflection

Deflection at installation is not critical as the planks are centrally propped.

Deflection at final stage :

$\delta = 5ql^4 / 384E_cI + Pl^3 / 48E_cI$
q = 1.70 + 2.50 = 4.20 kN/m²
P = 20.8 kN/m
E_c = 28 kN/mm²
I = $bh^3/12$
l = 7200mm

δ = 5 x 4.20 x 7200^4 x 12 / (384 x 28 x 10^3 x 1000 x 185^3) +
20.8 x 10^3 x 7200^3 x 12 / (48 x 28 x 10^3 x 1000 x 185^3)
= 9.9 + 10.9
= 20.8 mm

δ/l = 20.8/7200
= 1/346 ≈ 1/350 OK

B. Ultimate Limit State Design

Step 9 : Design for bending moment

Ultimate UDL = 1.40 x 1.70 + 1.60 x 2.50
= 6.38 kN/m²

Ultimate prop reaction = 1.4 x 20.8
= 29.1 kN/m

At mid-span : M_u = 6.38 x 7.2^2 / 8 + 29.1 x 7.2 / 4
= 93.7 kNm/m

Negative moment at mid-span moment during installation stage (step 1 (a))

M_u^- = 1.4 x 7.19
= 10.1 kNm/m

Hence net mid-span moment
M_u = 93.7 - 10.1
= 83.6 kNm/m

χ/d = 1.11[1 - $\sqrt{1 - (4.44M_u/bd^2f_{cu})}$]
d = 185 - 35
= 150 mm

χ/d = 1.11[1 - $\sqrt{1 - (4.44 \times 83.6 \times 10^6/(1000 \times 150^2 \times 35))}$]
= 0.303
χ = 45.4 mm

Concrete compression = $0.45f_{cu}\, b\, \chi$
= 0.45 x 35 x 1000 x 45.4 x 10^{-3}
= 715.0 kN/m

Total tension to be provided by prestressing tendons

= $0.87f_{pu}A_{ps}$ x 1.0 (From Part 1, Table 4.4)
= 0.87 x 1860 x 275 x 10^{-3} x 1.0
= 445.0 kN/m

Additional tension capacity to be provided by normal reinforcement

A_s = (715 - 445) x 10^3/(0.87 x 460)
= 675 mm²/m

Use T10 @ 100 c/c (A_s = 785 mm²/m), placed at the same level with the tendons.

Step 10 : Design for composite action

Total horizontal force = 715.0 kN/m
Contact width, b_e = 1000 mm
Contact length, l_e = 7200/2
= 3600 mm
Average v_h = 715.0 x 10^3/(1000 x 3600)
= 0.20 N/mm² OK

Proportioning to shear force distribution maximum horizontal shear stress at support

v_{hmax} = 2 x 0.20
= 0.40 N/mm²

which is less than 0.6 N/mm² (Table 5.5, Part 1) OK

Step 11 : Design for vertical shear

Total vertical shear at support

$V = 6.38 \times 3.6 + 29.1/2 + 1.4 \times 4.44 \times 0.375 \times 3.6$
$= 45.9$ kN/m

$v = 45.9 \times 10^3/(1000 \times 150)$
$= 0.30$ N/mm^2 < min 0.35 N/mm^2 OK

2.3 Design Of Precast Reinforced Concrete Beams

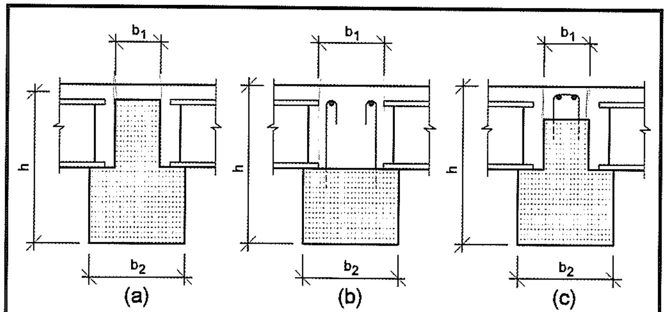
2.3.1 Design considerations

The design of precast reinforced concrete beams is affected by the following factors :

1. section properties of the precast beam,
2. construction methods,
3. sequence of the loads applied onto the beams, and
4. beam behaviour at the serviceability and ultimate limit state

2.3.2 Beam sections

Precast beams may be designed in either full, semi-precast or shell sections depending on the fabrication, jointing details, handling, delivery and lifting capacities of the cranes. The widths and depths in Figure 2.16 may be used in the design of the beam sections:



Flexural bending reinforcement use $\approx b_1$ at span
$\approx b_2$ at support (for continuous beam)
Shear reinforcement $\approx b_1$
Composite shear links $\approx b_1$ in (b) and (c)

Note : (a) full precast section
(b), (c) semi-precast sections with composite topping concrete

Figure 2.16 Effective Widths And Depths In Precast Beams Design

2.3.3 Construction methods and loading considerations

Figure 2.17 illustrates the various methods in construction using precast beams which can be broadly grouped into propped or unpropped construction with full or semi-precast sections. The final beam behaviour can be either simply supported or with semi-rigid or rigid moment connection at the supports for continuous composite beam behaviour.

At the installation stage, the load consists of essentially the self-weight of the beam, floor elements and wet concrete topping. In unpropped construction, the loads are carried wholly by the precast beams whereas in propped construction, part or all of the loads will be transferred to the props.

On removing the props, additional moments and shears will be created by the prop reactions which will be carried by the composite action of the beams. Precise instructions must, therefore, be given on the method of construction of the precast beams and the positions of the props if they are required.

At the service stage, the stresses in the beams are primarily due to imposed dead and live loads. Depending on the construction methods, the loading considerations on the beam design can be categorised into the following cases:

1. **unpropped construction with simply-supported beam behaviour or continuously propped precast beam :**

   The loads are applied as in the conventional cast in-situ beams design.

2. **unpropped construction with full or semi-precast section with continuous beam behaviour :**

   Apart from the dead and live loads, the beams are subjected to an additional live load of:

   0.4/1.6 x (beam self-weight + floor element + wet concrete)

   The load is treated as live load and is applied in order to satisfy the critical loading arrangement required in Part 1, clause 3.2.1.2.2, of the Code.

3. **propped construction with semi-precast section and continuous beam behaviour :**

   In addition to the imposed dead and live loads and the equivalent live load in (2) above, the beams will also be subjected to the action of prop forces. These are applied as point loads acting vertically downwards at the respective position of the props.

   The loading consideration of the semi-precast continuous beams in (2) and (3) above is illustrated in Figure 2.18.

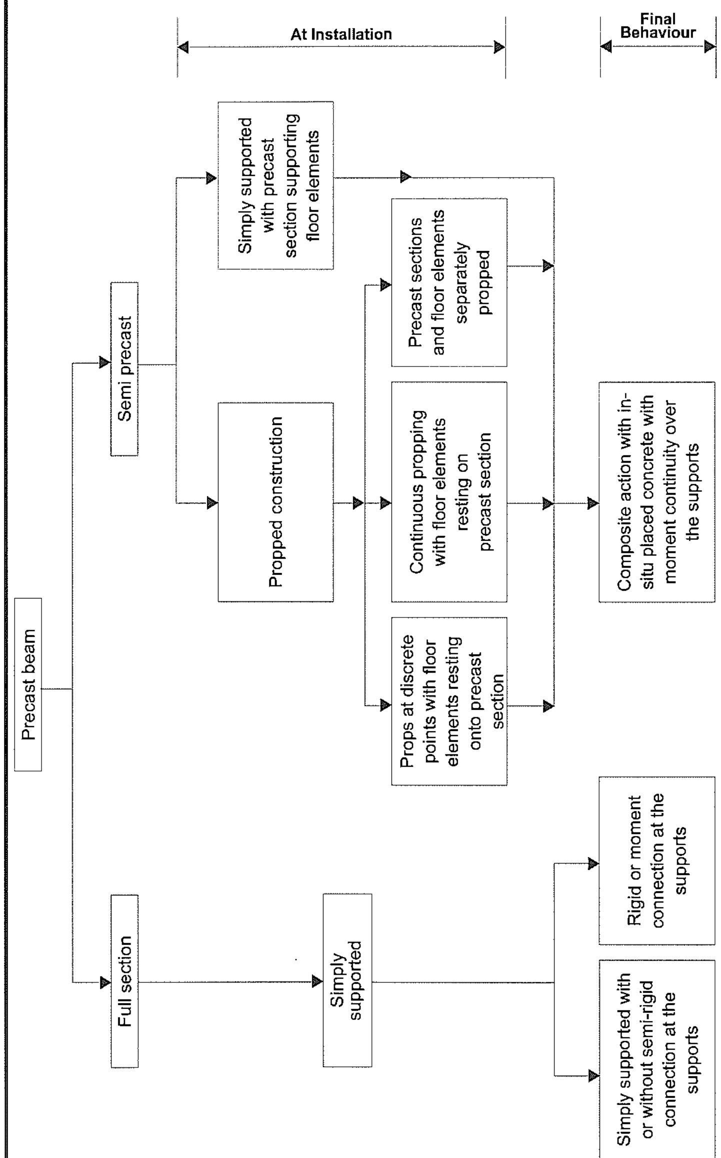
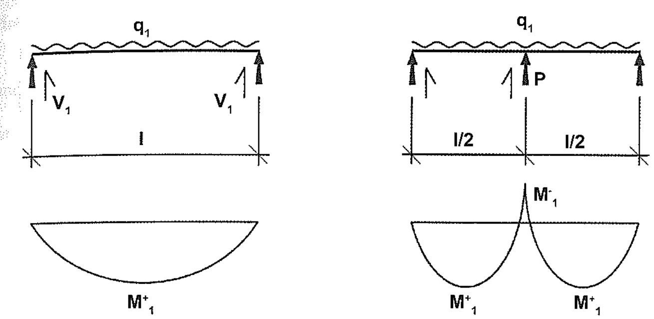


Figure 2.17 Precast Concrete Construction Methods And Beam Behaviours

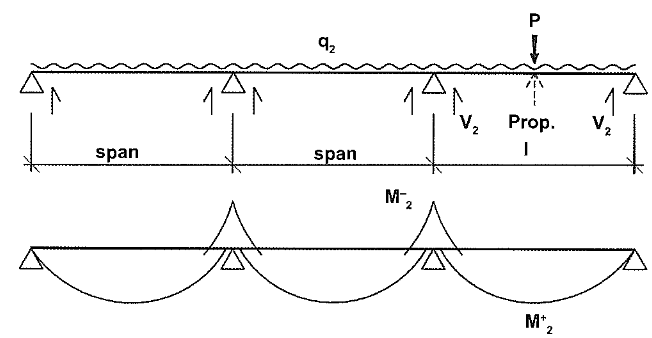


Unpropped Construction

Propped Construction

q_1 = Weight of beam + floor elements + wet concrete topping (load factor = 1.0)

(a) Semi-Precast Beams Supporting Floor Elements



q_2 = 1.4 x imposed dead load + 1.6 x (imposed live load + 0.4/1.6 x q_1)
P = reverse prop reaction (load factor = 1.4)

(b) Continuous Composite Beams At Ultimate Limit State

The final design moment is given as

Support moment $= M_2^-$
Span moment $= 1.0M_1^+ + M_2^+$

The design shear force at support is given as

Total shear $V = 1.0V_1 + V_2$

Figure 2.18 Loading Consideration In Semi-Precast Continuous Beam Design

2.3.4 Design for composite action

Design for composite action may follow the procedures under Part 1, clause 5.4.7, of the Code.

The determination of horizontal shear forces in composite design is shown in Figure 2.19 below :

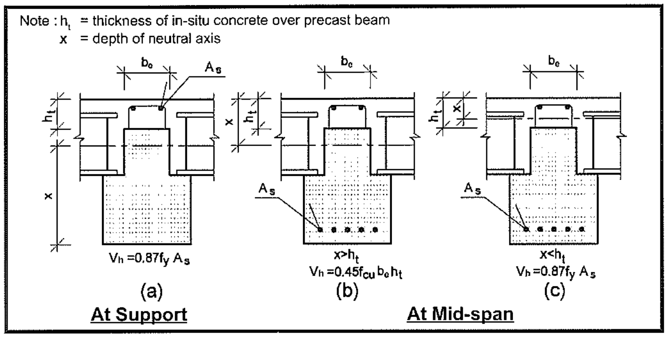


Figure 2.19 Horizontal Shear Force In Composite Concrete Section

The effective contact lengths may be determined as shown in Figure 2.20 :

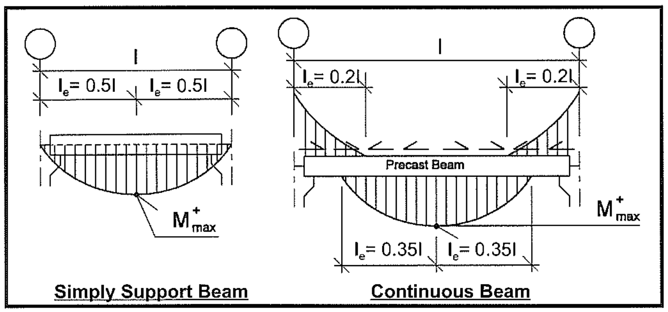


Figure 2.20 Effective Contact Lengths In Composite Design

The required shear links for composite action are calculated from :

$$\frac{A_{sv}}{s_v} = \frac{b_e v_h}{0.87 f_{yv}}$$

where
- b_e = effective contact width
- A_{sv} = area of shear links
- s_v = spacing of shear links
- v_h = average horizontal shear stress = $V_h/(b_e l_e)$

If the average horizontal shear stress is less than the permissible values in Table 5.5 of the Code, only norminal links equivalent to 0.15% of the contact area need to be provided. It is to be noted that the provisions of shear links is based on the larger of the requirements for vertical and horizontal shear and not addition of the two values.

2.3.5 Deflection

Deflection under serviceability requirements can be based on the effective span/depth approach in Part 1, clause 3.4.6 of the Code. When deflection needs to be determined, it may be calculated using the method outlined in Part 2, clause 3.7, BS 8110, which is given by:

$$\delta = Kl^2/r_b$$

and $$l/r_b = M_s/E_c I_e$$

where
- K = coefficient determined from Part 2, Table 3.1 of the Code
- l = span of the beam
- l/r_b = the mid-span curvature or, for cantilevers, at the support section
- M_s = bending moment at span or, for cantilevers, at the support section
- I_e = the effective moment of inertia of the beam
- E_c = modulus of elasticity of concrete

The effective moment of inertia of the beam I_e is calculated from :

$$I_e = (M_{cr} / M_s)^3 I_g + [1 - (M_{cr} / M_s)^3] I_{cr}$$

where
- M_{cr} = cracking moment (= $0.67\sqrt{f_{cu}} Z_b$)
- M_s = service load moment
- I_g = gross uncracked moment of inertia of the beam
- I_{cr} = cracked moment of inertia of the beam
- f_{cu} = design concrete cube strength
- Z_b = gross uncracked section modulus at tension face

In simply supported beams, I_e is calculated based on the mid-span value. For continuous beam, a weighted average of the support and span is more appropriate due to the varying degree of cracking at these two regions. The weighted average I_e is calculated from :

continuous span	$I_e = 0.70\, I_{em} + 0.15(I_{e1} + I_{e2})$
continuous span with simply supported at one end	$I_e = 0.85\, I_{em} + 0.15\, I_{e1}$

where
- I_{em} = effective moment of inertia for the mid-span
- I_{e1}, I_{e2} = effective moment of inertia for the negative moment sections at the beam ends

For long-term deflection calculations, effective modulus of elasticity $E_{ce} = E_c / (1 + \phi)$ is used where ϕ is the creep coefficient. In general, E_{ce} may be taken to be equal to about $0.5E_c$ unless a more precise ϕ has to be determined.

2.3.6. Crack width

Calculation of crack widths can follow the procedures in Part 2 clause 3.8, of the Code. The calculation of crack widths for beams with concrete covers under normal exposure conditions may not be necessary if the bar spacing rules in Part 1 clause 3.12.11, are observed. The permissible crack width is 0.3 mm for normal reinforced concrete section.

2.3.7 Design charts

The design charts for precast reinforced concrete beams are shown in Figures 2.21 to 2.25 for concrete grades of C30 to C50 respectively. In each of the design chart, the load capacity curves for beams with overall depths from 400 mm to 1200 mm at 100 mm increments are shown. The load capacity curves are based on a beam module of 50 mm width with simply supported conditions. The use of 50 mm module as a beam element has the following advantages :

1. For a given beam depth, the charts are able to cater to different beam widths with simple calculations. This reduces drastically the number of design charts.

2. It provides the designer quick means to explore other options in precasting the beams without carrying out extensive calculations.

The design charts are developed based on a main steel content of between 175 to 200 kg/m^3 of concrete. Including shear links and normal lappings, the steel contents in the final beam section may be 200 to 250 kg/m^3, which may be considered an optimum quantity in a beam section.

There are essentially three basic steps in the use of the charts:

Step 1. Calculate the ultimate uniform floor loading (i.e. 1.4DL + 1.6LL). Self weight of the precast beams need not be considered as the load capacity curves have incorporated the self-weight effect.

Step 2. Assume initially a beam width, b, which is in multiples of 50 mm and work out the total number of 50 mm modules for the assumed width i.e. n = b/50. Divide the ultimate uniform floor loads from Step 1 by n to obtain the loads on each beam module.

Step 3. Knowing the loaded span of the floor, the span of the precast beam and the uniform floor load on a 50 mm wide beam module, the designer can obtain the beam depth and hence the beam section from the design charts.

The design aids also include a bending steel design chart in Figure 2.26 and a permissible shear stress chart in Figure 2.27 which are derived from the code provisions in Part 1 clause. 3.4.4 and 3.4.5 respectively.

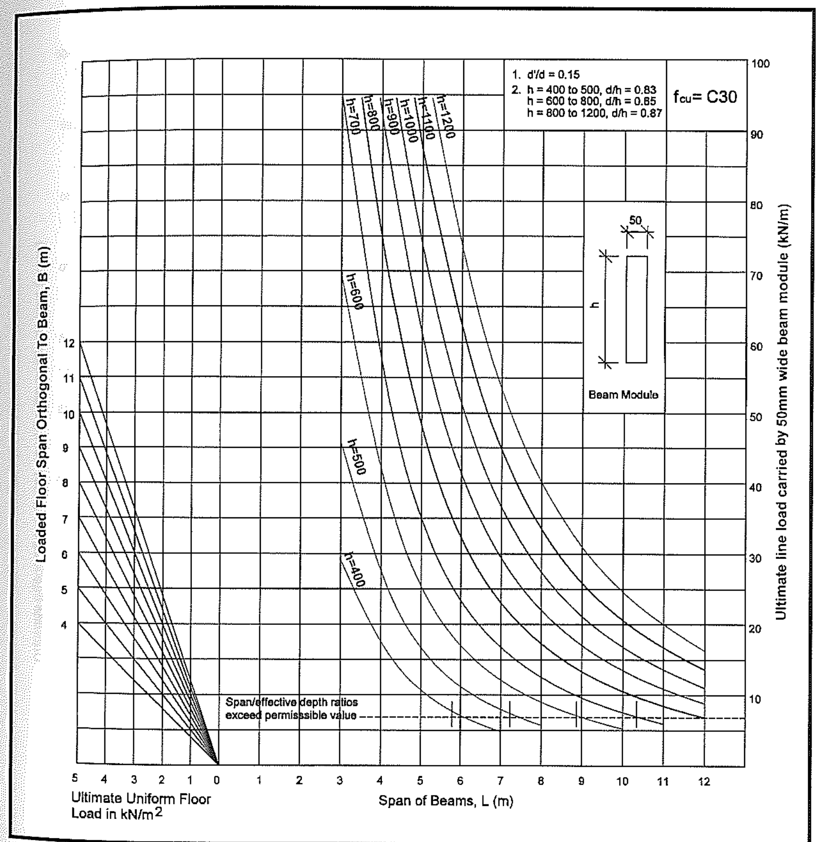


Figure 2.21 Reinforced Concrete Precast Beams Design Chart For f_{cu} = 30N/mm^2

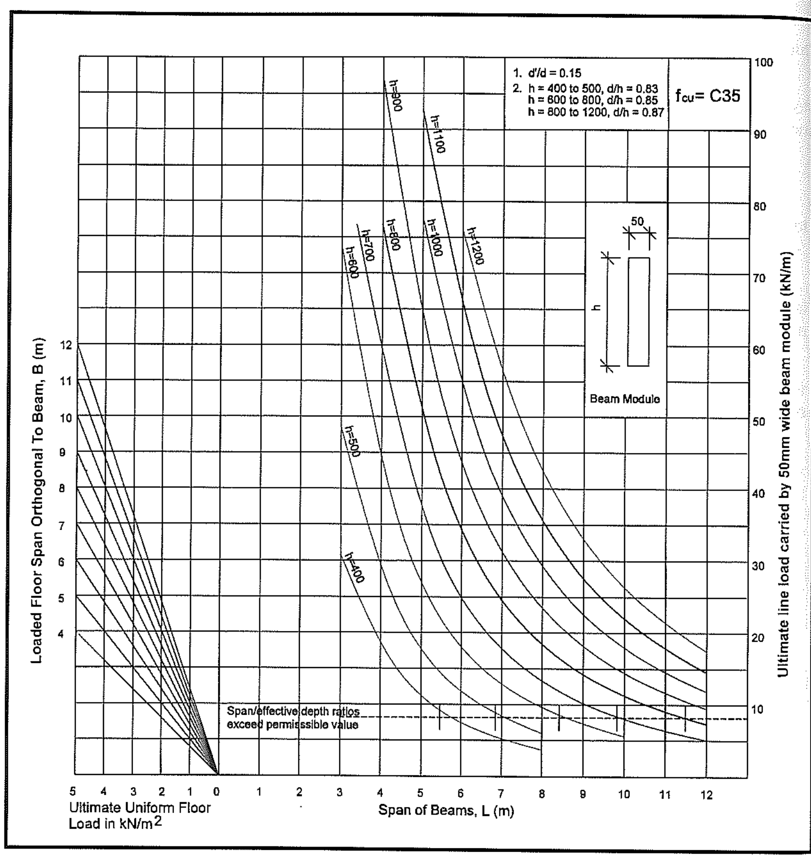


Figure 2.22 Reinforced Concrete Precast Beams Design Chart For f_{cu} = 35N/mm²

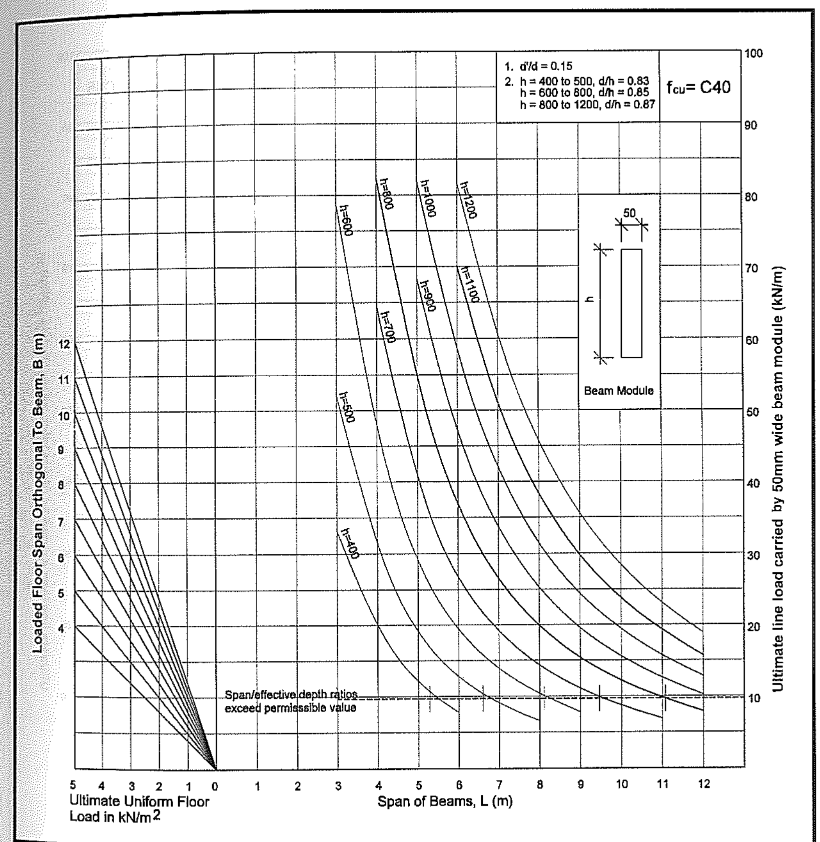


Figure 2.23 Reinforced Concrete Precast Beams Design Chart For f_{cu} = 40N/mm²

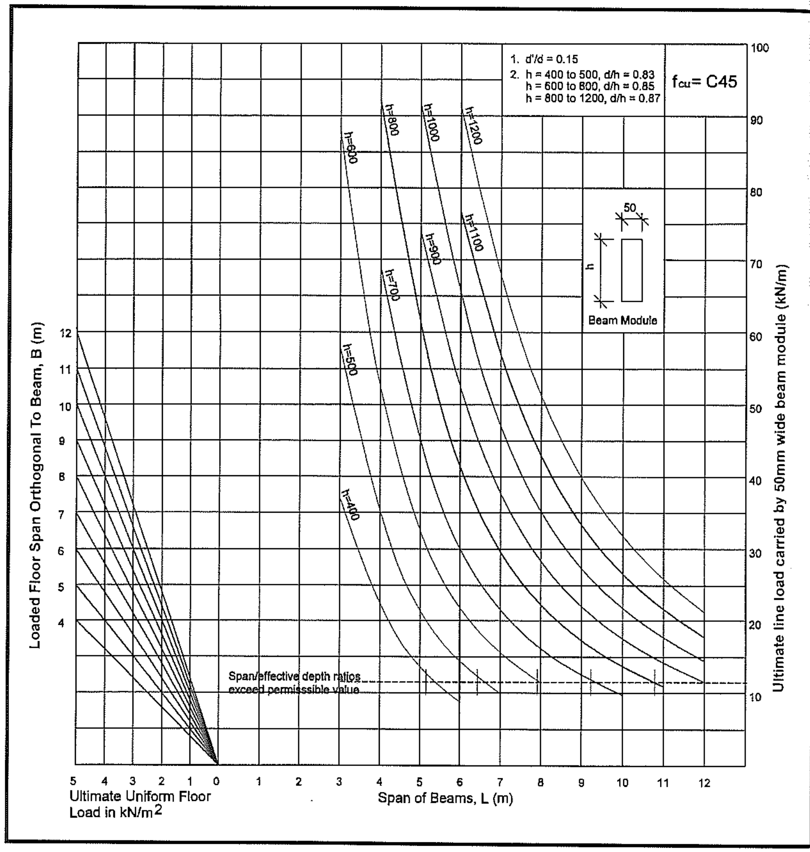


Figure 2.24 Reinforced Concrete Precast Beams Design Chart For f_{cu} = 45N/mm²

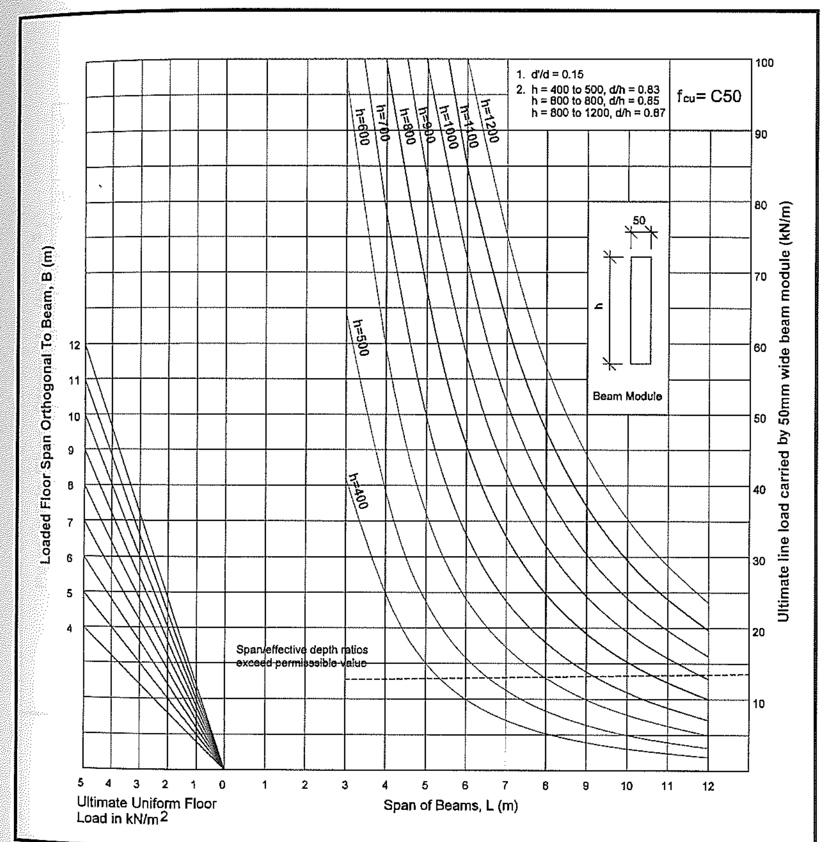


Figure 2.25 Reinforced Concrete Precast Beams Design Chart For f_{cu}=50N/mm²

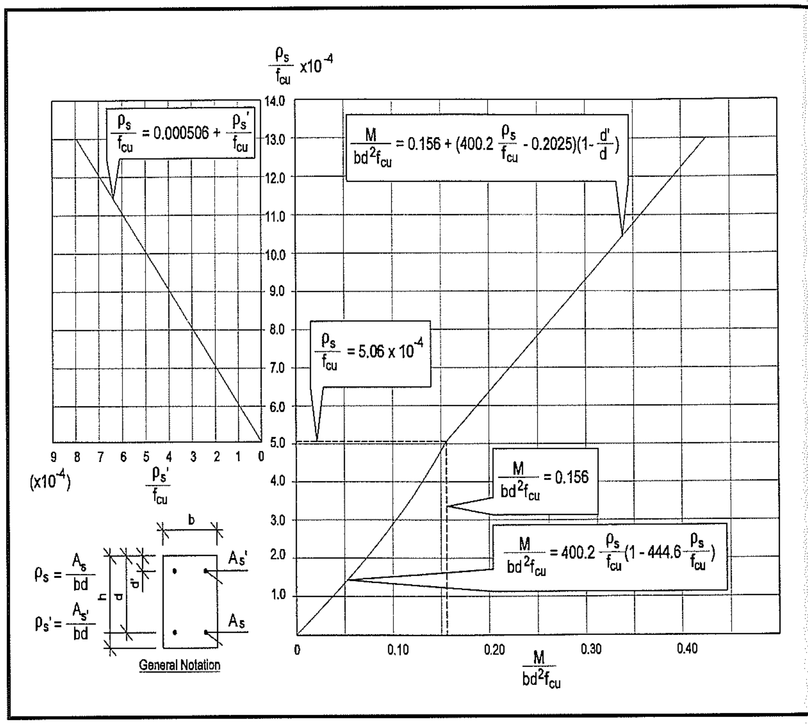


Figure 2.26 Bending Steel Design Chart

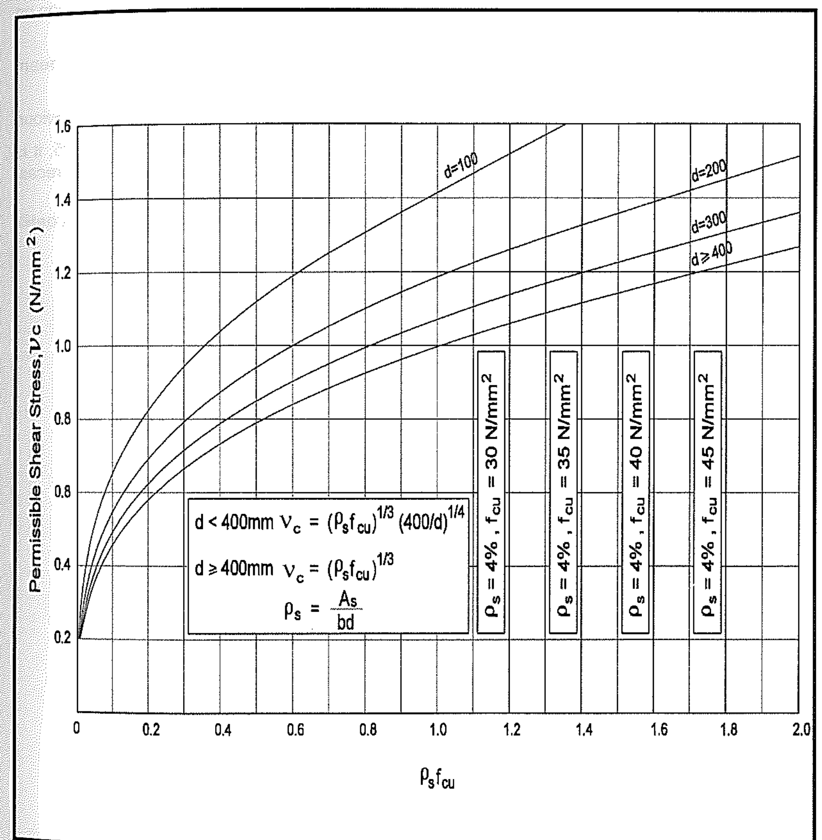


Figure 2.27 Permissible Concrete Shear Stress

Note : The ν_c values from the chart will need to be multiplied by a factor 1.063 to give the permissible ν_c value in CP65.

2.3.8 Design examples

To illustrate the use of the design charts, four design examples are presented. An internal precast beam, shown in the typical floor plan of an office in Figure 2.28 is used and the variations in the design examples are as follows:

Example 3 : Full precast section, unpropped construction and simply supported final beam behaviour. The overall depth of the beam is restricted by clear headroom requirement.

Example 4 : Resize and design of the precast beam in Example 3 if there is no restriction on the overall beam depth.

Example 5 : Semi-precast section, unpropped construction with final continuous beam behaviour.

Example 6 : Semi-precast section, propped construction with final continuous beam behaviour.

Although Example 6 does not involve the use of any of the charts in Figures 2.21 to 2.25, it is intended to illustrate the use of charts in Figures 2.26 and 2.27 which are applicable to both precast and conventional in-situ beam design.

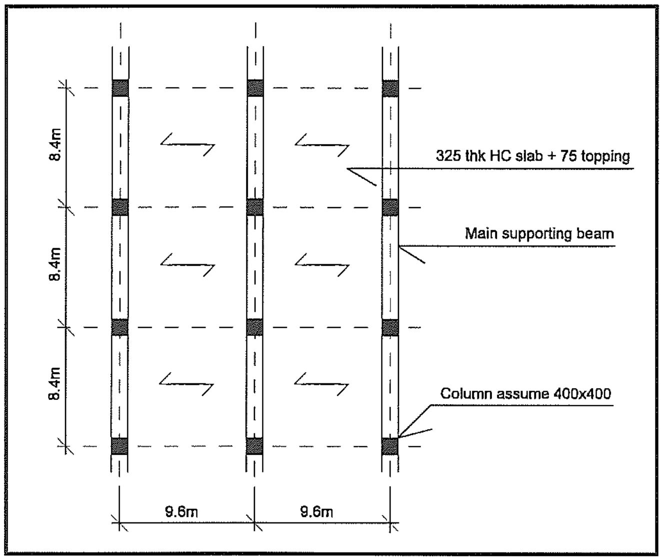


Figure 2.28 Typical Floor Plan For Precast Beam Design For Design Examples 3 to 6.

1. Design Loading :

Imposed dead load	:	Finishes	=	1.20 kN/m²
		Services	=	0.50 kN/m²
		Partition	=	1.00 kN/m²
Imposed live load			=	5.00 kN/m²

2. Materials

Concrete (C35)	f_{cu}	=	35 N/mm²
Steel	f_y	=	460 N/mm²

3. Net span of precast main beam = 8 m

Design Example 3: Full Section Precast Beam And Unpropped Construction

Design the simply supported main precast beams which are to be cast in full section. The beams are to be unpropped during installation. Due to ceiling height requirements, overall depth of the beam should not exceed 700 mm.

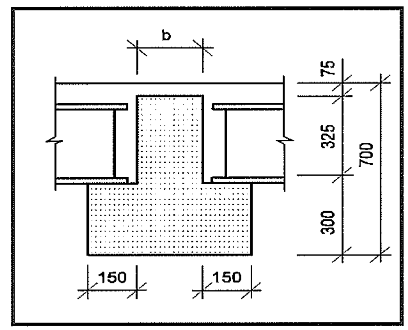
Step 1 : Calculate ultimate floor loading

Dead load:

HC slabs (jointed weight)	=	4.50 kN/m²
Topping (75 mm thk)	=	1.80 kN/m²
Finishes	=	1.20 kN/m²
Services	=	0.50 kN/m²
Partition	=	1.00 kN/m²
Total dead load	=	9.00 kN/m²
Live load	=	5.00 kN/m²
Ultimate UDL = 1.4DL + 1.6LL	=	20.6 kN/m²

Step 2 : Determine beam depth and width

Depth, h = 700 - 75
= 625 mm



Typical Beam Section

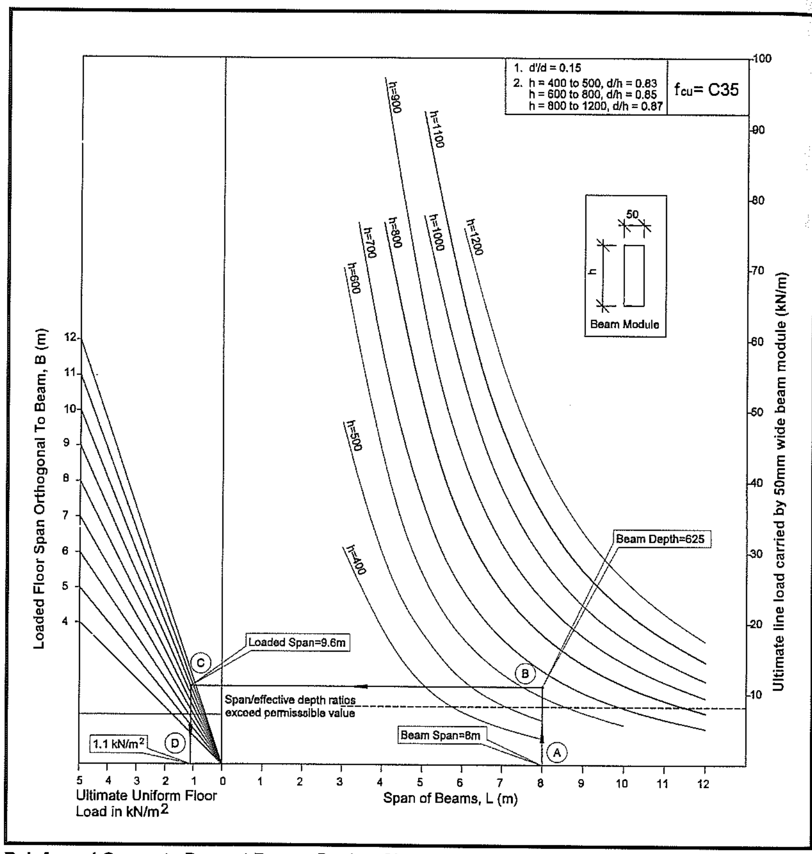
Using Figure 2.22 (concrete grade C35) and starting from Point A and following the sequence A⇨B⇨C⇨D (see following page for illustrations), an ultimate UDL for a 50mm wide beam module is determined to be 1.1 kN/m².

The number of beam modules for the total floor UDL of 20.6 kN/m² is derived from:

n = 20.6/1.1
= 18.7

Width, b = 18.7 x 50
= 935, say 950mm wide.

Adopt b = 950mm, h = 625mm



Reinforced Concrete Precast Beams Design Chart – (Design Example 3)

Step 3 : Calculate main steel reinforcement

Self-weight of beam $= (0.95 \times 0.625 + 0.15 \times 2 \times 0.3) \times 24$
$= 16.41$ kN/m

At mid-span, M $= [1.4 \times 16.41 + 20.6 \times 9.6]\ 8^2/8$
$= 1765.9$ kNm

h = 625mm, d say 515mm

$M / bd^2 f_{cu} = 1765.9 \times 10^6 / (950 \times 515^2 \times 35)$
$= 0.20$

Refer to Figure 2.26, for $M/bd^2 f_{cu} = 0.20$,

$\rho_s/f_{cu} = 6.3 \times 10^{-4}$
$\rho_s = 0.022$
$A_s = 10764$ mm²
Use 10T32 + 6T25 ($A_s = 10987$ mm²)

$\rho_s'/f_{cu} = 1.25 \times 10^{-4}$
$\rho_s' = 0.0044$
$A_s' = 2153$ mm²
Use 8T20 ($A_s' = 2514$ mm²)

Step 4 : Design for shear links

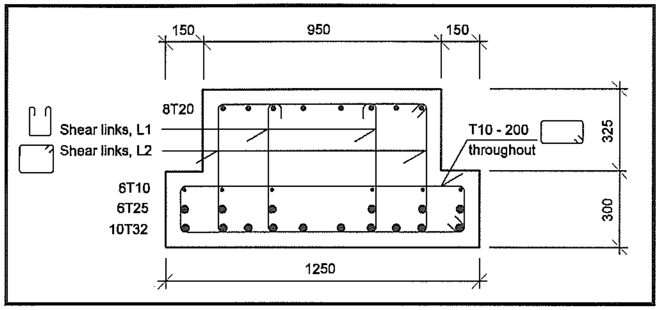
V $= 1.4 \times 16.41 \times 4 + 20.6 \times 9.6 \times 4$
$= 882.9$ kN

v $= 882.9 \times 10^3/(950 \times 515)$
$= 1.80$ N/mm²

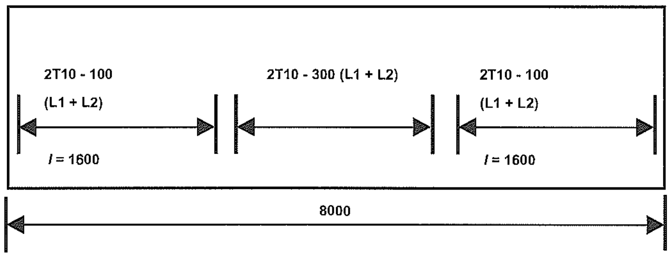
Assume ρ_s at the support to be 30% of mid-span A_s
$\rho_s f_{cu} = 0.231$
From Figure 2.27, $v_c = 0.61$N/mm²

$A_{sv}/ s_v = (1.80 - 0.61) \times 950/(0.87 \times 460)$
$= 2.82$
Use 2T10@100 mm for 1.6 m both ends ($A_{sv}/ s_v = 3.14$)
Remaining shear links to be 2T10@300 mm

Step 5 : Detailing



Typical Section



Distribution of Shear Links

Design Example 4: Full Section Precast Beam And Unpropped Construction

Re-design the precast beam in Design Example 3 if there is no restriction to the beam depth.

Step 1 : Calculate ultimate floor loading

As per Example 3, ultimate UDL = 20.6 kN/m^2

Step 2 : Determine beam depth

Assume beam width		=	500 mm
No. of beam modules	n	=	500/50
		=	10 nos.
UDL on each beam module		=	20.6/10
		=	2.06kN/m^2

Using Figure 2.22 and following the flow direction in the chart (shown on the following page), the depth of beam is about 825 mm at the intersection at point C. Including 75mm thick topping, the overall beam depth is 900 mm which is a reasonable beam dimension.

Adopt b = 500 mm, h = 825 mm

Step 3 : Calculate main steel reinforcement

Beam s/w = (0.5 x 0.825 + 0.15 x 2 x 0.5) x 24
= 13.5 kN/m

At mid-span, M = (1.4 x 13.5 + 20.6 x 9.6) x 8^2/8
= 1733.3 kNm

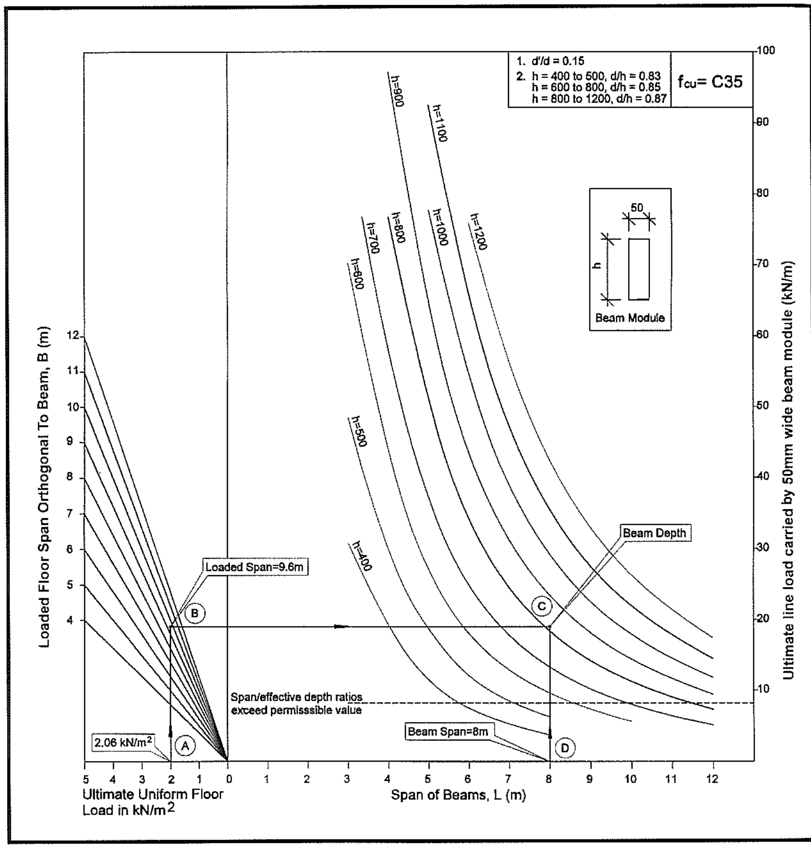
h = 825mm, d say 715mm

$M / bd^2 f_{cu}$ = 1733.3 x 10^6 / (500 x 715^2 x 35)
= 0.194

Refer to Figure 2.26, for M/bd^2f_{cu} = 0.194,

ρ_s/f_{cu} = 6.0 x 10^{-4}
ρ_s = 0.021
A_s = 7508 mm^2
Use 8T32 + 4T20 (A_s = 7690 mm^2)

ρ_s'/f_{cu} = 1.06 x 10^{-4}
ρ_s' = 0.0037
A_s' = 1326 mm^2
Use 5T20 (A_s' = 1571 mm^2)



Reinforced Concrete Precast Beams Design Chart – (Design Example 4)

Step 4 : Design for shear links

$V = (1.4 \times 13.5 + 20.6 \times 9.6) \times 4$
$= 866.6$ kN

$v = 866.6 \times 10^3/(500 \times 715)$
$= 2.42$ N/mm^2

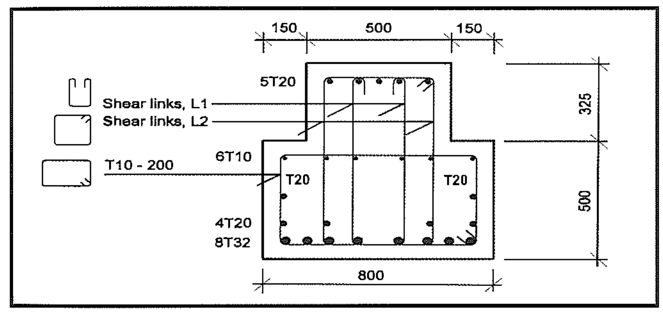
Assume ρ_s, to be 30% of mid-span A_s
$\rho_s f_{cu} = 0.22$

From Figure 2.27, $v_c = 0.60$ N/mm^2

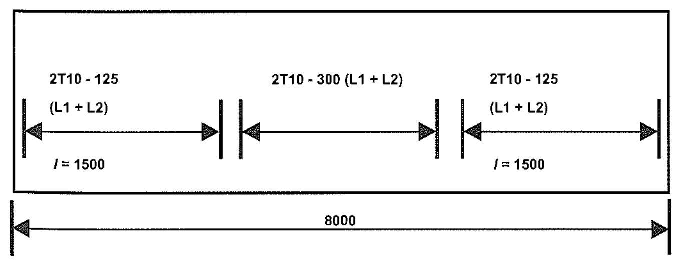
$A_{sv}/ s_v = (2.42 - 0.60) \times 500/(0.87 \times 460)$
$= 2.27$

Use 2T10@125 mm for 1.5 m both ends ($A_{sv}/ s_v = 2.51$)
Remaining shear links to be 2T10@300 mm

Step 5 : Detailing



Typical Section



Distribution Of Shear Links

Design Example 5: Semi Precast Beams And Unpropped Construction

Design the precast main beams which are to be semi-precast and unpropped during installation. The beams are designed to behave continuous at final stage. The design concrete grade for in-situ topping is C35.

A. At Installation stage

Step 1 : Calculate ultimate floor loading

Dead load :

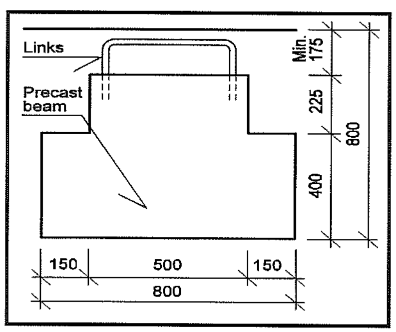
HC slabs (jointed weight)	=	4.50 kN/m^2
Topping (75 mm thk)	=	1.80 kN/m^2
Total	=	6.30 kN/m^2
Allow live load (construction)	=	1.50 kN/m^2
Ultimate UDL = 1.4DL + 1.6LL	=	11.22 kN/m^2

Step 2 : Determine beam depth and width

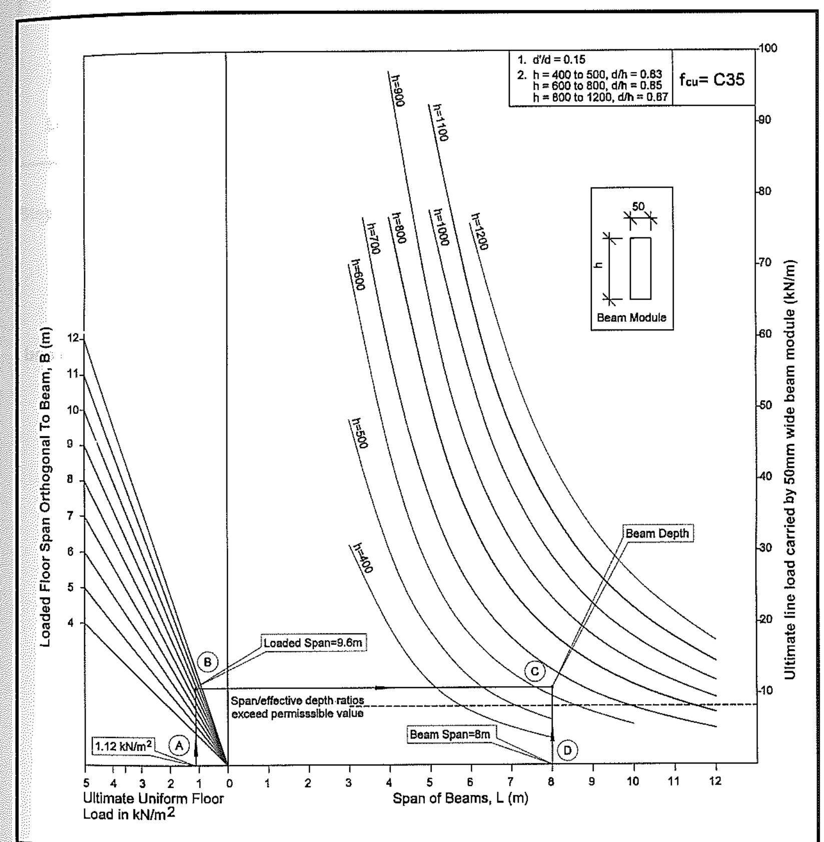
Assume beam width	=	500 mm
No. of beam modules, n	=	500/50
	=	10 nos.
UDL on each module	=	11.22/10
	=	1.12 kN/m^2

Using Figure 2.22 (see following page for illustrations) where the minimum semi-precast beam depth is 625 mm for an unpropped construction.

Adopt b = 500 mm, and h = 625 mm and overall beam depth = 800 mm as shown below.



Composite Beam Section



Reinforced Concrete Precast Beams Design Chart – (Design Example 5)

Step 3 : **Calculate main tension steel requirement at installation**

Beam s/w = (0.5 x 0.8 + 0.15 x 2 x 0.4) x 24
= 12.5 kN/m

At mid-span, M = $(1.4 \times 12.5 + 11.22 \times 9.6) \times 8^2/8$
= 1001.7 kNm

h = 625 mm, d say 530 mm

$M / bd^2 f_{cu}$ = $1001.7 \times 10^6 / (500 \times 530^2 \times 35)$
= 0.204

Refer to Figure 2.26, for $M/bd^2 f_{cu} = 0.204$,

ρ_s/f_{cu} = 6.41×10^{-4}
ρ_s = 0.0224
A_s = 5945 mm²

ρ_s'/f_{cu} = 1.40×10^{-4}
ρ_s' = 0.0049
A_s' = 1298 mm²
Use 5T20 (A_s' = 1571 mm²)

B. **At Service**

Step 4 : **Analysis of continuous beam behaviour under imposed dead and live load and an equivalent live load due to self weight of the structure**

Dead load :

Finishes	=	1.20 kN/m²
Services	=	0.50 kN/m²
Partition	=	<u>1.00 kN/m²</u>
		2.70 kN/m²

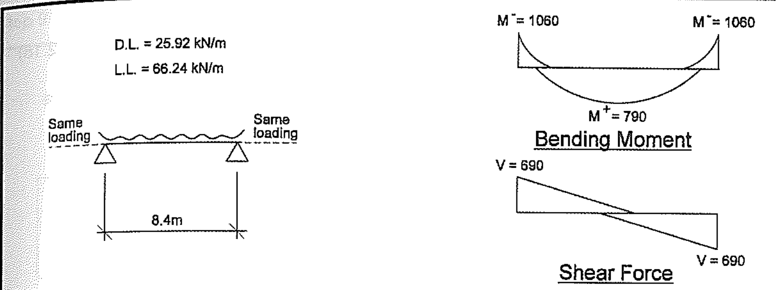
Total dead load = 2.70 x 9.6
= 25.92 kN/m

Live load = 5 x 9.6
= 48.0 kN/m

Equivalent live load = (0.4/1.6) x (12.5 + 6.3 x 9.6)
= 18.24 kN/m

Total live load = 48.0 + 18.24
= 66.24 kN/m

The moment and shear in final condition are shown in the following page.



Loading On Continuous Beam

Distribution Of Bending Moment And Shear (Typical Internal Beam)

Step 5 : **Design of main tension steel**

Span :
b = 500mm, h = 800 mm, d ≈ 700 mm

M^+ = 790 kNm

$M^+ / bd^2 f_{cu}$ = $790 \times 10^6 / 500 \times 700^2 \times 35$
= 0.092

Refer to Figure 2.26, for $M/bd^2 f_{cu} = 0.092$,

ρ_s/f_{cu} = 2.6×10^{-4}
ρ_s = 0.0091
A_{s2} = 3185 mm²

Total A_s = A_{s1} (From Step 3) + A_{s2}
= 5945 + 3185
= 9130 mm²
Use 9T32 + 4T25 (A_s = 9201 mm²)

Note : The above summation of A_s from installation and final stage for the mid-span main steel may appear to contradict Figure 2.20. However, the final result will be the same if a detail crack section analysis is carried out at both the installation and at the final stage and with the steel service stress limited to $5/8f_y$ (= 287 N/mm²). The calculation in Step 5 is

Support :
b = 800mm, h = 800 mm, d ≈ 700 mm

M^- = 1060 kNm

$M^- / bd^2 f_{cu}$ = $1060 \times 10^6 / (800 \times 700^2 \times 35)$
= 0.077

Refer to Figure 2.26, for $M/bd^2 f_{cu} = 0.077$

ρ_s/f_{cu} = 2.1×10^{-4}
ρ_s = 0.00735
A_s = 4116 mm²
Use 5T32 (A_s = 4021 mm²)

Step 6 : **Design shear links**

At support, $V = 690 + (12.5 + 6.30 \times 9.6) \times 4.0 \times 1.0$ (refer Figure 2.18)
$= 981.9$ kN

$v = 981.9 \times 10^3/(500 \times 700)$
$= 2.81$ N/mm²

ρ_s (5T32) $= 0.0115$ (b=500, refer to Figure 2.16)
From Figure 2.27, $v_c = 0.74$ N/mm²

$A_{sv}/s_v = (2.81 - 0.74) \times 500/(0.87 \times 460)$
$= 2.59$

Step 7 : **Shear links for composite action**

Support : contact length, $l_e \approx 0.2 \times 8.4 - 0.2$
$= 1.48$ m

$A_{sv}/s_v = A_s/l_e$
$= 4116/1480$
$= 2.78$

Span : contact length, $l_e \approx 0.35 \times 8.4$
$= 2.94$ m

Contact width, $b_e = 500$ mm

Neutral axis depth, $\chi = 0.87 f_y A_s/(0.45 f_{cu} \times b_e \times 0.9)$
$= (0.87 \times 460 \times 4116) / (0.45 \times 35 \times 800 \times 0.9)$
$= 145.2 \text{ mm} < 175 \text{ mm}$

Hence horizontal shear force,
$V_h = 0.45 f_{cu}\, b_e\, \chi$
$= 0.45 \times 35 \times 500 \times 145.2 \times 10^{-3}$
$= 1143.5$ kN

Average horizontal shear stress in mid-span,
$v_h = V_h/b_e l_e$
$= 1143.5 \times 10^3/(500 \times 2940)$
$= 0.78 \text{ N/mm}^2 < 1.9 \text{ N/mm}^2$ (Table 5.5 Part 1, BS 8110)

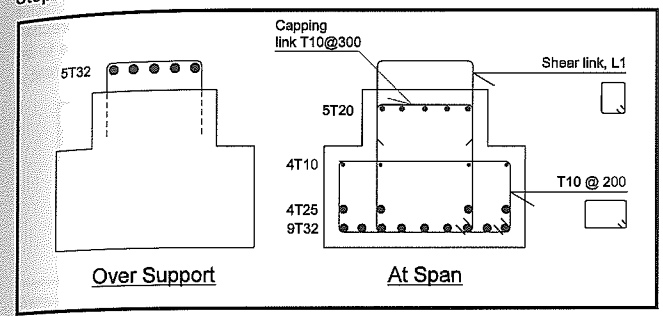
Step 8 : **Shear links provision**

Support : $A_{sv}/s_v = 2.78$ (the greater A_{sv}/s_v of Step 5 and Step 6)

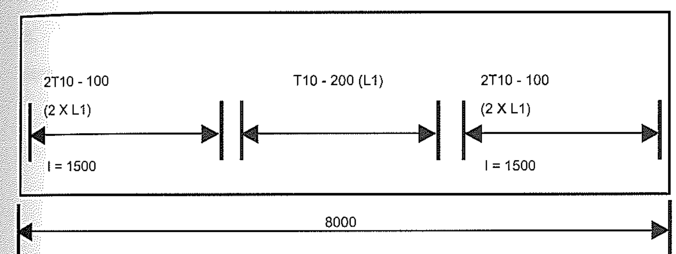
Use 2T10-100 links for 1.5 m both ends

Span : use nominal links T10-200

Step 9 : **Detailing**



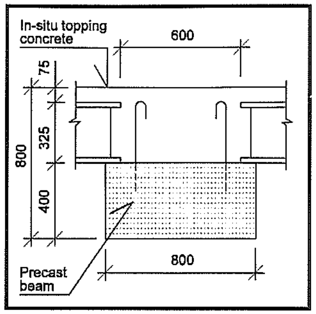
Typical Beam Section



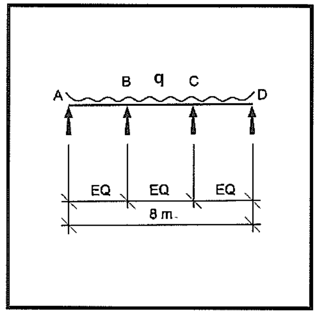
Distribution Of Shear Links

Design Example 6 : Semi Precast Beam And Propped Construction

Re-design the precast beam in Design Example 5 if only the bottom 400 mm deep section is precast. The beams are to be propped at 1/3 point during the floor slab installation. The beams are to be designed for continuous behaviour at the final condition. Topping concrete used is C35 concrete.



Typical Composite Section



Propping Of Precast Beam

A. At Installation Stage

Step 1 : Calculate prop reactions and check section strength

Dead load :

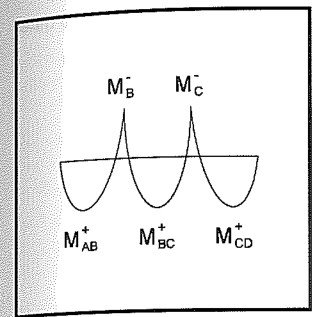
Self weight = 0.4 x 0.8 x 24	=	7.68 kN/m
HC slab = 4.5 x 9.0	=	40.50 kN/m
Topping = 1.8 x 9.6 + 0.6 x 0.4 x 24	=	23.04 kN/m
Total	=	71.22 kN/m
Live load (construction) = 1.5 x 9.6	=	14.40 kN/m
Ultimate load = 1.4 x DL + 1.6 x LL	=	122.75 kN/m

Propped reaction at A and D

Dead load = 0.4 x 8/3 x 71.22	=	76.0 kN
Live load = 0.4 x 8/3 x 14.40	=	15.4 kN

Propped reaction at B and C

Dead load = 1.1 x 8/3 x 71.22	=	208.9 kN
Live load = 1.1 x 8/3 x 14.40	=	42.2 kN



Bending Moment In Propped Precast Beam

Step 2: Design for installation

Support :

$M_B^- = M_C^- = 0.1 \times (8/3)^2 \times 122.75 = 87.3$ kNm

$M^- / bd^2 f_{cu} = 87.3 \times 10^6 / (800 \times 290^2 \times 35) = 0.037$

From Figure 2.26, ρ_s/f_{cu} (min.) $= 0.98 \times 10^{-4}$

$\rho_s = 0.00343$

$A_{s1} = 796$ mm^2

Use 4T16 ($A_s = 804$ mm^2)

Span :

$M_{AB}^+ = M_{CD}^+ = 0.08 \times (8/3)^2 \times 122.75 = 69.8$ kNm

$\rho_s = 0.0027$

$A_{s1} = 626$ mm^2

M_{BC}^+ is not critical and assumes $A_{s1} = 796$ mm^2

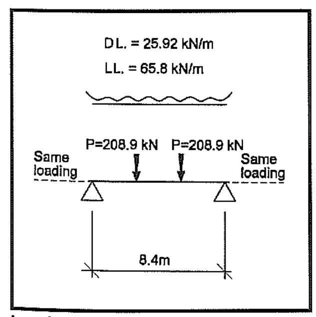
B. At Service

Step 3 : Analysis of beam under imposed dead and live load and reverse prop reactions

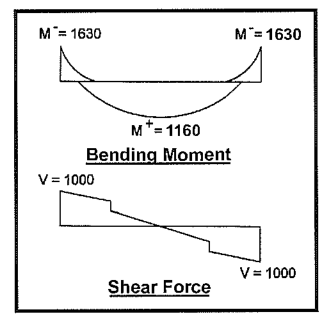
Dead load:

Finishes = 1.2 x 9.6	=	11.52 kN/m
Services = 0.5 x 9.6	=	4.80 kN/m
Partition = 1.0 x 9.6	=	9.60 kN/m
Total DL	=	25.92 kN/m

Live load = 5 x 9.6	=	48.0 kN/m
Equivalent L.L. = (0.4/1.6) x 71.22	=	17.8 kN/m
Total LL	=	65.8 kN/m
Propping reaction P = dead load (only due to s/w of structure)	=	208.9 kN



Loading On Continuous Beam



Distribution Of Bending Moment And Shear Force (Typical Internal Beam)

Step 4 : Design of main tension steel

Span : b = 600mm, h = 800mm, d = 700mm

$M^+ = 1160$ kNm

$M^+ / bd^2 f_{cu} = 1160 \times 10^6 / (600 \times 700^2 \times 35)$
$= 0.113$

From Figure 2.26, $\rho_s / f_{cu} = 3.4 \times 10^{-4}$
$\rho_s = 0.0119$
$A_{s2} = 4998$ mm²
Total $A_s = A_{s1}$ (from step 2) + A_{s2}
$= 626 + 4998$
$= 5624$ mm²

Use 7T32 (A_s = 5629 mm²)

Support : b = 800mm, h = 800mm, d = 700mm

$M^- = 1630$ kNm

$M^- / bd^2 f_{cu} = 1630 \times 10^6 / (800 \times 700^2 \times 35)$
$= 0.119$

From Figure 2.26, $\rho_s / f_{cu} = 3.49 \times 10^{-4}$
$\rho_s = 0.0122$
$A_{s2} = 6832$ mm²

Use 6T32 + 4T25 (A_s= 6789 mm², marginally under provided, OK)

Step 5 : Design for shear links

At support, $V = 1.0 \times$ (Step 1) + (Step 3)
$= 1.0 \times 76.0 + 1000$
$= 1076.0$ kN

$v = 1076.0 \times 10^3 / (600 \times 700)$ (b=600, refer to Figure 2.19)
$= 2.56$ N/mm²

ρ_s (6T32 + 4T25) $= 6789/(600 \times 700)$
$= 0.0162$
$\rho_s f_{cu} = 0.567$

From Figure 2.27, $v_c = 0.83$ N/mm²

$A_{sv} / s_v = (2.56 - 0.83) \times 600/(0.87 \times 460)$
$= 2.59$

Step 6 : Design for composite action

Support : Contact width, $b_e = 600$ mm
Contact length, $l_e = 0.2 \times 8.4 - 0.2$
$= 1.48$ m

$A_{sv}/s_v = A_s / l_e$
$= 6832/1480$
$= 4.62$

Span : Contact length, $l_e \approx 0.35 \times 8.4$
$= 2.94$ m

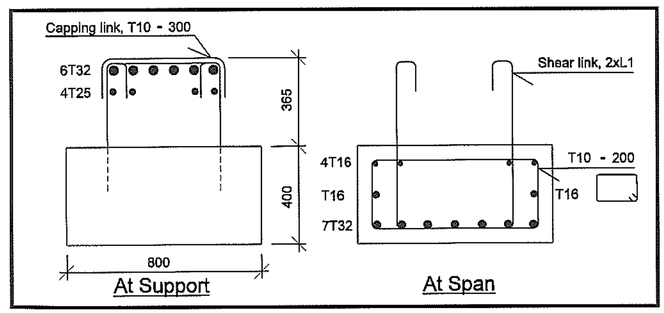
$A_{sv}/s_v = A_s / l_e$
$= 5624/2940$
$= 1.91$

Step 7 : Provide shear links for both composite action and shear resistance, whichever is greater

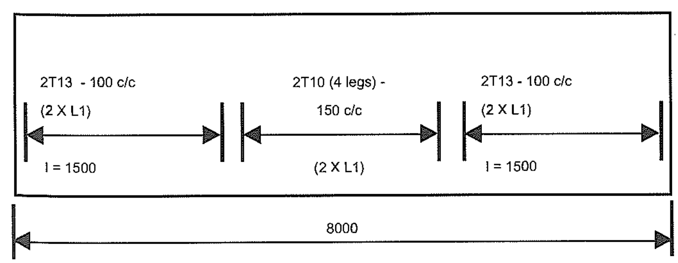
Support : $A_{sv}/s_v = 4.62$
Use 2T13 - 100 c/c links for 1.5 m at end span (A_{sv}/s_v = 5.31)

Span : $A_{sv}/s_v = 1.91$
Use 2T10 - 150 c/c links for mid-span (A_{sv}/s_v = 2.09)

Step 8 : Detailing



Typical Beam Section



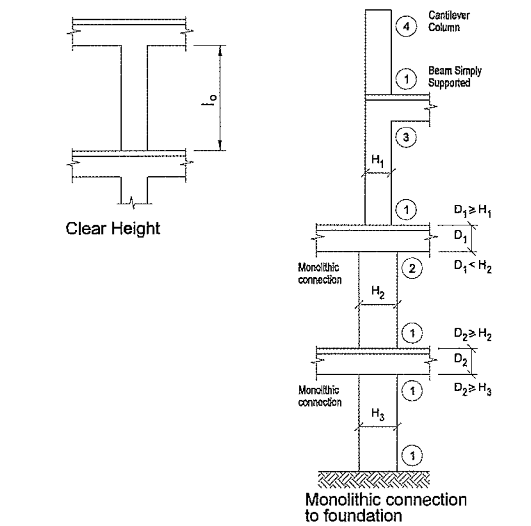
Distribution Of Shear Links

2.4 Design Of Precast Concrete Columns

The design of precast concrete columns is similar in approach to those for in-situ columns. The design methods complying to the code requirements are well documented in most standard texts and will not, therefore, be elaborated further in this section.

