

MULTISTOREYED APARTMENT

PROJECT REPORT

P-76

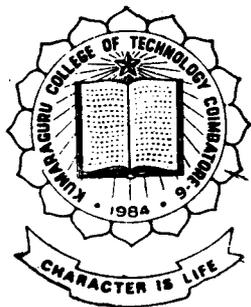
SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD
OF THE DEGREE OF BACHELOR OF ENGINEERING IN CIVIL ENGINEERING
OF THE BHARATHIYAR UNIVERSITY, COIMBATORE-46

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1989-90

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Certificate

This is to certify that the Report entitled
MULTISTOREYED APARTMENT
has been submitted by

Mr _____

in partial fulfilment for the award of Bachelor of Engineering
in the Civil Engineering Branch of the Bharathiyar University,
Coimbatore-641 046 during the academic year 1989-90

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Certified that the candidate was examined by us in the Project Work
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SYNOPSIS

This project work deals with planning, analysis and design of a multistoreyed apartment buildings. It is a typical reinforced concrete framed structure for apartments, with total number of 14 flats.

Each flat of the building is provided with ^{two}~~to~~ bed rooms, one drawing, dining room, one kitchen, one bathroom and one ~~wash~~ ^{WC}.

Car parking is provided in the ground floor of the back side building. Ducts are provided for service lines and for wastes. Lift and a staircase are provided from the easy passage of inmates.

The slabs are analysed by yield line theory and designed by Working Stress Method design procedure. Moment Distribution method is used for the analysis of frames. Single cycle moment distribution is done for dead load whereas three cycle moment distribution is done for live load using suitable substitute frames. The member have been designed using working stress method design procedure. High strength deformed bars have been used as reinforcement. Necessary architectural and structural drawings have been prepared.

INTRODUCTION

In urban areas, multi-storeyed apartments are now in vogue due to increase in population and growth in different activities, vacant and has also become scarce within the city limit to construct independent houses. This has driven many to seek refuge in flat system.

The object of this project work is to develop independent and creative thinking correlating fundamental, theoretical knowledge obtained in the course of the practical application in the field.

Analysis and synthesis are higher level of learning according to blooms taxonomy of education evaluation. Project work of learning incorporates mainly the above two methods.

In this project work attempts have been made to comprehend analysis and synthesis of what we have learnt during our course extended over last four years.

The design deals with a variety of structural elements like slabs, beams, frames foundations, staircases etc., The procedure followed here in generally is in accordance with Indian Standard specification and National Building code.

PLANNING OF APARTMENT BUILDING

INTRODUCTION

A building should be planned to make it comfortable, economical and to meet all the requirements. The effort of the planner should be to attain maximum comfort with limited money available. Functional utility, cost, habits, taste requirements etc., should also be considered in planning a building.

ORIENTATION OF BUILDING

The orientation of the building should be such that it attains maximum benefit from the nature and at the same time it is protected from harmful effects.

VENTILATION

The object of ventilation is to maintain air circulation in the rooms and in the building. For proper ventilation windows should have minimum area of $1/8$ of floor area of the room and the area of doors and windows should not be less than $1/4$ of the floor area of the room.

ROOM SIZE AND THEIR LOCATION

For the some area of the building, the bigger the sizes of the rooms the lesser the cost ;and vice

versa. But the size of the rooms will depend on the requirements and purpose for which they are meant.

DRAWING ROOM

Drawing room, the main aspect of a residential building is that, that it is well lighted and ventilated and located in the heart of the building having access from all the rooms. This room services as a recreation room, study room, entertaining room etc.

DINING ROOM

It should be close to the drawing room by its side and provided with a cup-board and a wash basin.

BED ROOM

Bed rooms should be so located that they are well ventilated and at the same time provide privacy. If space, good water supply and drainage is available, a bed room should have attached bath and W/C.

KITCHEN

It should be provided in rear corner of the building. It should be connected with dining room and well ventilated. Sink should be provided for washing

and sufficient number of shelves should also be provided.

BATH AND WATER CLOSURE

Bath and W/C should be provided in rear of the building separately so that the two can be used at a time. Good ventilation should be provided for rest room and should be fitted with shower, wash basin, shelves, brackets etc.

STAIRCASE AND LIFT

Staircases and lifts should be constructed for easy passage of the inmates. The staircases are well ventilated and lighted. The minimum width of staircase should be 0.9 m clear of railings and may range upto 1.5 m. The stairs should be in three flights having a landing at the end of each flight. As in this project, the number of stories are four lifts are also provided.

WATER SUPPLY AND DRAINAGE

A common water tank is provided separately at the side of the building. Water supply is given to each flat by proper network for pipelines. To receive the water from the corporation water supply an underground water tank has been provided.

Adequate arrangements shall be made for satisfactory drainage of sewage and waste water from each flat. Harmful waste water such as kitchen waters can be satisfactorily disposed. The drainage system should be met that no stagnation at the maximum discharge rate for which different units are designed.

ELECTRICAL WIRING AND DISTRIBUTION

The exact positions of all mains, plug points etc, should be determined in advance so that holes, silts etc, can be left in structural units. Recessed conduct type is suggested for wiring.

FLOORING

Now a days it is advisable to use mosaic tiles floorings for better performance and appearance. The type, dimenssion and laying be according to IS 809-1957.

DRAWINGS

The detailed drawings showing, floor plans, section, elevation, structural drawings are also attached.

STRUCTURAL DESIGN

Framed multistorey structure is an assembly of a numbers such as slabs, beams, columns, walls, lintels and sunshades etc.

Design of a structure includes assessing the loads and providing members of sufficient proportions to resist the assessed loads with sufficient margin of safety.

A structure is said to be efficiently designed to all the component members are so arranged that they transmit their self weight and other imposed loads to foundation by cheapest means so as to satisfy the requirements of architecture, structural stability and nature of the site with sufficient safety.

In the project all the structural members are designed according to the provisions of IS 456-1978. The concrete grade used is M_{15} and steel grade is Fe_{415}

DESIGN OF SLABS

INTRODUCTION:-

In our project the slab has been analysed by yield line theory and designed by working stress method. This method of analysis and design is more economical.

ROOF SLABS:

PANEL 5

We assume to provide 0.4% of gross area as steel

modification factor = 1.3. (from chart)

Basic value for continuous beam = 26

limiting L/d ratio = $26 \times 1.3 = 33.8$

d required = $\frac{3730}{33.8}$

33.8

= 110.36mm

Assume 140mm overall thickness of the slab.

LOADS:-

Self weight of slab = $0.14 \times 25 \times 1 = 3.5 \text{ KN/M}^2$

Weight of weathering course = 2.25 KN/M^2

Total dead Load = 5.75 KN/M^2

$$\begin{aligned} \text{Live load (access provided)} &= 1.5 \text{ KN/m}^2 \\ \text{Total load} &= \underline{\underline{7.25 \text{ KN/m}^2}} \end{aligned}$$

SHORT SPAN DIRECTION

$$\begin{aligned} \text{C/C distance between supports} &= 3.73\text{m} \\ \text{Clear span + effective depth} &= 3.62\text{m} \\ \text{Effective span } L_x &= 3.62\text{m} \end{aligned}$$

LONG SPAN DIRECTION

$$\begin{aligned} \text{C/C distance between supports} &= 6.46\text{m} \\ \text{Clear span + effective depth} &= 6.35\text{m} \\ \text{Effective span } l_y &= 6.35\text{m} \end{aligned}$$

$$\frac{l_y}{l_x} = \frac{6.35}{3.62} = 1.754$$

The end conditions of slab = One short edge discontinuous.

MOMENT ON SHORT SPAN DIRECTION

$$\begin{aligned} \alpha_x \text{ for mid span} &= 0.048 \\ \beta_x \text{ for edge} &= 0.064E \\ M_x \text{ for mid span} &= 0.048 \times 7.25 (3.62)^2 \\ &= 4.59 \text{ KN m} \end{aligned}$$

$$\begin{aligned}
 M_x \text{ for edge} &= 0.064 \times 7.25 \times (3.62)^2 \\
 &= -6.08 \text{ KN-m}
 \end{aligned}$$

MOMENT ON LONG SPAN DIRECTION

$$\begin{aligned}
 \mathcal{L}_y \text{ for mid span} &= 0.028 \\
 \mathcal{L}_y \text{ for edge} &= 0.037 \\
 M_y \text{ for mid span} &= 0.028 \times 7.25 \times (3.62)^2 \\
 &= 2.7 \text{ KN-m} \\
 M_y \text{ for edge} &= 0.037 \times 7.25 \times (3.62)^2 \\
 &= -3.52 \text{ KN-m}
 \end{aligned}$$

EFFECTIVE DEPTH

Out of the four moments 6.09 KN-m is the maximum moment. So that for the calculation of depth required, this moment is used

$$\begin{aligned}
 d \text{ required} &= \frac{M}{Q_b}^{\frac{1}{2}} \\
 &= \frac{6.08 \times 10^6}{0.658 \times 1000}^{\frac{1}{2}} \\
 &= 96.4 \text{ mm}
 \end{aligned}$$

d provided is greater than d required. So that d provided is safe.

$$d \text{ provided} = 140 - 20 - 10/2 = 115 \text{ mm.}$$

REINFORCEMENT

SHORT SPAN DIRECTION:-

$$\text{Area of steel required} = A_{st} = \frac{M}{st \times jd}$$

$$\begin{aligned} A_{st} \text{ of edge} &= \frac{6.09 \times 10^6}{230 \times 0.9 \times 115} \\ &= 255.83 \text{ mm}^2 \end{aligned}$$

Provide 10mm dia bars at 250 mm C/C.

$$\begin{aligned} A_{st} \text{ at mid span} &= \frac{4.57 \times 10^6}{230 \times 0.9 \times 115} \\ &= 192 \text{ mm}^2 \end{aligned}$$

Provide 8mm dia bars at 250 mm C/C

LONG SPAN DIRECTION:-

$$\begin{aligned} A_{st} \text{ at edge} &= \frac{3.52 \times 10^6}{230 \times 0.9 \times 115} \\ &= 148 \text{ mm}^2 \quad A_{st} \text{ minimum} \end{aligned}$$

∴ provide $A_{st} \text{ minimum} = 168 \text{ mm}^2$

Provide 8mm dia bars at 200mm C/C

$$\begin{aligned} A_{st} \text{ at mid span} &= \frac{2.7 \times 10^6}{230 \times 0.9 \times 115} \\ &= 113.4 \text{ mm}^2 \quad A_{st} \text{ minimum} \end{aligned}$$

∴ Provide Ast minimum = 168mm²
Provide 8mm dia bars at 250mm C/C.

MINIMUM AREA OF STEEL:-

For Fe 415 steel 0.12% of gross area is minimum area of steel

$$\begin{aligned} \text{Ast minimum} &= \frac{0.12}{100} \times 140 \times 1000 \\ &= 168\text{mm}^2 \end{aligned}$$

CHECK FOR DEFLECTION:-

Limiting L/d ratio = modification factor x Basic Value

$$\begin{aligned} \text{Percentage of steel} &= \frac{78}{250 \times 115} \times 100 \\ &= 0.2713\% \end{aligned}$$

Referring the chart is 15456 - 1978

$$\begin{aligned} \text{modification factor} &= 1.5 \\ \text{limiting L/d} &= 1.5 \times 2.6 \\ &= 39 \\ \text{Actual L/d ratio} &= \frac{3730}{115} \\ &= 32.44 \end{aligned}$$

Actual L/d ratio is less than limiting L/d ratio. SO that the slab is safe against deflection.

CHECK FOR SHEAR:-

$$\begin{aligned} \text{Maximum shear force} \quad V &= wL/2 \\ &= \frac{7.25 \times 3.73}{2} \\ &= 13.52 \text{ Km.} \end{aligned}$$

$$\begin{aligned} \text{Maximum shear stress} \quad \tau_v &= \frac{13.52 \times 10^3}{1000 \times 115} \\ &= 0.118 \text{ N/mm}^2 \end{aligned}$$

From is code 450 - 1978 Table 17.

$$\begin{aligned} 100A_s/bd &= \frac{100 \times 314}{1000 \times 115} \end{aligned}$$

$$\tau_c = 0.273$$

$$= 0.22 + \frac{(0.29-0.22)(0.023)}{0.25}$$

$$= 0.2265 \text{ N/mm}^2$$

$$K \tau_c = 0.2265 \times 1.3$$

$$= 0.294 \text{ N/mm}^2$$

$\tau_v < \tau_c$. Hence safe.

Hence no shear inforcement required.

FLOOR SLABS

PANEL 5

We assume to provide 0.4% of gross area as steel

Modification factor = 1.3 (from chart)

Basic value for continuous beam = 26

Limiting L/d ratio L3d ratio = $26 \times 1.3 = 33.8$

d required = $\frac{3730}{33.8}$

33.8

= 110.36mm

Assume 140mm overall thickness of the slab.

LOADS:-

Self weight of slab = $0.14 \times 25 \times 1$
= 3.5 KN/m²

Weight due to floor finish = 1KN/m²

Live load for floor = 3 KN/m²

As per Reynolds hand book for
partition wall = 1KN/m²

Design load for

(i) Bed room and kitchen = 8.5 KN/m²

(ii) Other rooms = 7.5 KN/m²

MOMENTS:-

$$M_x = \mathcal{L}_x W L_x^2$$

$$M_y = \mathcal{L}_y W l_x^2$$

where l_x and l_y are effective span along shaft and long direction respectively.

EFFECTIVE SPAN:-

Assume 230 mm width of beam is used. Support it less than 1/12 of clear span. Now the effective span is taken as

- (i) clear span + effective depth
- (ii) Centre to centre of supports Whichever is less.

SHORT SPAN DIRECTION:-

C/C distance between supports	= 3.73 m
Clear span + effective depth	= 3.62 m
Effective span l_x	= 3.62m

LONG SPAN DIRECTION:-

C/C distance between supports	= 6.46 m
clear span + effective depth	= 6.35 m
Effective span l_y	= 6.35 m

$$\frac{I_y}{I_x} = \frac{6.35}{3.62} = 1.754$$

The end conditions of slab = one short edge discontinuous

MOMENT ON SHORT SPAN DIRECTION:-

$$\begin{aligned} \alpha_x \text{ for mid span} &= 0.0481 \\ \alpha_x \text{ for edge} &= -0.0641 \\ M_x \text{ for mid span} &= 0.0481 \times 7.5 \times (3.62)^2 \\ &= 4.73 \text{ kr-m} \\ M_x \text{ for edge} &= 0.0641 \times 7.5 \times (3.62)^2 \\ &= 6.3 \text{ KN-m} \end{aligned}$$

MOMENT ON LONG SPAN DIRECTION:-

$$\begin{aligned} \alpha_y \text{ for mid span} &= 0.028 \\ \alpha_y \text{ for edge} &= 0.037 \\ M_y \text{ for mid span} &= 0.028 \times 7.5 \times (3.62)^2 \\ &= 2.75 \text{ Kn-m} \\ M_y \text{ for edge} &= 0.037 \times 7.5 \times (3.62)^2 \\ &= -3.64 \end{aligned}$$

$$\begin{aligned} \text{Ast at edge} &= \frac{6.3 \times 10^6}{230 \times 0.9 \times 115} \\ &= 264.65 \text{ mm}^2 \end{aligned}$$

Provide 10mm dia bars at 250mm C/C

$$\begin{aligned} \text{Ast at mid span} &= \frac{4.73 \times 10^6}{230 \times 0.9 \times 115} \\ &= 198.7 \text{ mm}^2 \end{aligned}$$

Provide 10mm dia bars at 250mm C/C.

LONG SPAN DIRECTION:-

$$\begin{aligned} \text{Ast at edge} &= \frac{3.64 \times 10^6}{230 \times 0.9 \times 115} \\ &= 152.9 \text{ mm}^2 \quad \text{Ast minimum} \end{aligned}$$

.. provide minimum Ast dia bars at 250 mm C/C

provide 8 mm dia bars at 250 mm C/C

$$\begin{aligned} \text{Ast at mid span} &= \frac{2.75 \times 10^6}{230 \times 0.9 \times 115} \\ &= 115.5 \text{ mm}^2 \quad \text{Ast} \end{aligned}$$

.. Provide minimum Ast = 168mm²

Provide 8mm dia bars at 250 mm C/C.

MINIMUM AREA OF STEEL:-

For Fe415 steel, 0.12% of gross area is minimum area of steel.

$$\begin{aligned} \text{Ast min} &= \frac{0.12}{100} \times 140 \times 1000 \\ &= 168 \text{ mm}^2 \end{aligned}$$

CHECK FOR DEFLECTION :-

$$\begin{aligned}\text{Limiting L/d ratio} &= MF \times BV \\ \text{Percentage of steel} &= \frac{78}{250 \times 115} \times 100 \\ &= 0.2713\end{aligned}$$

Referring the chart in IS 450 - 1978

$$\begin{aligned}\text{Modification factor} &= 1.5 \\ \text{Limiting L/d} &= 1.5 \times 26 = 39 \\ \text{Actual L/d ratio} &= \frac{3730}{115} = 32.44\end{aligned}$$

Actual L/d ratio is less than limiting l/d ratio. So that slab is safe against deflection.

CHECK FOR SHEAR :-

$$\begin{aligned}\text{Maximum shear force} &= \frac{Wl}{2} \\ &= \frac{7.5 \times 3.73}{2} \\ &= 13.99 = 14 \text{ KN}\end{aligned}$$

$$\text{Maximum shear stress} = \frac{14 \times 1000}{1000 \times 115}$$

$$\tau_v = 0.122 \text{ N/mm}^2$$

From IS code 456-1978 Table 17

$$\frac{100 AS}{bd} = \frac{314 \times 100}{1000 \times 115}$$

$$\tau_c = 0.22 + \frac{(0.29 - 0.22)(0.023)}{0.25}$$

$$= 0.2265 \text{ N/mm}^2$$

$$K\tau_c = 0.2265 \times 1.3 = 0.294 \text{ N/mm}^2$$

$\tau_c > \tau_v$ Hence safe.

CANTLEVER SLABS

PANEL 10 :-

Assume thickness of slab as 140 mm

LOADS :-

$$\text{Self weight of slab} = 0.14 \times 1 \times 25 = 3.5 \text{ KN/m}^2$$

$$\text{Weight due to floor finish} = 1 \text{ KN/m}^2$$

$$\text{Live load of floor} = 3 \text{ KN/m}^2$$

$$\begin{aligned} \text{As per Raynolds hand book for partition} \\ \text{walls} &= 1 \text{ KN/m}^2 \end{aligned}$$

Total load

$$= 8.5 \text{ KN/m}^2$$

MOMENT :-

Maximum BM at built in end

$$= \frac{Wl^2}{2}$$

$$= \frac{8.5 \times 1.415^2}{2}$$

$$= 6.014 \text{ KN-m}$$

EFFECTIVE DEPTH :-

For a balanced design d required $= \left[\frac{M}{Qb} \right]^{\frac{1}{2}}$

Using MS bars and M_{15} grade concrete

$$d \text{ required} = \left[\frac{6.014 \times 10^6}{0.87 \times 1000} \right]^{\frac{1}{2}} = 83.14 < 140$$

Hence safe

Let $D = 140$ mm and $d = 120$ mm

REINFORCEMENT :-

$$\begin{aligned} \text{Ast required} &= \frac{M}{\sigma_{st} j d} \\ &= \frac{6.014 \times 10^6}{140 \times 0.91 \times 120} \\ &= 393.4 \text{ mm}^2 \end{aligned}$$

Provide 10 mm dia bars at 150 mm c/c

The spacing of bar is less than $120 \times 3 = 360$ mm.

