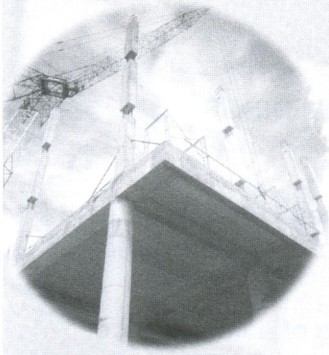


CHAPTER 4

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

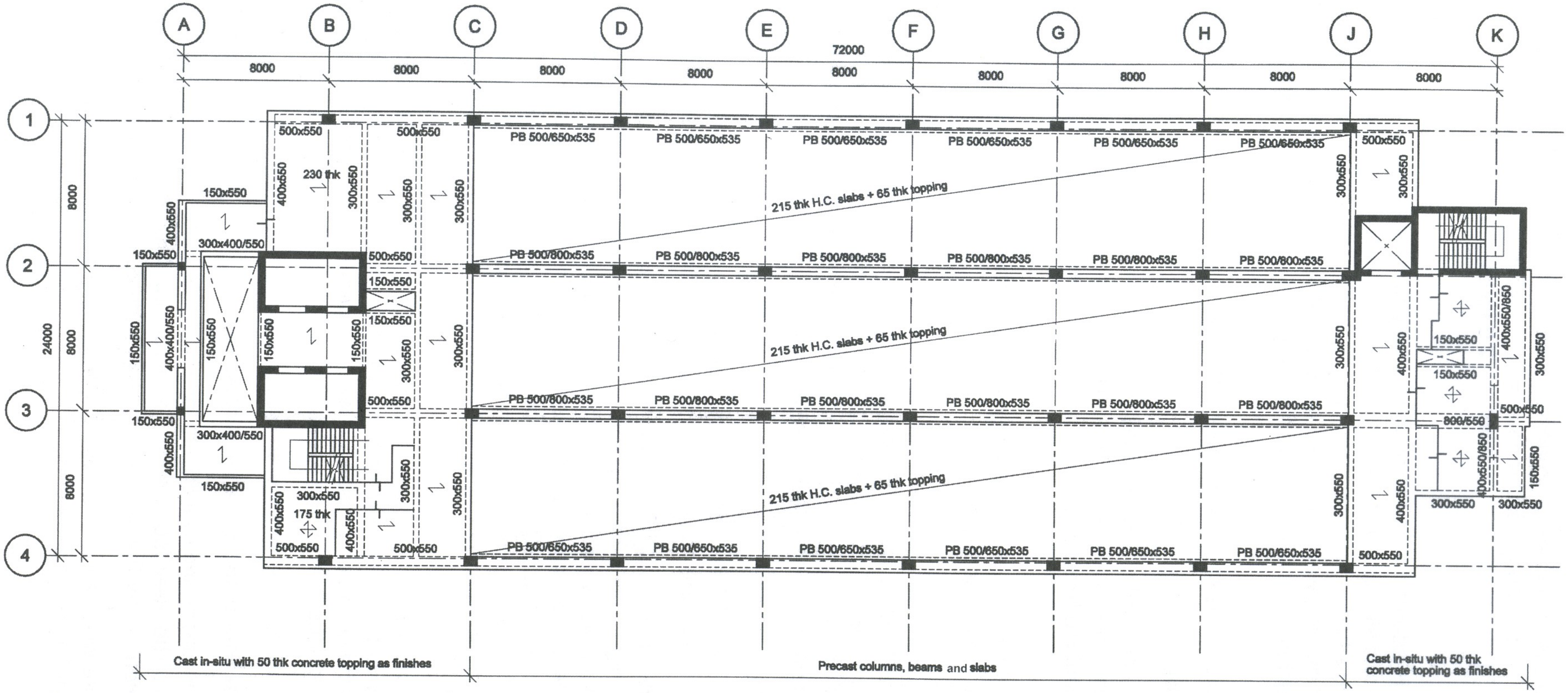
BS 6399	Design Loading for Building
CP 65	The Structural Use of Concrete
CP3, Chapter V	Wind Load

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/mm^2 mild steel reinforcement
	:	f_y = 460 N/mm^2 high yield steel reinforcement
	:	f_y = 485 N/mm^2 for steel fabric reinforcement

c. Dead loads

Concrete density	=	24	kN/m^3
Partitions, finishes and services	=	1.75	kN/m^2
Brickwalls	=	3.0	kN/m^2 in elevation



Notes :

1. All in-situ slab shall be 150mm thk, unless otherwise stated.
2. Column : 700x500
3. All wall : 300 thk

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.

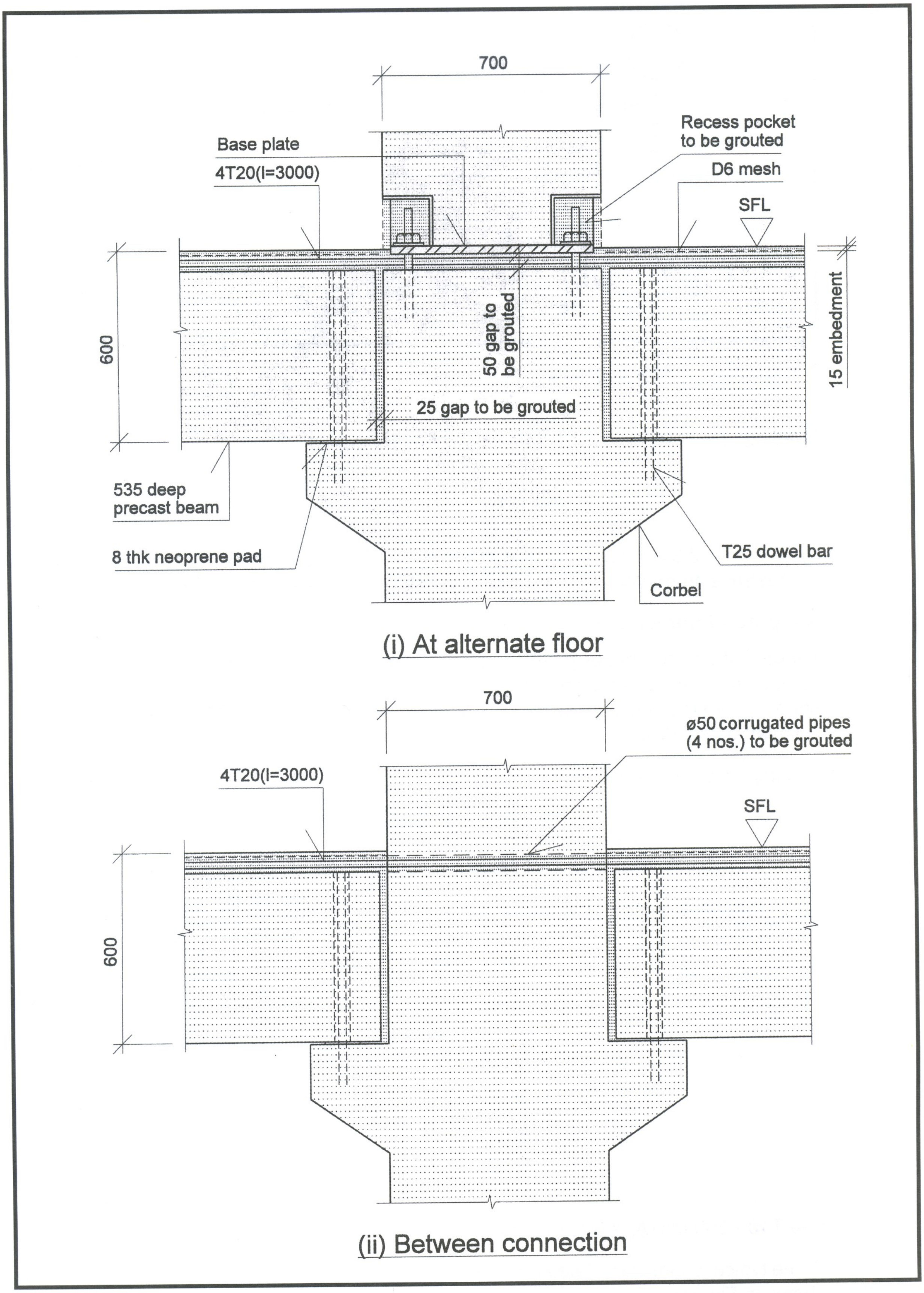
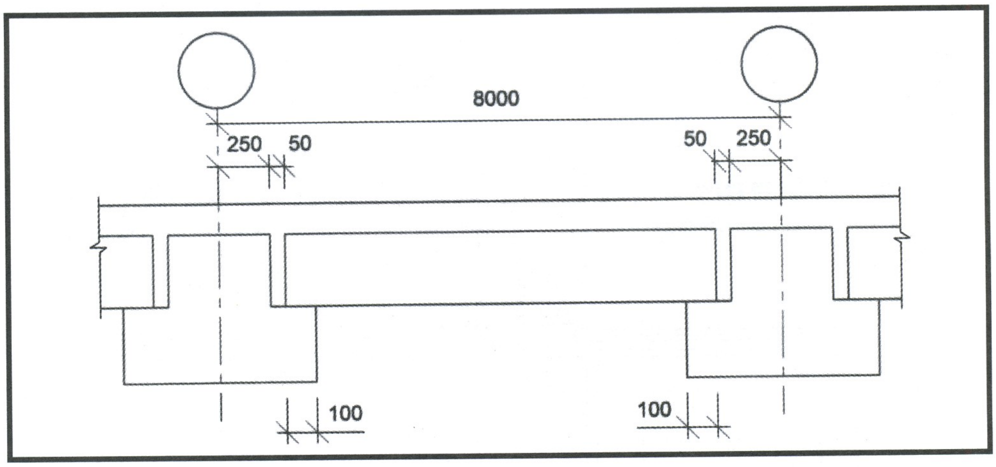


Figure 4.2 Typical Column/Column, Column/Beam Connections

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/m^2
Imposed = 1.75 kN/m^2
LL = 3.00 kN/m^2

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/m^2
LL = 3.00 kN/m^2

Ultimate load = 1.4 x 1.75 + 1.6 x 3.00
= 7.25 kN/m^2

Support moment = $0.083ql^2$
= 0.083 x 7.25 x 7.4^2
= 33.0 kNm/m

h = 215 + 65
= 280 mm

z ≈ 0.8h
= 224 mm

A_s = $M/(0.87f_y\, z)$
= $33.0 \times 10^6/(0.87 \times 460 \times 224)$
= 368 mm^2/m

Use T10 - 200 c/c (A_s = 393 mm^2/m)

Some typical connection details of hollow core slabs at the support and between units are shown in Figures 4.3 to 4.5.