In the design of precast concrete columns, the designer should be conversant with the various connection methods used in jointing column-to-foundation, column-to-column and column-to-beam in order to achieve the desired joint behaviour which could be either moment-rigid or pin-connected.

Particular attention should be given to ensure that the connection details will not jeopardise the structural stability of the building. In addition, the columns must have sufficient capacity to withstand failure from buckling due to slenderness effect. A summary of ß values for braced and unbraced columns in accordance with Part 1 clause.3.8.1.6, of the Code is shown in Figure 2.29.



Braced Columns

End condition at top	End condition at bottom 1	2	3
1	0.75	0.80	0.90
2	0.80	0.85	0.95
3	0.90	0.95	1.00

Unbraced Columns

End condition at top	End condition at bottom 1	2	3
1	1.2	1.3	1.6
2	1.3	1.5	1.8
3	1.6	1.8	-
4	2.2	-	-

Value of β

Figure 2.29 Effective Height Of Column, $l_e = \beta\, l_o$

2.4.1 Design charts

In practice, it is very common to make use of charts in the design of columns; a collection of which is found in BS8110 : Part 3. In this Handbook, design charts for rectangular and circular columns are shown with 2% and 3% steel content for concrete grades f_{cu} = 35, 45 and 50 N/mm² respectively. The reinforcement content represents a typical range in precast column design.

To reduce the total number of design charts for rectangular (or square) columns, the charts are presented for a 50mm wide module for 200mm to 1000mm deep column. The charts are applicable for pin-connected and moment rigid jointing of either a braced or unbraced column.

The load eccentricities shown in the charts arise from:

1. actual design eccentricity such as beam supported by corbel,
2. h/20 or minimum 20mm,
3. additional eccentricity due to column slenderness effect as determined in Part 1 clause. 3.8.3.,
4. eccentricity due to framing moment in a moment rigid column-beam connection where the eccentricity is calculated as e = M/N where N is the total column load at the level being considered.

For columns under biaxial bending, the enhanced bending moment in either the minor or major axis should be determined in accordance with Part 1, clause 3.8.4.5 of the Code.

In using the design charts, the following steps may be taken:

Step 1. Determine the total ultimate column load, N, at the level being considered.

Step 2. Divide N by n which is a multiple of 50mm module for an assumed or given column width.

Step 3. Determine the load eccentricities as described earlier.

It should be noted that it may not always be possible to obtain the framing moment in a column unless actual column stiffness is used in the analysis. This, however, cannot be done before a reasonable column size is fixed. To overcome this problem, the column is usually sized by assuming a value of bending stresses M/bh² in the column which is generally taken to be:

Internal columns = 1.5 to 2.5N/mm²
Edge and corner columns = 3.0 to 4.0N/mm²

The M/bh² graphs are shown in all the column design charts from Figures 2.30 to 2.35 for rectangular/square columns and Figures 2.36 to 2.41 for circular columns.

The design charts for circular columns follow similar approach as that for rectangular columns except the axial load capacities are given for the actual column size. Hence, Step 2 in the design process is omitted.

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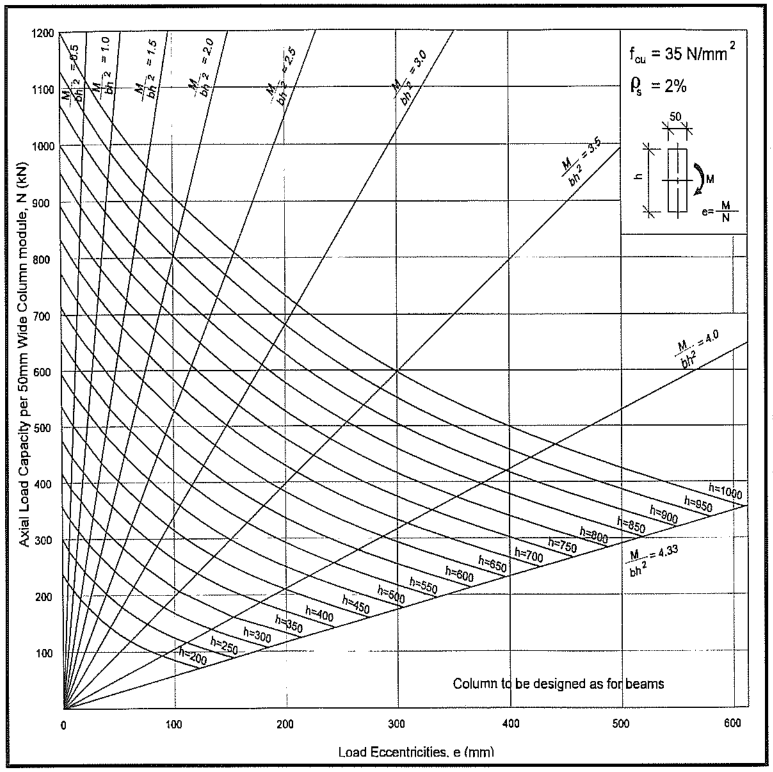


Figure 2.30 Reinforced Concrete Precast Rectangular/Square Column Design Chart For f_{cu}=35N/mm², And ρ_s=2%

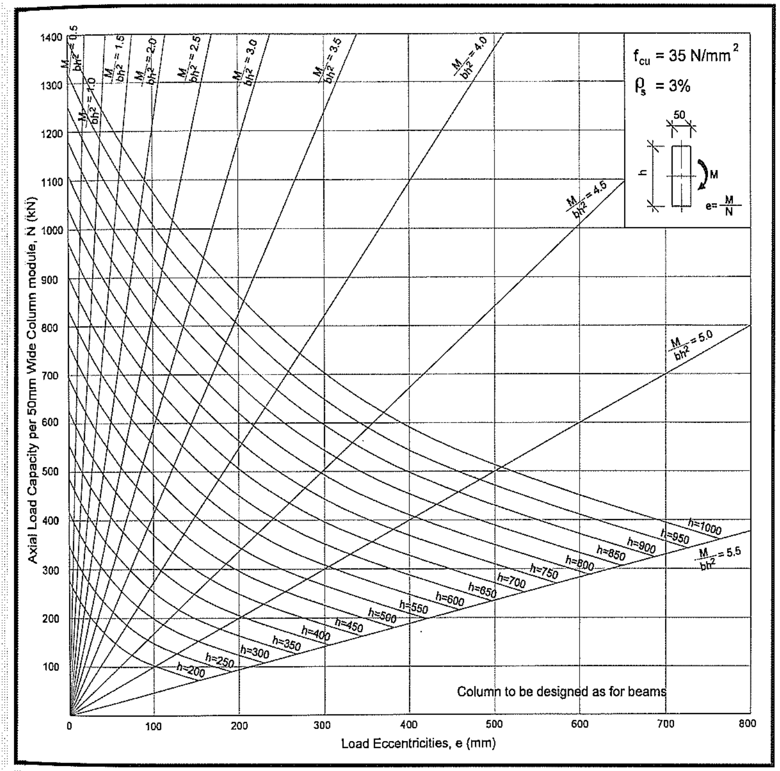


Figure 2.31 Reinforced Concrete Precast Rectangular/Square Column Design Chart For f_{cu}=35N/mm² And ρ_s=3%

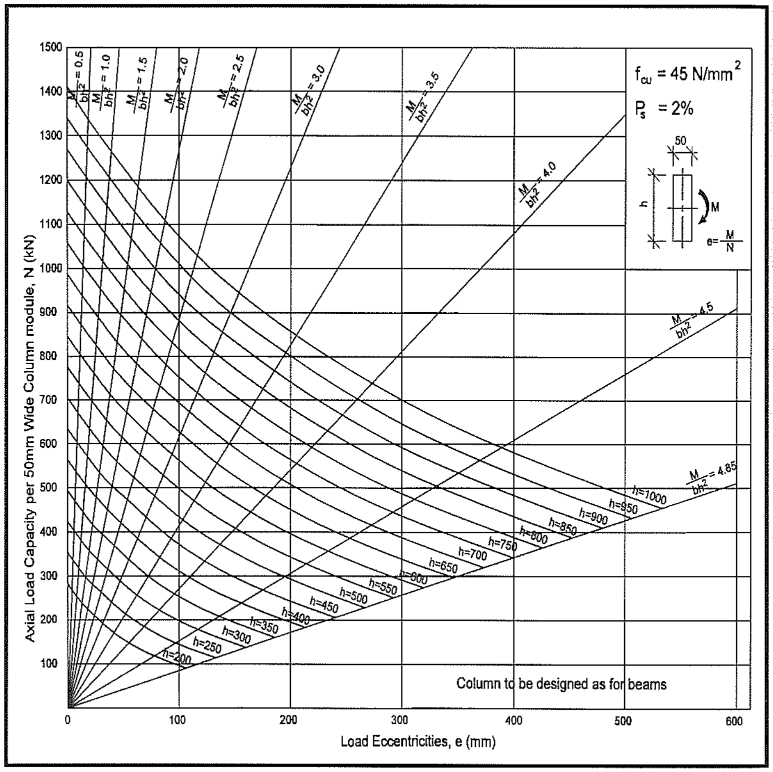


Figure 2.32 Reinforced Concrete Precast Rectangular/Square Column Design Chart For f_{cu}=45N/mm² And ρ_s=2%

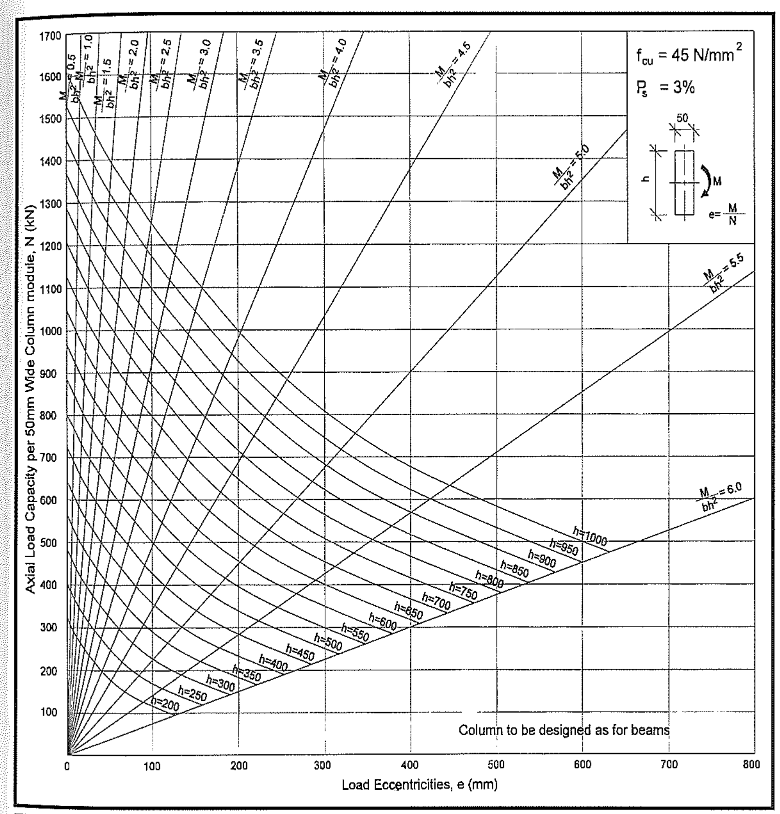


Figure 2.33 Reinforced Concrete Precast Rectangular/Square Column Design Chart For f_{cu}=45N/mm² And ρ_s=3%

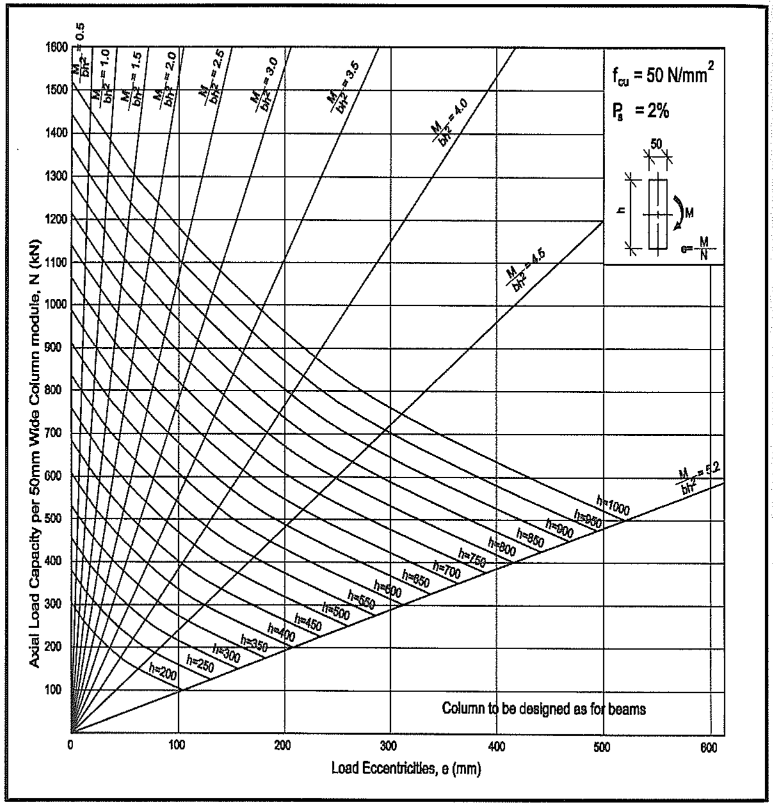


Figure 2.34 Reinforced Concrete Precast Rectangular/Square Column Design Chart For f_{cu}=50N/mm^2 And ρ_s=2%

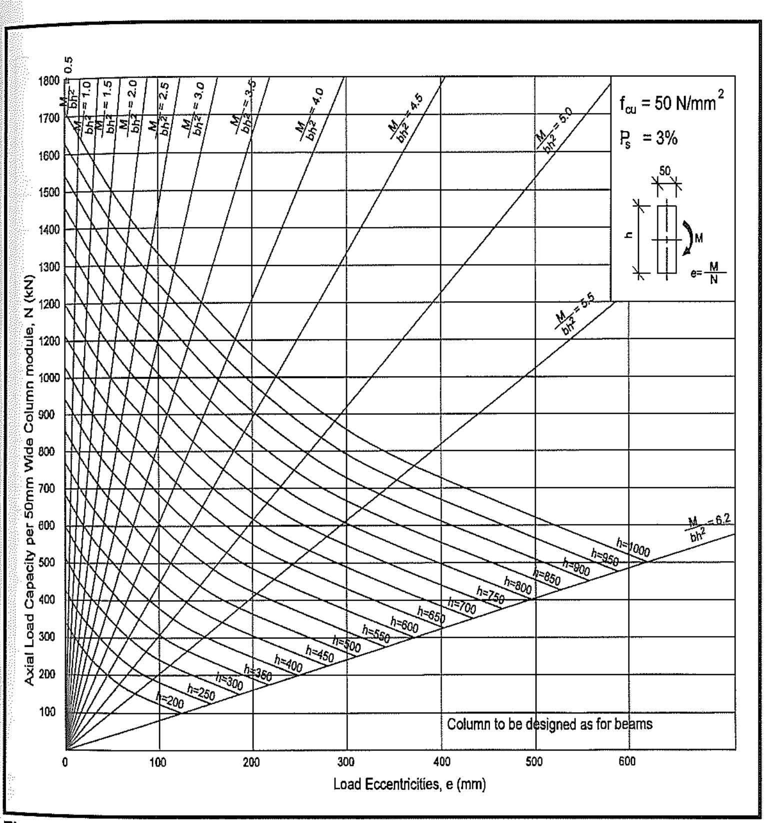


Figure 2.35 Reinforced Concrete Precast Rectangular/Square Column Design Chart For f_{cu}=50N/mm^2 And ρ_s=3%

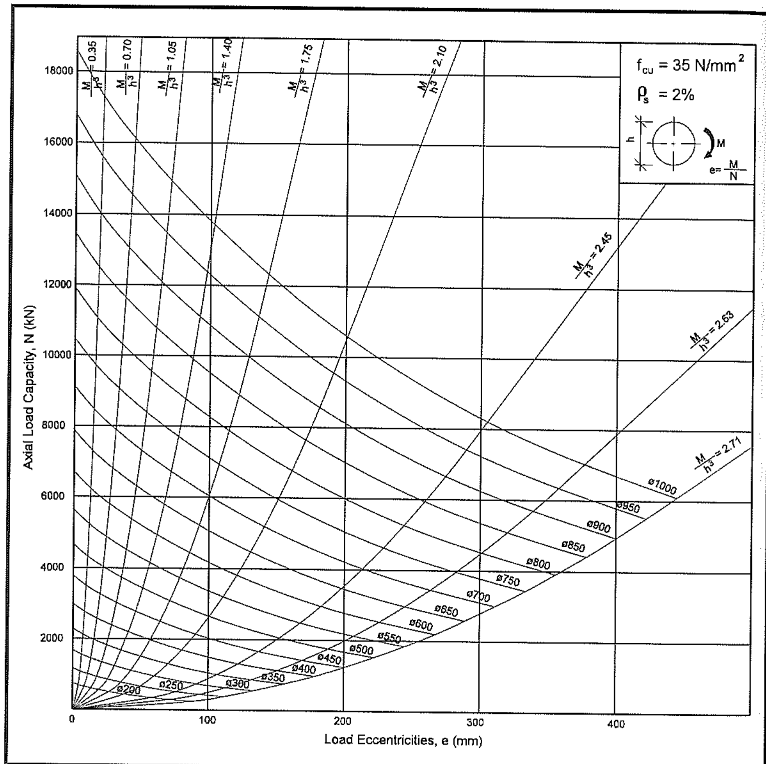


Figure 2.36 Reinforced Concrete Precast Circular Column Design Chart For f_{cu}=35N/mm^2 And ρ_s=2%

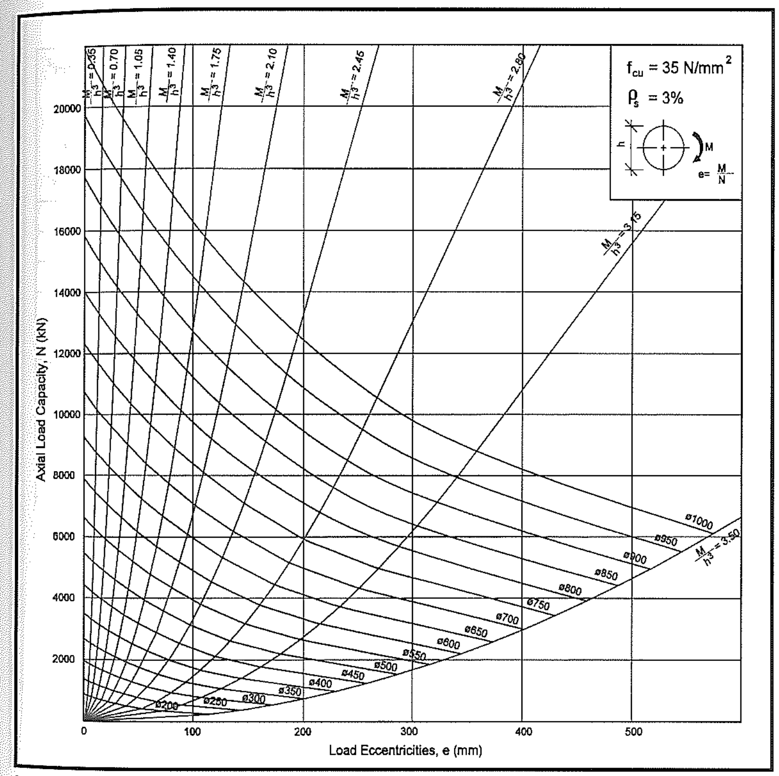


Figure 2.37 Reinforced Concrete Precast Circular Column Design Chart For f_{cu}=35N/mm^2 And ρ_s=3%

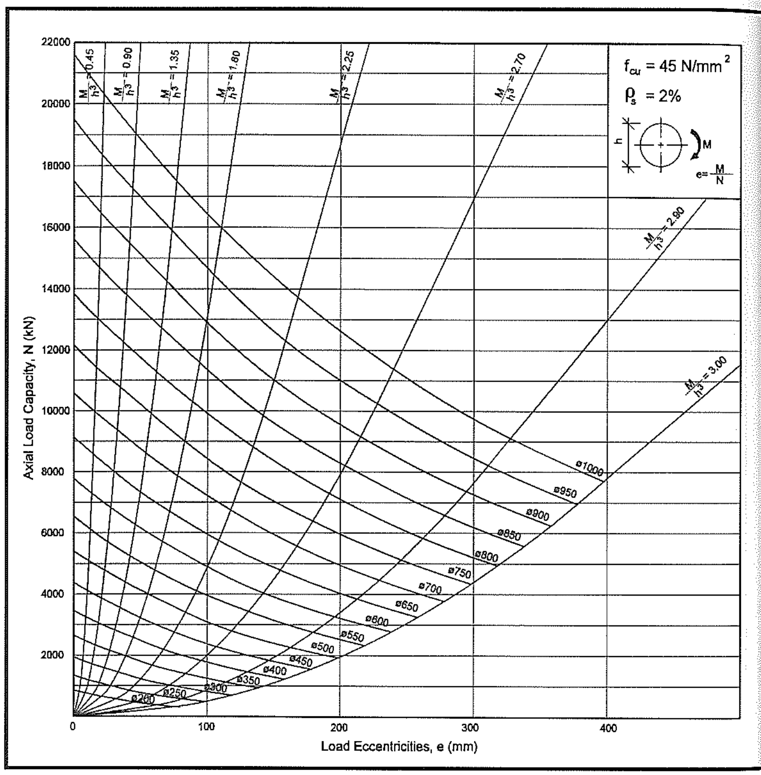


Figure 2.38 Reinforced Concrete Precast Circular Column Design Chart For f_{cu}=45N/mm² And ρ_s=2%

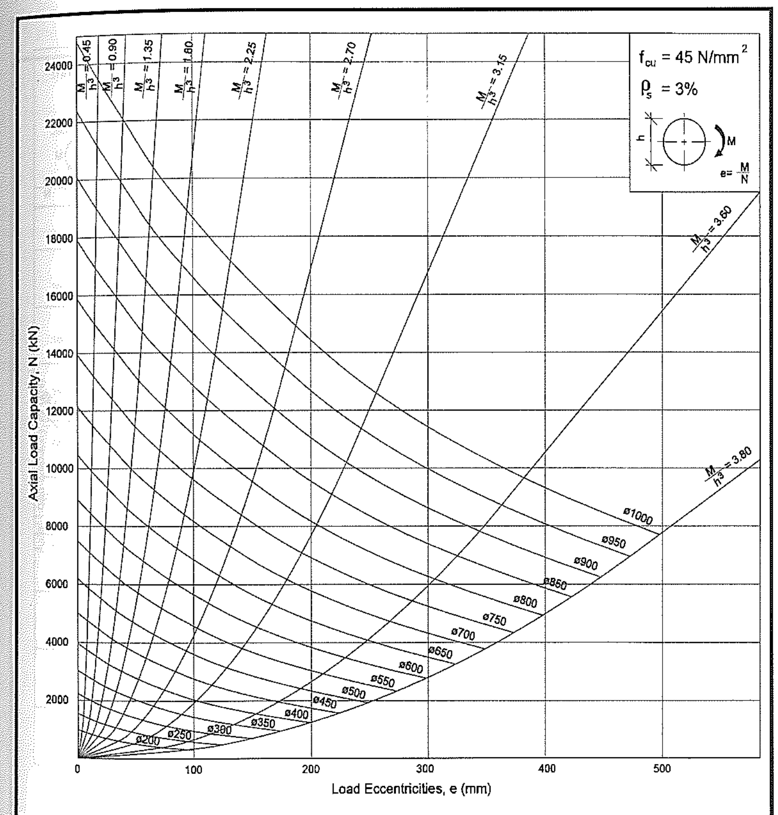


Figure 2.39 Reinforced Concrete Precast Circular Column Design Chart For f_{cu}=45N/mm² And ρ_s=3%

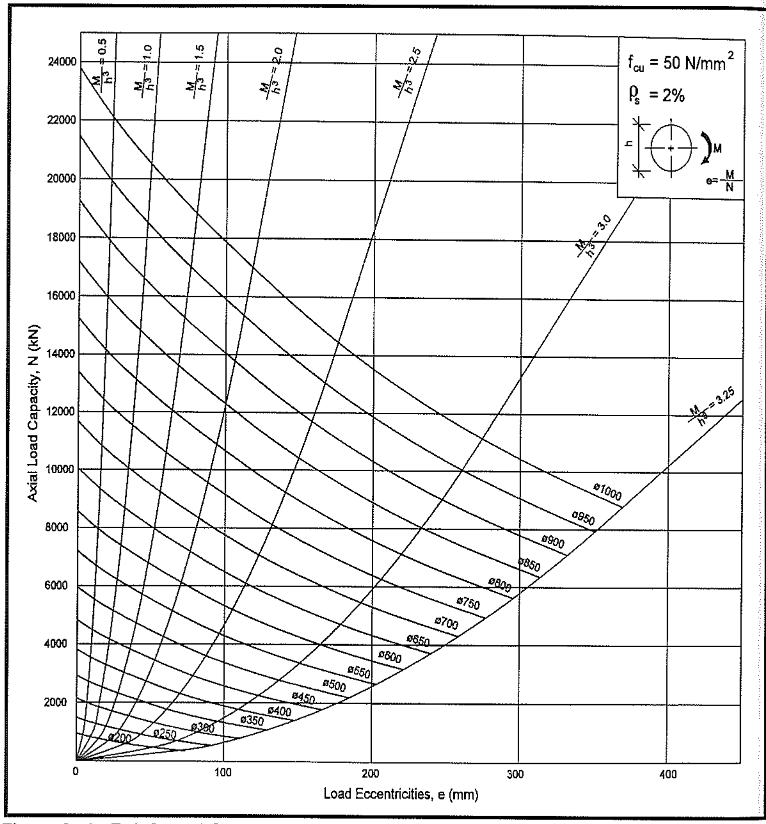


Figure 2.40 Reinforced Concrete Precast Circular Column Design Chart For f_{cu}=50N/mm² And ρ_s=2%

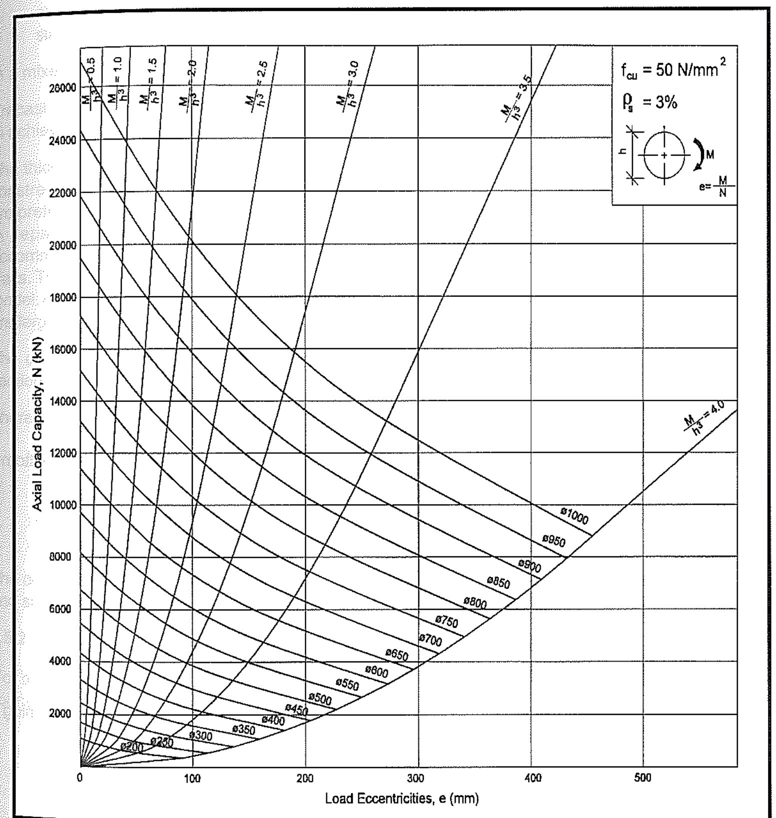
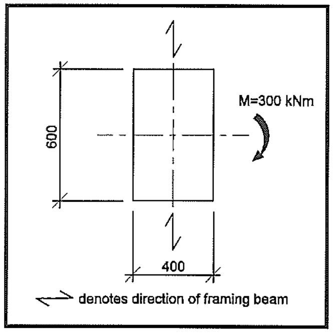


Figure 2.41 Reinforced Concrete Precast Circular Column Design Chart For f_{cu}=50N/mm² And ρ_s=3%

2.4.2 Design examples

Design Example 7: Precast Concrete Column

Determine whether a 400 x 600 precast column is adequate to carry a total ultimate load of 3500 kN and an ultimate framing moment of 300 kNm about the major axis. The column is braced with floor to floor height 4m and design concrete strength f_{cu}=35N/mm^2.



Step 1. Determine the column load for a 50 mm wide module

n = 400/50
= 8

The column load per 50 mm wide module = 3500/8
= 437.5kN

Step 2. Determine slenderness effect

From Figure 2.29, β value for a braced column with condition 1 at both top and bottom ends is 0.75.

$\therefore l_e$ = 0.75 x 4000
= 3000mm

Minor axis l_e/h = 3000/400
= 7.5 < 15, short column

Additional eccentricity due to slenderness effect is not critical.

Step 3. Determine load eccentricities

Since the column/beam connection is moment rigid, the load eccentricity from framing moment is calculated to be

e = M/N
= 300 x 1000/3500
= 85.7mm

Step 4. Check adequacy of column size

From Figure 2.30 (f_{cu} = 35N/mm^2, p_s = 2%), the required column length for column load of 437.5kN and e = 85.7mm is about h ≈ 500mm < 600mm provided.

Therefore the column size 400 x 600mm with 2% reinforcement content is adequate.

2.5 Design Of Precast Concrete Walls

2.5.1 General

The functions of precast concrete walls can be identified by the type of buildings in which they are used :

1. Skeletal frame structures : precast concrete walls are used as non-load bearing in-fill walls and may be designed to provide stability to the building
2. Shear wall structures : the precast walls are reinforced, cantilevered walls designed to carry the vertical loads and horizontal, lateral and in-plane forces.They are used as stabilising elements for the structure and may come in the form of single elements or forming boxes for staircases or lift shafts

In a mixed skeletal frame and shear wall building, both types of walls are provided.

Precast concrete walls may also be used as non-load bearing partitions to replace brickworks so as to achieve better surface quality and minimise site plastering.

The thickness of precast walls varies from 125 mm to 300 mm and is governed either by cranage constraints at the site or factory or by the ultimate shear and load carrying capacity in service. Walls are preferably designed as single elements. For wider walls, it may be necessary to assemble them in separate units with in-situ jointing. Openings for doors, windows and services may be accommodated provided their positions do not interrupt the structural integrity and continuity of the walls. This is particularly important when the wall design is based on the compressive diagonal strut model. Alternative load paths for the vertical and horizontal forces must be considered if the openings are large.

The construction of box walls for staircases and lift shafts can be obtained from individual wall sections or from a complete or partial box in single or part storey high.

There is a wide range of solutions in jointing the walls. These include:

1. in-situ concrete and steel tie
2. welded connections made by fully anchored plates
3. bolting
4. shear keys with or without interlacing steel and
5. simple mortar bedding

The design forces at the vertical and horizontal wall joints primarily consist of compression, tension and shear forces. Design considerations are shown in Chapter 3, section 3.17 in this Handbook.

2.5.2 Design classifications of concrete walls

For design purposes, CP 65 classifies the walls, which are defined as having their length exceeding four times their thickness, into :

1. reinforced concrete walls containing a minimum quantity of steel as given in Part 1, clauses 3.9.3 and 3.12.5.3. The steel is taken into account when determining the strength of the walls, and
2. plain concrete walls in Part 1, clause 3.9.4 where only minimum shrinkage steel is provided. The strength of the walls is based solely on the compression capacity of the concrete.

In addition, the walls are classified as :

1. braced if the walls are supported laterally by floors or other cross-walls
2. unbraced if the walls provide their own stability, such as cantilever walls.

The walls are considered stocky if the slenderness ratio, i.e. l_e/t, does not exceed 15 for a braced wall and 10 for an unbraced wall. Otherwise, the walls are considered as slender.

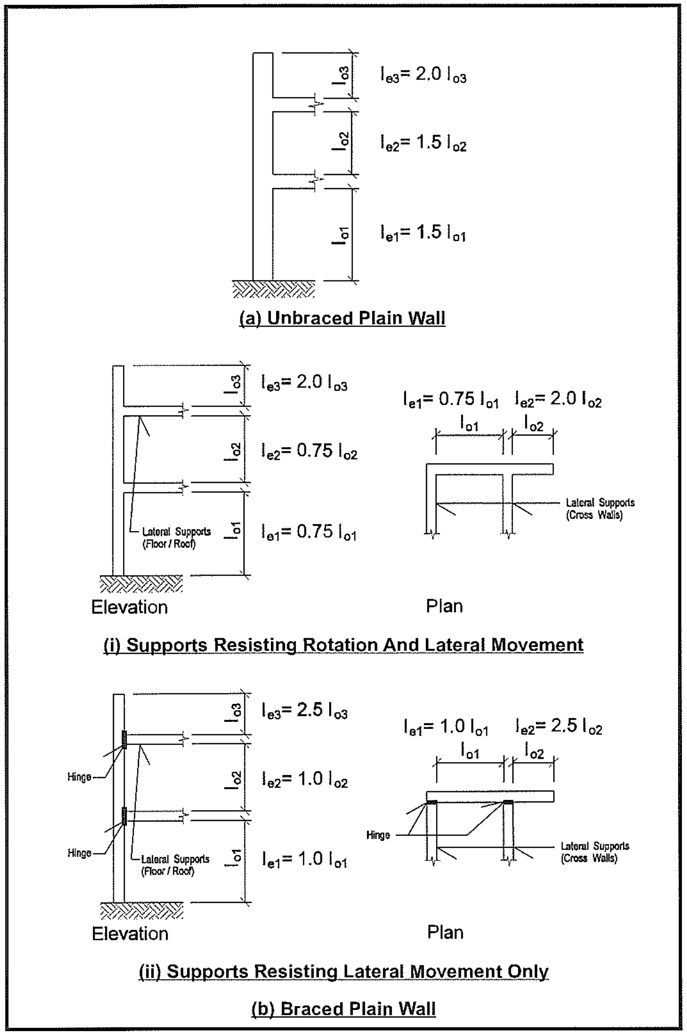


Figure 2.42 Effective Heights Of Plain Walls

The slenderness limits for reinforced concrete walls are given in Part 1, Table 3.25 of the Code. The effective heights are determined by similar methods used for columns and in accordance with Part 1, clause 3.8, Tables 3.21 and 3.22, of the Code. When the beams and slabs transmitting forces into the reinforced concrete walls are simply supported, the effective heights of the walls are assessed similarly for plain walls. These are shown in Figure 2.42 above.

2.5.3 Distribution of horizontal loads

Due to their large in-plane stiffness and strength, precast concrete walls offer the best solution to stability irrespective of the number of storeys in a multi-storey construction.

The horizontal forces, which consist of the greater of either the ultimate wind forces or 1.5% of the characteristic dead weight of the structure are transferred by diaphragm action of the floor to the stability walls in the manner shown in Figure 2.43.

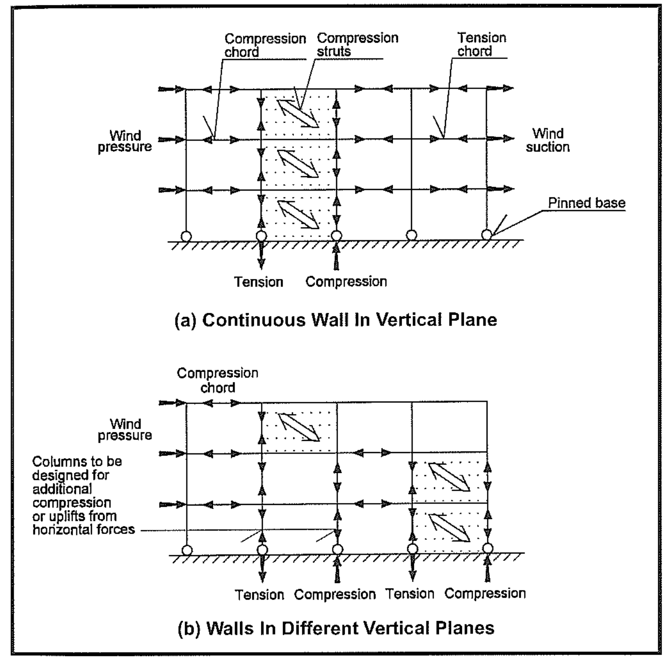


Figure 2.43 Horizontal Load Transfer In Braced Structure

It may be necessary to consider uplift particularly in gable end walls which do not carry any significant vertical loads.

The distribution of horizontal loads between stability wall elements is generally determined by the position and stiffness of the walls in the structure. When the location and distribution of the walls are such that their centre of resistance coincides closely with the centre of the mass and the geometric centroid of the completed building, the distribution of horizontal loads is proportional to their flexural stiffness. If the walls are more squat, i.e., its height to length is less than three, shear deflection will govern and the load distribution will be a function of the shear stiffness of the wall elements. If the Young's modulus and shear modulus are similar in all the walls, the stiffness of each wall element is then proportional to its second moment area, I, of the uncracked section or to the web cross sectional area, generally taken as 80% of the total web area if the wall deflection is predominantly due to shear.

In plain concrete walls, the Code stipulates in Part 1 clause 3.9.4.7, that when the resultant eccentricity from a horizontal force is greater than 1/3 of the length of the wall, the stiffness of the wall will be ignored and the horizontal forces adjusted to be carried by the remaining walls.

In majority of the buildings, the walls are normally non-symmetrical and torsional stability may need to be considered in the design. The torsions in the walls in a complicated layout can be determined using computer software with appropriate structural modelling. In relatively straight forward cases, it suffices just to carry out approximate and simple manual calculations to determine the horizontal loads due to torsion effect.

The approximate design method (reference 6) assumes that :

1. the floor plate is a rigid diaphragm,
2. the relative deflections of the walls are proportional to the distances from the centroid of flexural rigidity, and
3. shear deflections are small compared with flexural deflections even though the distribution of horizontal loads may vary depending on whether the wall system behaviour is predominantly flexure, shear or a combination of both.

Figure 2.44 shows a wall system containing n numbers of wall profiles. The centroid of the stiffness or flexural rigidities (EI) of the wall system is calculated from an arbitrary reference point O in a x-y cartesian coordinate system. If the Young's modulus E is similar in all the walls, the flexural rigidity of each wall element is then proportional to its second moment of area I.

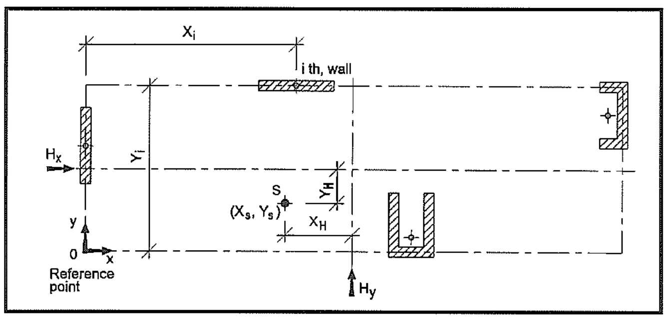


Figure 2.44 Notations Of Wall System Subjected To Torsion

The total stiffness of the wall system in the two principal directions is determined by :

$$I_\chi = \Sigma I_{i\chi} \qquad I_y = \Sigma I_{iy}$$

The centre of the shear force of the wall system is :

$$\chi_s = \Sigma(I_{i\chi}\,\chi_i)/I_\chi$$

$$y_s = \Sigma(I_{iy}\,y_i)/I_y$$

The resultant horizontal loads denoted by H_χ and H_y act at a distance χ_H, y_H from the centroid of the shear force.

The torsion, taking positive in anti-clockwise direction, is given by

$$T = H_y\,\chi_H - H_\chi\,y_H$$

The torsional strength of the individual wall system can be determined by means of the formula :

$$t = \Sigma I_{i\chi}\,(\chi_i - \chi_s)^2 + \Sigma I_{iy}\,(y_i - y_s)^2$$

The horizontal load on each wall section can then be calculated by means of the components in the χ and y directions.

$$H_{i\chi} = I_{iy}\,[H_\chi/I_y - T(y_i - y_s)/t]$$
$$H_{iy} = I_{i\chi}\,[H_y/I_\chi + T(\chi_i - \chi_s)/t]$$

It can be observed from the above that in symmetrical wall system where T = 0, the distribution of horizontal loads in the wall sections will then be directly proportional to the moment of inertia of the individual walls.

Design Example 8 is used to illustrate the design method outlined above.

2.5.4 Infill Precast Walls in Skeletal Frame Structure

Precast concrete walls are commonly used as infill walls between framing elements in a skeletal frame structure to function as stability walls. The walls are assumed not to carry any building loads and that beams between the wall panels are considered as separate structural elements even though the gap between them is grouted solidly. The walls are also assumed to be free from simultaneous in-plane and out-of-plane wind loadings.

The infill walls are generally designed as plain walls with sufficient reinforcement to resist diagonal cracking in the panel and to maintain the intrinsic shape of the panel particularly at the corners. The tensile strength of the concrete is ignored in the design.

The behaviour of infill concrete walls and general design principles subjected to horizontal loads are outlined in reference 7 and illustrated in Figure 2.45.

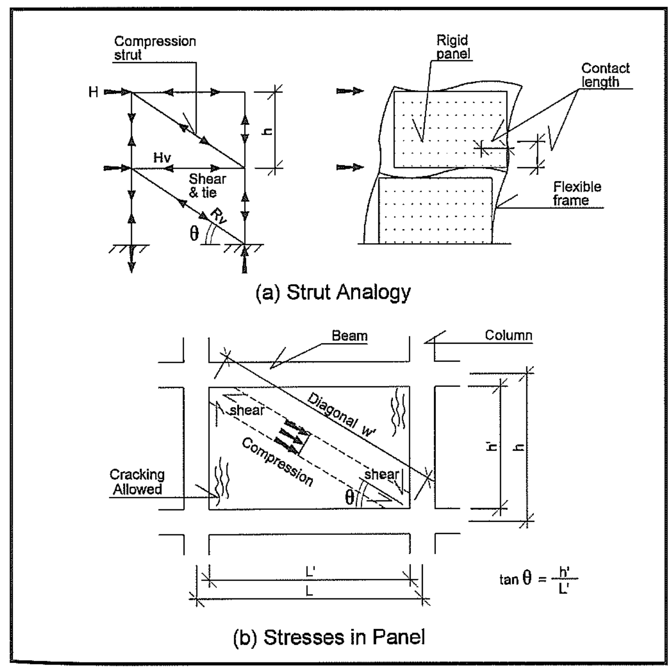


Figure 2.45 Infill Wall Panel Design Principles (reference 7)

The ultimate horizontal forces are resisted by a diagonal compressive strut across the wall panel. As the flexible frame deforms under horizontal loadings, compressive forces are transmitted at the two locked corners of the panels by interface shear stresses. The transmission of forces over the diagonal compressive strut at the corners of the panel occurs over an extended contact length. This gives rise to the effective width of the compressive strut. The width of the compressive strut is also dependent on the relative stiffness of the wall panel and the framing structure, and on the aspect ratio of the panel. An upper bound width of 0.1w' may generally be used in the design together with a consideration of strength reduction due to slenderness effect.

The contact lengths between panel and framing elements may be taken as

a. with beams = 0.5L'
b. with column : $\alpha = \pi/2\lambda$

and $\lambda = 4\sqrt{(E_i\ t \tan 2\theta)/(4E_f\ I\ h')}$

where E_i, E_f = modulus for infill and frame respectively
t = thickness of infill wall
I = moment of inertia of beams or columns whichever is lesser
h' = net height of infill panel
θ = slope of infill

The infill wall is considered braced as it is constructed on all sides. The effective length of the diagonal strut is then given by $l_e = 0.75w'$ where w' is the length of the diagonal strut. The ultimate concrete compression is assumed to be $0.3f_{cu}$ as in plain wall.

From Figure 2.45, the strength of the compressive strut, R_v, is given by:

$$R_v = 0.3f_{cu} \times 0.1w' \times t$$

$$\text{and } H_v = R_v \cos\theta = 0.03f_{cu}\ L'\ t$$

If the slenderness ratio w'/t > 12, then in accordance to Part 1, clause 3.9.4.15 of the Code,

$$R_v = 0.3f_{cu} \times 0.1w' \times (t - 1.2e_x - 2e_a)$$

where $e_x = 0.05t$
$e_a = l_e^2/2500t$
and $l_e/t = 0.75w'/t < 30$ as required by Part 1, clause 3.9.4 of the Code

Substituting e_x and e_a into the expression

$$R_v = 0.3f_{cu} \times 0.1w' \times t \times [0.94 - (l_e/t)^2/1250]$$

At the corner, the interface ultimate shear stresses between the panel and framing beams and columns may adopt 0.45 N/mm² as in Part 1, clause 5.3.7 of the Code for plain surfaces without castellations. This is valid as the joint is in compression as well as shear. Hence,

$$R_v \sin\theta = 0.45\ \alpha\ t$$

$$R_v \cos\theta = 0.45\ (0.5L't)$$

Any residual horizontal shear is taken up either by interface reinforcement anchored into beam and wall panel or by mechanical bolting or welding of anchored plates. The vertical residual shear is carried by the beam/column connector.

The bearing under the corner of the wall should be checked against the weakest interface jointing concrete by

$$R_v \sin\theta \le 0.6f_{cu}(0.5L't)$$

The design of infill plain walls in skeletal frame structure is illustrated in Design Example 9.

2.5.5 Cantilever precast concrete walls

Cantilever precast concrete walls are usually provided as enclosures for staircases and lift shafts. They are designed as conventional reinforced concrete walls with special emphasis on the connection details of vertical and horizontal joints of the walls for the transfer of compression, tension and shear forces.

The walls may fail in one of the following modes as shown in Figure 2.46.

a. Shear slip at the horizontal and vertical joints if the walls are assembled from separate units and when L/h is greater than three.

b. Flexural tension failure under large overturning moment and lightly loaded situation.

c. Flexural compression failure due to combined axial forces and moments.

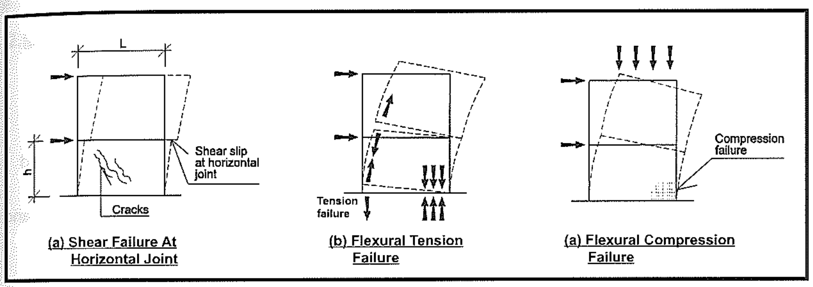


Figure 2.46 Failure Mode Of Cantilever Precast Concrete Walls

The design of the wall is carried out at the ultimate limit state as illustrated in Figure 2.47.

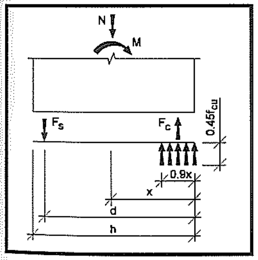


Figure 2.47 Design Principle Of Cantilever Precast Concrete Walls

The concrete compression forces under combined action of axial load and moment are given by

$$F_c = 0.45f_{cu} \times 0.9\chi \times t$$

Under axial loads

$$N = F_c - F_s$$

where F_s is the tensile resistance provided by reinforcement

$$F_s = 0.87f_yA_s$$

Hence $N = 0.45f_{cu} \times 0.9\chi \times t - 0.87f_yA_s$

and
$$\begin{aligned} M &= F_c(d - 0.45\chi) - N(d - h/2) \\ &= 0.45f_{cu} \times 0.9\chi \times t(d - 0.45\chi) - N(d - h/2) \\ &= 0.405f_{cu}\ \chi\ t(d - 0.45\chi) - N(d - h/2) \end{aligned}$$

To transfer tension force F_s across the joint, isolated connections using grouted pipe sleeves, bolting and welding may be used.

2.5.6 Vertical load capacity of precast walls

One of the primary structural functions of walls is to carry the vertical loads to the foundation. From a structural viewpoint, the walls can be regarded as columns of unit length. The theoretical calculations of such columns under axial load or axial load with eccentricities are dealt with in most standard literature on structural design and a set of column design charts can be found in BS8110 : Part 3. These column design charts can be applied to the design of in-situ as well as precast reinforced walls.

In low- to medium-rise buildings, plain precast walls may be used due to the simplicity in the wall connections as well as the inherent large carrying capacity of the walls based on concrete alone. However, as a result of the lack of structural reinforcement in plain walls, the vertical load capacity can be reduced drastically if the wall is subjected to excessive flexural tensile stresses resulting from transverse load eccentricities.

These eccentricities may arise from :

1. the floor elements with unequal span or with different design loadings on each side of the wall,
2. special support fittings for the floor elements,
3. vertical misalignment or tilting in the vertical plane of the wall panels
4. angular rotations of the ends of the floor element which introduce moments at the top and bottom of the walls.

The magnitude of the transverse load eccentricities is given in Part 1, clause. 3.9.4, of the Code and is illustrated in Figure 2.48 for the following cases :

1. loads from the floor
2. loads from special support fitting
3. resultant loads

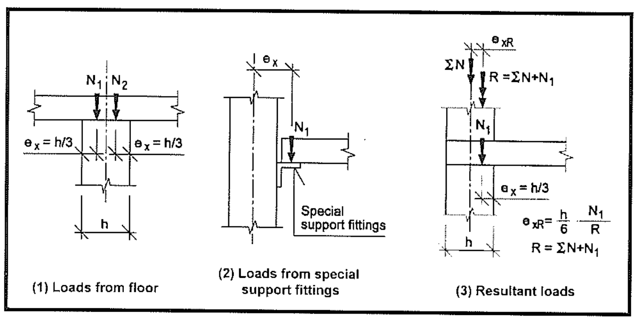


Figure 2.48 Design Load Eccentricities In Walls

For an unbraced wall, the eccentricities of all vertical loads and the moments due to lateral loads are considered when calculating the resultant eccentricity.

Minimum design transverse eccentricity is t/20 or 20 mm, whichever is greater.

The vertical load carrying capacity of plain walls is calculated using the following equations :

1. Stocky braced wall

   $n_w \le 0.3f_{cu}(t - 2e_\chi)$

2. Slender braced wall

   $n_w \le 0.3f_{cu}(t - 1.2e_\chi - 2e_a)$

3. Unbraced wall

   a. $n_w \le 0.3f_{cu}(t - 2e_{\chi,1})$ or

   b. $n_w \le 0.3f_{cu}(t - 2e_{\chi,2} - 2e_a)$

   whichever is smaller.

   where e_χ = actual resulting transverse load eccentricity t/20 or 20 mm, whichever is greater.

   $e_{\chi,1}$, $e_{\chi,2}$ = resulting transverse load eccentricity at the top and bottom of the wall respectively, as in e_χ

   e_a = additional deflection due to slenderness effect = $l_e^2/2500t$ where l_e is the effective height of the wall

2.5.7 Design charts

The design charts for stocky and slender braced plain walls are shown in Figures 2.49 and 2.50 respectively. The walls are assumed to be pin-connected at the floor levels.

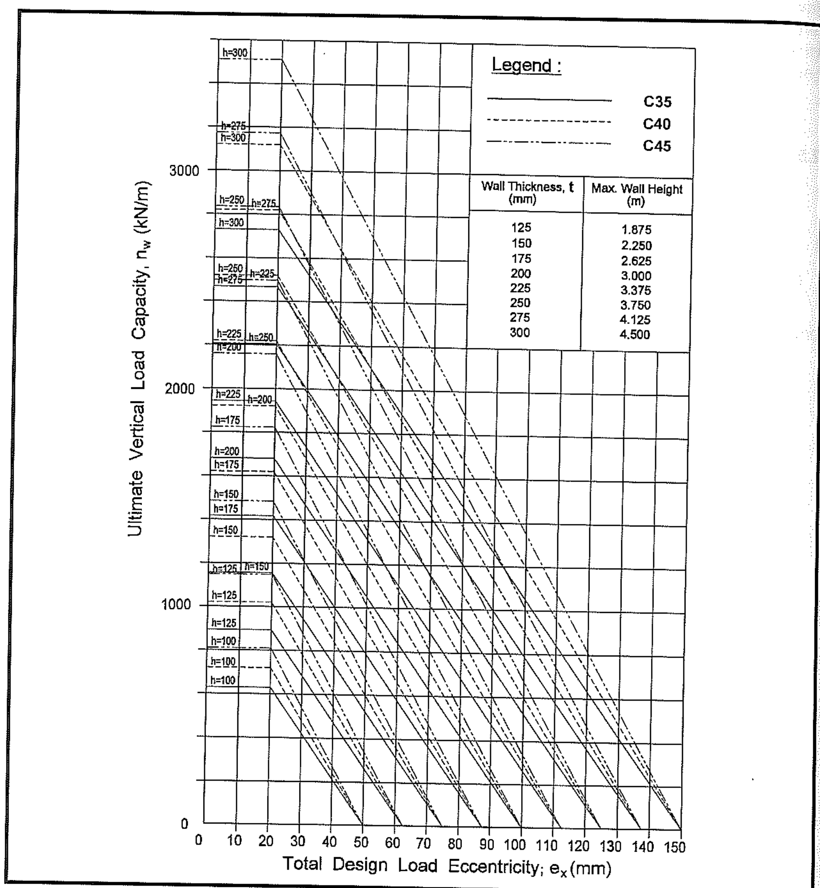


Figure 2.49 Ultimate Vertical Load Capacity (kN) Of Plain Stocky Braced Walls ($l_e/h \leq 15$)

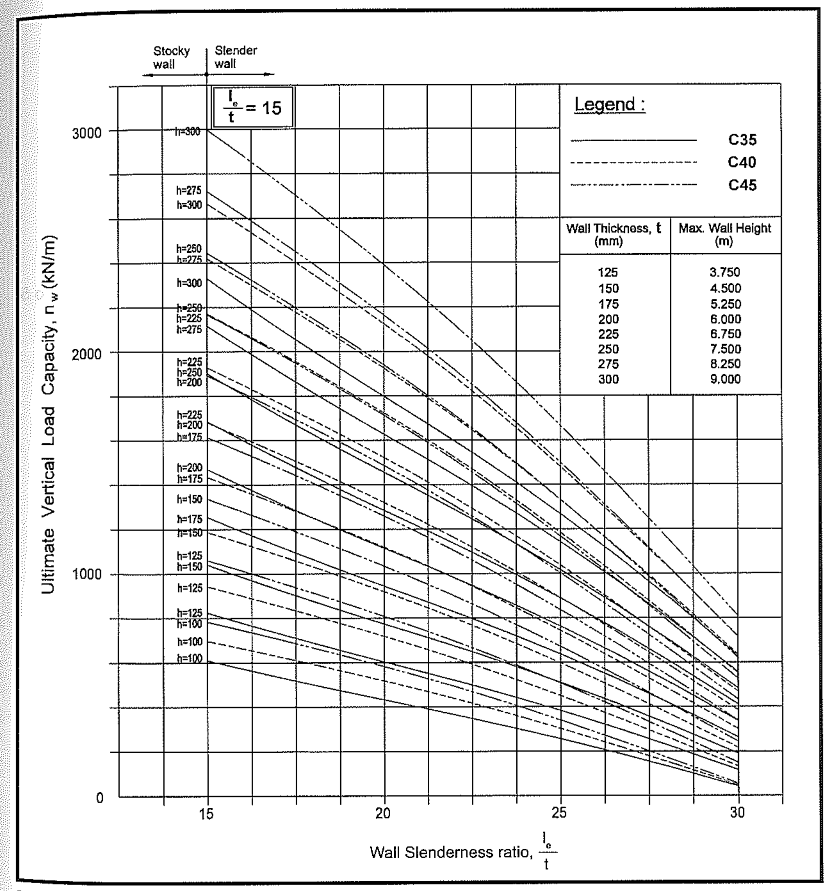
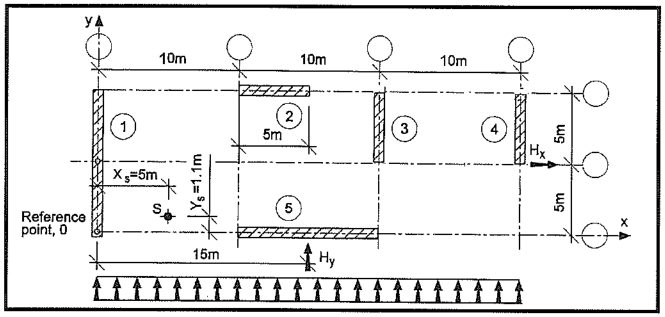


Figure 2.50 Ultimate Vertical Load Capacity Of Braced Slender Walls ($15 < l_e/h \leq 30$)

2.5.8 Design examples

Design Example 8 : Distribution of Horizontal Loads



Note : All walls to be of the same thickness

1. Centroid of wall stiffness

Using a 5 m long wall as reference and with a moment of inertia, I, the stiffness of walls 1 to 5 and the centroid of wall stiffness are calculated as below:

Wall	χ_i (m)	y_i (m)	I_χ	I_y
1	0	5.0	8I	0
2	12.5	10.0	0	I
3	20.0	7.5	I	0
4	30.0	7.5	I	0
5	15.0	0	0	8I
Total			10I	9I

The centroid of wall stiffness

$$\chi_s = \Sigma(I_{i\chi}\,\chi_i)/I_\chi = (I \times 20 + I \times 30)/10I = 5\text{ m}$$

$$y_s = \Sigma(I_{iy}\,y_i)/I_y = I \times 10/9I = 1.1\text{ m}$$

2. Torsion from horizontal load

Taking anti-clockwise as positive

$$T = H_y \times (15 - 5) = 10H_y$$

3. Torsional strength of individual wall sections

$$t = \Sigma I_{i\chi}(\chi_i - \chi_s)^2 + \Sigma I_{iy}(y_i - y_s)^2 = 8I \times 5^2 + I \times 15^2 + I \times 25^2 + I \times 8.9^2 + 8I \times 1.1^2 = 1138.9I$$

4. Distribution of horizontal forces

$$H_{i\chi} = I_{iy}\,[H_\chi/I_y - T(y_i - y_s)/t)]$$

$$H_{iy} = I_{i\chi}\,[H_y/I_\chi + T(\chi_i - \chi_s)/t)]$$

Substituting $H_\chi = 0$, $\Sigma I_y = 9I$, $\Sigma I_\chi = 10I$, $T = 10H_y$, $t = 1138.9I$, $\chi_s = 5$, $y_s = 1.1$

Hence :

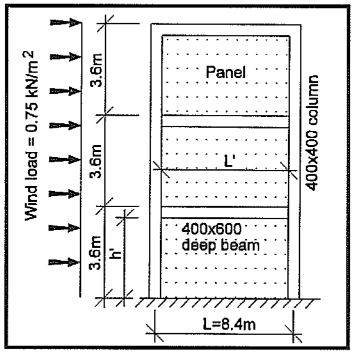
$$H_{i\chi} = -8.78 \times 10^{-3}(y_i - 1.1) \times H_y \times I_{iy} / I$$

$$H_{iy} = [0.1 + 8.78 \times 10^{-3}(\chi_i - 5)] \times H_y \times I_{ix} / I$$

Wall 1 : $H_{1\chi} = 0$, $H_{1y} = +0.45H_y$
Wall 2 : $H_{2\chi} = -0.08H_y$, $H_{2y} = 0$
Wall 3 : $H_{3\chi} = 0$, $H_{3y} = +0.23H_y$
Wall 4 : $H_{4\chi} = 0$, $H_{4y} = +0.32H_y$
Wall 5 : $H_{5\chi} = +0.08H_y$, $H_{5y} = 0$

Design Example 9 : Infill Precast Walls

The building is 60 m long and is braced at both ends with infill walls between the framing beams and columns. Determine the minimum thickness of the infill precast concrete walls at the first storey to provide stability to the building. Use f_{cu} = 35 N/mm² for the walls and frame. Design wind pressure = 0.75 kN/m².



Gable End Walls

$L' = 8.4 - 0.4 \quad = 8.0$ m

$h' = 3.6 - 0.6 \quad = 3.0$ m

$w' = \sqrt{(8.0^2 + 3.0^2)} \; = 8.544$ m

$\theta = \tan^{-1} 3/8.544 \quad = 19.35^0$

1. Maximum ultimate horizontal forces acting at second storey per frame

$H = 1.4 \times 0.75 \times (3.6 + 3.6 + 3.6/2) \times 60/2$
$= 283.5$ kN

$R_v = H/\cos\theta$
$= 300.5$ kN

Compression capacity of diagonal strut

$R_v = 0.3f_{cu} \times 0.1w' \times t[0.94 - (l_e/t)^2/1250]$
$= 0.3 \times 35 \times 0.1 \times 8544 \times t[0.94 - (6408/t)^2/1250] \times 10^{-3}$ (in kN)
$= 8.43t - 2.947 \times 10^5/t$

Minimum wall thickness required $300.5 = 8.43t - 2.947 \times 10^5/t$

Solving the quadratic equation t = 205.6 mm

Slenderness ratio $= 0.75w'/t$
$= 0.75 \times 8544/205.6$
$= 31.2 > 30$

Use $0.75w'/t = 30$
$t = 214$ mm

Say t = 225mm

2. Check interface shear stress

Column Interface $\lambda = \sqrt[4]{(E_i\, t \tan 2\theta)/(4E_f\, I\, h')}$

$E_i = E_f = 27$ kN/mm²

$I_{col} = 400 \times 400^3/12$
$= 2.133 \times 10^9$ mm⁴

$t = 225$ mm
$h' = 3.0$ m
$\lambda = 1.629 \times 10^{-3}$
$\alpha/h = \pi/2\lambda h'$
$= \pi/(2 \times 1.629 \times 10^{-3} \times 3000)$
$= 0.321$

Contact length $\alpha = 0.321 \times 3000$
$= 964.2$ mm

$R_v \sin\theta = 0.45\alpha t$
$R_v = 0.45 \times 964.2 \times 225 \times 10^{-3}/\sin 19.35^0$
$= 294.6$ kN < 300.5 kN

Residual vertical shear to be resisted by beam/column connection
$= (300.5 - 294.6)\sin 19.35^0$
$= 1.95$ kN (not critical)

3. Beam Interface

$R_v \cos\theta = 0.45 \times 0.5L't$

$R_v = 0.45 \times 0.5 \times 8000 \times 225 \times 10^{-3}/\cos 19.35^0$
$= 429.2$ kN > 300.5 kN — OK

Use 225 mm thick plain precast wall

4. Check Bearing Stress

$R_v \sin\theta < 0.6f_{cu}(0.5L't)$

$R_v \sin\theta = 300.5 \sin 19.35^0$
$= 99.6$kN

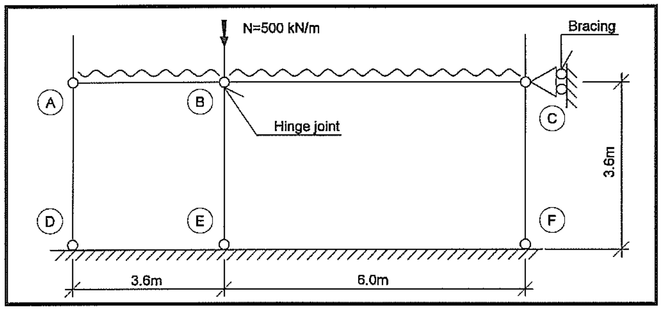
$0.6f_{cu}(0.5L't) = 0.6 \times 35 \times 0.5 \times 8000 \times 225 \times 10^{-3}$
$= 18900$ kN > 99.6 kN — OK

225mm thick plain precast wall is adequate

Design Example 10 : Braced Load Bearing Plain Walls

Check whether the internal 150 mm thick plain wall BE at the first storey in a braced building is adequate to carry an ultimate wall load of 500 kN/m and the floor loads shown in the figure below. Design concrete strength is f_{cu} = 35 N/mm².

Design dead load (unfactored)	= 6.0 kN/m²
live load (unfactored)	= 4.0 kN/m²



1. Calculate ultimate design load (per metre length)

Maximum design ultimate load = 1.4 x 6.0 + 1.6 x 4.0
= 14.8 kN/m

Minimum design ultimate load = 1.0 x 6.0
= 6.0 kN/m

2. Vertical floor reactions

For span AB : maximum floor reaction = 14.8 x 3.6/2
= 26.6 kN/m

minimum floor reaction = 6.0 x 3.6/2
= 10.8 kN/m

For span BC : maximum floor reaction = 14.8 x 6.0/2
= 44.4 kN/m

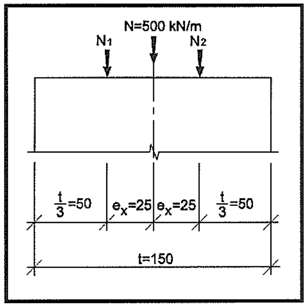
minimum floor reaction = 6.0 x 6.0/2
= 18.0 kN/m

3. Load eccentricities

For span AB : N_1 = maximum reaction = 26.6 kN/m
minimum reaction = 10.8 kN/m

For span BC : N_2 = maximum reaction = 44.4 kN/m
minimum reaction = 18.0 kN/m

e_x = t/20 = 7.5 mm < 20 mm
Use e_x = 20 mm



Case 1 : maximum floor load results in minimum load eccentricity

$$e_x = \frac{t}{6}\,\frac{N_2 - N_1}{N + \Sigma(N_1 + N_2)}$$

$$= \frac{150}{6}\,\frac{(44.4 - 26.6)}{500 + (44.6 + 26.6)}$$

= 0.78 mm < 20 mm

Use e_x = 20 mm

Case 2 : maximum load in span BC, minimum load in span AB, result in maximum load eccentricity

$$e_x = \frac{t}{6}\,\frac{N_2 - N_1}{N + \Sigma(N_1 + N_2)}$$

$$= \frac{150}{6}\,\frac{(44.4 - 10.8)}{500 + (44.6 + 10.8)}$$

= 1.5 mm < 20 mm

Use e_x = 20 mm

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Eccentricity from slenderness effect

Effective wall height = 1.0 x 3.6
= 3.6 m

l_e /h = 3600/150
= 24 > 15 for a braced wall

Hence, additional eccentricity e_a due to slenderness effect has to be considered.

e_a = l_e^2 / 2500t
= 3600^2 / 2500 x 150
= 34.6 mm

4. Total load eccentricities

Load eccentricity from floor is 20 mm for both cases 1 and 2. Hence the wall is designed using the following loads and load eccentricities :

ΣN = 500 + 44.4 + 26.6 + self weight of wall at mid-storey height
= 571.0 + 1.4 x 0.15 x 24 x 1.8
= 580.1 kN/m

e_x = 20 mm
e_a = 34.6 mm

5. Vertical load carrying capacity

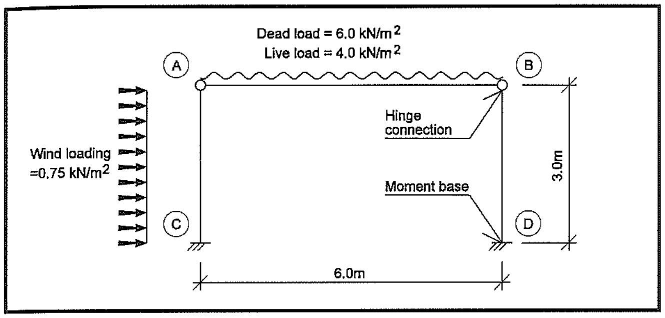
For braced slender plain walls :

$n_w \leq 0.3f_{cu}(t - 1.2e_x - 2e_a)$
$\leq 0.3 \times 35(150 - 1.2 \times 20 - 2 \times 34.6)$
≤ 596.4 kN/m > 580.1 kN/m

150 mm thick plain wall is adequate

Design Example 11 : Unbraced Load Bearing Plain Walls

Design the 250 mm thick wall AC in a single storey, unbraced building for the floor loading and lateral wind load shown in the figure below. The design concrete strength is f_{cu} = 35 N/mm².