DISTRIBUTION STEEL :-

$$\begin{aligned} \text{Ast} &= \frac{0.15}{100} \times 1000 \times 140 \\ &= 210 \text{ mm}^2 \end{aligned}$$

Provide 8 mm dia bars at 200 mm c/c

CHECK FOR SHEAR :-

$$\begin{aligned} \text{Maximum shear force } V &= Wl \\ &= 8.5 \times 1.45 = 12.03 \end{aligned}$$

$$\begin{aligned} \text{Nominal shear stress } \tau_v &= \frac{V}{bd} \\ &= \frac{12.03 \times 10^3}{1000 \times 120} = 0.1003 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} p &= \frac{\text{Ast} \times 100}{bd} \\ &= \frac{523 \times 100}{1000 \times 120} \\ &= 0.436 \end{aligned}$$

$$\tau_c = 0.22 + \frac{(0.29 - 0.22)(0.186)}{0.25}$$

$$= 0.272 \text{ N/mm}^2$$

$\tau_v < \tau_c$. Hence safe in shear

CHECK FOR DEFLECTION :-

$$\text{Actual } \frac{L_{eff}}{d} = \frac{1415}{120}$$

$$= 11.8$$

for $p = 0.436$ and for MS steel $MF = 2$

$$\text{Permissible } \frac{L_{eff}}{d} = 7 \times 2$$

$$= 14 > 11.8$$

Hence safe

CHECK FOR END ANCHORAGE/DEVELOPMENT LENGTH :-

$$L_d = \frac{M_1}{v} + l_0$$

$$l_d = \frac{\phi}{4bd} = \frac{140 + 10}{4 \times 0.6}$$

$$= 583.3 \text{ mm}^2$$

$$M_1 = \frac{78.5 \times 10^3}{83} \times 140 \times 0.87 \times 140 = 16.12 \times 10^6$$

$$v = 12.03 \text{ KN}$$

$$L_0 = 12 \phi \text{ or } d_{eff} \text{ whichever is more}$$

$$= 120 \text{ or } 140 \quad \text{Take } l_0 = 140 \text{ mm}$$

$$\frac{M_1}{v} + l_0 = \frac{16127494 + 140}{12030}$$
$$= 1480 > 583.3$$

Hence safe

PANEL 9 :-

Since the length of panel '9' is less than panel 10. Provide the same reinforcement as in panel 10 for panel '9' also.

PRELIMINARY DESIGN OF BEAMS

INTRODUCTION

The analysis of moments and forces in a continuous structure requires the prior knowledge of cross sectional dimensions. In multistoreyed building frames the size of beams are usually governed by the negative moments and shears at supports. To arrive at the dimensions of the beam, the following rules are used.

WIDTH OF BEAMS

Normally beam width should be assumed as equal to width of wall. In extreme case this should be equal to half the depth of beam.

DEPTH OF BEAMS

- (i) $\frac{1}{10}$ to $\frac{1}{15}$ of span (as considering deflection criteria)
(ii) $M = 1.5 Q b d^2$

Where M is fixed end moment due to both dead and live load.

b = breath of beam

d = depth of beam

Q = Co-efficient of moment of resistance.

FEM varies depending upon the loading pattern as follows.

UNIFORMLY DISTRIBUTED LOADS

$$FEM = \frac{1}{12} WL$$

where W is the total load and
L is the span of the member.

TRIANGULAR LOADS

$$FEM = \frac{5}{48} WL$$

Where W is the total load and
L is the span of the member.

TRAPEZOIDAL LOADS

$$FEM = KWL$$

Where W is the total load
L is the span of the member
K is the bending moment coefficient given by
Reynolds

$$K = \frac{(1 + a - a^2)}{12}$$

MODEL CALCULATION

ROOF AND FLOOR BEAM NO.1

Span of beam = 3.73m.

Shape of loading pattern on all beam from slab is
triangular and trapezoidal.

PRELIMINARY DESIGN OF COLUMN:

INTRODUCTION:

The preliminary estimate of approximate sizes of columns must be made. Generally, column sections are subjected to axial load, uniaxial bending and biaxial bending.

Moments in columns are considerably smaller than in beams, so that these sizes are primarily governed by the axial loads they have to carry.

To determine these size first it is best to compute the sections which would be required if axial load alone were present and then to increase them to provide the additional influence of moments (By Reynolds factor). The direct load coming on the column is computed and multiplied by a Reynolds factor which depends upon its partition in plan and elevation.

EQUIVALENT DIRECT LOAD ON COLUMN;

Axial	1.0	1.0	1.0
Uniaxial	4.5	2.0	1.4
Biaxial	6.0	2.3	1.8

Load from beam	=	153.05KN
Assume dimension of column	=	230 x 450mm
Self weight	=	4KN
Total load	=	157.05KN
Equivalent load factor	=	6.0
Therefore equivalent axial load	=	157.05 x 6
	=	942.3KN

Assume 2% steel.

P	=	ccAc + sc x Ast
942.3 x 10 ³	=	5x0.98Ag+190x0.02Ag
	=	8.7Ag

∴ Provided dimensions are safe

Area provided	=	115000mm ²
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TABLE NO. 3

Column	Floor Level	Loaded from Beam	Reynold's Factor	Assumed Dimension	Total Equal Load	Dimension Provided
	I	III	IV	V	VI	VII
C ₁	II			230 x 450	367.74	230 x 450
	F ₃		4.5	230 x 450	458.76	230 x 450
	F ₂		2	230 x 450	529.58	230 x 450
	F ₁		1.4	230 x 450	734.02	230 x 450
	G		1.4	230 x 450	739.62	230 x 450
C ₂	II			230 x 450	367.74	230 x 450
	F ₃	81.92	4.5	230 x 450	458.76	230 x 450
	F ₂	147.46	2	230 x 450	527.58	230 x 450
	F ₁	147.46	1.4	230 x 450	734.02	230 x 450
	G	147.46	1.4	230 x 450	739.62	230 x 450
FN		4	1.4	230 x 450		230 x 450

C ₃	F ₃	40.76	4.5	230 x 450	183.42	230 x 450
	F ₂	80.97	2	230 x 450	243.46	230 x 450
	F ₁	80.97	1.4	230 x 450	283.78	230 x 450
	G	80.97	1.4	230 x 450	397.14	230 x 450
	FN	4	1.4	230 x 450	402.74	230 x 450
C ₄	F ₃	109.83	4.5	230 x 450	494.24	230 x 450
	F ₂	204.82	2	230 x 450	629.93	230 x 450
	F ₁	204.82	1.4	230 x 450	727.26	230 x 450
	G	204.82	1.4	230 x 450	1014.00	230 x 450
	FN	4	1.4	230 x 450	1019.61	230 x 450
C ₅	F ₃	44.51	6	230 x 450	267.06	230 x 450
	F ₂	103.21	2.3	230 x 450	339.76	230 x 450
	F ₁	103.21	1.8	230 x 450	451.67	230 x 450
	G	103.21	1.8	230 x 450	637.45	230 x 450
	FN	4	1.8	230 x 450	644.65	230 x 450

C ₆	F ₃	44.51	6	230 x 450	267.06	230 x 450
	F ₂	103.21	2.3	230 x 450	339.76	230 x 450
	F ₁	103.21	1.8	230 x 450	451.67	230 x 450
	G	103.21	1.8	230 x 450	637.45	230 x 450
	FN	4	1.8	230 x 450	644.65	230 x 450
C ₇	F ₃	109.83	4.5	230 x 450	494.24	230 x 450
	F ₂	204.82	2	230 x 450	629.3	230 x 450
	F ₁	204.82	1.4	230 x 450	727.26	230 x 450
	G	204.82	1.4	230 x 450	1014.00	230 x 450
	FN	4	1.4	230 x 450	1019.61	230 x 450
C ₈	F ₃	40.76	4.5	230 x 450	183.42	230 x 450
	F ₂	80.97	1.4	230 x 450	243.46	230 x 450
	F ₁	80.97	1.4	230 x 450	283.78	230 x 450
	G	80.97	1.4	230 x 450	397.14	230 x 450
	FN	4	1.4	230 x 450	402.74	230 x 450

230 x 450

367.74

230 x 410

4.5

81.92

F₃

C₉

230 x 450

458.76

230 x 450

2

147.46

F₂

230 x 450

527.58

230 x 450

1.4

147.46

F₁

230 x 450

734.02

230 x 450

1.4

147.46

G

230 x 450

739.62

230 x 450

1.4

4

FN

230 x 450

267.74

230 x 450

4.5

81.92

F₃

C₁₀

230 x 450

458.76

230 x 450

2

147.46

F₂

230 x 450

527.58

230 x 450

1.4

147.46

F₁

230 x 450

734.02

230 x 450

1.4

147.46

G

230 x 450

739.62

230 x 450

1.4

4

fn

230 x 450

230.45

230 x 450

4.5

51.21

F₃

C₁₁

230 x 450

363.62

230 x 450

2

130.60

F₂

230 x 450

437.37

230 x 450

1.4

130.6

F₁

230 x 450

620.22

230 x 450

1.4

130.6

G

230 x 450

625.82

230 x 450

1.4

4

FN

C ₁₂	F ₃	137.89	1	230 x 450	137.89	230 x 450
	F ₂	210.11	1	230 x 450	348.0	230 x 450
	F ₁	210.11	1	230 x 450	558.11	230 x 450
	G	210.11	1	230 x 450	768.22	230 x 450
	FN	4	1	230 x 450	772.22	230 x 450
C ₁₃	F ₃	137.96	1	230 x 450	137.96	230 x 450
	F ₂	232.95	1	230 x 450	370.91	230 x 450
	F ₁	232.95	1	230 x 450	603.86	230 x 450
	G	232.95	1	230 x 450	836.81	230 x 450
	FN	4	1	230 x 450	840.81	230 x 450
C ₁₄	F ₃	111.8	4.5	230 x 450	503.1	230 x 450
	F ₂	161.13	2	230 x 450	545.86	230 x 450
	F ₁	161.13	1.4	230 x 450	607.68	230 x 450
	G	161.13	1.4	230 x 450	833.27	230 x 450
	FN	4	1.4	230 x 450	838.87	230 x 450

C ₁₅	F ₃	57.4	6.0	230 x 450	344.4	230 x 450
	F ₂	117.84	2.3	230 x 450	403.05	230 x 450
	F ₁	117.84	1.8	230 x 450	527.54	230 x 450
	G	117.84	1.8	230 x 450	739.66	230 x 450
	FN	4	1.8	230 x 450	746.86	230 x 450
C ₁₆	F ₃	119.88	4.5	230 x 450	539.46	230 x 450
	F ₂	186.3	2	230 x 450	612.36	230 x 450
	F ₁	186.3	1.4	230 x 450	689.47	230 x 450
	G	186.3	1.4	230 x 450	950.29	230 x 450
	FN	4	1.4	230 x 450	955.89	230 x 450
C ₁₇	F ₃	86.07	4.5	230 x 450	387.32	230 x 450
	F ₂	173.23	2	230 x 450	518.6	230 x 450
	F ₁	173.23	1.4	230 x 450	848.06	230 x 450
	FN	4	1.4	230 x 450	853.66	230 x 450

C ₁₈	F ₃	34.1	6	230 x 450	204.6	230 x 450
	F ₂	94.47	2.3	230 x 450	295.71	230 x 450
	F ₁	94.47	1.8	230 x 450	401.47	230 x 450
	G	94.47	1.8	230 x 450	571.52	230 x 450
	FN	4	1.8	230 x 450	578.72	230 x 450

FRAME ANALYSIS

INTRODUCTION:

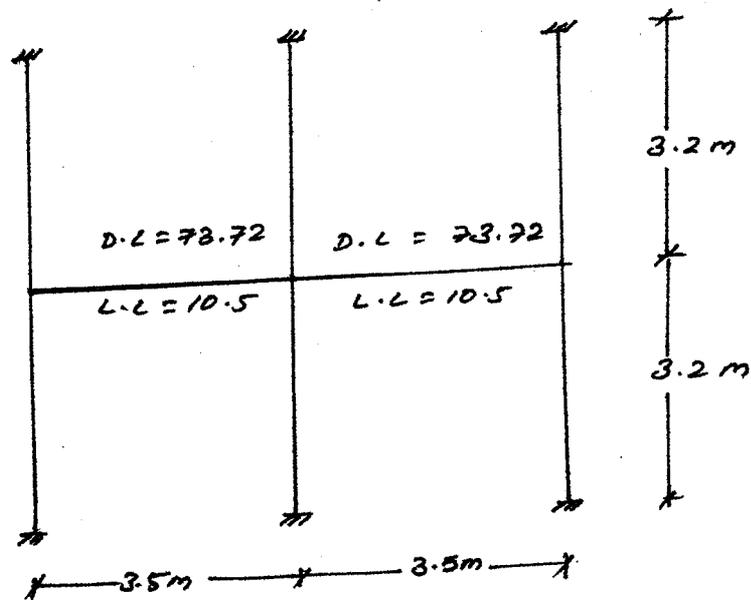
A frame consists of beams and columns built monolithic, having rigid joints at the junction. The load from slab is transferred to the foundation through beams and columns. The walls act as partition.

The frame under one consideration is a three dimensional structure and it is tedious to analyse as such. Hence the structure is analysed as two dimensional frame in two perpendicular directions neglecting the interaction.

The analysis of R.C frames can be carried out by the elastic and limit load (Plastic) method. But one I.S code does not specify limit analysis so we have to adopt elastic method of analysis.

Bending moments in various members are found out using "SUBSTITUTE FRAMES". This method is used because the moments in any beam or column are mainly due to the loads on spans very close to it. Loads on distance spans do not have appreciable effect.

ANALYSIS OF FRAME NO. I



LOADS:

Dead load from slab	= 19.25 kN/Space
Dead load from wall	= 44.82 kN/Space
Self weight of beam	= 9.65 kN/Space
Total dead load	= 73.72 kN/Space
Live load from slab	= 10.5 kN/Space.

DISTRIBUTION FACTORS:

JOINT	MEMBER	STIFFNESS	TOTAL STIFFNESS.	DISTRIBUTION FACTORS.
A	AB	1.02		0.3
	A1	1.25		0.35
	A2	1.25	3.57	0.35
B	BA	1.07		0.23
	BC	1.07		0.23
	B1	1.25		0.27
	B2	1.25	4.64	0.27
C	CB	1.07		0.3
	C1	1.25		0.35
	C2	1.25	3.57	0.35

FIXED END MOMENTS:

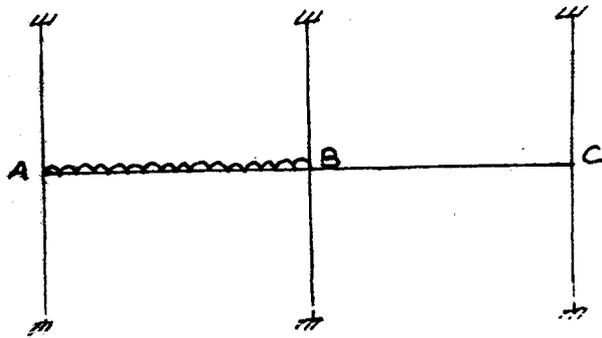
SPAN	D.L.FEM (KNm)	T.L.FEM (KNm)
AB	22.92	26.18
BC	22.92	26.18

MOMENT DISTRIBUTION OF MAXIMUM NEGATIVE BENDING MOVEMENT AT

JOINTS:

JOINT - A

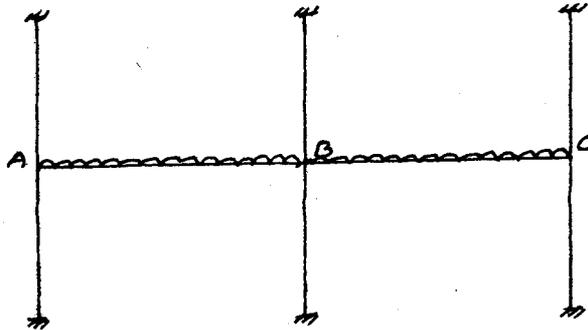
POSITION OF LOADS:



JOINT	A	B	C
Member	AB	BA	BC
DF	0.3	0.23	0.23
DLFEM			-22.92
TLFEM	-26.18	+26.18	+22.92
Distribute and C/O	-0.38		
Add	-26.56		
Distribute	+7.77		
Total	-18.59		

JOINT - B

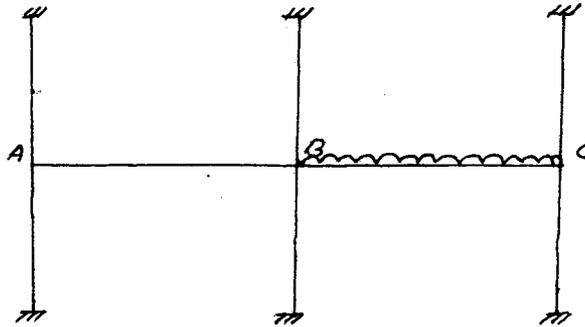
POSITION OF LOADS:



JOINT	A	B	C
Member	AB	BA	BC
DF	0.3	0.23	0.3
DLFEM			
TLFEM	-26.18	+26.18	+26.18
Distribute and C/O		+3.93	-3.93
Add		+30.11	-30.11
Distribute		0.00	0.00
Total		+30.11	-30.11

JOINT - C

POSITION OF LOADS:

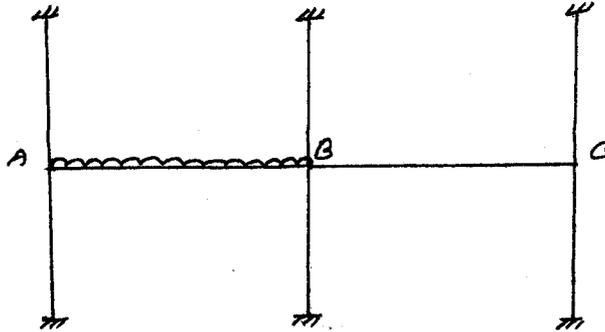


JOINT	A		B		C
Member	AB	BA	BC	CB	
DF	0.3	0.23	0.23	0.3	
DLFEM	-22.92	+22.92			
TLFEM			-28.18	+26.18	
Distribute and C/O					+0.38
Add					+26.56
Distribute					-7.97
Total					+18.59

MAXIMUM POSITIVE BENDING MOMENT AT MID SPAN:

SPAN - AB

POSITION OF LOADS

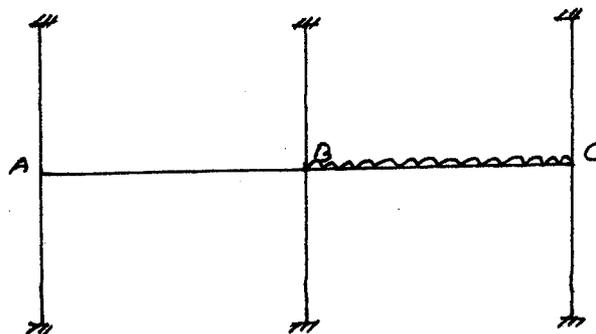


JOINT	A	B	A	C
Member	AB	BA	BC	CB
DF	0.3	0.23	0.23	0.3
DLFEM			-22.92	+22.92
TLFEM	-26.18	+26.18		
Distribute	7.85	-0.75		-6.88
C/O	-0.38	3.93	-3.44	
Distribute	0.114	-0.12		
Total	-18.61	+29.24		

$$\begin{aligned}
 \text{Maximum BM} &= \frac{3}{2} \text{TLFEM} - \frac{(18.61 + 29.24)}{2} \\
 &= \frac{3}{2} \times 26.18 - 23.925 \\
 &= 15.3 \text{KNm}
 \end{aligned}$$

SPAN BC

POSITION OF LOADS:



JOINT	A		B		C	
Member	AB	BA	BC	CB		
DF	0.3	0.23	0.23	0.3		
DLFEM	-22.92	+22.92				
TLFEM			-26.18	+26.18		
Distributes	+6.88		+0.75	-7.85		
C/O		+3.44	-3.93	+0.38		
Distribute			+0.12	-0.12		
Total			-29.24	+18.61		

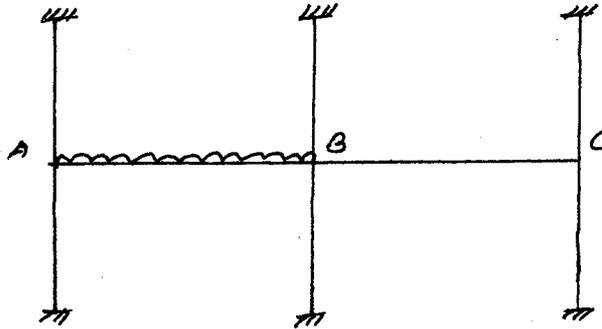
$$\text{Max BM} = \frac{3}{2} \text{TLFEM} - \frac{(18.61 + 29.24)}{2}$$

$$= \frac{3}{2} \times 26.18 - 23.925$$

$$= 15.3 \text{ KNm}$$

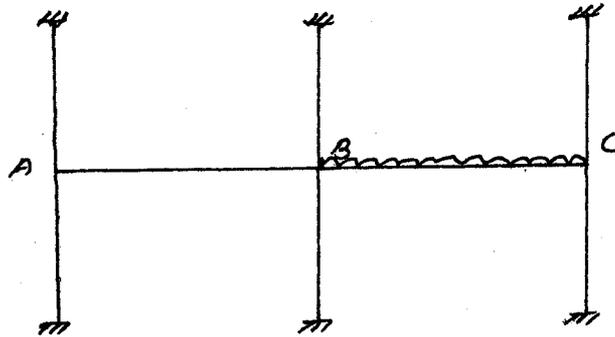
MOMENTS IN COLUMN:

LOADING PATTERN - I



JOINT		A	B	C
DF	TOP	0.35	0.27	0.35
	Bottom	0.35	0.27	0.35
Member (beam)		AB	BA	BC
DF		0.3	0.23	0.23
FEM		-26.18	+26.18	-22.92
Distribute and C/O		-0.38	+3.93	-3.44
Add		-26.56	+30.11	-26.36
Distribute top		+9.31	-1.02	-7.87
Distribute Bottom		+9.31	-1.02	-7.89

LOADING PATTERN - II



JOINT		A	B	C
DF	Top	0.35	0.27	0.35
	Bottom	0.35	0.27	0.35
Member (beam)		AB	BA	BC
DF		0.3	0.23	0.23
FEM		-22.92	+22.92	-26.18
Distribute and C/O		+0.38	+3.44	-3.93
Add		22.53	+26.36	-30.11
Distribute				
	Top	+7.89	+1.02	-9.31
	Bottom	+7.89	+1.02	-9.31

MOMENT IN BEAM

TABLE - 4

Joint	Member	Maximum Positive BM at Midspan (KN-m)	Maximum Negative BM at Midspan (KN-m)	Net Moment at Midspan (KN-m)	Maximum Shear force (KN)
I	II	III	IV	V	VI
FRAME II					
A C16	A C16 C13	21.4		2.14	57.00
B C16	B C16 C13	21.4		21.4	52.00
C C16	C C16 C13	21.4		21.4	57.00
D C16	D C16 C13	21.4		21.4	57.00
E C16	E C16 C13	21.4		21.4	57.00
A C13	A C13 C7	18.94	13.88	5.06	57.25
B C13	B C13 C7	18.94	13.88	5.06	57.25
C C13	C C13 C7	18.94	13.88	5.06	57.25
D C13	D C13 C7	18.94	13.88	5.06	57.25
E C13	E C13 C7	18.94	13.88	5.06	57.25

A C7	A C7 C4	11.77	11.45	0.32	39.25
B C7	B C7 C4	11.77	11.45	0.32	39.25
C C7	C7 C4	11.77	11.45	0.32	39.25
D C7	D C7 C4	11.77	11.45	0.32	39.25
E C7	E C7 C4	11.77	11.45	0.32	39.25

FRAME III

A C18	A C18 C11	15.44	15.44	15.44	41.76
B C18	B C18 C11	15.44	15.44	15.44	41.76
C C18	C C8 C11	15.44	15.44	15.44	41.76
D C18	D C8 C11	15.44	15.44	15.44	41.76
E C18	E C18 C11	15.44	15.44	15.44	41.76

A C11	A C11 C9	17.66	14.84	2.82	49.40
B C11	B C11 C9	17.66	14.84	2.82	49.40
C C11	C C11 C9	17.66	14.84	2.82	49.40
D C11	D C11 C9	17.66	14.84	2.82	49.40
E C11	E C11 C9	17.66	14.84	2.82	49.40

A C9	A C9 C2	18.22	13.91	4.31	55.52
B C9	B C9 C2	18.22	13.91	4.31	55.52
C C9	C C9 C2	18.22	13.91	4.31	55.52
D C9	D C9 C2	18.22	13.91	4.31	55.52
E C9	E C9 C2	18.22	13.91	4.31	55.52

FRAME IV

A C6	A C6 C7	25.83	25.83	25.83	57.85
B C6	B C6 C7	25.83	25.83	25.83	52.85
C C6	C C6 C7	25.83	25.83	25.83	57.85
D C6	D C6 C7	25.83	25.83	25.83	57.85
E C6	E C6 C7	25.83	25.83	25.83	57.85

A C7	A C7 C8	9.9	6.72	3.18	42.25
B C7	B C7 C8	9.9	6.72	3.18	42.25
C C7	C C7 C8	9.9	6.72	3.18	42.25
D C7	D C7 C8	9.9	6.72	3.18	42.25
E C7	E C7 C8	9.9	6.72	3.18	42.25

A C12	A C12 C13	11.76	5.4	6.36	49.25
B C12	B C12 C13	11.76	5.4	6.36	49.25
C C12	C C12 C13	11.76	5.4	6.36	49.25
D C12	D C12 C13	11.76	5.4	6.36	49.25
E C12	E C12 C13	11.26	5.4	6.36	49.25

A C13	A C13 C14	38.54		38.54	69.25
B C13	B C13 C14	38.54		38.54	69.25
C C13	C C13 C14	38.54		38.54	69.25
D C13	D C13 C14	38.54		38.54	69.25
E C13	E C13 C14	38.54		38.54	69.25

57

FRAME VI

A C18	A C18 C17	12.12		12.12	35.00
B C18	B C18 C17	12.12		12.12	35.00
C C18	C C18 C17	12.12		12.12	35.00
D C18	D C18 C17	12.12		12.12	35.00
E C18	E C18 C17	12.12		12.12	35.00

TABLE - 5

MOMENT IN COLUMN

COLUMN	FLOOR	MOMENT IN XX DIRECTION (KN-m)	MOMENT IN YY DIRECTION (KN-m)	AXIAL LOAD (KN)
I	II	III	IV	V
C ₁	F ₃	4.3	1.53	81.92
	F ₂	4.3	1.53	229.38
	F ₁	4.3	1.53	376.84
	G	4.3	1.53	524.3
	F _N	4.3	1.53	528.3
C ₂	F ₃	4.3	1.53	81.92
	F ₂	4.3	1.53	229.38
	F ₁	4.3	1.53	376.84
	G	4.3	1.53	524.3
	F _N	4.3	1.53	528.3
C ₃	F ₃	15.53	0.00	40.76
	F ₂	15.53	0.00	121.73
	F ₁	15.53	0.00	202.27
	G	15.53	0.00	283.67
	F _N	15.53	0.00	287.67

I	II	III	IV	V
C4	F3	10.25	6.86	109.83
	F2	10.25	6.86	314.65
	F1	10.25	6.86	519.47
	G	10.25	6.86	724.29
	FN	10.25	6.86	728.29
C5	F3	24.5	15.9	44.51
	F2	24.5	15.9	147.72
	F1	24.5	15.9	250.93
	G	24.5	15.9	354.14
	FN	24.5	15.9	358.14
C6	F3	24.5	15.9	44.51
	F2	24.5	15.9	147.72
	F1	24.5	15.9	250.93
	G	24.5	15.9	354.14
	FN	24.5	15.9	358.14
C7	F3	10.25	6.86	109.83
	F2	10.25	6.86	314.65
	F1	10.25	6.86	519.47
	G	10.25	6.86	724.29
	FN	10.25	6.86	728.29

DESIGN OF BEAMS

INTRODUCTION

In analysis part the final end moment of the beams (Hogging bending moments at support section, maximum sagging bending moment at middle sections and maximum shear force at support sections) are given in a tabular form. Individual designs are carried out for roof beams, floor beams and plinth beams. The middle and support sections are considered as rectangular sections and both are designed for sagging and hogging bending moments respectively.

For adequacy of depth, the sections are designed as singly reinforced or doubly reinforced beams.

FRAME NO:1 BEAM NO.6 and 12 :

Size of beam = 230 x 450 mm
d provided = 450 - 40
= 410 mm

EFFECTIVE WIDTH OF FLANGE :-

$$6F = \frac{I_n}{6} + b_w + 6 D_f$$

(or)

$$b_f = b_w + \frac{1}{2} (\text{clear span on either side})$$

$$Bf = \frac{3.73}{6} + .23 + 6 \times .14$$

$$= 1.692 \text{ m}$$

$$bf = .23 + \frac{1}{2} \times 3.5$$

$$= 1.98 \text{ m}$$

$$\text{Width of flange } bf = 1.692 \text{ m}$$

$$= 1692 \text{ mm}$$

Assuming X^n to be balanced

$$xc = 0.29 d$$

$$= 0.29 \times 410$$

$$= 118.9 \text{ mm}$$

$$M = bf Df \quad cbc \quad \frac{(2xc - Df)}{2xc} \quad \frac{d-Df}{3} \quad \frac{3xc-2Df}{2xc - Df}$$

$$= 1692 \times 140 \times 5 \frac{(2 \times 118.9 - 140)}{2 \times 118.9} \quad \frac{410 - 140}{3} \quad \frac{3 \times 118.9 - 2 \times 140}{2 \times 118.9 - 140}$$

$$= 181.88 \text{ KN-m}$$

MR is greater than maximum BM

The T beam section is safe

AREA OF STEEL REQUIRED AT MID SECTIONS:-

$$Ast = \frac{M}{\sigma_{stxz}}$$

$$= \frac{15.35 \times 10^6}{230 \times (410 - 118.9/3)}$$

$$= 180.19 \text{ mm}^2$$

Using 16 mm \emptyset bars.

$$\begin{aligned}\text{No of bars} &= \frac{180.19}{201} \\ &= 0.89\end{aligned}$$

provide 2 bars at the bottom and 2 bars of 12 mm \emptyset bars as hanger bars.

$$\text{Moment at support} = 30.11$$

AREA OF STEEL REQUIRED AT END SECTION :-

$$\begin{aligned}\text{Ast required} &= \frac{M}{\sigma_{st} \times z} \\ &= \frac{30.11 \times 10^6}{230 \times (410 - 118.9/3)} \\ &= 353.47 \text{ mm}^2\end{aligned}$$

using 16 mm \emptyset bars

$$\begin{aligned}\text{No. of bars} &= \frac{353.47}{201} \\ &= 1.75\end{aligned}$$

Provide 2 bars at top

CHECK FOR SHEAR :-

$$\begin{aligned}\text{maximum S.F V} &= \frac{85.5 \times 3.23}{2} \\ &= 138.08 \text{ KN}\end{aligned}$$

$$\frac{As \times 100}{, bd} = \frac{402 \times 100}{230 \times 410} = 0.4263$$

$$\tau_c = 0.22 + \frac{(0.29-0.22)}{0.25} (0.1763)$$

$$= 0.269$$

$$\tau_v = \frac{138.083 \times 10^3}{230 \times 410}$$

$$= 1.464$$

$\tau_v > \tau_c$ Hence shear reinforcement should be provided.

Provide 6 mm \emptyset 2 legged stirrups

$$S_v = \frac{2 \times 28 \times 250}{0.4 \times 230}$$

$$= 153$$

provide 6 mm \emptyset 2 legged stirrups at 150 mm c/c

DESIGN OF CANTI-LEVER BEAMS

Size of beam = 230 x 450

Self weight = 2.6 KN/m

Load from slab = 13.73 KN/m

Load from walls = 13.11 KN/m

Total Load = 29.44 KN/m

Maximum BM at fixed end = $\frac{wl^2}{2}$

= $\frac{29.44 \times 1.1^2}{2}$

= 17.81 KN-m

M = $Q bd^2$

For a balanced section $d = \sqrt{\frac{17.81 \times 10^6}{0.658 \times 230}}$

= 343.05 mm

D overall = 343.05 + 40 = 383.05 < 450 mm

Hence safe

d actual = 410 mm

Area of steel required = $\frac{M}{\sigma_{st} \times Jd}$

= $\frac{17.81 \times 10^6}{230 \times 0.9 \times 410}$

= 209.85 mm²

Using 16 mm dia bars

No. of bars = $\frac{209.85}{201}$

= 1.04

Provide 2 Nos. of 16 mm dia bars

$$A_{st} = 402 \text{ mm}^2$$

CHECK FOR SHEAR :-

$$\begin{aligned} p &= \frac{100 \times A_{st}}{bd} \\ &= \frac{100 \times 402}{230 \times 410} \\ &= 0.43 \\ &= 0.22 + \frac{(0.29 - 0.22)}{0.25} (0.18) \\ &= 0.27 \text{ N/mm}^2 \end{aligned}$$

Maximum shear force at fixed end $V = 29.44 \times 1.1$

$$= 32.38 \text{ KN}$$

$$\begin{aligned} \tau_v &= \frac{32.38 \times 10^3}{230 \times 410} \\ &= 0.34 \text{ N/mm}^2 \end{aligned}$$

Hence shear reinforcement is required.

$$\begin{aligned} \text{Design shear } V_s &= (v - c) bd \\ &= (0.34 - 0.27) 230 \times 410 \\ &= 6601 \text{ N.} \end{aligned}$$

Using 6 mm dia 2 legged shear stirrups

$$\begin{aligned} \text{spacing} &= \frac{140 \times 2 \times 28 \times 410}{6601} \\ &= 486.95 \end{aligned}$$

$$\begin{aligned} \text{Maximum spacing} &= 0.75 d \\ q &= 0.75 \times 410 \\ &= 307.5 \text{ mm} \end{aligned}$$

307.5 or 450 mm whichever is less

Spacing of 6 mm dia 2 legged shear stirrups for minimum shear reinforcement

$$\begin{aligned} &= \frac{2 \times 28 \times 250}{0.4 \times 230} \\ &= 152.17 \end{aligned}$$

Adopt spacing of 150 mm c/c

CHECK FOR DEFLECTION :-

$$\begin{aligned} \text{Actual span/effective depth ratio} &= \frac{1100}{410} \\ &= 2.68 \end{aligned}$$

$$\text{For } P = 0.43, K_1 = 1.3$$

$$\begin{aligned} \text{Permissible span/effective depth ratio} &= 7 \times 1.3 = 9.1 > 2.68 \\ q \text{ Hence safe} \end{aligned}$$

REINFORCEMENT IN BEAMS

TABLE - 6

BEAM	MIDDLE SECTION				END SECTION				SHEAR REINFORCEMENT
	BOTTOM		TOP		BOTTOM		TOP		
	DIAMETER OF BARS	NUMBER OF BARS							
II	III	IV	V	VI	VII	VIII	IX	X	
ROOF BEAMS 6, 12	16	2	16	2	16	2	16	2	6 mm dia 2 legged stirrups at 150 mm c/c
ROOF BEAMS 11,7,4	16	2	16	2	16	2	16	3	6 mm dia 2 legged stirrups at 150 mm c/c
ROOF BEAMS 9,8,2	16	2	16	2	16	2	16	3	6 mm dia 2 legged stirrups at 150 mm c/c
ROOF BEAMS 16,15,14,13,18,19,20	16	2	16	2	16	2	16	3	6 mm dia 2 legged stirrups at 150 mm c/c
ROOF BEAMS 21,22,23	16	3	16	2	16	2	16	3	6 mm dia 2 legged stirrups at 150 mm c/c

I	II	III	IV	V	VI	VII	VIII	IX	X
ROOF BEAMS 24,25,26	16	2	16	2	16	2	16	3	6mm dia 2 legged stirrups at 150mm c/c
FLOOR BEAMS 6,12	16	2	16	2	16	2	16	2	6mm dia 2 legged stirrups at 150 mm c/c
FLOOR BEAMS 11,7,4	16	2	16	2	16	2	16	3	6mm dia 2 legged stirrups at 150mm c/c
FLOOR BEAMS 9, 8, 2	16	2	16	2	16	2	16	3	6mm dia 2 legged stirrups at 150 mm c/c
FLOOR BEAMS 16, 15, 14,13,17,18,19,20	16	2	16	2	16	2	16	3	6mm dia 2 legged stirrups at 150 mm c/c
FLOOR BEAMS 21,22,23	16	3	16	2	16	2	16	3	6mm dia 2 legged stirrups at 150 mm c/c
FLOOR BEAMS 24,25,26	16	2	16	2	16	2	16	3	6mm dia 2 legged stirrups at 150 mm c/c

DESIGN OF COLUMNS

INTRODUCTION :-

From the frame analysis chapter we have taken the bending moment and actual forces carried by the sections are neatly represented in a tabular form. The columns are designed either as short column or as long column subjected to uniaxial or biaxial bending whichever is available and occurs respectively.

DESIGN OF BIAXIAL BENDING COLUMNS :-

COLUMN No. C₁, C₂

$$\begin{aligned} \text{Axial load} &= 528.3 \\ M_x &= 4.3 \text{ KN-m} \\ M_y &= 1.53 \text{ KN-m} \\ l_{\text{eff}} &= 3.2 \text{ m} \\ \frac{l_{\text{eff}}}{b} &= \frac{3200}{230} = 13.91 \quad 12 \\ &= \end{aligned}$$

Hence the column is long column.

Assume to use 10 Nos. of 20 mm dia bars.

$$\begin{aligned} A_c &= 230 \times 450 + (1.5 \times 13-1) \times 10 \times 314 \\ &= 161590 \text{ m}^2 \\ I_{cx} &= \frac{230 \times 450^3}{12} + (1.5 \times 13-1) \times 10 \times 314 \times 200^2 \\ &= 4.07 \times 10^9 \text{ mm}^4 \end{aligned}$$

$$I_{cy} = \frac{450 \times 230^3}{12} + (1.5 \times 13-1) \times 10 \times 314 \times 90^2$$

$$= 9.268 \times 10^8 \text{ mm}^4$$

$$cc \text{ cal} = \frac{528.3 \times 10^3}{161590} = 3.269 \text{ N/mm}$$

$$cbc \text{ cal}_x = \frac{4.3 \times 10^6 \times 225}{4.07 \times 10^9}$$

$$= 0.238 \text{ N/mm}^2$$

$$cbc \text{ cal}_y = \frac{1.53 \times 10^6 \times 115}{9.268 \times 10^8}$$

$$= 0.19 \text{ N/mm}^2$$

$$\text{Total bending stress} = cbc \text{ cal } x + cbc \text{ cal } y$$

$$= 0.238 + 0.19$$

$$= 0.428 \text{ N/mm}^2$$

$$\frac{cc \text{ cal}}{cc} + \frac{cbc \text{ cal}}{cbc} = \frac{3.269}{4} + \frac{0.428}{5}$$

$$= 0.9028 < 1$$

Hence safe

LATERAL TIES:-

Use 6 mm dia ties with spacing of

i) 250 mm

ii) 16 times of main bar dia = $16 \times 20 = 320$ mm

iii) 48 times of tie bar dia = $48 \times 6 = 288$ mm

Adopt a spacing of 250 mm c/c

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Adopt a spacing of 250 mm c/c

COLUMN NO. C₃ AND C₈ :-

Provide 20 mm dia bars 6 Nos. and lateral ties of 6 mm dia at 250 mm c/c

COLUMN NO. C₅, C₆ and C₁₈ :-

Provide 20 mm dia bars 8 Nos. and lateral ties of 6 mm dia at

COLUMN NO. C₁, C₂, C₉, C₁₀, C₁₁, C₁₂, and C₁₅ :-

Provide 20 mm dia bars 10 Nos and lateral ties of 6 mm dia at 250 mm c/c.

COLUMN No. C₁₄, C₁₆, AND C₁₇ :-

Provide 20 mm dia bars 12 Nos. and lateral ties of 6 mm dia at

COLUMN NO. C₇, C₄ AND C₁₃:-

Provide 20 mm dia bars 14 Nos. and lateral ties of
6 mm dia at

DESIGN OF COLUMN FOOTINGS

INTRODUCTION :-

The design of foundations for columns are designed based on working stress method. All columns footings are designed as isolated rectangular footings. RC footings are provided to transmit the load of structure through column to the soil.

The requirements in design of footings are

- i) The bearing capacity of soil is assumed as 450 KN/M^2 as per IS 1904-1961 for coarse sand compact and dry..
- ii) The settlement of the structure should be within the permissible limit.
- iii) All footings are subjected to only actual load because the moments are very less compared with axial loads..
- iv) M_{15} concrete and F_e_{415} steel are used.