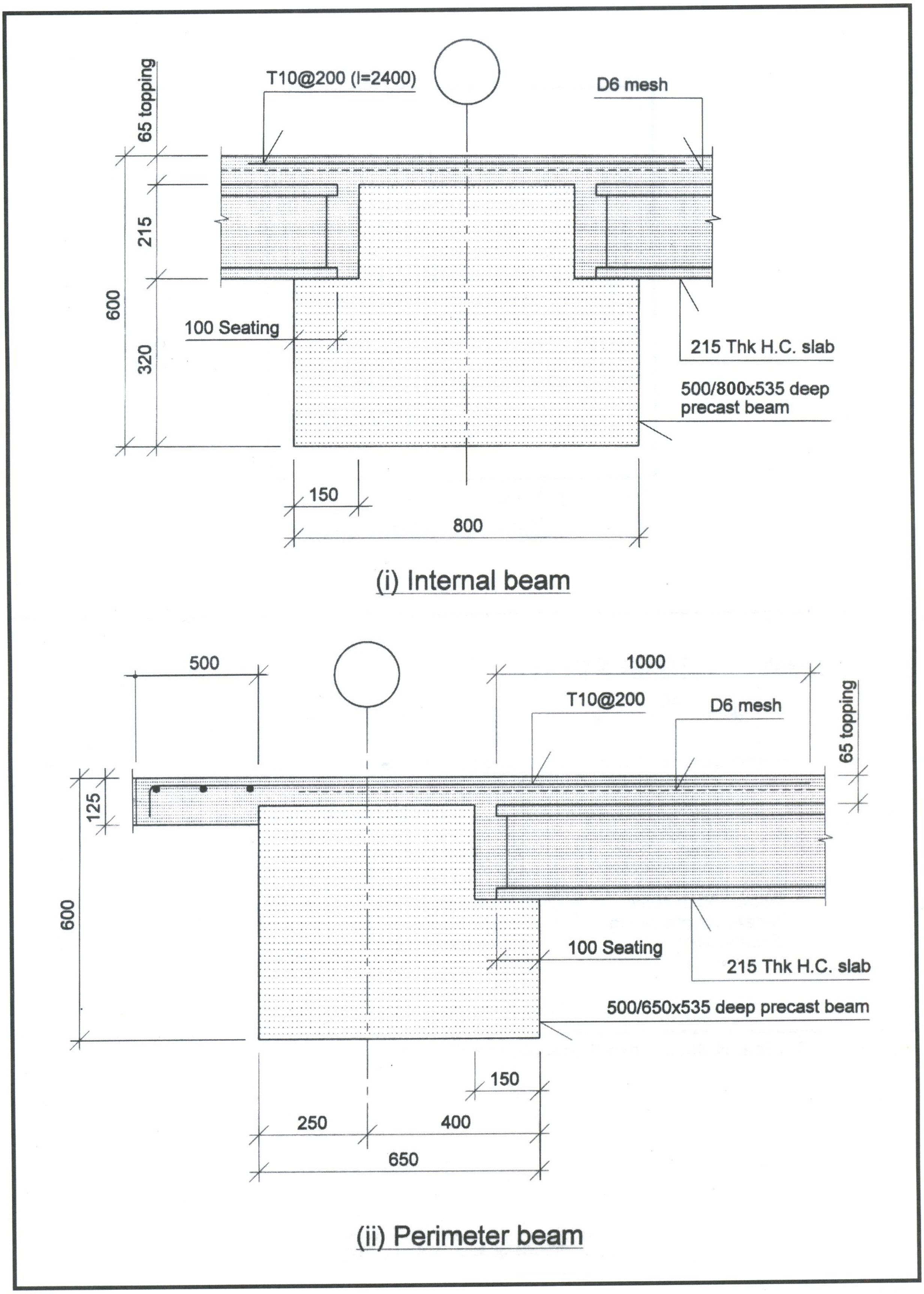


Figure 4.3 Typical Details At Hollow Core Slab Supports

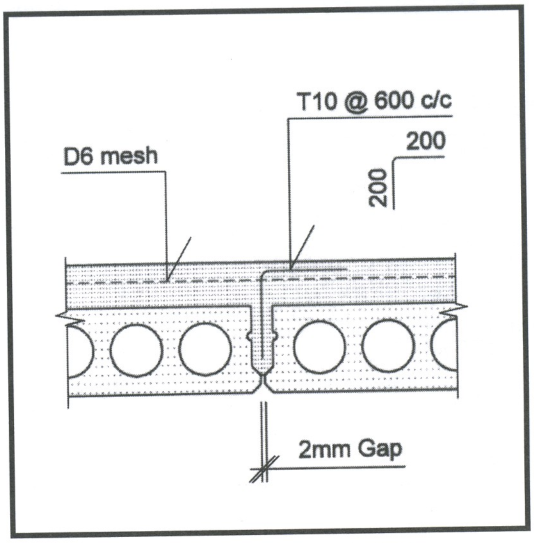


Figure 4.4 Typical Joint Details Between Hollow Core Slabs

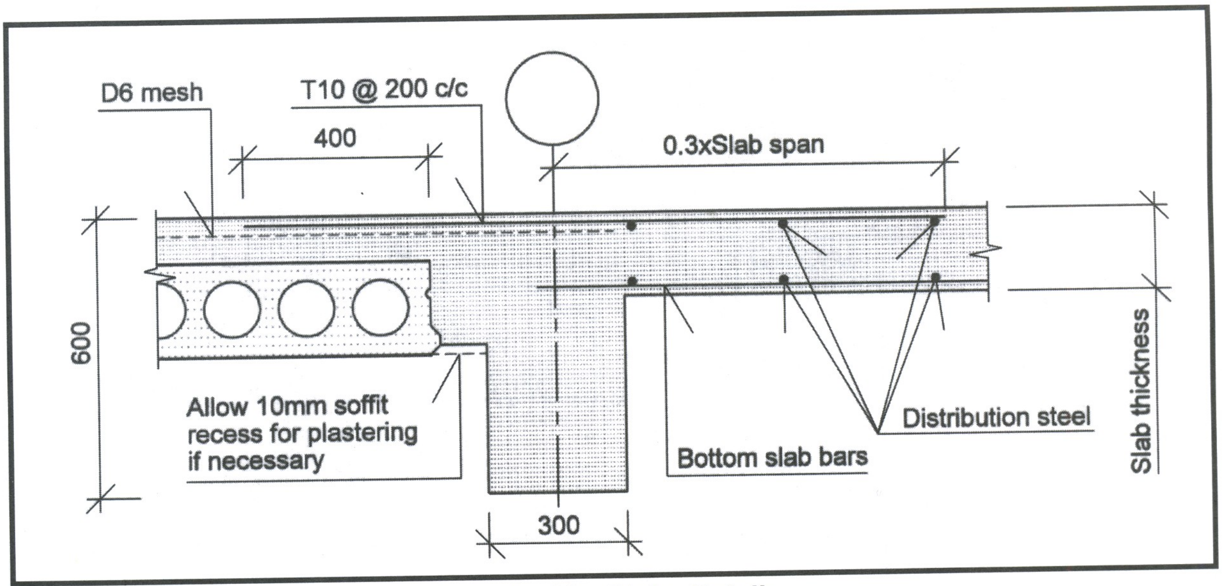
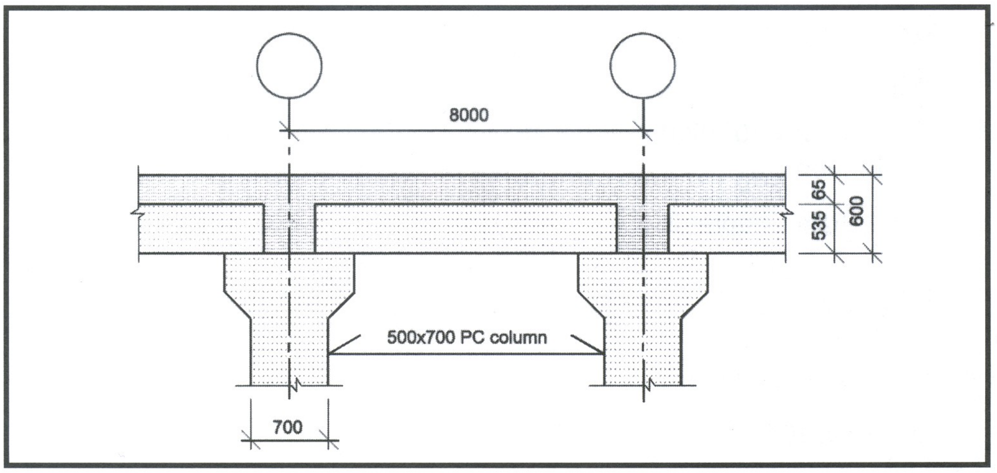


Figure 4.5 Typical Hollow Core Slabs/In-Situ Slabs Details

B. Design of precast beam



1. Design loading

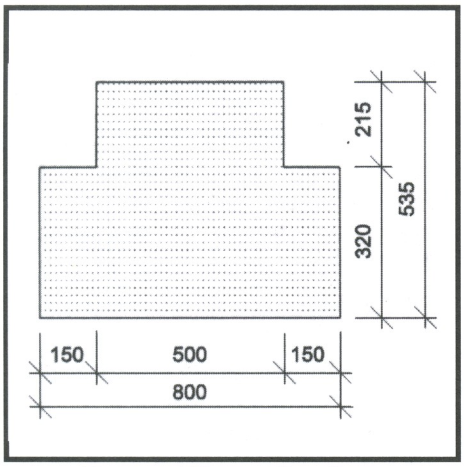
q = DL (beam s/w)	=	(0.8 x 0.32 + 0.215 x 0.5) x 24	=	8.72 kN/m
Hollow core slab (jointed wt)	=	2.89 x 7.5	=	21.67 kN/m
Topping	=	0.065 x 24 x 8	=	12.48 kN/m
Imposed	=	1.75 x 8	=	14.00 kN/m
		Total	=	56.87 kN/m
LL	=	3.0 x 8	=	24.00 kN/m

Design ultimate load = 1.4 x 56.87 + 1.6 x 24.00
= 118.03 kN/m

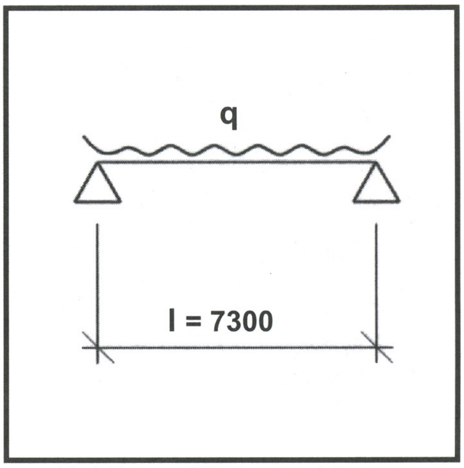
2. Limit state design

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

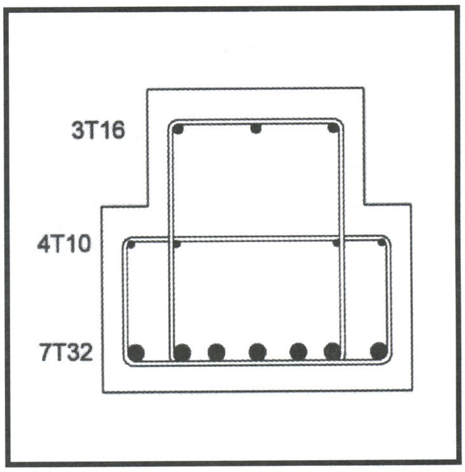
$b = 500$ mm

$M^+/(bd^2 f_{cu}) = 786.2 \times 10^6/(500 \times 460^2 \times 45)$
$= 0.165$

From Figure 2.26 in Chapter 2, $\rho_s/f_{cu} = 5.3 \times 10^{-4}$
$\rho_s = 5.3 \times 10^{-4} \times 45$
$= 0.0239$
$A_s = 5486$ mm^2

Provide 7T32 ($A_s = 5629$ mm^2)

$\rho_s'/f_{cu} = \text{min. } 4.4 \times 10^{-4}$
$\rho_s' = 0.02$
$A_s' = 460$ mm^2
Provide 3T16 ($A_s = 603$ mm^2) 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/mm^2

Minimum steel to be provided at the beam end to prevent bond slip failure is:

$A_s = V/0.87 f_y$
$= 430.8 \times 10^3/(0.87 \times 460)$
$= 1076$ mm^2
$= 0.47$ %

From Figure 2.27, in Chapter 2, $v_c = 0.59$ N/mm^2
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

$\nu = 265.6 \times 10^3 / (500 \times 460)$
$= 1.16$ N/mm^2

$A_{sv}/s_v = (1.16 - 0.59) \times 500 / (0.87 \times 460)$
$= 0.71$

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 $= 0.5 l_e$
$= 0.5 \times 7.3$
$= 3.65$ m

Average interface horizontal shear stress,
$\nu_h = 438.8 \times 10^3 / (500 \times 3650)$
$= 0.24$ N/mm^2
peak $\nu_h = 2 \times 0.24$
$= 0.48$ N/mm^2

which is less than 0.55 N/mm^2 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 $\approx 0.08 q l^2$
$= 0.08 \times 58.0 \times 7.3^2$
$= 247.3$ kNm

$A_s = M / (0.87 f_y z)$
$z \approx 0.8h$
$A_s = 247.3 \times 10^6 / (0.87 \times 460 \times 0.8 \times 600)$
$= 1287$ mm^2

Provide 4T20 ($A_s = 1257$ mm^2) 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/mm^2

From Part 1, Table 3.11, CP65 the modification factor for tension reinforcement = 0.75 and for compression reinforcement = 1.08

Minimum d $= 7300 / (20 \times 0.75 \times 1.08)$
$= 450.6$mm < 460 mm OK

f. End bearing

i. Effective bearing width

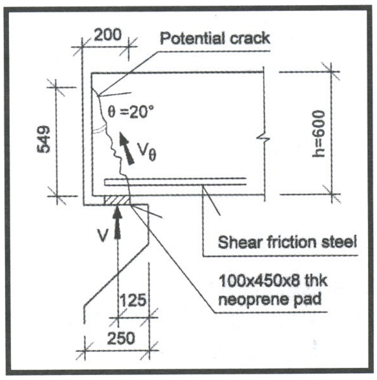
Permissible ultimate bearing stress from Part 1, clause 5.2.3.4, CP 65

$$= 0.4f_{cu}$$
$$= 0.4 \times 45$$
$$= 18 \text{ N/mm}^2$$

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



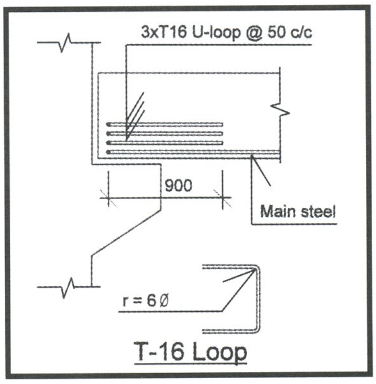
Bearing At Support

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 $= 0.87 \times 460 \times 201 \times 10^{-3}$