1. Calculate ultimate design vertical load (per metre length)

Case 1 : maximum design load = 1.4 x 6.0 + 1.6 x 4.0
= 14.8 kN/m

Case 2 : Dead and wind = 1.4 x 6.0
= 8.40 kN/m

Case 3 : Dead, live and wind = 1.2(6.0 + 4.0)
= 12.0 kN/m

2. Vertical load and moment at bottom of wall

Wind moment is shared between two walls.
Moment per wall = 0.75 x 3² x 0.5/2
= 1.7 kNm/m

Case 1 : Vertical load = 14.8 x 6/2 + self weight of wall
= 44.4 + 1.4 x 0.25 x 24 x 3.0
= 69.6 kN/m

Wind moment = 0

Case 2 : Vertical load = 8.4 x 6/2 + self weight of wall
= 25.2 + 1.4 x 0.25 x 24 x 3.0
= 50.4 kN/m

Wind moment = 1.4 x 1.7
= 2.4 kNm/m

Case 3 : Vertical load = 12.0 x 6/2 + self weight of wall
= 36.0 + 1.2 x 0.25 x 24 x 3.0
= 57.6 kN/m

Wind moment = 1.2 x 1.7
= 2.0 kNm/m

3. **Load eccentricities**

a. **Vertical load and wind moment**

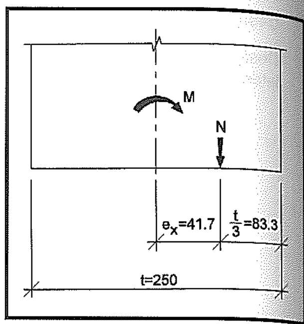
Case 1 : e_x = 41.7 mm

Case 2 : Total moment $= Ne_x + 2.4$
$= 50.4 \times 0.0417 + 2.4$
$= 4.5$ kNm/m

Resultant eccentricity e_{x2} $= M/N$
$= 4.5 \times 10^3/50.4$
$= 89.3$ mm

Case 3 : Total moment $= Ne_x + 2.0$
$= 57.6 \times 0.0417 + 2.0$
$= 4.4$ kNm/m

Resultant eccentricity e_{x2} $= M/N'$
$= 4.4 \times 10^3/57.6$
$= 76.4$ mm



Vertical Load And Wind Moment

b. **Slenderness effect**

Effective wall height $l_e = 1.5 l_o$
$l_o = 3.0$ m
$l_e = 1.5 \times 3.0$
$= 4.5$ m

Additional eccentricity e_a $= l_e^2/2500t$
$= 4500^2/(2500 \times 250)$
$= 32.4$ mm

4. **Vertical load carrying capacity**

The vertical load carrying capacity of the unbraced concrete plain wall is the smaller of the following:

a. $n_w \le 0.3 f_{cu}(t - 2e_{x1})$
b. $n_w \le 0.3 f_{cu}(t - 2e_{x2} - 2e_a)$

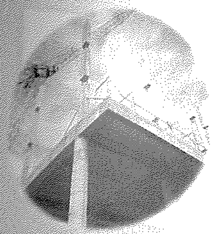
By inspection, the load carrying capacity is given by the second expression which is applied at the bottom of the wall.

Case 1 : e_{x2} = 41.7 mm e_a = 32.4 mm
$n_w \le 0.3 \times 35(250 - 2 \times 41.7 - 2 \times 32.4)$
≤ 1068.9 kN/m > 69.6 kN/m OK

Case 2 : e_{x2} = 89.3 mm e_a = 32.4 mm
$n_w \le 0.3 \times 35(250 - 2 \times 89.3 - 2 \times 32.4)$
≤ 69.3 kN/m > 50.4 kN/m OK

Case 3 : e_{x2} = 76.4 mm e_a = 32.4 mm
$n_w \le 0.3 \times 35(250 - 2 \times 76.4 - 2 \times 32.4)$
≤ 340.2 kN/m > 57.6 kN/m OK

250 mm thick plain wall is adequate.



CHAPTER 3

DESIGN OF PRECAST CONCRETE CONNECTIONS

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3.1 General

Connections form the most vital part of precast concrete construction. The ingenuity of engineers, researchers and manufacturers has, over the years, developed an extensive range of solutions, theoretical concepts and design equations for various types of connection. The knowledge of precast construction and design particularly those from UK, Continental Europe and North America is well disseminated through the publication of technical reports, research papers, design handbooks and product design information. However, an uninitiated design engineer will face considerable problems when looking for the necessary information to begin his design in precast construction. One of the aims of this part of the Handbook is, therefore, to bring together the wide and varied design methods used in the industry for the benefit of everyone. It is not possible, for obvious reasons, to present a complete overview of all the existing solutions and hence only current practices are included.

3.2 Design Criteria

Connections must meet a variety of design and performance criteria. A satisfactory connection design should, through careful detailing, include the following considerations:

1. a connection must be able to resist the ultimate design forces in a ductile manner
2. precast components are manufactured economically, easy to handle and simple to erect
3. tolerances for manufacturing and field erection must not jeopardise and adversely affect the intended structural behaviour of the components in service
4. the final appearance of the joint must satisfy fire, durability and visual requirements

3.3 Design Considerations

3.3.1 Strength

A connection must resist the forces to which it will be subjected during its lifetime. Some of these forces such as those caused by dead and live gravity loads, winds, earth and water pressures are obvious. Some are not so apparent and will often be overlooked. These are forces caused by the restraint of volume changes in the precast components and those required to maintain stability. Joint strength may be categorised by the type of forces that may be induced such as compressive, tensile, shear, flexural and torsion.

A connection may have a high degree of resistance to one type of force but with little or no resistance to another. While acknowledging the existence of these forces, the designer should not focus on a connection which would resist every conceivable forces. A better solution would be to utilise more than one type of joints to achieve the overall result.

3.3.2 Ductility

Ductility is the ability of a connection to undergo large deformation without failure. Deformation is measured between the first yield and ultimate failure of the structural materials used in the connection. Ductility in building frame is usually associated with moment resistance with the flexural tension capacity provided by the reinforcing bars or structural steel sections. The ultimate failure may be due to the rupture of reinforcing bars, crushing of concrete, the failure of connectors or steel embedded in the concrete.

3.3.3 Volume change

The combined shortening effect due to creep, shrinkage and temperature reductions induces tensile stresses in the precast components. The stresses must be accounted for in the connection design by either providing stress relieve details in unrestrained joint or by providing additional reinforcing steels to resist tensile forces in a restrained joint.

3.3.4 Durability

Exposed sections in a connection should be periodically inspected and maintained. Evidence of poor durability is usually exhibited by corrosion of exposed steel elements, cracking and spalling of concrete. Components which are exposed to weather should have the steel elements adequately encased in concrete or grout, painted, galvanised or using stainless steel sections.

3.3.5 Fire resistance

Connections which may be weakened by exposure to fire should be protected by concrete or grout, enclosed or sprayed with fire resistance materials. The connections should be protected to the same degree as that required for the components and the building frame.

3.4 Manufacturing Considerations

Maximum economy of precast concrete construction is achieved when connection details are kept as simple as possible and they are designed with adequate performance to facilitate easy field erection. The following manufacturing considerations should be noted :

1. Avoid congestion – where large quantity of reinforcing bars, embedded plates, inserts, blockouts, etc, are required, congestion is inevitable. When this occurs, study the area in question using large scale drawings. Use actual physical sizes and dimensions of steel bars, plates, inserts, bolts, etc, instead of line representations. Attention should be focused on areas which may not be reinforced as a result of minimum bending radii of reinforcing bars. It may be necessary to increase the precast components size to avoid congestion and to ensure that sufficient room is left for concrete to be placed without leaving any honeycombs.

2. Avoid penetration of forms – projections such as corbels, nibs, or starter bars which require cutting through the forms are difficult and costly to place. Where possible, these projections should be located at the top surface of the components when casting concrete.

3. Minimise embedded items – embedded items such as inserts, connectors, plates, etc, which require precise positioning and secure fixing are labour and time consuming. They should be kept to a minimum.

4. Avoid operations after stripping the form – operations which are required after stripping the form such as special cleaning, finishing, welding on projecting steel elements should be avoided whenever possible. These operations require additional handling, added labour (often with skilled trade) and extra working space.

5. Allow manufacturing tolerances – dimensional tolerances which are more stringent than industrial standards are difficult to achieve. Connections requiring close fitting parts with little provision for field adjustments should be avoided.

6. Use standard items – wherever possible, embedded items of inserts, plates, bolts, steel sections etc. should be standardised. The items should be available from more than one supplier. Standardisation of items will reduce errors made and will enhance and improve productivity.

7. Use repetitive details – utilise similar connection details as much as possible even if they may result in slight over-sizing. Repeated use of similar details will involve fewer modifications and form setup.

8. Allow alternatives – manufacturers should be allowed to propose alternative methods and detailing provided the design requirements are met. Allowing alternative solutions would generally provide more economical and better performing connections.

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3.5 Construction Considerations

Hoisting equipment and lifting operation may constitute a substantial cost item in precast construction. The design of connections should enable the components to be lifted, set and unhooked in the shortest possible time. This will mean that temporary shoring, staying guy ropes, bearing pads, levelling shims and other loose attachments such as angles, bolts, nuts and plates shall be in place or made available at a short notice prior to lifting. The following list of items should be considered in the design to improve on the erection of precast components:

1. Allow for field adjustment – design of connections should avoid details for perfect fit in the field. Certain amount of field adjustment should be allowed by means of oversize holes for bolts and dowels, shimming, welding and grouting.

2. Accessibility for works – connections should be detailed and planned to allow accessibility for working and sufficient room for equipment. For example, wrenches require minimum swinging arc for bolt tightening. Working under the deck in an overhead position should be avoided.

3. Repetitive and standard details – connection details should be standardised and repeated as much as possible. Connections requiring special skills such as welding, prestressing should be reduced so as to make the construction more economical.

4. Robust projections – projections from precast components such as reinforcing bars, bolts, steel sections should be robust and rugged in order to withstand damages due to handling. Threads on projecting bolts should be taped and greased to protect against damage and rust.

5. Allow alternatives – precast manufacturers and erectors should be allowed to propose alternative methods and details not anticipated by the designer. This would often result in more economical and easy erection of precast components.

3.6 Types of Joints

3.6.1 Compressive joints

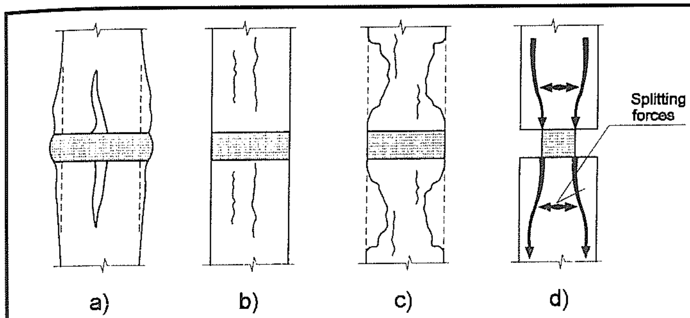
Compressive forces can be transmitted between adjacent precast components by direct bearing or through intermediate medium such as in-situ mortar, fine concrete, bearing pads or other bearing elements.

Direct contact between elements should only be used when great accuracy in manufacturing and erection is achieved and when the bearing stresses are small, usually less than $0.2f_{cu}$. Particular attention should be given in the reinforcement detailing of the precast components when a large concrete cover is required to achieve a high fire rating or when high strength steel bars with large bending radii are used.

It is more common and advisable to use intermediate bedding material for direct transmission of compression forces between precast elements as there will always be surface irregularities at the jointing surfaces. Cementatious materials such as in-situ mortar, fine concrete or grouting are often used in the joints between load bearing elements in columns and walls as well as for beams and floor elements. The nominal thickness is about 10 to 30 mm for mortar and grout, and 30 to 50 mm for fine concrete. The bedding is usually without reinforcing bars. The mode of failure is precipated by crushing of the mortar or splitting of the precast components in contact with it.

Although the mortar, grout or fine concrete is in a highly confined state under predominantly plane stress conditions and should achieve compressive strength higher than f_{cu}, a low design strength is normally used because the edges of the bedding tend to spall off. This will lead to non-uniform stress distribution. The situation can be exacerbated by poor workmanship, unintentional eccentricity, spurious bending moments and shear forces. Another factor which leads to a reduction of the joint strength is when there is a great difference in the elastic response between the bedded material and the precast concrete which may result in localised contraction, lateral tensile stress and splitting forces as shown in Figure 3.1. This effect may become important when the joint thickness is greater than 50 mm.

Hard bearing elements which usually consist of cast-in steel sections or plates with confining reinforcement are used when large concentrated forces are to be transmitted.



Force transfer through compression joint having an elastic modulus
a) less than the precast
b) equal to precast
c) greater than precast
d) as (c) but with reduced breadth

Figure 3.1 Vertical Transfer Of Compressive Forces

There are no explicit design equations in the Code to determine the joint strength taking into consideration the mentioned factors. The Code, however, suggests in Part 1, clause 5.3.6 that in calculating the compressive strength of the mortar joint, the area of the concrete in the joint should be the greater of :

a. the area of the in-situ concrete ignoring the area of any intruding components, but not greater than 90% of the contact area, or
b. 75% of the actual contact area

Vambersky (reference 8) proposed that the bearing capacity of unreinforced mortar joint can be calculated from:

$$n_w = \eta_0 \beta f_{cu} \qquad (3.1)$$

where f_{cu} = weaker concrete compressive strength of either the joint mortar or the precast components adjacent to the joint

η_0 = reduction factor reflecting the trapped air content

For a precast component placed onto a mortar bed η_0=0.3. For the case of joint infill after the components are placed:

η_0 = 0.7 for dry packed mortar and

= 0.9 for colloidal pouring mortar

The effects of joint geometry and the different quality conditions between site mixed mortar and under laboratory tests are reflected in the expression:

$$\beta = K \frac{5(1-K)+\delta^2}{5(1-K)+K\delta^2} \qquad (3.2)$$

where $K = \eta_m f_{cw}/f_{cu}$

f_{cw}/f_{cu} = concrete compressive strength of mortar and precast component respectively

η_m = 0.75 if site cubes are tested
= 1.00 if cores are cut and tested

δ = ratio of joint width to joint thickness
= t/v or x/v where x is the effective compression length of the joint under eccentric loading

It may be noted from the above expressions that a design stress of $0.4f_{cu}$ (or $0.4f_{cw}$, whichever is lower) as in Part 1, clause 5.2.3.4 of the Code may be adopted as the bearing capacity of the joint provided that:

a. t/v ratios are between 8 and 10

b. the difference in strength between the mortar and the precast component does not differ by more than 25%.

In practice, both the above criteria are generally satisfied.

Soft bedding materials such as neoprene pads are also used to even out surface irregularities. The thickness of the bearing pads may vary from 2 mm to 20 mm or even more. The larger thickness is used to allow displacements and rotations in order to reduce force built-up at the connection as shown in Figure 3.2.

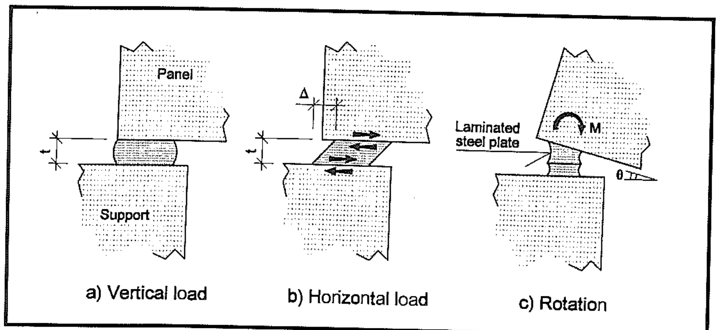


Figure 3.2 Loading Condition At Bearing Pads

3.6.2 Tensile joints

Tensile forces are transferred between concrete elements by means of various types of steel connectors which are anchored into each side of the elements at the joint with continuity achieved by overlapping of steel bars, dowel action, bolting or welding as shown in Figure 3.3.

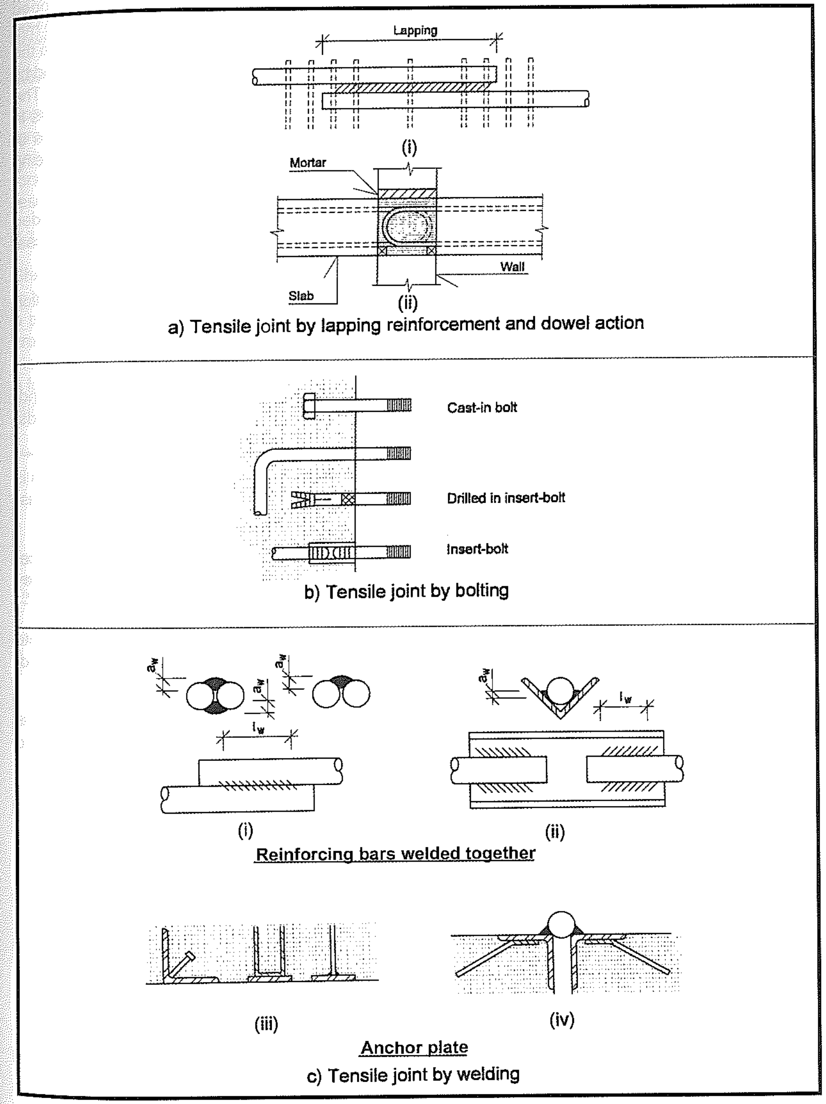


Figure 3.3 Tensile Force Transfer (reference 3)

The tensile capacity of the connection can be determined by either the strength of the steel elements or by the anchorage capacity. The latter is normally achieved by bonding along reinforcing bars or by means of end anchorage devices.

For the transfer of tensile forces such as vertical tie forces or tension in the force couple of a section under moments, one of the popular methods is the grouted pipe sleeves with in-situ lapped reinforcement. The method involves an annulus metal duct with a diameter of at least 20 to 30 mm larger than the bar diameter projected from a component to be jointed. The bar is inserted into the duct and grout is then injected through a hole at the base. Alternatively, the grout may be placed by gravity pouring. In either cases, the duct must be vented to prevent formation of air pockets. The lapped reinforcement can be placed either singly or symmetrically by the sides of the duct. As in normal lapping, the transfer of forces between bars can be visualised to consist of a series of compressive strut-tension ties. To ensure effective force transfer, stirrups are placed along the lapping length.

Bolting is used extensively to transfer tensile and shear forces. Anchorages such as bolts, threaded sockets and captive nuts are attached to the rear of the plates which are anchored into the precast units. Tolerances are provided using oversized holes in the connecting members.

Welding is used to connect between projections from adjacent precast components. The connection is either made direct or via an intermediate piece.

3.6.3 Shear joints

Shear forces between adjacent precast concrete components can be transferred through bond, interface joint friction, interlocking by shear keys, dowel action of transverse steel bars or rods, welding or by other mechanical means.

Shear transfer by bond between precast and in-situ elements is possible when the shear stress is low. It is not necessary to deliberately roughen the surface texture of precast units beyond the as-cast finish which may be of slip-forming, extrusion or tamped finish.

Shear transfer by shear friction requires the presence of a permanent normal compressive force. The force may arise from permanent gravity loads, by prestressing or artificially induced by reinforcement bars placed across the joint.

Shear keys for the transfer of shear forces between elements are obtained by cast in-situ concrete or grout in joints between the elements with surface castellations. Under the action of shear load, the shear keys act as mechanical locks that prevent significant slip at the interface.

When steel bars or rods are placed across the joint, shear forces can be transferred between elements by dowel action as shown in Figure 3.4. The dowel is loaded by shear at the joint interface and

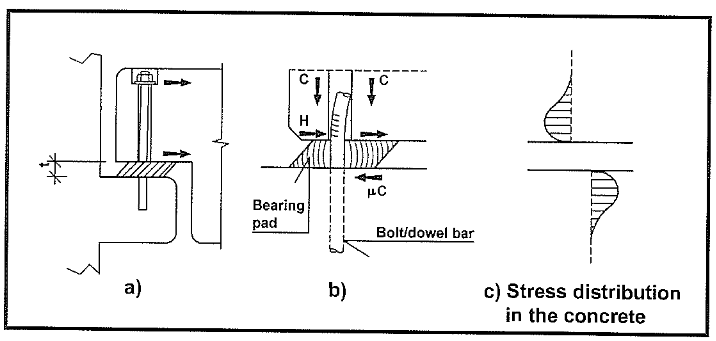


Figure 3.4 Shear Transfer By Dowel Action (reference 3)

supported by contact stresses in the concrete which result in significant bending deformation in the dowel. In the ultimate state, the concrete crushes locally at the contact area and plastic hinge forms at the dowel. Shear capacity depends on the bar diameter and the strength of the concrete. The capacity by dowel action decreases considerably when the dowel is loaded by eccentric shear away from the interface. It is necessary to provide splitting reinforcement around the dowels particularly if they are placed near to the edge or corner of a component. Combined action by shear friction and dowel action can be obtained if the dowels are anchored by bond or by end anchorages.

3.6.4 Flexural and torsional joints

Forces acting in a flexural or torsional joint can always be resolved into tension and compression force couples. The principle in the connection design is based on splicing of reinforcement between units by means of overlapping, bolting or welding as discussed earlier. In the case of torsion, the resulting torque in a precast component is resisted by force couples at the support. The resulting torque is transformed into bending moment in the supporting members.

3.7 Shear Friction Design Method

Shear friction is a very useful and simple method in connection design as well as in the application to the design of composite structures. A basic assumption in the concept is that a crack will form at a potential failure plane where direct shear stresses are high; or at actual planes of weakness created during construction. Propagation of this crack is inhibited by ductile steel reinforcement placed across the cracks. The tension developed by these shear friction reinforcement will provide a force normal to the crack plane as shown in Figure 3.5. If the shear friction coefficient at the crack surface is μ, the resistance to the shear force parallel to the crack, V, is given as

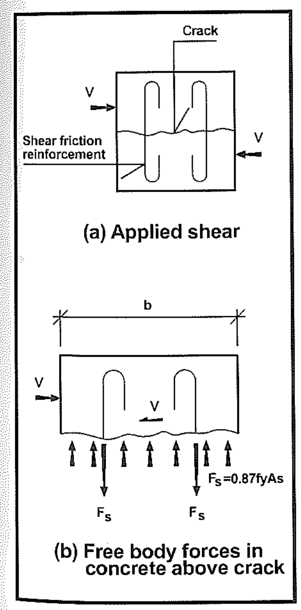


Figure 3.5 Basis Of Shear Friction Design Method

$$V = 0.87 f_y A_s \mu$$

$$A_s = V / (0.87 f_y \mu) \quad \text{— (3.3)}$$

Values of shear friction coefficient μ for various surface conditions are shown in the table below (reference 1).

Type of Surface	μ
Smooth untreated concrete interface	0.7
Artificially roughened or castellated surface	1.4
Monolithic concrete	1.7

Table 3.1 Shear Friction Coefficients For Various Concrete Surfaces

If there is an applied force N, acting normal to the crack plane, equation 3.3 will be modified as follow:

If N is compression : $A_s = (V / \mu - N) / (0.87 f_y)$ — (3.4)

If N is tension : $A_s \quad = (V / \mu + N) / (0.87f_y)$ ——(3.5)

When the interface shear stress reaches the limiting value of $v_c = 0.8\sqrt{f_{cu}}$ or 5 N/mm^2 whichever is smaller, no further increase of shear friction is allowed. This limiting shear stress will provide a limit to the steel proportion ($\rho_s = A_s/bd$) placed across a crack as

$$\rho_s \quad = v_c / (0.87 f_y \, \mu) \qquad \text{——(3.6)}$$

3.8 Static Friction

The maximum force resulting from frictional restraint of axial movements can be determined by :

$$N \quad = \mu_s V \qquad \text{——(3.7)}$$

where V = design vertical force
μ_s = static coefficient of friction
N = horizontal frictional resistance

The static coefficients of friction are shown in Table 3.2. The coefficients of friction are for a dry condition and the values should be reduced by 15% to 20% for moist conditions. If friction is to be depended upon for support of temporary loads at construction, the coefficients should include a safety factor of 5 (reference 9).

Materials	μ_s
Elastomeric pads to steel or concrete	0.7
Concrete to concrete	0.8
Concrete to steel	0.4
Steel to steel (not rusted)	0.25
Hardboard to concrete	0.5
Laminated cotton fabric to concrete	0.6
Multipolymer plastic (non-skid) to concrete	1.2
Multipolymer plastic (smooth) to concrete	0.4

Table 3.2 Static Coefficients Of Friction Of Dry Material (reference 9)

3.9 Bearing On Concrete

The effective bearing area for a structural member may be determined, based on the weaker of the bearing surfaces and using the ultimate concrete bearing stress as follows (reference 1):

1. direct bearing without bedding or bearing pads = 0.4 f_{cu}
2. with intermediate bedding = 0.6 f_{cu}
3. with cast-in steel bearing plates = 0.8 f_{cu}

Bearing using flexible padding may be designed using stresses intermediate between (1) and (2). When the bearing stresses exceed the above limits, reinforcement is to be provided in the bearing area. Reinforcement may be determined using the shear friction design method.

Figure 3.6 shows a bearing area where reinforcement is provided across the potential vertical and horizontal cracks induced by vertical load V and horizontal load N.

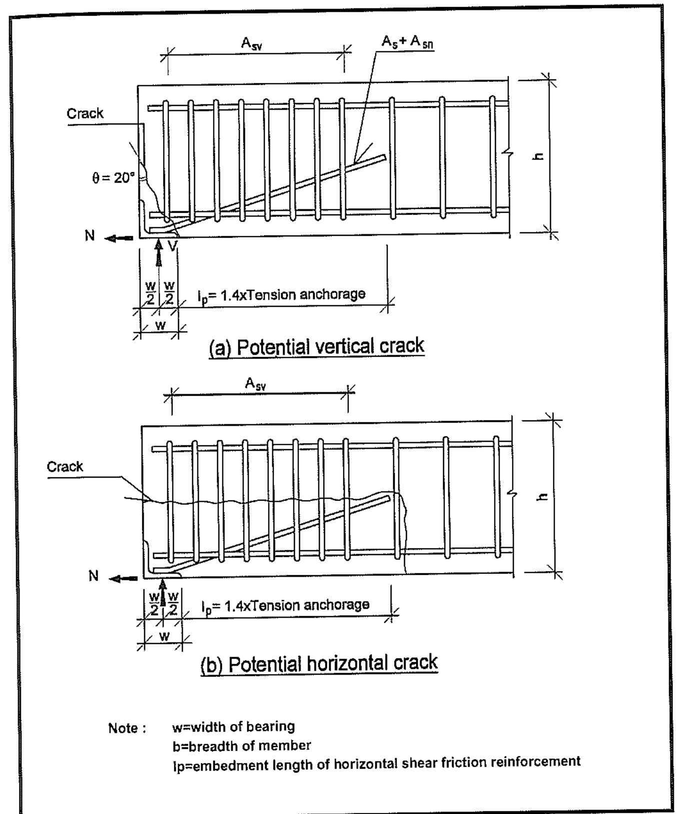


Figure 3.6 Shear Friction Reinforcement At Potential Cracks At Bearing Area

3.9.1. Horizontal shear friction reinforcement

The horizontal shear friction reinforcement in Figure 3.6a is calculated as follows:

$$A_s \quad = \quad V_\theta / (0.87 f_{yn} \, \mu) \qquad \text{— (3.8)}$$

where V_θ = V/cos θ
θ = shearing angle (assumes to be 20°)
μ = shear friction coefficient
f_{yn} = characteristic strength of horizontal shear friction reinforcement

The average shear stress, v_θ at the inclined crack plane is given as

$$v_\theta = V \tan\theta / bw \qquad (3.9)$$

and $v_\theta < v_c$, where $v_c = 0.8\sqrt{f_{cu}}$ or 5 N/mm², whichever is smaller.

If N is a tensile force

$$A_{sn} = N / (0.87\, f_{yn}) \qquad (3.10)$$

Because of the uncertainty of the exact location of the crack, the horizontal shear friction reinforcement should at least have a total embedment length of 1.4 times the tension anchorage value.

3.9.2 Vertical shear friction reinforcement

Potential horizontal crack may be formed as the entire anchorage assembly has the tendency to be pulled horizontally out of the member. The required vertical shear friction reinforcement across the horizontal crack can be calculated as:

$$A_{sv} = 0.87\, f_{yn} (A_s + A_{sn}) / (0.87 f_{yv}\, \mu) \qquad (3.11)$$

where f_{yv} = yield strength of vertical reinforcement

If $f_{yn} = f_{yv}$

$$A_{sv} = (A_s + A_{sn}) / \mu \qquad (3.12)$$

Shear links used for diagonal tension reinforcement can be considered to act as A_{sv}. The average shear stress, v_h, along the horizontal crack plane is given as

$$v_h = 0.87 f_{yv} A_{sv} / (b l_p) \qquad (3.13)$$

where l_p = embedment length of the horizontal shear friction reinforcement

and $v_h < v_c$

The designer should note that the use of cast-in steel items in reinforced end bearing of precast components as those shown in Figure 3.6 may entail higher production cost and complicate manufacturing processes. Unless necessary, the preferred method of reinforced end bearing should adopt looped reinforcement as shown in Figure 3.7.

The design of the looped reinforcement is similar in approach as outlined above.

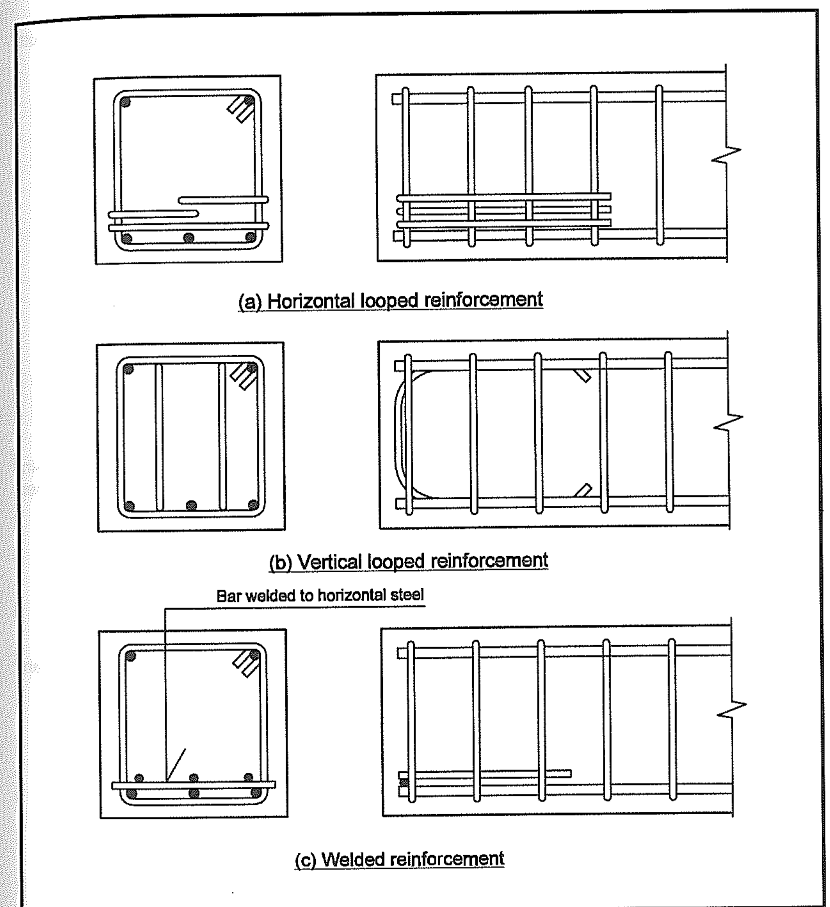
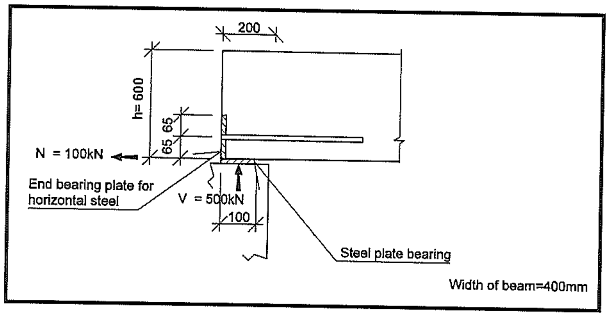


Figure 3.7 Concrete Bearing With Looped Reinforcement

Design Example 1 : Reinforced Concrete Bearing

Design the end bearing of a precast beam 400 x 600mm deep which is subjected to an ultimate vertical reaction of V = 500 kN and a horizontal tension force of N = 100 kN. The beam rests on steel plate at the support. The design concrete cube strength for both beams and supporting member is f_{cu} = 35 N/mm²

Figure below shows general details of the end bearing with the horizontal shear friction steel butt-welded to a cast-in end plate in the beam:



1. **Effective bearing width**

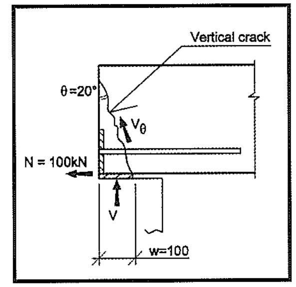
Steel plate is used as bearing for the beam at the support. As the bearing plate is not cast-in, the design of the ultimate concrete bearing stress is taken as $0.6f_{cu}$ in the support member. Assuming length of the plate l_b= 300mm, the effective width (w) of bearing

$$w = V / (0.6\ f_{cu} \times l_b)$$
$$= 500 \times 10^3 / (0.6 \times 35 \times 300)$$
$$= 79\ \text{mm}$$

adopt w = 100 mm

For the most critical vertical shear crack, the plate is assumed to be flushed with the beam end face.

2. **Horizontal shear friction reinforcement**



Vertical Shear Crack

a. **Determination of A_s**

From equation 3.8, the shear friction steel area perpendicular to the inclined vertical crack is

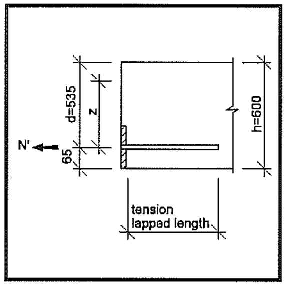
$$A_{s\theta} = V_\theta / (0.87 f_{yn}\ \mu)$$
$$V_\theta = V / \cos\theta$$
$$\theta = 20°$$
$$\mu = 1.7\ \text{(monolithic concrete)}$$
$$A_{s\theta} = 500 \times 10^3 / (0.87 \times 460 \times 1.7 \times \cos 20°)$$
$$= 782\ \text{mm}^2$$

The crack is inclined at an angle of 70° to the horizontal shear friction reinforcement. Resolve V_θ vertically, i.e., crack plane perpendicular to the horizontal shear friction reinforcement.

$$A_s = A_{s\theta} \cos\theta$$
$$= 782 \times \cos 20°$$
$$= 735\ \text{mm}^2$$

Note: The above intends to show, the determination of the shear friction reinforcement across a potential crack from first principle. The calculations can in fact be simplified by directly calculating $A_s = V/(0.87 f_y \mu)$.

b. Determination of A_{sn}



Horizontal Force At Bearing

Resolving the force N at the level to horizontal shear friction reinforcement which is 65 mm above the beam soffit.

$$N' = N\,(h/z - d/z + 1)$$

where z is the lever arm of tension steel to centroid of the compressive block. Assume conservatively z = 0.8d then

$$\begin{aligned} N' &= 1.25N\,(h/d - 0.20) \\ &= 1.25 \times 100\,(600/535 - 0.20) \\ &= 115.2 \text{ kN} \end{aligned}$$

$$\begin{aligned} A_{sn} &= 115 \times 10^3 / (0.87 \times 460) \\ &= 287 \text{ mm}^2 \end{aligned}$$

c. Total horizontal shear friction reinforcement

$$\begin{aligned} A_s + A_{sn} &= 735 + 287 \\ &= 1022 \text{ mm}^2 \end{aligned}$$

Provide 4T20 (A_s = 1257 mm^2) welded to the beam end plate.

d. Check ultimate shear stress

From equation 3.9, the average shear stress V_θ along the crack plane is:

$$\begin{aligned} v_\theta &= V \tan\theta / bw \\ &= 500 \times 10^3 \tan 20° / (400 \times 100) \\ &= 4.55 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} v_c &= 0.8\sqrt{35} \\ &= 4.73 \text{ N/mm}^2 > v_\theta \end{aligned}$$ OK

3. Vertical shear friction reinforcement

The vertical reinforcement, A_{sv}, across potential horizontal shear cracks is calculated from equation 3.11.

$$\begin{aligned} A_{sv} &= (A_{sv} + A_{sv}) / \mu \\ &= 1022 / 1.7 \\ &= 601 \text{ mm}^2 \end{aligned}$$

The A_{sv} is to be checked against shear links provision. If the shear links provision is less than A_{sv}, then provide A_{sv}.

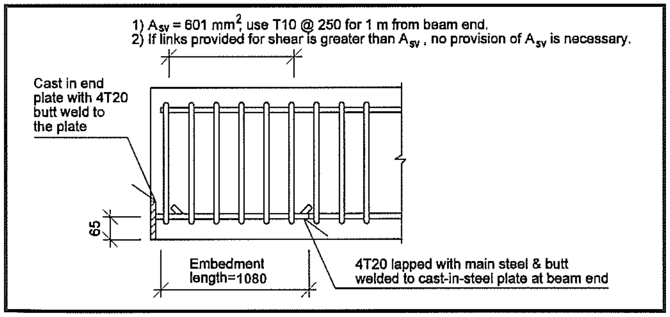
Check shear stress at horizontal crack face

$$\begin{aligned} \text{Embedded length of T20} &= 1.4 \times 35\,\varphi + w \\ &= 980 \text{ mm} + 100 \\ \text{say} \quad l_p &= 1080 \text{ mm} \end{aligned}$$

Average shear stress v_h at the horizontal crack face is calculated using equation 3.13

$$\begin{aligned} v_h &= 0.87\, f_{yv}\, A_{sv} / (b \times l_p) \\ &= 0.87 \times 460 \times 601 / (400 \times 1080) \\ &= 0.56 \text{ N/mm}^2 < 4.73 \text{N/mm}^2 \end{aligned}$$ OK

4. Detailing



3.10 Reinforced Concrete Corbel

Figure 3.8 illustrates the definitions of reinforced concrete corbel in accordance with the Code in Part 1, clause 5.2.7.1.

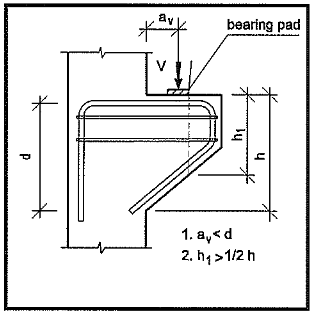


Figure 3.8 Definition Of Corbel

Reinforced concrete corbel may be designed either by:

1. Strut and tie system
2. Shear friction method

3.10.1 Corbel design by strut and tie force system

The design of reinforced concrete corbel may assume a strut and tie force system as shown in Figure 3.9.

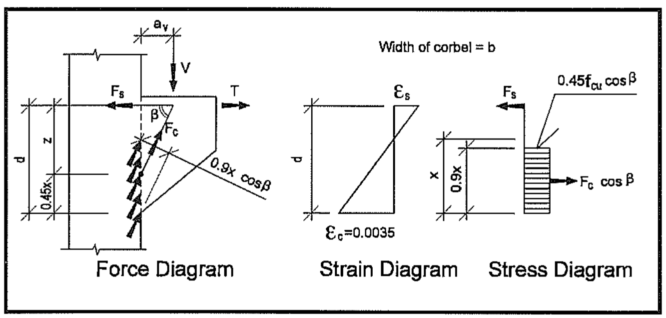


Figure 3.9 Strut And Tie Force System

The design may follow the following steps:

1. **Shear at support face**

The minimum depth of corbel is determined from

a. $$V / bd \le 2dv_c / a_v$$

$$d \ge \sqrt{Va_v / (2b\, v_c)} \qquad (3.14)$$

b. $$V / bd \le v_c$$

$$d \ge V / bv_c \qquad (3.15)$$

where v_c in b is equal to $0.8\sqrt{f_{cu}}$ or $5N/mm^2$, whichever is smaller.

2. **Strut and tie forces**

From Figure 3.9, the strut and tie forces are derived as :

Tension tie forces :

$$F_s = T + F_c \cos\beta$$

$$F_s = T + V\, a_v/z \qquad (3.16)$$

where T is the horizontal force acting at the support.

3. **Compressive strut forces**

$$F_c = 0.45f_{cu}\, b(0.9\, \chi \cos\beta)$$

$$F_c = 0.405f_{cu}\, b\, \chi \cos\beta \qquad (3.17)$$

4. **Derivation of z/d**

$$V = F_c \sin\beta$$

From equation 3.17, $V = 0.405f_{cu}\, b\, \chi \cos\beta \sin\beta$

$$\cos\beta = \frac{a_v}{(a_v^2+z^2)^{1/2}}$$

$$\sin\beta = \frac{z}{(a_v^2+z^2)^{1/2}}$$

$$V = 0.405f_{cu}\, b\, \chi \frac{a_v\, z}{(a_v^2 + z^2)}$$

$$v = V/bd$$

$$v = 0.405f_{cu} \left(\frac{z}{d}\right) \frac{\chi\, a_v}{(a_v^2 + z^2)} \qquad (3.18)$$

Substituting $\chi = (d - z)/0.45$ into equation 3.18 and rearranging the terms will result in the following simplified expressions :

$$\frac{v}{f_{cu}} = \frac{0.9(z/d)(a_v/d)(1 - z/d)}{(a_v/d)^2 + (z/d)^2} \quad \text{— (3.19)}$$

From equation 3.19, a graph for the determining of z/d is shown in Figure 3.10.

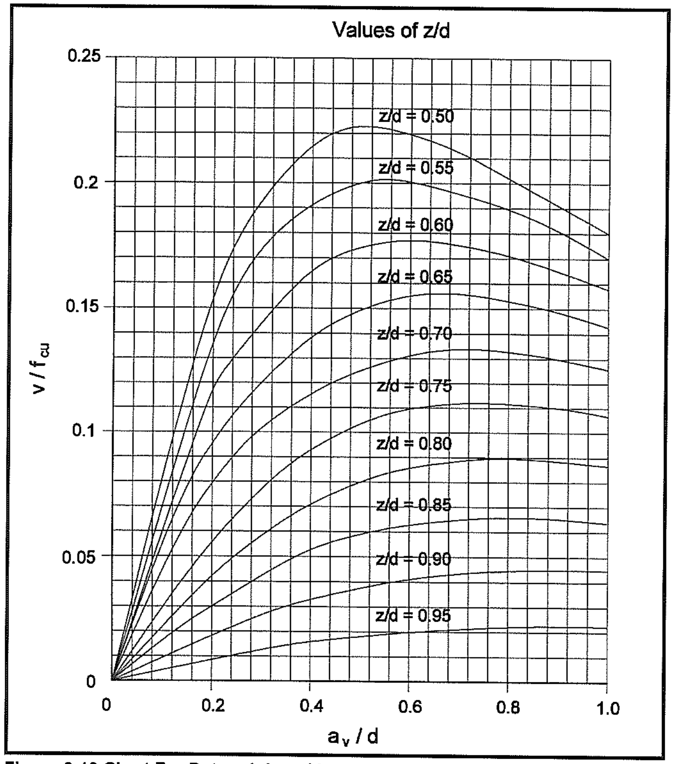


Figure 3.10 Chart For Determining z/d

Part 1, clause 5.2.7.2 requires that the resistance provided to horizontal tie force should not be less than one-half of the design vertical load and in the absence of external horizontal forces,

$$F_s = V(a_v / z)$$
$$= V(a_v / d) / (z / d)$$

If

If $F_s = 0.5V$, then

$$z/d \geq 2\, a_v/d \quad \text{— (3.20)}$$

Substituting equation 3.20 into equation 3.19

$$v/f_{cu} = 0.36(1 - 2a_v/d) \quad \text{— (3.21)}$$

Equation 3.21 provides the boundary below which F_s is taken as $F_s = V/2$

5. **Steel stresses**

From the strain diagram in Figure 3.9,

$$\varepsilon_s/(d - \chi) = \varepsilon_c/\chi$$

Substituting $\varepsilon_c = 0.0035$, $\chi = (d - z)/0.45$, $f_s = E_s\varepsilon_s$ and $E_s = 200$ kN/mm^2 into the above equation, the steel stress can be calculated :

$$f_s = 700\,(z/d - 0.55) / (1 - z/d)\ (\text{N/mm}^2) \quad \text{— (3.22)}$$

The main tension reinforcement may be considered anchored by:

a. welding to a transverse bar of equal strength or
b. bending the bars to form a loop

The length geometry of the corbels with anchorage of reinforcement using method (a) or (b) and the relative positioning of bearing plates or bedding are illustrated in Figure 3.11.

Shear reinforcement, if required, must not be less than 50% of the main tension steel area and it consists of horizontal links located within the upper 2/3 of the effective depth of the corbel.

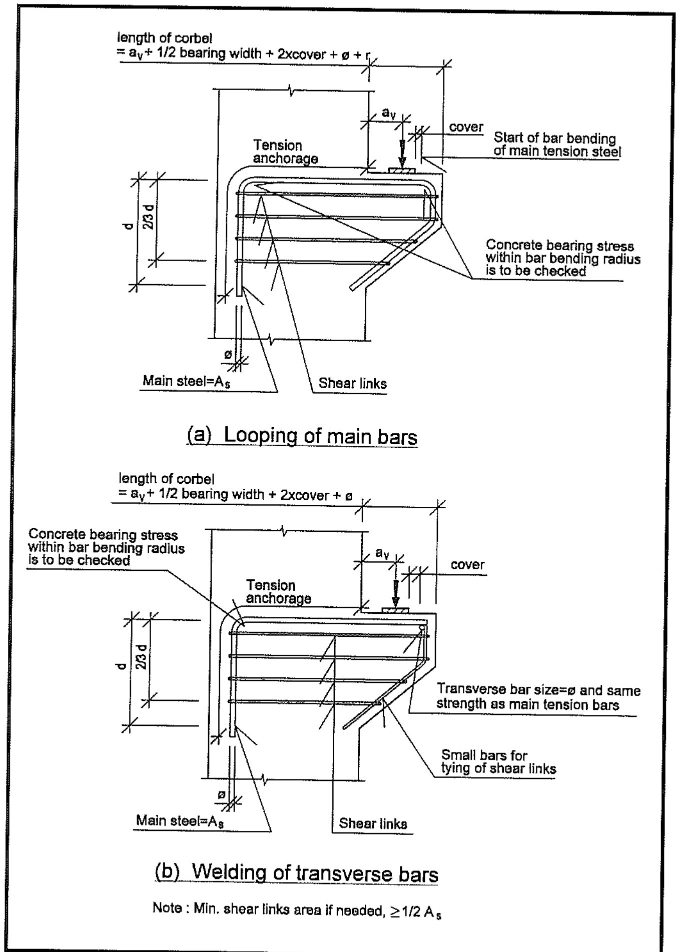
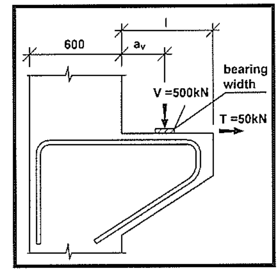


Figure 3.11 Anchorage And Relative Position Of Bearing Plates In R.C. Concrete Corbel

Design Example 2: Corbel Design By Strut And Tie Forces System

Design a reinforced concrete corbel to support a vertical ultimate load V = 500 kN and an ultimate horizontal force of **T** = 50 kN. The corbel is projected from a column 600 x 400 with width 400 mm and a_v = 100 mm from the face of the column. Bearing plate is used to transmit both vertical and horizontal load to the corbel. Assume cover to all steel = 35 mm and f_{cu}= 35 N/mm².



1. Corbel geometry

a. Assume tension bar = $\phi16$
link size = $\phi10$

b. Bearing width :
Length of bearing width l_b = 350 mm
Maximum bearing stress = $0.6f_{cu}$

Width of bearing, w $= V/0.6f_{cu}\ l_b$
$= 500 \times 10^3/(0.6 \times 35 \times 350)$
= 63.5 mm
say w = 65 mm

c. Length of corbel (assuming looped tension reinforcement)

Length of corbel $= a_v + 0.5w + 2 \times \text{cover} + \phi_{link} + \phi_{main}$ + bend radius of main steel (4ϕ)
= 100 + 0.5 x 65 + 2 x 35 + 10 + 16 + 4 x 16
= 292.5 mm,
say = 300 mm

d. Depth of corbel

Assume overall depth of corbel h= 400mm and with the following dimensions :

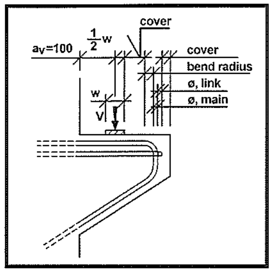
h_1 = 200 + (167.5 x 200/300)
= 311mm > h/2 OK

d = 400 - 35 - 8
= 357mm > a_v OK

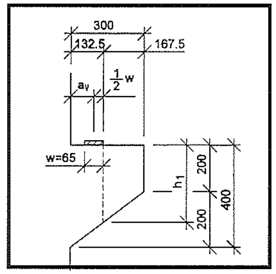
e. Check shear stress

ν = V / bd
$= 500 \times 10^3/(400 \times 357)$
= 3.50 N/mm² < $0.8\sqrt{f_{cu}}$ OK
= 4.73 N/mm²

The final dimensions of the corbel are shown below:



Length Of Corbel



Depth Of Corbel

2. Corbel reinforcement

a. Main tie steel

Check if $F_s > V / 2$

From equation 3.21, $v/f_{cu} = 0.36(1 - 2a_v/d)$

$v/f_{cu} = 0.158$

But $v/f_{cu} = 3.5/35 = 0.100 < 0.158$

Hence, design for minimum tension

$F_s = 0.5V$
$= 0.5 \times 500$
$= 250$ kN

Total tension force in tie $= F_s + T$
$= 250 + 50$
$= 300$ KN

b. Check steel stress

From equation 3.19,

$$\frac{v}{f_{cu}} = \frac{0.9(z/d)(a_v/d)(1 - z/d)}{(a_v/d)^2 + (z/d)^2}$$

$v/f_{cu} = 0.10$
$a_v/d = 0.28$

Substituting $v/f_{cu} = 0.10$ and $a_v/d = 0.28$ into the equation and re-arranging the terms, the following quadratic equation for z/d is obtained :

$(z/d)^2 - 0.716(z/d) + 0.0227 = 0$
$z/d = 0.683$

Alternatively, z/d may be obtained directly from the graph in Figure 3.9.
Substituting z/d = 0.683 into equation 3.22

$f_s = 700(0.683 - 0.55)/(1 - 0.683)$
$= 293$ N/mm^2

Hence, tension steel area

$A_s = (F_s + T)/f_s$
$= 300 \times 10^3/293$
$= 1024$ mm^2 $>$ 0.004bd (min A_s) OK

Provided 6T16 (A_s=1207mm^2)

c. Check bearing stress within bend

Tensile force per bar

$$F_s' = \frac{(F_s + T)}{\text{numbers of bar}} \times \frac{A_{s\,req}}{A_{s\,prov}}$$

$= (300/6) \times (1024/1207)$
$= 42.4$ kN

Minimum bend radius

$$r = \frac{F_s'}{\phi} \times \frac{1 + 2(\phi/a_b)}{2f_{cu}}$$

$a_b = (400 - 35 - 35 - 16)/5 = 62.8$ mm say $= 60$ mm

$r = 42.4/16 \times 10^3 \times (1 + 2 \times 16/60) / (2 \times 35)$
$= 58$mm

$r = 4\phi$ as assumed OK

d. Shear links

$\rho_s = 100A_s/bd$
$= 100 \times 1207/(400 \times 357)$
$= 0.845$

From Table 3.9 Part 1 of the Code

$v_c = 0.67$ N/mm^2

Enhanced $v_c' = 2d\, v_c/a_v$
$= 2 \times 357 \times 0.67/100$
$= 4.77 > 0.8\sqrt{f_{cu}}$

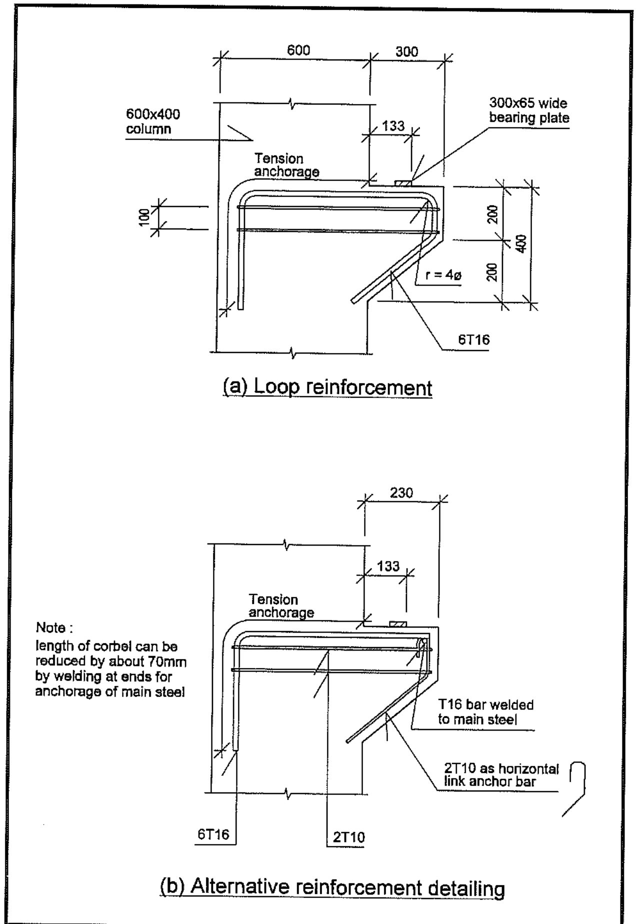
Hence $v_c' = v_c = 4.73$ N/mm^2

Shear stress $v = 500 \times 10^3/(400 \times 357)$
$= 3.50$ N/mm^2 $< v_c'$ OK

No shear links needed. But provide 25% of main steel for confinement purpose as well as for binding of the main steel.

Link area $= 0.25 \times A_{s,\,req}$
$= 0.25 \times 1045$
$= 261$ mm^2

Provide 2 numbers of T10 links (4 legs)



(a) Loop reinforcement

(b) Alternative reinforcement detailing

Corbel Reinforcement Design Using Strut And Tie Forces System

3.10.2. Corbel design by shear friction method

The design of corbel by shear friction method involves the investigation of several potential crack planes as illustrated in Figure 3.12. The associated reinforcement for each of the crack plane being considered is as follows:

1. Flexural (cantilever bending) and axial tension at the corbel projection. Provide reinforcement A_s (flexural) and A_{sn} (axial tension).
2. Direct shear at corbel junction with main member from which the corbels are projected. Provide shear friction A_s and A_{sh}.
3. Diagonal tension in corbel. Provide shear friction reinforcement A_{sh}
4. Inclined shear crack at bearing. Provide A_s and A_{sh} and ensure anchorage of these steel is achieved.

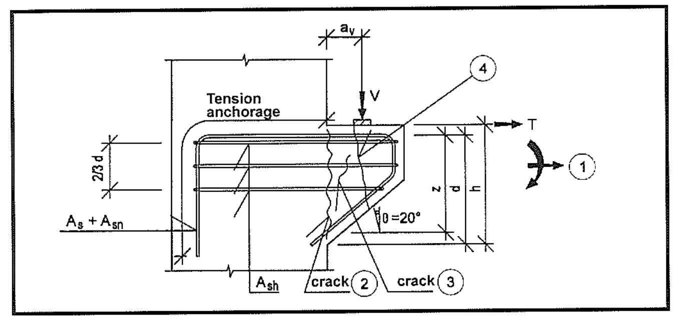


Figure 3.12 Potential Cracks In Corbel

The reinforcement calculated above is not cumulative but the greater of the different considerations. As stated in the shear friction design method, the average shear stress at any crack plane must be less than or equal to $0.8\sqrt{f_{cu}}$ or 5 N/mm², whichever is smaller.

The reinforcement by shear friction design method is calculated as below:

1. Flexural and axial tension steel

Flexural steel : $A_s = V(a_v / z) / (0.87f_y)$

Axial steel : $A_{sn} = T(h / z - d / z + 1) / (0.87f_y)$

Assuming z = 0.8d, then

$$A_s = 1.25V \times (a_v / d) / (0.87f_y) \quad \text{— (3.23)}$$

$$A_{sn} = 1.25T \times (h / d - 0.2) / (0.87f_y) \quad \text{— (3.24)}$$

2. **Direct shear at junction with main member**

$$A_s = \frac{2}{3} \times V / (0.87 f_y \mu) \qquad \text{— (3.25)}$$

$$A_{sh} = \frac{1}{3} \times V / (0.87 f_y \mu) \qquad \text{— (3.26)}$$

A_{sn} is as in equation 3.22 above. A_{sh} should be distributed uniformly within (2/3)d of the corbel depth and minimum $A_{sh} = 0.5(A_s + A_{sn})$

3. **Bearing on corbel**

The design approach is described in Section 3.9.

Design Example 3: Corbel Design By Shear Friction Method

Redesign the corbel in Design Example 2 using shear friction design method. Adopt the corbel geometry and all relevant design parameters.

$V = 500$ KN
$T = 50$ KN
$a_v = 100$ mm
$d = 357$ mm
$z = 0.8d$
$= 286$ mm
$\mu = 1.7$ (monolithic concrete)
$f_y = 460\text{N/mm}^2$ for all steel bars

1. **Flexural and axial tension steel**

From equation 3.23,

$$\begin{aligned} A_s &= 1.25V\,a_v/(0.87 f_y\, d) \\ &= 1.25 \times 500 \times 10^3 \times 100/(0.87 \times 460 \times 357) \\ &= 437 \text{ mm}^2 \end{aligned}$$

From equation 3.24,

$$\begin{aligned} A_{sn} &= 1.25T(h/d - 0.2)/0.87 f_y \\ &= 1.25 \times 50 \times 10^3(400/357 - 0.2)/(0.87 \times 460) \\ &= 144 \text{ mm}^2 \end{aligned}$$

$A_s + A_{sn} = 581 \text{ mm}^3$

2. **Direct shear at junction**

From equation 3.25,

$$A_s = \frac{2}{3} \times V / (0.87 f_y\, \mu)$$

$$\begin{aligned} A_s &= 2 \times 500 \times 10^3/(3 \times 0.87 \times 460 \times 1.7) \\ &= 490 \text{ mm}^2 \end{aligned}$$

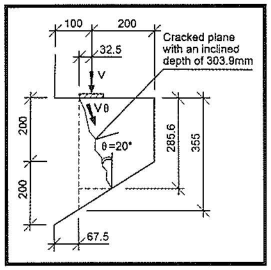
From equation 3.26,

$$\begin{aligned} A_{sh} &= \frac{1}{3} \times V / (0.87 fy\, \mu) \\ &= 500 \times 10^3/(3 \times 0.87 \times 460 \times 1.7) \\ &= 245 \text{ mm}^2 \end{aligned}$$

$A_{sn} = 144 \text{ mm}^2$ as in 1 above.

Required total steel area:
$A_s + A_{sn} = 634 \text{ mm}^2$
$A_{sh} = 245 \text{ mm}^2$

3. **Bearing on corbel**



Inclined Crack At Bearing

From equation 3.8 and resolving inclined crack plane vertically

$A_s = V/0.87f\mu$
$= 500 \times 10^3/(0.87 \times 460 \times 1.7)$
$= 735 \text{ mm}^2$

From equation 3.8 and transferring the axial force to the A_{sn} level

Then $A_{sn} = 1.25T\,(h/d - 0.2)/(0.87f_y)$

as in (1) above, $A_{sn} = 144 \text{ mm}^2$

$A_s + A_{sn} = 735 + 144$
$= 879 \text{ mm}^2$

4. **Check average shear stress at inclined crack plane at bearing**

$v_\theta = V/(\cos\theta \times bh)$
$= 500 \times 10^3/(\cos 20^\circ \times 400 \times 303.9)$
$= 4.38 \text{ N/mm}^2 < 0.8\sqrt{f_{cu}} = 4.73 \text{ N/mm}^2$

5. **Design steel area**

Steel area calculated from 1 to 3

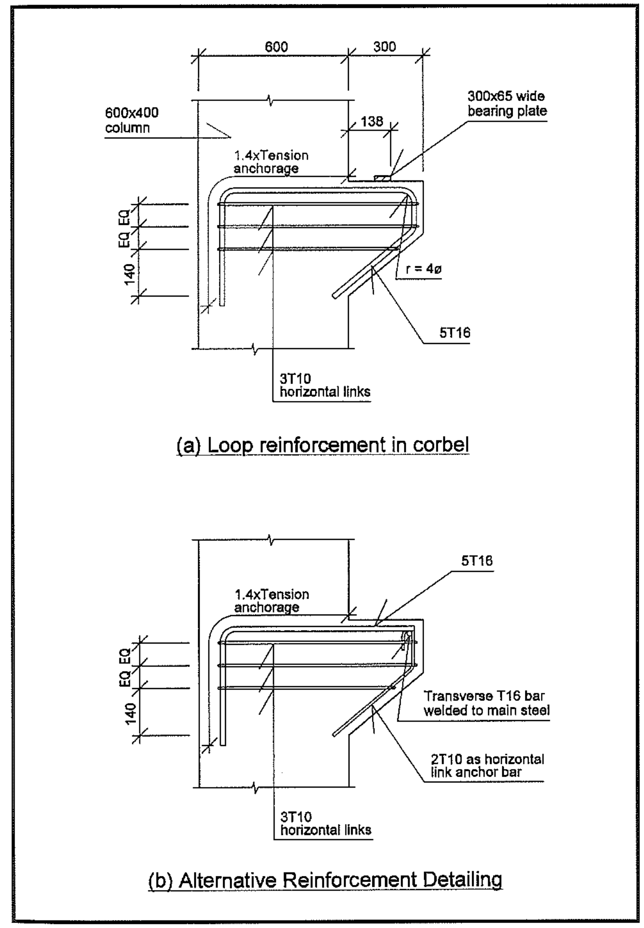
i. Max $A_s = 735 \text{ mm}^2$
Max $A_{sn} = 144 \text{ mm}^2$
$A_s + A_{sn} = 879 \text{ mm}^2$, use 5T16 ($A_s$=1006mm^2)

ii. Max $A_{sh} = 245 \text{ mm}^2$
Check Minimum $A_{sh} = 0.5\,(A_s + A_{sn})$
$= 0.5\,(735 + 144)$
$= 440 \text{ mm}^2$

use 3T10 (A_{sh} = 471mm^2, 2 legs) and space them at (2/3)d in the corbel.

The checks for concrete bearing stress for T16 bars are similar to Design Example 2.

6. **Detailing**



(a) Loop reinforcement in corbel

(b) Alternative Reinforcement Detailing

Corbel Reinforcement Designed Using Shear-Friction Method

3.11 Reinforced Concrete Nib

Reinforced concrete nibs are short cantilever projections from walls, columns or beams to provide support for floor elements. The nibs are usually less than 300 mm deep with a_v greater than the effective depth. Figure 3.13 illustrates the line of action of vertical load and the various a_v as defined in accordance with Part 1 clause 5.2.8.1 of the Code.

The concrete nibs are designed as cantilever slab where the moment in the nib is taken as :

$$M = V \times a_v \qquad \text{— (3.27)}$$

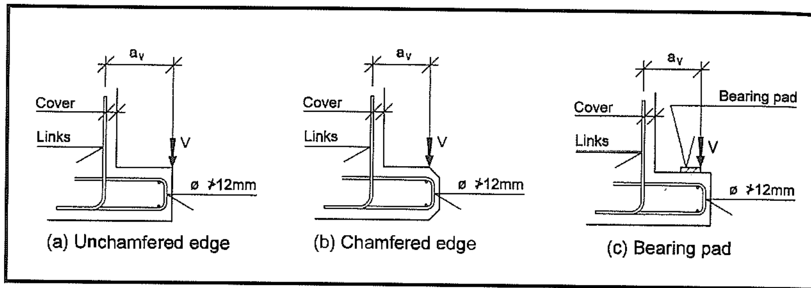


Figure 3.13 a_v In Concrete Nib

The design concrete shear stress v_c may be enhanced by a factor $2d/a_v$. As in the slab design, the depth should be sufficiently deep to avoid the provision of shear links within the nib.

The main tension reinforcement may be anchored in a similar manner as in corbel. The size of main tension steel should be less than 12 mm in diameter.

Vertical reinforcement consisting of links should be provided in the member from which the nib projects. The reinforcement should be designed to carry the vertical load on the nib.

For isolated loads in a continuous nib, the effective width of load dispersal may assume to be at a 45° angle line of failure as shown in Figure 3.14.

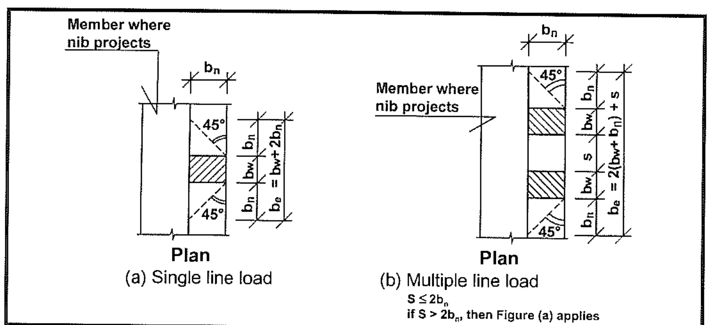
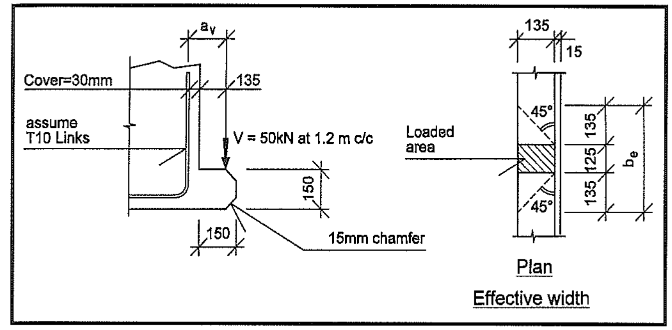
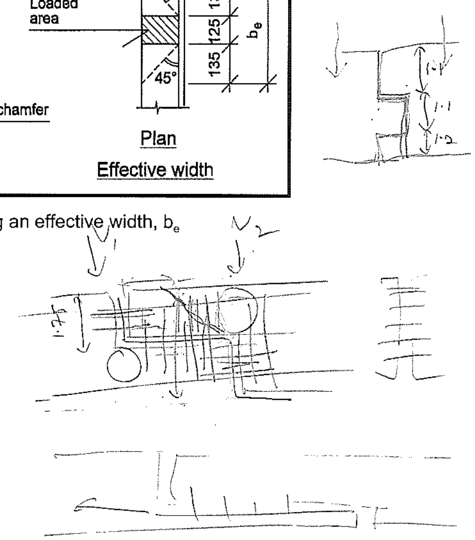


Figure 3.14 Effective Width In Nib Design For Single And Multiple Line Loads On Nib

Design Example 4: Reinforced Concrete Nib

Design the reinforcement in the nib of a beam supporting multiple ultimate line loads of 50 kN at 1.2 m spacing. The loaded width is 125 mm; the nib is 150 mm wide and 150 mm deep with 15 mm chamfered at the outer edge. Concrete mortar is used as bedding material. Adopt design concrete strength f_{cu} = 35 N/mm² and concrete cover = 30 mm.