COLUMN No C₁ :-

Size of column = 230 x 450 mm

Load on column = 528.3 KN

Assume self weight = 52.83 KN
of footing as 10% _____

Total load = 581.13

Area of footing required = $\frac{\text{Total load}}{\text{Bearing capacity}}$

= $\frac{581.13}{450}$

= 1.2914 m²

Provide 0.9 x 1.8 m size footing

$q_0 = \frac{581.13}{1.8 \times 0.9}$

= 358.72 KN/m²

$M_{xx} = \frac{q_0 \times [(2l + a) (B-b)^2]}{24}$

= $\frac{358.72 [(2 \times 1.8 + 0.45) (0.9 - 0.2)^2]}{24}$

= 27.2 KN-m

$M_{yy} = \frac{q_0 [(2B+b) (L-a)^2]}{24}$

= $\frac{358.72 (2 \times 0.9 + 0.23) (1.8 - 0.45)^2}{24}$

= 85.3 KN-m

$$d_{xx} = \frac{27.2 \times 10^6}{0.9 \times 450}$$

$$= 259 \text{ mm}$$

$$d_{yy} = \frac{85.3 \times 10^6}{0.9 \times 230}$$

$$= 641.9$$

Provide a thickness of 700 mm

$$d = 700 - 40 = 660 \text{ mm}$$

$$A_{st \ xx} = \frac{27.2 \times 10^6}{230 \times 0.9 \times 660}$$

$$= 199.1 \text{ mm}^2$$

Provide 6 Nos. of 10 mm dia bars.

$$A_{st \ yy} = \frac{85.3 \times 10^6}{230 \times 0.9 \times 660}$$

$$= 624.4 \text{ mm}^2$$

Provide 12 Nos. of 10 mm dia bars

CHECK FOR SHEAR :-

$$\begin{aligned} \text{Maximum shear force } V &= q_0 \times [L \times B - (a+d)(b+d)] \\ &= 358.72 [1.8 \times 0.9 \\ &\quad - (0.45 + 0.66)(0.23 + 0.66)] \\ &= 226.75 \text{ KN.} \end{aligned}$$

$$\begin{aligned}
 \text{Maximum shear stress } \tau_v &= \frac{V}{2(a+d)+(b+d)dc} \\
 &= \frac{226.75 \times 10^3}{2(450+660)+(230+660)455.6} \\
 &= 0.125 \text{ N/mm}^2
 \end{aligned}$$

$$\text{Permissible shear stress} = K_s \tau_c$$

$$K_s = \left(0.5 + \frac{230}{450}\right) = 1.01$$

$$\tau_c = 1.01 \times 0.16 = 15$$

$$= 0.626$$

Hence safe in shear

TABLE - 7

COLUMN FOOTING REINFORCEMENT

COLUMN NUMBER	SIZE OF FOOTING		THICKNESS OF FOOTING	REINFORCEMENT ON XX DIREC- TION		REINFORCEMENT ON YY DIREC- TION	
	m	x m		DIA OF BARS	NUMBER OF BARS	DIA OF, BARS	NUMBER OF BARS
C ₁	0.9	x 1.8	700	10	6	10	12
C ₂	0.9	x 1.8	700	10	6	10	12
C ₃	0.6	x 1.2	425	10	4	10	8
C ₄	1	x 2	725	10	8	10	16
C ₅	0.7	x 1.4	425	10	6	10	12
C ₆	0.7	x 1.4	425	10	6	10	12
C ₇	1	x 2	725	10	8	10	16
C ₈	0.6	x 1.2	425	10	4	10	8
C ₉	0.9	x 1.8	700	10	6	10	12
C ₁₀	0.9	x 1.8	700	10	6	10	12
C ₁₁	0.75	x 1.5	475	10	6	10	12
C ₁₂	0.9	x 1.8	600	10	6	10	12
C ₁₃	1.2	x 2.4	875	10	8	10	16
C ₁₄	0.9	x 1.8	600	10	6	10	12
C ₁₅	0.75	x 1.5	450	10	4	10	8
C ₁₆	1	x 2	725	10	8	10	16
C ₁₇	0.9	x 1.8	625	10	6	10	12
C ₁₈	0.7	x 1.4	400	10	4	10	8

DESIGN OF THREE QUARTER TURN STAIRCASE

The arrangement of staircase as shown in drawings.

Width of staircase = 1.2 m

Vertical distance between floors = 3 m

Provide 3 flights

Height of each flight = $\frac{3000}{3} = 1000$ mm

Assuming to provide 150 mm risers.

No. of risers required = $\frac{1000}{150} = 6.67$.

Provided = 7 risers.

Actual height of each riser = $\frac{1000}{7} = 142.86$ mm

Number of treads in each flight = Number of risers - 1

$$= 7 - 1 = 6$$

Let the tread be 300 mm

DESIGN OF FLIGHT AB AND C D :-

The bearing of flight is 230 mm
effective horizontal span

$$\begin{aligned} &= 1.8 + 1.2 + 0.23 + \frac{0.23}{2} \\ &= 3.345 \text{ m} \end{aligned}$$

Let the thickness of waist be 250 mm

LOADS :-

Dead load of 250 mm waist

$$= 0.25 \times 25 = 6.25 \text{ KN/m}^2$$

Ceiling finish(12.5 mm) = 0.0125 x 25

$$= 0.313 \text{ KN/M}^2$$

Total load = 6.563 KN/m²

Corresponding load per square metre

$$\begin{aligned} &= \frac{R^2 + T^2}{T} \times W \\ &= \frac{(0.14286^2 + 0.3^2)^{\frac{1}{2}}}{0.3} \times 6.563 \\ &= 7.269 \text{ KN} \end{aligned}$$

Hence the actual load per square metre of
plan area will consist of

waist and ceiling finish = 7.269

$$\begin{aligned}
 \text{Dead load of steps} &= 0.7143 \times 25 = 1.786 \\
 \text{Top finish} &= 0.313 \\
 \text{Live loads} &= 3 \\
 \hline
 \text{Total load} &= 12.368 \\
 &= 12.4 \text{ KN}
 \end{aligned}$$

$$\begin{aligned}
 \text{BM} &= \frac{Wl^2}{8} \\
 &= \frac{12.4 \times 3.345^2}{8} \\
 &= 17.34 \text{ KN-m}
 \end{aligned}$$

$$\begin{aligned}
 \text{Depth required} &= \frac{M}{Qb}^{\frac{1}{2}} \\
 &= \frac{17.34 \times 10^6}{0.658 \times 1000}^{\frac{1}{2}} \\
 &= 162 \text{ } 229 \text{ mm}
 \end{aligned}$$

Hence safe

$$\begin{aligned}
 d \text{ provided} &= 250 - 15 - 12/2 \\
 &= 229 \text{ mm (using 12mm } \emptyset \text{ bars)}
 \end{aligned}$$

$$\begin{aligned}
 A_{st} &= \frac{M}{st \cdot jd} \\
 &= \frac{17.34 \times 10^6}{230 \times 0.9 \times 229} \\
 &= 265.8 \text{ mm}^2
 \end{aligned}$$

using 12 mm dia bars

$$\text{spacing} = \frac{113 \times 1000}{365.8} = 308.9 \text{ mm}$$

Provide 12 mm \emptyset bars at 250 mm c/c

DISTRIBUTION STEEL:

$$\begin{aligned} \text{Ast required} &= \frac{0.12}{100} b D \\ &= \frac{0.12}{100} \times 1000 \times 250 \\ &= 300\text{mm}^2 \end{aligned}$$

Using 8mm ϕ bars

$$\begin{aligned} \text{Spacing} &= \frac{50}{300} \times 1000 \\ &= 166.7\text{mm} \end{aligned}$$

Provide 8mm ϕ bars at 150 mm C/C

DESIGN OF FLIGHT BC.

$$\begin{aligned} \text{Effective horizontal span} &= 1.2+1.2+0.23+1.5+0.23+0.23\frac{1}{2} \\ &= 4.59\text{m} \end{aligned}$$

$$\begin{aligned} \text{BM} &= \frac{Wl^2}{8} \\ &= \frac{12.4 \times 4.59^2}{8} \\ &= 32.7 \text{ KN} - \text{M} \end{aligned}$$

$$\begin{aligned} \text{depth required} &= \left(\frac{M}{Qb} \right)^{\frac{1}{2}} \\ &= \frac{32.7 \times 10^6}{0.658 \times 1000}^{\frac{1}{2}} \\ &= 223\text{mm} < 229\text{m} \end{aligned}$$

Hence safe.

$$\begin{aligned}
 \text{Ast.} &= \frac{M}{\sigma_{st} \cdot j d} \\
 &= \frac{32.7 \times 10^6}{230 \times 0.9 \times 229} \\
 &= 690 \text{ mm}^2
 \end{aligned}$$

Using 12mm \emptyset bars

$$\begin{aligned}
 \text{Spacing} &= \frac{113}{690} \times 1000 \\
 &= 164 \text{ mm}
 \end{aligned}$$

provide 12mm \emptyset bars at 150 mm C/C

DISTRIBUTION STEEL:

$$\begin{aligned}
 \text{Ast. required} &= \frac{0.12}{100} \times 60 \\
 &= \frac{0.12}{100} \times 1000 \times 250 \\
 &= 300 \text{ mm}^2
 \end{aligned}$$

Using 8 mm \emptyset bars

$$\begin{aligned}
 \text{Spacing} &= \frac{50}{300} \times 1000 \\
 &= 166.7 \text{ mm}
 \end{aligned}$$

Provide 8mm \emptyset bars at 150 mm C/C

DESIGN OF SUNSHADE AND LINTELS

SUNSHADE

All sunshade are having a overhang of 450 mm

Assume thickness of sunshade = 100 mm

Considering 1 metre width of sunshade

Self weight of sunshade:

$$0.1 \times 1 \times 25 = 2.5 \text{ KN/m}^2$$

$$\text{Live load} = 0.75 \text{ KN/m}^2$$

$$\text{Total load} = 3.25 \text{ KN/m}^2$$

$$\text{Eff. length of sunshade} = 0.45 + 0.23/2$$

$$= 0.565\text{m}$$

or

$$= 0.45 + 0.8$$

$$= 0.53\text{m}$$

$$\text{Effective length} = 0.53\text{m}$$

$$\text{Maximum BM} = \frac{3.25 \times 0.53^2}{2}$$

$$= 0.46 \text{ KN - m}$$

$$\text{'d' required} = \left[\frac{M}{Qb} \right]^{\frac{1}{2}}$$

$$= \left[\frac{0.46 \times 10^6}{0.658 \times 1000} \right]^{\frac{1}{2}}$$

$$= 26.4 \text{ mm} \quad 80\text{mm}$$

Hence safe.

d provided

$$= 100 - 15 - 10/2$$

$$= 80\text{mm}$$

Ast required

$$\begin{aligned} &= \frac{M}{\sigma_{st} j d} \\ &= \frac{0.46 \times 10^6}{230 \times 0.9 \times 80} \\ &= 27.8 \text{ mm}^2 \end{aligned}$$

Minimum Area of steel
 \bar{c}

$$\begin{aligned} &= \frac{0.12 \times b D}{100} \\ &= \frac{0.12}{100} \times 1000 \times 100 \\ &= 120 \text{ mm}^2 \end{aligned}$$

Using 8mm \emptyset bars.

Spacing .

$$\begin{aligned} &= \frac{50}{120} \times 100 \\ &= 416 \text{ mm} \end{aligned}$$

Provide 8mm \emptyset bars at 200 mm C/C in both directions.

LINTELS:

The maximum length of lintel is equal to 1.8m
which is placed over windows (W_1)

The width of lintel beam	= 230 mm
Take depth of lintel beam	= 150 mm
Effective depth	= 125 mm
Effective span = 1.8 + 0.125	= 1.925 m.
Load from wall	= 0.6 x 1.925 x 0.23 x 20
	= 5.313 KN
Self weight of lintel	= 0.15 x 0.23 x 25
	= 0.8625 KN/m
Maximum BM	= $\frac{W_2 l^2}{8} + \frac{W_1 l}{6}$
	= $\frac{0.8625 \times 1.925^2}{8} + \frac{5.313 \times 1.925}{6}$
	= 2.11 KN-m
d required	= $\left[\frac{M}{Qb} \right]^{\frac{1}{2}}$
	= $\left[\frac{2.11 \times 10^6}{0.658 \times 230} \right]^{\frac{1}{2}}$
	= 118.1 mm
This is less than provided depth d	= 125 mm
Ast. required	= $\frac{2.11 \times 10^6}{230 \times 0.9 \times 125}$
	= 81.55 mm ²

$$\begin{aligned} \text{Minimum Ast.} &= \frac{0.85 \text{ bd}}{f_y} \\ &= \frac{0.85 \times 230 \times 125}{415} \\ &= 58.88 \text{mm}^2 \end{aligned}$$

Using 10mm \emptyset bars

$$\begin{aligned} \text{No. of bars} &= \frac{81.55}{78} \\ &= 1.05 \end{aligned}$$

Provide 2 Nos. of 10mm \emptyset bar at bottom and 2 nos 8mm \emptyset at top.

CHECK FOR SEAL

$$\begin{aligned} \text{Max SF} &= \frac{W_1}{2} + \frac{W_2 \times l}{2} \\ &= \frac{5.313}{2} + \frac{0.8625 \times 1.925}{2} \\ &= 3.5 \text{ KN} \end{aligned}$$

$$\begin{aligned} \text{Actual sheal stress} &= \frac{3.5 \times 10^3}{230 \times 125} \\ &= 0.122 \text{N/mm}^2 \end{aligned}$$

This is less than minimum allowable sheal stress of 0.22N/mm²

Provide 8mm 22 legged strups at 300mm c/c .

DESIGN OF WATER TANK

INTRODUCTION

The water tank is designed as a rectangular water tank. The concrete mix M20 and steel grade Fe415 are used.

Calculation for capacity

No of person in each flat = 5
Total No. of persons in the apartment = 14 x 5
= 70

Assuming the daily average demand/person/day

= 240 lit/day
Total average demand = 70 x 240

= 16800 lit
Peak demand = 1.8 x 16800

= 30240 lit
= 30.24 m³
Take the design capacity = 35m³

Dimension of tank

Assume to provide a tank of size 5m x 4m x 2m

Concrete mix used is M₂₀

Design of Roof Slab:

Roof slab is designed for a live load of 1.5 kN/m²

Assume thickness of roof slab as 150mm

Self weight

$$= 0.15 \times 1 \times 1 \times 25$$

$$= 3.75 \text{ KN/m}^2$$

Live load

$$= 1.5 \text{ KN/m}^2$$

Total load

$$= 5.25 \text{ KN/m}^2$$

L/B ration

$$= \frac{5}{4} = 1.25$$

Maximum BM

$$= \frac{Wl^2}{8}$$

$$= \frac{5.25 \times 4^2}{8}$$

$$= 10.5 \text{ KNM}$$

d required

$$= \left(\frac{M}{Qb} \right)^{\frac{1}{2}}$$

$$= \left(\frac{10.5 \times 10^6}{0.658 \times 1000} \right)^{\frac{1}{2}}$$

$$= 126.3 \text{ mm.}$$

Provided depth is 130 mm 126.3mm

Hence safe.

Area of steel

$$= \frac{M}{\sigma_{st} jd}$$

$$= \frac{10.5 \times 10^6}{230 \times 0.9 \times 130}$$

$$= 390.2 \text{ mm}^2$$

Provide 12mm dia bars at 250mmc/c

Distribution steel

$$= \frac{0.12 \times 150 \times 1000}{100}$$

$$= 180 \text{ mm}^2$$

Provide 8mm dia bars at 250 mm c/c

DESIGN OF LONG WALL:

As the length of long wall is greater than twice the height, the wall will be designed spanning vertically

$$\begin{aligned} P &= 2 \times 10 \\ &= 20 \text{ KN/m}^2 \\ &= 5.33 \text{ KNm} \end{aligned}$$

The tension will be on water side.

$$\begin{aligned} d \text{ required} &= \left(\frac{M}{Qb} \right)^{\frac{1}{2}} \\ &= \left(\frac{5.33 \times 10^6}{1.411 \times 1000} \right)^{\frac{1}{2}} \\ &= 61.5 \text{ mm} \end{aligned}$$

Provide overall depth of 150 mm with effective depth of 100mm

$$\begin{aligned} \text{Area of steel required} &= \frac{M}{\sigma_{st} j d} \\ &= \frac{5.33 \times 10^6}{100 \times 0.84 \times 100} \\ &= 634.9 \text{ mm}^2 \end{aligned}$$

Provide 12mm dia bars at 150mm c/c

Maximum positive

$$\begin{aligned} \text{BM} &= \frac{Pl^2}{33.5} \\ &= \frac{20 \times 2^2}{33.5} \\ &= 2.4 \text{ KNm} \end{aligned}$$

Area of steel required.

$$\begin{aligned} &= \frac{M}{\sigma_{st} j d} \\ &= \frac{2.4 \times 10^6}{100 \times 0.84 \times 100} \\ &= 285.7 \text{ mm}^2 \end{aligned}$$

Provide 8mm dia bars at 150mm c/c.

Distribution Steel

$$\begin{aligned} \text{Distribution steel} &= \left(0.3 - \frac{0.1 \times 50}{350}\right) \% \\ &= 0.286\% \end{aligned}$$

$$\begin{aligned} \text{Area steel of steel} &= \frac{0.286 \times 50 \times 1000}{100} \\ &= 143 \text{ mm}^2 \end{aligned}$$

Half reinforcement is provided on each face. Provide 8mm \emptyset bars at 300mm c/c on each face. There will be direct load on the long wall due to self weight of roof. It is assumed that whole load from roof is transferred to long walls.

Load per metre length of wall

$$\begin{aligned} &= 5.25 \times 5/2 + 4 \times 2 \\ &= 17.13 \text{ KN} \end{aligned}$$

Direct load is very small, its effect is neglected.

Actually reinforcement is provided on both faces vertically as well as horizontally.

There will be direct tension in long walls due to pressure on short walls but its effect is negligible.

Design of short walls

$$\frac{L_H}{L_r} = \frac{4}{2} = 2$$

Coefficients of BM are obtained from Tables.

Maximum -ve BM for vertical span

$$= 0.055 \times 20 \times 2^2$$
$$= 4.4 \text{ KNM}$$

Maximum +ve BM for vertical span

$$= 0.032 \times 20 \times 2^2$$
$$= 2.56 \text{ KNm}$$

Maximum -ve BM for horizontal span

$$= 0.002 \times 20 \times 2^2$$
$$= 0.16 \text{ KNm}$$

Maximum +ve BM for horizontal span

$$= 0.001 \times 20 \times 2^2$$
$$= 0.08 \text{ KNM.}$$

Steel in vertical span

For -ve BM

$$= \frac{4.4 \times 10^6}{100 \times 0.84 \times 100}$$
$$= 523.81 \text{ mm}^2$$

Provide 12mm dia bars at 150mm c/c

For +ve BM

$$= \frac{2.56 \times 10^6}{100 \times 0.84 \times 100}$$
$$= 304.8 \text{ mm}^2$$

Provide 8mm dia bars at 150mm c/c

Steel in Horizontal span

$$\begin{aligned}\text{For -ve BM} &= \frac{0.16 \times 10^6}{0.84 (100-6-6) \times 100} \\ &= 21.64 \text{ mm}^2\end{aligned}$$

Provide 12mm dia bars at 300 mm c/c

$$\begin{aligned}\text{For +ve BM} &= \frac{0.08 \times 10^6}{0.84 (100-6-6) \times 100} \\ &= 10.83 \text{ mm}^2\end{aligned}$$

Provide 8mm dia bar at 300mm c/c

Design of Slab

$$\text{L/B ratio} = 5/4 = 1.25$$

Assume 200 mm thick slab.

$$\begin{aligned}\text{Self weight} &= 0.20 \times 1 \times 25 \\ &= 5 \text{ KN/m}^2\end{aligned}$$

$$\text{Load of water} = 20 \text{ KN/m}^2$$

$$\text{Total load} = 25 \text{ KN/m}^2$$

BM in short span

$$\begin{aligned}\text{+ve BM at mid span} &= 0.0755 \times 25 \times 4^2 \\ &= 30.2 \text{ KNm}\end{aligned}$$

BM in long span:

$$\begin{aligned}\text{+ve BM at mid span} &= 0.056 \times 25 \times 4^2 \\ &= 17.92 \text{ KNM}\end{aligned}$$

$$d \text{ required} = \left[\frac{30.2 \times 10^6}{1.411 \times 1000} \right]^{\frac{1}{2}}$$

$$= 146.3 \text{ mm}$$

Provide overall depth of 200mm with effective depth of 160mm

Steel in short span:

$$\text{Area of steel required} = \frac{30.2 \times 10^6}{0.84 \times 100 \times 160}$$

$$= 2247 \text{ mm}^2$$

Provide 16mm dia bars at 75 mm c/c

Steel, in long span

$$\text{Effective depth} = 160 - 6 - 6$$

$$= 148 \text{ mm}$$

$$\text{Area of steel required} = \frac{17.92 \times 10^6}{0.84 \times 100 \times 148}$$

$$= 1441.4 \text{ mm}^2$$

Provide 12mm dia bars at 75 mm c/c

Design of Beams.