$= 80.4$ kN

Actual tension force per bar F_{bt} $= 80.4 \times 1076/1207$

$= 71.7$ kN

$a_b = 50$ mm

$\phi = 16$ mm

Minimum r $= F_{bt}[1 + 2(\phi/a_b)] / 2 f_{cu}\phi$ (Equation 50 Part 1)

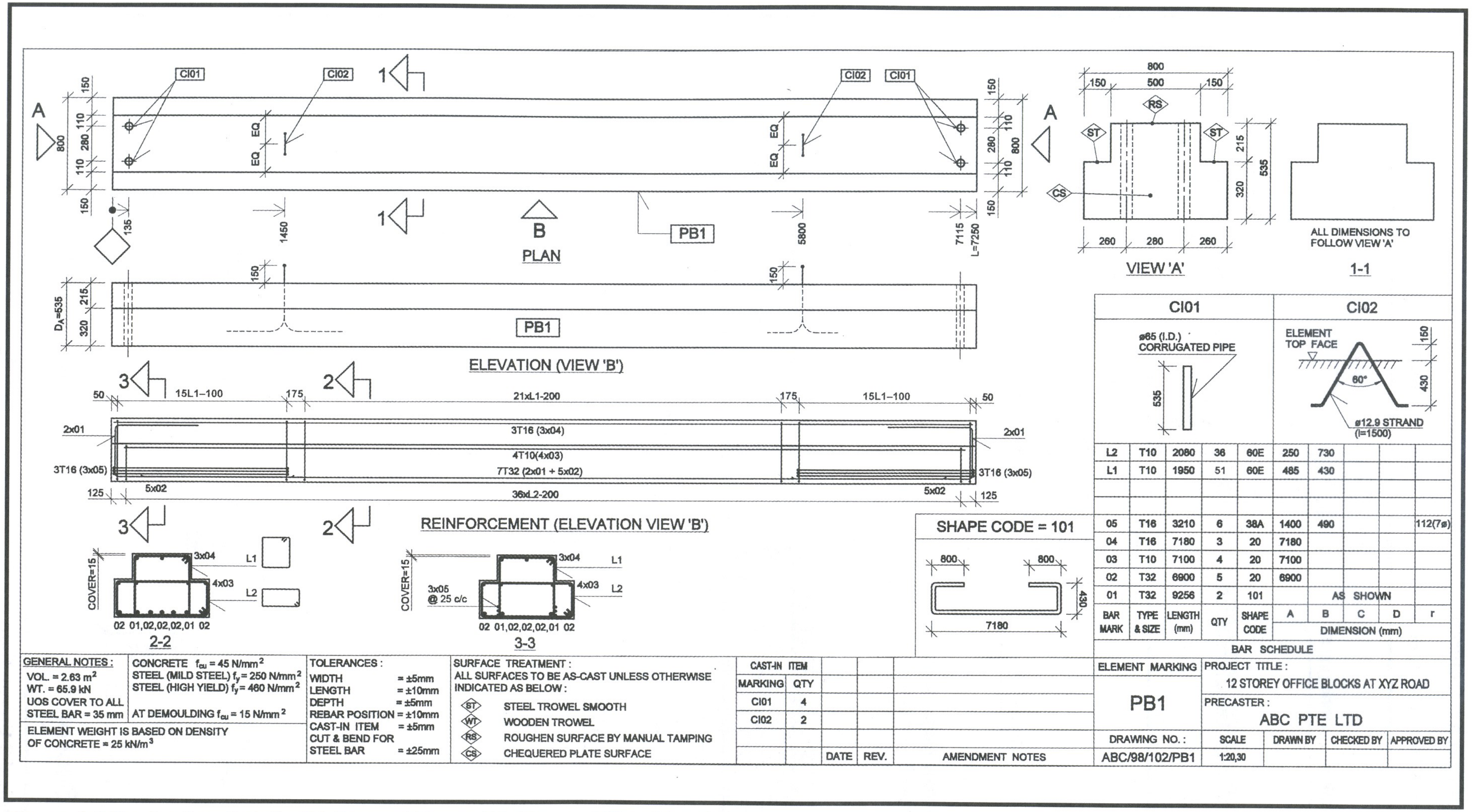
$= 71.7 \times 10^3[1 + 2(16/50)] / (2 \times 45 \times 16)$

$= 82$ mm

Say r $= 6\phi$ (96 mm)

g. Production Details

Typical production details of the precast beam are shown in Figure 4.6.



BAR MARK	TYPE & SIZE	LENGTH (mm)	QTY	SHAPE CODE	A	B	C	D	r
L2	T10	2080	36	60E	250	730			
L1	T10	1950	51	60E	485	430			
05	T16	3210	6	38A	1400	490			112(7ø)
04	T16	7180	3	20	7180				
03	T10	7100	4	20	7100				
02	T32	6900	5	20	6900				
01	T32	9256	2	101	AS SHOWN				

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:

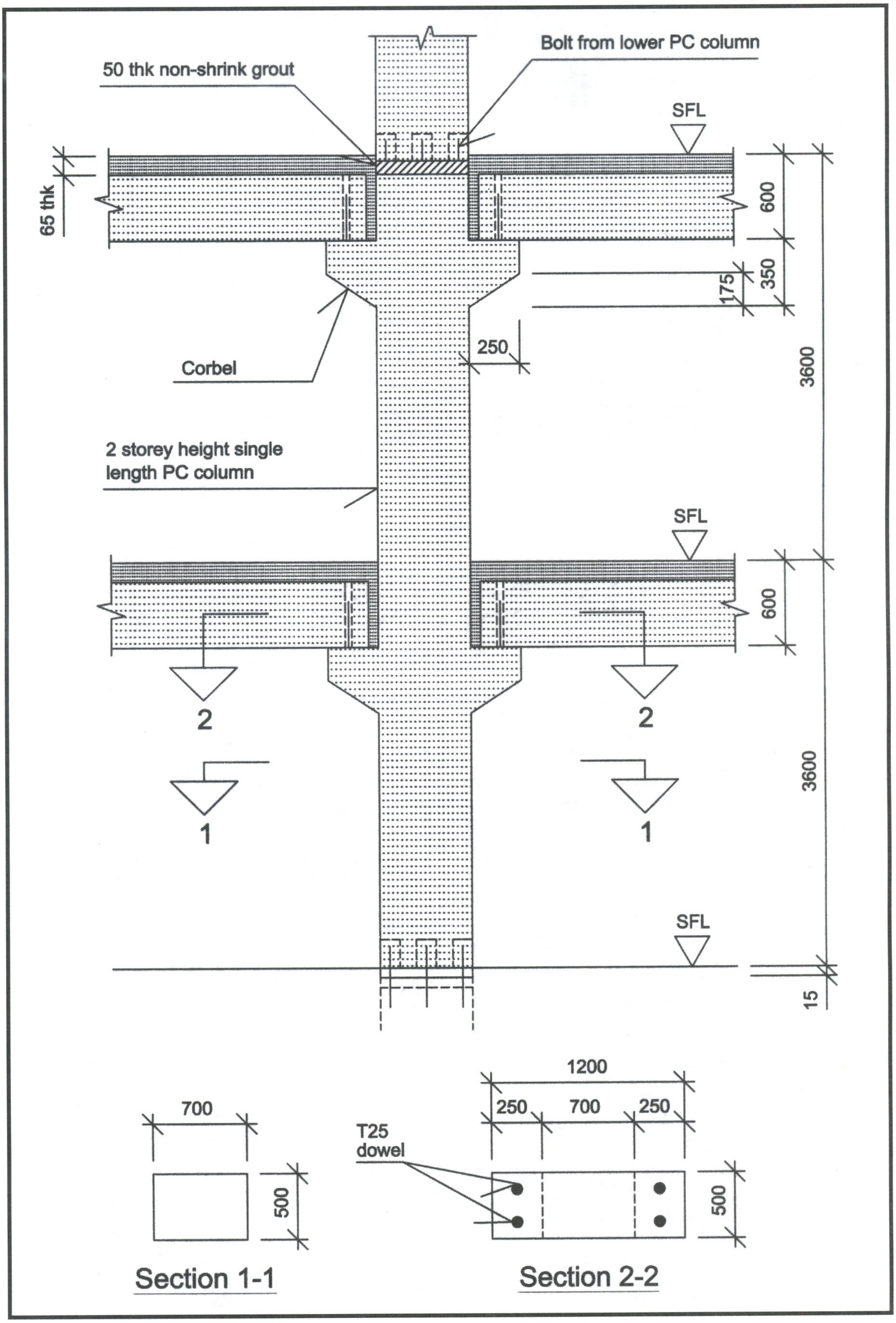


Figure 4.7 Typical 2 Storey High Precast Columns

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 2nd storey (12 floors), total column load (above 1st 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/mm^2

$A_{sc} = (9919.0 \times 10^3 - 0.4 \times 50 \times 500 \times 700) / (0.75 \times 460)$
$= 8461$ mm^2

Provide 12T32 (A_{sc} = 9650 mm^2, 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/mm^2,

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

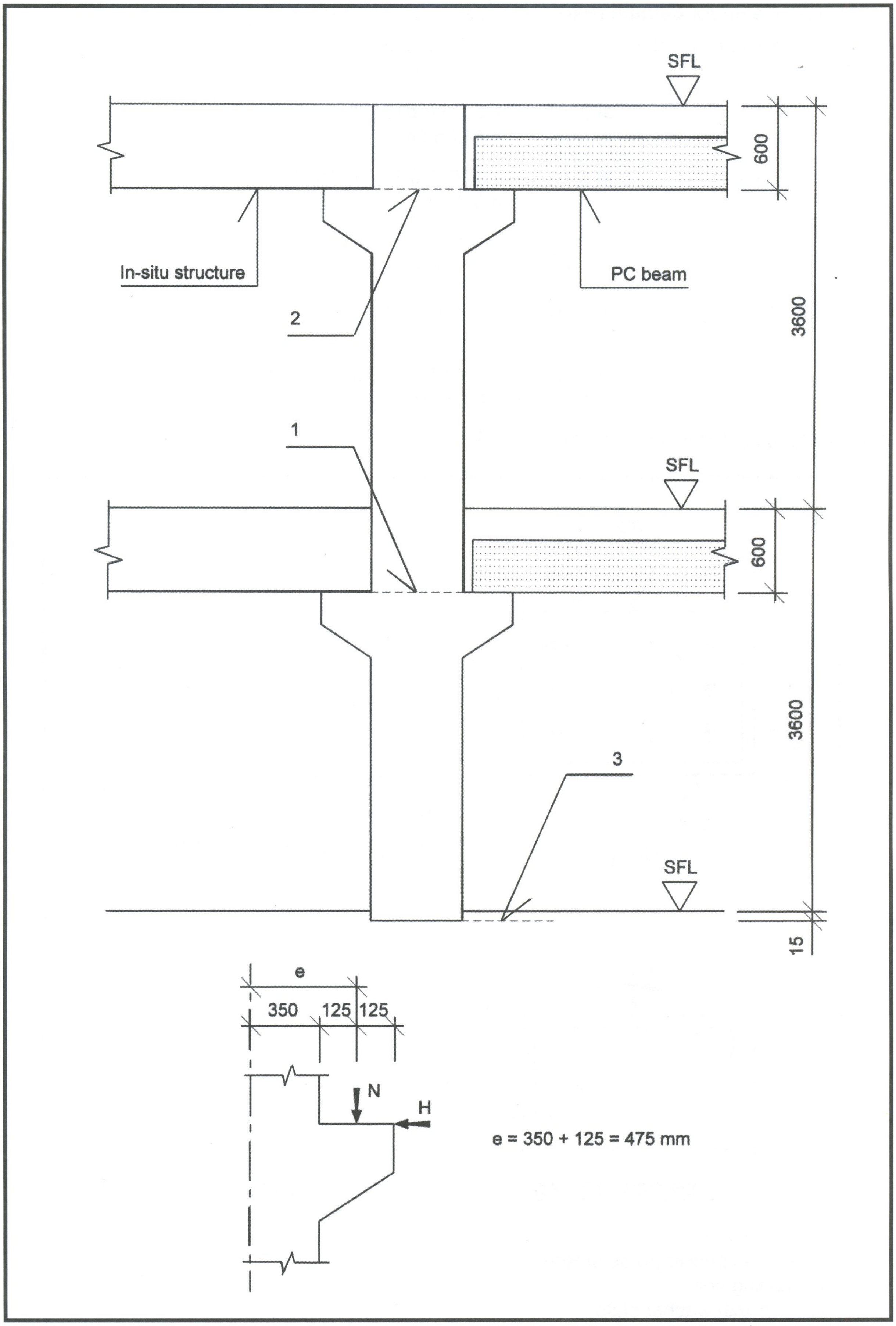
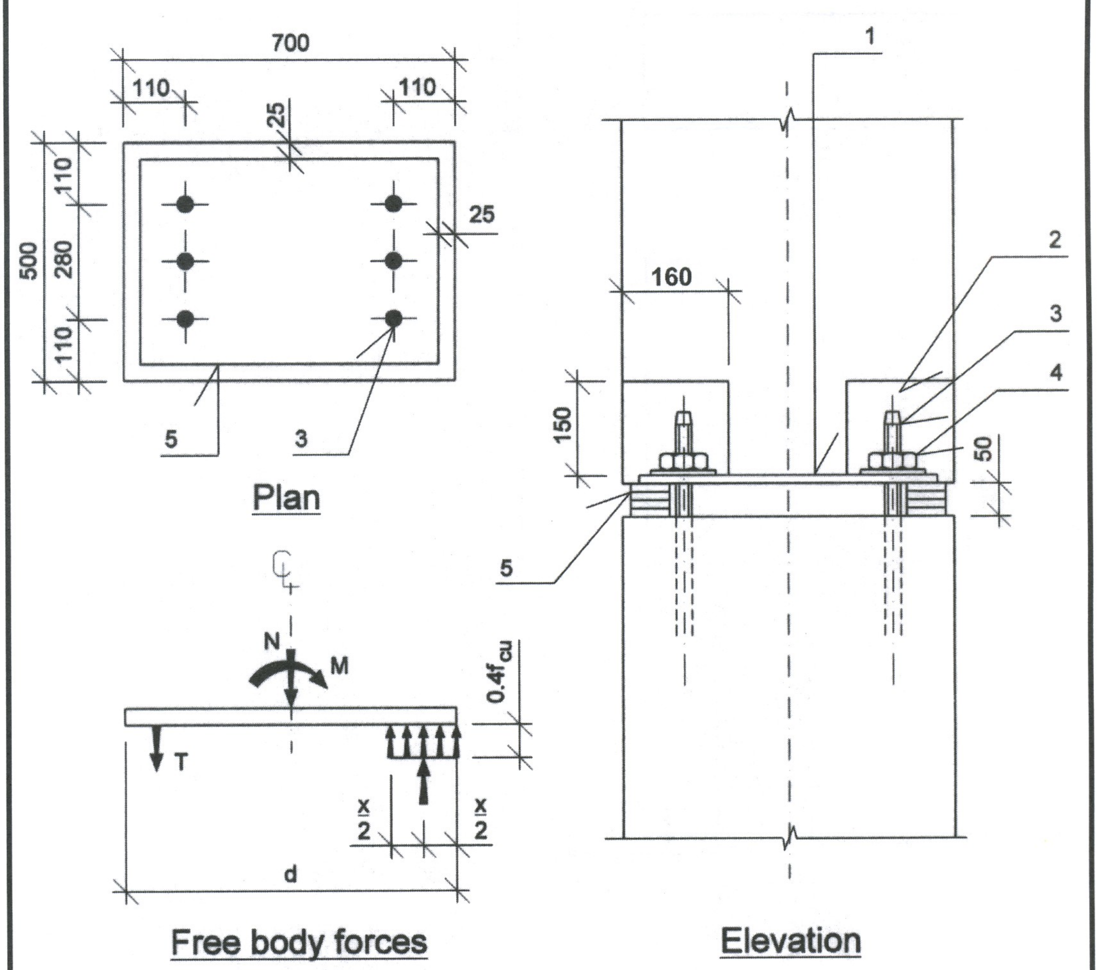