The nib is designed as isolated single line load using an effective width, b_e

b_e = 2 x 135 +125
= 395 mm < 1200 mm

1. Main tension steel

a_v = 150 - 15 + 30 + 5
= 170 mm

d = 150 - 30 - 5
= 115 mm

Moment, M = V x a_v
= 50 x 0.17
= 8.5 kNm

$M / b_e d^2$ = 8.5 x 10⁶ / (395 x 115²)
= 1.63

z = 0.946d

A_s = 8.5 x 10⁶ / (0.87 x 460 x 0.946 x 115)
= 195 mm²
= 494 mm²/m

For practical reasons, the bars are to be uniformly placed in the nib with T10-100 c/c (A_s = 785 mm²/m)

2. Check bearing stress within bend

Tensile force per bar

$F_s = 0.87 f_y A_s \times (A_{s,req} / A_{s,pro})$
= 0.87 x 460 x 78 x (494/785) x 10⁻³
= 19.9 kN

Minimum bending radius :

$$r = \frac{F_s}{\phi} \times (1 + \frac{2\phi}{a_b})/2f_{cu}$$

$a_b = 100$mm

$r = (19.9 \times 10^3 / 10) \times (1 + 2 \times 10/100) / (2 \times 35)$

$= 34.1$ mm

use $r = 4\phi$

3. Vertical links within beams

Vertical links area $A_{sv} = V / (0.87f_y)$

$= 50 \times 10^3 / (0.87 \times 460)$

$= 125$ mm^2

$= 104$ mm^2/m

The required steel links are in addition to any other links resisting shear forces in the beam.

4. Check shear stress

$b_e = 395$ mm

$v = V/b_e d$

$= 50 \times 10^3/(395 \times 115)$

$= 1.10$ N/mm^2

$r_s = 0.68\%$ (T10-100 c/c)

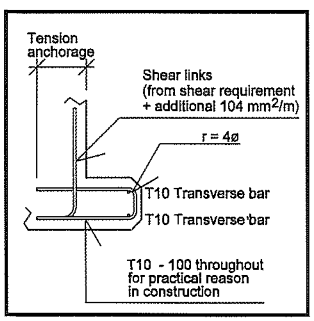
$v_c = 0.85$N/mm^2

Enhanced $v_c' = v_c \times 2d/a_v$

$= 0.85 \times 2 \times 115 / 170$

$= 1.15$N/mm^2 > 1.10mm^2 OK

5. Detailing



3.12 Beam Half Joint

The design of beam half joints involves the investigation of several potential crack planes which are illustrated in Figures 3.15 and 3.16 respectively.

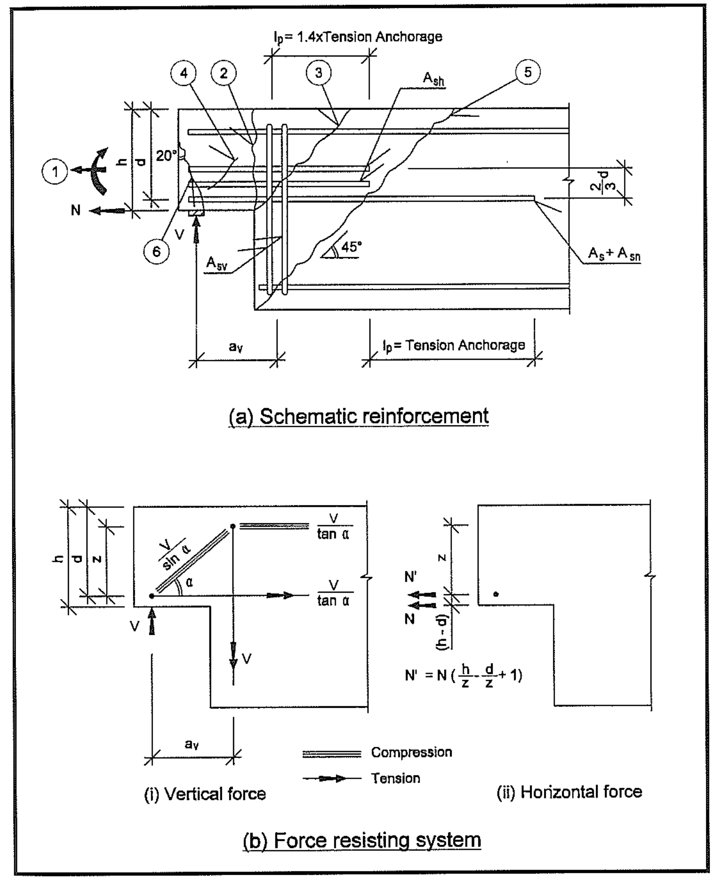
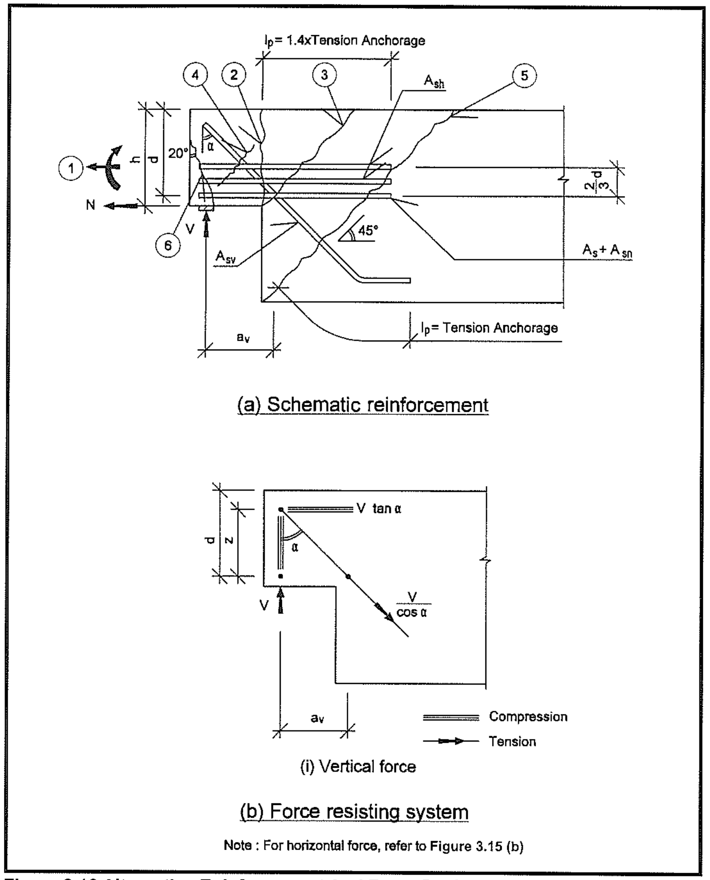


Figure 3.15 Reinforcement And Force Resisting System In Beam Half Joint



(a) Schematic reinforcement

(b) Force resisting system

Note : For horizontal force, refer to Figure 3.15 (b)

Figure 3.16 Alternative Reinforcement And Force Resisting System In Beam Half Joint

In Figures 3.15 and 3.16, the reinforcement for each of the crack plane considered is listed as below :

1. Flexural (cantilever bending) and axial tension at the extended end. Provide reinforcement A_s (flexure) and A_{sn} (axial tension).
2. Direct shear at joint junction with main member. Provide shear friction reinforcement A_s and A_{sh}.
3. Diagonal tension at re-entrant corner. Provide shear friction reinforcement A_{sv}.
4. Diagonal tension in half-joint. Provide reinforcement A_{sh}.
5. Diagonal tension in main member. Provide A_s and A_{sv} with full tension anchorage beyond the potential crack plane.
6. Inclined shear crack at beam half-joint bearing. Provide A_s and A_{sn}. Refer Section 3.9 when investigating potential horizontal crack at the bearing, although it is generally not critical.

The reinforcement as determined from (1) to (6) above is not cumulative and should be :

A_s, the greater of (1), (2), (5) or (6)
A_{sh}, the greater of (2) or (4)
A_{sv}, the greater of (3) or (5)

The determination of reinforcement in the beam half joint is based on the shear friction design method. The reinforcement across each of the potential cracks is calculated as follows :

3.12.1 Reinforcement as in Figure 3.15

1. Flexural and axial tension steel

Flexural steel : $A_s = V(a_v / z) / (0.87f_y)$ —— (3.28)

Axial tension steel : $A_{sn} = N(h / z - d / z + 1) / (0.87f_y)$ —— (3.29)

Assuming $z = 0.8d$, then

$A_s = 1.25V\ (a_v / d) / (0.87f_y)$ —— (3.30)

$A_{sn} = 1.25\ N\ (h / d - 0.2) / (0.87f_y)$ —— (3.31)

2. Direct shear at joint junction

$$A_s = \frac{2}{3} \times V / (0.87 f_y \mu) \qquad \text{—— (3.32)}$$

$$A_{sh} = \frac{1}{3} \times V / (0.87 f_y \mu) \qquad \text{—— (3.33)}$$

A_{sn} is as in equation 3.29.

A_{sh} should be uniformly distributed within 2/3d of the half joint depth.

The maximum shear stress at the joint junction is determined as

$v = V/bd \le v_c$

where $v_c = 0.8\sqrt{f_{cu}}$ or 5.0 N/mm², whichever is smaller.

3. Diagonal tension at re-entrant corner

$A_{sv} = V/0.87f_{yv}$ —— (3.34)

Based on shear friction design method, A_{sv} should be

$A_{sv} = V/0.87f_{yv}\mu$

Due to high stress concentration at the re-entrant corner, μ = 1.0 should be conservatively adopted at re-entrant corner crack plane.

4. Bearing at half joint

Refer to Section 3.9 for the design of shear friction reinforcement at the half joint bearing.

3.12.2. Reinforcement as in Figure 3.16

The alternative reinforcement in the force resisting system in Figure 3.16 is calculated as follow:

1. **Axial tension steel**

   Assuming z = 0.8d

   A_{sn} = 1.25N (h / d - 0.2) / (0.87f_y)   as in equation 3.31

2. **Diagonal tension at re-entrant corner**

   Adopting μ = 1.0

   A_{sv} = V / (0.87f_{yv} cos α)

   where α is the angle between the diagonal tension and V.

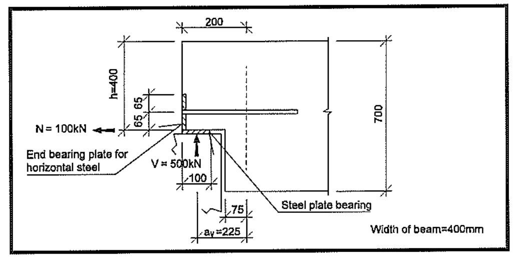
$$\cos \alpha = \frac{z}{\sqrt{a_v^2 + z^2}}$$

Assuming z = 0.8d

$$A_{sv} = \frac{1.25V}{0.87 f_{yv} d} \sqrt{a_v^2 + (0.8d)^2} \qquad \text{(3.35)}$$

Design Example 5: Beam Half Joint

Design the beam half joint shown in the figure below for an ultimate vertical reaction V = 500 kN and an ultimate horizontal tension of N = 100 kN. Design concrete cube strength is f_{cu} = 35 N/mm² and f_y = 460 N/mm² for all steel.



General design data for half-joint

h	= 400 mm
b	= 400 mm
d	= 400 - 65 = 335 mm
z	= 0.8d = 268mm
a_v	= 200 + 75 - 50 = 225 mm
V	= 500 kN
N	= 100 kN
f_{cu}	= 35 N/mm²
f_y	= 460 N/mm²
μ	= 1.0

1. **Reinforcement as in Figure 3.15**

   a. **Flexural and tension steel**

   From equation 3.30

   A_s = 1.25V(a_v / d) / (0.87f_y)
   = 1.25x 500 x 10³ x (225 / 335) / (0.87 x 460)
   = 1049 mm²

   From equation 3.31

   A_{sn} = 1.25N (h / d - 0.2) / (0.87f_y)
   = 1.25 x 100 x 10³ x (400 / 335 - 0.2) / (0.87 x 460)
   = 310 mm²

   $A_s + A_{sn}$ = 1359 mm²

b. Direct shear at joint junction

From equation 3.32

$$A_s = \frac{2}{3} \times V / (0.87 f_y \mu)$$
$$= \frac{2}{3} \times 500 \times 10^3 / (0.87 \times 460)$$
$$= 833 \text{ mm}^2$$

From equation 3.33

$$A_{sh} = \frac{1}{3} \times V / (0.87 f_{yv} \mu)$$
$$= \frac{1}{3} \times 500 \times 10^3 / (0.87 \times 460)$$
$$= 417 \text{ mm}^2$$

c. Check shear stress at joint junction

$$v = 500 \times 10^3 / (400 \times 335)$$
$$= 3.73 < 0.8\sqrt{f_{cu}} = 4.73 \text{ N/mm}^2$$

O K

d. Diagonal tension at re-entrant corner

From equation 3.34

$$A_{sv} = V / 0.87 f_{yv}$$
$$= 500 \times 10^3 / (0.87 \times 460)$$
$$= 1249 \text{ mm}^2$$

e. Bearing at half-joint

Refer to Design Example 1 where $A_s + A_{sn} = 1022$ mm²

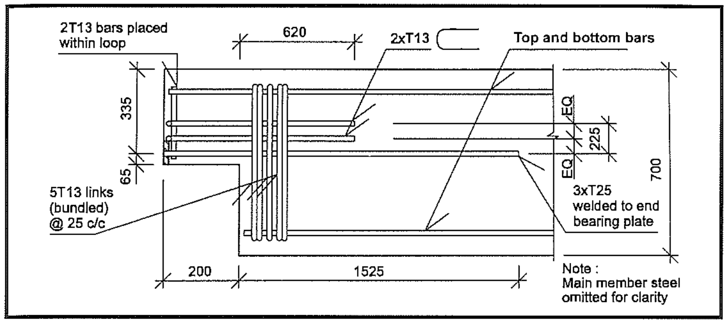
f. From (a) to (e) the greater value of A_s, A_{sn}, A_{sv}, and A_{sh} is adopted

i. $A_s + A_{sn} = 1359$ mm²
Provide 3T25 welded to bearing plate at beams end ($A_s = 1473$ mm²)

ii. $A_{sh} = 417$ mm²
Provide 2 numbers of T13 (4 legs = 531 mm²)

iii. $A_{sv} = 1249$ mm²
Provide 5T13 (10 legs = 1327 mm²)

g. Detailing



2. Alternative reinforcement as in Figure 3.16

a. Axial tension steel

From equation 3.31

$$A_{sn} = 1.25N\,(h/d - 0.2) / (0.87 f_y)$$
$$= 1.25 \times 100 \times 10^3 \times (400 / 335 - 0.2) / (0.87 \times 460)$$
$$= 310 \text{ mm}^2$$

b. Diagonal tension at re-entrant corner

From equation 3.35

$$A_{sv} = \frac{1.25V}{0.87 f_{yv} d} \sqrt{a_v^2 + (0.8d)^2}$$
$$= \frac{1.25 \times 500 \times 10^3}{0.87 \times 460 \times 335} \sqrt{225^2 + (0.8 \times 335)^2}$$
$$= 1631 \text{ mm}^2$$

c. Bearing at half joint

As in Design Example 1, $A_s + A_{sn} = 1022$ mm²

d. Adopted steel area

i. $A_s + A_{sn} = 1022$ mm²
Use 4T20 butt welded to cast-in steel plate at beam end. (Area = 1257mm²)

ii. $A_{sv} = 1631$ mm²
Use 6T20 welded with T20 transverse bar at the beam top to ensure effective anchorage.

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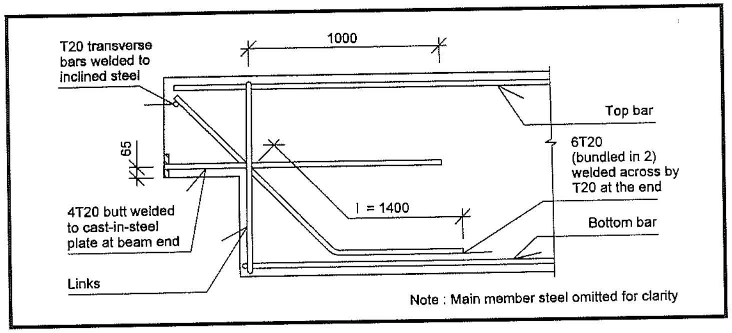
e. **Detailing**



3.13 Steel Sections Inserts

Connections between precast units may be made using embedded structural steel sections to form a simple bearing or bolted connection as illustrated in Figure 3.17. Minimum width of the steel inserts should preferably be 75 mm so that in the event of failure, it will be concrete compression rather than splitting failure. I-sections, channels, angles or hollow sections are commonly used in such connections.

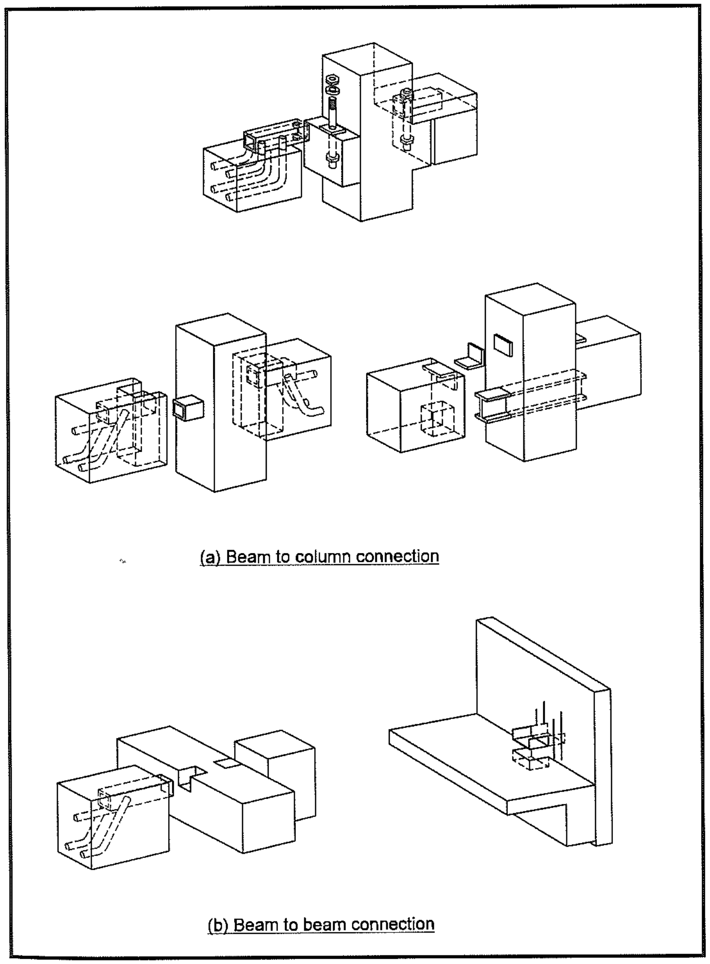


Figure 3.17 Examples Of Structural Steel Insert Connection

3.13.1. Steel inserts cast in column

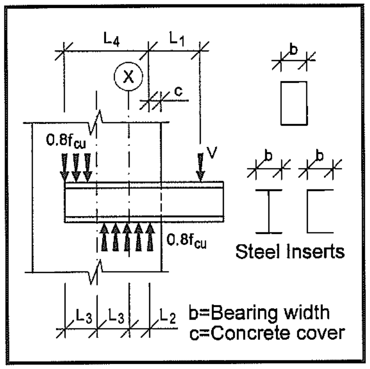


Figure 3.18 Distribution Of Forces In Column Inserts

Figure 3.18 shows a steel insert in column loaded on one side only with an embedded length L_4, and the distance between the applied load V and the effective bearing edge being L_1. The distribution length of concrete bearing stresses is defined as over L_2 and L_3. The ultimate concrete bearing stress is $0.8f_{cu}$ (clause 5.2.3.4, Part 1).

The following equations can be derived :

By geometry : $2L_3 + L_2 = L_4$ —— (3.36)

Resolve forces vertically : $V = 0.8f_{cu}L_2b$ —— (3.37)

Taking moment about point x : $0.8f_{cu}b(L_3^2+0.5L_2^2)=(L_1+L_2)V$ —— (3.38)

Combining the above equations and after rearranging,

$V = 0.8f_{cu}b\alpha L_4$ —— (3.39)

where $\alpha = [(1+2L_1/L_4)^2 + 1]^{1/2} - 2L_1/L_4 - 1$ —— (3.40)

The variations of α with L_1/L_4 are shown in Figure 3.19 below.

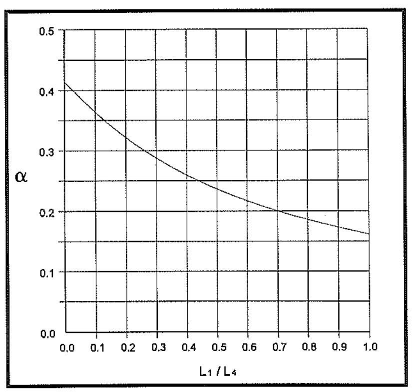


Figure 3.19 Variations Of α With L_1/L_4

The capacity of the inserts may be increased as shown in Figure 3.20 by:

a. welding additional flanges to the inserts to increase the effective bearing width, and

b. welding vertical reinforcing bars to the steel section

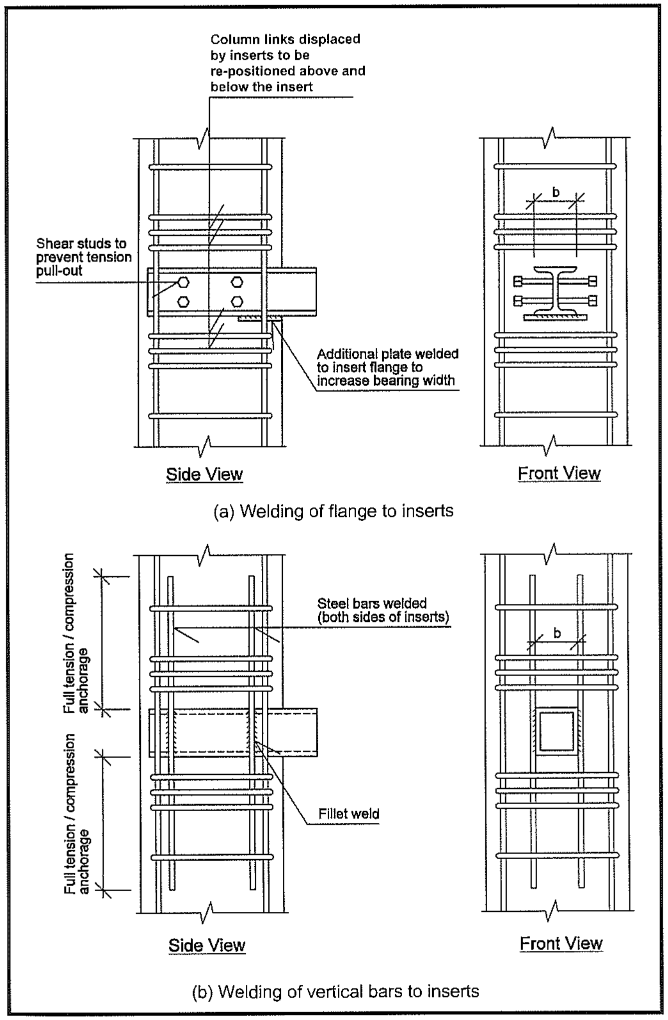


Figure 3.20 Increasing The Capacity Of Steel Inserts

The steel section is determined by

Flexural strength, $Z_p = M/p_y$ —— (3.41)

Shear strength, $V = 0.6dt\,p_y$ —— (3.42)

$M = V \times L_1$

where p_y = yield strength of steel section

d, t = depth and thickness of steel web respectively

Z_p = plastic section modulus of steel section

If a horizontal force N is acting, in addition to vertical loads, the force is resisted by bond on the perimeter of the section. The perimeter bond stress should be within the permissible values in Part 1, clause 3.12.8 of the Code. Treating steel section as mild steel (under tension), the bond stress is given as:

$$f_b = N/\Sigma p \leq 0.28\sqrt{f_{cu}} \qquad \text{—— (3.43)}$$

where Σp = perimeter of steel section

f_b = design perimeter bond stress per unit length of steel section

If the resultant bond stress exceeds the limiting values, headed studs or reinforcing bars can be welded to the section to increase the pull-out resistance.

For inserts installed near to the top of column, the bearing resistance of the column concrete to the upward force on the insert cannot be relied upon. This force is to be fully resisted by welding reinforcing bars to the steel inserts with full anchorage length into the column concrete below the insert. The design force for the bar is given by:

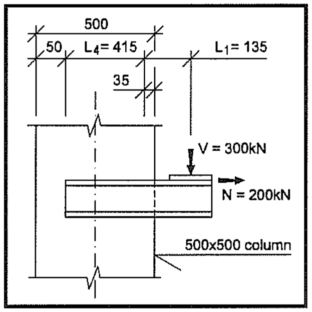
$$F_s = 0.8f_{cu}bL_3$$

From equation 3.36, $L_3 = 0.5\,(L_4 - L_2)$ and from equations 3.37 and 3.39, $L_2 = \alpha\, L_4$. Making these substitutions to the above expression:

$$F_s = 0.8f_{cu}b \times 0.5\,(L_4 - L_2)$$
$$= 0.4f_{cu}b\,(L_4 - \alpha\, L_4)$$
$$= 0.4f_{cu}b\,(1 - \alpha)\,L_4 \qquad \text{—— (3.44)}$$

Design Example 6: Steel Insert Cast In Columns

Design a column steel insert to support a vertical ultimate load of 300 kN. The column inserts are to be within a 500 x 500mm square column with f_{cu} = 35 N/mm². Also check the adequacy of the insert design if an ultimate horizontal force of 200 kN acts at the support. Concrete cover = 35 mm and yield strength of steel section p_y = 275N/mm².



Column Insert Embedment

1. **Effective bearing width of insert**

L_1 = 135 mm
L_4 = 415 mm
L_1/L_4 = 0.325

From equation 3.40 or from Figure 3.19, α = 0.279

Hence minimum effective bearing width of insert required from equation 3.39 is

$$b = V/0.8f_{cu}\alpha\, L_4$$
$$= 300 \times 10^3/(0.8 \times 35 \times 0.279 \times 415)$$
$$= 92.5 \text{ mm}$$

2. **Steel section for insert**

Moment $M = 300 \times 0.135$
$= 40.5$ kNm

Shear $V = 300$ kN

Horizontal Force, $N = 200$ kN

a. Plastic section modulus required

$$Z_p = M/p_y$$
$$= 40.5 \times 10^6/(275 \times 10^3)$$
$$= 147 \text{ cm}^3$$

O K

b. Minimum d x t required

$$d \times t = V/0.6p_y$$
$$= 300 \times 10^3/(0.6 \times 275)$$
$$= 1818 \text{ mm}^2$$

c. Minimum steel area required under tension

$$\text{Area} = N/p_y$$
$$= 200 \times 10^3/(275 \times 10^2)$$
$$= 7.3 \text{ cm}^2$$

Try 254 x 146 x 43 kg/m I-section

Z_p = 567.4 cm³ > 147 cm³ OK
A = 55 cm² > 7.3 cm² OK
d = 260 mm, t = 7.3 mm
d x t = 1898 mm² > 1818 mm² OK

d. Check moment and tension interaction :

Section capacity for moment M = 275 x 567.4 x 10^{-3}
= 156 kNm

Section capacity for tension N = 275 x 55 x 10^{-1}
= 1512.5 kN

Hence interaction
40.5/156 + 200/1513 = 0.26 + 0.13
= 0.39 < 1.0 OK

Column insert design is governed by shear. Section is adequate.

e. Check bearing capacity :

b = 147 mm > 95.2 mm OK

Max. bearing capacity of insert

$V = 0.8\, f_{cu}\, b\, \alpha\, L_4$
= 0.8 x 35 x 147 x 0.279 x 415 x 10^{-3}
= 477 kN > 300 kN OK

f. Check bond stresses

Total perimeter of steel section is about 1095 mm

Total contact area between steel section and concrete = 1095 x 415
= 454425 mm²

Average bond stress = 200 x 10³ / 454425
= 0.44 N/mm²

Permissible ultimate bond stress

$f_b = 0.28\sqrt{f_{cu}}$
$= 0.28\sqrt{35}$
= 1.66 N/mm² > 0.44 N/mm² OK

3.13.2 Steel inserts cast in beam

Steel section inserts used in beam connection may generally consist of:

a. wide flange section
b. steel plate with welded bearing flanges, and
c. exposed section on beam top;

as illustrated in Figure 3.21 below.

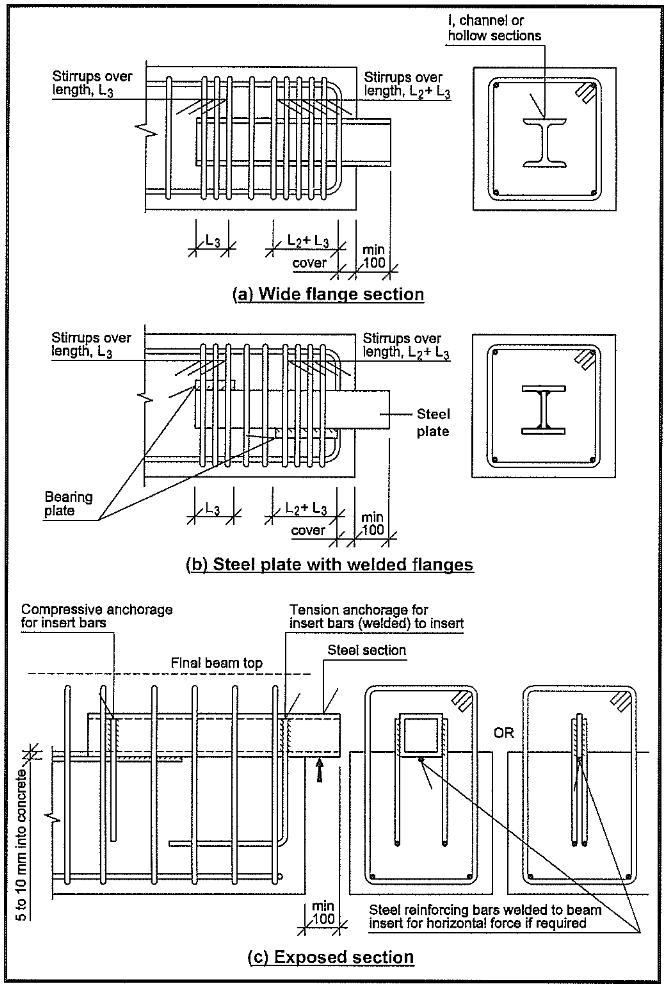


Figure 3.21 Steel Inserts Cast In Beams

Distribution of forces in the inserts is illustrated in Figure 3.22 (a) to (c) and the method of analysis may be as follows :

a. Wide flange section (Figure 3.22a)

The design and force distribution of wide flange section are similar to column inserts as shown in Figure 3.18. However, unlike column inserts, there is less depth (d_1 and d_2) of concrete above and below the insert to resist the bearing pressure. To be conservative, the bearing stress is limited to $0.4f_{cu}$ and the capacity of the insert is given by:

$$V = 0.4f_{cu}\, b\, \alpha\, L_4 \qquad \text{—— (3.45)}$$

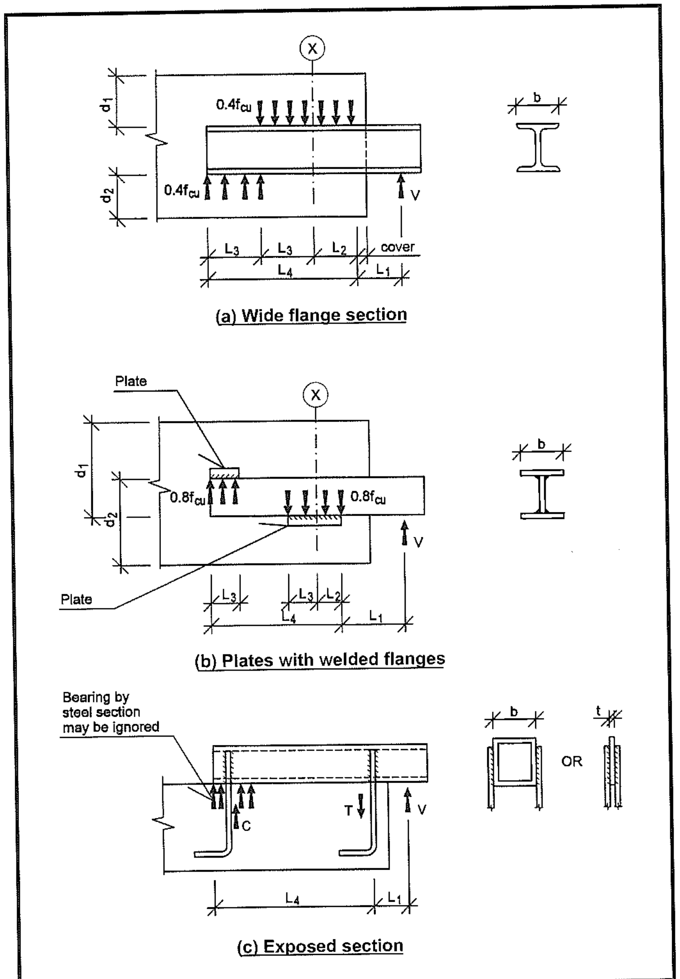


Figure 3.22 Distribution Of Forces In Beam Inserts

The designer should further ensure that:

i. the breadth and depth of the steel inserts should not exceed 1/3 of the respective breadth and depth of the concrete beam.

ii. reinforcing bars are provided at both the inner and outer bearing, in order to resist fully the bearing forces generated. The provision of steel area may be detemined as in plate with welded flanges insert below.

If horizontal forces are present, the bond stress around the section perimeter should not exceed $0.28\sqrt{f_{cu}}$ as in the column insert design.

b. Plate with welded flanges (Figure 3.22b)

The method of analysis of plate with welded flanges insert may be as below:

i. Resolve the forces vertically

$$V = 0.8f_{cu}\, b\, L_2 \qquad \text{—— (3.46)}$$

ii. Taking moment about X

$$V(L_1+L_2) = 0.8f_{cu}b[(L_4 - L_2 - L_3)L_3 + 0.5L_2^2] \qquad \text{—— (3.47)}$$

From the above equations, L_4 can be calculated from :

$$(L_4 - L_2 - L_3)L_3 = L_2(L_1 + 0.5L_2) \qquad \text{—— (3.48)}$$

A hanger system consisting of steel stirrups is to be provided at the inner and outer bearing plates. The area of the steel required may be :

Inner bearing plate: $A_{sv} = 0.8bf_{cu}L_3/0.87f_y$ —— (3.49)

Outer bearing plate: $A_{sv} = 0.8bf_{cu}(L_2 + L_3)/0.87f_y$ —— (3.50)

It is important to ensure that the main tension steel in the beam is fully anchored at the beam ends to prevent shear tension or shear bond failures at the beam support.

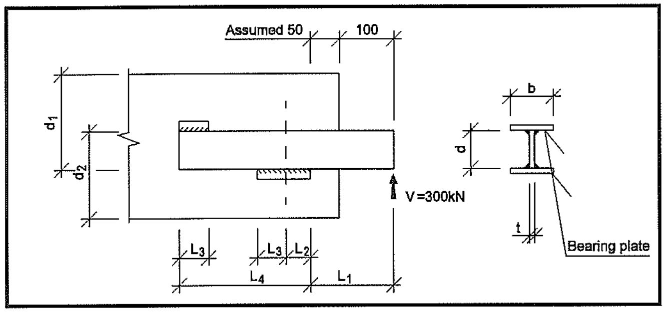
The dimensions of d_1 and d_2 should be proportional to the respective bearing forces i.e.

$$d_1/d_2 = (L_2 + L_3)/L_3 \qquad \text{—— (3.51)}$$

If horizontal tension forces are present, the bond stress must be checked as described earlier.

Design Example 7: Steel Plate Insert Cast In Beams

Design the steel plate with bearing flanges as an insert to support a vertical ultimate reaction from the beam of 300 kN acting at 100 mm from the beam end. The beam size is given as 300 x 600 mm deep and concrete strength f_{cu} = 30 N/mm². The ultimate yield strength of the steel plate is p_y = 275 N/mm². Also determine the appropriate position of the insert within the beam depth.



1. **Insert geometry**

Assuming bearing plate width b ≈ 1/3 beam width
= 100 mm

For equation 3.46

$V = 0.8 f_{cu} b L_2$

$L_2 = 300 \times 10^3 / (0.8 \times 30 \times 100)$
=125mm

Taking L_1 = 150 mm, L_2 = 125 mm and L_3 = 125 mm as trial. Then from equation 3.48 :

$$(L_4 - L_2 - L_3)L_3 = L_2(L_1 + 0.5L_2)$$
$$(L_4 - 125 - 125)125 = 125(150 + 0.5 \times 125)$$
$$L_4 = 462.5\text{mm}$$

Check $2 \times L_3 + L_2 \le L_4$ 2 x 125 + 125
= 375mm < 462.5mm   O.K.

Dimensions assumed in the insert are acceptable.

2. **Design of steel plate**

a. Shear

Required section for shear strength

$d \times t = V/0.6p_y$
$= 300 \times 10^3 / (0.6 \times 275)$
$= 1818\text{mm}^2$

$d = 1818 / t$

b. Bending

Ultimate bending moment, M $= V \times L_1$
$= 300 \times 0.15$
$= 45$ kNm

Plastic section modulus required $Z_p = M/p_y$
$= 45 \times 10^6/(275 \times 10^{-3})$
$= 163.6$ cm³

Using 2 numbers of t = 15 mm thick plates

Required minimum d $= \sqrt{[163.6 \times 10^3 \times 4/(2 \times 15)]}$
= 147.6, say d = 150 mm

3. **Bearing flanges design**

Steel plate inserts are spaced at 70 mm.
Contact bearing pressure = $0.8f_{cu}$
Required bearing flange thickness

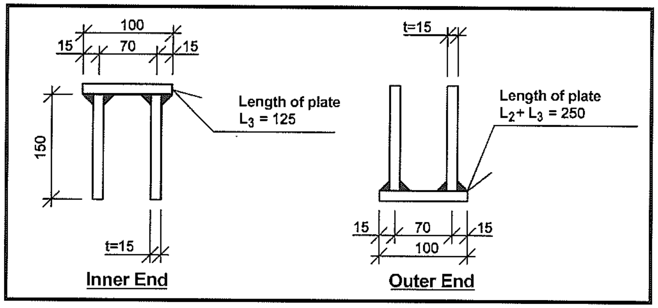
$(t^2/4) \times 275 = 0.8 \times 30 \times 70^2/8$
t = 14.6, say t = 15 mm thick

Check welding

Tension pull out force per mm run $= 0.8 \times 30 \times 50 \times 10^{-3}$
= 1.20 kN/mm

Use = 6 mm fillet weld
Weld strength $= 2 \times 6 \times 215 \times 10^{-3}/\sqrt{2}$
= 1.82 kN/mm > 1.20 kN/mm   OK

Steel plate insert = 2 numbers of 150 x 15 mm thick plates. The plates are as shown in the figure below.



4. **Reinforcing bars to resist bearing forces**

At inner plate : $A_s = 0.8f_{cu}\, b\, L_3/0.87f_y$
$= 0.8 \times 30 \times 100 \times 125/(0.87 \times 460)$
= 750 mm²
use 3T13 stirrups (6 legs) (A_s = 796 mm²)

At outer plate : $A_s = 0.8f_{cu} b(L_2 + L_3)/0.87f_y$
$= 0.8 \times 30 \times 100 \times (125 + 125)/(0.87 \times 460)$
= 1500mm²
use 6T13 stirrups (12 legs) (A_s = 1592 mm²)

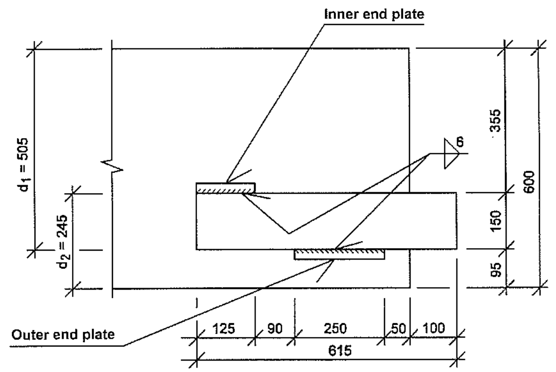
5. **Position of insert**

$$d_1/d_2 \approx (L_2 + L_3)/L_3$$
$$= (125 + 125)/125$$
$$\approx 2$$

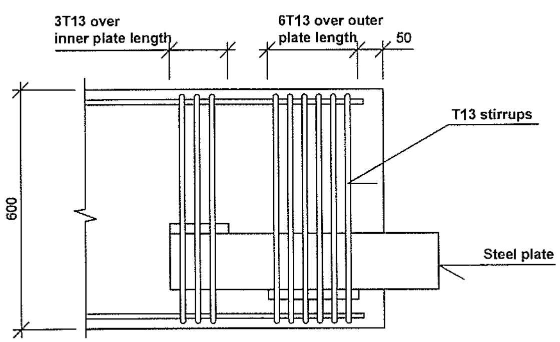
The final insert position is as shown in the detailing

$$d_1/d_2 = 505/245$$
$$= 2.06$$

6. **Detailing**



Positioning of insert



Links over bearing plates

3.13.3 Exposed sections

The exposed section inserts are commonly used in beam-to-beam connection when it is necessary to keep the structural beam shallow. As shown in Figure 3.21c, the exposed sections may consist of either wide flange sections or plates with reinforcing bars welded to the sides to provide the tension and compression reactions in a simple cantilever beam behaviour. The insert assembly will be embedded eventually in concrete.

The bearing pressure created at the far end of the insert is conservatively ignored, as it may be lost due to shrinkage, plastic cracking or surface grazing. There may also be partial or total loss of contact at the interface between the beam surface and insert as a result of fresh concrete settlement.

Referring to Figure 3.22c, the tension and compression reactions provided by the reinforcing bars may be obtained simply as :

Inner compression $C = V \times L_1/L_4$ —— (3.52)

Outer tension $T = V + C$ —— (3.53)

The reinforcing bars are calculated as :

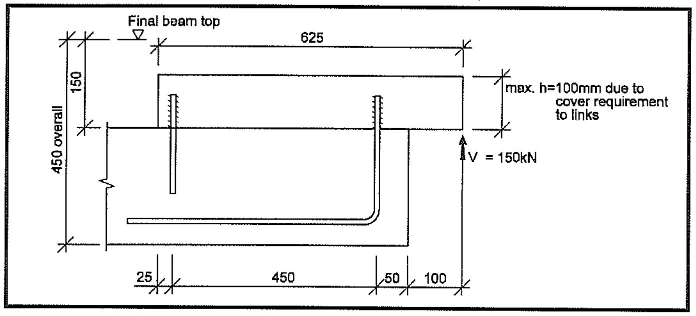
Tension and compression steel $A_s = T \text{ (or C)} / (0.87f_y)$

It is important to ensure that the welding is capable of developing the full strength of the reinforcing bars which are of full anchorage embedment length. The bearing stress within the bending radius of the bars has to be checked against local concrete bearing failure.

The choice of L_1 and L_4 is left entirely to the designer, bearing in mind that the larger the ratio of L_1/L_4, the heavier is the reinforcement needed in the steel insert.

Design Example 8: Exposed Steel Inserts Cast In Beam

Design an exposed steel insert at the ends of a precast beam subjected to an ultimate reaction of 150 kN. The semi-precast beam dimension is 300 x 300 and concrete strength is f_{cu} = 40 N/mm². The overall beam depth is restricted to 450 mm. The ultimate strength of the steel section p_y=275N/mm² and the characteristic strength of reinforcing bars f_y = 460N/mm²



1. **Geometry of inserts**

Assuming a ratio of cantilever to internal span of 1:3 will result in a tension of 4/3 x V and a compression of 1/3 x V in the reinforcing bars. The geometry of the insert is shown in the above figure.

2. **Tension and compression forces**

T = 4/3 x V
= 200 kN

C = T - V
= 200 - 150
= 50 kN

3. **Reinforcement design**

a. Bar in tension

Tension T = 200 kN

A_s = T / 0.87f_y (f_y = 460 N/mm²)
= 500 mm²

Say 4 numbers of T13 with 2 bars welded to each side of the insert. (A_s=530mm²)

Check tension anchorage

Tensile force per bar F_s = 0.87f_yA_s
= 0.87 x 460 x 132 x 10^{-3}
= 52.8 kN

Tensile anchorage length required $l_p = F_s / (\pi\phi f_b)$

where $f_b = \beta\sqrt{f_{cu}}$
= 0.5√40
= 3.16 N/mm²

l_p = 52.8 x 10³ / (π x 13 x 3.16)
= 409mm, say 450mm

The anchorage length of straight bars will exceed the overall beam depth of 300mm. Therefore, a bend will be provided in the tension bar.

Check bearing stress within the bend

$$\text{Bearing stress} = \frac{F_s}{r\phi} \leq \frac{2f_{cu}}{1 + 2(\phi/a_b)}$$

$$\text{Minimum} \quad r = \frac{F_s}{\phi} \times \frac{1 + 2(\phi/a_b)}{2f_{cu}}$$

Assuming plate thickness of inserts = 35 mm

a_b = 35 + ϕ
= 48mm

Hence minimum bending radius

r = [52.8 x 10³/13] x [1 + 2(13/48)] / (2 x 40)
= 78 mm

Use r = 6ϕ (ϕ=13)
= 78 mm

b. Bar in Compression

Compression force in bar, C = 50 kN

A_s = C/0.87f_y
= 50 x 10³/(0.87 x 460)
= 125 mm²

Use 2 numbers of T13, one on each side of insert. (A_s=265mm²)

Check compression anchorage

Force per bar F_s = 50/2
= 25 kN

Compression anchorage required f_b = 0.63$\sqrt{f_{cu}}$
= 3.98 N/mm²

l_b = 25 x 10³/(π x 13 x 3.98)
= 154 mm

Provide l_b = 200 mm

4. **Steel insert design**

Ultimate bending moment M = 0.15 x V
= 0.15 x 150
= 22.5 kNm

Plastic section modulus required Z_p = 22.5 x 10⁶/(275 x 10³)
= 81.8 cm³

Restrict height of plate to 100 mm due to cover requirements to shear links.

Hence t = 81.8 x 10³ x 4/100²
= 33 mm, say t = 35 mm

Note: When t > 16mm thick, p_y should be 265N/mm². However, the adopted thickness of t=35mm remains valid in this case.

Check shear

Plate thickness under shear for d = 100

d x t = $V/0.6p_y$

t = $150 \times 10^3/(0.6 \times 275 \times 100)$
= 9 mm $\leq$ 35mm OK

5. Welding design of steel bars

Maximum tension force = 52.8 kN per bar

Weld length available = 100mm

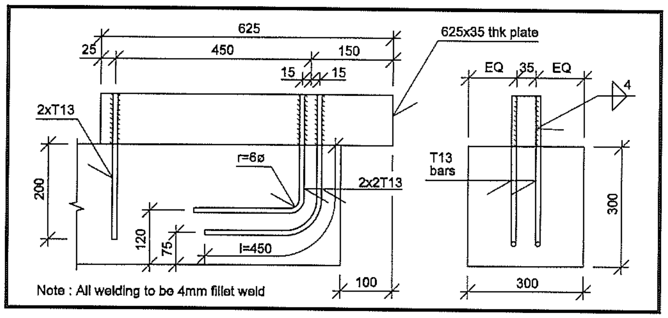
$\therefore$ Weld strength required = 52.8/100
= 0.528 kN/mm

Try 4 mm fillet weld on both sides of the bar.

Welding strength provided = $0.707 \times 4 \times 215 \times 10^{-3} \times 2$
= 1.22 kN/mm > 0.528 kN/mm OK

Provide continuous welding of 4 mm fillet weld on both sides of the bar.

6. Detailing



3.14 Force Transfer By Welding

Welding for the purpose of force transfer between reinforcing bars or between bars and other steel sections is permitted under Part 1, clause 7.6, of the Code. Provided the steels are weldable and suitable safeguards and techniques are employed, welding is a practical method to achieve the force transfer between connections in precast construction.

There are two types of welds, namely butt and fillet weld. Butt welds may be considered as strong as the parent steel as long as full penetration for the weld is achieved. The size of butt weld is specified by its throat thickness and is the smaller value of the two materials being joined. Figure 3.23 shows some typical butt welds between bars and other steel sections. Fillet welds are more commonly used as they do not require special surface preparation of the reinforcing bars or plates and are therefore cheaper than butt welds. Figure 3.24 shows some typical fillet welds between bars and other steel sections. The strength of fillet weld is given as:

P_w = (throat thickness) x (unit length) x (design shear stress)
= $a_w \times p_w$
= $0.7 \times$ leg length $\times p_w$
= $0.7 \times s \times p_w$ —— (3.54)

Table 3.3 shows the design strength for fillet welds.

<table>
<tr><th rowspan="3">Leg length s (mm)</th><th colspan="5">Design strength per unit length in kN/mm</th></tr>
<tr><th>Grade 43</th><th colspan="2">Grade 50</th><th colspan="2">Grade 55</th></tr>
<tr><th>E43, E51</th><th>E43</th><th>E51</th><th>E51</th><th>E51*</th></tr>
<tr><td></td><td>p_w = 215 N /mm^2</td><td colspan="2">p_w = 255 N/mm^2</td><td colspan="2">p_w = 275 N/mm^2</td></tr>
<tr><td>4</td><td>0.602</td><td colspan="2">0.714</td><td colspan="2">0.770</td></tr>
<tr><td>5</td><td>0.753</td><td colspan="2">0.893</td><td colspan="2">0.963</td></tr>
<tr><td>6</td><td>0.903</td><td colspan="2">1.071</td><td colspan="2">1.155</td></tr>
<tr><td>8</td><td>1.204</td><td colspan="2">1.428</td><td colspan="2">1.540</td></tr>
<tr><td>10</td><td>1.505</td><td colspan="2">1.785</td><td colspan="2">1.925</td></tr>
<tr><td>12</td><td>1.806</td><td colspan="2">2.142</td><td colspan="2">2.310</td></tr>
<tr><td>16</td><td>2.257</td><td colspan="2">2.667</td><td colspan="2">2.887</td></tr>
<tr><td>18</td><td>2.709</td><td colspan="2">3.213</td><td colspan="2">3.465</td></tr>
<tr><td>20</td><td>3.010</td><td colspan="2">3.750</td><td colspan="2">3.850</td></tr>
<tr><td>22</td><td>3.311</td><td colspan="2">3.927</td><td colspan="2">4.235</td></tr>
<tr><td>25</td><td>3.763</td><td colspan="2">4.463</td><td colspan="2">4.813</td></tr>
</table>

Note (1) E denotes electrode complying with BS639.
(2) Grade 43, 50, 55 complying with BS 4360.
(3) * only applies to electrodes having a minimum tensile strength of 550 N/mm^2 and a minimum yield strength of 450 N/mm^2.

Table: 3.3 Design Strength For Fillet Weld

For welding between reinforcing bars or reinforcing bars with steel sections, the reader may refer to guidelines on welding design from the latest edition of ANSI/AWS D1.4 - Structural Welding Code, Reinforcing Steel.

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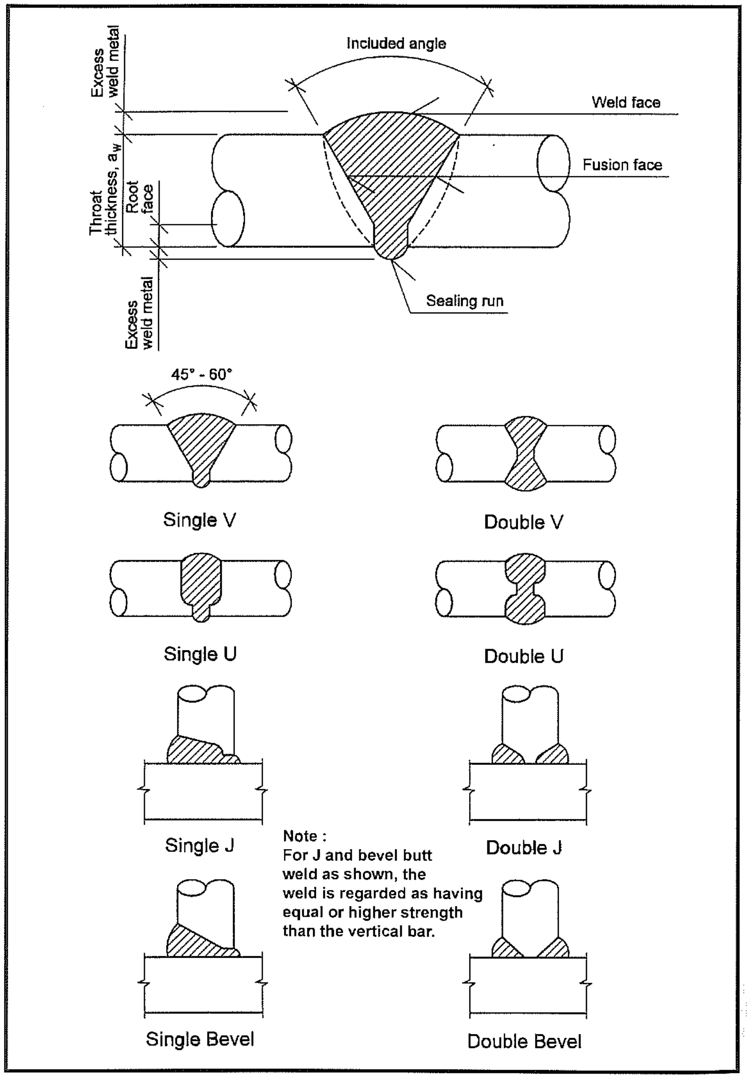


Figure 3.23 Typical Butt Welding

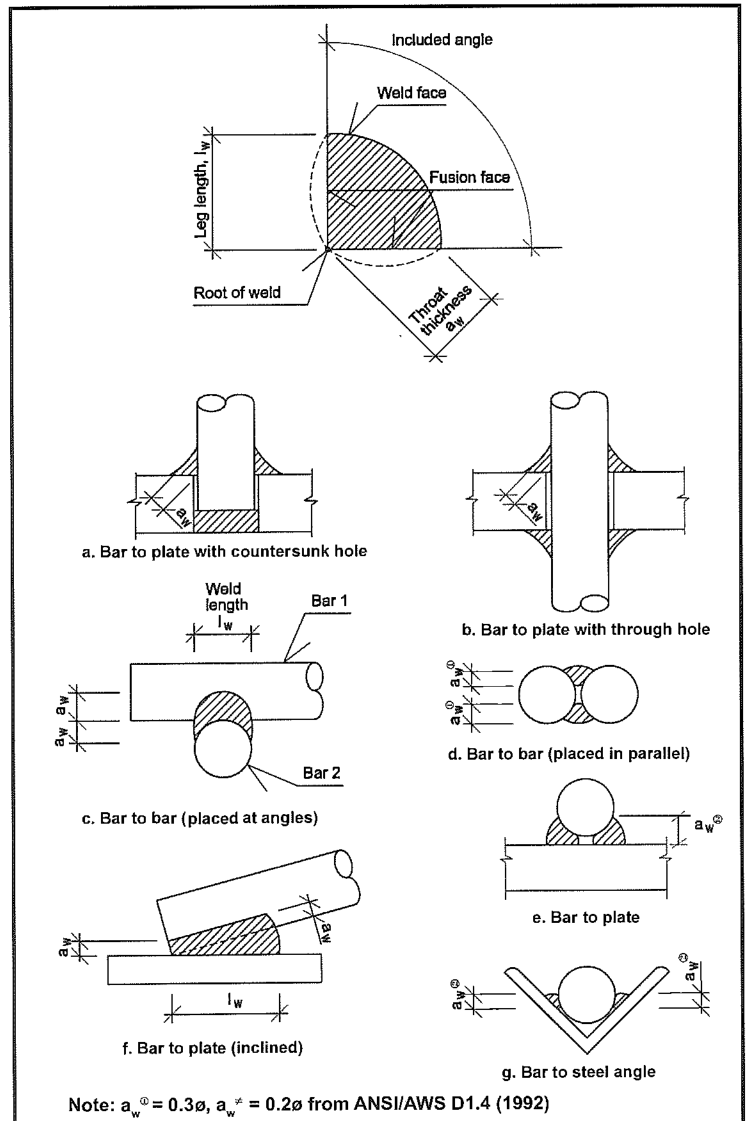


Figure 3.24 Typical Fillet Welding

3.15 Shear Key Connection

Shear key connections may be un-reinforced . When the elements are prevented from moving apart under shear loading, usually by means of reinforcing ties at top and bottom of the joint as shown in Figure 3.35, tests have shown that there is similar deformation behaviour as those of reinforced keys. However, there is also a definite increase in joint strength if the shear keys are reinforced.

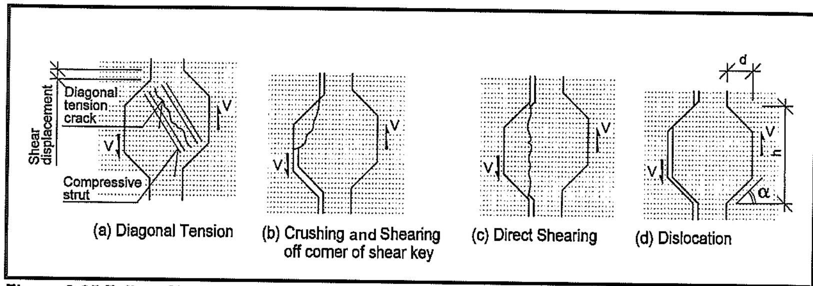


Figure 3.25 Failure Modes Of Joint Concrete In Shear Key

Figure 3.25 illustrates the failure modes of shear key joints which are :

a. diagonal cracks across the joint,
b. crushing and shearing off corner of the shear key,
c. direct shearing cracks parallel to the joint, and
d. slippage or dislocation at the contact face.

The different failure modes of shear keys are dependent on the compressive strength of the joint concrete, surface adhesion and bonding strength at the contact surfaces and the profile of the keys.

In general, the mode of failures in (a) and (c) takes place in normal shear keys having $6 \leq h/d \leq 8$ and $\alpha = 30^{\circ}$, where h and d being the respective height and depth of the shear key and α the slope of contact faces as shown in Figure 3.26. The minimum dimension of d should be 10mm. Test results have shown that maximum joint capacity is obtained for shear key having the above dimensions. (reference 10)

Failure mode (b) generally occurs in long shear key where $h/d > 8$ and the key capacity is not fully mobilised. In (d), failure occurs when $\alpha > 30^{\circ}$.

Depending on the number of keys, the forces acting on a single key joint due to a distributed vertical shear force V can be obtained as shown in Figure 3.26 :

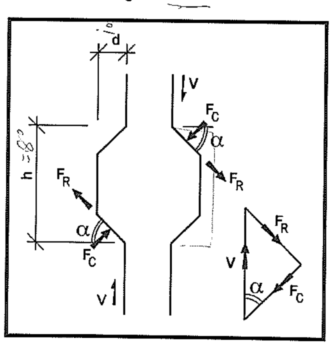


Figure 3.26 Forces In Shear Key Joint

Compression force perpendicular to the slope face of the key:

$$F_c = V \cos \alpha \qquad \text{— (3.55)}$$

Sliding force parallel to the key slope:

$$F_R = V \sin \alpha \qquad \text{— (3.56)}$$

The resistance to the sliding force F_R is given as :

$$\begin{aligned} F_R &= 0.6\mu F_c \\ &= 0.6\,\mu \times V \cos \alpha \end{aligned} \qquad \text{— (3.57)}$$

where μ is the shear friction coefficient.

When F_R exceeds the frictional resistance by shear friction, shear friction reinforcement is to be provided for the total F_R. Reinforcement is calculated as in accordance with the Code:

$$A_s = V / (0.6\,\mu \times 0.87 f_y)$$

Value of μ may be obtained from Table 3.1. In general it may be taken as 1.4 for shear key joint and A_s is then simplified as:

$$A_s = 1.2V / (0.87 f_y) \qquad \text{— (3.58)}$$

3.16 Column Base Connection

3.16.1 General

Column to foundation connections can be achieved through one of the following means :

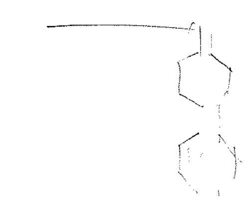
1. socket connection
2. base plate connection
3. grouted pipe sleeves

Column to column connections are, in principle, similar to column to foundation connections and methods (2) and (3) above are most commonly adopted in practice. The connection by pipe sleeves is explained in Section 3.6.

3.16.2 Socket connection

In socket connection, the precast columns are fixed rigidly to the foundation and loads are transmitted by skin friction in the socket and by end bearing. It is common to roughen the surfaces or form shear keys at the sides of the socket or columns in order to enhance the transfer of axial load by shear wedging. In the case of large overturning moment where the column reinforcement is in tension, the column bars extending into the socket must be fully anchored by bond or other means. The bars may be hooked at their ends for the purpose of reducing the depth of the socket. Additional links are also required in the precast columns to resist the bursting pressure generated by end bearing forces.

Figure 3.27 shows two variations of the socket connection in practice. The socket may also be precast when soil condition allows pad footing design.



In-situ socket is cast using a box shutter. The clearance gap between the socket wall and column should be at least 50 to 75 mm all round for ease of grouting or concreting as well as to allow for construction tolerances. The socket walls may be used to support precast or in-situ ground beams or slabs. In general, the structural floor may be 200 to 300 mm or greater above the socket. A minimum of 40 mm root depth should be allowed at the bottom of the socket.

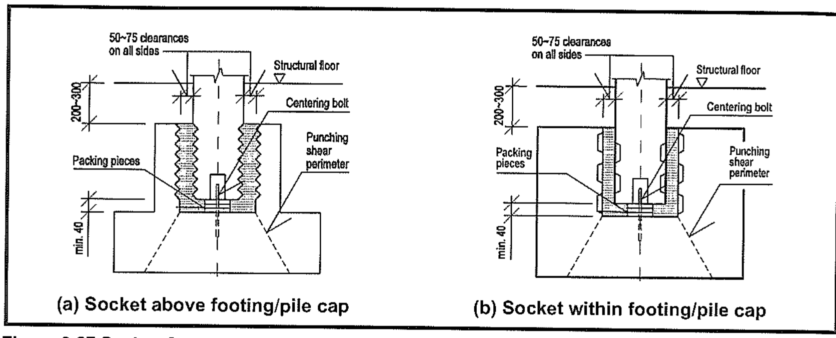


Figure 3.27 Socket Connection

During erection, the precast column rests on packing pieces and is wedged into position using timber pieces as shown in Figure 3.28. Alternatively, column props at orthogonal directions may be used.

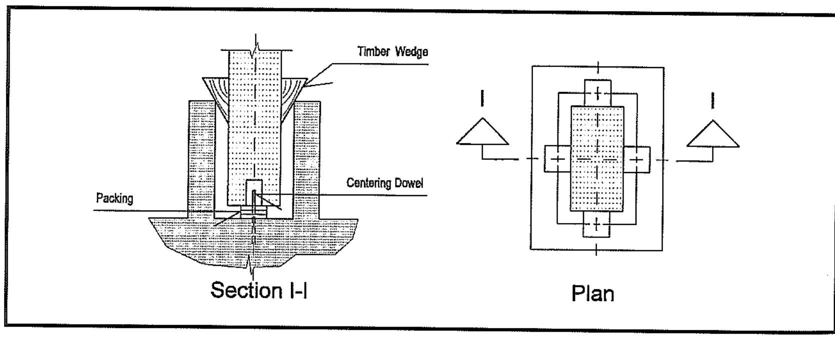


Figure 3.28 Timber Wedging At Column Installation

To assist in centering the column, a dowel may be provided at the base of the socket.

The design of the socket connection may adopt the following steps: (reference 11)

1. Support reactions

Referring to Figure 3.29, a rotation of the column in the socket under moment shifts the support reaction at the column base from the centre line towards the edge. The resultant reaction R at the base may be assumed to act at a distance a/6 from the column centre line.

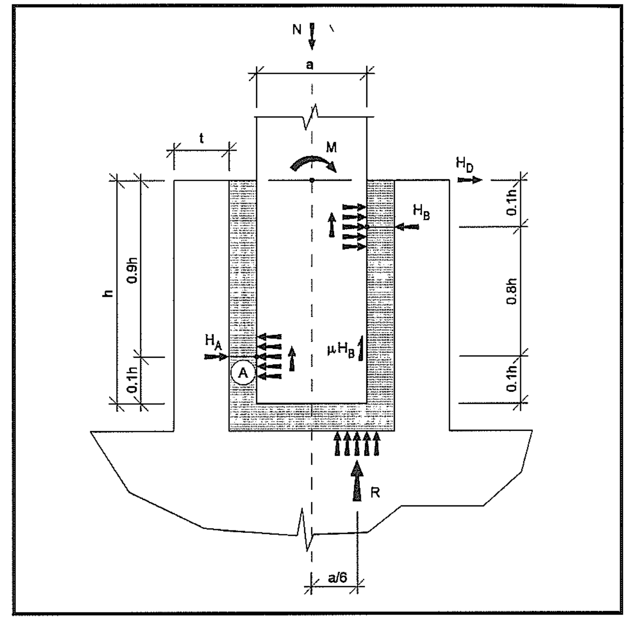


Figure 3.29 Distribution Of Forces In Column Socket

From Figure 3.29 the following forces may be derived :

a. **Horizontal force H_B**

Moment about point A

$M + N(a/2) + H_D \times 0.9h = H_B \times 0.8h + \mu H_B \times a + R(a/2 + a/6)$

Support reaction at column base is

$R = N - \mu H_B$

Substituting R into the above equation and after rearranging

$$H_B = (M - 0.17aN + 0.9hH_D) / (0.8h + 0.33a\mu) \qquad \text{—— (3.59)}$$

If the height of the socket is taken as h = 1.5a and ignoring the top 0.1h (assumed cover zone) of the socket height, the effective column embedment length is given as

$h = 0.9 \times 1.5a$

i.e. $a = 0.74h$

For a smooth column face, the coefficient of friction $\mu = 0.3$ is used and substituting $a = 0.74h$ and $\mu = 0.3$ into equation 3.59, the following simplified expression for horizontal force H_B is obtained:

$$H_B = 1.14\,M / h - 0.15N + 1.03H_D \qquad \text{—— (3.60)}$$

b. **Horizontal force H_A**

By equilibrium : $H_A = H_B - H_D$ —— (3.61)

c. **Vertical reaction R**

Resolve vertically : $R = N - \mu H_B$ —— (3.62)

Hence knowing H_D, N and M, H_A, H_B and R can be calculated.

d. **Reinforcement in socket**

The ring reinforcement at H_B level is calculated from

$$A_{SB} = H_B / (0.87 f_y) \quad \text{—— (3.63)}$$

Ring reinforcement at H_A level is calculated from

$$A_{SA} = (H_A - \mu R) / (0.87 f_y) \quad \text{—— (3.64)}$$

If μR is greater than H_A, no ring reinforcement is needed.

Vertical reinforcement A_{sv} in the socket is calculated at the lower level using the moment

$$M_v = M + H_D \times h \quad \text{—— (3.65)}$$

Schematic reinforcement in the socket is shown in Figure 3.30.

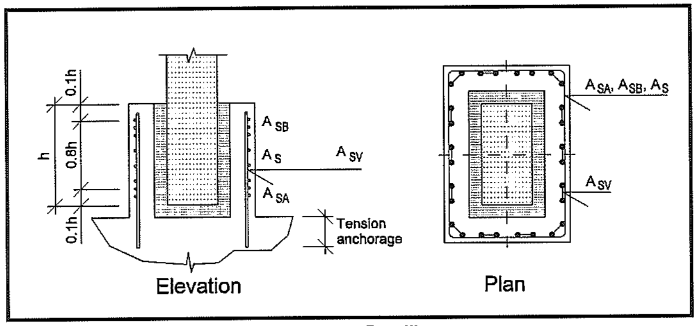


Figure 3.30 Typical Socket Reinforcement Detailing

2. **Column reinforcement in socket**

To reduce the depth of the socket, the main column reinforcement in the socket may be lapped with looped bars. Minimum reinforcement at the lower end of the column should be

$$A_s = H_A / (0.87 f_y)$$

Typical column reinforcement in the socket is shown in Figure 3.31.