Design of Beam B

$$\text{load from slab} = 12.5 \text{ KN/m}^2$$

$$\text{Assume size of rib as } 300 \times 300 \text{ mm}$$

$$\text{self weight} = 0.3 \times 0.3 \times 25$$

$$= 2.25 \text{ KN/m}$$

Maximum BM at centre

$$= 0.8 \frac{17.13 \times 5^2}{8} + 3 \times 25 \times 2.5$$

$$- 2 \times 25 \times \frac{1}{2} \times 2 \left(\frac{2}{3} + 0.5 \right)$$

$$- 2 \times 25 \times 0.5 \times 0.25$$

$$= 173.56 \text{ KNM}$$

$$\begin{aligned}
 \text{Over all depth} & \quad d = \hat{c} \quad 2150\text{mm} \\
 \text{Assume effective depth} & \quad = 2100\text{mm} \\
 \text{Area of steel required} & \quad = \frac{173.56 \times 106}{0.86 \times 2100 \times 125} \\
 & \quad = 768.82\text{mm}^2
 \end{aligned}$$

Provide 3 bars of 20mm dia.

$$\begin{aligned}
 \text{Shear force} & \quad = 3 \times 25 + 17.3 \times 5/2 \\
 & \quad = 117.83 \text{ KN}
 \end{aligned}$$

Provide 8mm dia 2 legged stirrups at 250 mm c/c

Design of columns:

Load on column when tank is full

$$\begin{aligned}
 & \quad = \text{Reaction from beams } B_1 \text{ \& } B_2 \\
 & \quad = 117.83 \times 2 \\
 & \quad = 235.66 \text{ KN}
 \end{aligned}$$

Total weight of water

$$\begin{aligned}
 & \quad = 5 \times 4.2 \times 10 \\
 & \quad = 400 \text{ KN}
 \end{aligned}$$

weight of water on column

$$\begin{aligned}
 & \quad = \frac{400}{4} \\
 & \quad = 100 \text{ KN}
 \end{aligned}$$

load on column when tank is

$$\begin{aligned}
 \text{empty} & \quad = 235.66 - 100 \\
 & \quad = 135.66 \text{ KN}
 \end{aligned}$$

Wind force on tank

Intensity of wind pressure is assumed as 1.5 KN/m²

Wind force acting on water tank.

$$\begin{aligned} &= 5.16 \times 2.15 \times 1.5 \\ &= 16.64 \text{ KN acting at } 15.15 \text{ m} \\ &\text{from base.} \end{aligned}$$

Wind force on staging

$$\text{On columns} = 4 \times 0.3 \times 5 \times 1.5$$

$$= 9 \text{ KN}$$

$$\text{On bracing} = 3 \times 0.3 \times 5 \times 1.5$$

$$= 6.75 \text{ KN}$$

Total wind force on staging

$$= 9 + 6.75$$

$$= 15.75 \text{ KN acting at } 7 \text{ m}$$

from base

$$\text{Moment at base} = 16.64 \times 15.15 + 15.75 \times 7$$

$$= 362.35 \text{ KNm.}$$

AS the excentricity is very small no tension is induced.

Equivalent area of section

$$= 300 \times 300 + 6 \times 314 \times 12$$

$$= 112608 \text{ mm}^2$$

Equivalent moment of inertia

$$= \frac{1}{12} \times 300 \times 300^3 + 12 \times 6 \times 314 \times (100)^2$$

$$= 8.25 \times 10^8$$

$$\begin{aligned} \text{Direct stress} &= \frac{281.66 \times 10^3}{112608} \\ &= 2.5 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} \text{Bending stress} &= \frac{12.15 \times 10^6 \times 100}{8.25 \times 10^8} \\ &= 1.47 \text{ N/mm}^2 \end{aligned}$$

As the effect of wind is also taken into consideration allowable stress are increased by 33.33%

$$\frac{2.5}{6.667} + \frac{1.47}{9.333} = 0.532 < 1$$

Hence safe.

Design of braces:

$$\begin{aligned} \text{Maximum BM in brace} &= 2 \times 12.15 \end{aligned}$$

$$= 24.4 \text{ KNM}$$

Provide 300 x 300mm brace.

$$\begin{aligned} \text{Effective depth} &= 300 - 40 \\ &= 260 \text{ mm} \end{aligned}$$

$$\begin{aligned} \text{M R of section} &= Qbd^2 \\ &= 1.21 \times 300 \times 260^2 \\ &= 24.54 \text{ KNM} \end{aligned}$$

Actual of steel required

$$= \frac{24.54 \times 10^6}{0.87 \times 2260 \times 140}$$

$$= 774.92 \text{ mm}^2$$

Provide 3 bars of 20mm at top and bottom as the wind may blow from either side.

$$\begin{aligned}\text{Maximum shear force} &= \frac{2 \times 12.15}{5} \\ &= 4.86 \text{ KN}\end{aligned}$$

Shear force is very small. Shear stress is negligible.

Provide 8mm dia stirrups at 250 mm c/c.

$$\begin{aligned}I^2 &= 4 \times (2.5)^2 \\ &= 25 \\ \text{Force in column} &= \frac{M_1}{I^2} \\ &= \frac{362.35}{25} \\ &= 14.5 \text{ KN}\end{aligned}$$

There will be downward force of 14.5 KN in each of leeward columns and upwards force of 14.5 KN on each of windward columns.

$$\begin{aligned}\text{Self weight of columns} &= 4 \times 0.3 \times 0.3 \times 25 \\ &= 31.5 \text{ KN}\end{aligned}$$

$$\begin{aligned}\text{Maximum force in column} &= 235.66 + 14.5 + 31.5 \\ &= 281.66 \text{ KN}\end{aligned}$$

$$\begin{aligned}\text{Minimum force in column} &= 135.66 - 14.5 \\ &= 121.16 \text{ KN}\end{aligned}$$

No upward load is produced in column when tank is empty.

Design of columns;

Maximum dead load

$$= 235.66 + 31.5$$

$$= 267.16 \text{ KN}$$

section of 300 x 300 mm and provided

$$267.16 \times 10^3 \text{ (300} \times \text{300 - AC) } \times 5$$

$$+ \text{AC} \times 130$$

$$= 450000 - 5 \text{ AC} + 130 \text{ AC}$$

$$\text{AC} = \frac{182840}{125}$$

$$= 1462.72 \text{ mm}$$

Provide 6 No of 20mm dia bars.

Provide 8mm dia ties at 250 mm c/c

Check for stress with wind blowing:

Total horizontal force.

$$= 16.64 + 15.75$$

$$= \underline{32.39} \text{ KN}$$

force taken by each column

$$= \frac{32.39}{4}$$

$$= 8.1 \text{ KN}$$

$$= 8.1 \times 1.5$$

$$= 12.15 \text{ KNm}$$

$$= 281.66$$

Maximum lload

Maximum lload

$$= \frac{12.15 \times 1000}{281.66}$$

Eccentricity

$$= 43.14 \text{ mm}$$

DESIGN OF FOUNDATION :-

$$\begin{aligned} \text{Maximum load} &= 267.16 \\ \text{Assume self weight} &= 40.1 \\ \text{Total load} &= 307.26 \text{ KN} \\ \text{Area of footing} &= \frac{307.26 \times 10^6}{150} \\ &= 2.048 \text{ mm}^2 \end{aligned}$$

Provide 1.5 x 1.5 m footing

$$\text{Net upward pressure on footing} = \frac{267.16}{1.5 \times 1.5}$$

$$= 118.74 \text{ KN/m}^2$$

$$\text{Maximum BM} = 1.5 \times 0.6 \times \frac{118.74 \times 0.6}{2}$$

$$= 32.06 \text{ KNm}$$

$$\text{d required} = \frac{32.06 \times 10^6}{0.87 \times 300}^{\frac{1}{2}}$$

$$= 350.5 \text{ mm}$$

$$\text{Provide D} = 400 \text{ mm. } d = 375 \text{ mm}$$

$$\begin{aligned} \text{Area of steel} &= \frac{32.06 \times 10^6}{140 \times 0.87 \times 375} \\ &= 715.05 \text{ mm}^2 \end{aligned}$$

Provide 10 Nos. of 10 mm dia bars both ways

CHECK FOR SHEAR :-

$$\begin{aligned}\text{Shear Force } V &= 118.74 \times 1.5 \times 1.5 - (0.3+0.375)^2 \\ &= 213.06 \text{ KN}\end{aligned}$$

$$\begin{aligned}\text{Actual shear stress} &= \tau_V = 213.06 \times 10^3 \\ &= 0.28 \text{ N/mm}^2\end{aligned}$$

$$\text{Permissible shear stress} = K_s \tau_c$$

$$K_s = 0.5 + \frac{300}{300} = 1.5 \text{ greater than } 1$$

$$K_s = 1.0$$

$$\begin{aligned}\text{Permissible shear stress} &= 1 \times 0.16 \times 15^{1/2} \\ &= 0.6197 \text{ N/mm}^2\end{aligned}$$

Hence safe

DESIGN OF SEPTIC TANK

Population = 80 person

average daily sewage flow = 150 lpcd

Assume period of cleaning = 6 months

Sewage entering is 90%

Quantity of sewage entering = $80 \times 150 \times 0.9$
= 10800 lit/day
= $10.8 \text{ m}^3/\text{day}$

Assume detention period = 24 hours

24 hours capacity = $\frac{10.8}{24} \times 24 = 10.8 \text{ m}^3$

Assume a design period of 2 years

Sludge and sewage storage = $0.035 + 0.01$
= $0.045 \text{ m}^3/\text{capacity}$
= 0.045×80
= 3.6 m^3

Quantity of sludge digestion = $80 \times 0.028 = 2.24 \text{ m}^3$

Add future expansion as 20% of sewage volume
= $10.8 \times 0.2 = 2.16 \text{ m}^3$

Total volume of tank = $10.8 + 34.6 + 2.24 + 2.16$
= 18.8 m^3

Assume depth of the tank = 1.5m

Area in plan = $\frac{18.8}{1.5} = 12.533 \text{ m}^2$

Assume L = 3 B

Area = L x B = $3B \times B = 3B^2 = 12.533 \text{ m}^2$

$$B = 2.042$$

Provided $B = 2.25\text{m}$

$$L = 3B = 3 \times 2.25 = 6.75 \text{ m}$$

Provide 0.5m free board over all dimension of septic tank.

$$= 6.75 \times 2.25 \times 2\text{m}$$

DESIGN OF DISPERSION TRENCH:

Width of the trench

$$= 50\text{cm to } 100\text{cm}$$

Depth

$$= 50\text{cm to } 100 \text{ cm}$$

length must be less than 30 m

Absorption weight

$$= 0.140 + 0.15\text{m}^3/\text{m}^2/\text{day}$$

minimum clear distance between trench 1.8m

Assuming 3 trench of breath and depth as 1m each.

Bottom area

$$= L \times 1 = L\text{m}^2$$

Since $B = 1$

$$D = 1$$

Side Area

$$= 2[L \times 1 + 1 \times 1]$$

Total area

$$= (3L + 2) \times 3$$

Total quantity of sludge absorbed by 3 trench at the

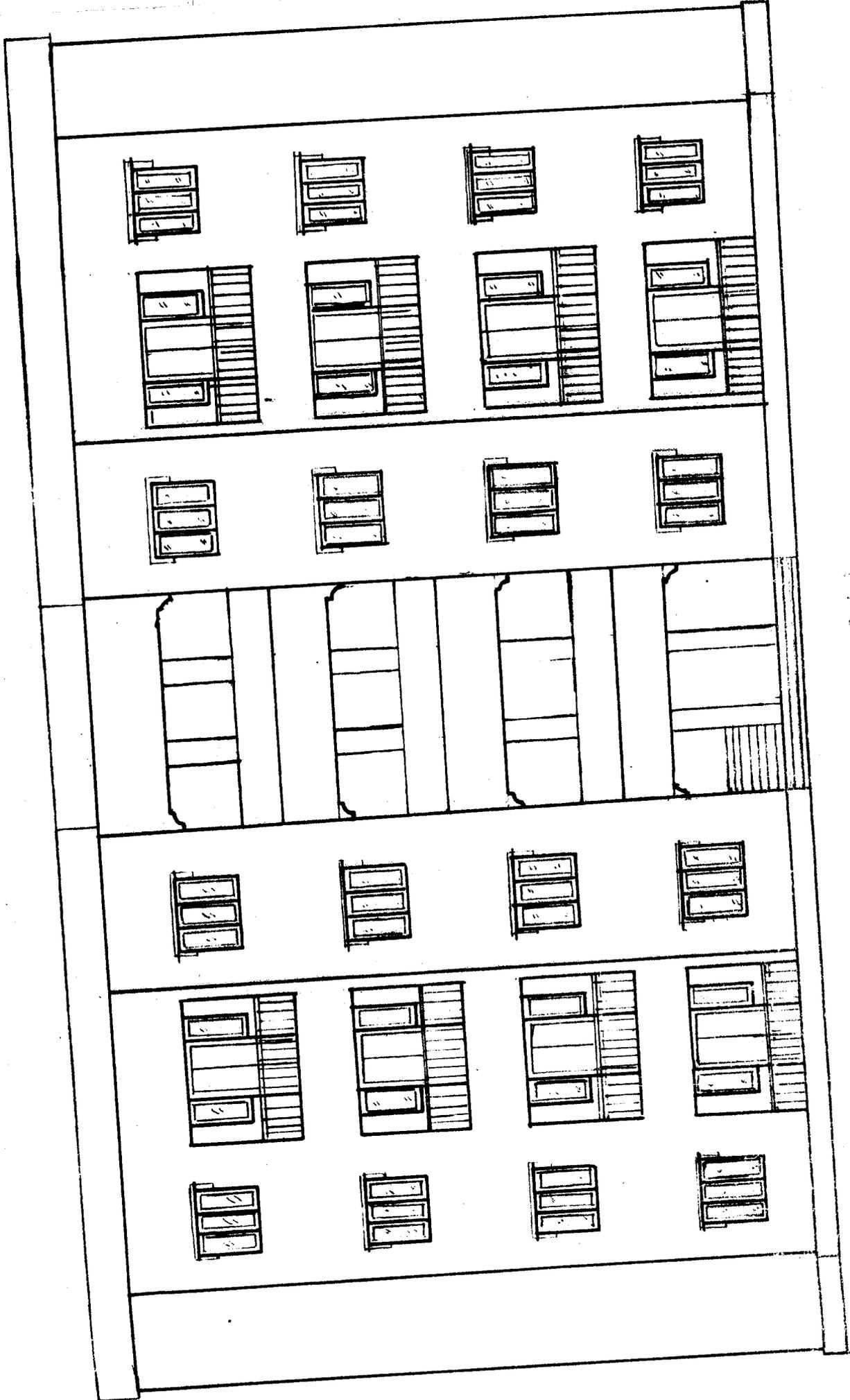
absorbing rate = $0.15\text{m}^3/\text{day}$.