Figure 4.8 Loadings At Edge Column

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

$$T = 363.7/3$$
$$= 121.2 \text{ kN}$$

$$\text{Bolt area } A_s \text{, required} = 121.2 \times 10^3/(0.87 \times 460)$$
$$= 303 \text{ mm}^2$$

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:

$$f_{bu} = \beta\sqrt{f_{cu}}$$
$$\beta = 0.50$$
$$f_{bu} = 0.5\sqrt{50}$$
$$= 3.53 \text{ N/mm}^2$$

$$\text{Bolt perimeter} = \pi d$$
$$= \pi \times 32$$
$$= 100.5 \text{ mm}$$

$$\text{Tension} = 121.2 \text{ kN}$$

$$\text{Hence anchorage length } l_p = 121.2 \times 10^3/(3.53 \times 100.5)$$
$$= 342 \text{ mm}$$

$$\text{Say } l_p = 400 \text{ mm}$$

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:

$$t = \sqrt{0.8 f_{cu}\, a^2/p_y}$$

$$a = 50 + 85 = 135 \text{ mm}$$
$p_y = 265 \text{ N/mm}^2$ (grade 43 steel, assuming plate thickness > 16mm)

$$\text{Hence } t = \sqrt{0.8 \times 50 \times 135^2/265}$$
$$= 52.4 \text{ mm}$$

Based on tension side:

$$t = \sqrt{4Tc/b\, p_y}$$

$$T = 363.7 \text{ kN}$$
$$c = 50$$
$$b = 450$$

$$t = \sqrt{4 \times 363.7 \times 10^3 \times 50/(450 \times 265)}$$
$$= 24.7 \text{ mm}$$

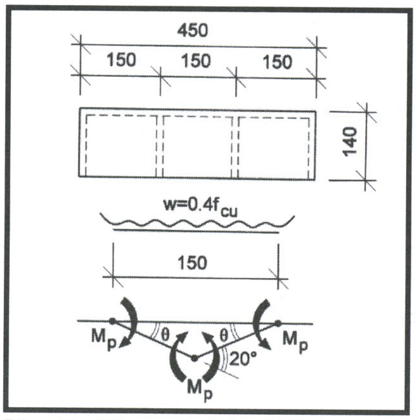
Minimum plate thickness = 51.5 mm. However, this is considered excessive. Instead, 4 numbers of stiffener plates are added to dissipate the compression from the base plate to the column section at the top of the block-out.

Total compression = $0.4f_{cu}\, b\, \chi$ (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
= $w \times 140 \times 150 / 2$
= $0.4 \times 50 \times 140 \times 150 / 2$
= 2.10×10^5

Energy spent in plastic hinges = $M_p\theta + 2M_p\theta + M_p\theta$
= $4M_p\theta$

$M_p = p_y (t^2 \times 100) / 4 = 35 p_y t^2$
$\theta = 1/75$
$p_y = 265$ N/mm²

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 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

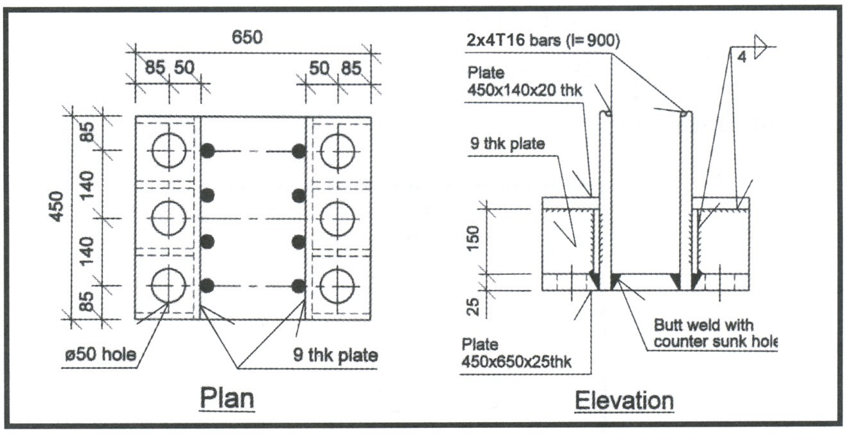
iii. Anchorage of base plate to column

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 x 480.7 + 0.33 x 192.0

= 568.1 kN

Vertical tie reinforcement A_s = 568.1 x 10^3/460

= 1235 mm^2

Total provision of bolt (root area) = 362 x 6

= 2172 mm^2 > 1235 mm^2 OK

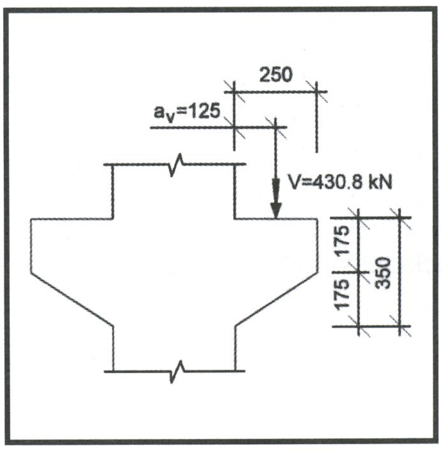
Total provision of welded bars to base plate

= 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

ν = V/bd

= 430.8 x 10^3/(500 x 275)

= 3.13 N/mm^2 < 0.8$\sqrt{f_{cu}}$ = 5.66 N/mm^2

adopt max. ν_c = 5.00 N/mm^2 OK

b. Reinforcement

i. Main steel

$$\nu/f_{cu} = \frac{0.9(z/d)(a_v/d)(1 - z/d)}{(a_v/d)^2 + (z/d)^2}$$

$\nu/f_{cu} = 3.13 / 50 = 0.063$
$a_v/d = 125 / 275 = 0.454$

Hence $(z/d)^2 - 0.866(z/d) + 0.0276 = 0$
$z/d = 0.833$

Steel stress $f_s = 200\ (z/d - 0.55) / (1 - z/d)$
$= 200\ (0.833 - 0.55) / (1 - 0.833)$
$= 339\ \text{N/mm}^2$

$F_t = V \times a_v/z$
$= 430.8 \times 125 / (0.833 \times 275)$
$= 235.1\ \text{kN} > V/2$

$A_s = F_t/f_s$
$= 235.1 \times 10^3 / 339$
$= 694\ \text{mm}^2$

Provide 3T13 U-loop (6 legs, $A_s = 796\ \text{mm}^2$) at 25 c/c.

ii. Check bearing stress within bend

Tension per leg of T13 bar, $F_t = 235.1 \times 694 / (6 \times 796)$
$= 34.2\ \text{kN}$

Minimum bending radius r $= \dfrac{F_t}{\phi} \times \dfrac{1 + 2(\phi/a_b)}{2f_{cu}}$

$F_t = 34.2\ \text{kN}$
$\phi = 13\text{mm}$
$a_b = 25\text{mm}$

$r = 34.2 \times 10^3 \times [1 + 2(13/25)] / (13 \times 2 \times 50)$
$= 53.6\ \text{mm}$

Use $r = 5\phi$

$= 65\text{mm}$

iii. Shear links

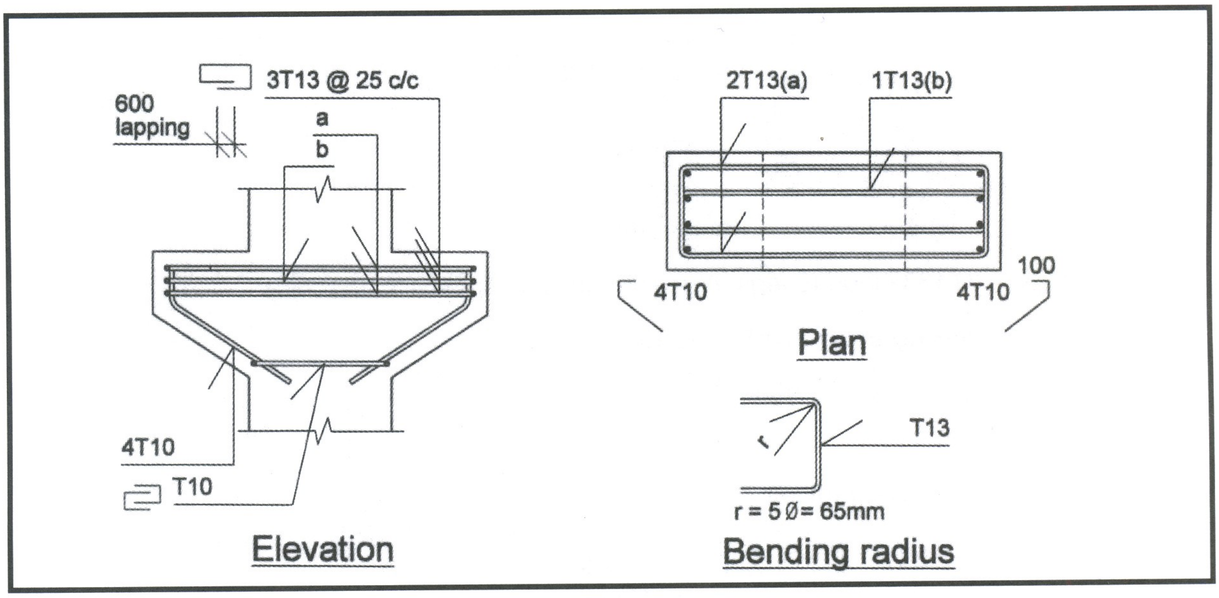
$$\rho_s = 100A_s/bd$$
$$= 100 \times 796/(500 \times 275)$$
$$= 0.58\%$$

$$\nu_c = 0.66 \times 1.10$$
$$= 0.73 \text{ N/mm}^2$$

Enhanced $\nu_c' = 2d \times \nu_c / a_v$
$$= 2 \times 275 \times 0.73/125$$
$$= 3.21 \text{ N/mm}^2 > 3.13 \text{ N/mm}^2$$

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.