Within the socket depth, the column section should be designed for shear using H_A. Effect of increases in concrete shear capacity due to axial compression may be ignored. There should be additional stirrups in the embedded column as it will improve the main steel bar anchorage. The stirrups will also act as reinforcement for bursting forces at the base due to axial vertical load. The amount of stirrups to be provided (assumed to be uniformly distributed) for bursting effect may be calculated from

$\zeta = H_{bst}/N$
$= 0.11$ (Table 4.7, Part 1, CP 65)

$$H_{bst} = 0.11N \quad \text{—— (3.66)}$$

where H_{bst} is the bursting force and N is the axial vertical force.

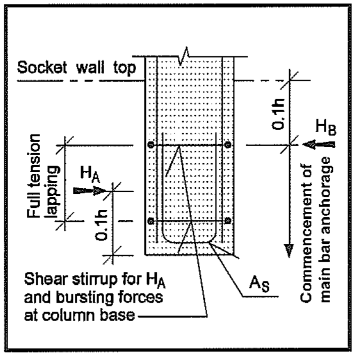


Figure 3.31 Typical Reinforcement Of Column In Socket

3. **Socket dimensions**

The following considerations have been commonly used in the dimensioning and design of column sockets :

a. strength of concrete in socket ≥ C35

b. strength of concrete/grout for the socket gap infill ≥ C35

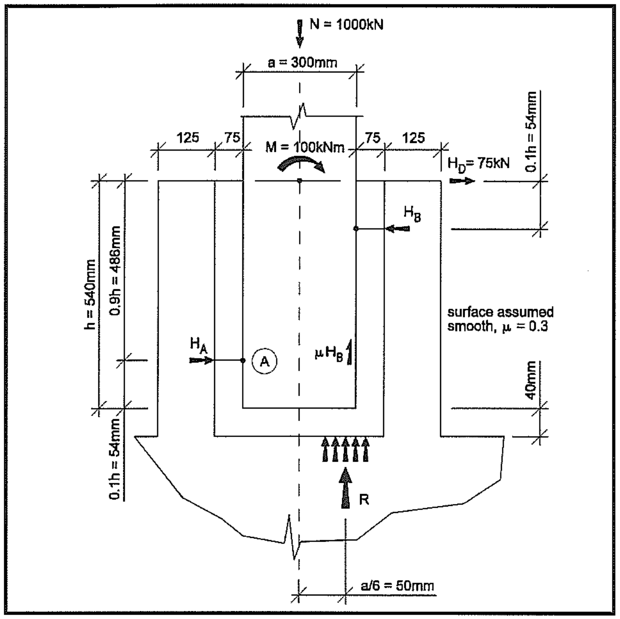
c. height of socket should be a < h ≤ 1.5a where a is the greater dimension of the column

d. the minimum wall thickness of the socket is t = 0.18b + 70 (mm) where b is the smaller column dimension

e. minimum static friction coefficient
$\mu = 0.3$ (smooth surface)
$\mu = 0.7$ (roughened surface)

Design Example 9: Socket Connection For Column

Design a column socket connection at the foundation which is required to support a 300 x 300 mm, C50 precast column subjected to an ultimate axial load of N = 1000 kN, a moment of M = 100 kNm and a horizontal force of H_D = 75 kN. Determine the minimum strength of the in-situ fill. Cover to column bars = 35 mm and to foundation bars = 50 mm. The column is reinforced with 4T16 and with links R10@150 c/c.



Forces In The Column Socket

Column size = 300 x 300mm
N = 1000 kN
M = 100 kNm
H_D = 75 kN

1. **Socket dimensions**

a. Socket depth

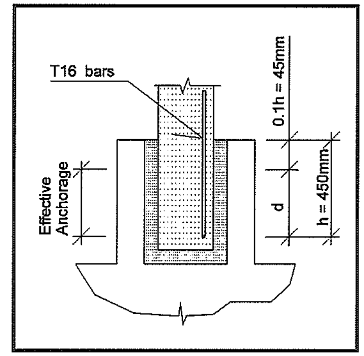
h = 1.5a
= 1.5 x 300
= 450 mm

Allow 40 mm at root depth, hence overall socket depth
= 450 + 50
= 490 mm, say 500 mm

Ultimate bond stress in tension anchorage

$f_b = \beta\sqrt{f_{cu}}$
$= 0.5\sqrt{50}$
$= 3.54$ N/mm^2

For T16, tension anchorage $l_p = (0.87 f_y A_s) / (f_b \times \pi\phi)$
$= (0.87 \times 460 \times 201) / (3.54 \times \pi \times 16)$
= 452 mm



Anchorage Of Column Bars

Depth of socket

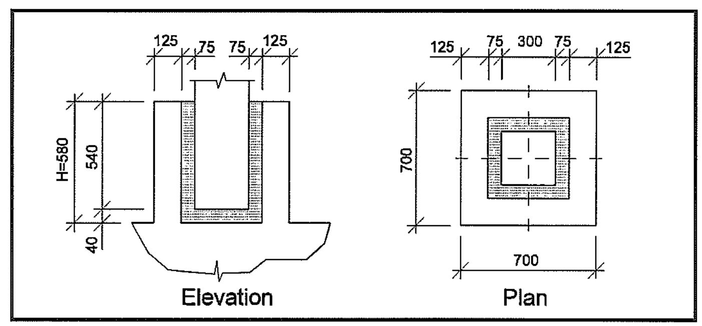
h = 452 + 0.1h + cover
= 452 + 0.1 x 452 + 35
= 532mm, say 540mm

Allowing 40mm root depth at the base, the total depth of the socket is 540+40=580mm

b. Socket wall thickness

t = 0.18b + 70
= 0.18 x 300 + 70
= 124mm, say 125 mm

The dimensions of the socket are shown below.



Column Socket Dimensions

2. Forces in socket

a. Horizontal force H_B

$$H_B = 1.14\,M / h - 0.15N + 1.03H_D$$
$$= 1.14 \times (100/0.54) - 0.15 \times 1000 + 1.03 \times 75$$
$$= 211.1 - 150 + 77.3$$
$$= 138.3 \text{ kN}$$

b. Horizontal force H_A

$$H_A = H_B - H_D$$
$$= 138.3 - 75$$
$$= 63.3 \text{ kN}$$

c. Vertical reaction R

$$R = N - \mu H_B$$
$$= 1000 - 0.3 \times 138.3$$
$$= 958.5 \text{ kN}$$

Width of R acting at the base = 2/3a = 200 mm

3. Reinforcement design in socket

a. Ring reinforcement at H_B level

$$A_{SB} = H_B/0.87f_y$$
$$= 138.3 \times 10^3/(0.87 \times 460)$$
$$= 346 \text{ mm}^2\text{, use 3T10 (6 legs) } (A_s = 471 \text{ mm}^2)$$

b. Ring reinforcement at H_A level

$$A_{SA} = (H_A - \mu R)/0.87f_y$$
$$= (63.3 - 0.3 \times 958.8)/(0.87 \times 460)$$
$$= \text{-ve value}$$

No ring reinforcement is theoretically required but nevertheless provide 3T10 (6 legs).

c. Vertical reinforcement in socket wall

Design moment M_v $= M + H_D \times h$
$= 100 + 75 \times 0.54$
$= 140.5$ kNm

Lever arm between the steels in walls $z = 700 - 70 - 70$
$= 560$ mm

$$A_{sv} = M/(0.87 \times f_y \times z)$$
$$= 140.5 \times 10^6/(0.87 \times 460 \times 560)$$
$$= 627 \text{ mm}^2$$

Use 8T10 (628 mm²) at each wall. Use U-shaped bars to ensure full anchorage.

d. Column reinforcement in socket

i. Bursting Reinforcement

$$H_{bst} = 0.11N$$
$$= 0.11 \times 1000$$
$$= 110 \text{ kN}$$

$$A_{bst} = 110 \times 10^3/(0.87 \times 250)$$
$$= 505 \text{ mm}^2 \text{ in column}$$

Links R10-150 in column (A_s = 523 mm²/m) are adequate.

ii. End reinforcement

$$A_s = H_A/0.87f_y$$
$$= 63.3 \times 10^3/(0.87 \times 460)$$
$$= 158 \text{ mm}^2 < \text{4T16, use 4T16}$$

iii. Check shear at column base

Max. shear force $= H_A$
$= 63.3$ kN

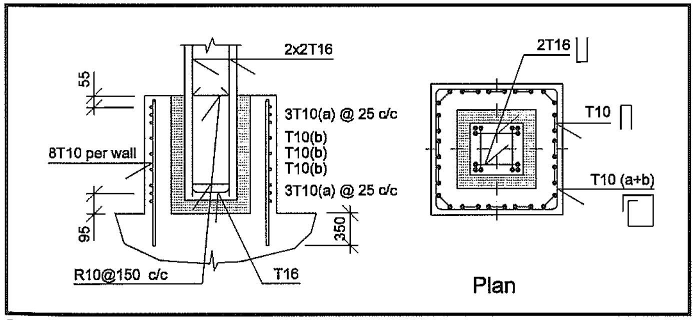
$v = 0.94$ N/mm²
r_s = 4T16 = 1.14%
$v_c = 0.95$ N/mm² > 0.94 N/mm² OK

4. Strength of infills

$R = 958.5$ kN
Contact area $= 300 \times 200$mm²
$f_c = 958.5 \times 10^3 / (300 \times 200)$
$= 16.0$ N/mm²

From clause 5.2.3.4, the concrete grade of infill $f_{cu} = 16.0/0.6$
$= 26.7$ N/mm², say min. C35

5. Detailing



Reinforcement Detailing

3.16.3 Base plate connection

Column base plates can be used when a moment connection is required. The base plates must be designed for both the erection loads and loads which occur in service. Unlike socket or grouted pipe sleeves connections which require time to develop the necessary strength, columns using base plate connections can achieve immediate stability which will greatly facilitate erection of other precast components. The choice in using base plates rather than sockets or grouted pipes is based more on work productivity than on structural requirements.

Two commonly used base plate details are shown in Figure 3.32. The column base plates may be larger than, flushed or smaller than, the column dimensions. In cases where base plates are larger than the columns, the plates must be protected against corrosion by concrete haunching, electro or hot-dip galvanising.

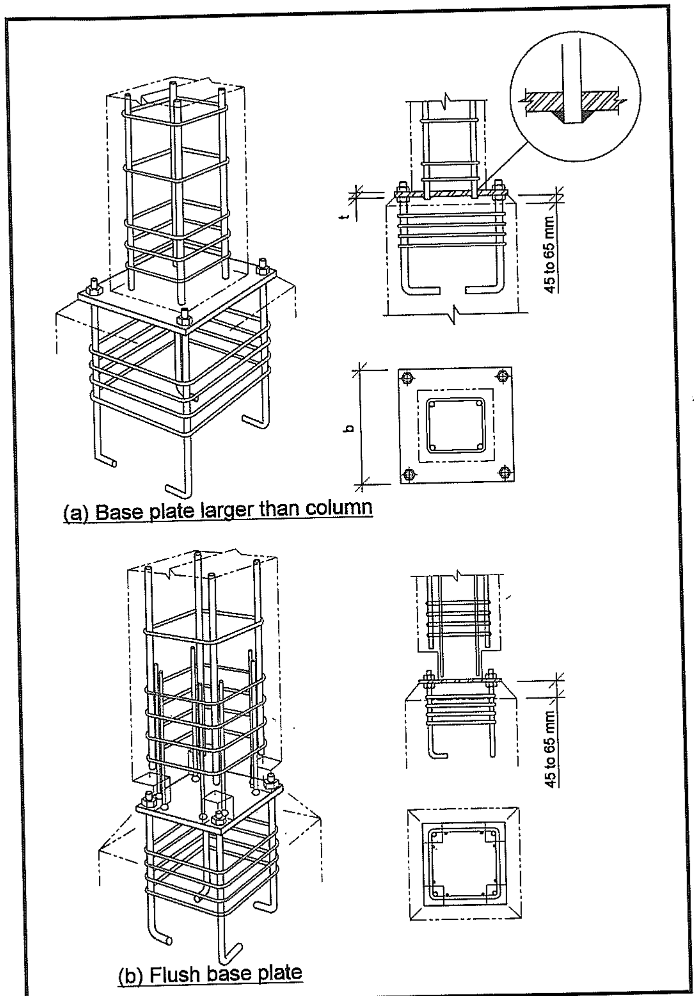


Figure 3.32 Column Base Plate Connections (references 5 and 9)

The base plates are fixed to the column by steel reinforcement welded to the plates. Additional links are usually provided to resist bursting pressure.

Holding down bolts are often specified to be either grade 4.6 or 8.8. The length of bolts should be designed in accordance with the acting forces. The end of the bolts may be hooked, L-shaped or fitted with a steel plate typically 100 x 100 x 9 mm thick to increase the pull-out capacity. Confined reinforcement in the form of links around the bolts is recommended and should not be less than 4 numbers of R8 links at 50 mm near to the top of the bolts.

The thickness of the base plate depends on the overhang projection from the column face. It is subjected to biaxial bending from the resultant compressive forces acting on the surface. For this reason, the overhang projection is normally limited to 100 to 125 mm; a minimum practical limit for detailing and site erection purposes. Holes in the plate are normally oversized to offset construction setting out and production tolerances. In general, there should be an all round gap of between 10 to 15 mm between the edges of the bolt and plate hole.

Figure 3.33 shows the distribution of forces in a column base plate connection.

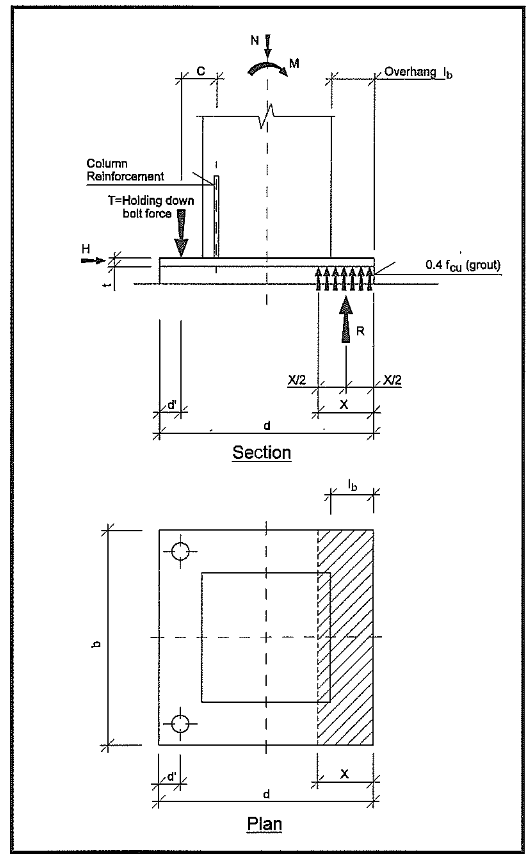


Figure 3.33 Force Distribution In Column Base Plate

The following steps may be used to calculate the base plate thickness:

a. Resolve forces vertically

$$N = 0.4 f_{cu} b \chi - T \qquad (3.67)$$

where χ = compressive stress block depth

b. Moment about centreline of compressive stress block

$$T = [M - N(d/2 - \chi/2)] / (d - d' - \chi/2) \qquad (3.68)$$

Substituting equation 3.67 into 3.68 and putting M=Nxe, the following simplified expression is obtained.

$$(\chi / d)^2 - 2(1 - d'/d)(\chi/d) + 5\,N(e + 0.5d - d') / (f_{cu} b d^2) = 0 \qquad (3.69)$$

from which χ/d and hence T can be calculated.

If $\chi/d > N/0.4 f_{cu} bd$, then T is positive and the area of holding down bolts can be calculated from :

$$A_b = T / (\Sigma n \times f_{yb}) \qquad (3.70)$$

where Σn = number of bolts
f_{yb} = ultimate tensile strength of bolts
= 195 N/mm² for grade 4.6 bolts
= 450N/mm² for grade 8.8 bolts

The thickness t of the column base plate may be obtained using plastic analysis and is the greater of :

i. Based on compression side

$$t = \sqrt{0.8 f_{cu} l_b^2 / p_y} \quad \text{in mm} \qquad (3.71)$$

ii. Based on tension side

$$t = \sqrt{4Tc/(bp_y)} \quad \text{in mm} \qquad (3.72)$$

where p_y = yield strength of steel plate (refer to table 6, Part 1, BS 5950)
= not greater than 275 N/mm² for grade 43 and 355 N/mm² for grade 50 steel.
l_b = plate overhang beyond column face
c = distance between centres of bolt and column bar

If $\chi/d < N/0.4 f_c bd$, then T is negative and the above equations are not valid. The analysis simplifies to the following :

$$N = f_c b \chi \qquad (3.73)$$

$$\chi/d = 1 - 2e/d \qquad (3.74)$$

where f_c is the uniformly distributed concrete compressive stresses under the combined action of N and M. The required plate thickness is given by

$$t = \sqrt{2 f_c l_b^2 / p_y} \qquad (3.75)$$

Design Example 10: Column Base Plate Connection

Design a column base plate connection for a 300 x 300mm precast column subjected to an ultimate axial load N = 1000 kN and a moment M = 100 kNm. Use grade 43 base plate and grout strength f_{cu} = 40 N/mm².

Try 500 x 500 plate with 100 mm overhang all round from the column face.

From equation 3.67

$$(\chi/d)^2 - 2(1-d'/d)(\chi/d) + 5\,N(e + 0.5d - d') / (f_{cu} b d^2) = 0$$

e = M/N = 100 mm
b = d = 500 mm
d' = 50 mm
f_{cu} = 40 N/mm²

$$(\chi/d)^2 - 2(1 - 50/500)(\chi/d) + 5 \times 1000\,(100 + 0.5 \times 500 - 50) \times 10^3 / (40 \times 500 \times 500^2) = 0$$

$$(\chi/d)^2 - 1.8(\chi/d) + 0.3 = 0$$

$$(\chi/d) = 0.186$$

$$N / (0.4 f_{cu} bd) = 1000 \times 10^3 / (0.4 \times 40 \times 500 \times 500) = 0.25 > 0.186$$

Hence there is no tension in the holding down bolts.
Revise the design using equation 3.74.

χ/d = 1 - 2e/d
= 1 - 2 x 100/500
= 0.6

χ = 300 mm

From equation 3.73, $N = f_c b \chi$
f_c = 1000 x 10³/(500 x 300)
= 6.67 N/mm²

From equation 3.75, the plate thickness

$$t = \sqrt{(2 f_c l_b^2 / p_y)}$$

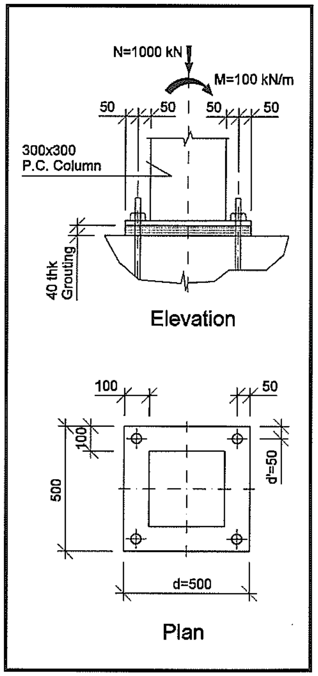
Using grade 43 steel plate, p_y = 275 N/mm² and l_b = 100 mm

$$t = \sqrt{(2 f_c l_b^2 / p_y)} = \sqrt{(2 \times 6.67 \times 100^2/275)} = 22 \text{ mm}$$

Use t = 25 mm

Plate size = 500 x 500 x 25 mm thick

Note: When steel plate thickness t>16mm, the required plate thickness should be recalculated using p_y = 265 N/mm². The adopted plate t = 25mm is, however, still valid in the above calculations.



Column Base Plate

3.17 Connection Of Precast Walls

Precast wall panels are usually single-storey high panels which are connected to each other and to the floor slabs. The connections are an integral part of the structural support system for vertical gravity dead and live load as well as for the transfer of horizontal in-plane forces from the floor diaphragm action. Figure 3.34 illustrates the different resulting joint force systems from internal and external forces.

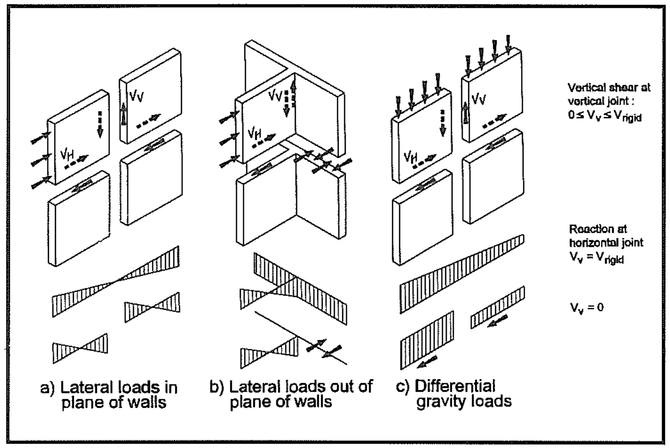


Figure 3.34 Exterior Forces And Joint Force System (references 5 and 9)

There are two principal types of joint in precast wall panels :

1. vertical joint for the purpose of transferring vertical shear forces from one wall component to the next with minimum relative movement
2. horizontal wall to floor and wall to foundation joint for the transferring of compressive, tensile and shear from one component to the other.

3.17.1. Vertical wall joint

Vertical wall joints may be formed by :

1. concentrating on the reinforcement crossing the joint at the top and bottom of the wall panels within the structural floor depth. The reinforcement serves as structural ties and provides artificial clamping forces to prevent the wall panels from separating. See Figure 3.35a.
2. embossing the edges of the wall panels with castellations or shear keys which act as mechanical locks when the panels deform under shear loading. Interlacing reinforcement projecting from the edges of the panels and running along the joint area as shown in Figure 3.35b may also be incorporated. The joint space is finally filled with concrete or grout.
3. mechanical connectors which consist of cast-in anchorage devices in the walls and steel section crossing the joint. As shown in Figure 3.36, the final connection is normally made by bolting or welding. The connections are usually located at the upper and lower region of the wall joint. The mechanical assemblies are eventually encased with concrete for protection against exposure to weather and fire.

Figures 3.35 and 3.36 show the differences between each of the above joints.

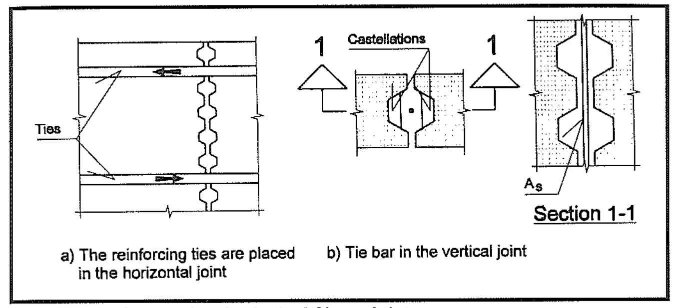


Figure 3.35 Connection At Vertical Shear Joints

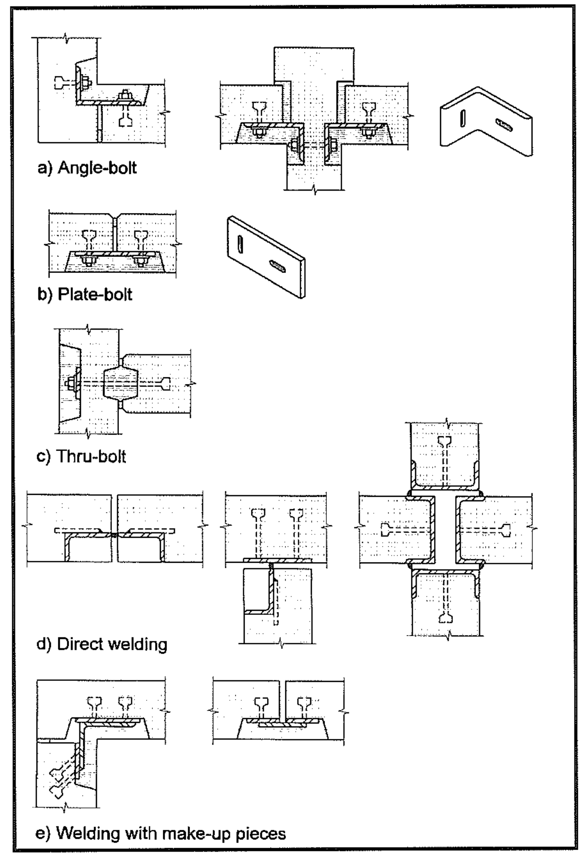


Figure 3.36 Mechanical Connection In Vertical Wall Joint (references 5 and 9)

3.17.2 Horizontal wall joint

Horizontal wall joints occur at floor levels and at the transition to the foundation or transfer beams. The principal forces to be transferred at the connection are essentially vertical gravity loads and horizontal forces from floor diaphragm action. The resulting forces acting at the joint for design considerations are shown in Figure 3.37 and consist of:

1. normal to joint – compressive and tensile
2. horizontal to joint – horizontal shear
3. vertical to joint at face – vertical shear
4. perpendicular to joint – compressive and tensile from floor diaphragm action and bending stresses from framing action of the wall with floor slabs

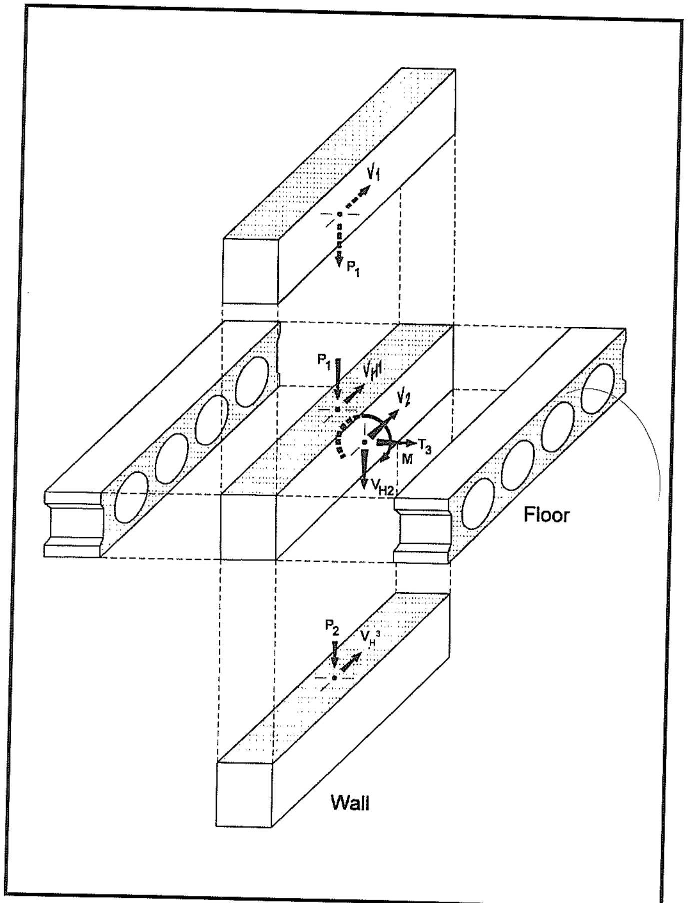


Figure 3.37 Exploded View Of An Interior Floor To Wall Connection And The Various Connection Forces (references 5 and 9)

The capacity of the joint to transfer vertical loads depends on a number of factors:

1. compressive strength of joint concrete and wall panels
2. ratio of loaded width to wall thickness
3. ratio of loaded width to joint thickness
4. splitting strength of wall ends and joint concrete
5. confinement of joint concrete
6. tensile strength of mechanical connectors

The horizontal load transfer capacity will depend on :

1. shear friction resistance
2. frictional resistance at interface
3. horizontal shear strength of mechanical connectors

Depending on the floor elements and their supporting details, the horizontal joint may be constructed continuous or with connections at isolated locations.

In a continuous joint, there are, in general, three basic types of details as shown in Figure 3.38.

1. thin mortar joint
2. wedged or open joint
3. platform or close joint

The isolated connections can be grouted pipe sleeves, dowels or mechanical connections with welded plates and bolts.

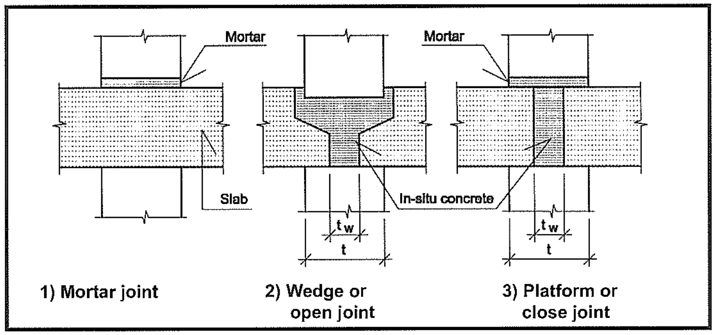


Figure 3.38 Different Forms Of Horizontal Wall Joints

3.17.3 Vertical load capacity of joint concrete or mortar

The mortar joint is cheap, simple and the most widely used continuous horizontal joint connection in precast wall construction. The mortar may be in fluid colloidal or dry-packed form and is either poured or packed in the joint space before or after the walls are erected. The joints are usually unreinforced.

The strength of the mortar joint depends primarily on the relative strength (and hence the elastic response) of the mortar and the walls as well as the dimensions of the joint. Under vertical loads, the joint may fail in one of the manners as shown in Figure 3.1.

The vertical load capacity of the mortar joint may be determined either by stipulations in Part 1, clause 5.3.6 of the Code or by other approaches as given in Section 3.6.1.

For wedge or platform horizontal wall joints shown in Figure 3.38, a simple design method to determine the vertical load capacity of the precast wall panels is outlined in the SBI Direction 115 - Danish Building Research Institute 1981 (reference 6).

The design method assumes the occurrence of split failure in the wall panel concrete as shown in Figure 3.39. The vertical stresses are assumed to be uniformly distributed over the panel width, t, and the width of the joint concrete, t_w. In the mid-vertical section of the wall, transverse stresses occur in the form of compressive stresses at the top and tensile stresses a little further down. The maximum transverse stresses in the wall panel concrete, denoted as f_t, is related to the vertical compressive stresses, f_c, by the simple relationship

$$f_t / f_c = (1 - t_w / t) / 2 \qquad \text{—— (3.76)}$$

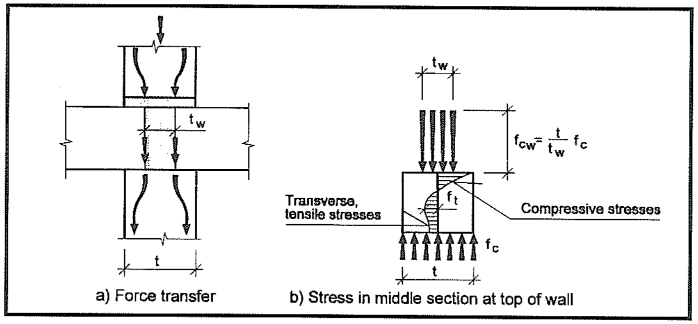


Figure 3.39 Stress Systems In Horizontal Wall Joint

Failure will occur if f_t is equal to the tensile strength of the concrete. If the tensile strength of the concrete is conservatively taken to be 10% of the compressive strength, then the vertical load capacity per unit length of the wall panel is calculated as below:

From equation 3.76 $\quad f_c = 2f_t / (1 - t_w / t)$

Assuming $f_t = 0.1 f_{cyd}$ $\quad f_c = 0.2 f_{cyd} / (1 - t_w / t)$

Where f_{cyd} is the cylinder compressive strength of the wall panel.

The vertical load carrying capacity per unit length of the wall is given as:

$$n_w = f_c \times t$$
$$= 0.2 f_{cyd} / (1 - t_w / t) \times t$$
$$= 0.2\, t\, f_{cyd} / (1 - t_w / t)$$

Assuming $f_{cu} = 1.25 f_{cyd}$

$$n_w = 0.16 \cdot f_{cu} / (1 - t_w / t) \qquad \text{—— (3.77)}$$

According to reference 6, equation 3.77 is valid up to a compressive failure of the wall panel when the compressive stresses in the wall panels reach $0.75 f_{cyd}$. (equivalent to $0.6 f_{cu}$).

Hence $n_w \leq 0.6 f_{cu} \times t$

3.17.4 Isolated connection

Typical examples of isolated connections in precast walls are shown in Figure 3.40. Since all forces are concentrated at a few points, special attention must be taken to ensure that the concentrated forces at the connection can be safely dispersed to the upper and lower wall panels. Small diameter reinforcement in the form of stirrups or loops should be detailed in the immediate vicinity of the connection in the wall panels to prevent splitting, bursting and spalling of concrete.

In cantilever shear walls subjected to overturning in-plane moment, the moment can be resolved into tension and compression force couple. The tension force can be resisted by grouted pipe sleeves, dowels, couplers, bolts or welded plates connection. Similar details may also be applied to the compression force as in most cases, the moment is reversible.

3.17.5 Structural ties in wall joint

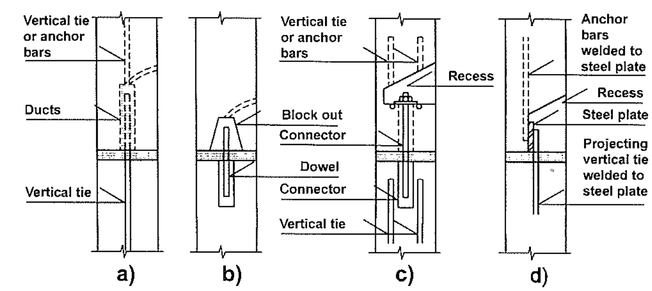
Structural vertical and horizontal integrity ties can be incorporated into the joint details and some typical examples are shown in Figure 3.35.

3.17.6 In-plane shear capacity

The in-plane shear capacity in vertical and horizontal wall joints with smooth or cast surfaces can be determined based on the permissible ultimate shear stress values given in Part 1, clause 5.3.7, under the following conditions :

1. 0.23 N/mm² in the absence of compressive forces across the joint
2. 0.45 N/mm² when the joint is under compression under all design conditions. The compressive force may be generated from gravity dead and live load or artificially created by reinforcement placed across the joint.

The in-plane shear capacity is only valid provided the joints are prevented from opening up.



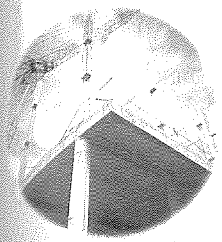
a) Vertical ducts and lapping of ties by grouting

b) Simple dowel pin connection for horizontal shear forces only

c) Mechanical bolting of vertical tie reinforcement

d) Welded joint

Figure 3.40 Isolated Connection In Wall Panels



CHAPTER 4

ILLUSTRATIONS ON DESIGN OF PRECAST CONCRETE BUILDINGS

CHAPTER 4: ILLUSTRATIONS ON DESIGN OF PRECAST CONCRETE BUILDINGS

4.1 Introduction

The principles and methods of design of precast concrete members and their connections as expressed in earlier chapters are demonstrated in this chapter by the design of a 12-storey office block and a 30-storey residential apartment.

The structural system of the office block is based on skeletal frame consisting of a framework of columns, beams and slabs. The skeletal frames are the most common structural system due to the advantage of greater flexibility in the building space arrangement and utilisation. They are also the most demanding of all precast structures because of the greater complexity of interplay of forces and movements of the structural components as compared to cast in-situ structures. It is important to understand the physical effects of these forces and how they are being transferred through the completed structure.

The residental apartment is constructed using cast in-situ prestressed flat plates and precast load bearing walls. It is an efficient structural system in this type of building as compared with a total precast solution.

In the following pages, the specimen calculations for the design of precast columns, beams,staircases, walls and their connections have been selected to illustrate as many as possible of the types of design in a building project for which calculations may have to be prepared. The drawings and calculations provided are illustrative examples and they are not intended to be sufficient to construct, or to obtain the necessary authority to construct, a building of the type and size considered.

Typical production details or shop drawings are also shown for each type of the precast components being considered. The layout and presentation are for illustrative purposes and will be different between manufacturers.

4.2 12-Storey Office Block

1. Project Description

The building is a 12-storey office block in a mix commercial development comprising carparks, shopping malls and service apartments.

A typical floor of the building measuring 24 m x 72 m with 8 m building grids in both directions is shown in Figure 4.1. The design floor-to-floor height is 3.6 m. Staircases, lift cores and other building services such as toilets, AHU, M&E risers are located at each end of the floor which are to be cast in-situ.

2. Design Information

a. Codes of Practice

BS 6399	Design Loading for Building
CP 65	The Structural Use of Concrete
CP3, Chapter V	Wind Load

b. Materials

Concrete	:	C30 for topping, walls and all other in-situ works
	:	C45 for precast beam
	:	C50 for precast columns and hollow core slabs
Steel	:	f_y = 250 N/mm² mild steel reinforcement
	:	f_y = 460 N/mm² high yield steel reinforcement
	:	f_y = 485 N/mm² for steel fabric reinforcement

c. Dead loads

Concrete density	=	24	kN/m³
Partitions, finishes and services	=	1.75	kN/m²
Brickwalls	=	3.0	kN/m² in elevation

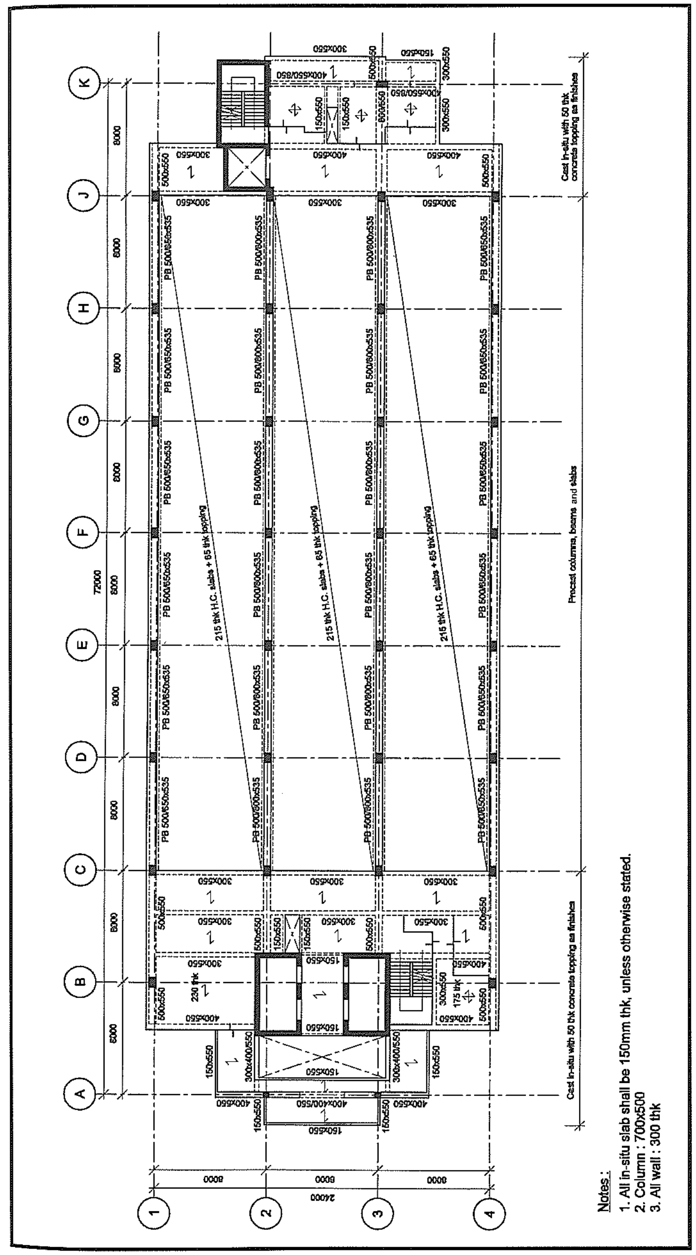


Figure 4.1 Typical Floor Plan

d. Live loads

Offices, staircases, corridors	=	3.0 kN/m^2
Lift lobby, AHU rooms	=	5.0 kN/m^2
Toilet	=	2.0 kN/m^2

3. Structural System

Precast construction is adopted from the second storey upwards to take advantage of the regular building grids and simple structural layout. The areas from grids A to C and from J to K are, however, cast in-situ due to drops, floor openings and water-tightness considerations. Beside acting as load bearing walls, staircase wells and lift cores also function as stabilising cores for the superstructure. The walls are 300 mm thick, cast in-situ and are tied monolithically at every floor.

The precast components consist of hollow core slabs, beams, columns and staircase flights.

a. Hollow core slabs

The design of hollow core slabs (215 mm thick) is based on class 2 prestressed concrete structure with minimum 2 hours fire rating. The hollow core slabs are cast with C50 concrete. Each unit (1.2 m nominal width) is designed as simply supported with nominal 100 mm seating at the support.

Resultant stresses are checked at serviceability and at prestress transfer. Design of the slab is carried out by the specialist supplier.

b. Precast beams

535 mm deep full precast beams are used in the office area. The beams, which are unpropped during construction, are seated directly onto column corbels and are designed as simply supported structures at the final stage. To limit cracking of the topping concrete at the supports, site placed reinforcement is provided as shown in the typical details in Figure 4.2.

c. Precast columns

The columns are 500 mm x 700 mm and are cast 2–storey in height with base plate connection at every alternate floor. They are designed as pin-ended at the ultimate limit state.

The base plate connection is designed with moment capacity to enable the columns to behave as a 2-storey high cantilever. This is to facilitate floor installation works which are to be carried out two floors in advance of a finally tied floor at any one time during the construction of the office block. The use of base plate connection will eliminate heavy column props and result in a safer and neater construction site. A nominal 50 mm gap is detailed in the design of the column-to-column connection in order to provide sufficient tolerances for the insertion of in-situ reinforcement at the beam support regions. The gap will be filled with C50 non-shrink grout.

Each column is cast with reinforced concrete corbels in the direction of the precast beams. The corbels are provided with T25 dowel bars which are used to prevent toppling of the precast beams when the hollow core slabs are laid. The depth of the corbel is designed to be concealed visually within the final ceiling space.

At the final state, all columns are considered braced in both directions.

d. Floor diaphragm action and structural integrity

All precast components are bound by a 65 mm thick concrete topping which is reinforced with a layer of steel fabric. The steel fabric serves as structural floor ties in order to satisfy the integrity ties requirement under the building robustness design considerations.

The final floor structure will behave as a rigid diaphragm which transmits horizontal loads to the stabilising cores at each end of the floor.

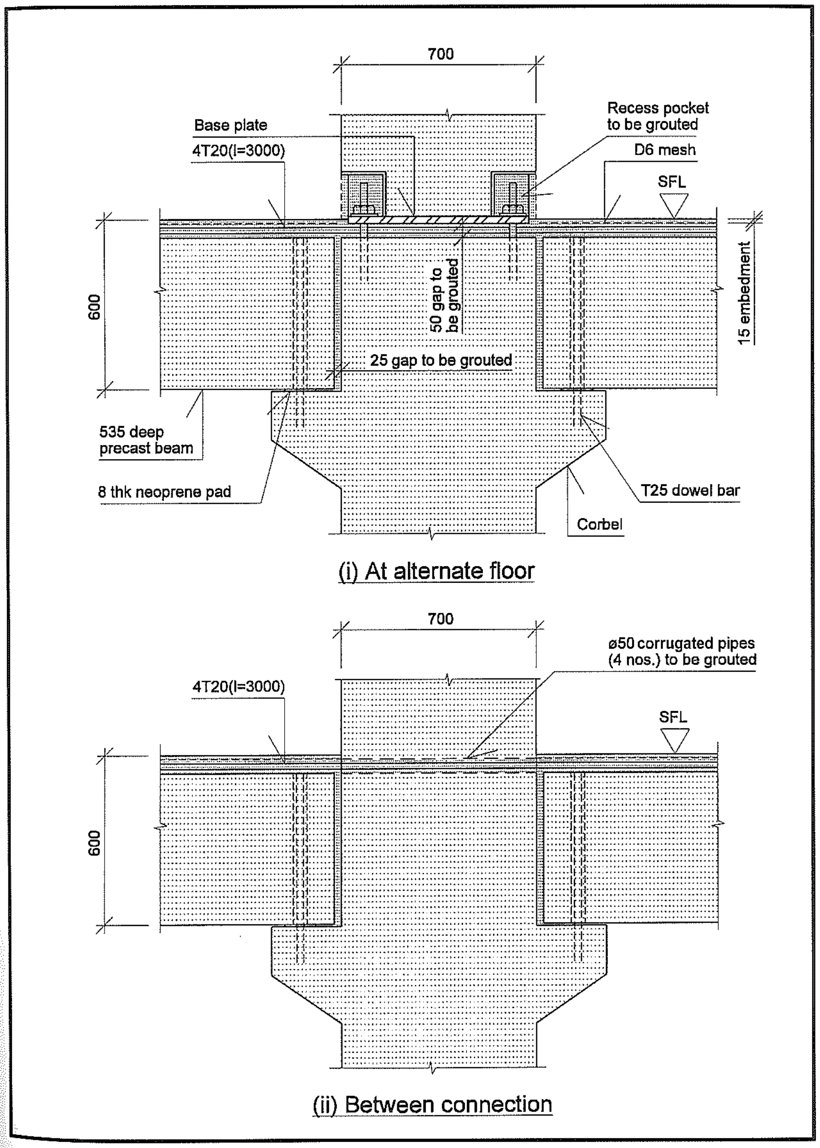
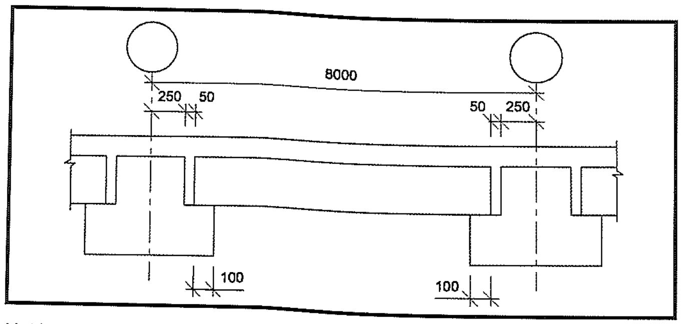


Figure 4.2 Typical Column/Column, Column/Beam Connections

4. Design Of Precast Components

A. Design of hollow core slabs



Net length of hollow core slab = 8000 – 500 –100
= 7400mm

Loading : DL = Topping (65 mm thick) = 0.065 x 24
= 1.56 kN/m²
Imposed = 1.75 kN/m²
LL = 3.00 kN/m²

From Figure 2.9 of Chapter 2, 215 mm thick hollow core slab is adequate. Actual design of prestressing reinforcement will be furnished by the producer of the slabs.

Support reinforcement

Although the slabs are designed as non-composite and simply supported, additional loose reinforcement should be placed in the topping concrete over the support in order to minimise surface cracking and limit the crack widths. The additional support reinforcement may be calculated as below :

Loading : DL (imposed) = 1.75 kN/m²
LL = 3.00 kN/m²

Ultimate load = $1.4 \times 1.75 + 1.6 \times 3.00$
= 7.25 kN/m²

Support moment = $0.083ql^2$
= $0.083 \times 7.25 \times 7.4^2$
= 33.0 kNm/m

h = 215 + 65
= 280 mm

z ≈ 0.8h
= 224 mm

A_s = $M/(0.87 f_y z)$
= $33.0 \times 10^6/(0.87 \times 460 \times 224)$
= 368 mm²/m

Use T10 - 200 c/c (A_s = 393 mm²/m)

Some typical connection details of hollow core slabs at the support and between units are shown in Figures 4.3 to 4.5.

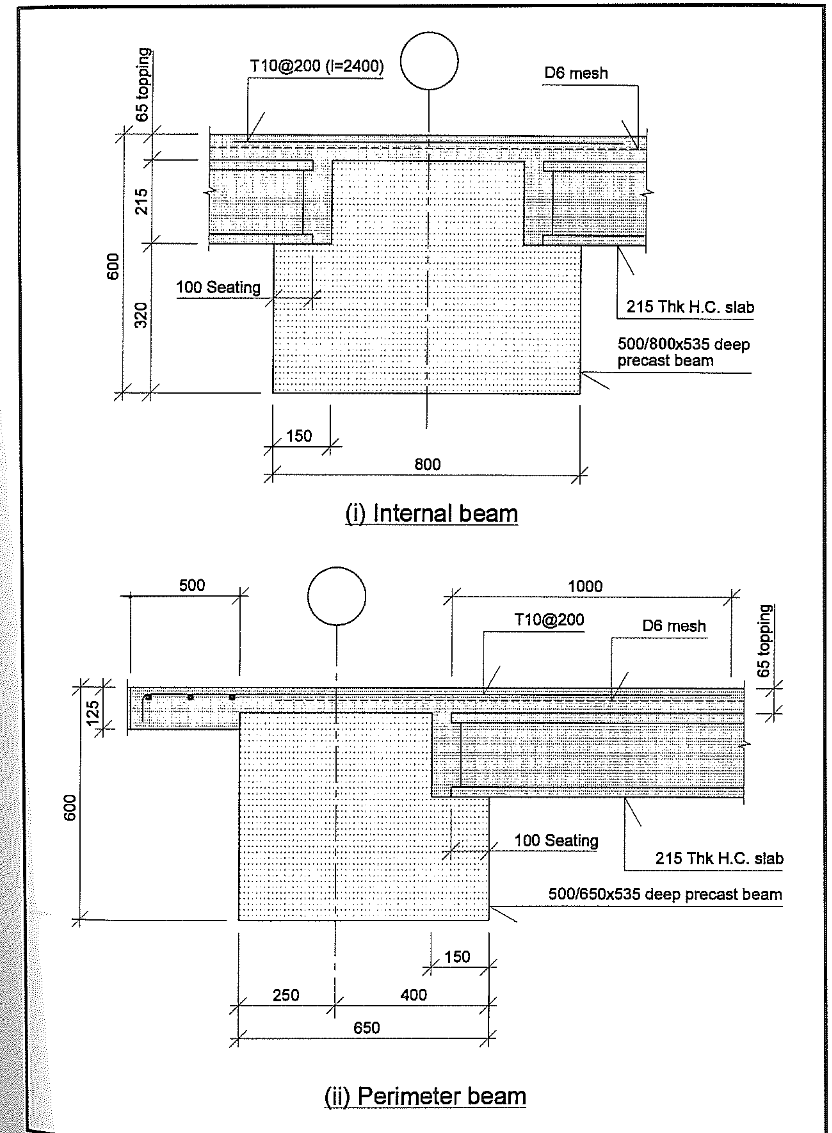


Figure 4.3 **Typical Details At Hollow Core Slab Supports**

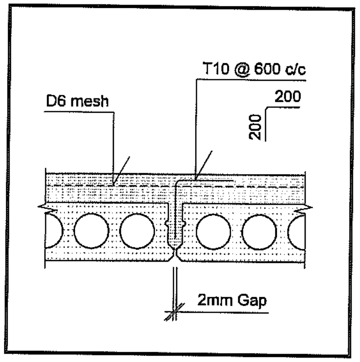


Figure 4.4 Typical Joint Details Between Hollow Core Slabs

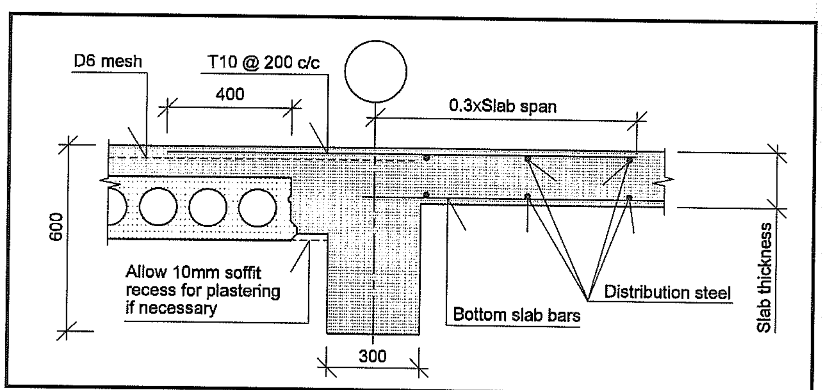
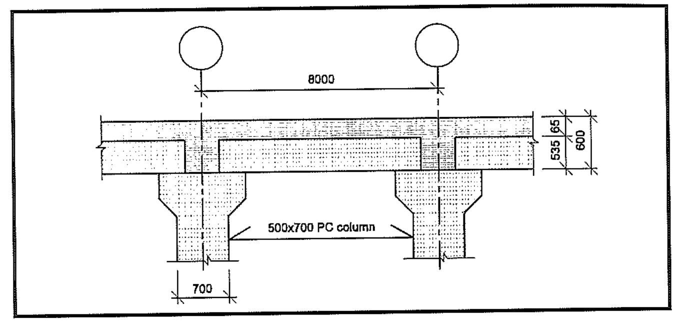


Figure 4.5 Typical Hollow Core Slabs/In-Situ Slabs Details

B. Design of precast beam



1. Design loading

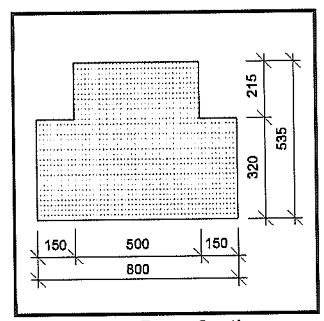
q = DL (beam s/w)	=	(0.8 x 0.32 + 0.215 x 0.5) x 24	=	8.72 kN/m
Hollow core slab (jointed wt)	=	2.89 x 7.5	=	21.67 kN/m
Topping	=	0.065 x 24 x 8	=	12.48 kN/m
Imposed	=	1.75 x 8	=	14.00 kN/m
		Total	=	56.87 kN/m
LL	=	3.0 x 8	=	24.00 kN/m

Design ultimate load = 1.4 x 56.87 + 1.6 x 24.00
= 118.03 kN/m

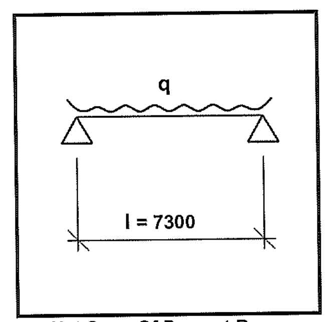
2. Limit state design

a. Bending moment

At mid-span $M^{+} = ql^2/8$
$= 118.03 \times 7.3^2/8$
$= 786.2$ kNm



Precast Beam Section



Net Span Of Precast Beam

$h = 535$ mm

$d \approx 535 - 75$
$= 460$ mm

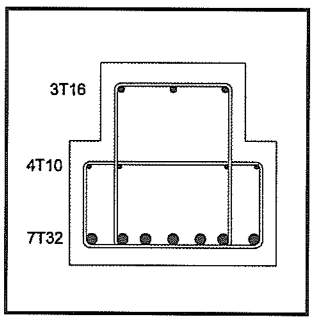
$b = 500$ mm

$M^+/(bd^2 f_{cu}) = 786.2 \times 10^6/(500 \times 460^2 \times 45)$
$= 0.165$

From Figure 2.26 in Chapter 2, $\rho_s/f_{cu} = 5.3 \times 10^{-4}$
$\rho_s = 5.3 \times 10^{-4} \times 45$
$= 0.0239$
$A_s = 5486$ mm²

Provide 7T32 ($A_s = 5629$ mm²)

$\rho_s'/f_{cu} =$ min. 4.4×10^{-4}
$\rho_s' = 0.02$
$A_s' = 460$ mm²
Provide 3T16 ($A_s = 603$ mm²) in top face as shown below.



b. Vertical shear

Total shear V $= 118.03 \times 7.3/2$
$= 430.8$ kN

$v = 430.8 \times 10^3/(500 \times 460)$
$= 1.87$ N/mm²

Minimum steel to be provided at the beam end to prevent bond slip failure is:

$A_s = V/0.87f_y$
$= 430.8 \times 10^3/(0.87 \times 460)$
$= 1076$ mm²
$= 0.47$ %

From Figure 2.27, in Chapter 2, $v_c = 0.59$ N/mm²
Hence $A_{sv}/s_v = (1.87 - 0.59) \times 500/(0.87 \times 460)$
$= 1.60$

Provide T10 – 75c/c for 1.4 m both ends. (0.2ℓ)
($A_{sv}/s_v = 2.09 > 1.60$, OK)

At 1.4 m from beam end, $V = 430.8 - 118.0 \times 1.4$
$= 265.6$ kN

$v = 265.6 \times 10^3 / (500 \times 460)$
$= 1.16$ N/mm²

$A_{sv}/s_v = (1.16 - 0.59) \times 500 / (0.87 \times 460)$
$= 0.71$

Provide T10-200 links ($A_{sv}/ s_v = 0.78 > 0.71$) for the remaining middle span.

c. Interface horizontal shear

Although the topping is considered non-structural, it is prudent to check that there should be no separation of the topping from the precast beam at the interface. At mid-span, compression generated in the topping concrete is:

Compression Force $= 0.45 f_{cu}\, b_e\, t$
$= 0.45 \times 30 \times 500 \times 65 \times 10^{-3}$
$= 438.8$ kN

Contact width $= 500$ mm
Contact length $= 0.5\ell_e$
$= 0.5 \times 7.3$
$= 3.65$ m

Average interface horizontal shear stress,
$v_h = 438.8 \times 10^3 / (500 \times 3650)$
$= 0.24$ N/mm²
peak $v_h = 2 \times 0.24$
$= 0.48$ N/mm²

which is less than 0.55 N/mm² in Part 1, Table 5.5, CP65 for as-cast surface without links. Hence the topping layer should not separate at the interface.

d. Support reinforcement

Although the beams are designed as simply supported structures, it is advisable to place additional reinforcement over the support to prevent excessive cracking when the beams rotate at the support under service load. The additional steel may be calculated as follows :

DL (imposed) $= 1.75 \times 8 = 14.0$ kN/m
LL $= 3.0 \times 8 = 24.0$ kN/m

Ultimate loading $= 1.4 \times 14.0 + 1.6 \times 24.0$
$= 58.0$ kN/m

Support moment $\approx 0.08ql^2$
$= 0.08 \times 58.0 \times 7.3^2$
$= 247.3$ kNm

$A_s = M / (0.87 f_y\, z)$
$z \approx 0.8h$
$A_s = 247.3 \times 10^6 / (0.87 \times 460 \times 0.8 \times 600)$
$= 1287$ mm²

Provide 4T20 ($A_s = 1257$ mm²) over the support as shown in Figure 4.2.

e. Deflection

At mid-span, $M^+ = 786.2$ kNm
$M^+/bd^2 = 786.2 \times 10^6 / (500 \times 460^2)$
$= 7.43$

$f_s = (5/8) f_y \times (A_{s\ req} / A_{s\ prov})$
$= (5/8) \times 460 \times (5486 / 5629)$
$= 280$ N/mm²

From Part 1, Table 3.11, CP65 the modification factor for tension reinforcement = 0.75 and for compression reinforcement = 1.08

Minimum d $= 7300 / (20 \times 0.75 \times 1.08)$
$= 450.6\text{mm} < 460$ mm OK

f. End bearing

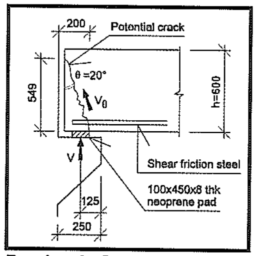
i. Effective bearing width

Permissible ultimate bearing stress from Part 1, clause 5.2.3.4, CP 65

$= 0.4f_{cu}$
$= 0.4 \times 45$
$= 18\ \text{N/mm}^2$

To even out surface irregularities at the beam and corbel surfaces, provide 100 x 450 x 8 mm thick neoprene pad at the beam support.

Contact pressure $= 430.8 \times 10^3/(100 \times 450)$
$= 9.6\ \text{N/mm}^2 < 18\ \text{N/mm}^2$ OK



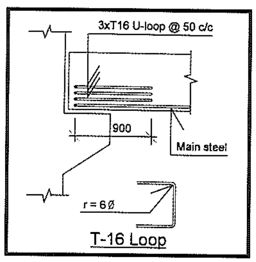
Bearing At Support

ii. Shear friction steel

Horizontal shear friction steel is calculated as $A_s = V/0.87f_y$ assuming conservatively $\mu = 1.0$

$A_s = 430.8 \times 10^3/(0.87 \times 460)$
$= 1076\ \text{mm}^2$

Provide 3T16 loops at 50 c/c at the beam ends as shown. ($A_s = 1207\ \text{mm}^2$)



Check minimum bar bending radius, r

Tension force per bar $= 0.87 \times 460 \times 201 \times 10^{-3}$
$= 80.4\ \text{kN}$

Actual tension force per bar $F_{bt} = 80.4 \times 1076/1207$
$= 71.7\ \text{kN}$

$a_b = 50\ \text{mm}$
$\phi = 16\ \text{mm}$

Minimum r $= F_{bt}[1 + 2(\phi/a_b)] / 2 f_{cu}\phi$ (Equation 50 Part 1)
$= 71.7 \times 10^3[1 + 2(16/50)] / (2 \times 45 \times 16)$
$= 82\ \text{mm}$

Say r $= 6\phi$ (96 mm)

g. Production Details

Typical production details of the precast beam are shown in Figure 4.6.

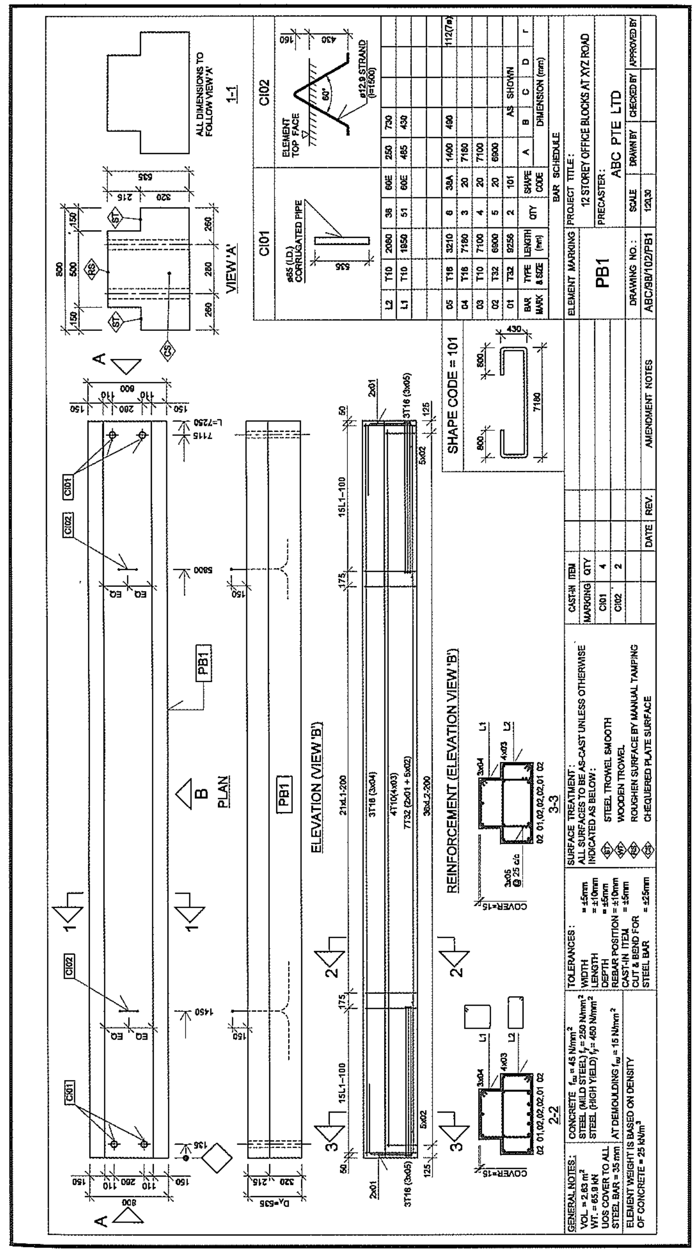


Figure 4.6 Production Details Of Precast Beam (12-Storey Office Block)

C. Design Of Precast Column

A typical 2-storey high precast column is shown in Figure 4.7 below:

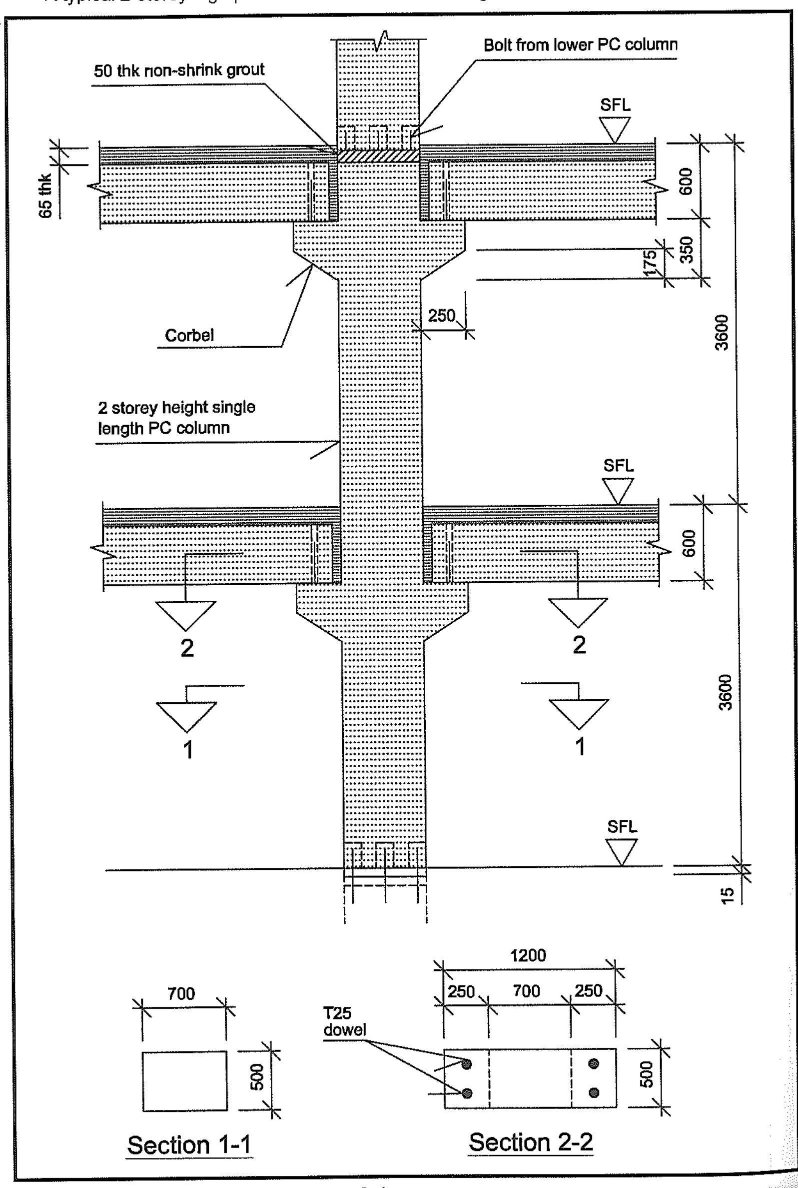


Figure 4.7 Typical 2 Storey High Precast Columns

1. Design loading

Per floor :

DL : beams = (0.8 x 0.32 + 0.215 x 0.5) x 24 x 7.3	= 63.7 kN
h.c.s. (jointed weight) = 2.89 x 7.5 x 8	= 173.4 kN
topping = 0.065 x 24 x 8 x 8	= 99.8 kN
imposed dead load = 1.75 x 8 x 8	= 112.0 kN
self weight = 0.7 x 0.5 x 24 x 3.6	= 30.2 kN
corbel = 0.2625 x 0.5x 0.5 x 24 x 2	= 1.6 kN
	480.7 kN
LL = 3 x 8 x 8	= 192.0 kN

From roof to 2[nd] storey (12 floors), total column load (above 1[st] storey):

DL = 480.7 x 12 = 5768.4 kN
LL (50% reduction) = 192.0 x 12 x 0.5 = 1152.0 kN

Ultimate axial load = 1.4 x 5768.4 + 1.6 x 1152.0
= 9919.0 kN

2. Column design

All columns are braced and pinned at the floor.

About minor axis : l_e $= \beta l_o$
$= 1.0 \times 3.6$
$= 3.6$ m

$l_e/b = 3600/500$
$= 7.2 < 15$

Columns are considered as short columns.