$$= (3L + 2) \times 3 \times 0.15 = 18.8$$

$$L = 13.259 \quad 14\text{m}$$

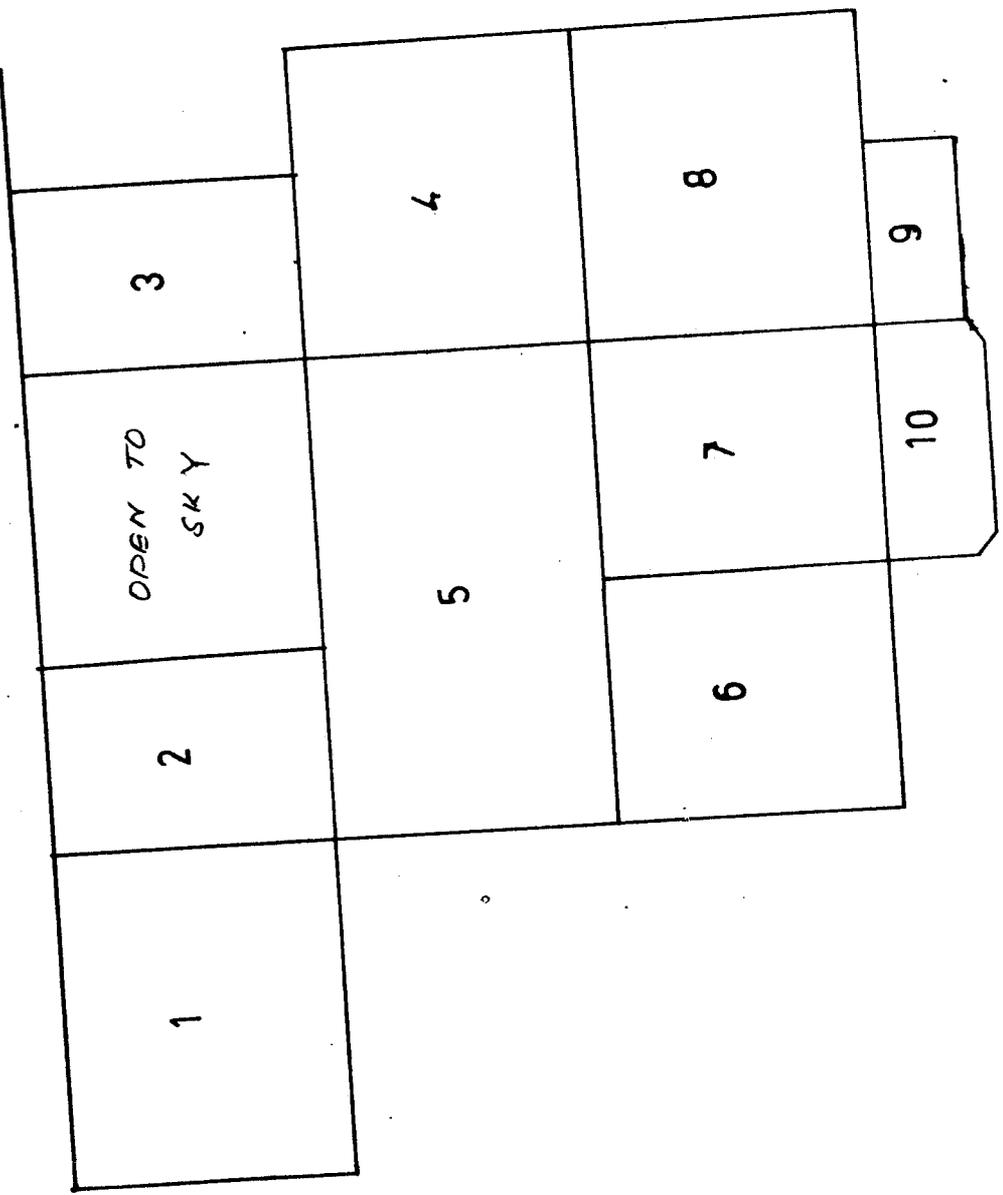
Size of dispersion trench =-

$$14\text{m} \times 1\text{m}.$$

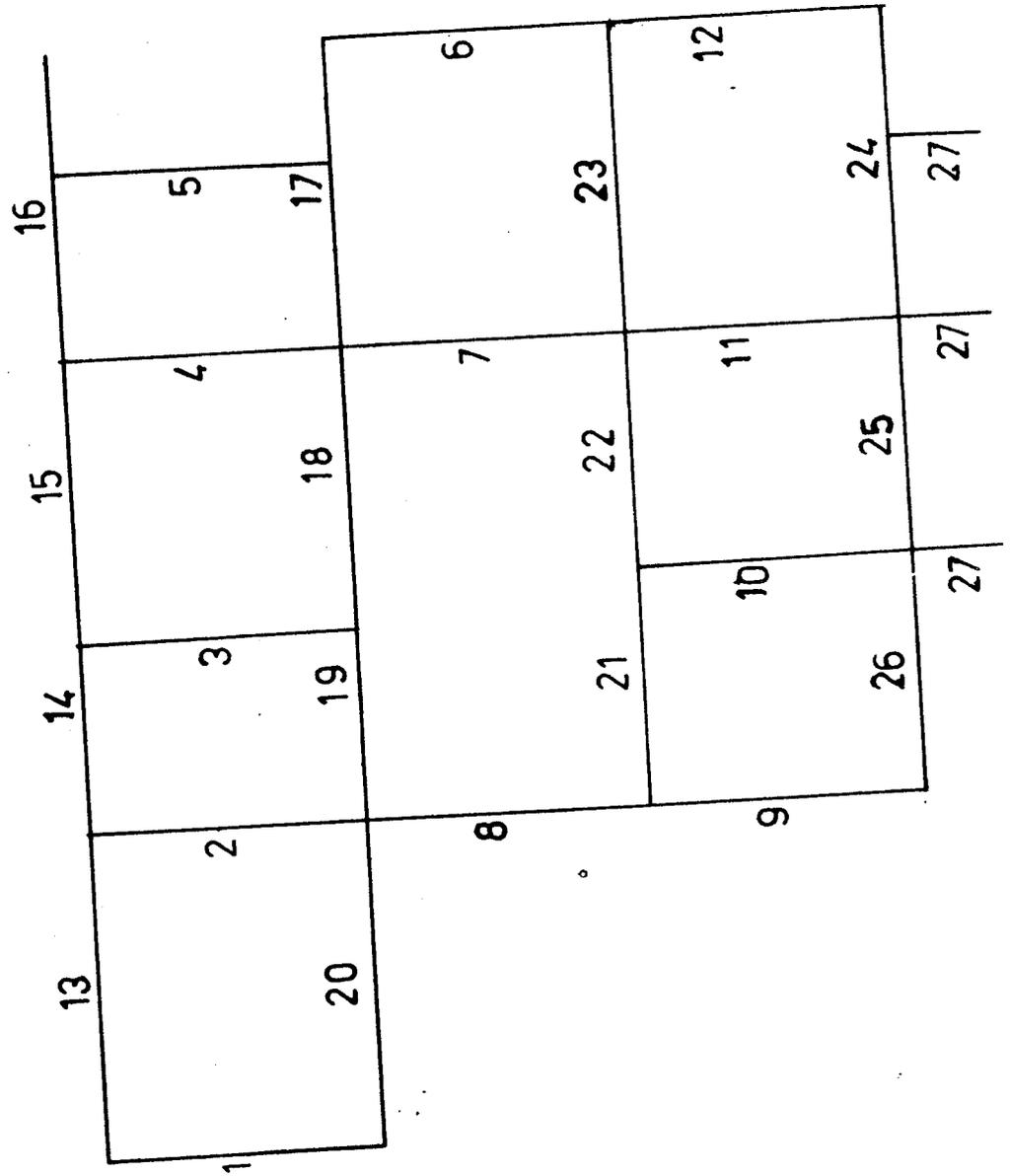


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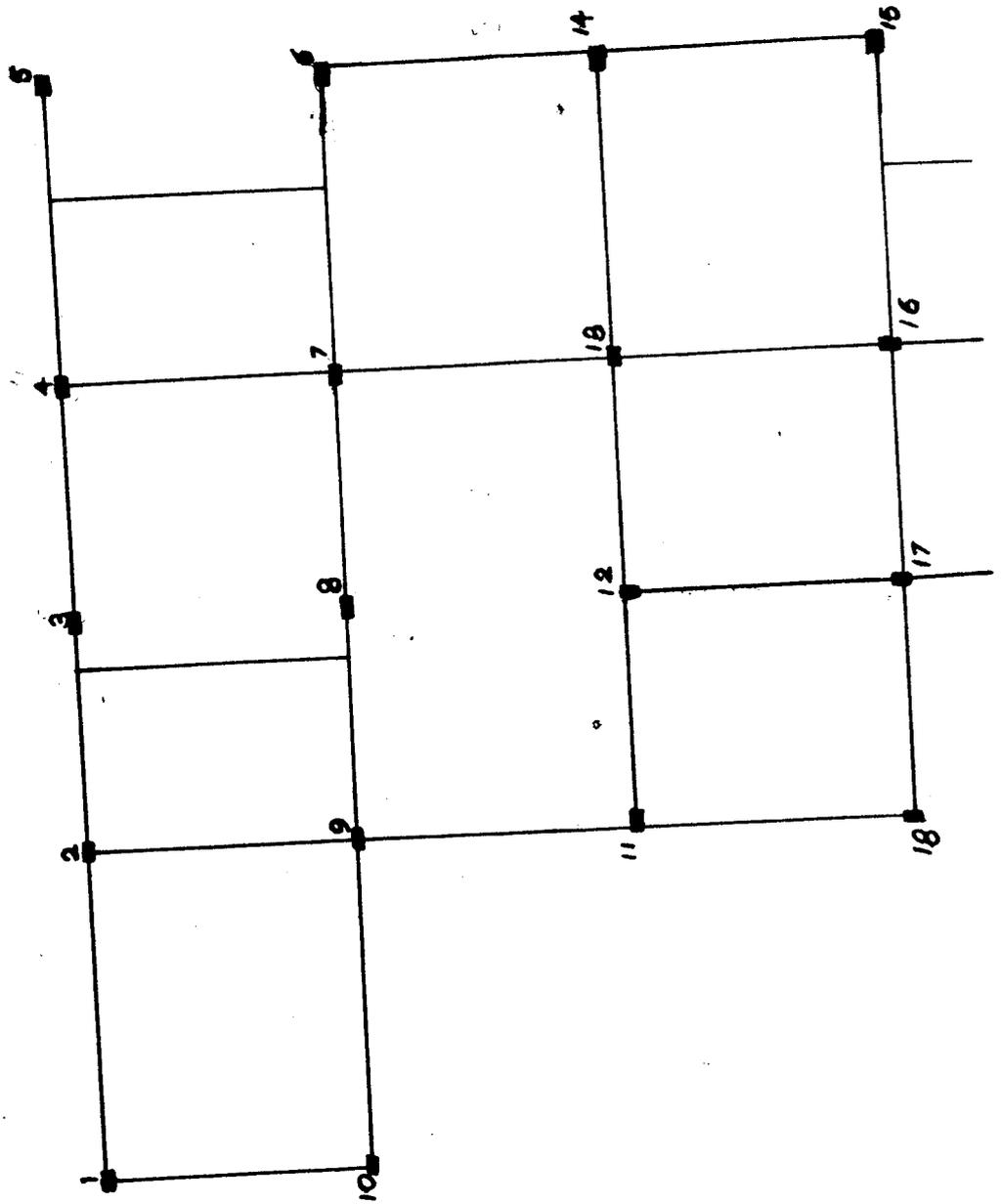
KEY PLAN FOR SLABS.



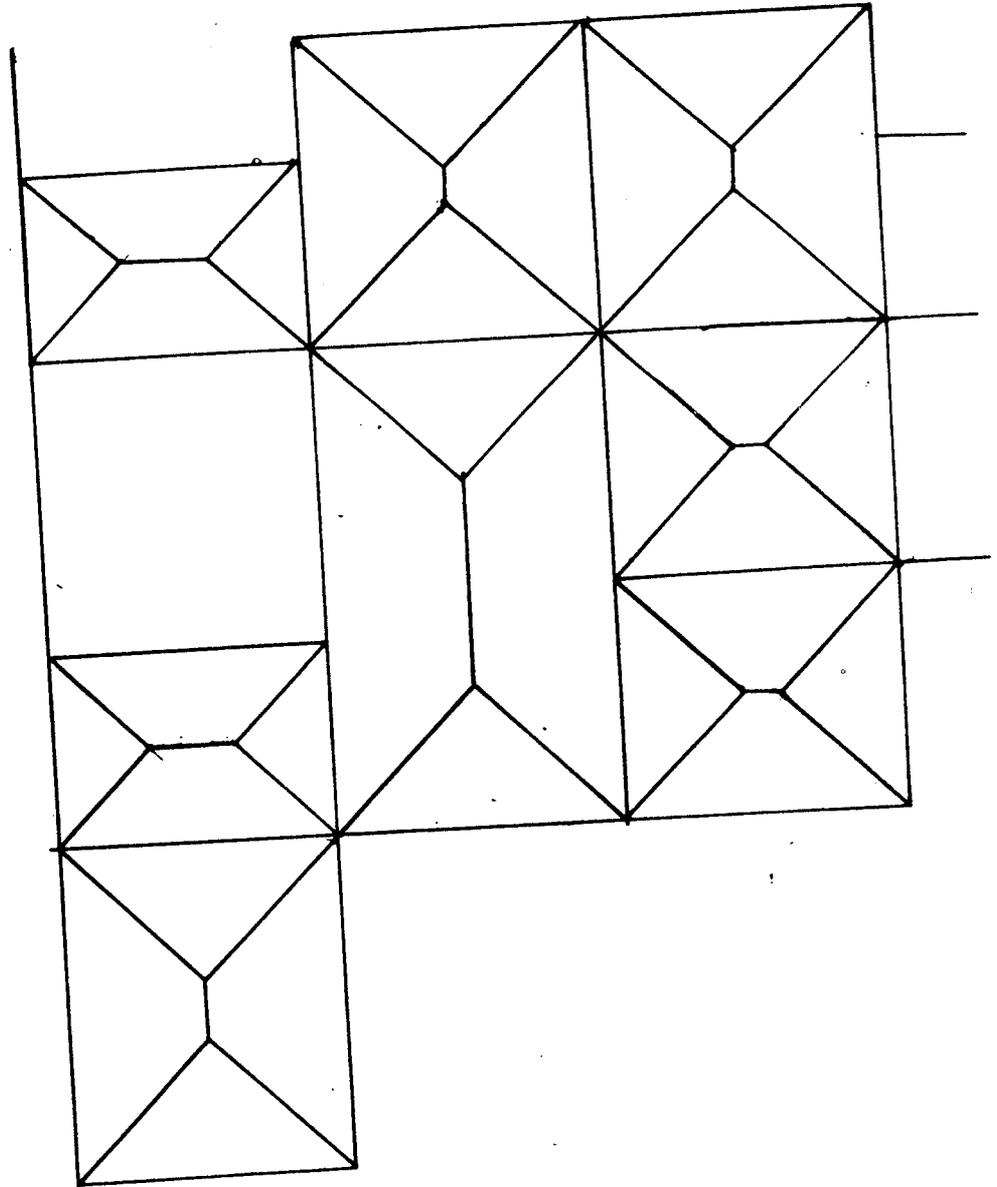
ARRANGEMENT OF BEAMS.