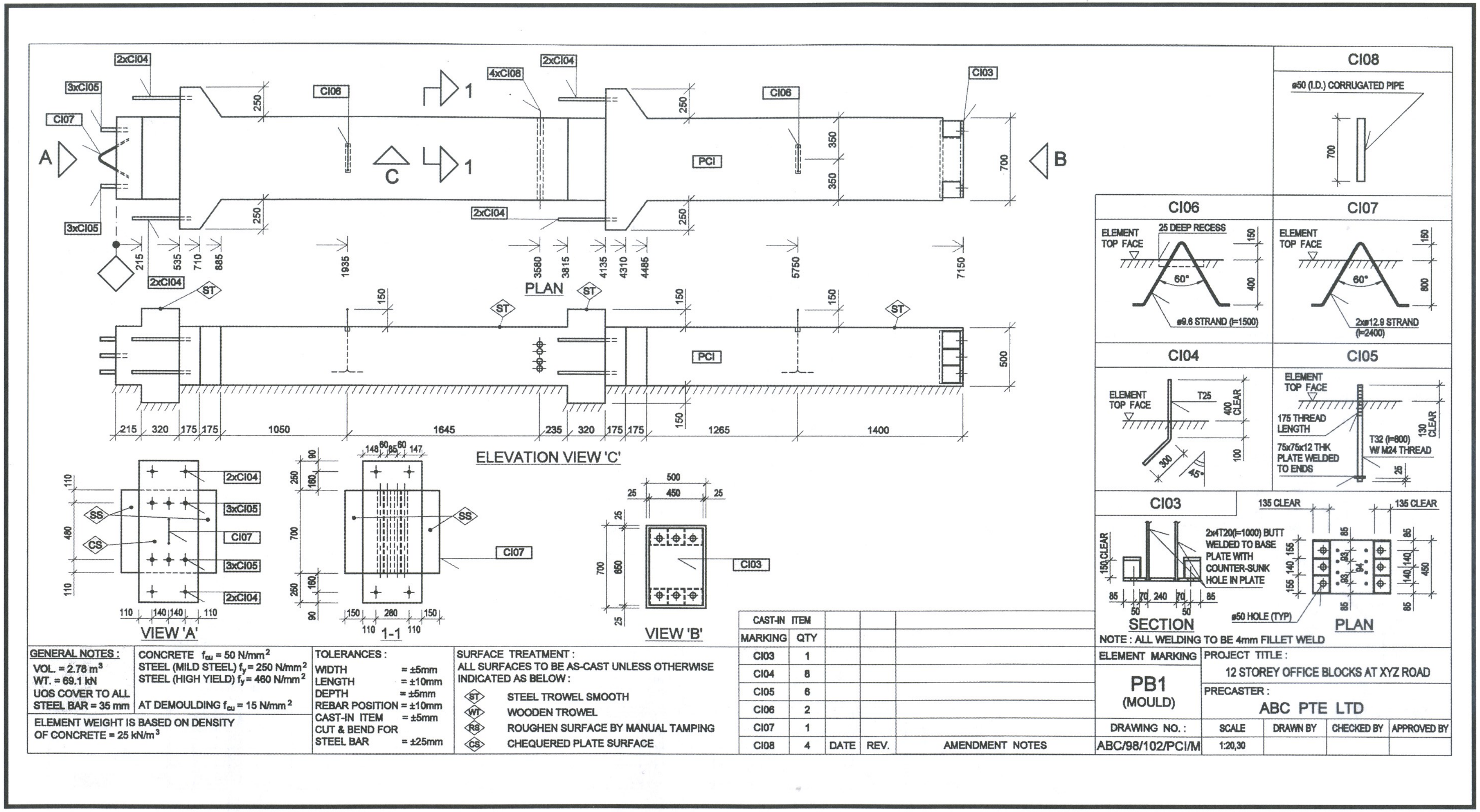


Figure 4.9 Mould Details Of Precast Column (12-Storey Office Block)

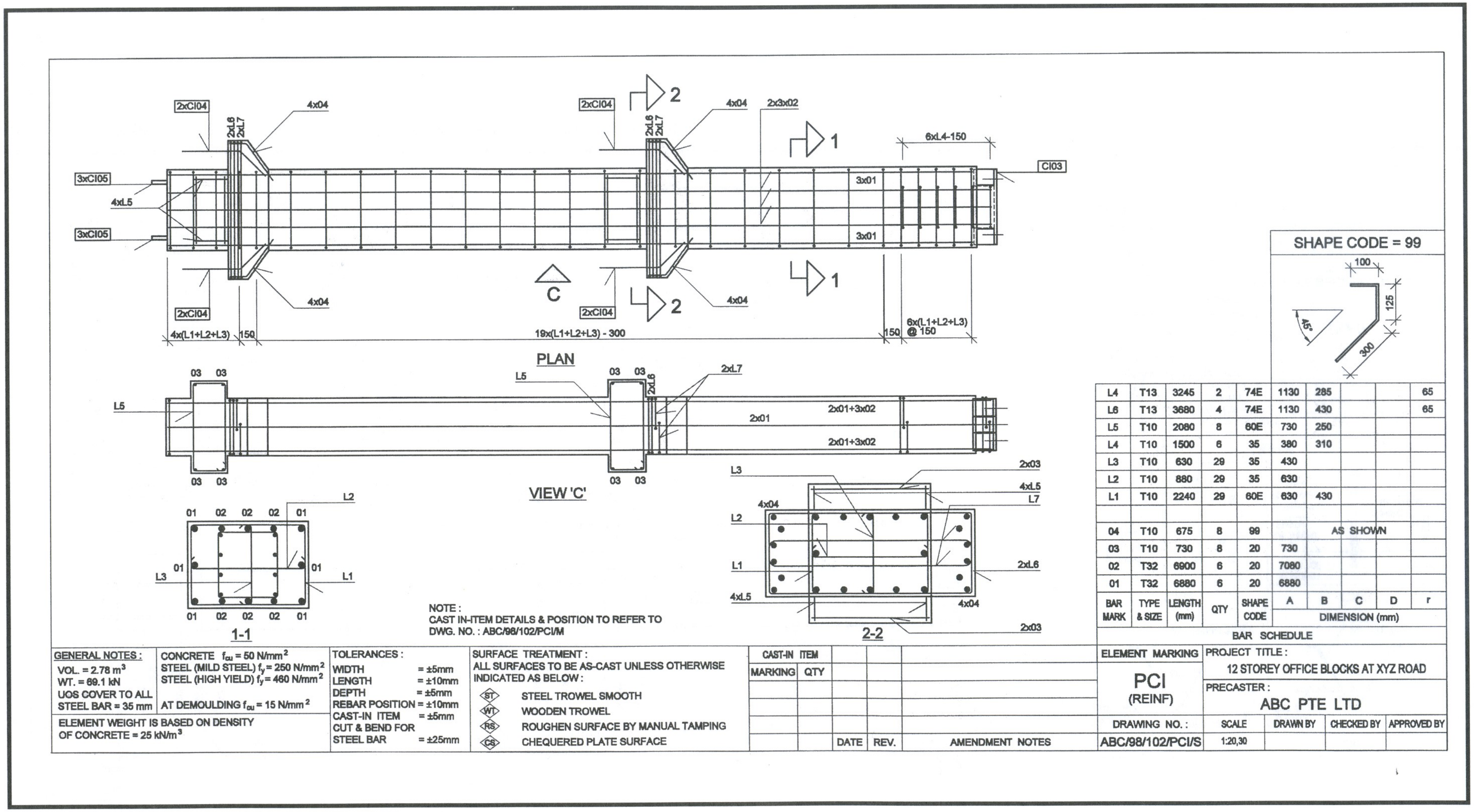
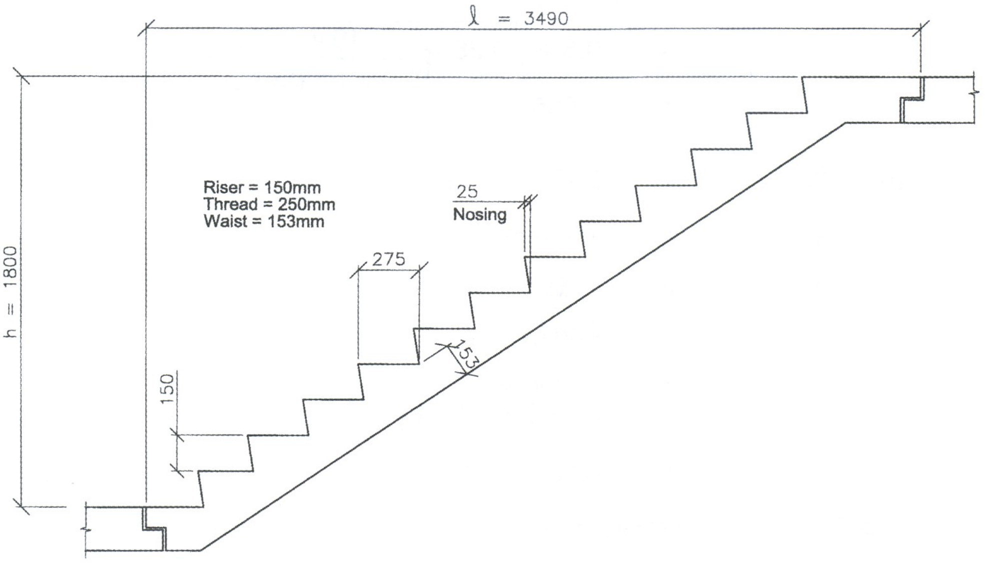


Figure 4.10 Reforcement Details Of Precast Column (12-Storey Office Block)

D. Precast Staircase Flight



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 x 3.49/8
= 19.41 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 $A_s = 523 \text{ mm}^2/\text{m}$

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

$$ø = 0.55 + \frac{(477 - f_y)}{120\ (0.9 + M/bd^2\ )}$$

$$= 0.55 + \frac{477 - 234}{120(0.9 + 1.35)}$$

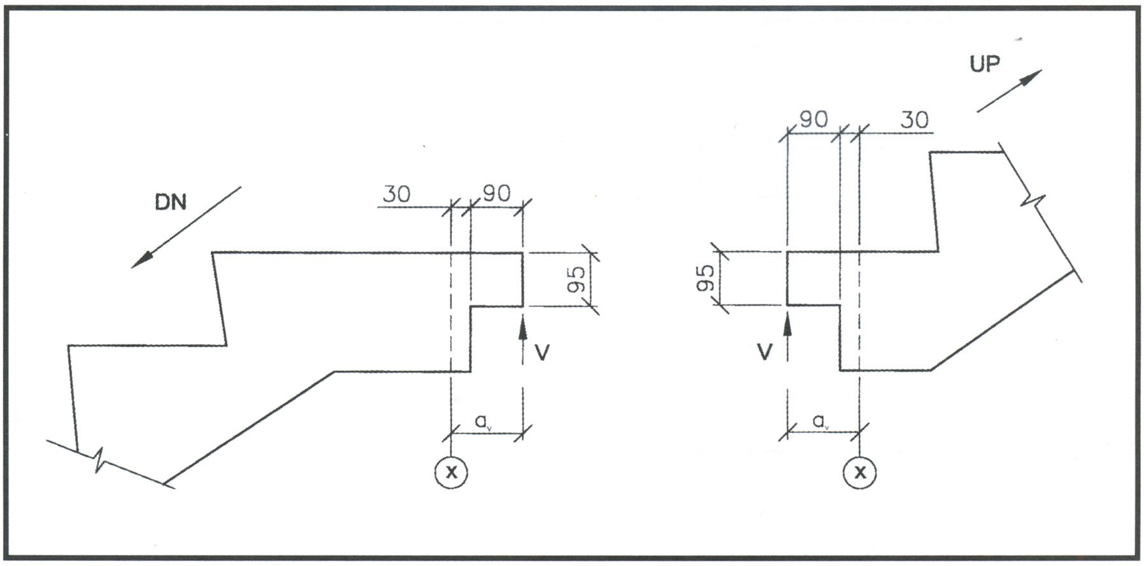
CI.3.10.2.2, Part1, CP 65, basic span/depth ratio of staircase flight $= 20 \times 1.15$
$= 23$

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 x a_v
= 22.25 x 0.12
= 2.67kNm/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