From Part 1, equation 38, CP 65,

$N = 0.4 f_{cu} A_c + 0.75 A_{sc} f_y$
$N = 9919.0$ kN
$f_{cu} = 50$ N/mm²

$A_{sc} = (9919.0 \times 10^3 - 0.4 \times 50 \times 500 \times 700) / (0.75 \times 460)$
$= 8461$ mm²

Provide 12T32 (A_{sc} = 9650 mm², 2.8%)

Check bearing capacity of column section in contact with base plate

The ultimate bearing stress at interface with steel plate = $0.8 f_{cu}$ (Part 1, clause 5.2.3.4)

Plate size = 450 x 650mm

Hence direct bearing capacity = $0.8 \times 50 \times 450 \times 650 \times 10^{-3}$
= 11700 kN > 9919.0 kN

Steel reinforcement need not be provided from the steel plate into the column section.

3. **Column to column joint**

a. **Vertical joint strength**

Joint between column to column is 50 mm thick and is embedded 15 mm below the floor's final structural level (Figure 4.2). Based on the lower column section, the ultimate bearing stress for bedded bearing = $0.6f_{cu}$ (clause 5.2.3.4).

Assuming non-shrink grout strength f_{cu} = 50 N/mm²,

Total bearing strength = $0.6 \times 50 \times 700 \times 500 \times 10^{-3}$
= 10500 kN > 9919.0 kN OK

b. **Base plate connection**

The steel base plate connection at alternate floors is used to provide moment capacity for the stability of two-storey high cantilever column at the installation stage. The base plate connection is designed to allow for precast components to be installed two storeys ahead of a completed floor. The columns are unpropped in order to achieve fast installation and minimise site obstruction by props. The design of the base plate connection needs to consider different load combinations so as to arrive at the most critical stresses. In accordance with the Code, the following load combinations are considered:

Case 1 = full dead and construction live loads
= 1.4DL + 1.6LL

Case 2 = full dead + construction live + wind (or notional load)
= 1.2 (DL + LL + WL) or
1.2 (DL + LL) + Notional load

Case 3 = full dead + wind (or notional load)
= 1.4 (DL + WL) or
1.4 (DL) + Notional load

The design wind load is considered less critical and its effects are mainly felt from the exposed faces of the beam and column perpendicular to the wind direction. This will be less than the notional load of 1.5% dead load.

The base plate connection is be checked in both the major and minor column axes and taking into account the column slenderness effect.

By inspection, the most critical stresses will take place at the edge columns next to the in-situ floor as there will be minimum axial load with maximum overturning moment and hence maximum tensile forces in the bolts during the temporary stage. The transfer of floor loading to the edge column is shown in Figure 4.8.

A final check on the connection should be made to ensure that continuous vertical ties are provided through the column joint and throughout the building height.

Axial column load per floor

DL : Beams = (0.8 x 0.32 + 0.215 x 0.5) x 24 x 7.3/2 = 31.8 kN
Hollow core slab (jointed weight) = 2.89 x 7.5 x 4 = 86.7 kN
Topping = 0.065 x 24 x 4 x 8 = 50.0 kN
168.5 kN

LL (construction) = 1.5 x 4 x 8 = 48.0 kN

Notional horizontal load

Per floor H = 1.5 x 168.5/100 = 2.5 kN acting at the top of the corbel

Bending moment at point 3 in Figure 4.8 = 2.5 x (3.015 + 6.615) = 24.1 kNm

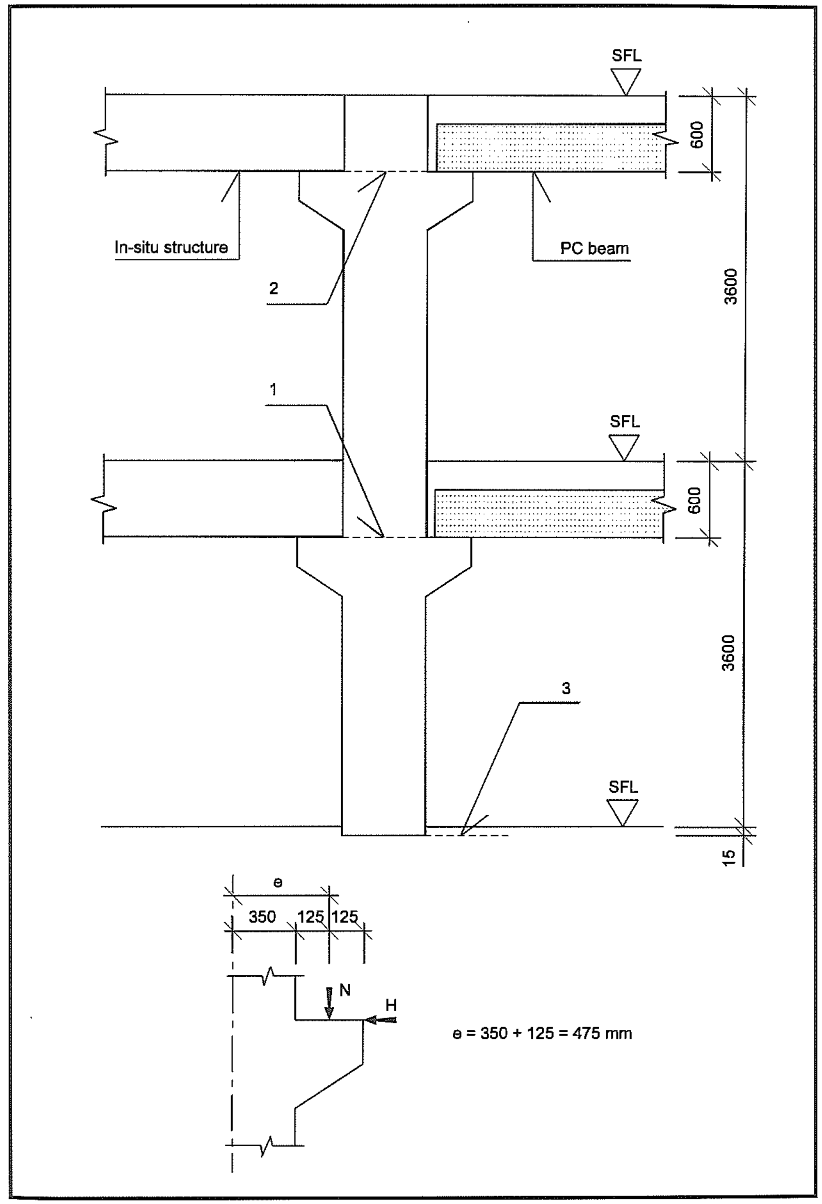


Figure 4.8 Loadings At Edge Column

Deflection in slender column (Part 1, clause 3.8.3, assuming K=1.0)

Effective column height at 1, l_{e1} $= 2.2 \times 3.015$
$= 6.633$ m

at 2, l_{e2} $= 2.2 \times 6.615$
$= 14.553$ m

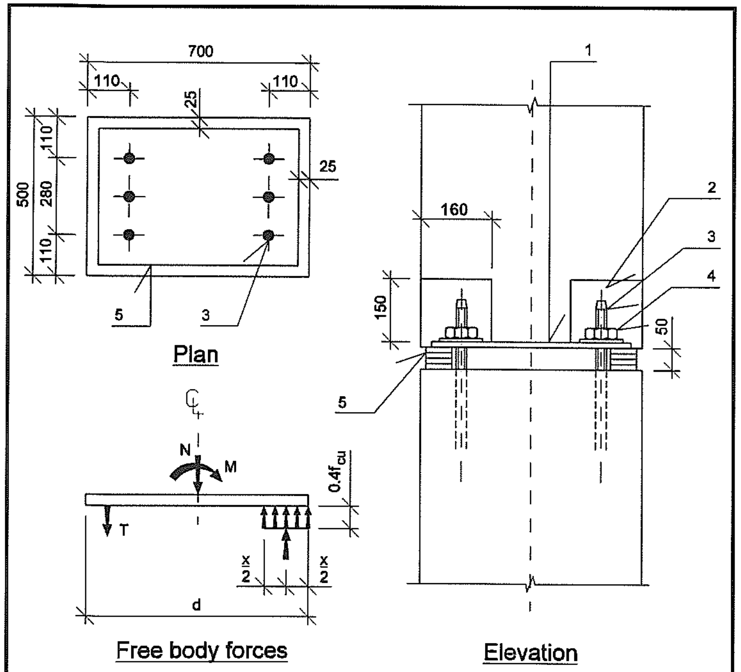
About major axis at 1, a_{u1} $= (6.633/0.70)^2 \times (1/2000) \times 1.0 \times 0.7$
$= 0.031$ m

at 2, a_{u2} $= (14.553/0.70)^2 \times (1/2000) \times 1.0 \times 0.7$
$= 0.151$ m

About minor axis at 1, a_{u1} $= (6.633/0.5)^2 \times (1/2000) \times 1.0 \times 0.5$
$= 0.044$ m

at 2, a_{u2} $= (14.553/0.5)^2 \times (1/2000) \times 1.0 \times 0.5$
$= 0.212$ m

Additional moment at point 3, $M = N(a_{u1} + a_{u2})$



1. Base plate
2. Recess pocket (to be grouted)
3. Holding bolt
4. Nuts with washer plate
5. Shimming plates at each corner

Free Body Forces In Base Plate

Referring to the free body force diagram in the base plate and using force and moment equilibrium conditions, the general expression for χ is given by:

$(\chi/d)^2 - 2(1 - d'/d)(\chi/d) + 5[M + N(d/2 - d') / f_{cu}bd^2] = 0$

Major axis bending : b = 450, d = 650, d' = 85

Minor axis bending : b = 650, d = 450, d' = 85

The total tension developed in the holding down bolt is given by:

$$T = C - N$$

where $C = 0.4f_{cu}\, b\, \chi$

Calculations of tension forces in the bolts are shown in the table below.

	Case 1		Case 2		Case 3	
	1.4DL + 1.6LL		1.2 (DL + LL) + Notional Load		1.4DL + Notional Load	
Axial load ΣN (kN)	312.7 x 2 = 625.4		259.8 x 2 = 519.6		235.9 x 2 = 471.8	
Bending moment (kNm) at Point 3 (Fig 4.8): About	Major axis	Minor axis	Major axis	Minor axis	Major axis	Minor axis
Notional load	-	-	24.1	24.1	24.1	24.1
Column slenderness $N(a_{u1} + a_{u2})$	56.9	80.1	47.3	66.5	42.9	60.4
Load eccentricity (N x e)	297.1	-	246.8	-	224.1	-
Total	354.0	80.1	318.2	90.6	291.1	84.5
χ/d	0.169	0.083	0.146	0.081	0.132	0.074
χ (mm)	109.9	37.4	94.9	36.5	85.8	33.3
Total tension in bolt, kN	363.7	No tension	334.5	No tension	300.4	No tension

Tension Forces In Base Plate Bolts

i. **Design of holding down bolts**

There is no tension in the bolts where the column bends about its minor axis. The maximum tension in bending about the major axis is in Case 1 with T = 363.7 kN. If 3 numbers of bolts are provided per face of the column, then the tension developed in each bolt is

$$T = 363.7/3 = 121.2 \text{ kN}$$

Bolt area A_s, required $= 121.2 \times 10^3/(0.87 \times 460) = 303 \text{ mm}^2$

Use T32 bars with M24 threading. The total root tension area $\approx 0.8 \times 24^2 \times \pi/4 = 362 \text{ mm}^2 > 303 \text{ mm}^2$ OK

From equation 49 and Table 3.28 in Part 1 of the Code, the ultimate anchorage bond stress in the bolt is:

$$f_{bu} = \beta\sqrt{f_{cu}}$$
$$\beta = 0.50$$
$$f_{bu} = 0.5\sqrt{50} = 3.53 \text{ N/mm}^2$$

Bolt perimeter $= \pi d = \pi \times 32 = 100.5$ mm

Tension $= 121.2$ kN

Hence anchorage length $l_p = 121.2 \times 10^3/(3.53 \times 100.5) = 342$ mm

Say $l_p = 400$ mm

ii. **Design of base plate**

Block-out pocket for bolt = 160 x 160 x 150 height at the column face. Centre of bolt to the edge of inner block-out

$$c = 160 - 110 = 50 \text{ mm}$$

Based on compression side:

$$t = \sqrt{0.8 f_{cu}\, a^2/p_y}$$

$a = 50 + 85 = 135$ mm

$p_y = 265 \text{ N/mm}^2$ (grade 43 steel, assuming plate thickness > 16mm)

Hence $t = \sqrt{0.8 \times 50 \times 135^2/265} = 52.4$ mm

Based on tension side:

$$t = \sqrt{4Tc/b\, p_y}$$

T = 363.7 kN
c = 50
b = 450

$$t = \sqrt{4 \times 363.7 \times 10^3 \times 50/(450 \times 265)} = 24.7 \text{ mm}$$

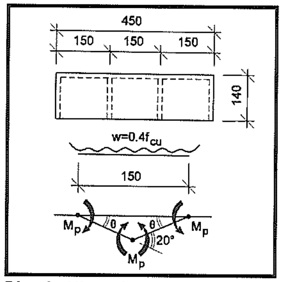
Minimum plate thickness = 51.5 mm. However, this is considered excessive. Instead, 4 numbers of stiffener plates are added to dissipate the compression from the base plate to the column section at the top of the block-out.

Total compression $= 0.4 f_{cu}\, b\, \chi$ (Refer to case 1 for χ value)
$= 0.4 \times 50 \times 450 \times 109.9 \times 10^{-3}$
$= 989.1$ kN

Minimum stiffener plate thickness $t = 989.1 \times 10^3/(4 \times 135 \times 265) = 6.9$ mm

Say $t = 9$ mm thick

Design of top plate in block-out



Plastic Hinge In Steel Plate

Using plastic analysis, the work done by load w
$= w \times 140 \times 150 / 2$
$= 0.4 \times 50 \times 140 \times 150 / 2$
$= 2.10 \times 10^5$

Energy spent in plastic hinges $= M_p\theta + 2M_p\theta + M_p\theta = 4M_p\theta$

$$M_p = p_y(t^2 \times 100)/4 = 35 p_y t^2$$
$$\theta = 1/75$$
$$p_y = 265 \text{ N/mm}^2$$

Hence $4 \times 35 \times 265 \times t^2 \times 1/75 = 2.10 \times 10^5$

$$t = \sqrt{2.10 \times 10^5 \times 75/(4 \times 35 \times 265)} = 20.6 \text{ mm}$$

Say $t = 20$ mm

Adopt : Base plate = 450 x 650 x 25 mm thick
Vertical stiffener = 9 mm thick
Top plate at block-out = 450 x 140 x 20 mm thick

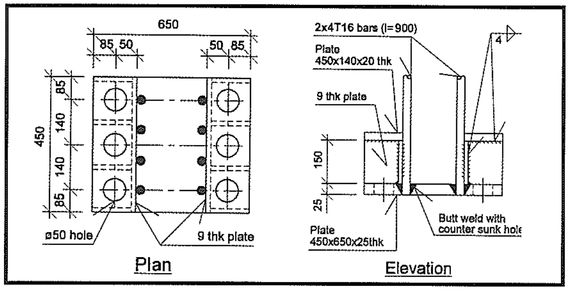
iii. Anchorage of base plate to column

Tension per bolt $= 121.2$ kN

$A_s = 303$ mm^2 (see earlier calculations)

Provide 8T16 ($A_s = 1609$ mm^2, see below calculation for vertical tie requirements) butt welded to the base plate.

Details of the base plate are shown in the figure below.



Base Plate Details

iv. Vertical tie requirement

Maximum axial load per floor $= 1.05 \times 480.7 + 0.33 \times 192.0$
$= 568.1$ kN

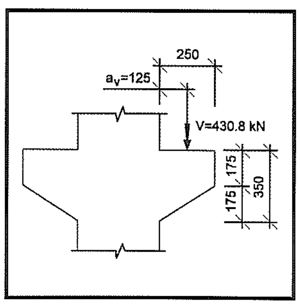
Vertical tie reinforcement $A_s = 568.1 \times 10^3/460$
$= 1235$ mm^2

Total provision of bolt (root area) $= 362 \times 6$
$= 2172$ mm^2 > 1235 mm^2 OK

Total provision of welded bars to base plate
$=$ 8T16
$= 1609$ mm^2 > 1235 mm^2 OK

Vertical column tie requirement is satisfactory at the column to column joint.

4. Design of corbel



a. Check shear stress

$v = V/bd$
$= 430.8 \times 10^3/(500 \times 275)$
$= 3.13$ N/mm^2 $< 0.8\sqrt{f_{cu}} = 5.66$ N/mm^2
adopt max. $v_c = 5.00$ N/mm^2 OK

b. Reinforcement

i. Main steel

$$v/f_{cu} = \frac{0.9(z/d)(a_v/d)(1 - z/d)}{(a_v/d)^2 + (z/d)^2}$$

$v/f_{cu} = 3.13 / 50 = 0.063$
$a_v/d = 125 / 275 = 0.454$

Hence $(z/d)^2 - 0.866(z/d) + 0.0276 = 0$
$z/d = 0.833$

Steel stress $f_s = 200\ (z/d - 0.55) / (1 - z/d)$
$= 200\ (0.833 - 0.55) / (1 - 0.833)$
$= 339$ N/mm^2

$F_t = V \times a_v/z$
$= 430.8 \times 125 / (0.833 \times 275)$
$= 235.1$ kN $> V/2$

$A_s = F_t/f_s$
$= 235.1 \times 10^3 / 339$
$= 694$ mm^2

Provide 3T13 U-loop (6 legs, $A_s = 796$ mm^2) at 25 c/c.

ii. Check bearing stress within bend

Tension per leg of T13 bar, $F_t = 235.1 \times 694 / (6 \times 796)$
$= 34.2$ kN

Minimum bending radius r $= \dfrac{F_t}{\phi} \times \dfrac{1 + 2(\phi/a_b)}{2f_{cu}}$

$F_t = 34.2$ kN
$\phi = 13$mm
$a_b = 25$mm

$r = 34.2 \times 10^3 \times [1 + 2(13/25)] / (13 \times 2 \times 50)$
$= 53.6$ mm

Use $r = 5\phi$
$= 65$mm

iii. Shear links

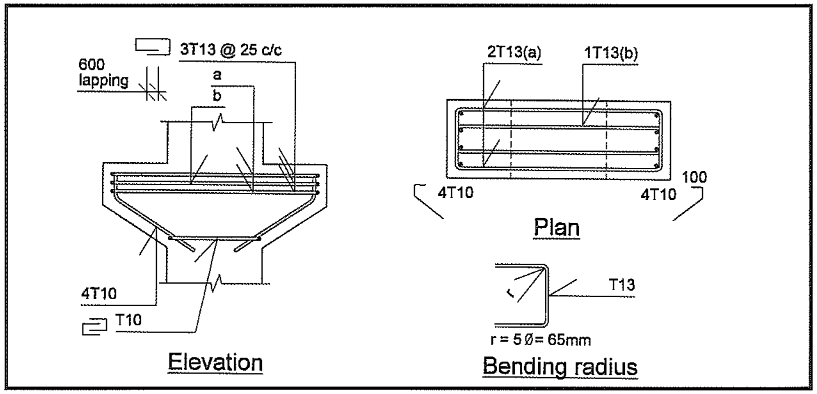
$$\rho_s = 100A_s/bd$$
$$= 100 \times 796/(500 \times 275)$$
$$= 0.58\%$$

$$v_c = 0.66 \times 1.10$$
$$= 0.73 \text{ N/mm}^2$$

Enhanced $v_c' = 2d \times v_c/a_v$
$$= 2 \times 275 \times 0.73/125$$
$$= 3.21 \text{ N/mm}^2 > 3.13 \text{ N/mm}^2$$

No shear links needed.

Detailing of corbel is as shown below.



Corbel Reinforcement

5. Production Details

The production details of a typical precast column are shown in Figures 4.9 and 4.10.

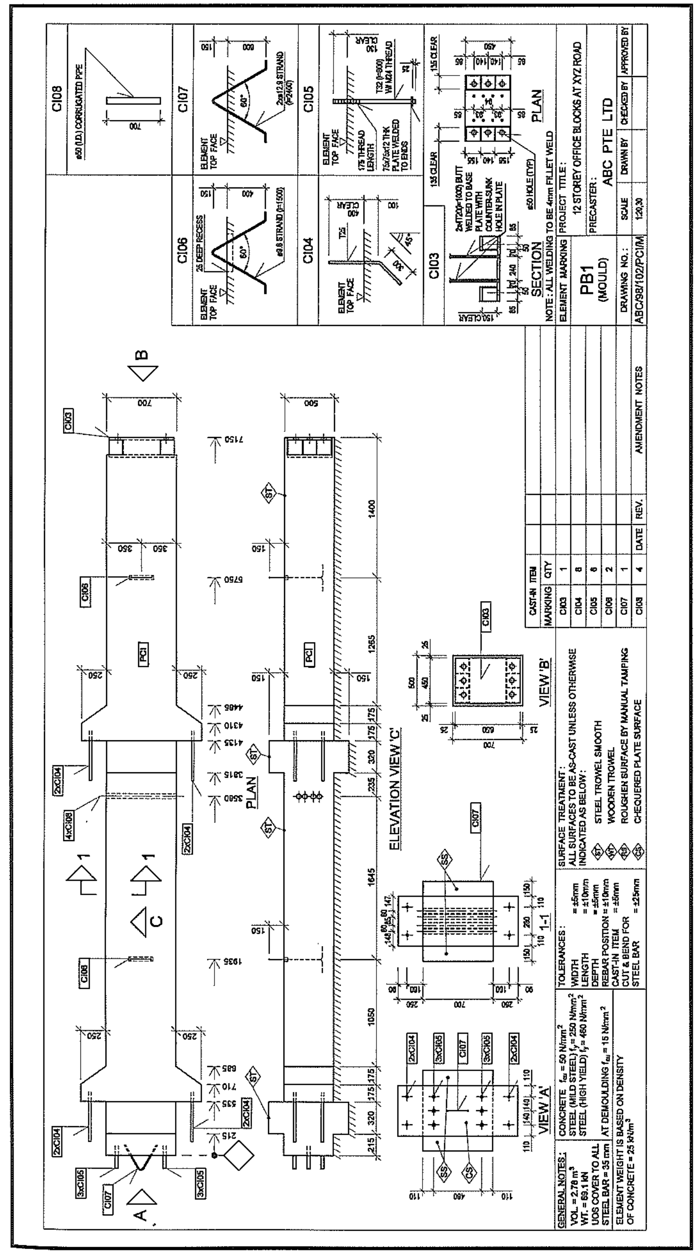


Figure 4.9 Mould Details Of Precast Column (12-Storey Office Block)

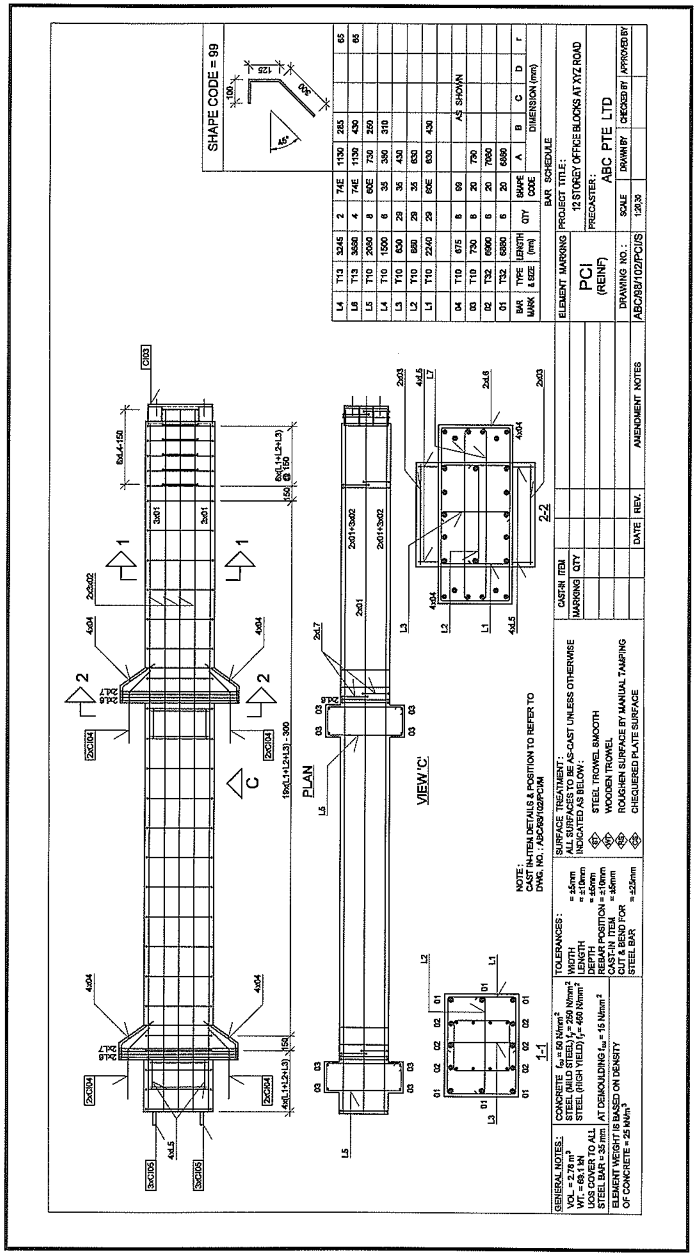
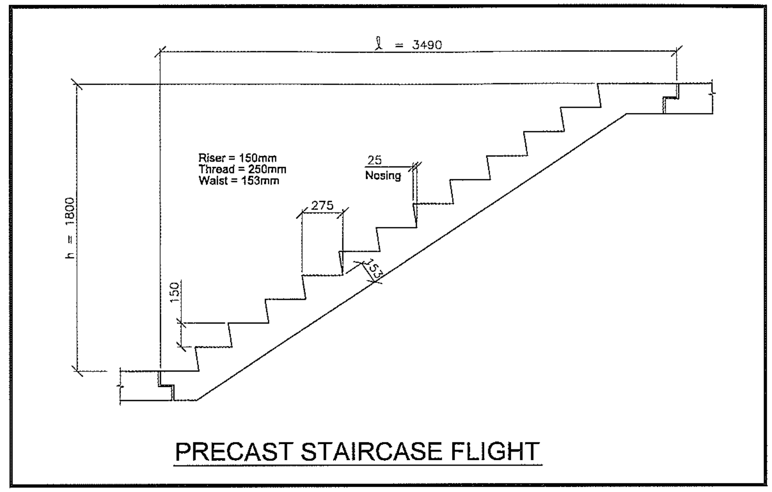


Figure 4.10 Reforcement Details Of Precast Column (12-Storey Office Block)

D. Precast Staircase Flight



Design Data:

concrete	:	f_{cu} = 45 N/mm^2	
steel	:	High Yield Steel :	f_y = 460 N/mm^2
		Mild Steel :	f_y = 250 N/mm^2

Design Live Load = 3.0 kN/m^2

Consider per metre width of stairs:

Slope length of stairs = $\sqrt{1.8^2 + 3.49^2}$
= 3.927 m

Weight of waist plus steps = (0.153 x 3.927 + 0.25 x 0.150 x 12/2) x 24
= 19.82 kN/m

Live Load = 3.0 x 3.49
= 10.47 kN/m

Ultimate Load F = 1.4 x 19.82 + 1.6 x 10.47
= 44.50 kN/m

1. **Beading Reinforcement**
Assuming no effective restraint at support:
Ultimate bending moment at mid-span

$$M = \frac{F\ell}{8}$$
$$= 44.50 \times 3.49/8$$
$$= 19.41 \text{ kN/m}$$

Effective d = 153 - 25 - 5
= 123 mm

Say $d = 120$ mm

$$\frac{z}{d} = 0.5 + \sqrt{0.25 - \left(\frac{M}{0.9bd^2f_{cu}}\right)}$$

$$= 0.5 + \sqrt{0.25 - \left(\frac{19.41 \times 10^6}{0.9 \times 1000 \times 120^2 \times 45}\right)}$$

$$= 0.965$$

use $\frac{z}{d} = 0.95$

$z = 114$ mm

$$A_s = \frac{M}{0.87f_yz}$$

$$= \frac{19.41 \times 10^6}{0.87 \times 460 \times 114}$$

$$= 425 \text{ mm}^2/\text{m}$$

Use T10-150 c/c. provided $A_s = 523$ mm²/m

2. Deflection

$$\frac{M}{bd^2} = \frac{19.41 \times 10^6}{1000 \times 120^2}$$

$$= 1.35$$

$$f_s = 5/8\ f_y \times A_{s\ req} / A_{s\ prov}$$

$$= 5/8 \times 460 \times 425/523$$

$$= 234 \text{ N/mm}^2$$

Tension modification factor

$$ø = 0.55 + \frac{(477 - f_y)}{120\ (0.9 + M/bd^2)}$$

$$= 0.55 + \frac{477 - 234}{120(0.9 + 1.35)}$$

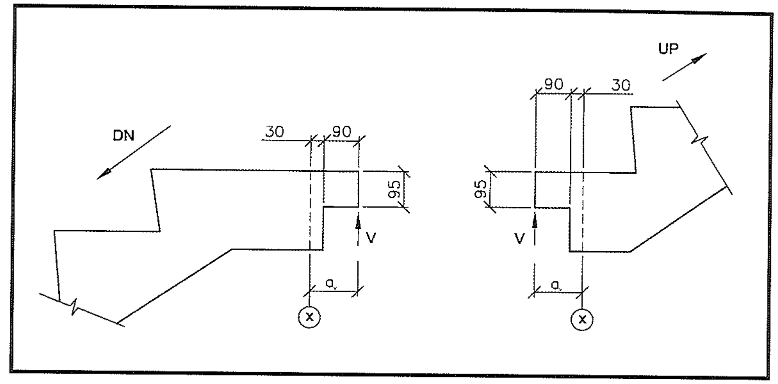
Cl.3.10.2.2, Part1, CP 65, basic span/depth ratio of staircase flight $= 20 \times 1.15$
$= 23$

Minimum effective d $= \dfrac{\ell}{ø \times (\text{span/depth ratio})}$

$$= \frac{3490}{1.45 \times 23}$$

$= 105$ mm < 120 mm OK

3. Design of supporting Nibs



Reaction at supporting nibs V $= 44.50/2$
$= 22.25$ kN/m

$a_v = 90 + 30$
$= 120$ mm

Bending moment at section X $= M = V \times a_v$
$= 22.25 \times 0.12$
$= 2.67$kNm/m

Effective d $= 95 - 20 - 5$
$= 70$ mm

Bending Steel

$$\frac{z}{d} = 0.5 + \sqrt{0.25 - \left(\frac{M}{0.9bd^2f_{cu}}\right)}$$

$$= 0.5 + \sqrt{0.25 - \left(\frac{2.67 \times 10^6}{0.9 \times 1000 \times 70^2 \times 45}\right)}$$

$$= 0.97$$

use $\frac{z}{d} = 0.95$

$z = 66.5$ mm

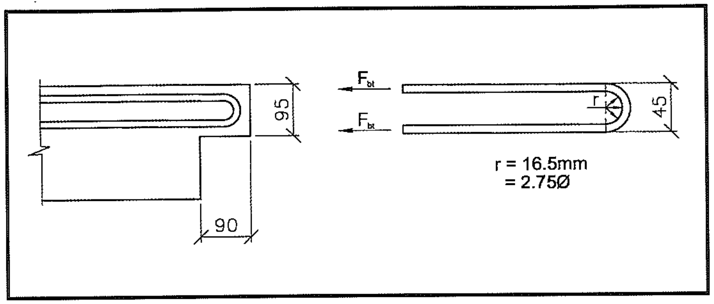
Use mild steel: $A_s = \dfrac{M}{0.87f_yz}$

$$= \frac{2.67 \times 10^6}{0.87 \times 250 \times 66.5}$$

$$= 185 \text{ mm}^2/\text{m}$$

use R6 @150 c/c ($A_s = 188$ mm²/m)

Check anchorage



$$F_{bt} = 0.87 f_y A_s$$
$$= 0.87 \times 250 \times 28 \times 10^{-3}$$
$$= 6.09 \text{ kN}$$

Minimum bending radius

$$r \geq \frac{F_{bt}}{ø} \times \frac{1 + 2(ø/a_b)}{2f_{cu}}$$

$ø = 6$ mm
$a_b = 150$ mm
$f_{cu} = 40$ N/mm^2

$$r \geq \frac{6.09 \times 10^3}{6} \times \frac{1 + 2(6/150)}{2 \times 45}$$

≥ 12.2 mm
$\geq 2.03\ ø$
provided $r = 2.75\ ø > 2.03\ ø$ OK

Check shear

$v = 22.25$ kN/m

$$v = \frac{22.25 \times 10^3}{1000 \times 70}$$

$= 0.32$ N/mm^2

$$r_s = \frac{188}{1000 \times 70} \times 100\%$$

$= 0.27\%$

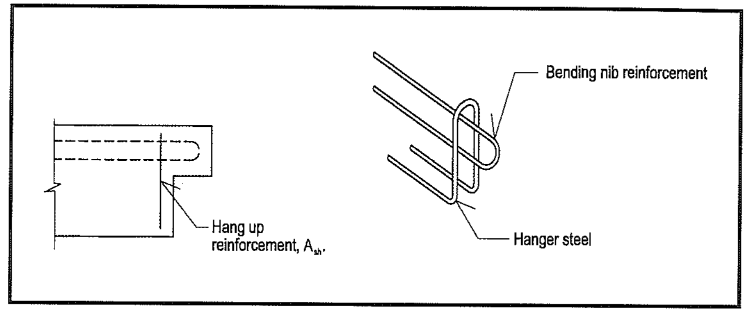
$$v_c = \frac{0.84}{\gamma_m} (r_s)^{1/3} (400/d)^{1/4} (f_{cu}/30)^{1/3}$$

$\gamma_m = 1.25$

$$v_c = \frac{0.84}{1.25} (0.27)^{1/3} (400/70)^{1/4} (45/30)^{1/3}$$

$= 0.77$ N/mm^2 $>$ 0.32 N/mm^2 OK

Check hang-up reinforcement

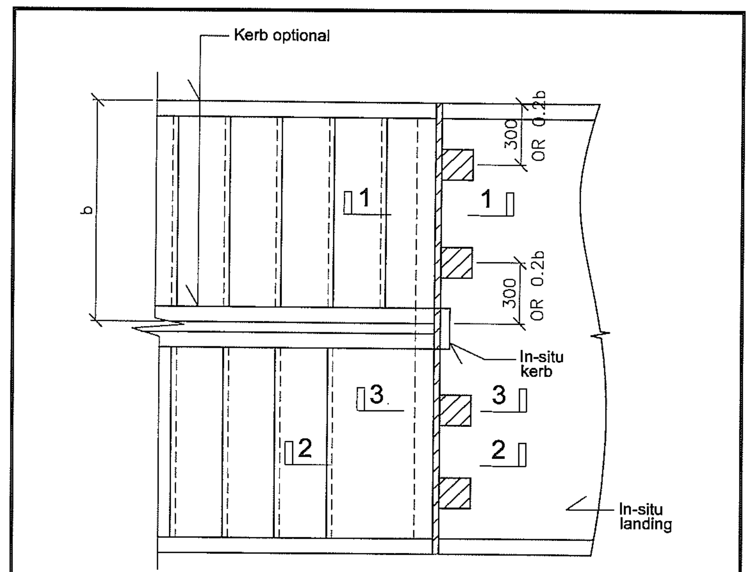


$$A_{sh} = \frac{V}{0.87 f_y}$$

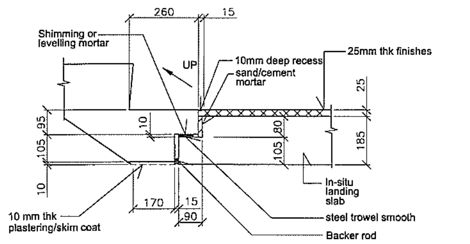
$$= \frac{22.25 \times 10^3}{0.87 \times 250}$$

$= 102$ mm^2/m

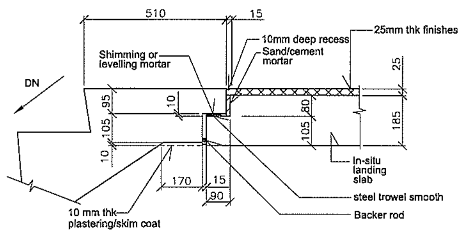
For practical reason, provide at every bending R6 reinforcement a looped R6 as hanger steel as above.



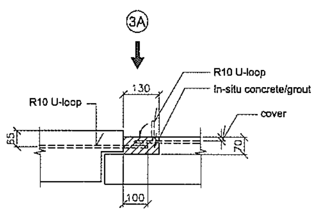
PLAN



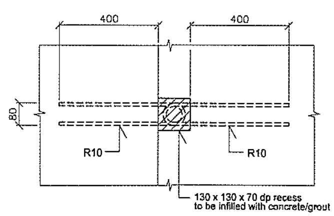
SECTION 1-1



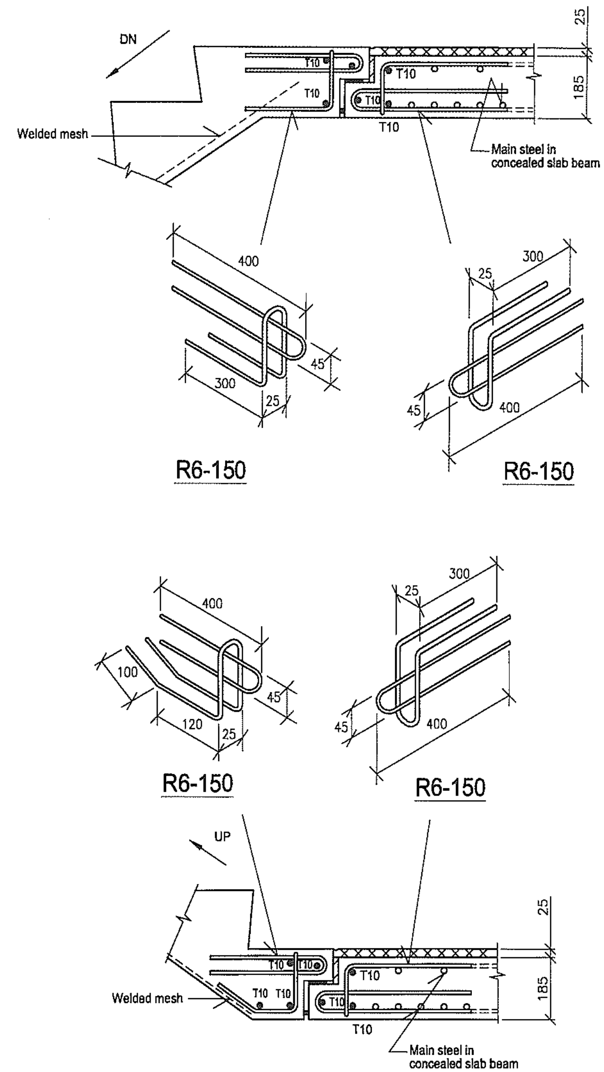
SECTION 2-2



SECTION 3-3



TOP VIEW 3A



NIB REINFORCEMENT DETAILS

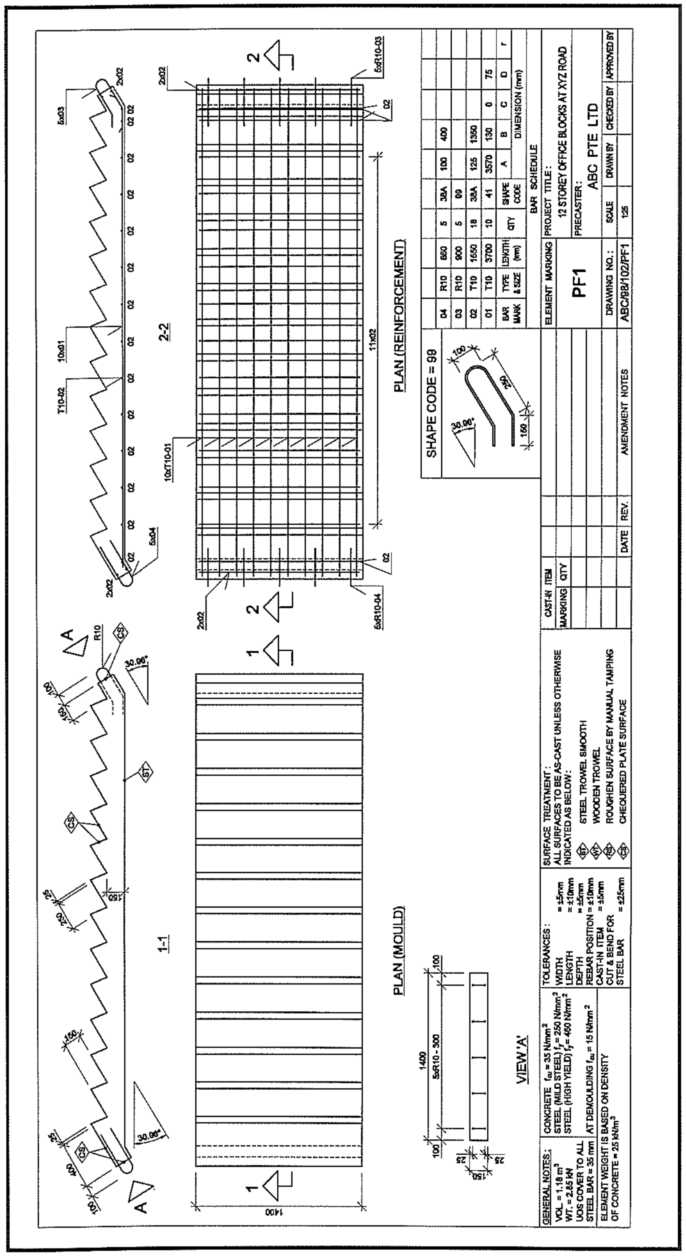


Figure 4.12 Mould And Reinforcement Details Of Precast Flight (12-Storey Office Block)

E. Design Of Structural Integrity Ties

Basic design data : Total number of floor = 12

Average UDL dead load = 485.4/(8 x 8)

= 7.6 kN/m^2

Average UDL live load = 3.0 kN/m^2

Basic tie force F_t = 20 + 4n_o

= 20 + 4 x 12

= 68 kN > 60kN

Hence F_t = 60 kN/m

1. Horizontal ties (Part 1, clause 3.12.3.5,)

a. Periphery ties

Design tie force = 1.0 F_t

= 60 kN

Steel area A_s = 60 x 10^3/460

= 130 mm^2

b. Internal ties

Case 1 : Tie force = 0.0267(g_k + q_k) F_t l_r

= 0.0267(7.6 + 3.0) x 60 x 8.0

= 135.8 kN/m

Case 2 : Tie force = 1.0 F_t

= 60 kN/m

Hence Case 1 governs.

Using steel mesh as floor ties,

A_s = 135.8 x 10^3/485

= 280 mm^2/m

Use D6 mesh (ϕ6 @ 100 c/c both ways, A_s = 283 mm^2/m) within the 65 mm thick topping. This provides automatically the required quantity of floor ties in the other direction as the spans between the bays are similar in both directions.

c. Column ties

Internal columns are tied by D6 mesh in both directions and all corner columns are automatically tied as the adjacent floors are cast in-situ. Hence only edge columns need to be checked.

Case 1 : Tie force = 2.0F_t (l_s F_t/2.5 is less critical)

= 2.0 x 60

= 120 kN

Case 2 : Tie force = 3% of ultimate axial column load

At 2nd storey, ultimate column axial load for edge column is conservatively estimated (allowing for brickwalls) to be :

N = (1.05 x 5768.4 + 0.33 x 1152.0) x 2/3

= 4291.3 kN

Hence tie force = 3 x 4291.3/100

= 128.7 kN

Therefore Case 2 governs the design.

A_s = 128.7 x 10^3/460

= 280 mm^2

= 35 mm^2/m over 8m bay width

The D6 mesh is anchored to the perimeter beams and hence no separate edge column horizontal ties are required.

2. Vertical ties

The vertical ties in the columns have been considered in the design of column joint connection.

4.3 Residential Block

1. Project Description

The project consists of four blocks of 30-storey residential apartments with basement carparks. There are four units on each floor which includes a common passenger lift, fire lifts and two staircases. Floor to floor height is typically at 3.2 m. For design illustration, only a typical floor structural system is presented. It is assumed to carry all lateral loads.

2. Structural System

The structural system shown in a typical layout in Figure 4.13 consists of cast in-situ prestressed flat plates of typically 175 mm thickness. The floor is supported by a series of precast load bearing walls which are generally 200 to 250 mm thick depending on the gravity loads the wall carries. The precast walls are located at the gable ends of each living unit as well as in between rooms. In areas where precast walls are not possible due to architectural reasons, the floor plates are supported by cast in-situ beams of typically 250 x 600 mm deep.The precast walls are braced in two directions by a cast in-situ stabilising lift and staircase cores which are 300 mm thick. They are assumed to carry all lateral loads.

3. Design Information

a. Loading

Finishes	=	1.20 kN/m^2
Live load general	=	1.50 kN/m^2

b. Materials

Concrete : All walls	f_{cu}	=	45 N/mm^2
Steel :	f_y	=	460 N/mm^2

4. Design Of Precast Walls

For design illustration, only W1 in Figure 4.13 is considered. The precast wall is 250 mm thick, 3 m long and is designed as braced structure with pin-connection at the floor levels. The wall is erected with a 20 mm thick horizontal joint which is to be grouted. Vertical ties from foundation to roof are provided at intervals along the wall length.

a. Loading:

Total statical floor area carried by the wall is determined to be about 25 m^2.

Dead load :

Floor s/w	= 0.175 x 24 x 25	=	105.0kN
Beams	= 0.25 x 0.425 x (3 + 1.5 + 1.5) x 24	=	15.3kN
Finishes	= 1.2 x 25	=	30.0kN
Brickwalls (100 mm thick)	= 3.0 x 3.025 x 14 m	=	127.1kN
Precast Wall s/w	= 0.25 x 3 x 3.025 x 24	=	54.5kN
			331.9kN
Live load	= 1.5 x 25	=	37.5kN

Above 1st storey

Total ultimate dead load = 1.4 x 331.9 x 30 = 13939.8 kN

Total ultimate live load with 50% live load reduction = 1.6 x 37.5 x 30 x 0.5
= 900 kN

Total N = 14839.8 kN

Say N = 14840 kN

n_w = 14840/3 = 4947 kN/m

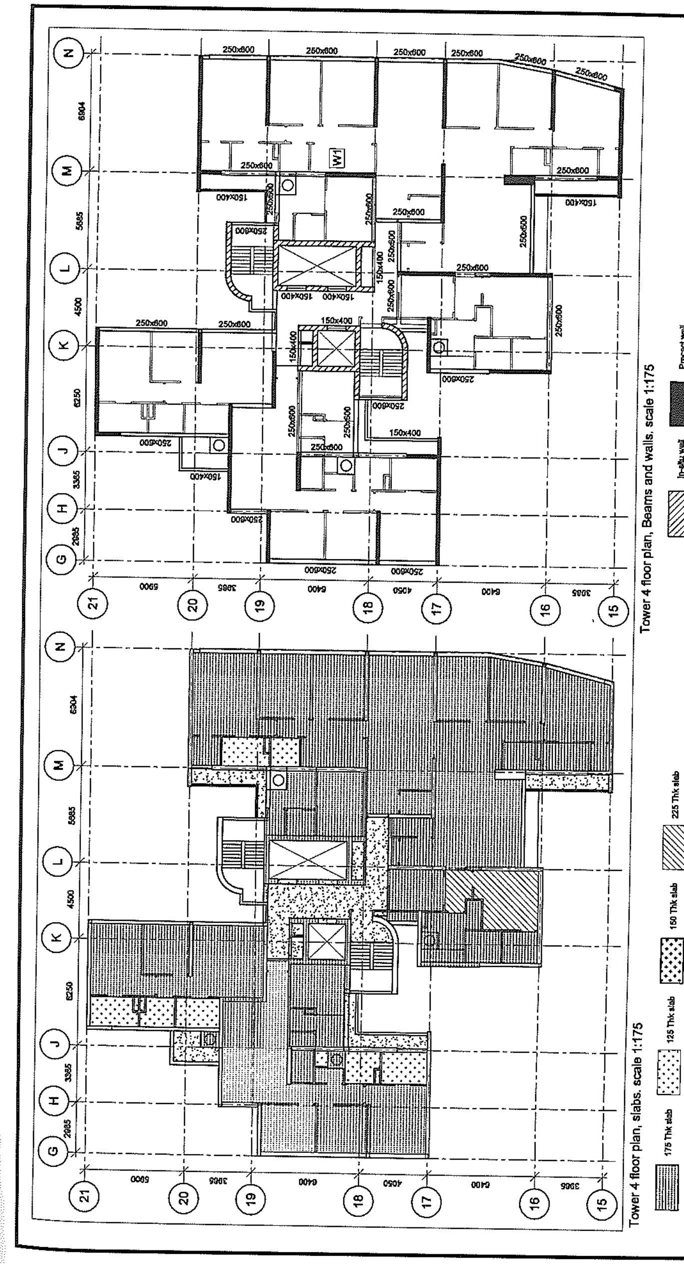


Figure 4.13 Typical Floor Layout

b. **Vertical load capacity**

Effective wall height l_e = $1.0l_o$
= 1.0 x (3.2 – 0.175)
= 3.025 m

Slenderness ratio l_e/h = 3025/250
= 12.1

The wall is considered a short wall under Part 1, clause 3.8.1.3. CP65.
From equation 42 (Part 1, clause 3.9.3.6.1) of the Code, the vertical load capacity is given as :
$n_w \leq 0.35f_{cu}A_c + 0.67A_{sc}f_y$

n_w = 4947 kN/m

A_{sc} = (4947 x 10^3 - 0.35 x 45 x 1000 x 250) / (0.67 x 460)
= 3275 mm^2/m

Per face = 1638 mm^2/m
Use T16 -125 c/c per face (A_s = 1675mm^2/m)

c. **Vertical structural ties**

Vertical tie force per floor = 1.05 x 331.9 + 0.33 x 37.5
= 360.9 kN

Tension steel area A_s = 360.9 x 10^3/(460)
= 785 mm^2

Use 3 numbers of T20 (ΣA_s = 943 mm^2) with one at each end of the wall and one number at the middle section

d. **Horizontal joint capacity**

Joint thickness t_w = 20 mm
Permissible concrete compressive stress (Part 1, clause 5.2.3.4) = $0.6f_{cu}$
Net area for vertical load transfer (Part 1, clause 5.3.6 of the Code) = 75%A_c

n_w = 4947 kN/m and
$n_w = 0.6f_{cu} \times 0.75A_c$

Minimum concrete strength required for the joint infill :

f_{cu} = 4947 x 10^3 / (0.6 x 0.75 x 1000 x 250)
= 44 N/mm^2

Use f_{cu} = 45 N/mm^2 as joint infill

e. **Connection details**

Some typical connection details of the wall W1 and floor are shown in Figure 4.14 to Figure 4.16.

f. **Production details**

Typical production details for the precast wall W1 are shown in Figure 4.17.

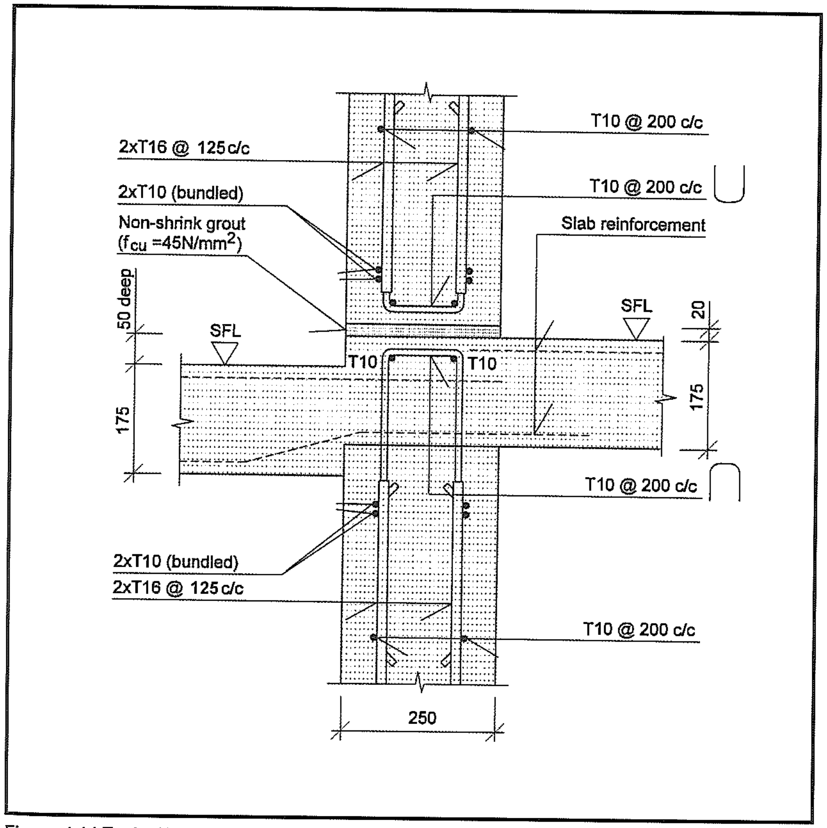


Figure 4.14 Typical Wall / Floor Joint

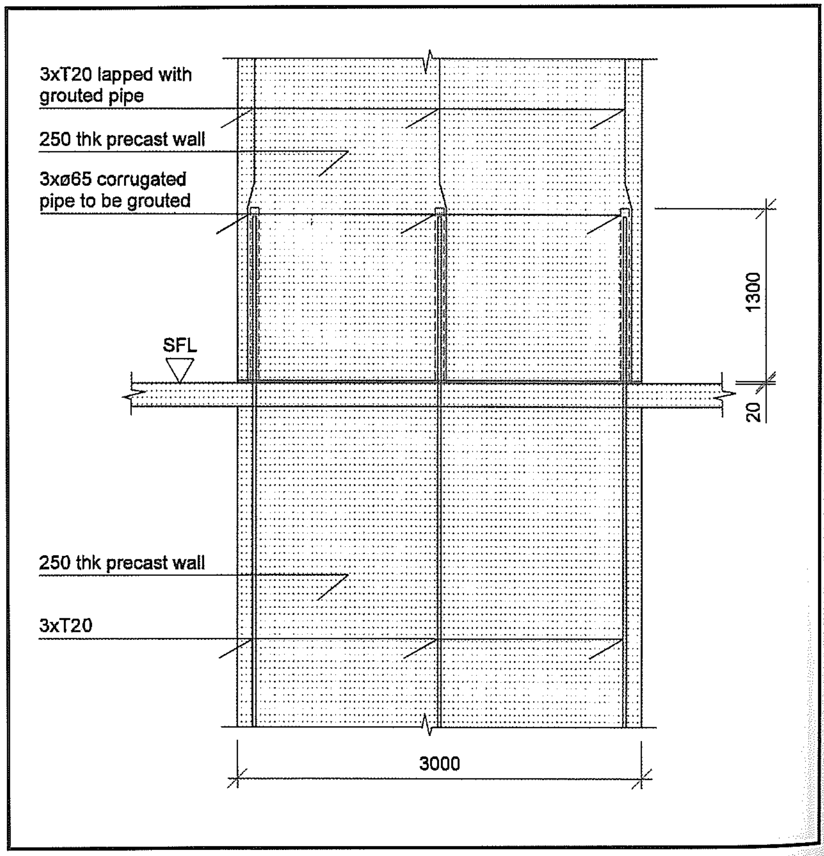


Figure 4.15 Vertical Ties

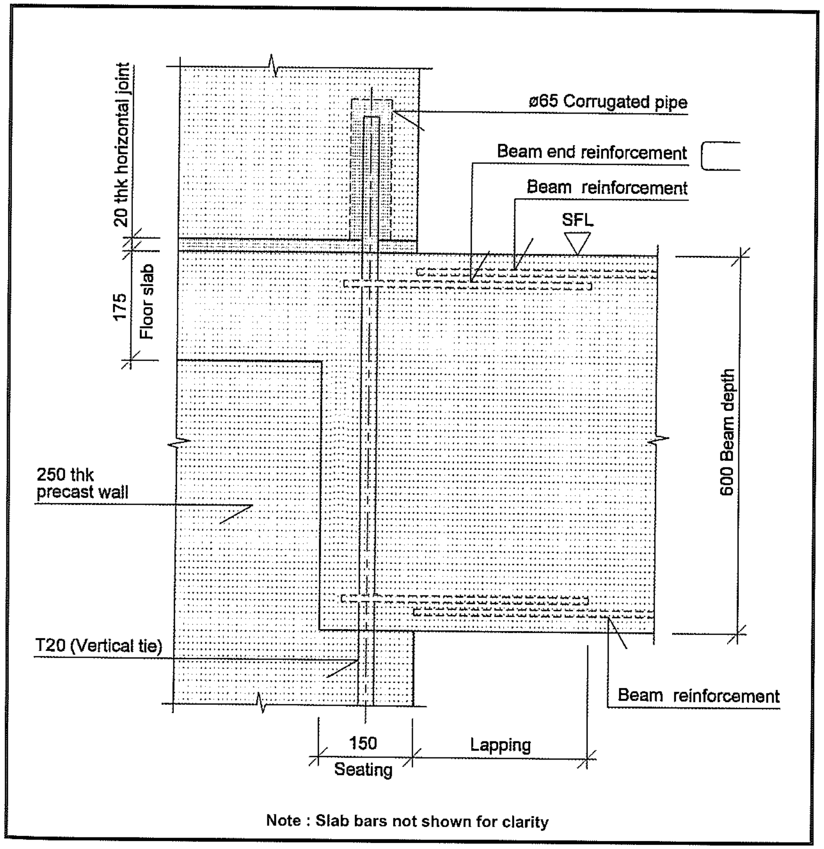


Figure 4.16 Typical Wall/Beam Joint

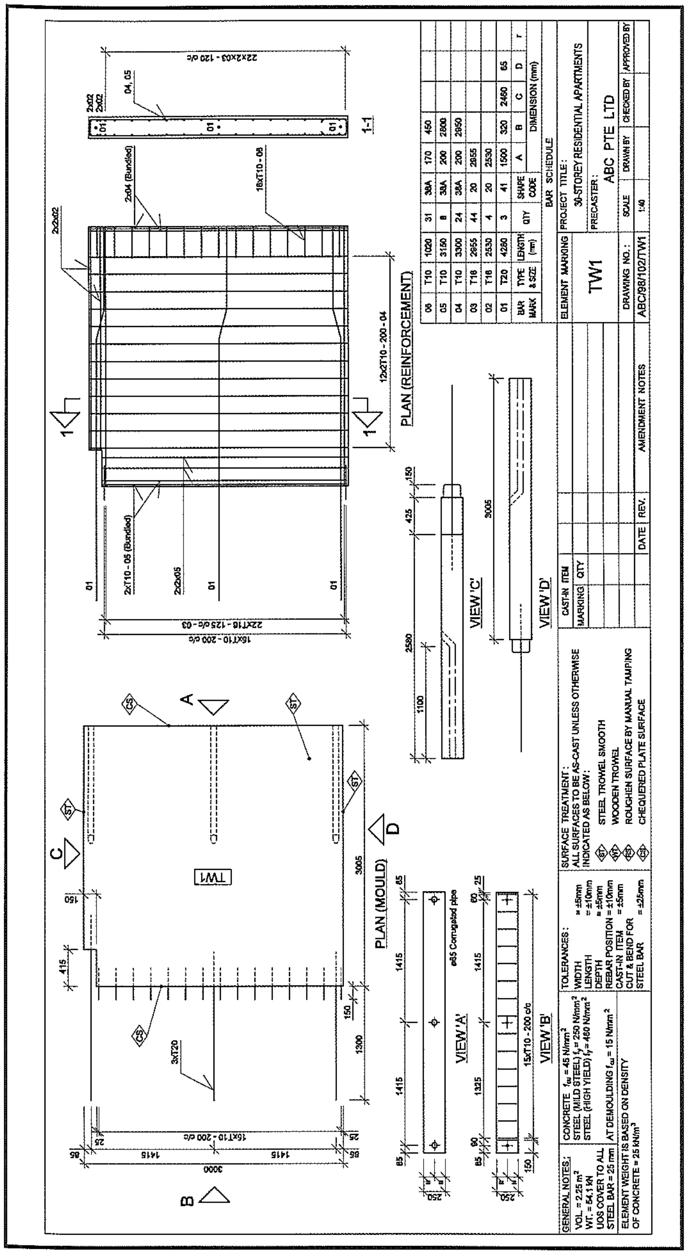
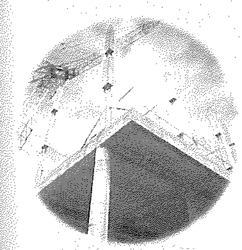


Figure 4.17 Mould And Reinforcement Details Of Precast Wall (Residential Block)



CHAPTER 5

PRECAST CONCRETE ASSEMBLY

Chapter 5 PRECAST CONCRETE ASSEMBLY

5.1 General

A complete statical evaluation of prefabricated buildings includes the following phases:
1. load assessments
2. formulation of a model of the main structural system
3. description of load paths
4. design of structural members
5. design of connections

Construction drawings for a prefabricated building project will normally consist of architectural sketches, modular details, general modular drawings, production drawings and assembly details.

For designers not familiar with prefabicated building designs, there will be considerable confusion with the selection of the appropriate methods of jointing and connections of precast components. With the exception of some common ones, the development of connection details is often on a project to project basis. Even the same details may be subjected to various modifications as dictated by the different structural and architectural requirements.

Despite the different manners in which a connection detail may be executed, the type of joint details must fulfil the following requirements:
1. compatibility with adjoining structural members
2. economy of materials
3. manufacturing and erection techniques
4. ease of assembly and speed of construction
5. final aesthetic joint appearances

There are basically three different types of connections used in precast concrete structures, namely, the cast in-site joint including grouted pipe sleeves, mechanical joint by bolting or welding, or a combination of both. Generally, the cast in-site type is the cheapest and it fits well into the craftsmanship already available on the building site. Bolted connections are easy to execute with both skilled and unskilled labour, whereas welding operations call for skilled labour or specialists. In terms of execution, bolted and welded joints normally need closer supervision and higher quality control.

This chapter presents an overview of the various jointings and connections of precast components which may assist the designer and architect in the design of prefabricated buildings. The details represent a cross-section of some of the proven techniques used locally in residential buildings, public housing, commercial and industrial buildings. As the local construction industry is relatively new to precast construction techniques, a selection of connection details from overseas are also included to provide readers with a wider spectrum of the connection techniques.

The details should be used a guide instead of as a design manual. It must be emphasised that the responsibility for a proper connection fulfilling all relevant performance requirements rests with the structural designer and the architect working often in close consultation with the precaster and contractor in an integrated approach during the design, development, fabrication and construction of the precast buildings.

5.2 Joints Used In Singapore Projects

The table below shows a compilation of connection and jointing details commonly used in Singapore. These details have been used in various projects such as private residential buildings, public housing, commercial and industrial buildings.