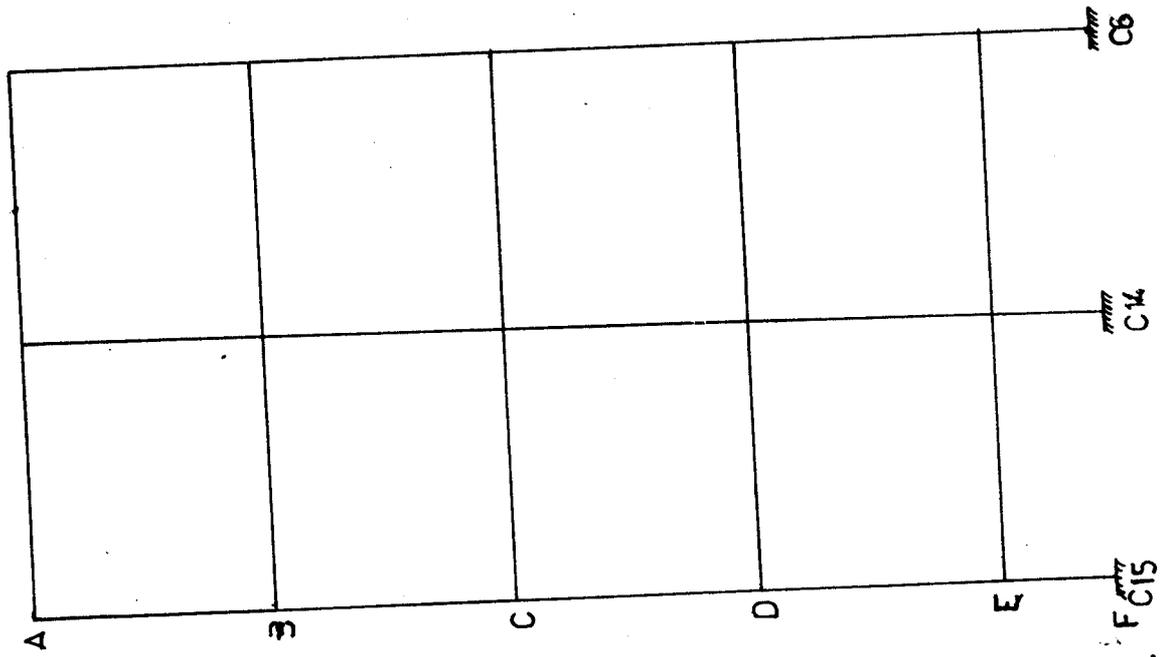
ARRANGEMENT OF COLUMNS



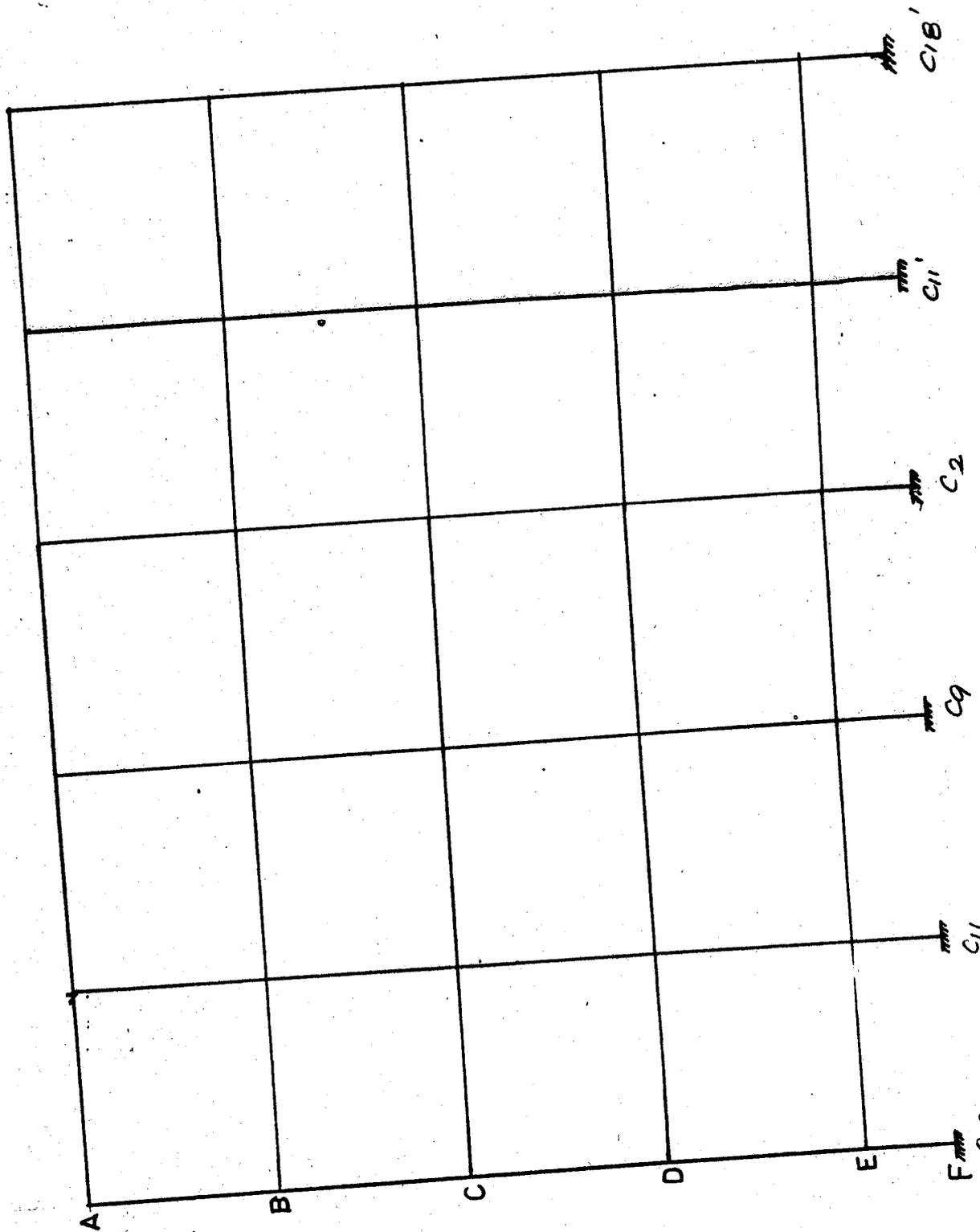
BEAM · LOADING · DIAGRAM



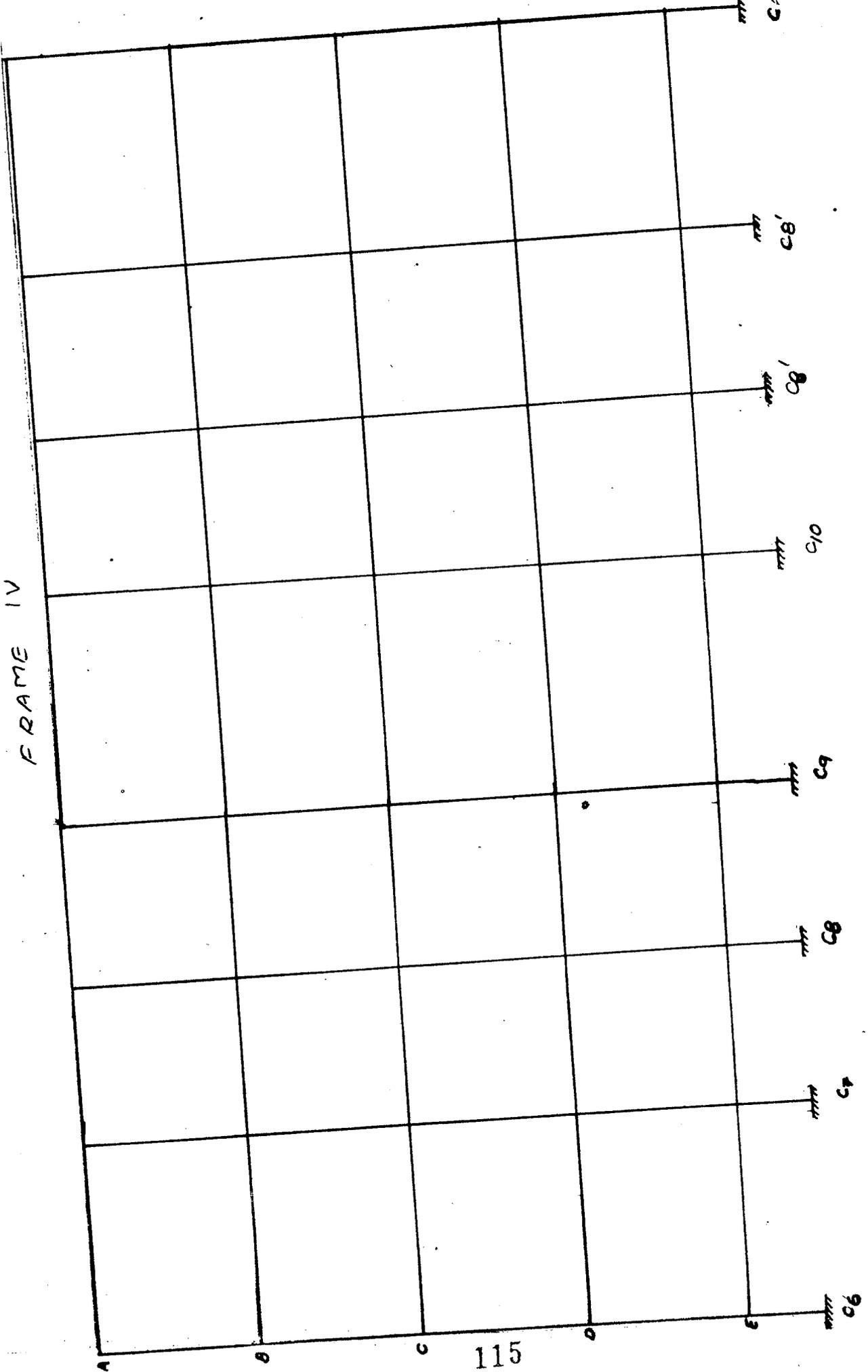
FRAME I



FRAME III

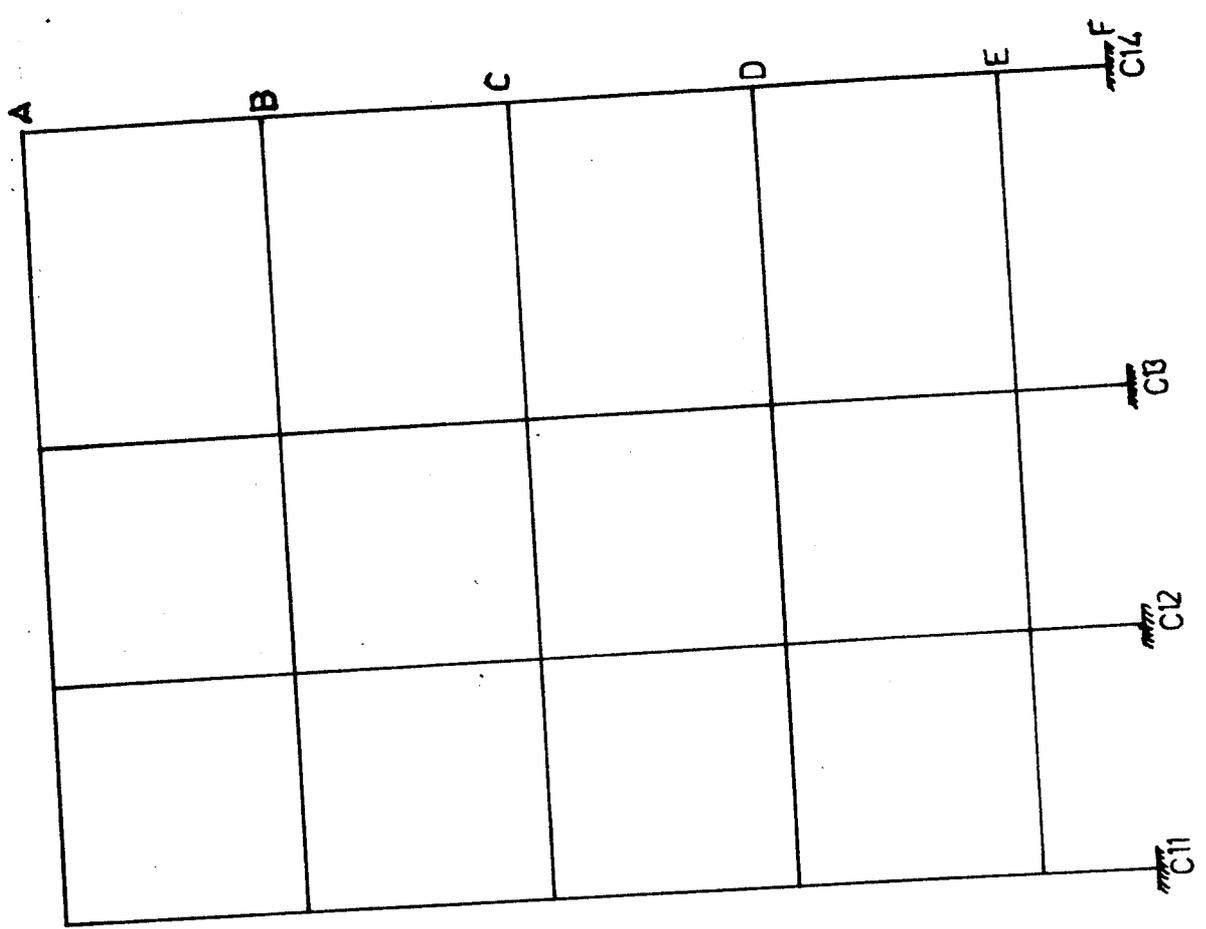


FRAME IV

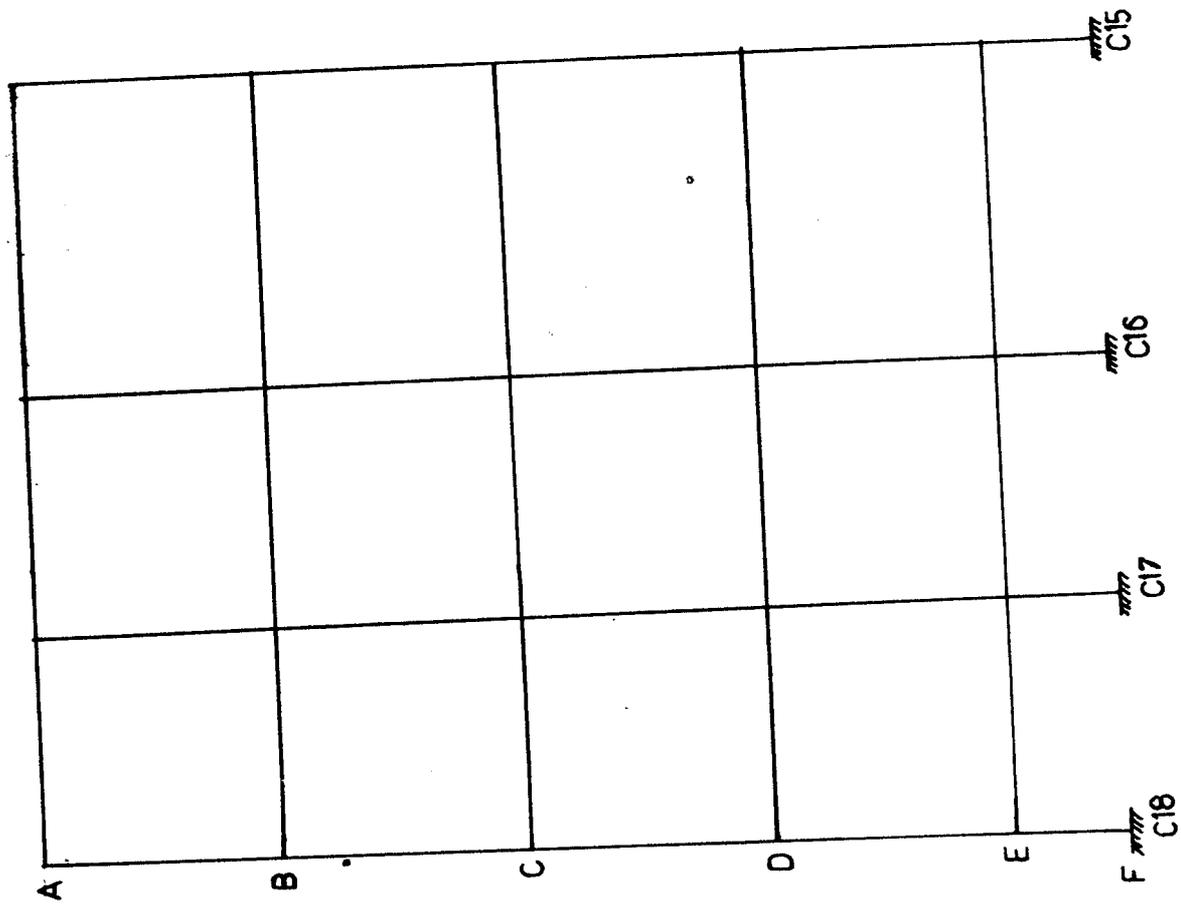


115

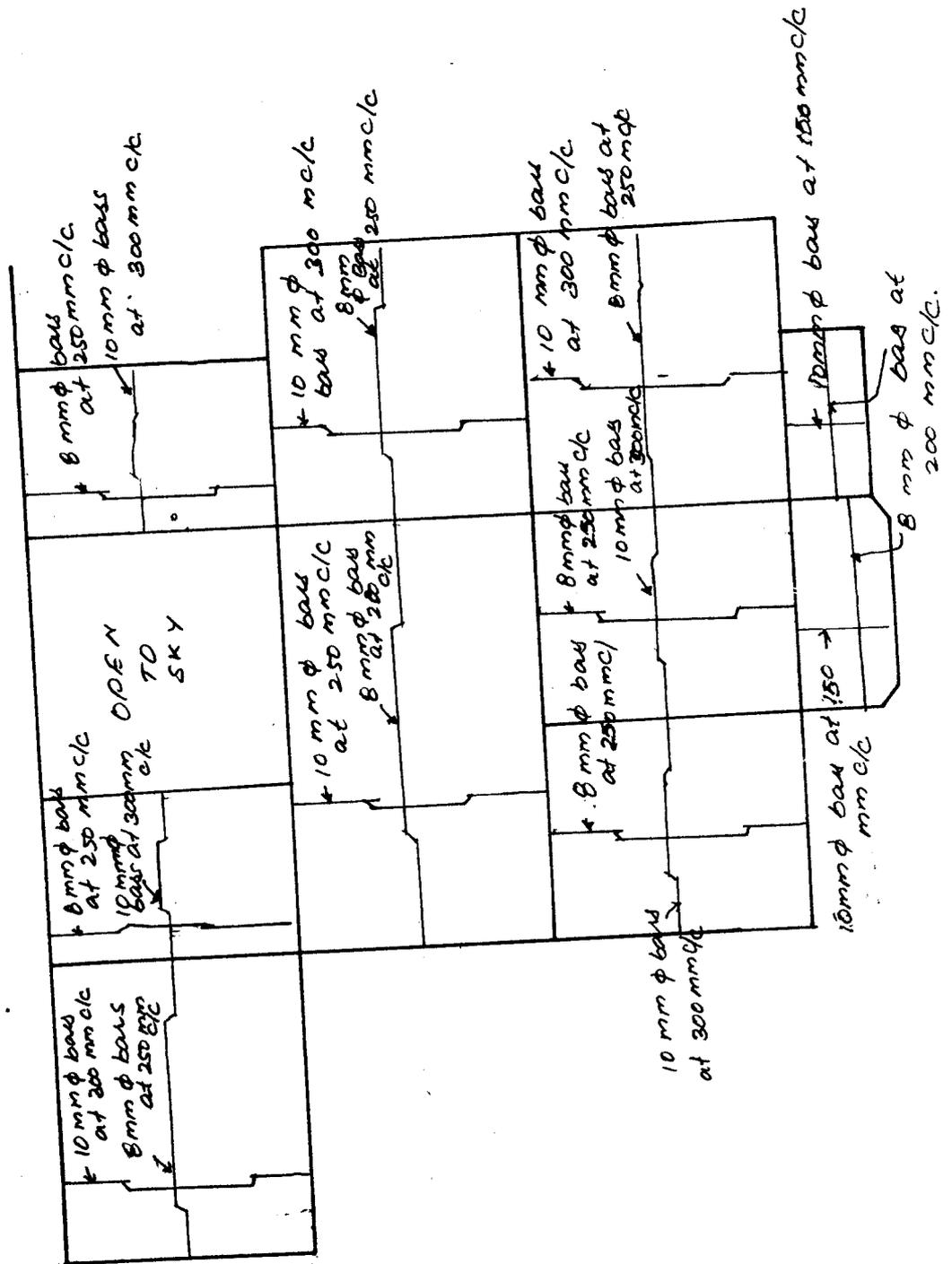
FRAME V



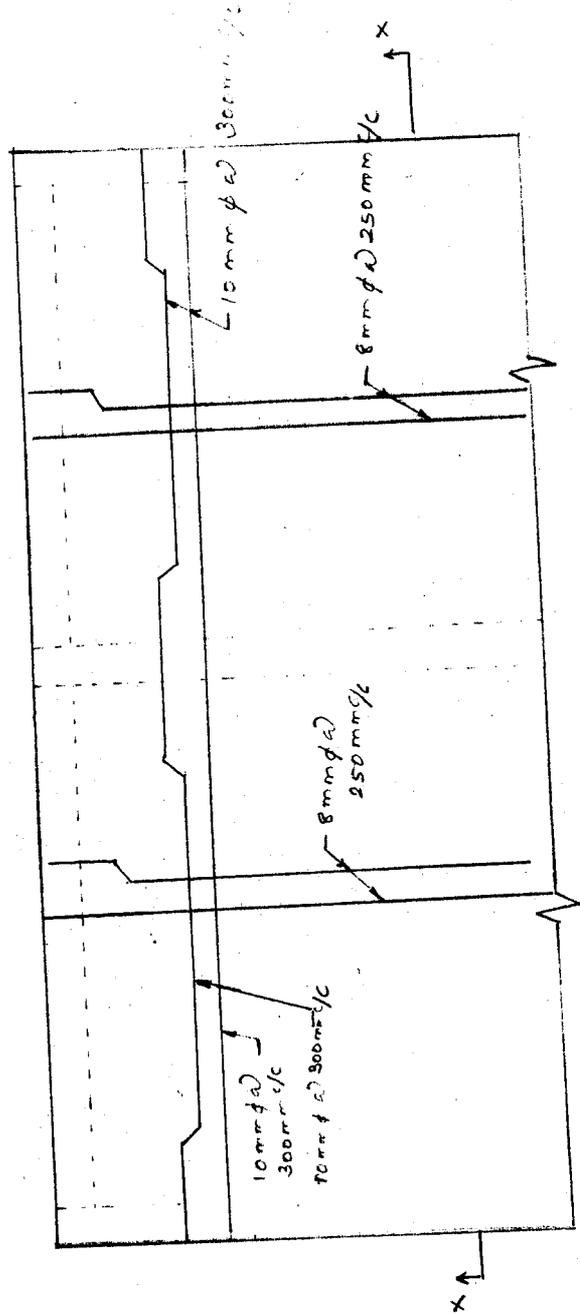
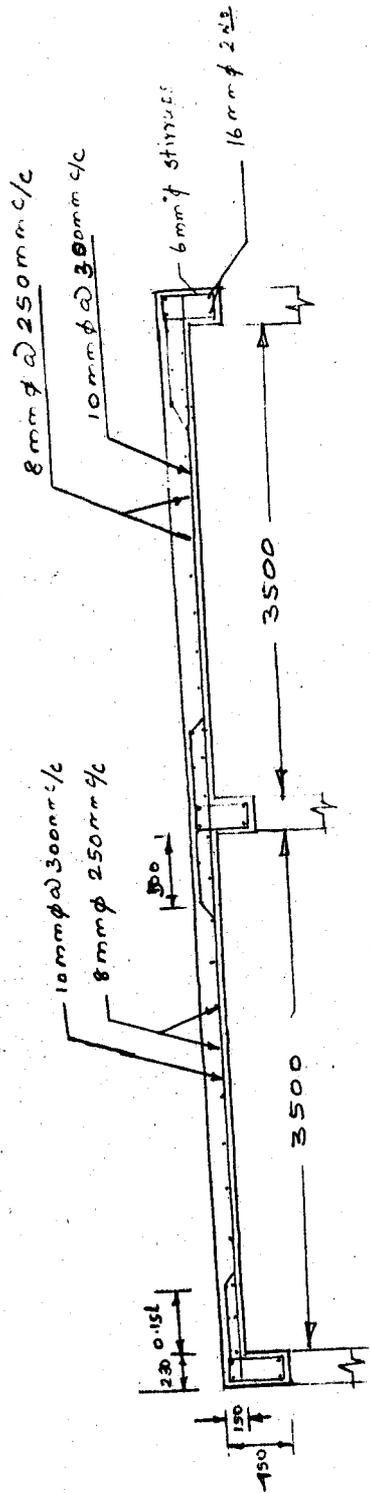
FRAME VI