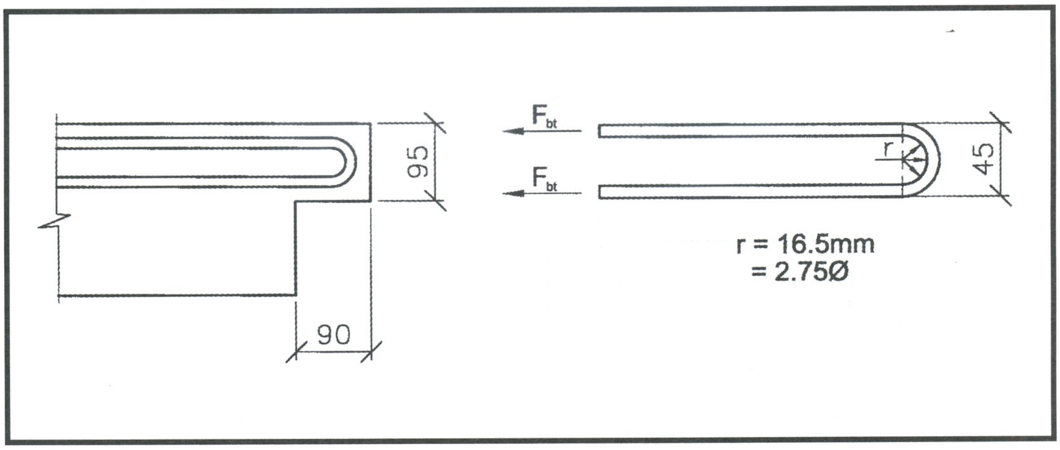
Use mild steel: $A_s = \frac{M}{0.87f_yz}$

$$= \frac{2.67 \times 10^6}{0.87 \times 250 \times 66.5}$$

$= 185$ mm^2/m

use R6 @150 c/c (A_s = 188 mm^2/m)

Check anchorage



F_{bt} = $0.87 f_y A_s$
= $0.87 \times 250 \times 28 \times 10^{-3}$
= 6.09 kN

Minimum bending radius

$$r \geq \frac{F_{bt}}{\varnothing} \times \frac{1 + 2(\varnothing/a_b)}{2f_{cu}}$$

ø = 6 mm
a_b = 150 mm
f_{cu} = 40 N/mm^2

$$r \geq \frac{6.09 \times 10^3}{6} \times \frac{1 + 2(6/150)}{2 \times 45}$$

$\geq$ 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%

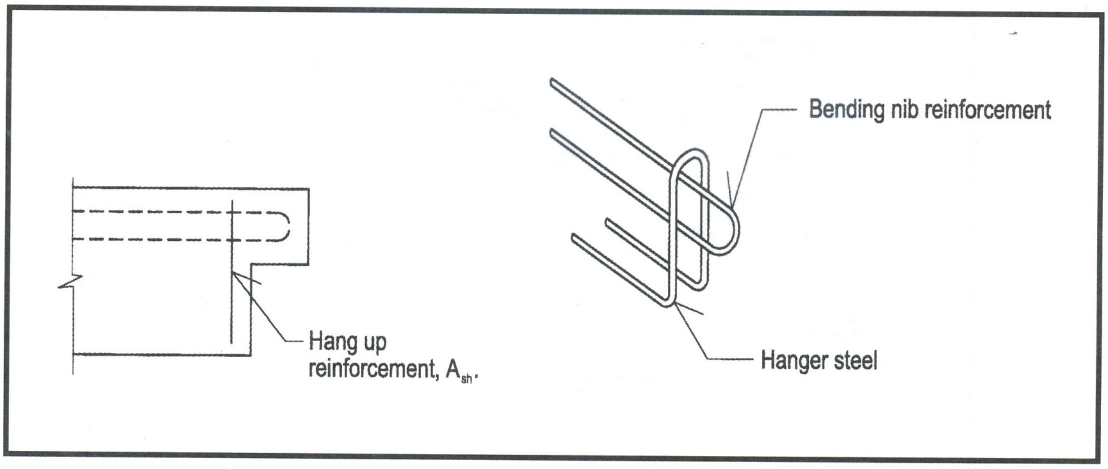
$$v_c = \frac{0.84}{\gamma_m} (r_s)^{1/3} (400/d)^{1/4} (f_{cu}/30)^{1/3}$$

γ_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

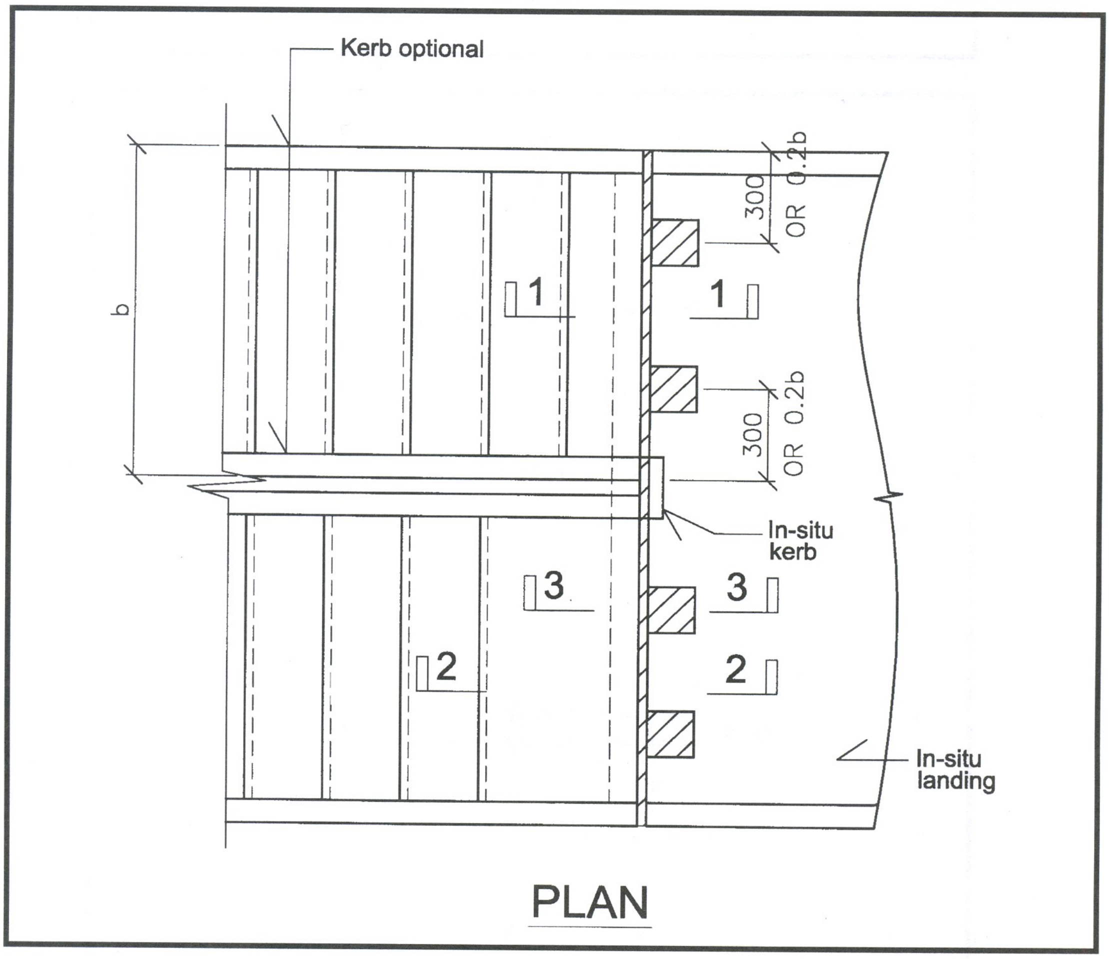


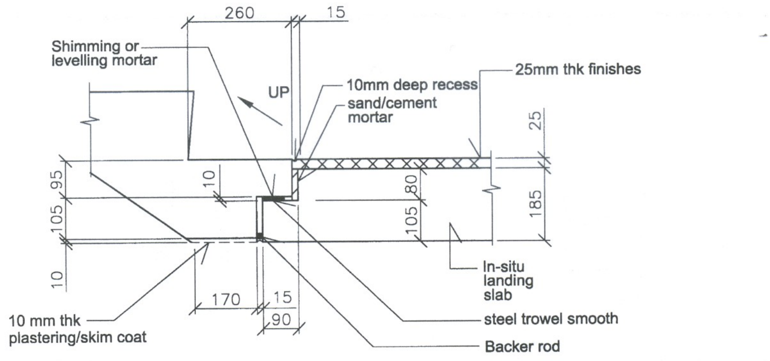
$$A_{sh} = \frac{V}{0.87f_y}$$

$$= \frac{22.25 \times 10^3}{0.87 \times 250}$$

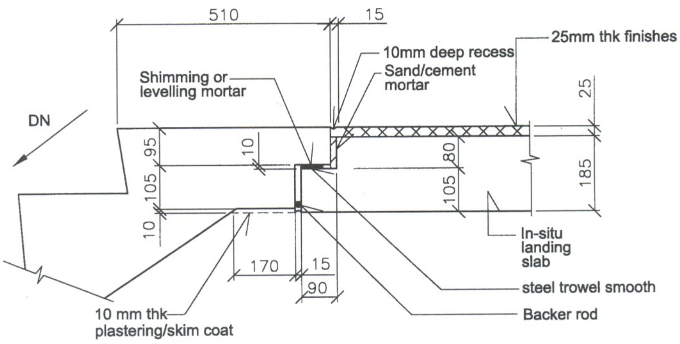
$$= 102 \text{ mm}^2/\text{m}$$

For practical reason, provide at every bending R6 reinforcement a looped R6 as hanger steel as above.