Types Of Joints Used Locally	Column	Beam	Slab/Plank	Wall	Stump	Precast Panel/Façade	Staircase
Column	5.9, 5.11, 5.12	5.9, 5.11, 5.12		5.17, 5.18	5.9, 5.10		
Beam			5.1, 5.3, 5.4, 5.5, 5.6, 5.8, 5.14	5.20			
Slab/Plank			5.1, 5.2	5.7, 5.8, 5.15, 5.16, 5.20		5.7, 5.16	5.27, 5.28, 5.29, 5.30, 5.31
Wall				5.21, 5.22, 5.23, 5.24, 5.25, 5.26			

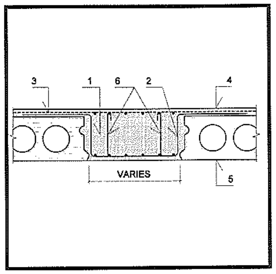


Figure 5.1 Tie Strip Between Hollow Core Slabs

1. Tie beam
2. Links
3. Steel mesh
4. Concrete topping
5. Hollow core slab
6. Shear links (if structurally required)

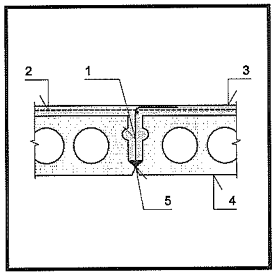


Figure 5.2 Hollow Core Slab to Slab Joint Detail

1. L shape T10 or R10 at 600c/c
2. Steel mesh
3. Concrete topping
4. Hollow core slab
5. Foam or cement/sand block-out

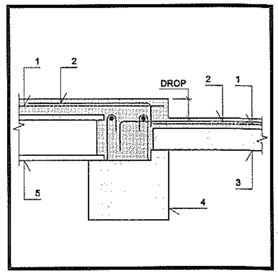


Figure 5.3 Hollow Core Slab / Beam/ Plank Connections With Drop

1. Steel mesh
2. Reinforcement to engineer's design
3. Plank
4. Beam
5. Hollow core slab

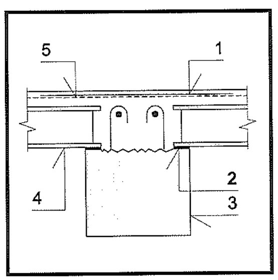


Figure 5.4 Hollow Core Slab Seating

1. Reinforcement to engineer's design
2. Neoprene strip
3. PC/RC beam
4. Hollow core slab
5. Steel mesh

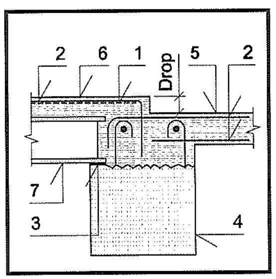


Figure 5.5 Beam/Hollow Core Slab/In-Situ Slab

1. Steel mesh
2. Reinforcement to engineer's design
3. Neoprene strip
4. PC/RC beam
5. RC slab
6. Concrete topping
7. Hollow core slab

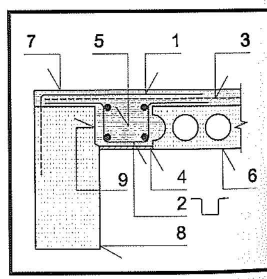


Figure 5.6 In-Situ Strip In Hollow Core Slab With Beam

1. Reinforcement to engineer's design
2. Shear links
3. Steel mesh
4. Plastering
5. Reinforcement in pour strip
6. Hollow core slab
7. Concrete topping
8. PC/RC beam
9. 20mm recess in beam lab

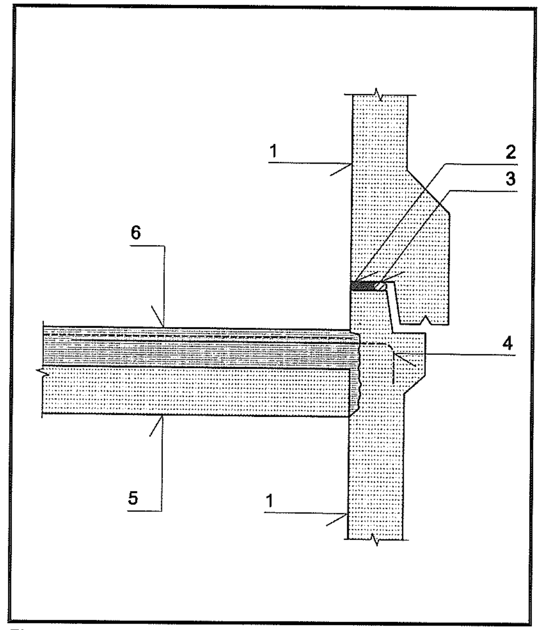


Figure 5.7 Plank And External PC Wall Panel

1. PC external wall
2. 20mm dry pack under external wall
3. Backer rod
4. Reinforcement (from PC wall)
5. Plank
6. In-situ topping

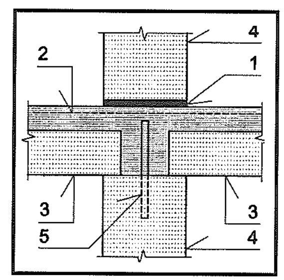


Figure 5.8 Plank And Wall

1. Sand/cement mortar or grouting
2. Steel mesh
3. Plank
4. PC beam / wall
5. Dowel bars

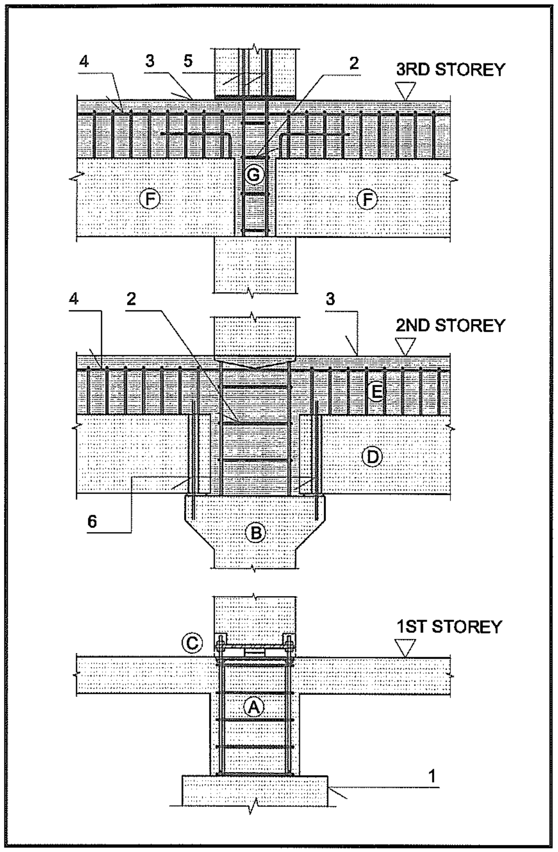


Figure 5.9 Illustration Example For Column/Stump, Column/ Column And Column/Beam Connection

1. Pilecap
2. Links loop round steel bars
3. Cast in-situ concrete
4. In-situ beam bars
5. Column steel bars to extend above floor for grouted pipe sleeve column connection
6. Steel dowel for site safety

Stages of Construction

A. Construct foundation and 1st storey RC work
B. Erect precast columns
C. Grouting of column base
D. Launch precast beams and floor slab elements
E. Cast in-situ RC work
F. Launch precast beams and floor slab elements
G. Cast in-situ RC work

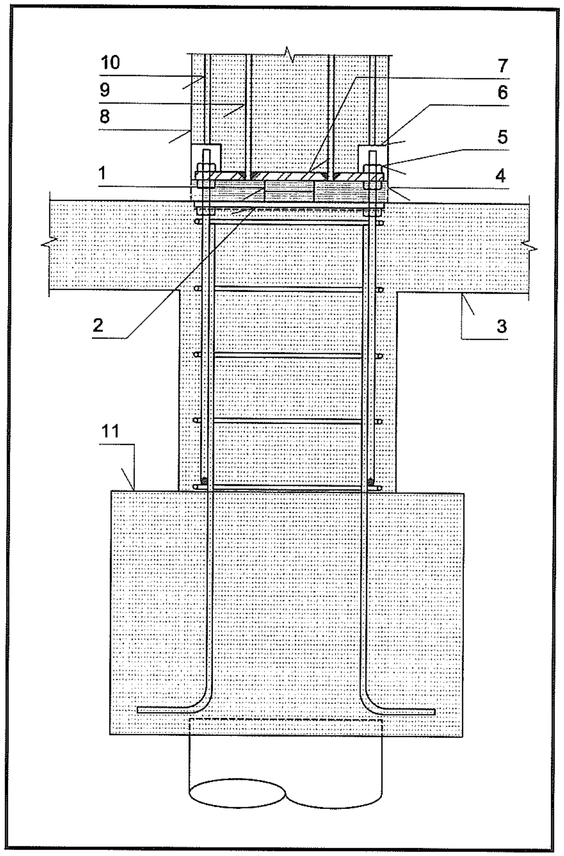


Figure 5.10 Bolted Column Base Connection

1. The gap is to be formed by seating precast column onto shimming plates (with the adjusting bolts screwed down to the lowest position).
2. Template to position bolts
3. Ground slab
4. Grouting
5. Nuts and washer plates
6. Base angle
7. Base plate (recessed from column faces)
8. Precast column
9. Bars welded to base plate
10. Column bars
11. Pilecap

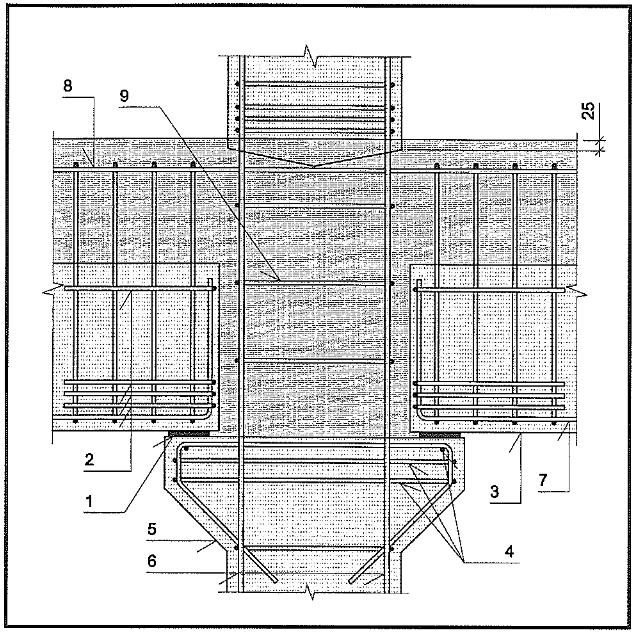


Figure 5.11 Column/Beam Connection With Corbel

1. Neoprene strip
2. Horizontal loop at beam ends
3. PC beam
4. Corbel reinforcement to engineer's design
5. PC column
6. Column bars
7. Bend-up beam reinforcement
8. In-situ reinforcement
9. Column links

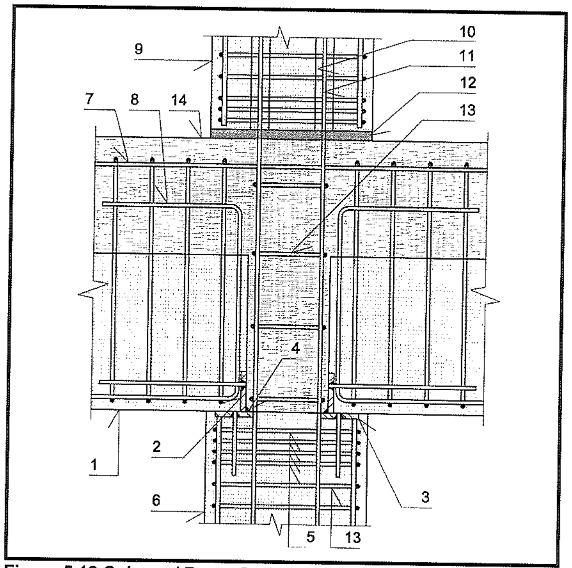


Figure 5.12 Column / Beam Connection Without Corbel

1. Precast beam
2. Cast-in end plate with horizontal welded bars in PC beam
3. Cast-in steel plate with vertical welded bars at column face
4. Site welding to prevent beam toppling
5. Shear friction steel
6. Precast column
7. In-situ placed reinforcement
8. Bend up beam reinforcement
9. Upper floor precast column
10. Pipe sleeve
11. Starter column bars from lower precast column
12. 25 thick non-shrink grout
13. Column links
14. In-situ concrete

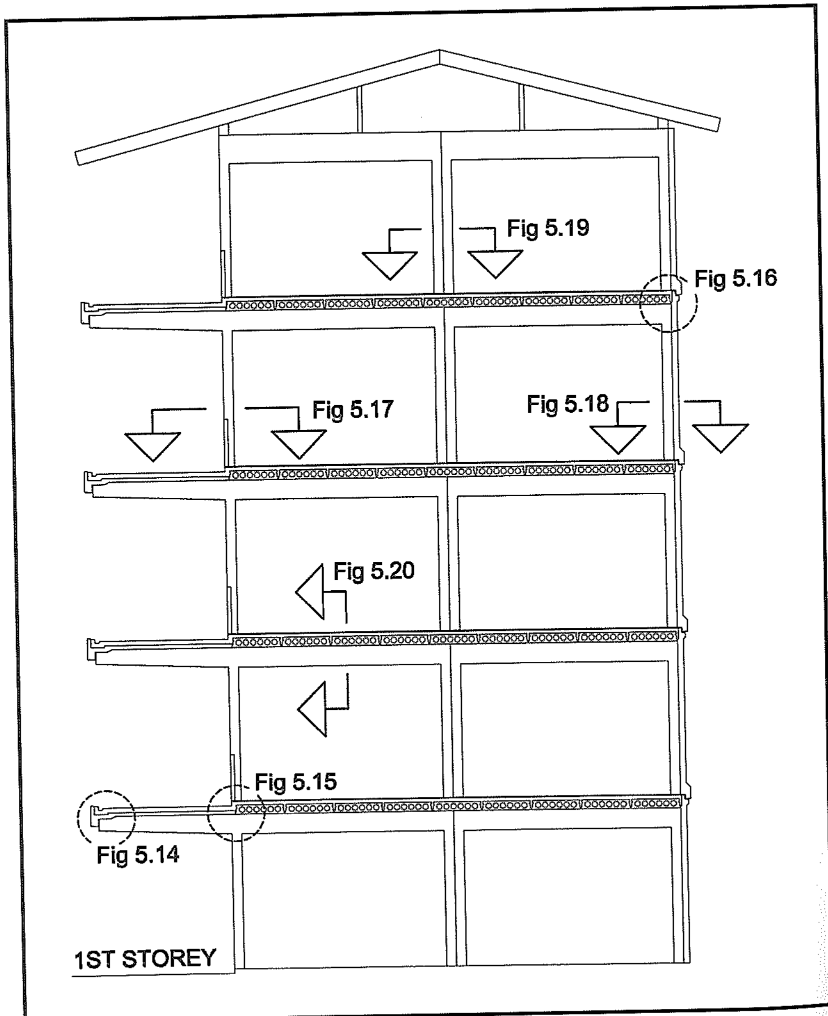
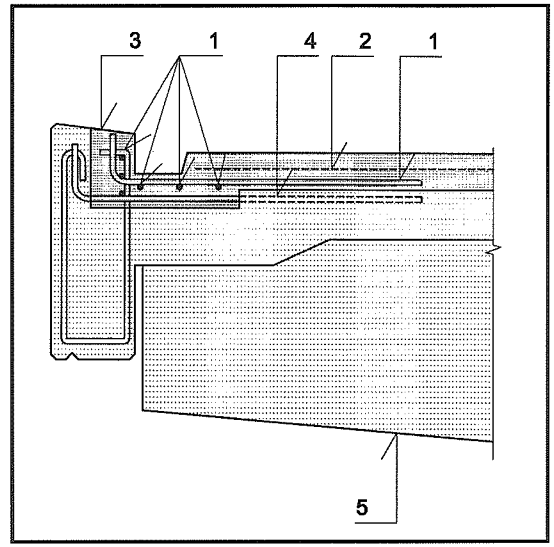


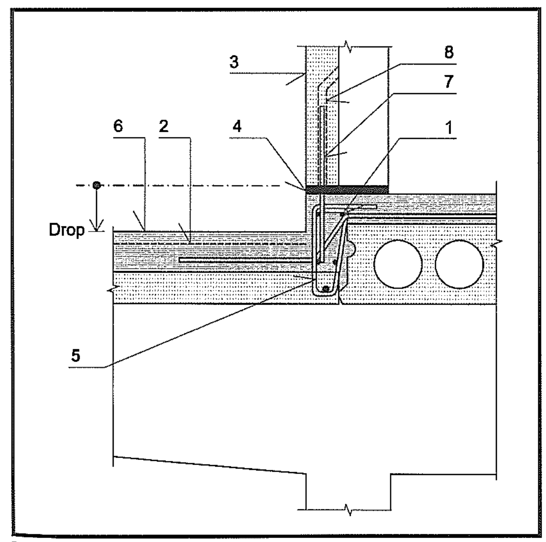
Figure 5.13 Typical Section Of A Precast Building (Section And Details To Refer To Figures 5.14 To 5.20)

This is a typical section of a precast concrete structure for a dormitory. The precast members consist of single storey high column and beam with integrated partition walls. A cantilever beam is incorporated in the outer frame at the corridor which is column free. Hollow core slabs are used as internal floor structure and profiled precast planks incorporating perimeter beams are used at the corridor. All internal and external walls are precast with architectural features of window sills, copings, drips etc. The precast components are simple pin-connected both vertically and horizontally with either cement/sand mortar or grouted joints. An in-situ 75mm thick concrete topping throughout the floor will bind all members together to provide the necessary structural action to resist horizontal forces.



1. Reinforcement added on site
2. Steel mesh
3. Cast in-situ concrete
4. Rebar from precast plank
5. Cantilever beam from precast frame

Figure 5.14 Perimeter Beam And Plank With Surface Scupper Drain



1. Reinforcement added on site
2. Steel mesh
3. Precast wall
4. Drypack cement mortar
5. Reinforcement to engineer's design
6. Concrete topping (75mm - 100mm)
7. Dowel bar
8. Corrugated pipe sleeve (to be grouted)

Figure 5.15 Internal Floor To Corridor And Partition Wall Connection To Floor

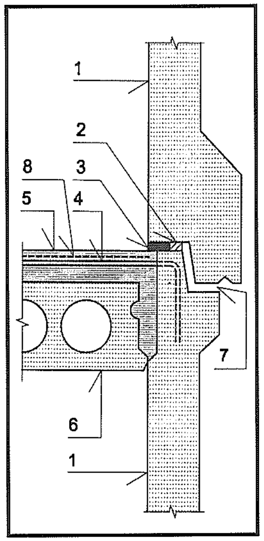


Figure 5.16 Floor To External Wall Connection

Figure 5.16
1. Precast wall
2. Backer rod
3. Drypack cement mortar
4. Reinforcement
5. Concrete topping
6. Precast hollow core slab
7. Open horizontal joint
8. Steel mesh

Figure 5.17
1. Precast wall
2. Cottering reinforcement (placed on site)
3. Precast column
4. Reinforcement to engineer's design
5. Plastering
6. Column and wall joint to be cast in-situ
7. Recess in wall for plastering

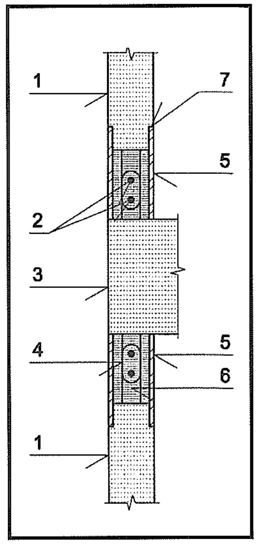


Figure 5.17 Column To Wall Connection

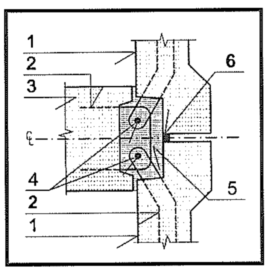


Figure 5.18 Column/External Wall Connection

1. Precast wall
2. Reinforcement to engineer's design
3. Precast column
4. Cottering reinforcement (placed on site)
5. Column and wall joint to be cast in-situ
6. Backer rod

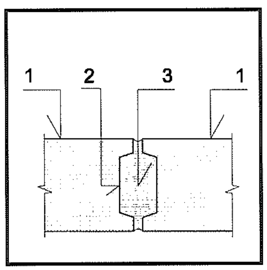


Figure 5.19 Column To Column Vertical Joint

1. Column
2. Vertical castellations
3. Grouting

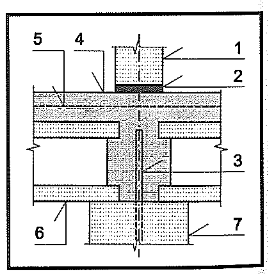


Figure 5.20 Hollow Core Slabs Beam/Wall Connection

1. Precast wall
2. Drypack cement mortar
3. Dowel reinforcement to engineer's design
4. Concrete topping
5. Steel mesh
6. Hollow core slab
7. Precast beam

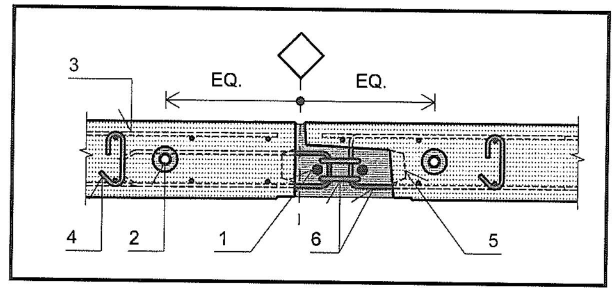
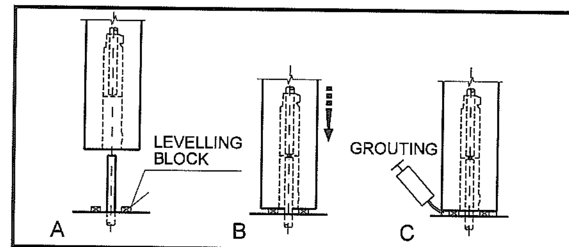


Figure 5.21 Wall-Wall Cast In-Situ Vertical Joint

1. Joint rebar
2. Pipe sleeve
3. Steel mesh
4. Space bar
5. Edge castellation
6. Reinforcement



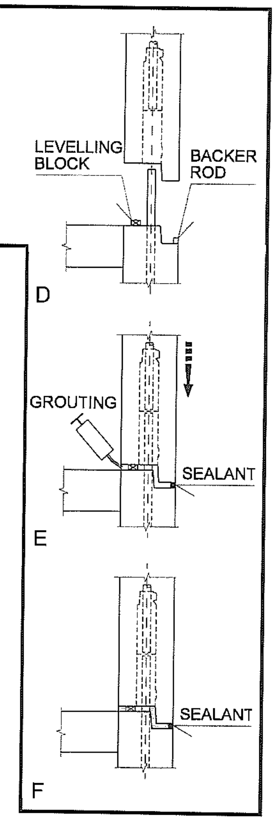


Figure 5.22 Installation Of Wall With Spliced Sleeve Connection

Procedures for connecting internal walls using pipe sleeves

A. Levelling blocks are placed
B. Lower the upper member into position
C. Grouting of horizontal joint

Procedures for connecting external walls using pipe sleeves

D. Levelling block is placed
E. Lower the upper member into position
F. Grouting of horizontal joint

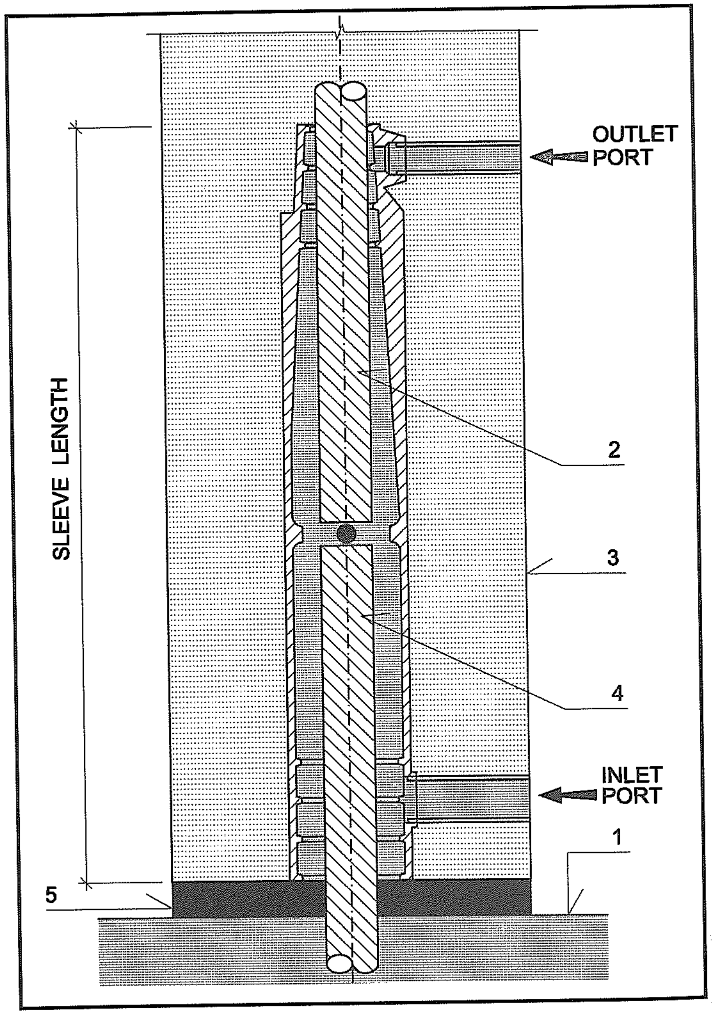


Figure 5.23 Spliced Sleeve Connection In Walls

1. Cast in-situ slab
2. Rebar embedment; dowel installed in factory
3. PC Wall
4. Rebar embedment; dowel installed in factory
5. Grouting

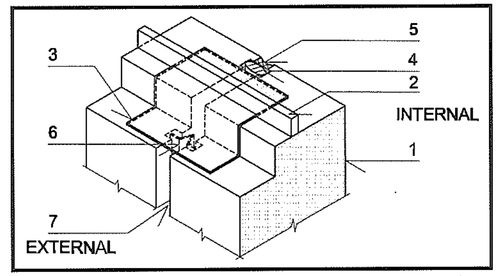


Figure 5.24 Joint Details Of Abutting External PC Wall

1. PC wall panel
2. Watertightness sealing strip
3. Aluminium or stainless steel flashing
4. Backing
5. Sealant or mortar
6. Baffle strip
7. Design gap

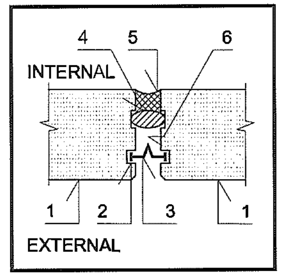


Figure 5.25 Open Drained Joint

1. PC wall panel
2. Recess profile for baffle strip
3. Baffle strip
4. Backing
5. Sealant or mortar
6. Drained and ventilated air space

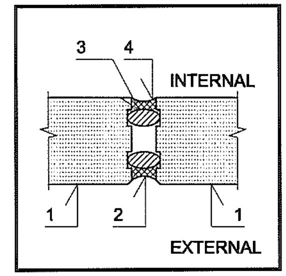


Figure 5.26 Closed Joint

The close and open joints have both been successfully used to ensure watertightness in the joints of precast wall panels.

The close joint adopts a single stage sealing using either polysulphide mastic or silicone sealants to prevent the penetration of air and water. The open joint has a 2-stage sealing feature namely a front protection baffle and a rear air-tight seal. Water penetrating the front baffle will be drained away in the airspace between the 2 seals.

Most failures in close joints involve aging, loss of surface adherence or elasticity of the sealant material. For good performance, it is necessary, therefore, to note the followings:

1. good workmanship preferably by specialists
2. good sealant material with correct proportioning or resin and inert materials
3. sufficient joint dimensions with enough width and depth for proper joint filling
4. regular and plain joint edges to ensure good surface adherence
5. seal should be set back in the joint for protection from weather elements

The watertightness of open joint depends critically upon the effectiveness of the air-tight seal at the rear of the joint. Damages to the seal may be due to the movement of the wall panels, loss of adherence of air-tight sealant or dropping off of the mortar or grouting. The loss of front baffle may arise from inadequate end stoppages which cause the baffle to fall out of the joint or from simple vandalism. Compared with close joint, the placement of open joint sealing features is carried out during the erection of the panels. It is thus necessary to check that the insert grooves for baffle in the adjoining panels run relatively parallel to each other at the joint. A further disadvantage of the open joint is that it is difficult to inspect and maintain the joint particularly at the intersections of the horizontal and vertical joints. Repair of leaking joints is also difficult. Despite these shortcomings, the open joints are less exposed to the weather than close joint and movement of the panels does not generally affect the effectiveness of the joints. Most open joints are found to give little trouble in service.

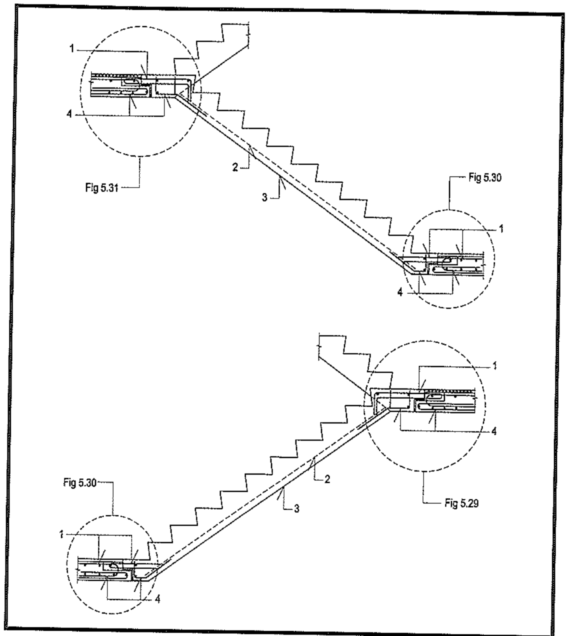


Figure 5.27 Typical Staircase Sections

1. Nib reinforcement
2. Welded wire
3. Precast staircase
4. Hanger steel

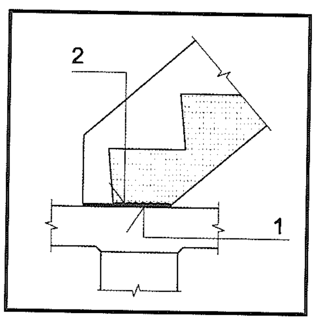


Figure 5.28 Floor Slab & Staircase Flight

1. Cement mortar (10 - 15mm thick)
2. Roughened contact faces

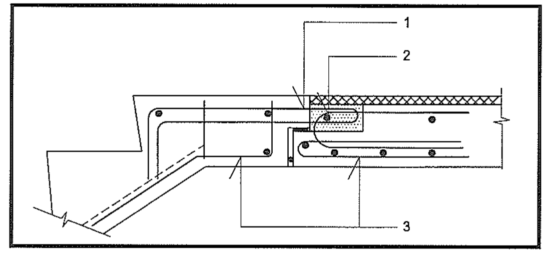


Figure 5.29 Landing Slab And Staircase Flight

1. Nib reinforcement
2. T13 cottering bar
3. Hanger steel

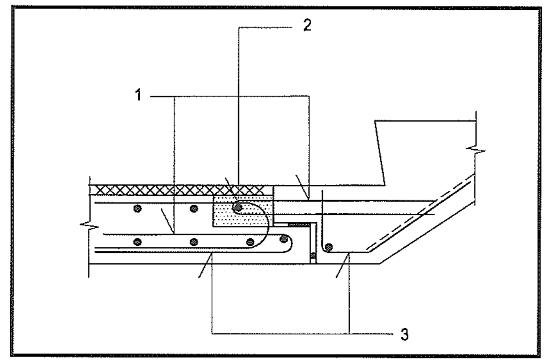


Figure 5.30 Landing Slab And Staircase Flight

1. Nib reinforcement
2. T13 cottering bar
3. Hanger steel

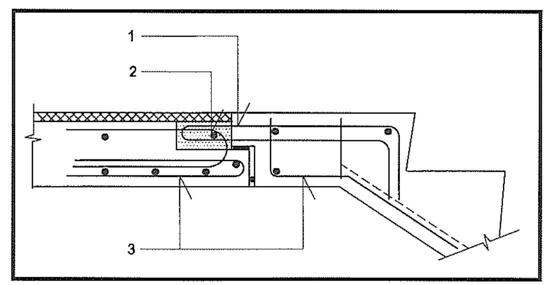


Figure 5.31 Landing Slab And Staircase Flight

1. Nib reinforcement
2. T13 cottering bar
3. Hanger steel

5.3 Joints Used In European Countries

The details indicated in the table below cover the most commonly adopted structural joints in European countries. For external joints in facades, the two-step open joint principle is chosen.

Types of Joints Used In European Countries	Column	Beam	HC Slab	Plank	Double T	Load Bearing Wall	Cladding	Panel/Façade	Frame	Balcony Slab	Balcony Parapet	Staircase	Column Socket
Column	5.32 to 5.37	5.38 to 5.45		5.115			5.116 to 5.117	5.118		5.119			5.135 to 5.138
Beam		5.46 to 5.49	5.50 to 5.54	5.55 to 5.56	5.57 to 5.59	5.60 to 5.62	5.63 to 5.66	5.67 to 5.69	5.101 to 5.103	5.120		5.131 to 5.134	
HC Slab			5.70 to 5.72	5.73 to 5.75	5.76 to 5.77	5.78 to 5.79	5.80	5.81	5.104 to 5.106	5.121 to 5.122			
Plank				5.82 to 5.83	5.84	5.85	5.86	5.87	5.107	5.125			
Double T					5.88	5.89	5.90	5.91	5.108				
Load Bearing Wall						5.92 to 5.95	5.96 to 5.97	5.98	5.109	5.123 to 5.124, 5.126		5.129 to 5.130	
Cladding							5.99		5.110 to 5.111				
Façade								5.100	5.112				
Frame									5.113 to 5.114				
Baloony Slab										5.127	5.128		

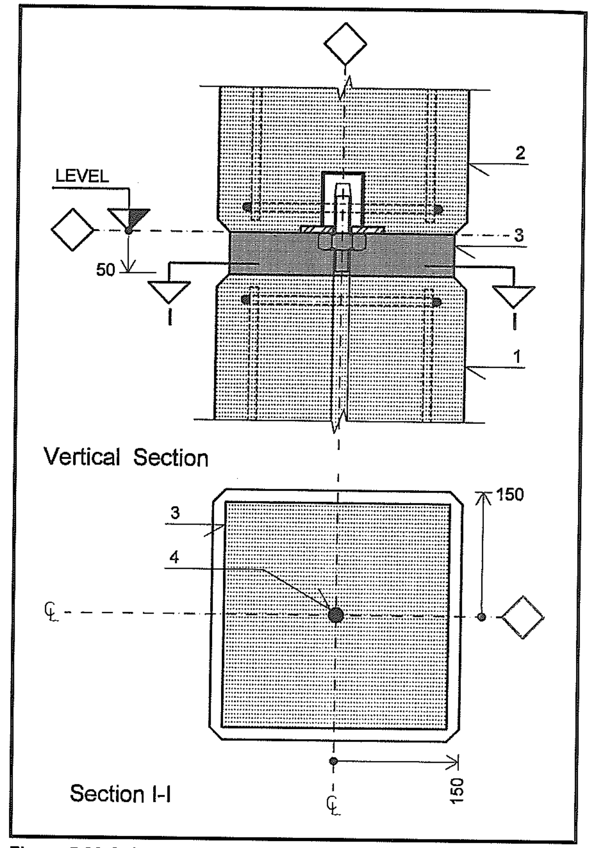
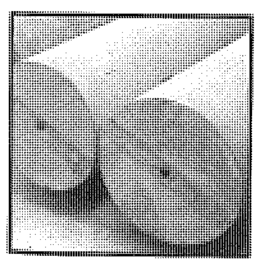


Figure 5.32 Column/Column Dowel Connection



1. Column with cast-in assembly bolt at the top
2. Column with cast-in steel plate and recess at the bottom
3. Grouting with cement mortar
4. Assembly bolt with nut

The simply supported column is temporarily placed on the assembly bolt. The grout will transmit the compression force.

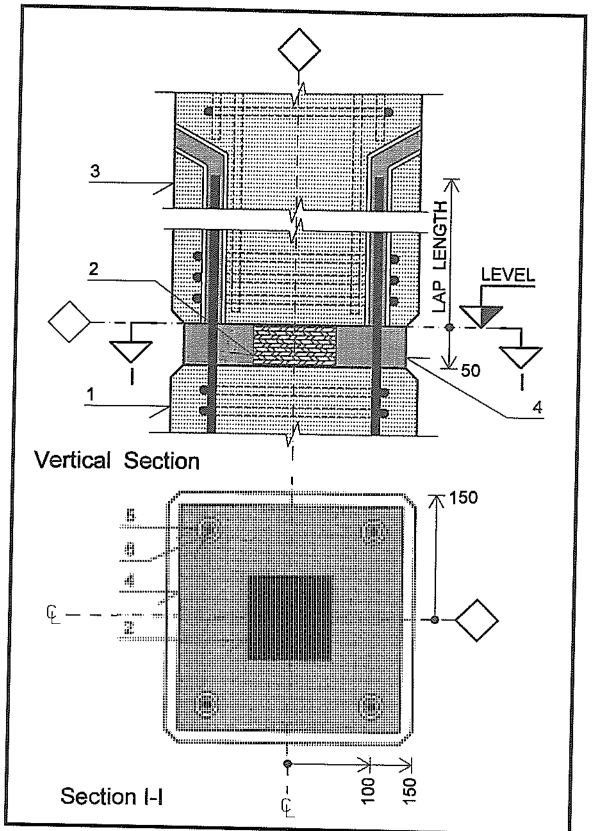
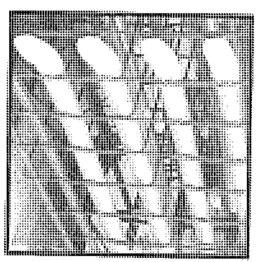


Figure 5.33 Column/Column Connection By Grouted Pipe Sleeves



1. Column with protruding rebars at the top
2. Support shims
3. Column with corrugated pipes at the bottom
4. Grouting with cement mortar
5. Corrugated pipe
6. Protruding reinforcement bar

The column is temporarily supported by the shims. After finishing the joint, the column can be considered as restrained.

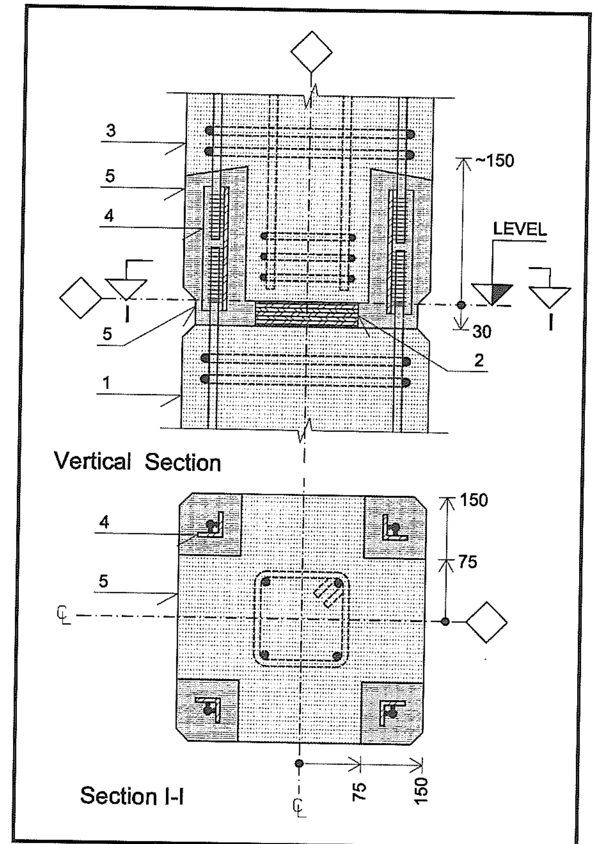
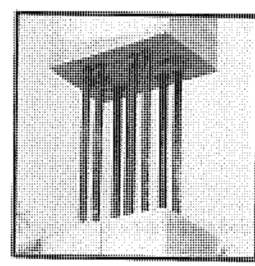


Figure 5.34 Column/Column Connection By Welded Angles



1. Column with protruding rebars at the top
2. Support shims
3. Column with protruding rebars at the bottom
4. Steel angle welded to rebars protruding from columns
5. Grouting with cement mortar

As the main reinforcement bars are welded together, the column acts as fixed-ended.

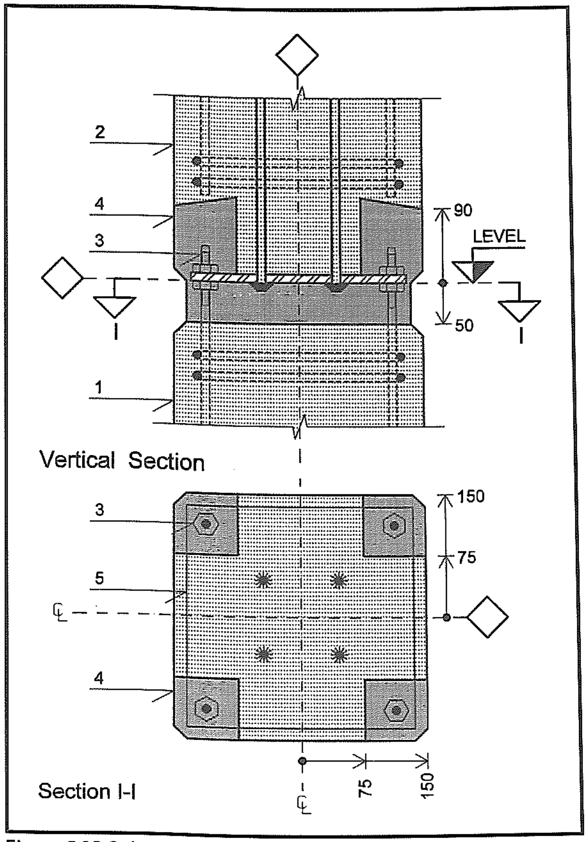
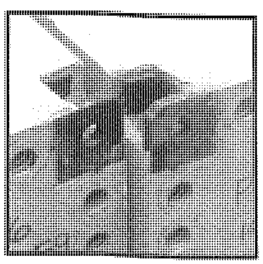


Figure 5.35 Column/Column Connection By Bolting



1. Column with assembly bolts at the top
2. Column with steel plate at the bottom
3. Bolts fastened to steel plate
4. Grouting with cement mortar
5. Steel plate welded to column bars

The four assembling bolts and the embedded steel plate form a fixed-ended column.

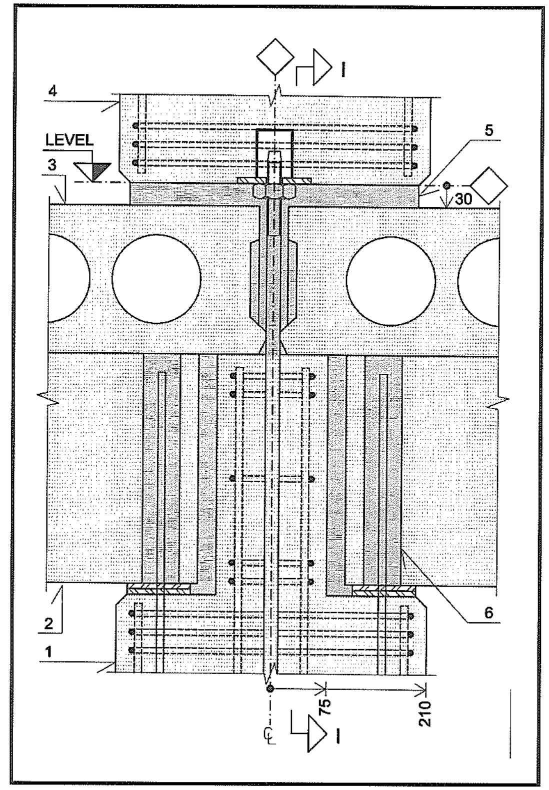


Figure 5.36 Column/Beam/Floor Pinned Connection

1. Column with assembly bolt at the top
2. Precast beam
3. Hollow core slab
4. Column with steel plate support and recess
5. Grouting with cement mortar
6. Dowel-hole connection

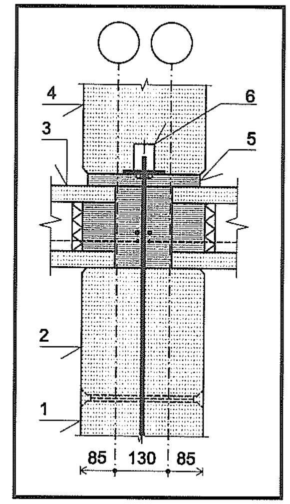


Figure 5.37 Section I-I

1. Column with assembly bolt
2. Beam component
3. Hollow core slab
4. Column in the next storey
5. Grouting
6. Recess for assembly bolt

Section I-I shows vertical forces being transmitted via the cast in-situ zone.

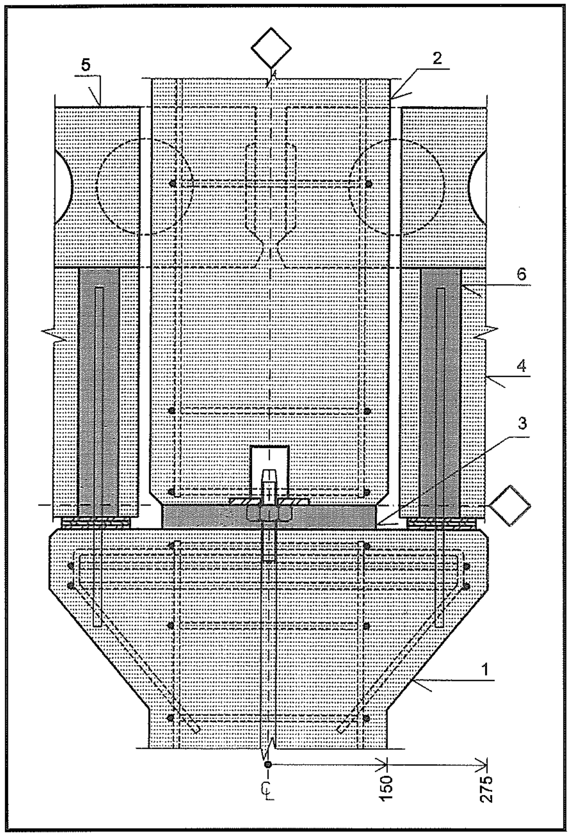
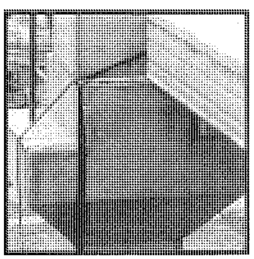


Figure 5.38 Column/Column Connection With Beams Onto Corbel



1. Column with corbels and assembly bolt
2. Column with steel plate and recess
3. Grouting with cement mortar
4. Precast beam
5. Hollow core slab
6. Dowel-hole connection

The assembly detail shows a normal column-to-column joint and two simply supported beams.

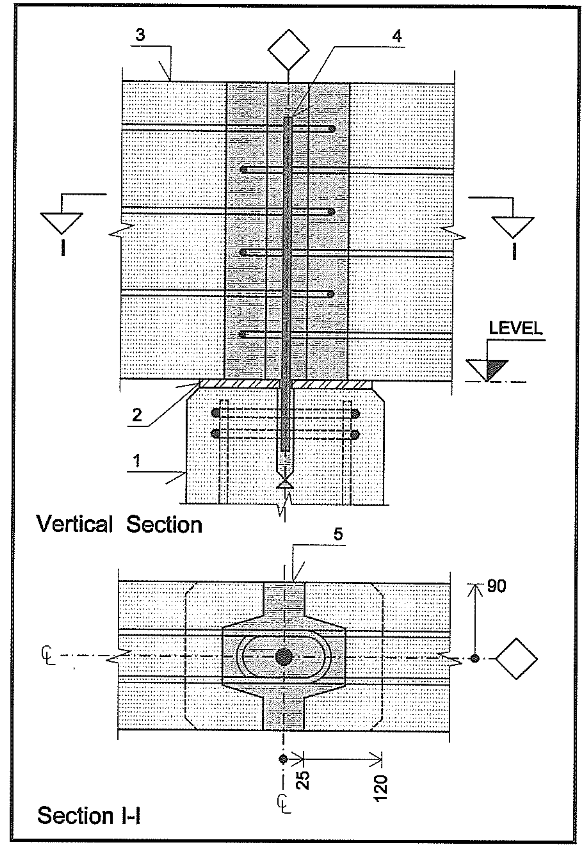
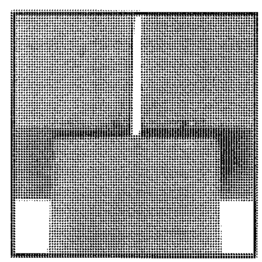


Figure 5.39 Column/Beam Connection With In-Situ Jointing And Looped Bars



1. Column with anchor insert at the top
2. Steel plate as support
3. Precast beam with protruding stirrups
4. Dowel screwed into insert
5. In-situ concrete or grouting

The two beams are restrained by the use of protruding stirrups and locking dowel.

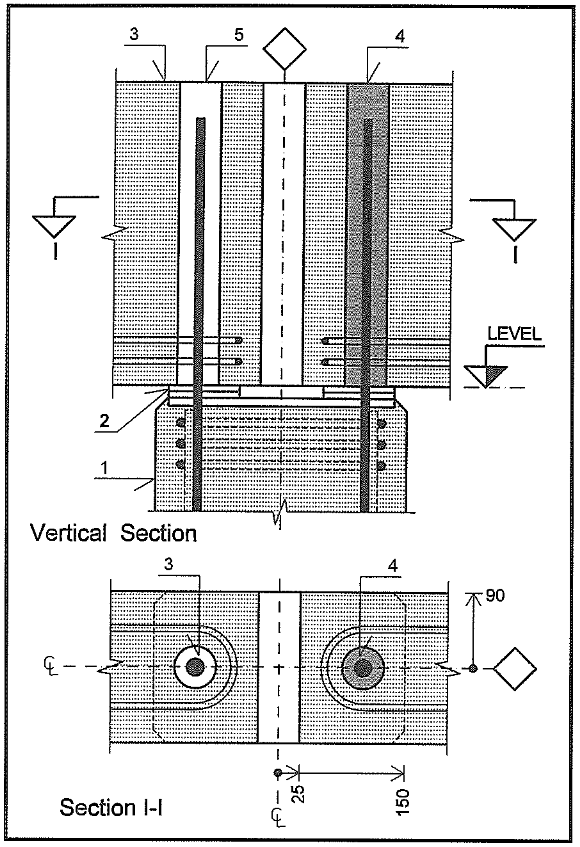
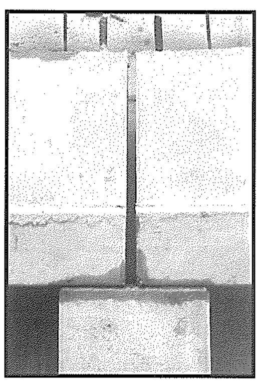


Figure 5.40 Column/Beam Connection With Vertical Dowel Bars



1. Column with cast-in steel plate and protruding dowels at the top
2. Reinforced neoprene rubber bearing
3. Beam with dowel holes
4. Grouted dowel-hole connection
5. Non-grouted connection

To the left, a movable bearing is shown; to the right, a fixed bearing is used.

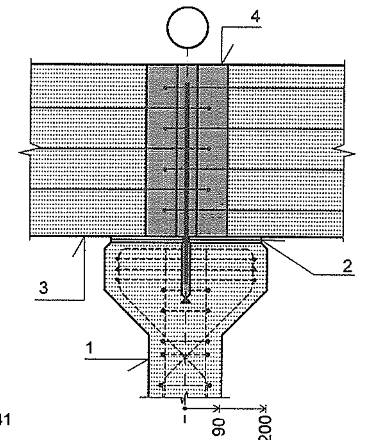


Figure 5.41

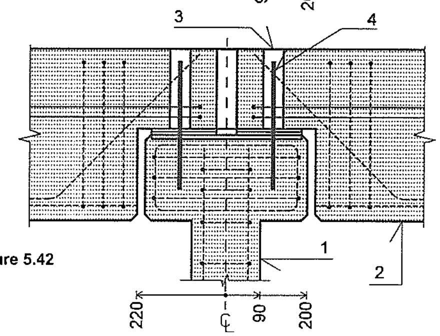


Figure 5.42

Figure 5.41 : Connection Similar to 5.39
Figure 5.42 : Connection Similar to 5.40

Figure 5.41 Column-Beam

1. Column with corbels and anchor insert with dowel
2. Bearings: steel or neoprene
3. Beam with stirrups
4. Column-beam joint cast in-situ

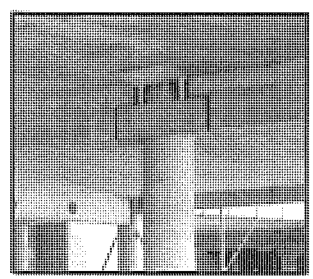


Figure 5.42 Half Joint Column/Beam Connection

1. Column with corbels and cast-in dowels
2. Precast beam with half joint
3. Grouted dowel-hole connection
4. Dowel protruding from column

In Figure 5.42, the components are placed on top of each other. Using a beam with half joint, the corbels can be hidden and the structural height can be reduced as shown in Figure 5.42.

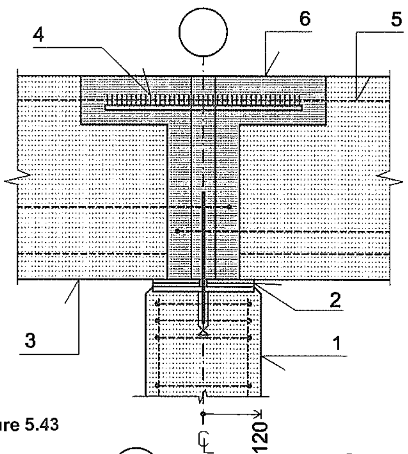


Figure 5.43

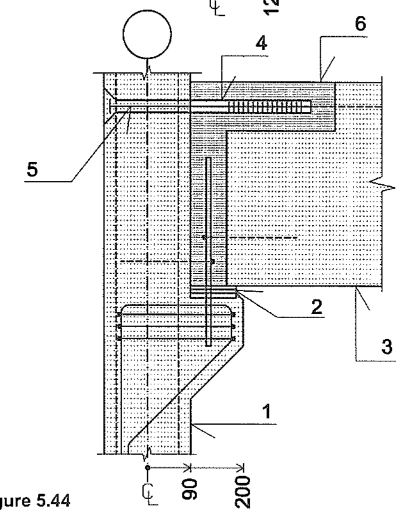


Figure 5.44

Figure 5.43 Column-Beam With Moment And Shear Continuity

1. Column with anchor insert
2. Steel bearing plate
3. Beam with protruding stirrups and rebars at the top
4. Welded connection using steel angle
5. Reinforcement bar
6. In-situ concrete or grouting

Figure 5.44 Edge Column/Beam Connection With Moment And Shear Continuity

1. Column with corbel and cast-in dowel
2. Steel bearing plate
3. Beam with stirrups and protruding rebars at the top
4. Welded connection
5. Reinforcement bar or bolt
6. In-situ concrete or grouting

The beams can be connected to both the bottom and top by stirrups, locking bars or welding.

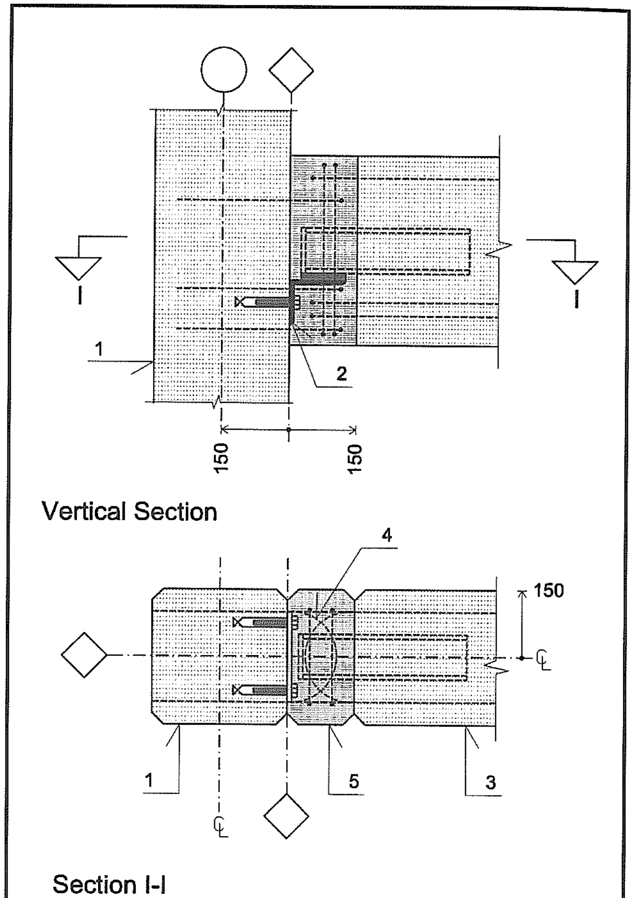


Figure 5.45 Column/Beam Connection With Temporary Steel Section Supports

1. Column component with anchor insert
2. Steel angle bolted to column
3. Beam with embedded steel section
4. Connection using protruding stirrups and reinforcement placed on site
5. Casting on site

In Figure 5.45, a combination of a temporary and a permanent column-beam support is shown. First the embedded steel profile acts as support, later the stirrups and locking bars take over.

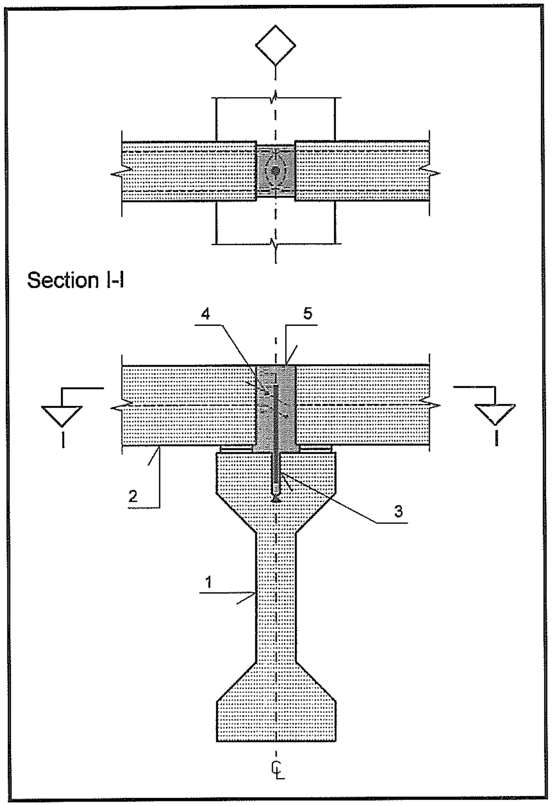
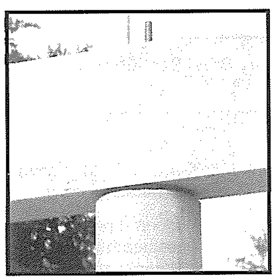
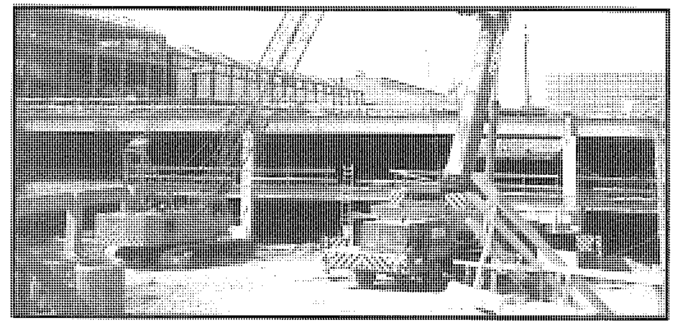


Figure 5.46 Secondary To Main Beam Connection



1. Prestressed main beam with anchor inserts at the top
2. Secondary beam or purlin with protruding stirrups
3. Dowel screwed into insert
4. Stirrup-dowel connection
5. Cast in-situ joint

The connecting detail is simple, cheap and easy to execute.



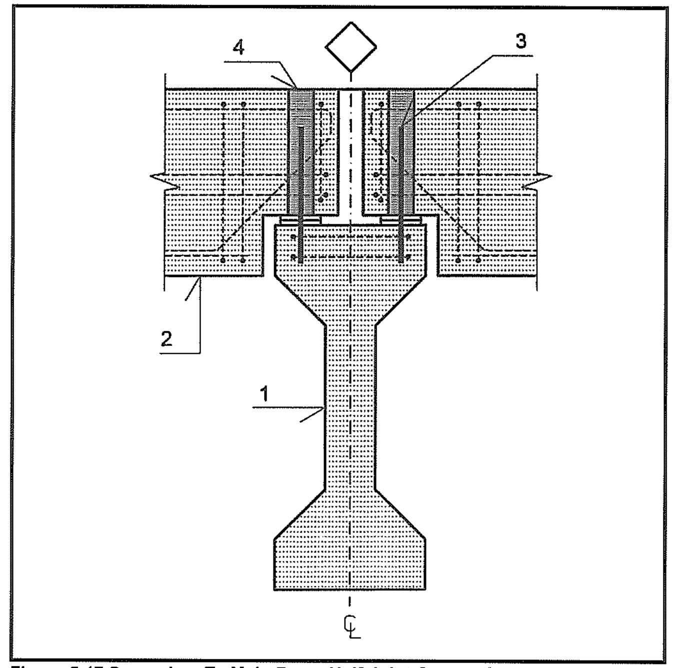
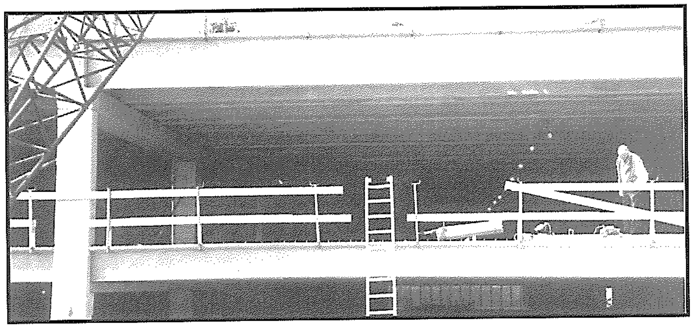


Figure 5.47 Secondary To Main Beam Half-Joint Connection

1. Prestressed beam
2. Secondary beam
3. Dowel protruding from beam
4. Grouted dowel-hole connection

With the use of beam half joints, a more aesthetic solution has been obtained.



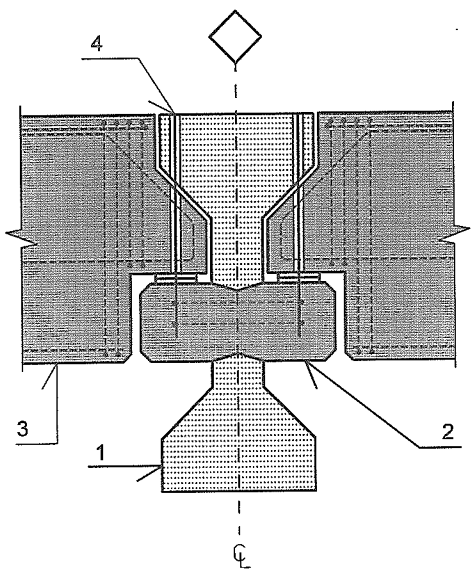


Figure 5.48 Secondary To Main Beam Half Joint Connection

1. Prestressed main beam
2. Transverse beam as double corbel
3. Secondary beam with half joint
4. Dowel-hole connection

To decrease the total construction height, double corbels can be used.

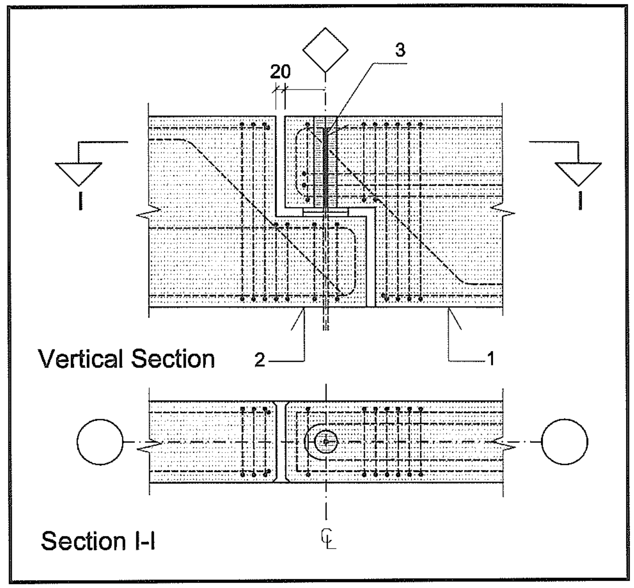
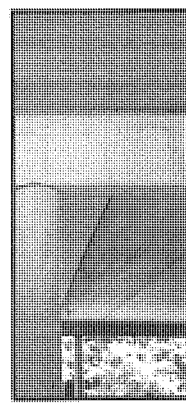
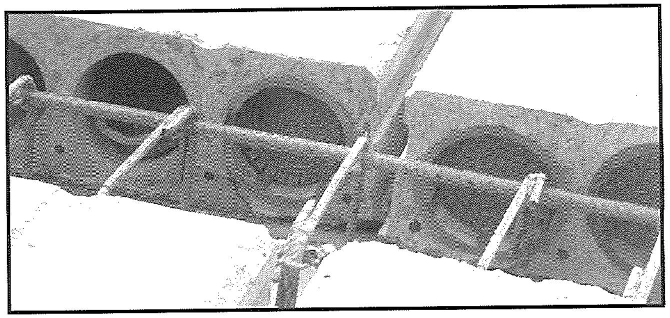


Figure 5.49 Half Joint Connection In Beams



1. Beam with half joint
2. Cantilevered beam
3. Dowel-hole connection, half-beam joint

Two half-beam joints have to be considered.



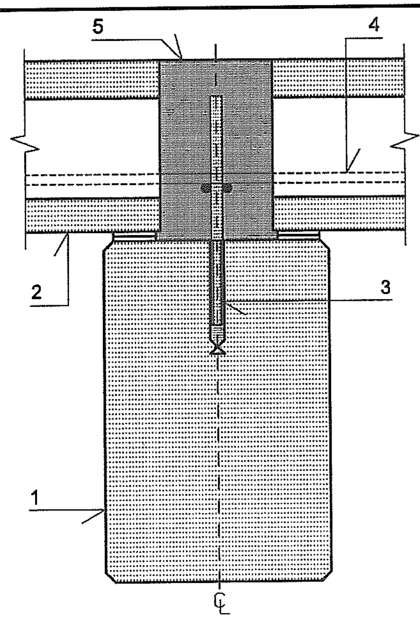


Figure 5.50 Beam And Hollow Core Slab Connection

1. Beam with anchor inserts at the top
2. Hollow core slab
3. Dowel screwed into insert
4. Reinforcement in slab joint
5. In-situ concrete or grouting

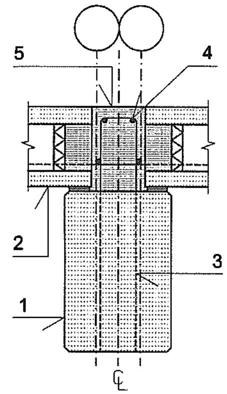


Figure 5.51

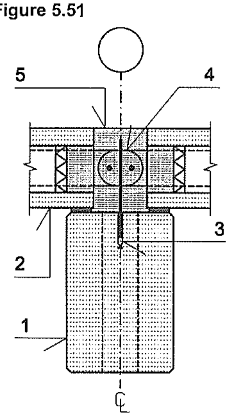


Figure 5.52

Figure 5.51 Beam-H.C.Slab
1. Precast beam
2. Hollow core slab
3. Stirrups protruding from the top of beam
4. Joint reinforcement
5. Casting on site

Figure 5.52 Beam-H.C.Slab
1. Precast beam
2. Hollow core slab
3. Dowel in insert
4. Joint reinforcement
5. Casting on site

Figure 5.53 Beam-H.C.Slab
1. Precast beam
2. Hollow core slab
3. Dowel in insert
4. Locking stirrups
5. Casting on site

Figure 5.51 shows modular slabs and a neutral zone; Figure 5.52 uses non-modular slabs. Both solutions result in a wider cast-on-site joint.

Figure 5.54 Beam-H.C.Slab
1. Precast beam: half inverted T
2. Hollow care slab
3. Anchor insert
4. Joint reinforcement screwed into insert
5. Casting on site

It is important to anchor the joint reinforcement to, for instance, edge beams.

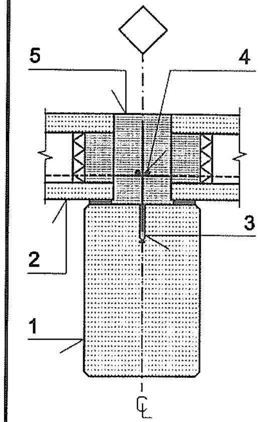


Figure 5.53

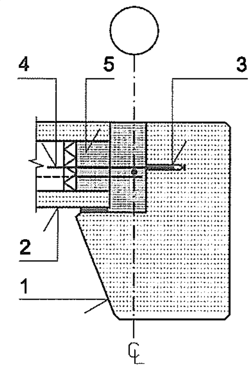
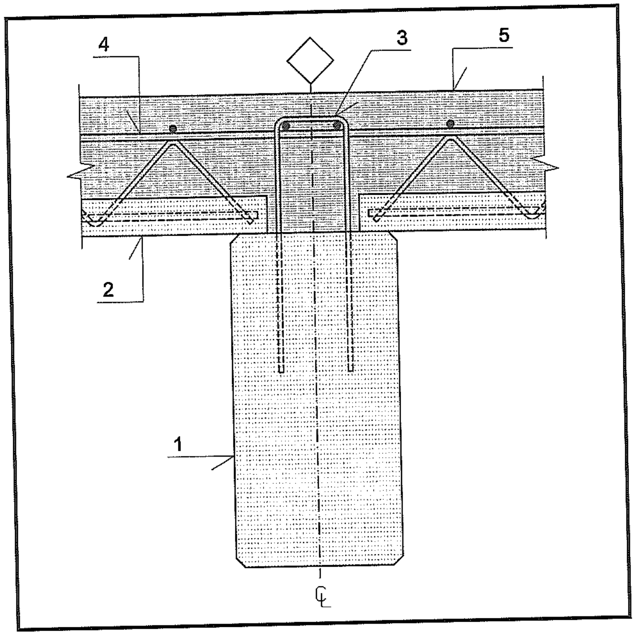
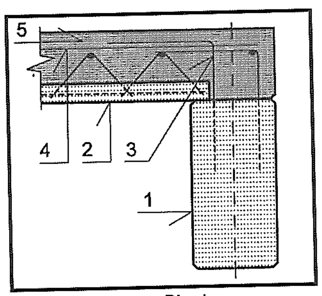


Figure 5.54



1. Precast beam
2. Plank
3. Stirrups protruding from beam
4. Reinforcement placed on site
5. Cast in-situ structural topping

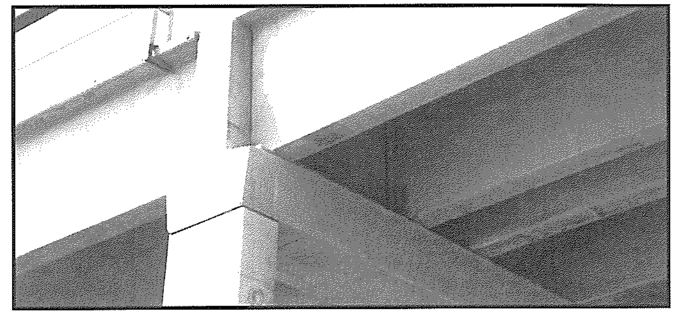
Figure 5.55 Beam-Plank



1. Precast beam
2. Plank
3. Reinforcement protruding from beam
4. Reinforcement placed on site
5. Cast in-situ structural topping

Figure 5.56 Beam-Plank

The combination of precast and cast in-situ structures gives good opportunities for the transmission of forces in the main structural system.



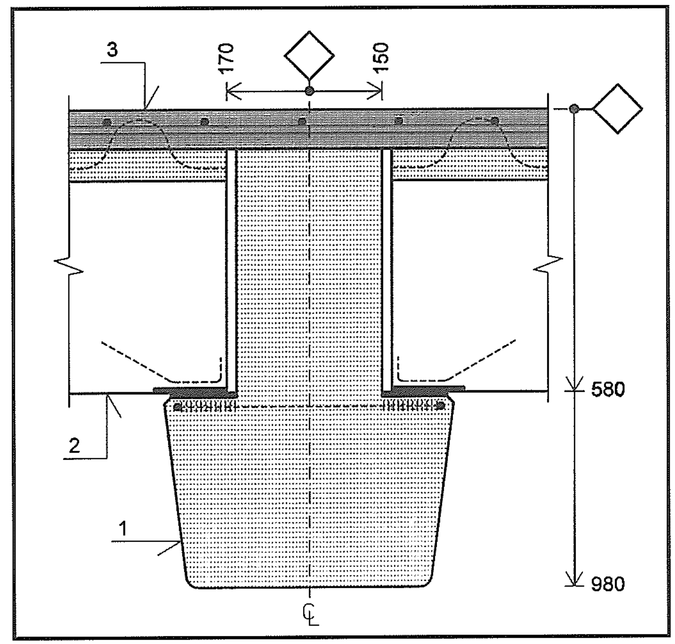


Figure 5.57 Beam And Double T

1. Beam: inverted T
2. Double T-slab
3. Structural topping

Normally the double T-slabs are produced with cast-in steel plates in the ribs for a welded support, and with protruding stirrups in the plate for the cast in-situ structural topping.