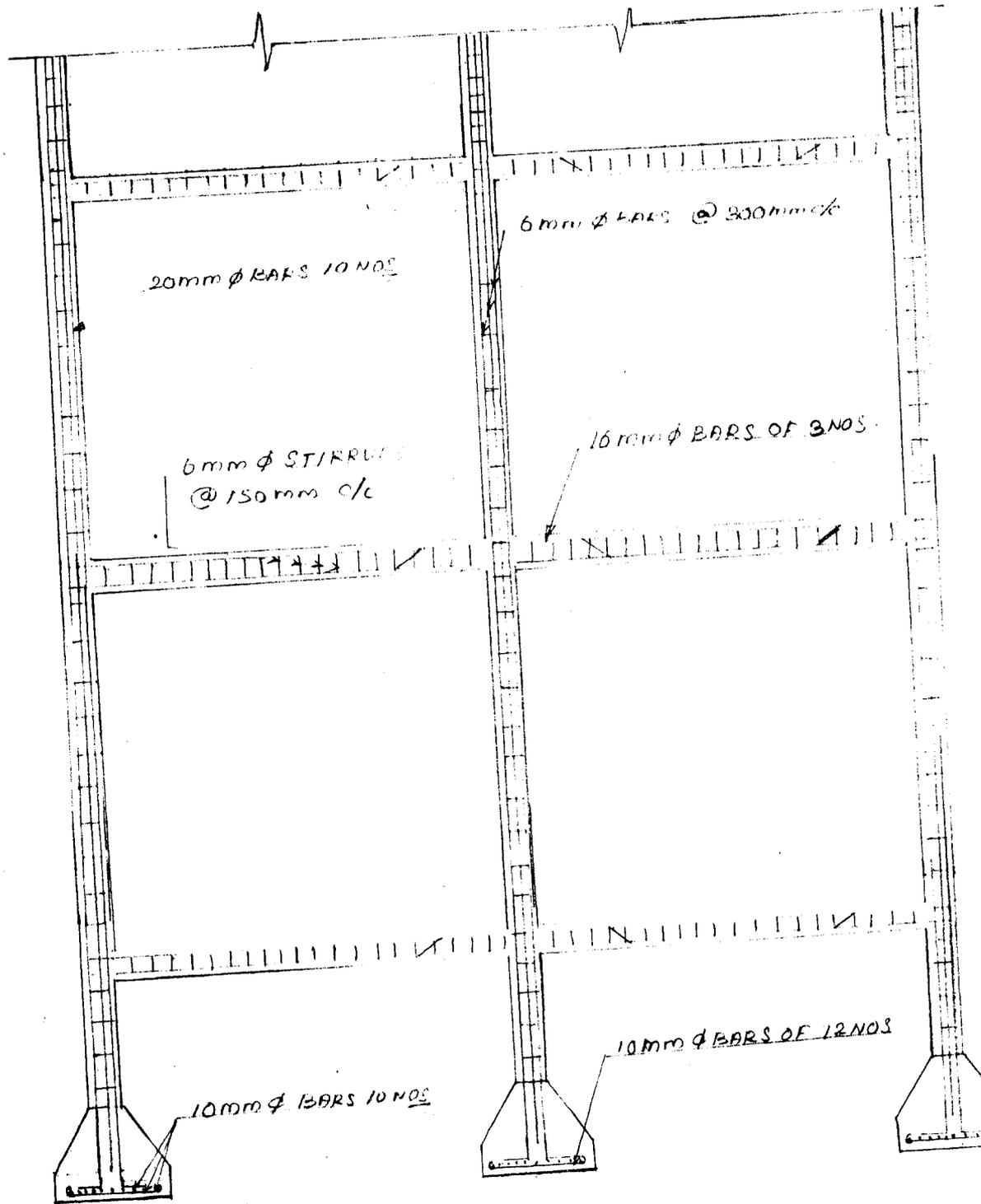
SLAB REINFORCEMENT DETAILS



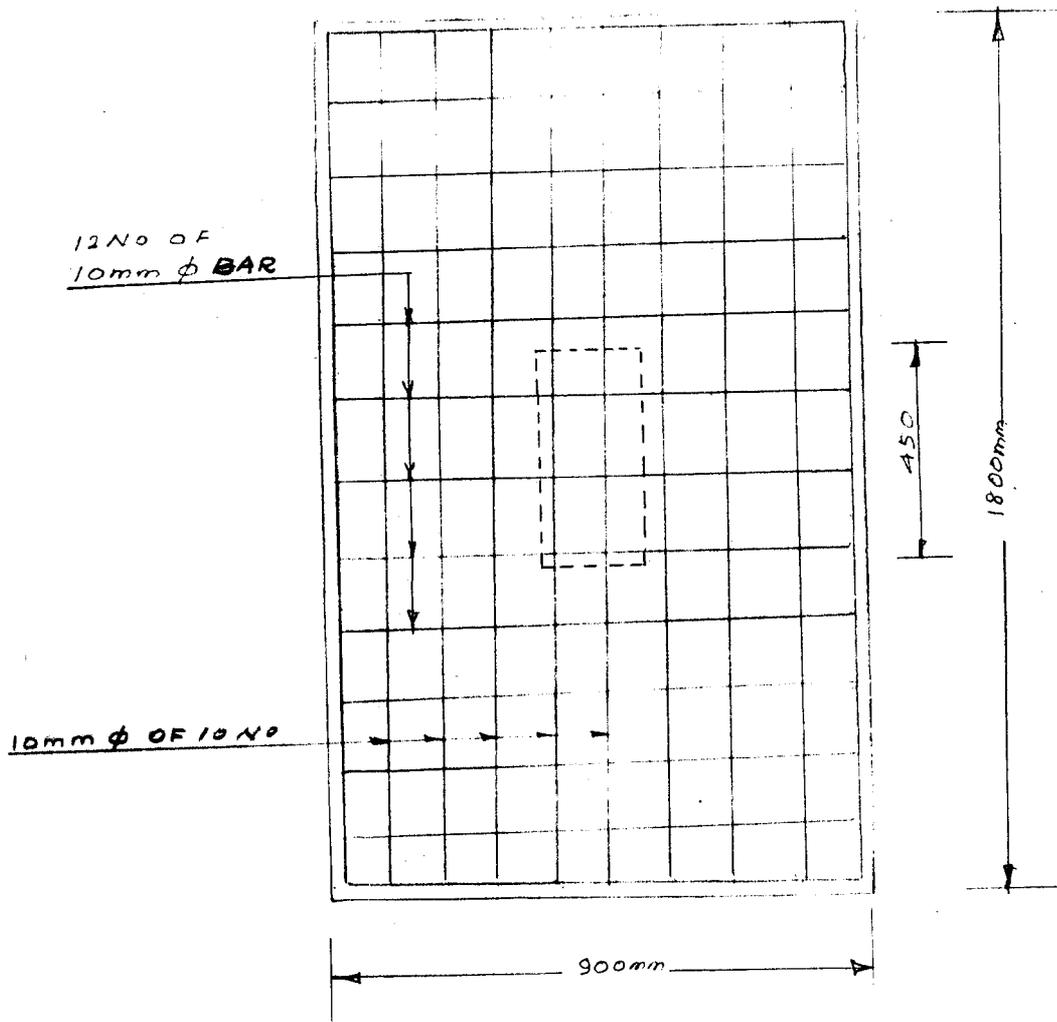
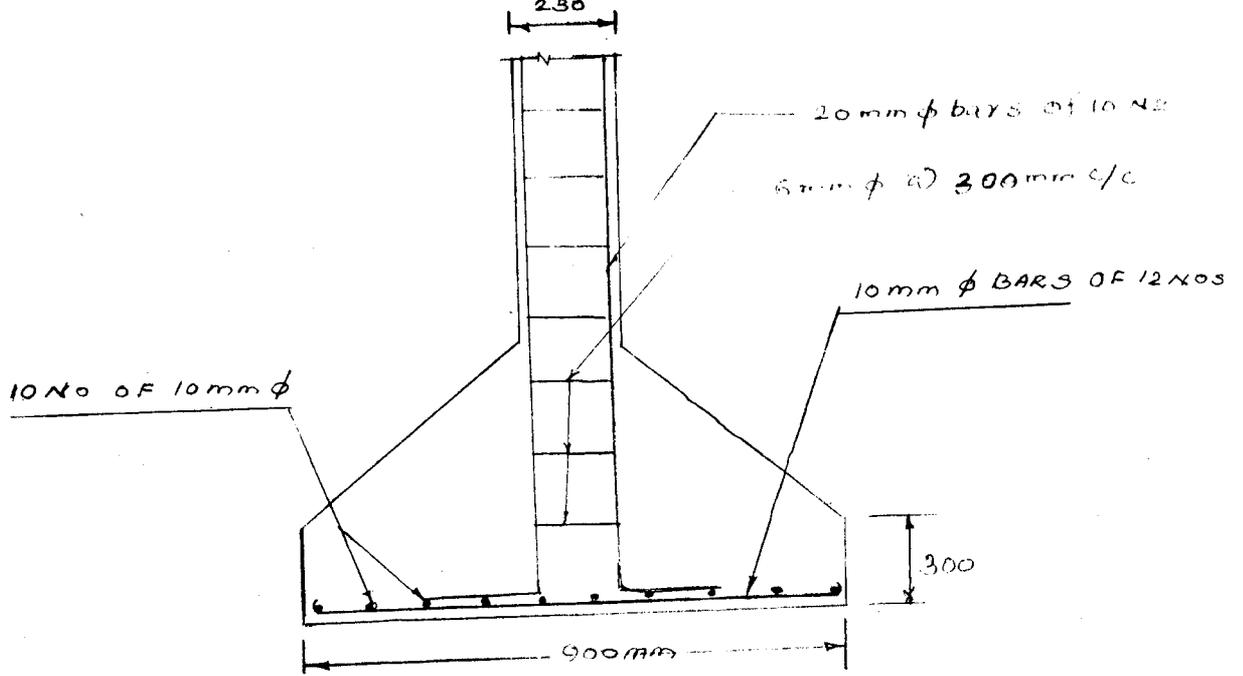
REINFORCEMENT DETAILS OF SLAB (PANEL 428)



SCALE: 1:20

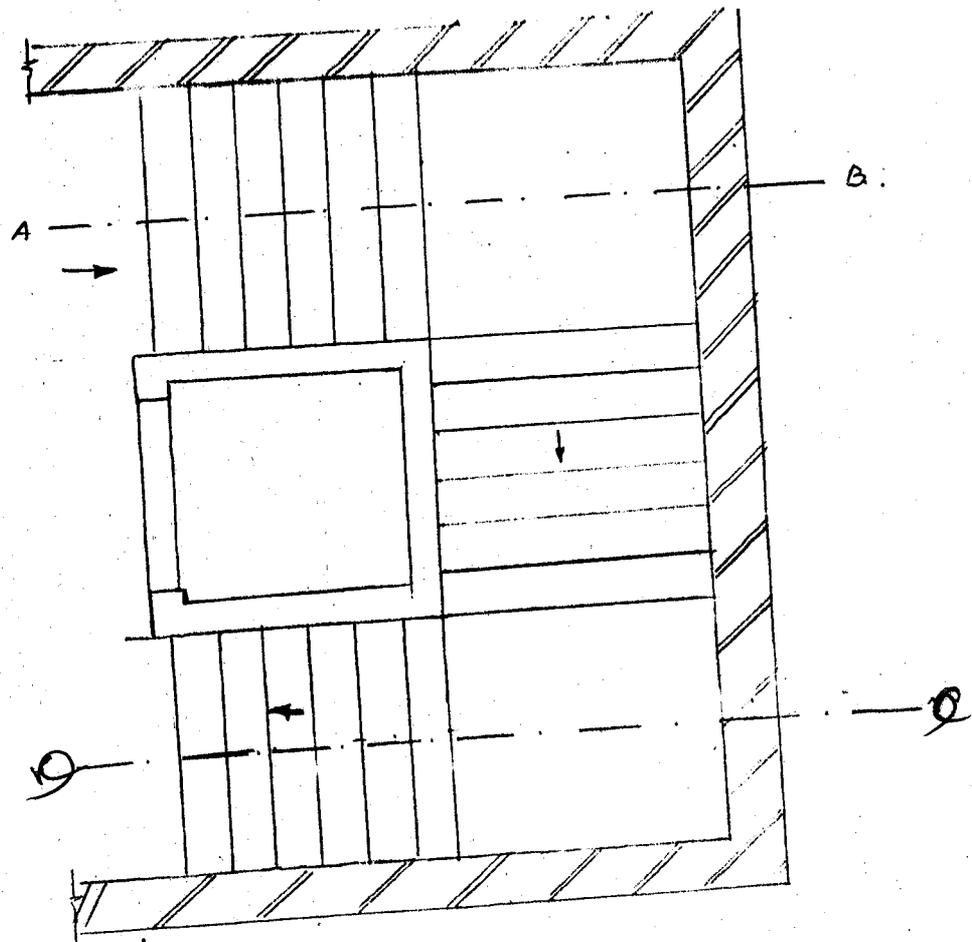


REINFORCEMENT DETAILS FOR BEAMS AND COLUMNS
 FRAME NO: 1 ALL DIMENSION IN MM . SCALE 1:50

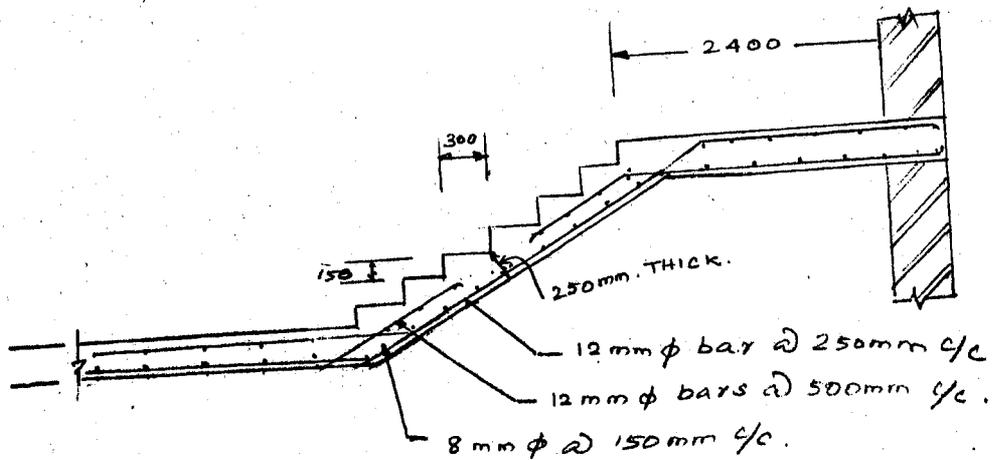


FOOTING FOR COLUMN NO C₁ SCALE 1:40

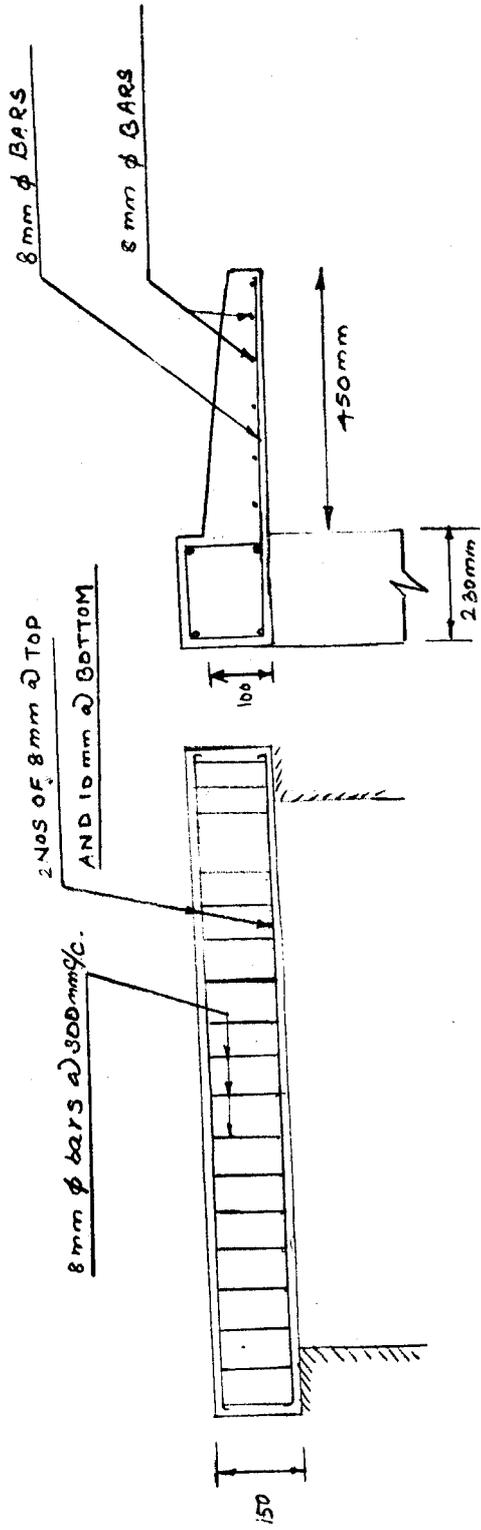
ALL DIMENSIONS ARE IN mm



PLAN

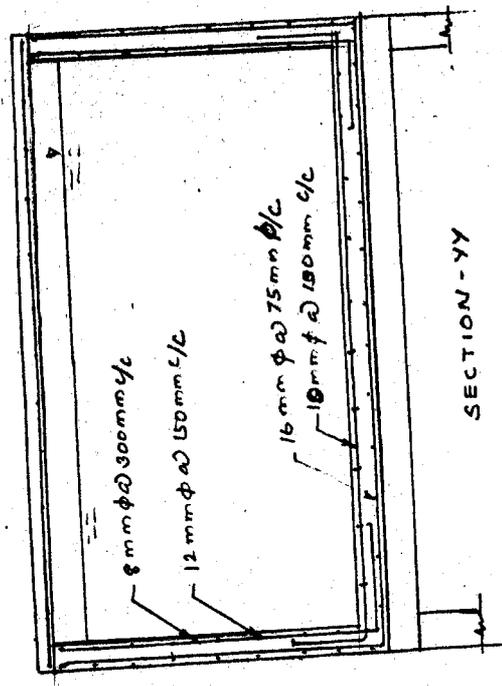
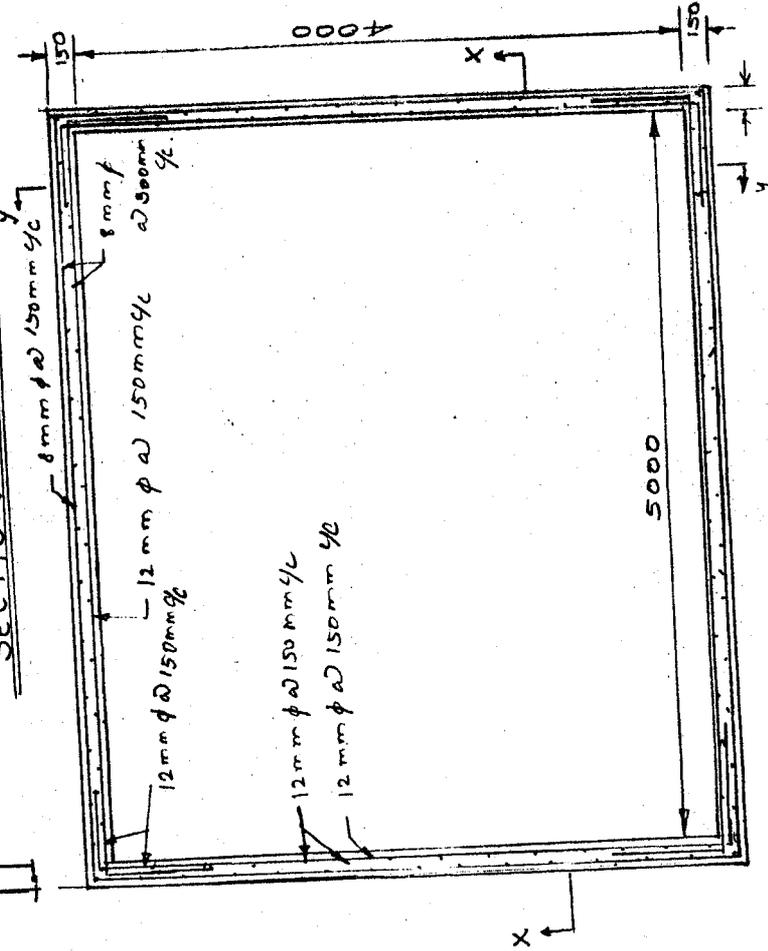
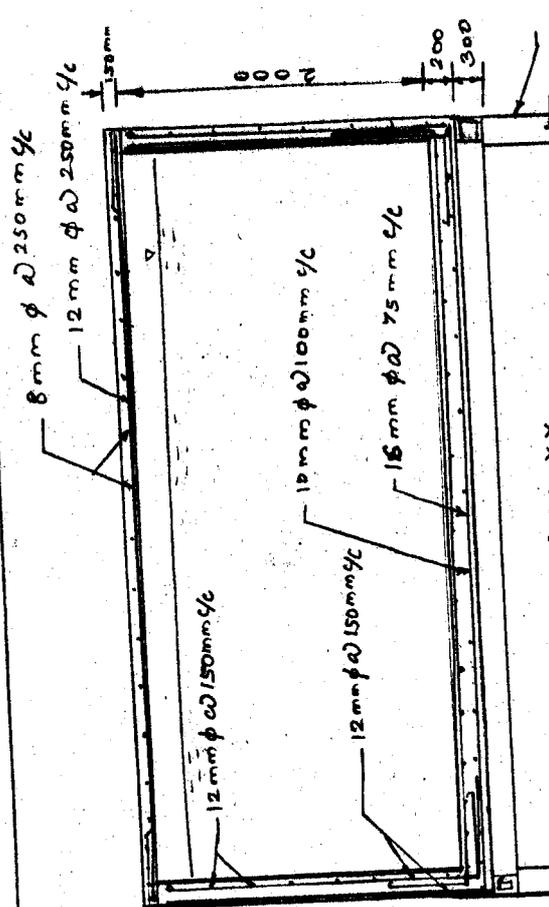


SECTION AT AB



C/S OF SUNSHADE

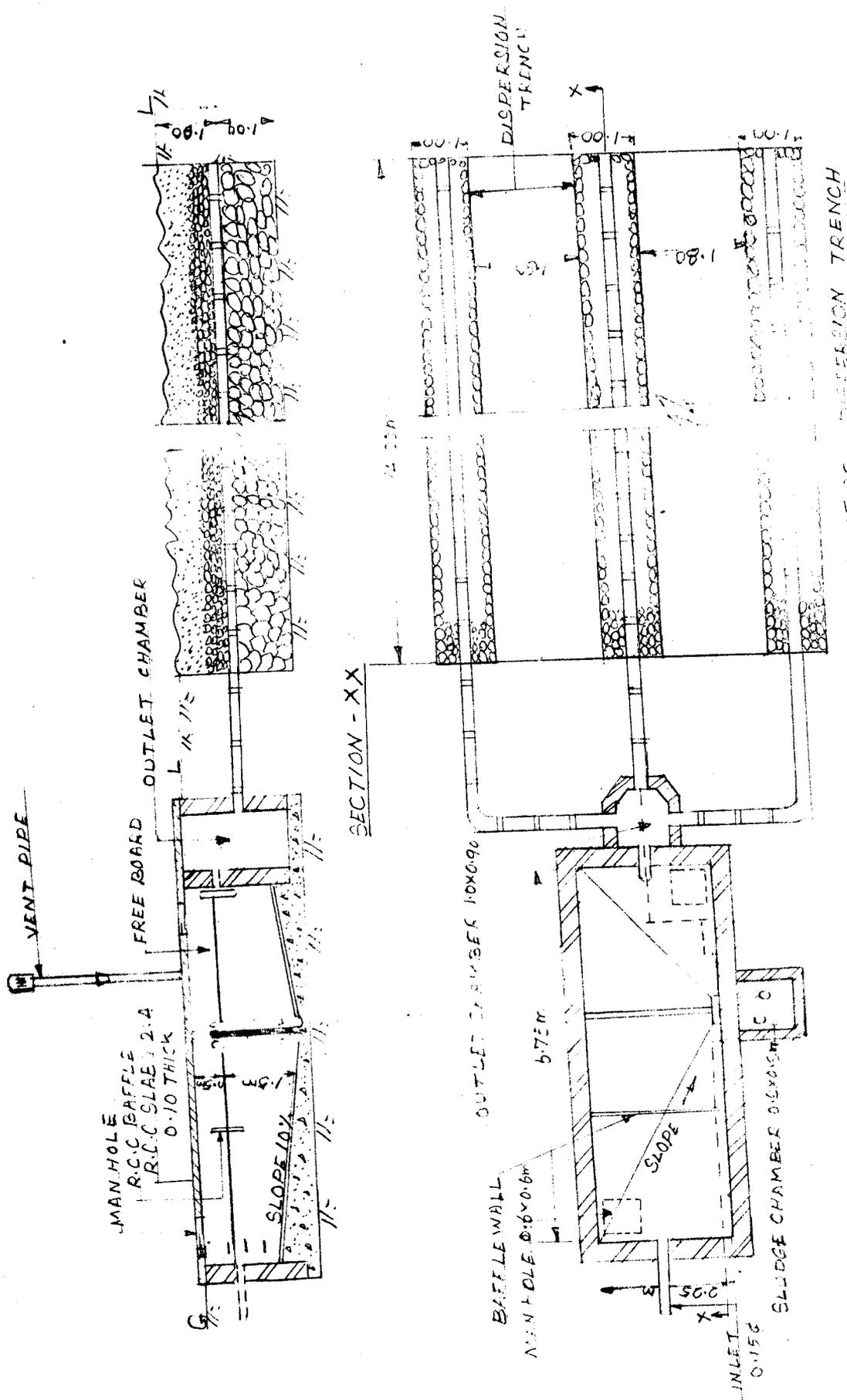
C/S OF LINTEL



WATER TANK REINFORCEMENT DETAIL

SCALE: 1:50

SEPTIC TANK



APPROXIMATE ESTIMATION

AREA

Total plinth area = 442.15m²

RATES

The rates are assumed as follows:

For ground floor	= Rs.2000.00/m ²
For first floor	= Rs.2100.00/m ²
For second floor	= Rs.2200.00/m ²
For third floor	= Rs.2300.00/m ²

APPROXIMATE COST:

For ground floor	= Rs. 884300.00
For first floor	= Rs. 928515.00
For second floor	= Rs. 972730.00
For third floor	= <u>Rs.1016945.00</u>
Total approximate cost	= <u>Rs.2966490.00</u>

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Concrete