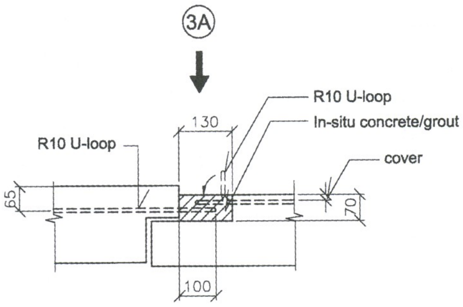




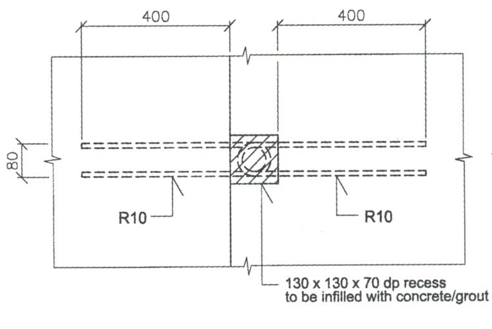
SECTION 1-1



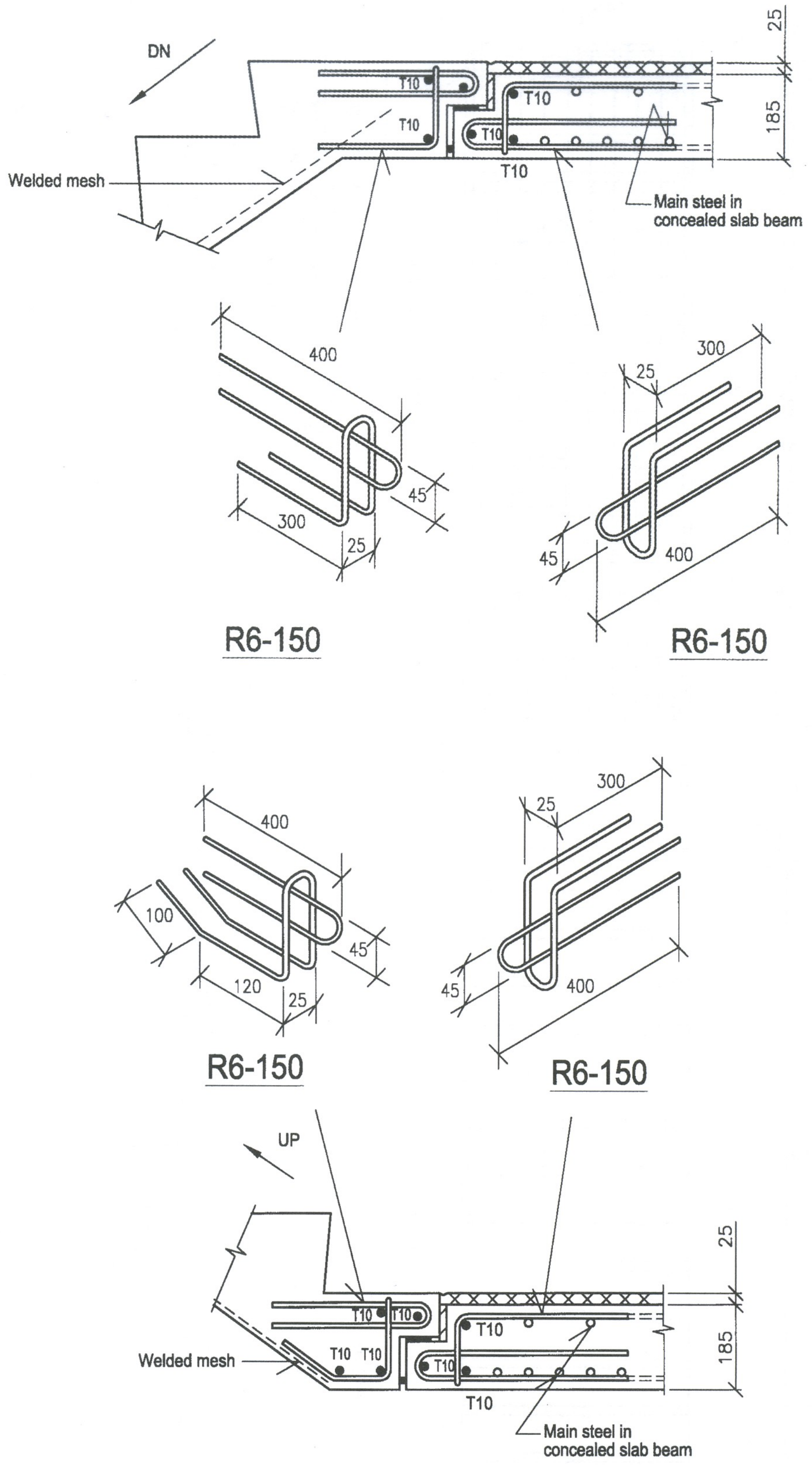
SECTION 2-2



SECTION 3-3



TOP VIEW 3A



NIB REINFORCEMENT DETAILS

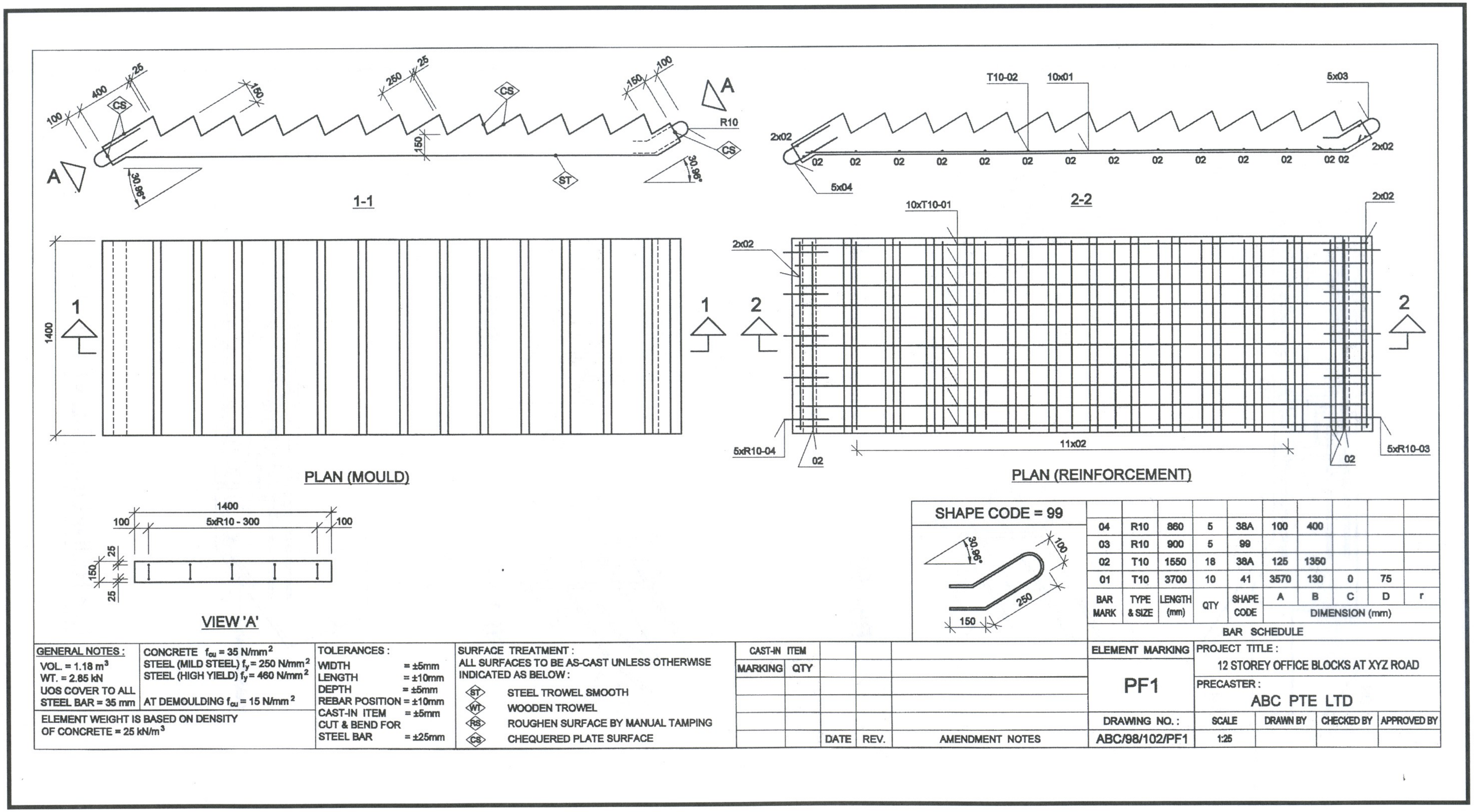


Figure 4.12 Mould And Reinforcement Details Of Precast Flight (12-Storey Office Block)

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	= <u>54.5kN</u>
		331.9kN
Live load	= 1.5 x 25	= 37.5kN

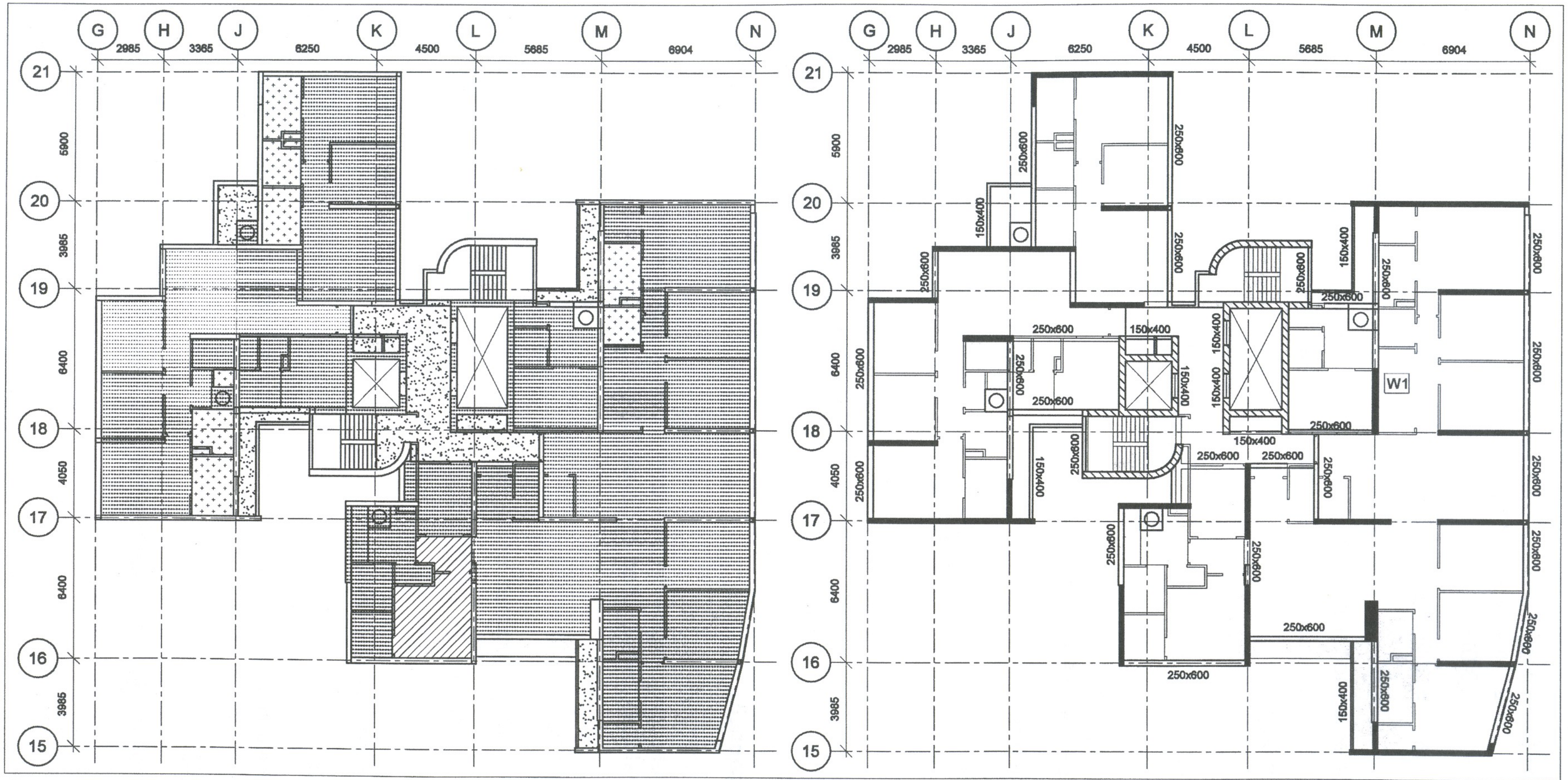
Above 1st storey

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
= <u>900 kN</u>

Total N = 14839.8 kN

Say N = 14840 kN
n_w = 14840/3 = 4947 kN/m



Tower 4 floor plan, slabs. scale 1:175

Tower 4 floor plan, Beams and walls. scale 1:175

175 Thk slab
125 Thk slab
150 Thk slab
225 Thk slab

In-situ wall
Precast wall

Figure 4.13 Typical Floor Layout

b. Vertical load capacity

Effective wall height l_e $= 1.0l_o$
$= 1.0 \times (3.2 - 0.175)$
$= 3.025$ 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} $= (4947 \times 10^3 - 0.35 \times 45 \times 1000 \times 250) / (0.67 \times 460)$
$= 3275$ mm²/m

Per face = 1638 mm²/m
Use T16 -125 c/c per face ($A_s = 1675$mm²/m)

c. Vertical structural ties

Vertical tie force per floor $= 1.05 \times 331.9 + 0.33 \times 37.5$
$= 360.9$ kN

Tension steel area A_s $= 360.9 \times 10^3/(460)$
$= 785$ mm²

Use 3 numbers of T20 ($\Sigma A_s = 943$ mm²) 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 :

$f_{cu} = 4947 \times 10^3 / (0.6 \times 0.75 \times 1000 \times 250)$
$= 44$ N/mm²

Use $f_{cu} = 45$ N/mm² 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.