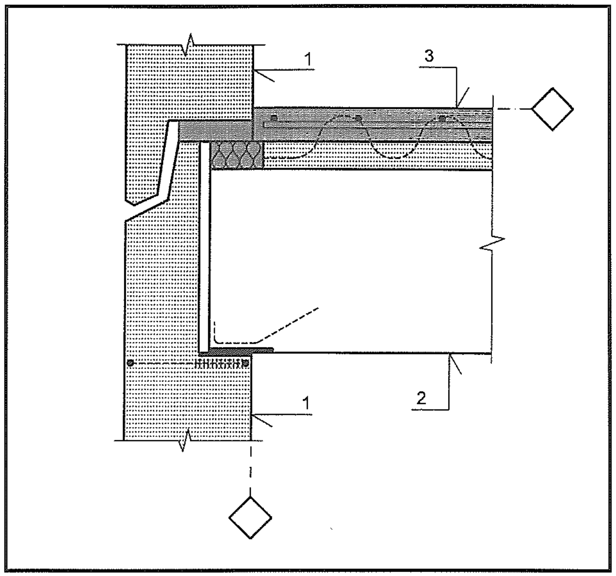


Figure 5.58 Beam-Double T

1. Facade beam (or frame or facade)
2. Double T-slab
3. Structural topping

As double T's are normally long-spanning components, moments and movements in the joints have to be considered in the structural design.

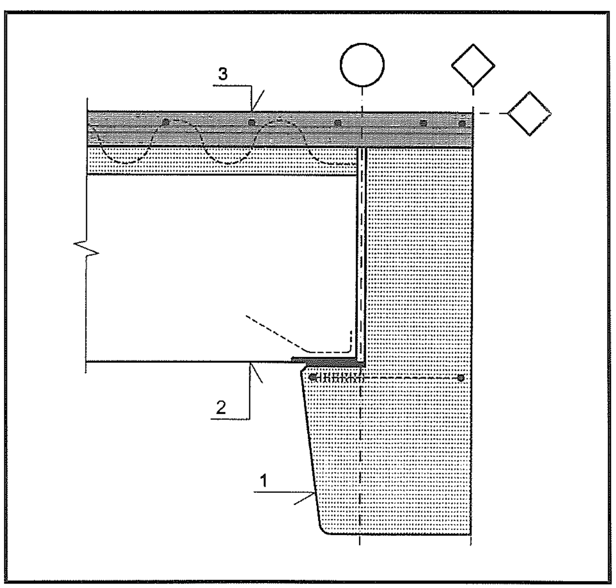


Figure 5.59 Beam-Double T

1. Beam, half inverted T
2. Double T-slab
3. Structural topping

Steel studs at the top of the edge beam could provide a better anchorage of the structural topping.

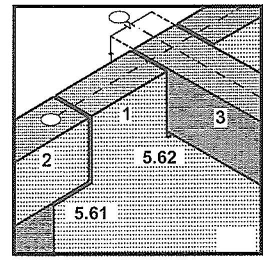
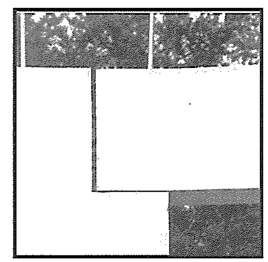


Figure 5.60 Beam-Wall Connections

1. Wall
2. Beam placed in the wall plane of symmetry
3. Beam placed perpendicular to the wall



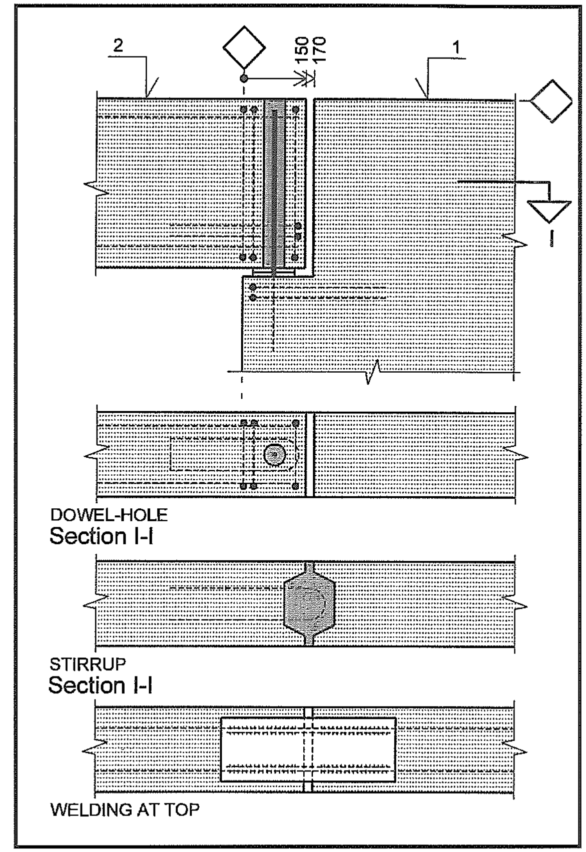
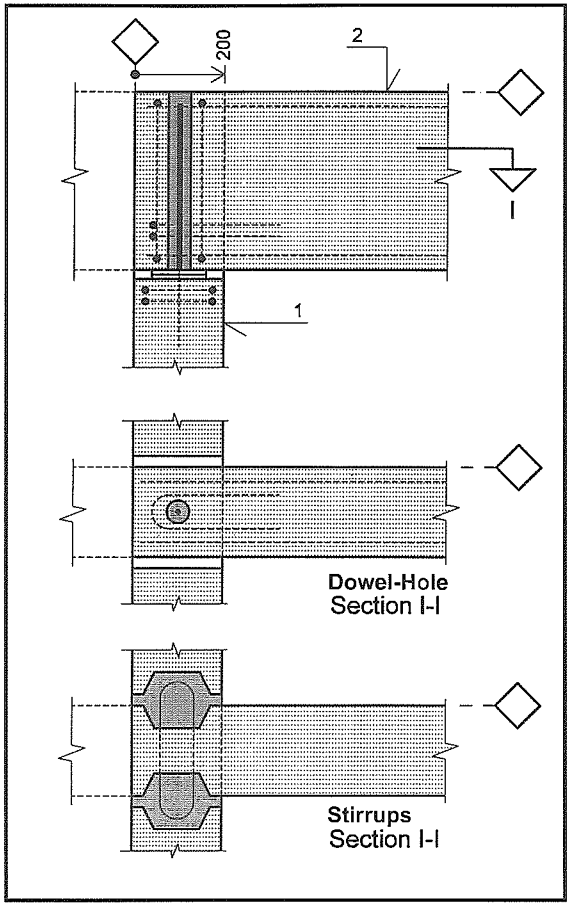


Figure 5.61 Beam-Load Bearing Wall

1. Wall
2. Beam

The beam-wall assembly could consist of either a dowel-hole, or stirrup or a welded connection as shown in Section I-I.



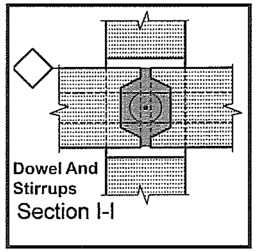


Figure 5.62 Beam-Load Bearing Wall

1. Wall
2. Beam

As shown in Section I-I, assembly could be done using a dowel-hole connection or protruding stirrups in cast in-situ joints.

Figure 5.63

1. Edge beam
2. Plank
3. Cladding
4. Structural topping

The cladding is fixed using dowel-hole connections which are locked to the structural topping.

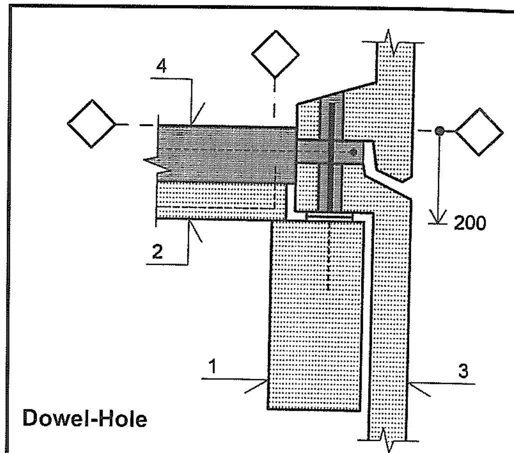


Figure 5.63 Beam-Cladding

Figure 5.64

1. Edge beam
2. Plank
3. Cladding
4. Structural topping

The cladding is fixed using embedded steel profiles and via placed-on-site steel angles.

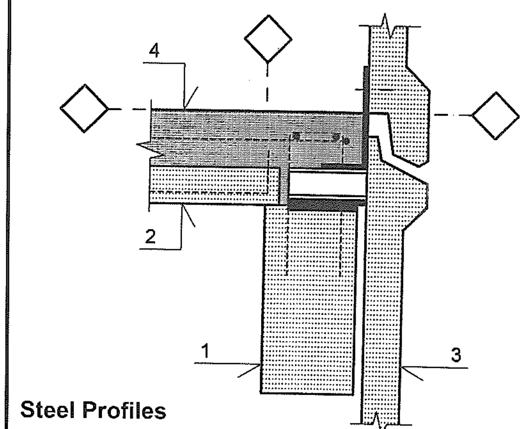
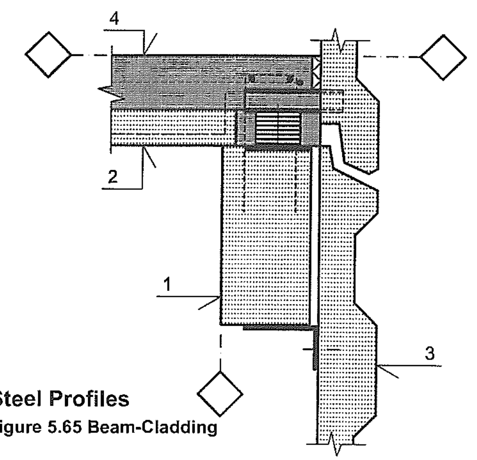


Figure 5.64 Beam-Cladding



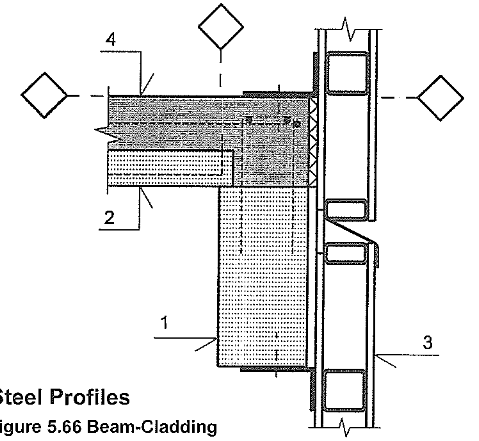
Steel Profiles

Figure 5.65 Beam-Cladding

Figure 5.65

1. Edge beam
2. Plank
3. Cladding
4. Structural topping

The cladding components are fixed to the beam by steel angles and to the floor slab using embedded steel sections locked to the structural topping.



Steel Profiles

Figure 5.66 Beam-Cladding

Figure 5.66

1. Edge beam
2. Plank
3. Cladding
4. Structural topping

The lightweight cladding shown can be fastened to the beam and finished floor slab using steel angle profiles.

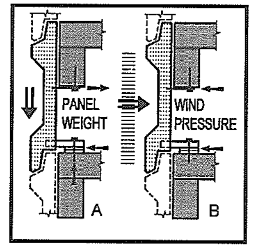


Figure 5.67 Beam-Panel Connections

Statically determinate connections should be chosen if possible. Load reactions are shown as a response to panel weight and to wind pressure.

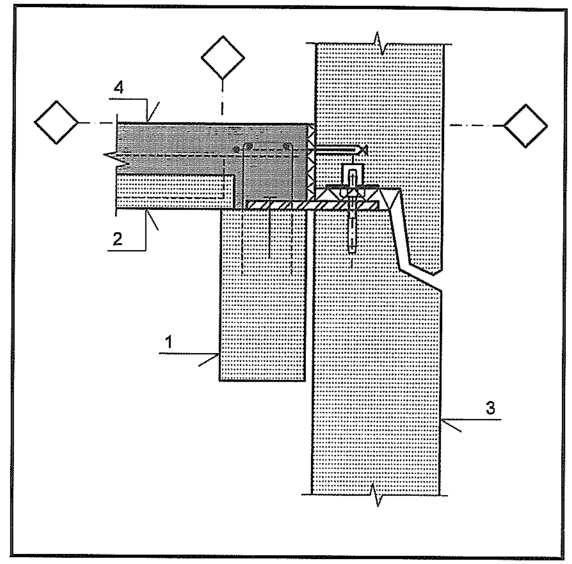


Figure 5.68 Beam-Facade

1. Edge beam
2. Plank
3. Facade
4. Structural topping

Vertical forces are transmitted through the horizontal joint. Horizontal forces are carried by the steel plate and reinforcement anchorage.

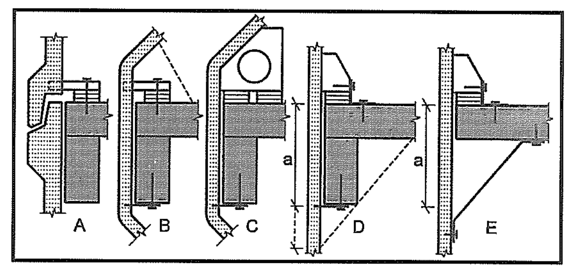


Figure 5.69 Beam-Panel Supports

Schematic solutions for positioning of joints.

A. Panels are fastened to floor slab
B. Joints are placed at floor slab and beam
C. Cross walls on panel plus beam joint
D. Corbels on panel plus beam fixing
E. Corbels on panel plus steel bracing

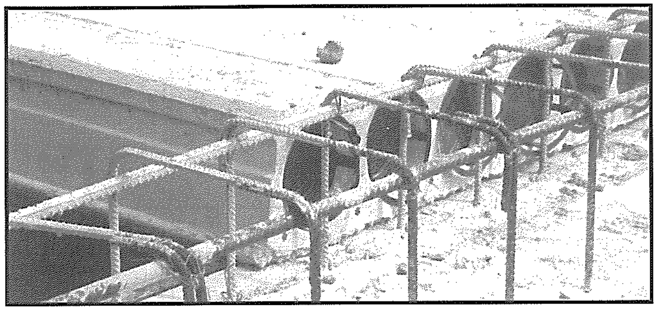


Figure 5.70 H.C.Slab-H.C. Slab

1. Hollow core slab with reinforcement bar placed in joint

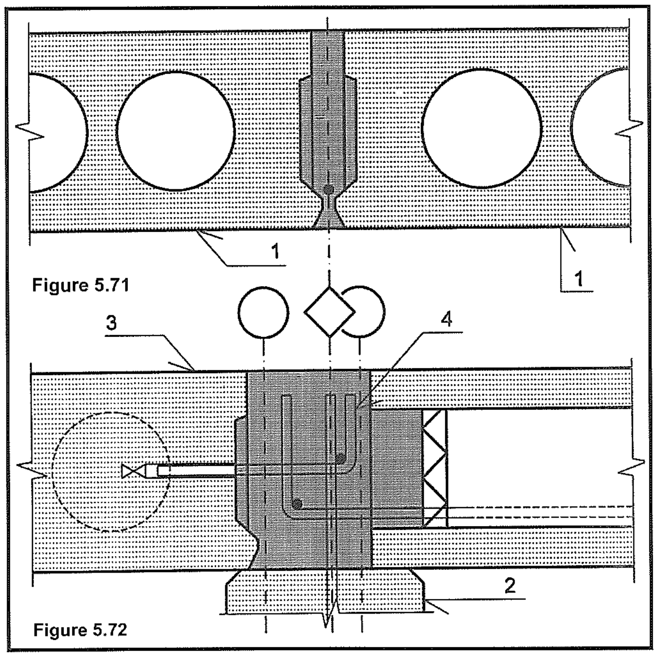


Figure 5.71 And 5.72: H.C.Slab-H.C.Slab

2. Beam, wall or frame
3. Edge of HCS without void
4. Locking rebars and dowels in joint

It is important to reinforce structural joints, for instance as shown with interaction between dowel in beam, reinforcement from slab joint and locking bar screwed into insert.

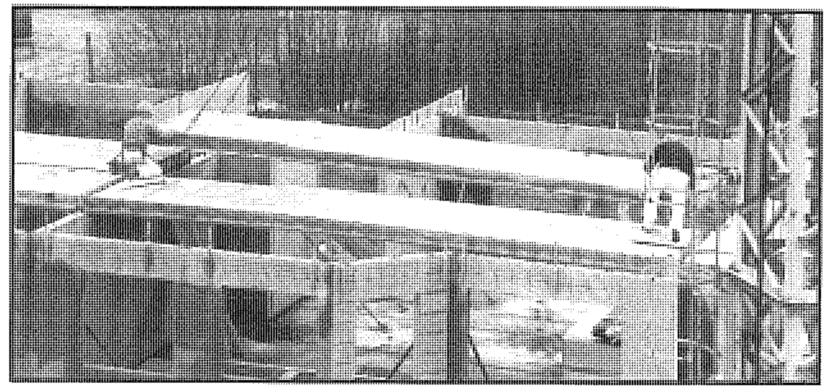
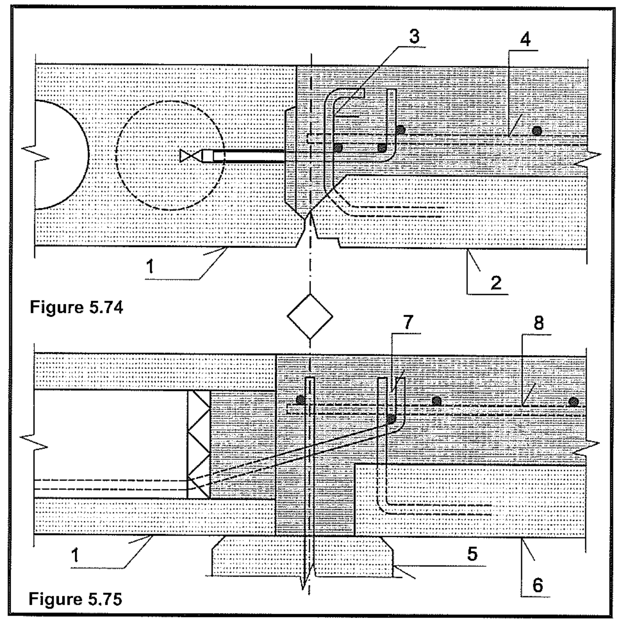


Figure 5.73 H.C.Slab-Plank



Figures 5.74 And 5.75 H.C.Slab-Plank

1. Hollow core slab
2. Plank
3. Locking reinforcement system
4. Reinforcement in structural topping

To create structural continuity in the floor slab system, it is important to anchor joint reinforcement in all adjoining members.

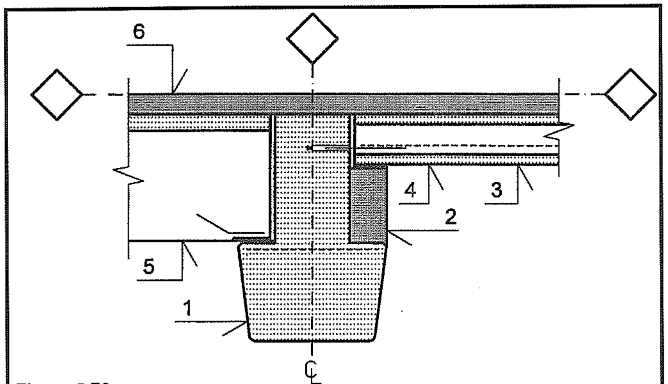


Figure 5.76

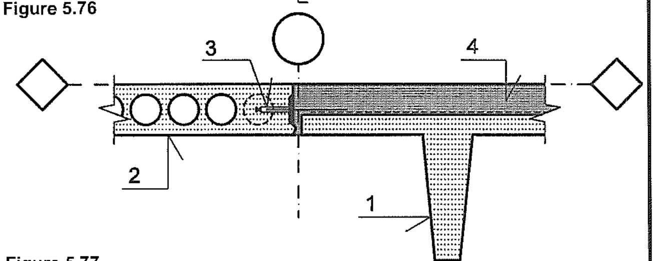


Figure 5.77

Figure 5.76 H.C.Slab-Double T

1. Beam as inverted T
2. Precast or cast in-situ members
3. Hollow core slab
4. Joint reinforcement anchored to insert
5. Double T-slab component
6. Structural topping

Figure 5.77 H.C.Slab-Double T

1. Double T-slab component
2. Hollow core slab
3. Steel anchor in edge of hollow core slab
4. Structural topping

Normally, the double T-support is secured by a welded connection. Structural continuity between different types of slabs is obtained by cast in-situ structural topping anchored to precast members.

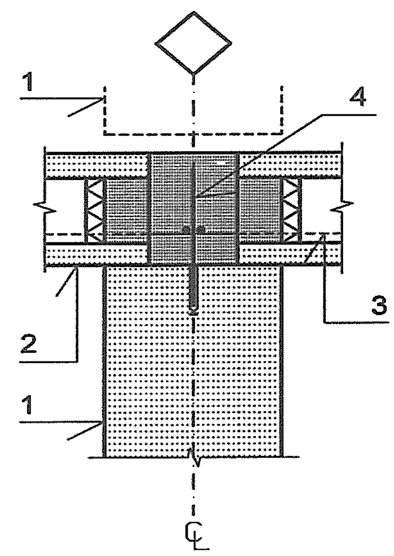


Figure 5.78

Figure 5.78 H.C.Slab-Load Bearing Wall

1. Load bearing wall
2. Hollow core slab, load bearing support
3. Slab joint reinforcement
4. Dowel screwed into steel insert

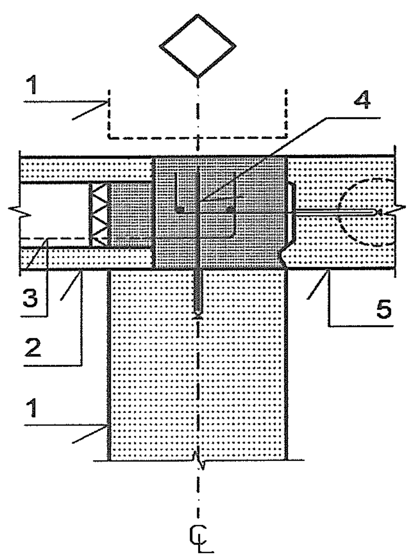


Figure 5.79

Figure 5.79 H.C.Slab-Load Bearing Wall

1. Load bearing wall
2. Hollow core slab, load bearing support
3. Slab joint rebar to be locked in wall joint
4. Dowel screwed into steel insert
5. Hollow core slab, non-load bearing support

Crossing reinforcement bars are locked together in the wall-slab joint. Wall in next storey (if any) is shown dotted.

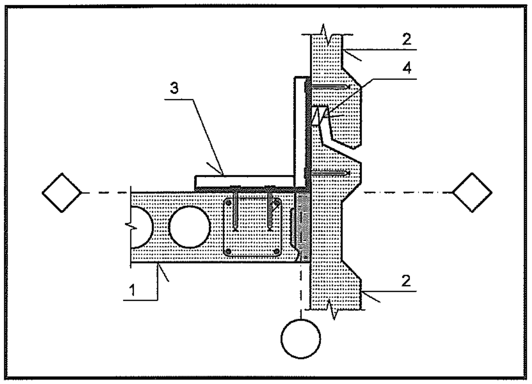


Figure 5.80 H.C.Slab-Cladding

1. Hollow core slab, non-load bearing edge
2. Cladding component
3. Steel angle bolted to hollow core slab and cladding
4. Joint material

The slab component has been reinforced along the edge to absorb vertical as well as horizontal forces from the cladding.

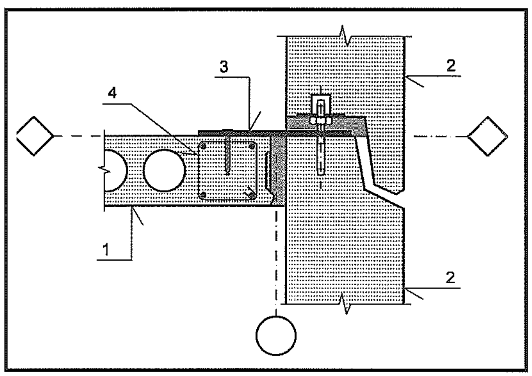


Figure 5.81 H.C.Slab-Facade

1. Hollow core slab, non-load bearing edge
2. Facade
3. Steel anchor plate
4. Edge beam reinforcement in hollow core slab

Facade weight is transmitted via the horizontal facade joint. The facade is anchored for horizontal forces using a steel plate bolted to the floor slab structure.

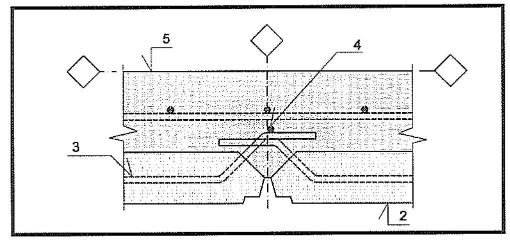


Figure 5.82 Plank-Plank

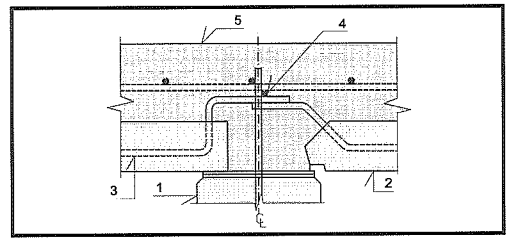


Figure 5.83 Plank-Plank

Figures 5.82 And 5.83

1. Supporting wall or beam
2. Plank
3. Reinforcement protruding from plank
4. Locked bar in joint
5. Structural topping

Figure 5.82 shows the connection along the non-load bearing edge while Figure 5.83 the connection between planks and supporting member. Protruding reinforcement bars from the planks are to be connected and locked in a joint cast in-situ together with the structural reinforced topping.

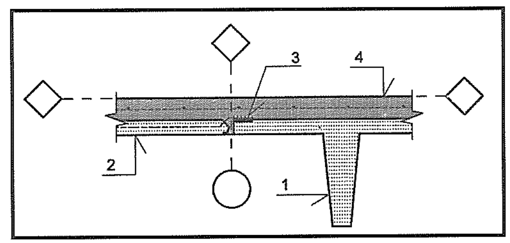


Figure 5.84 Plank-Double T

Figure 5.84

1. Double T-slab
2. Plank
3. Welded connection
4. Structural topping

Normally, steel plates are cast-in per 1.2 rib along the edge of double T's. Protruding rebars from planks could be welded to the plates and covered by the topping.

Figure 5.85

1. Supporting wall
2. Plank
3. Structural topping
4. Joint reinforcement locked to assembly bolt

Protruding rebars from planks are locked using joint reinforcement and assembly bolts from the wall.

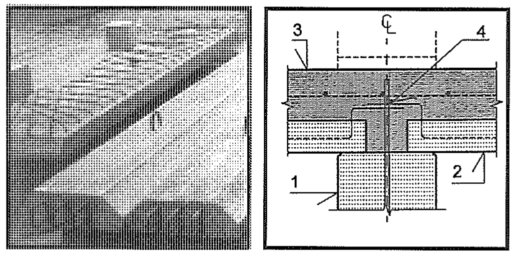


Figure 5.85 Plank-Load Bearing Wall

Figure 5.86

1. Structural topping on planks
2. Cladding component
3. Steel angle bolted to plank and cladding

As the cladding is lightweight, the steel angle must be designed to transmit vertical as well as horizontal forces.

Figure 5.87

1. Structural topping on planks
4. Facade component

Vertical forces are transmitted from facade to facade whereas horizontal loads go from assembly bolt via cast-in steel insert to reinforcement in the structural topping.

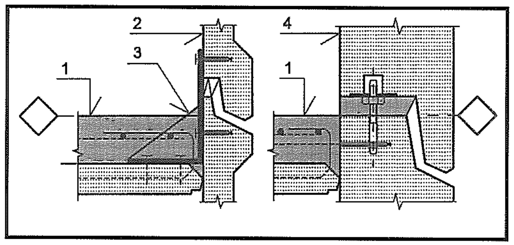


Figure 5.86 Plank-Cladding **Figure 5.87 Plank-Facade**

Figure 5.88

1. Double T-slab
2. Cast-in steel plate
3. Protruding stirrups
4. Connecting steel plate
5. Structural topping

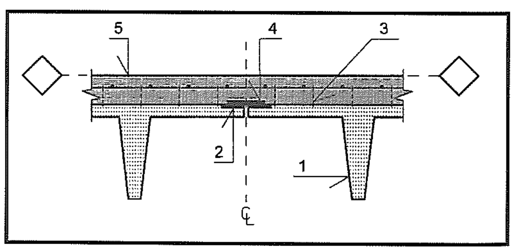


Figure 5.88 Double T

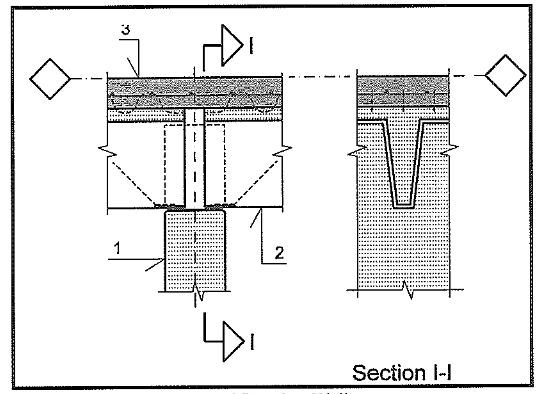


Figure 5.89 Double T-Load Bearing Wall

Figure 5.89

1. Supporting Wall
2. Double T, the rib
3. Structural topping

Welded connection between wall and ribs together with the structural topping locked to the plate create continuity in the floor slab structure.

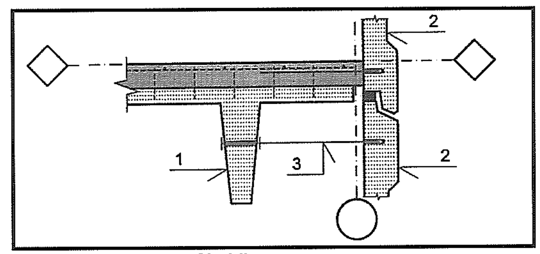


Figure 5.90 Double T-Cladding

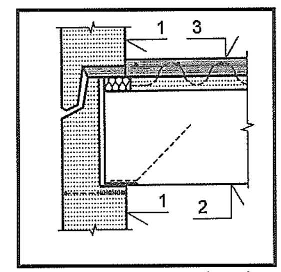


Figure 5.91 Double T-Facade

Figure 5.90

1. Double T, the rib
2. Cladding components
3. Connecting steel bar

The cladding components are placed on top of each other, horizontal forces are transmitted to the floor slab structure via steel anchors.

Figure 5.91

1. Facade components
2. Double T, the rib
3. Structural topping

The rib-facade joint is welded. A recess in the plate over the ribs allows the double T to rotate at the support.

Figure 5.92

1. Solid precast wall
2. Cast in-situ joint
3. Reinforced cast in-situ joint

Sketch A shows the standardised vertical joint between two wall components with castellated vertical edges. Sketch B shows a T-shaped wall connection which is normally reinforced like the L-shaped wall connection shown in Sketch C.

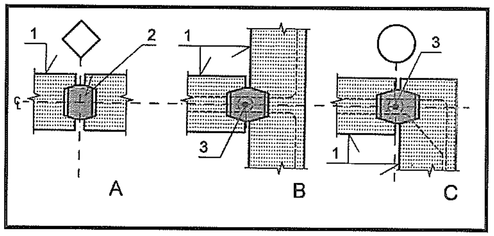


Figure 5.92 Load Bearing Wall-Loading Bearing Wall

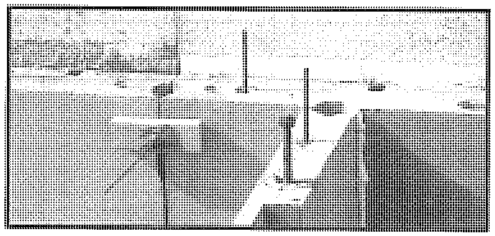


Figure 5.93 Load Bearing Wall-Load Bearing Wall

1. Standardised wall component
2. Grouting with cement mortar

The horizontal wall-to-wall joint is shown as a simple concrete joint with assembly bolts.

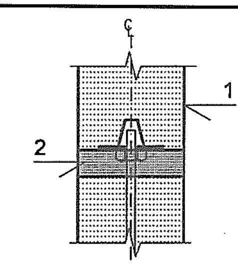


Figure 5.93

Figure 5.94 Load Bearing Wall-Load Bearing Wall

1. Standardised wall components
2. Hollow core slab
3. Plank
4. Structural topping

The wall-to-wall joint is often connected to slab structures as well. Reinforcement bars from adjoining members are locked using the structural topping.

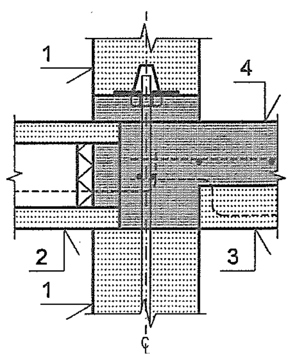


Figure 5.94

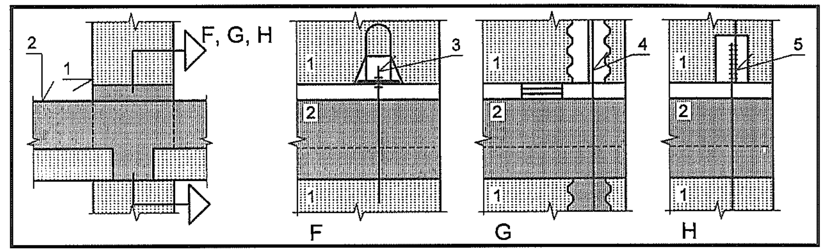
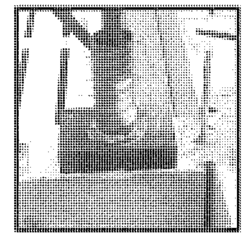


Figure 5.95 Load Bearing Wall-Load Bearing Wall, Tensile Joints

1. Load bearing and bracing walls
2. Concrete joint between walls and floor slabs
3. Bolted connection
4. Cast-in reinforcement bars
5. Welded connection



In Sketch F, the assembly bolt forms the tensile connection to the next wall. Sketch G shows reinforcement bars placed-on-site in corrugated pipes. Protruding rebars are welded together in Sketch H allowing tensile forces to be transmitted from wall to wall.

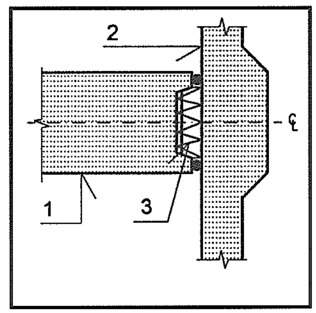


Figure 5.96 Load Bearing Wall-Cladding

Figure 5.96

1. Wall component
2. Cladding
3. Non-load bearing joint

As cladding components are normally fastened to floor slabs, the joint must be designed to resist sound and fire.

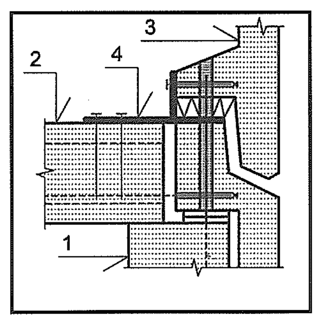


Figure 5.97 Load Bearing Wall-Cladding

Figure 5.97

1. Facade wall
2. Floor slab structure
3. Cladding
4. Steel angle to fix the cladding

Vertical forces are absorbed in a locked dowel-hole connection at the top of the cladding. Horizontal forces are transmitted to the floor slab via a steel anchor.

Figure 5.98

1. Facade wall
2. Plank
3. Standardised facade
4. Joint reinforcement
5. Structural topping

The assembly bolts in facades and walls act as connecting devices between the facade components and the structural topping.

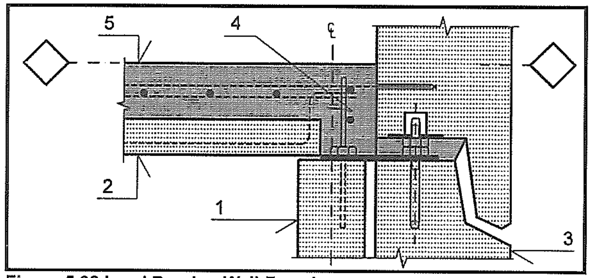


Figure 5.98 Load Bearing Wall-Facade

Figure 5.99

1. Cladding components
2. Fixing points, if any
3. Soft airtight joint
4. Partly open cladding joint
5. Soft airtight joint

When designing the joints between cladding components, it is important to know prevailing climatic conditions and movements in the cladding.

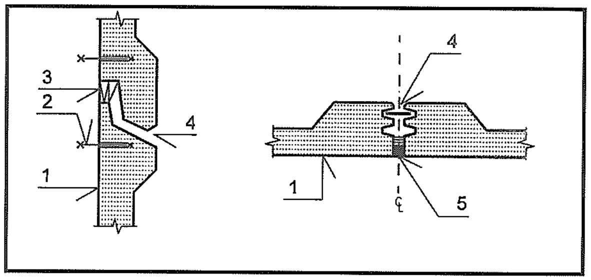


Figure 5.99 Cladding-Cladding

Figure 5.100

1. Facade component
2. Fixing point, if any
3. Grouting with cement mortar
4. Partly open facade joint
5. Reinforced castellated concrete joint

Normally, the facade-joint is designed according to the two-step principle: Airtightness inside and watertightness outside. Figure 5.100 shows a concrete joint inside and an open joint or a neoprene strip as an outside protection against the rain.

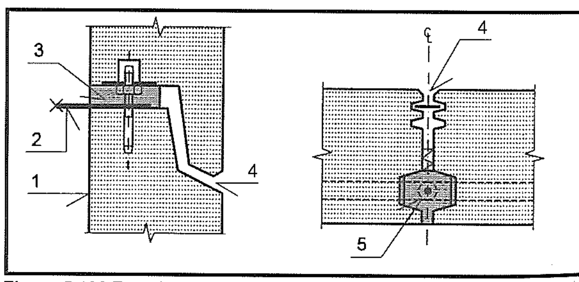


Figure 5.100 Facade-Facade

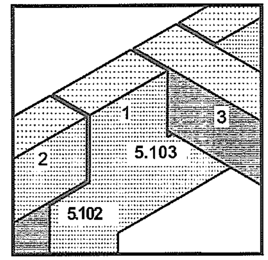


Figure 5.101 Beam-Frame Connections

Figure 5.101

1. Frame
2. Beam placed in frame plane of symmetry
3. Beam placed perpendicular to frame

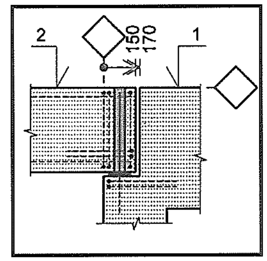


Figure 5.102 Beam-Frame

Figure 5.102

1. Frame
2. Beam

A normal dowel-hole solution is shown with reinforcing stirrups near the narrow support area.

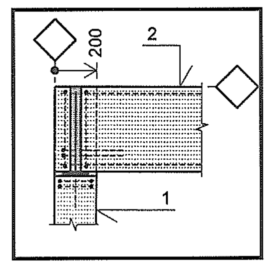


Figure 5.103 Beam-Frame

Figure 5.103

1. Frame
2. Beam

To get the components flushed, the whole width of the frame is used as supporting area. A dowel-hole connection is chosen.

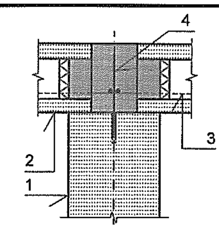


Figure 5.104

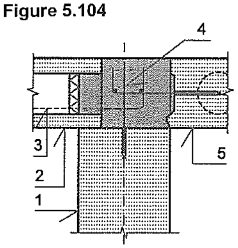


Figure 5.105

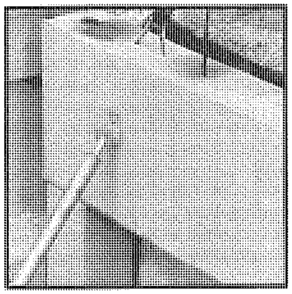
Figure 5.104 H.C.Slab-Frame

1. Frame
2. Hollow core slab
3. Reinforcement in slab joint
4. Dowel screwed into insert and grouted

Figure 5.105 H.C.Slab-Frame

1. Frame
2. Hollow core slab
3. Reinforcement in slab joint
4. Grouted joint with locking dowel and bars
5. HCS, non-load bearing edge

The two vertical sections in Figure 5.104 and Figure 5.105 show how continuity from slab to slab is obtained using joint reinforcement or protrusion of components.



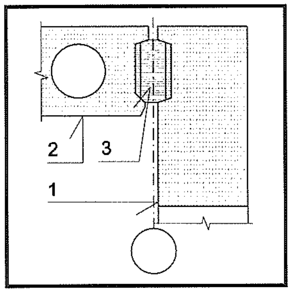


Figure 5.106 H.C.Slab-Frame

1. Frame
2. Hollow core slab
3. Cast in-situ castellated joint

A rough joint is able to absorb horizontal as well as vertical shear.

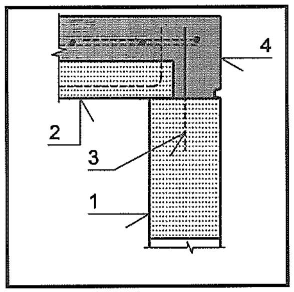


Figure 5.107 Plank-Frame

1. Frame
2. Plank
3. Protruding rebar
4. Casting on site

The connection is able to support the plank and absorb horizontal shear as well.

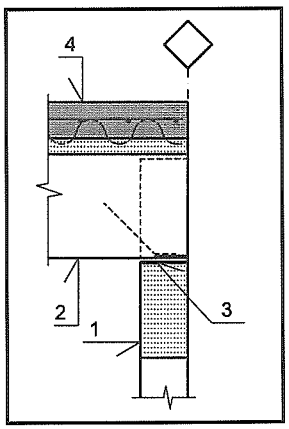


Figure 5.108 Double T-Frame

1. Frame
2. Double T-slab
3. Welded connection
4. Structural topping

Cast-in steel plates in frame and in ribs make it possible to execute a welded supporting connection.

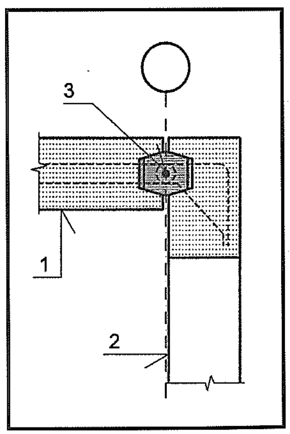


Figure 5.109 Load Bearing Wall-Frame

1. Standardised wall component
2. Frame, beam and column
3. Reinforced castellated joint

This horizontal section shows the use of a normal rough vertical joint in both the wall and frame column.

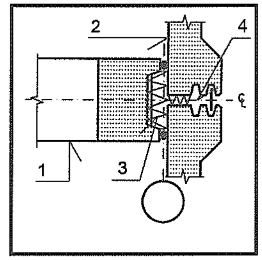
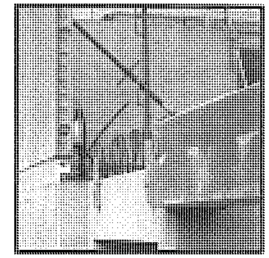


Figure 5.110 Cladding-Frame

Figure 5.110

1. Frame, beam and column
2. Cladding component
3. Soft joint material
4. Ventilated space/neoprene strip

The connection is considered only as a climatic joint. All forces are transmitted to the floor slab structure.



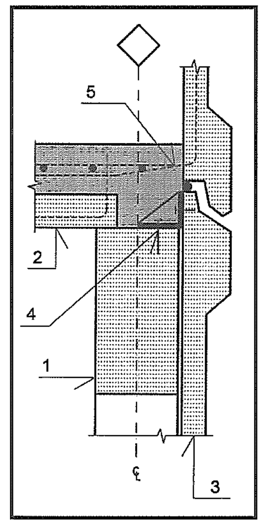


Figure 5.111 Cladding-Frame

Figure 5.111

1. Frame component
2. Plank
3. Cladding
4. Steel angle
5. Rebar protruding from cladding

The cladding is fastened at the top using a steel angle device and, at the bottom using rebars to be cast into the structural topping.

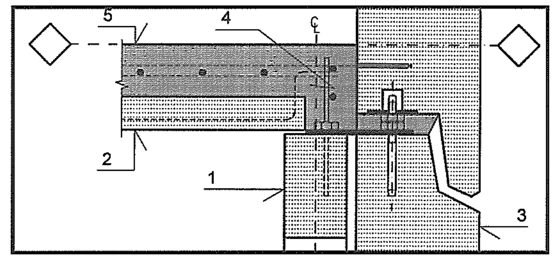
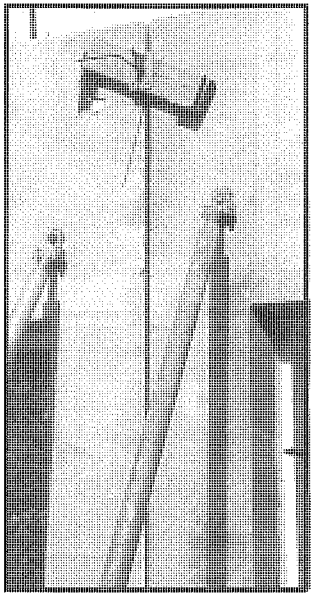


Figure 5.112 Facade-Frame

Figure 5.112

1. Frame component
2. Plank
3. Facade
4. Cast in-situ joint
5. Structural screed

Self weight from facade components is transmitted direct from facade to facade. Horizontal forces go to the floor slab structure using assembly bolt, steel plate and dowel.



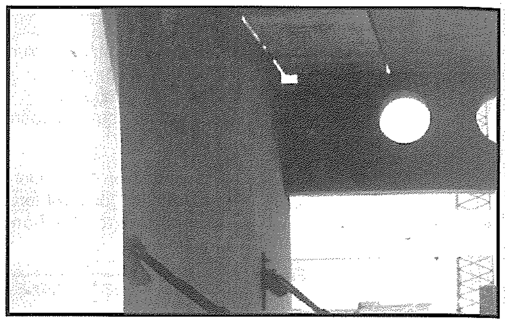


Figure 5.113 and 5.114 Frame-Frame

1. T-shaped frame
2. V-shaped frame

A two-dimensional load-bearing and bracing frame system can be created by using T- and V-shaped frame components. The T-shaped frames are placed longitudinally while the V-shaped frames are placed in the transverse direction.

Figure 5.113 shows the reinforced and rough joint between T-components while Figure 5.114 features the narrow bearing support between the two frame types. These bearings are reinforced by cast-in steel plates.

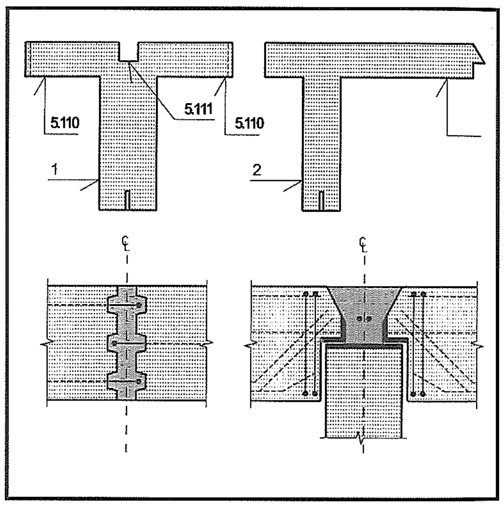
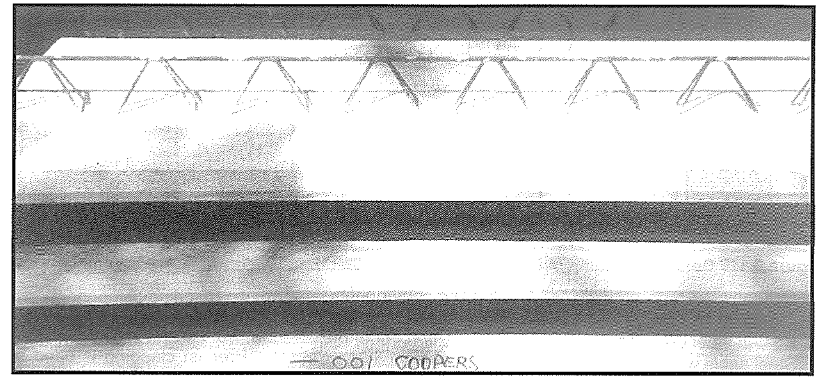


Figure 5.113 Figure 5.114



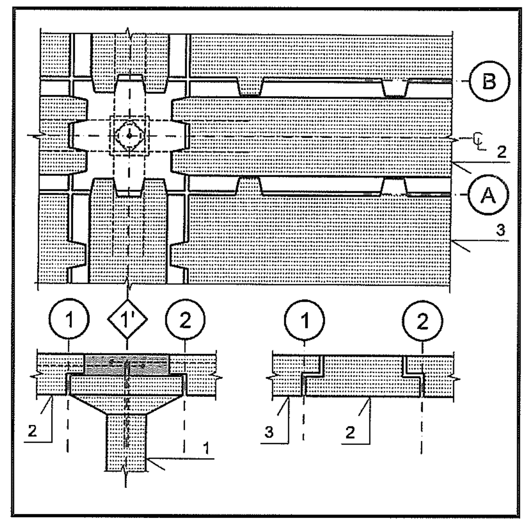
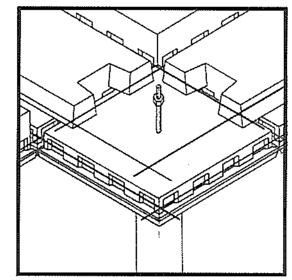


Figure 5.115 Column-Plank



1. Column with column head
2. Narrow planks or slabs
3. Planks or floor slabs

Precast building systems without beams may create problems due to very small support areas between slabs and columns.

Figure 5.115 provides a solution to this: A column head supports the narrow slabs, which are, in turn, supported by the main floor slabs.

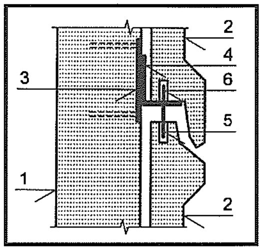


Figure 5.116 Column Cladding

1. Column
2. Cladding
3. Cast-in steel plate
4. Steel angle welded to plate
5. Dowel-hole connection
6. Dowel as assembly bolt

Vertical movement can take place at the top of the cladding components. Vertical and horizontal forces are transmitted via the welded steel connection.

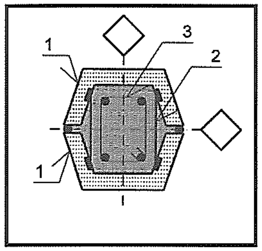


Figure 5.117 Column Cladding

1. Cladding components
2. Steel bar welded to the cladding components
3. Reinforcement in cast in-situ column

Precast cladding components are used as a finished surface and a form panel for cast in-situ structures.

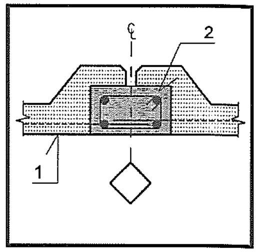


Figure 5.118 Column Facade

1. Cladding component
2. Cast in-situ column

Precast cladding is used as shuttering in a skeleton main structure.

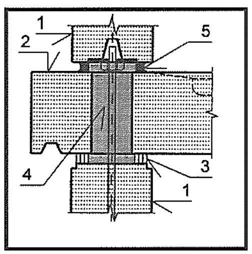


Figure 5.119 Column Balcony Slab

1. Column components
2. Balcony slab
3. Soft packing
4. Joint cast in-situ
5. Mastic joint

The designer must ensure a sound transfer of the vertical load and check on temperature movements in the balcony structure.

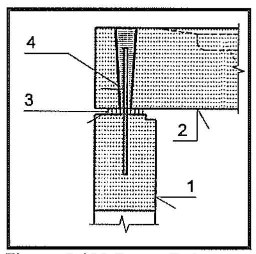


Figure 5.120 Beam-Balcony Slab

1. Beam or frame
2. Balcony slab
3. Soft bearing, for instance, neoprene
4. Dowel-hole connection

Temperature movements between beam and slab have to be considered.

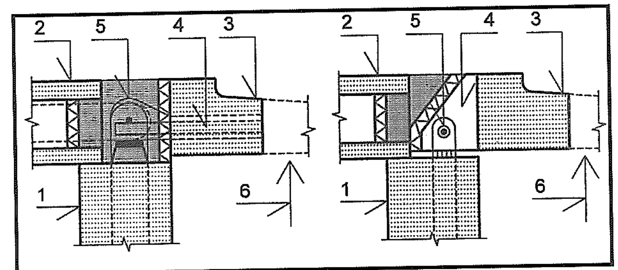


Figure 5.121 H.C.Slab-Balcony Slab

1. Beam, frame or wall
2. Hollow core slab
3. Balcony slab
4. Embedded steel section
5. Locking stirrups protruding from slabs
6. Vertical support

Figure 5.122 H.C.Slab-Balcony Slab

1. Beam, frame or wall
2. Hollow core slab
3. Balcony slab
4. Support cam or nib
5. Dowel in nib plus locking stirrup
6. Vertical support

Figures 5.121 and 5.122 have been designed to eliminate coldbridges and to allow temperature movements to take place.

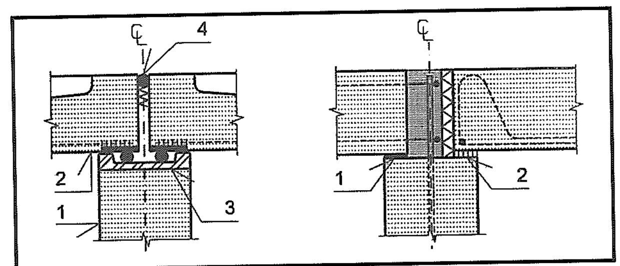


Figure 5.123 Load Bearing Wall-Balcony Slab

1. Wall
2. Balcony slab
3. Cast-in steel plate
4. Soft joint material

Figure 5.123 shows two movable supports in a wall-balcony connection, whereas Figure 5.124 shows one fixed and one movable joint.

Figure 5.124 Load Bearing Wall-Balcony Slab

1. Balcony slab with fixed support
2. Balcony slab with movable support

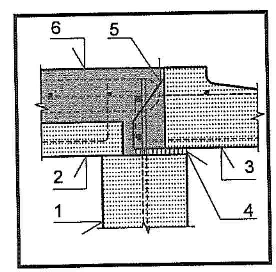


Figure 5.125 Plank-Balcony Slab

1. Beam, frame or wall
2. Plank
3. Balcony slab with nibs
4. Bearing, for instance, neoprene
5. Nib with protruding reinforcement
6. Structural topping

The balcony slab is considered as fixed-ended in Figure 5.125

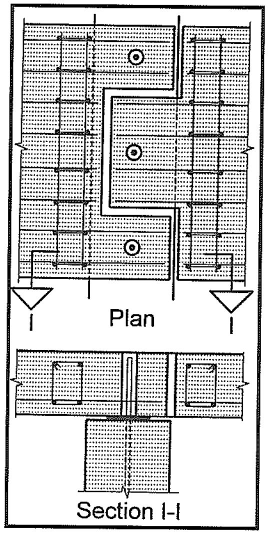


Figure 5.126 Load Bearing Wall- Balcony Slab

The assembly in Figure 5.126 shows how two balcony slabs can share the support area on top of a wall, each having the full width.

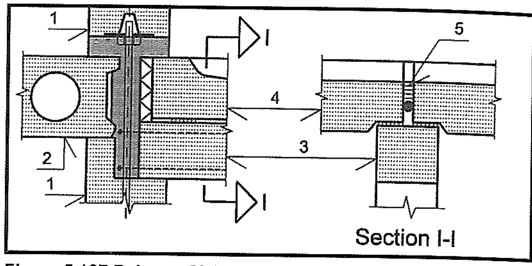


Figure 5.127 Balcony Slab-Balcony Slab

1. Facade wall
2. Hollow core slab
3. Balcony frame
4. Balcony slab

The balcony slabs are simply supported on the frame which is locked to the assembly bolt in the facade.

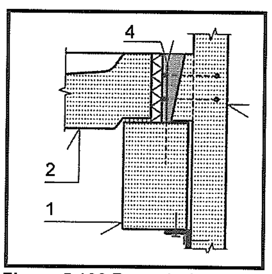
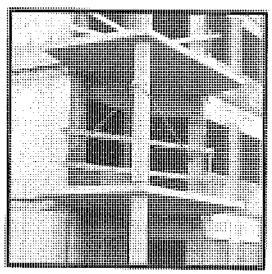
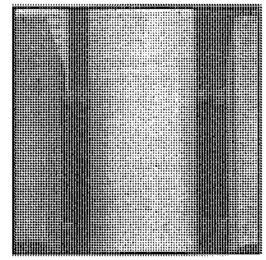


Figure 5.128 Beam-Balcony Parapet

1. Balcony beam
2. Balcony slab
3. Balcony parapet
4. Stirrup-dowel connection

The parapet is fastened to the top and bottom of the beam component.





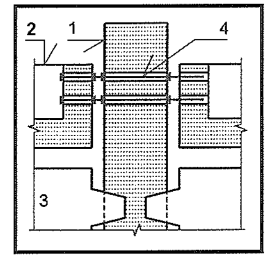


Figure 5.129 Load Bearing Wall Balcony Parapet

1. Load bearing wall
2. Balcony parapet
3. Balcony slab
4. Assembly bolts

The assembly bolts with nuts allow temperature movements from the right hand parapet to take place, but do not allow movements from the parapet to the left.

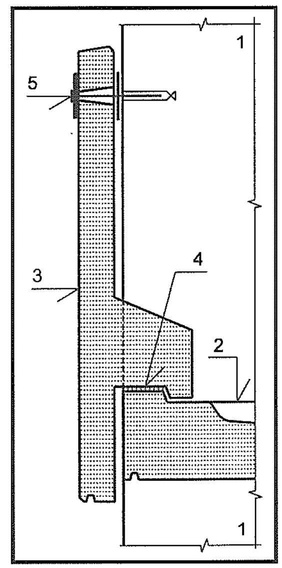
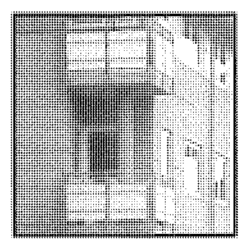
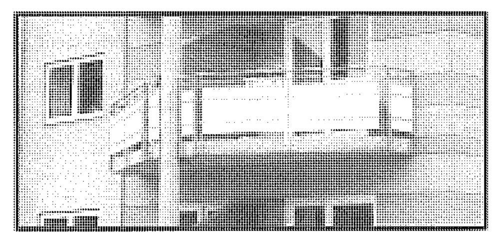


Figure 5.130 Balcony Slab-Balcony Parapet

1. Load bearing wall
2. Balcony slab
3. Balcony parapet
4. Neoprene bearing
5. Bolt into cast-in steel insert

Temperature movements in the parapet have to be considered before designing the parapet joint.





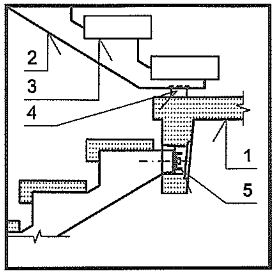
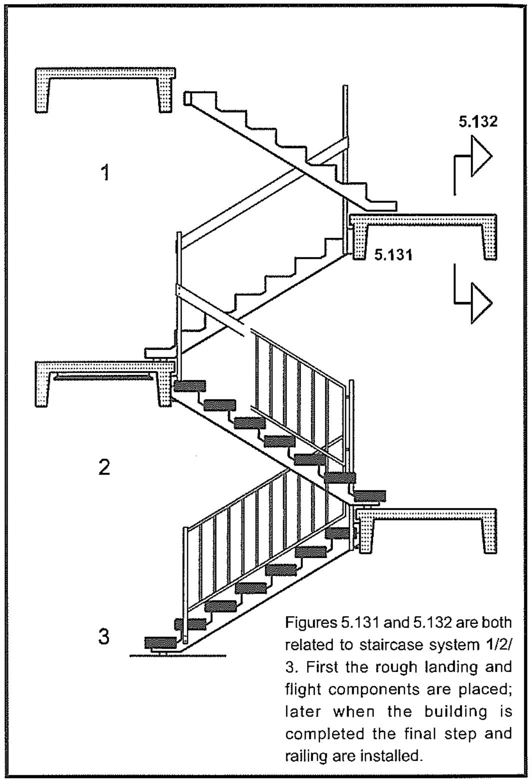


Figure 5.131 Beam-Staircase

1. Landing component
2. Flight component
3. Loose steps
4. Supporting steel studs
5. Bolted connection between flight and landing



Figures 5.131 and 5.132 are both related to staircase system 1/2/3. First the rough landing and flight components are placed; later when the building is completed the final step and railing are installed.

Staircase System 1/2/3

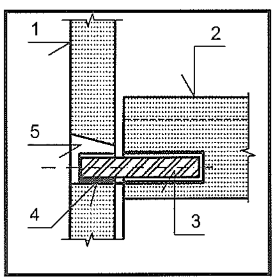
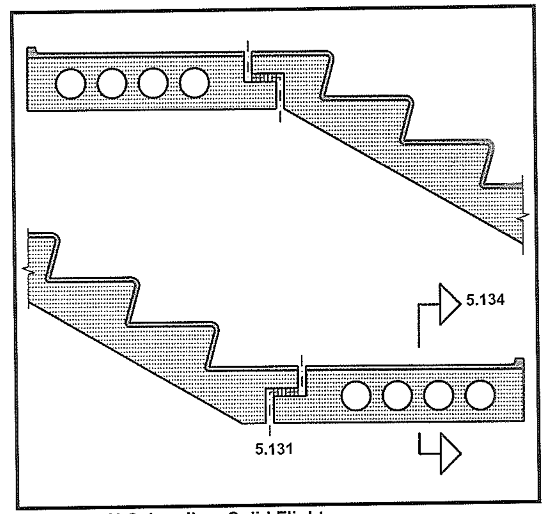


Figure 5.132 Load Bearing Wall-Staircase

1. Load bearing wall
2. Landing component
3. Loose dowel in steel section socket
4. Soft support, for instance, neoprene
5. To be grouted



Staircase: H.C. Landing, Solid Flights

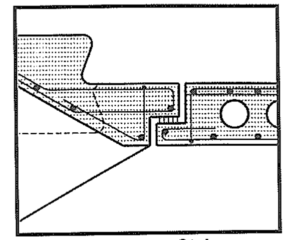


Figure 5.133 Beam-Stair

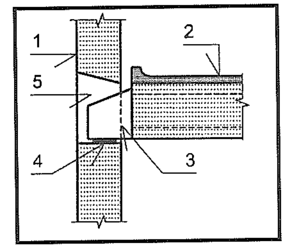


Figure 5.134 Load Bearing Wall-Staircase

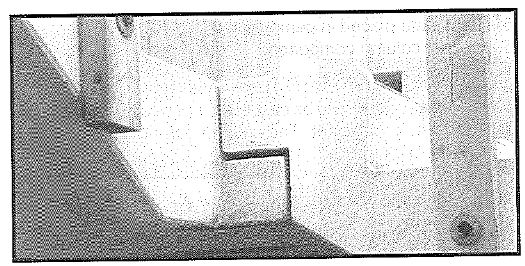


Figure 5.133

Half-beam joints are used between landings and flights.

Figure 5.134

1. Load bearing wall
2. Landing component
3. Support nib
4. Soft material, for instance, neoprene
5. To be grouted

Figures 5.133 and 5.134 are both related to the staircase system shown here which uses a hollow core slab as landing and a normal solid plate as flight component.

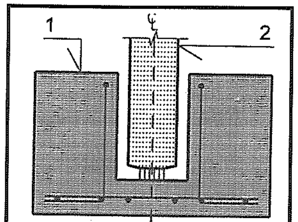


Figure 5.136 1st Generation

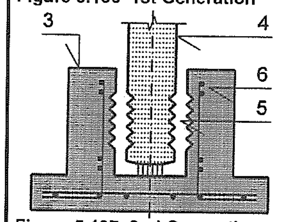


Figure 5.137 2nd Generation

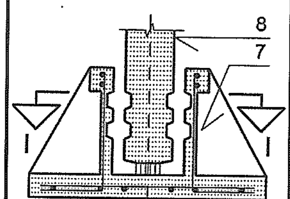
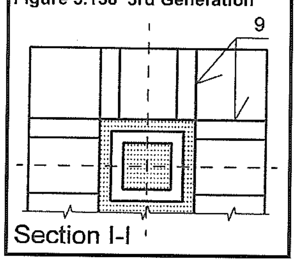


Figure 5.138 3rd Generation



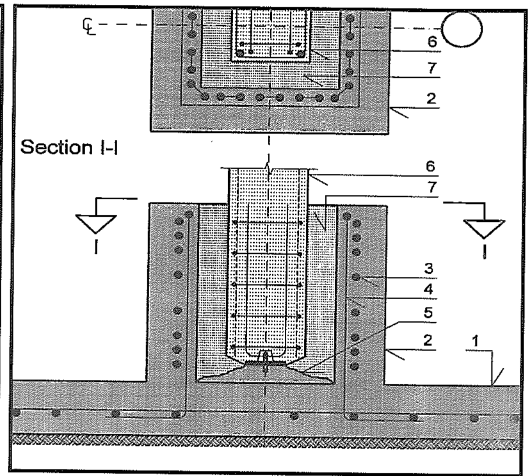


Figure 5.135 Column-Column Socket

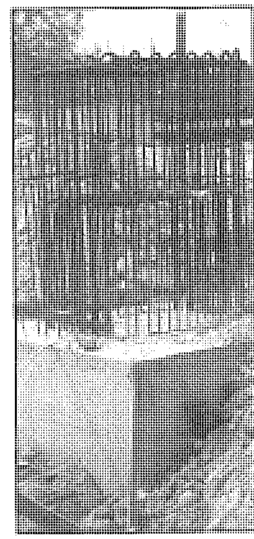
1. Bottom plate
2. Socket wall
3. Ring reinforcement
4. Vertical stirrups
5. Steel plate placed in cement mortar
6. Precast column component
7. Casting on site

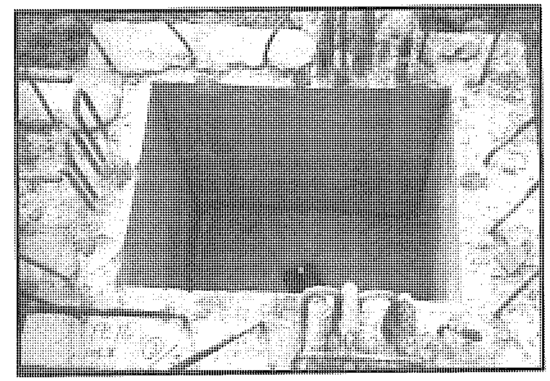
The column socket could be cast-on-site or it could be produced as a precast component. The column can be designed as a fixed-ended column.

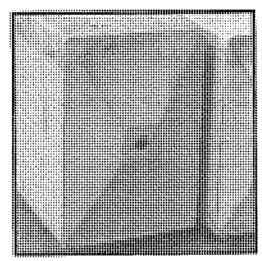
Figures 5.136, 5.137 and 5.138 Column Sockets

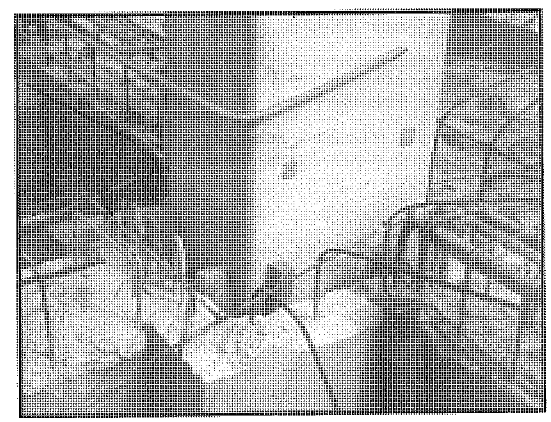
1. Thick cast-on-site socket walls
2. Precast column
3. Thin cast-on site socket wall with rough surfaces
4. Precast column with rough surface
5. Cast-in-situ rough joint
6. Ring reinforcement and stirrups
7. Precast column socket with very thin walls and fins
8. Precast column with castellated surface

Figures 5.136, 5.137 and 5.138 show the development of the column socket from very solid concrete boxes cast in-situ to more light precast structures with thin walls supported by fins.









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