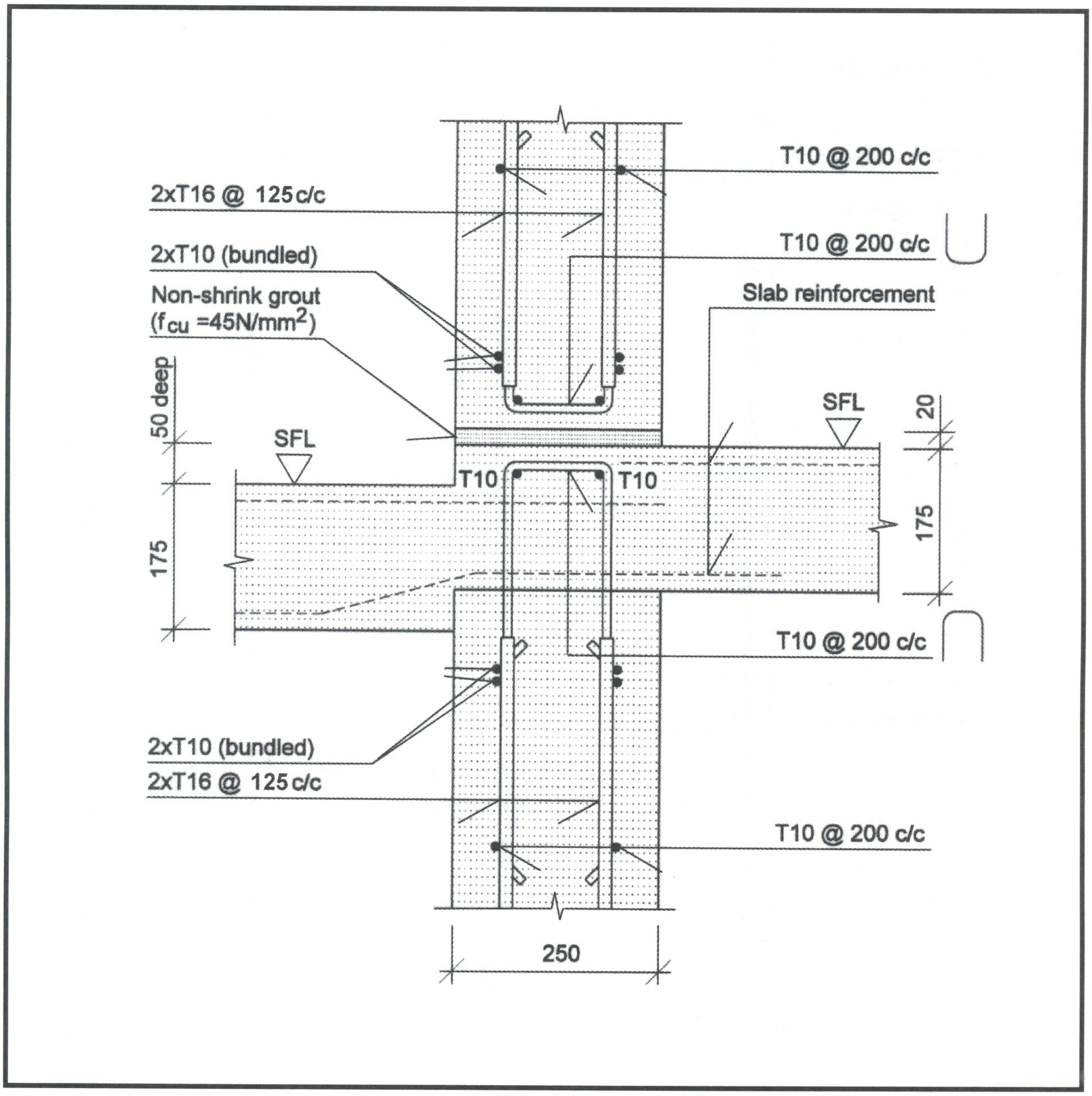


Figure 4.14 Typical Wall / Floor Joint

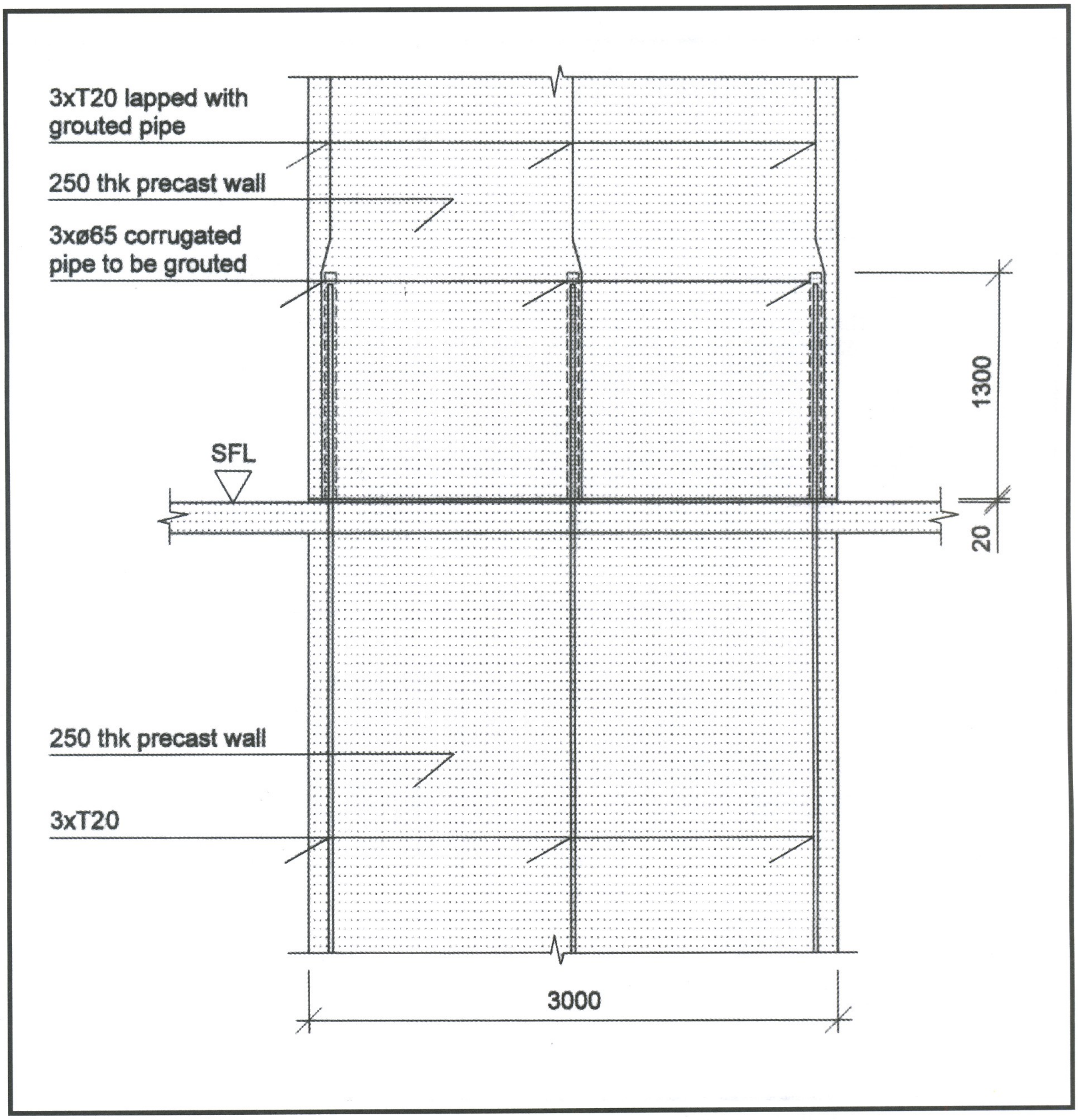


Figure 4.15 Vertical Ties

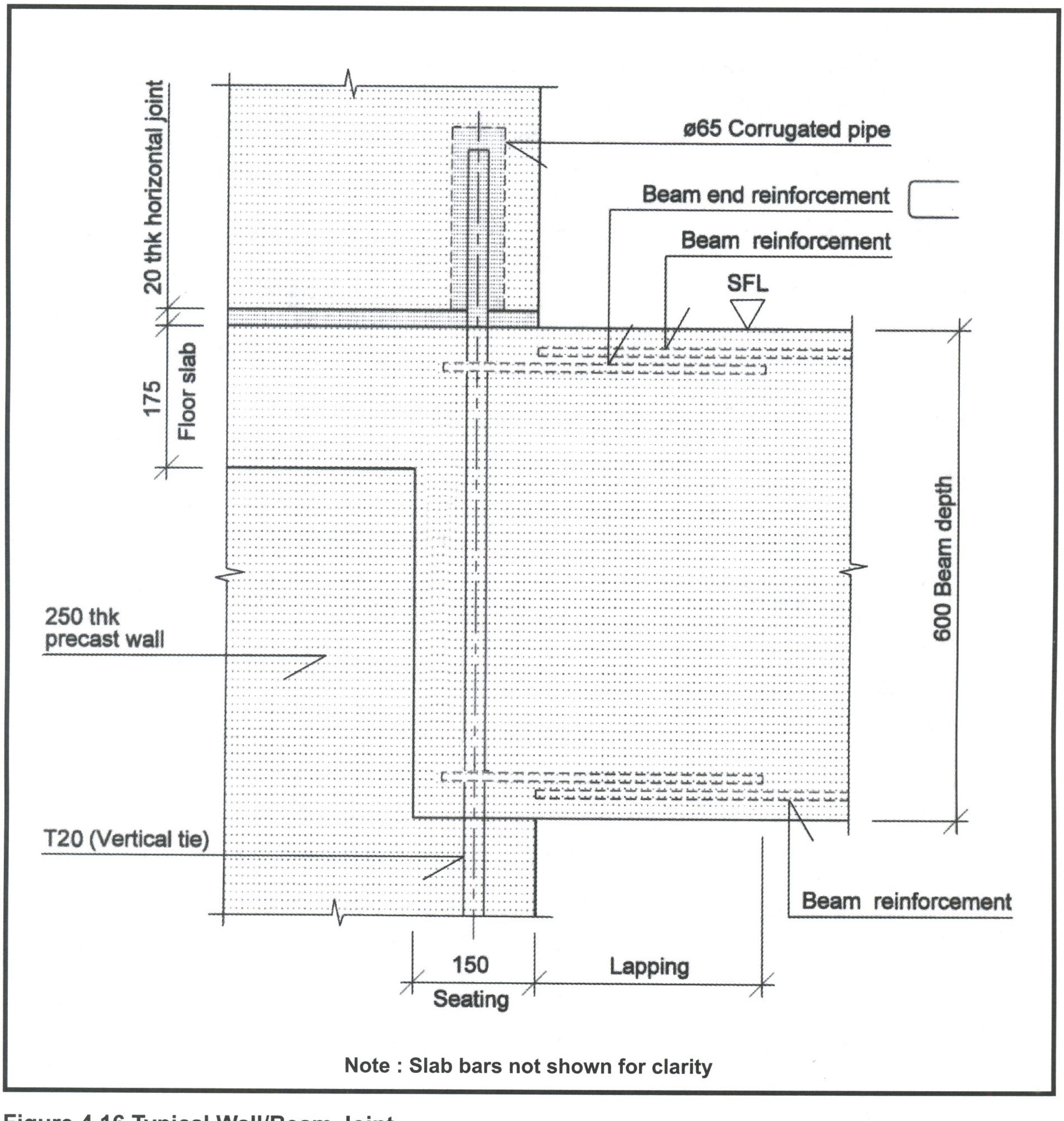
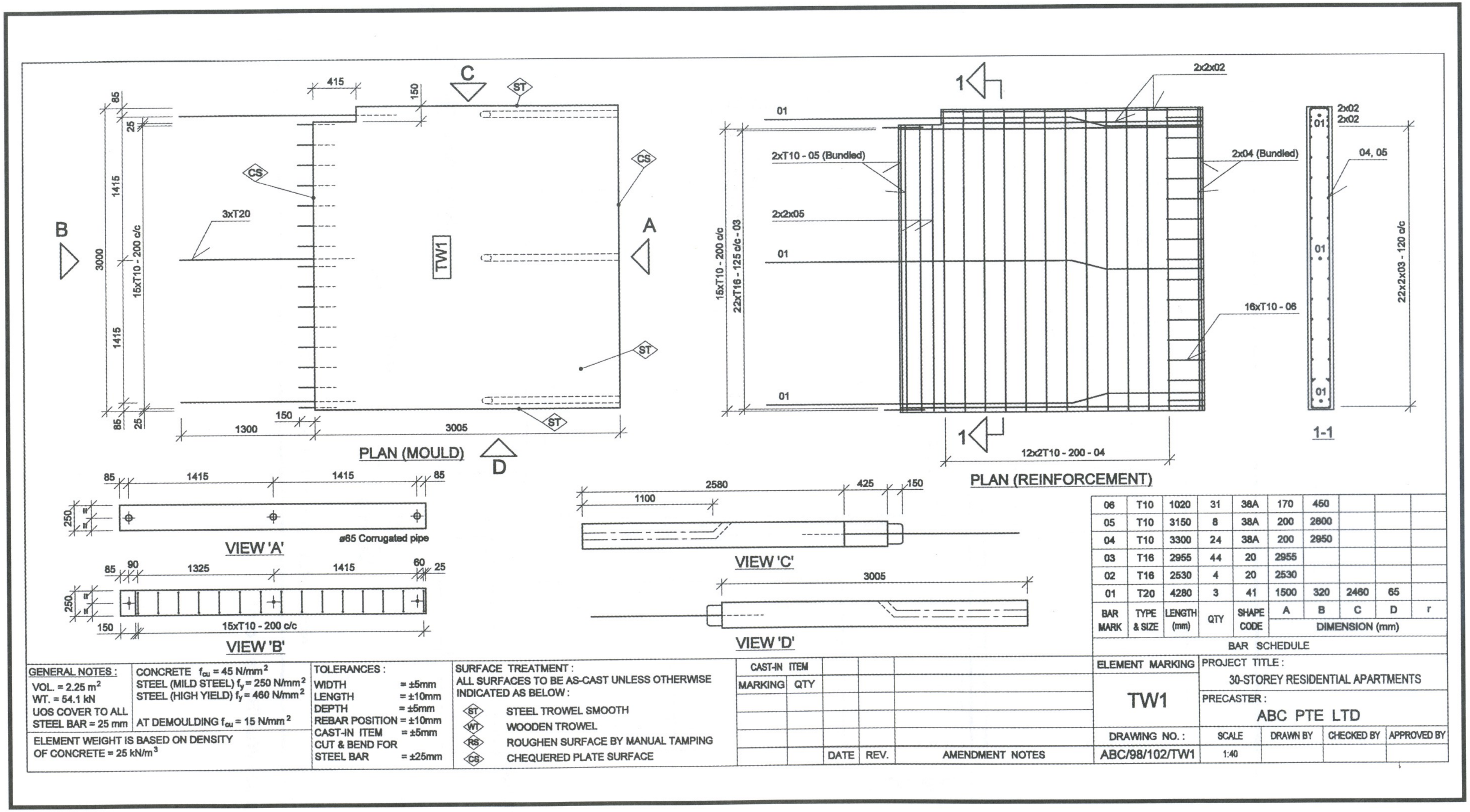


Figure 4.16 Typical Wall/Beam Joint



BAR MARK	TYPE & SIZE	LENGTH (mm)	QTY	SHAPE CODE	A	B	C	D	r
06	T10	1020	31	38A	170	450			
05	T10	3150	8	38A	200	2800			
04	T10	3300	24	38A	200	2950			
03	T16	2955	44	20	2955				
02	T16	2530	4	20	2530				
01	T20	4280	3	41	1500	320	2460	65	

DIMENSION (mm) — BAR SCHEDULE

Figure 4.17 Mould And Reinforcement Details Of Precast Wall (Residential Block)